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Chapter 1: Introduction

Part B of the design report presents a prestressed concrete interior girder as well as the reinforced concrete deck of a 25-meter bridge. Unlike Part A that focuses on the design procedure, detailed calculations with respect to three different design standards were conducted in this part of the design report. Seven chapters are included in this section, and each chapter follows the design procedure described in Part A.

In order to complete the design, the design scope must be defined at first. Thus, this part of the report starts off with a problem statement that includes the detailed information such as the geometry and material properties in Chapter 2. Structural analysis was conducted in Chapter 3 using the concept of influence line to analyze the impact that moving truck loads could bring to the structure. The truck loads are specified differently in each of the *Canadian Highway Bridge Design Code S6-14*, *AASHTO LRFD 2014 Bridge Design Specification*, and *Design of Highway Bridges S6-66*. Therefore, three different designs based on each of the design standard are included in this project.

Upon the completion of structure analysis, the truck load as well as other live loads and dead loads were calculated under both serviceability and ultimate limit states for all three design standards, and the corresponding shear force and bending moment profile along the span were determined in Chapter 4 as well. Once the design loads have been determined, AASHTO type 4 girder was chosen as the girder used in this design. In Chapter 5, three separate tendon profile and shear reinforcement design of the prestressed concrete girders were presented to satisfy each of the design standards. Similarly, the reinforced concrete deck is also designed in accordance with each code in Chapter 6. As it is mentioned in Part A, durability design should never be ignored. Thus, Chapter 7 develops a detailed concrete mix design. Information presented includes the size and distribution of aggregates, the amount of cement and aggregates, and the type of admixtures that needs to be added in. At the end of the Part B, detailed design drawings are presented with respect to all three designs.
Overall, Part B of the project report presents three detailed designs of a 25 meter prestressed concrete bridge with respect to three design standards, and the strength, serviceability and durability designs are all included. The entire design process follows the description in Part A.
Chapter 2 Project Statement

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2.1 Introduction

In this chapter, it will introduce all the detailed information for this required bridge design such as its geometric properties, material properties as well as the assumption that is being made during the design process. The assigned task for our group is to design a simply supported bridge with four prestressed concrete bridge girder, the specific requirements are in the following sections.

Figure 2.1 Project Render and Explored Views
2.2 Geometric properties

In the following figure, it contains the required dimensions for the assigned bridge design[Figure B.1]. All dimensions are in millimeter.

![Overall Geometry of the Bridge](image)

*Figure 2.2 Overall Geometry of the Bridge*

As the figure above presented, the bridge is a simply supported bridge that have four prestressed concrete bridge girders that support a concrete slab that has a span length of 25 m, width 10 m and slab thickness of 200 mm [Figure 2.2].

2.3 Material properties

The bridge contains two types of material, the first type is the concrete and the prestressed steel is the second type. The table below provides assumptions on specific capacity, strength and young's modulus for these two types of material [Table 2.1][Table 2.2]. Elastic young’s modulus for the concrete and steel is coming from the code instead of given by the client.

<table>
<thead>
<tr>
<th>Material</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>fc’ (Girder)</td>
<td>50 MPa</td>
</tr>
<tr>
<td>fc’ (Deck)</td>
<td>40 MPa</td>
</tr>
<tr>
<td>fci</td>
<td>40 MPa</td>
</tr>
</tbody>
</table>
Table 2.2 Material Properties of Reinforcing Steel

<table>
<thead>
<tr>
<th>Material</th>
<th>Reinforcing steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_y$</td>
<td>400 MPa</td>
</tr>
<tr>
<td>$E_s$</td>
<td>200000 MPa</td>
</tr>
</tbody>
</table>

Table 2.3 Material Properties of Prestressed Tendon

<table>
<thead>
<tr>
<th>Material</th>
<th>Prestressed tendon (Seven-wire strand)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_u$</td>
<td>1860 MPa</td>
</tr>
<tr>
<td>$E_s$</td>
<td>200000 MPa</td>
</tr>
</tbody>
</table>

2.4 Conclusion

This chapter provides the basis of design and assumptions for the simply supported bridge that the group has been assigned to design. This information includes geometric and material properties will be applied in the following chapters. The primary code that the design is referencing from are the CSA 16 code.
# Chapter 3 Structural Analysis – Influence Line

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3.1 Introduction to Influence Line method of analysis

An influence line is a profile representation of the variation of either the reaction, shear, moment, or deflection at a specific location in a member as an applied concentrated force moves along the member length. Influence lines is a powerful analysis method for the design of bridges, industrial crane rails, conveyors, and other structures where loads move across their span because the influence line enables direct measure of where the moving load should be placed on the structure to create the greatest influence at the specified location. In addition, the magnitude of the associated reaction, shear, moment, or deflection at the specified location can be calculated from the influence-line diagram easily [1]. This section starts with the background information related to the influence line method of analysis with respect to determinate and indeterminate structures, then influence lines specific to the problem statement presented in Ch. 2 are developed for truck loads calculation as specified in the design standards of CSA S6-14, AASHTO LRFD-14, and CSA S6-66.

3.2 Procedure for Determining Influence Line

There are two widely-used procedures for the construction of influence line at a specific point P in a member for reaction, shear, or moment. Both methods use a moving force with dimensionless magnitude of unity.

3.2.1 Tabulate Value Procedure

The first method is to use tabulate values. A unit load is to be placed at various locations along the member. At each location, equations of equilibrium are used to statically determine the value of the function (reaction, shear, or moment). Once the load has been placed at various points along the span of the member, the tabulated values can be plotted and the influence-line segments constructed. [1]

3.2.2 Influence-Line Equation

The influence line can also be constructed by placing the unit load at a variable position
x on the member and then directly calculate the value of reaction, shear and moment as a function of x. The equation of the various line segments can then be determined and plotted. [1]

3.2.3 Qualitative Influence Line

Influence lines can be determined using both qualitative and quantitative methods. Qualitative influence lines are developed by Heinrich Müller-Breslau for rapidly constructing the shape of an influence line. The method is referred to as the Müller-Breslau principle, which states that the influence line for a function (reaction, shear, or moment) is to the same scale as the deflected shape of the beam when the beam is acted upon by the function[1]. For statically determinate and indeterminate structures the shape of the influence line is slightly different, which are shown in Figure 3.1 below. For the purpose of this project, determinate influence lines are utilized.

![Qualitative Influence Line](image)

*Figure 3.1 – Different Shape of Qualitative Influence Line for Statically Determinate and Indeterminate structures [2]*
3.3 Influence Lines for the Design of Bridge

In this section, unit load influence lines were developed based on the 25 m, simply supported span that will be designed as part of this project. A quantitative method was employed and the equation of the influence line was determined with respect to a variable, \( x \), the distance of the unit load from the left support. Figure 3.2 and Figure 3.3 shows the unit load influence lines for the reaction at the supports, the shear at 1/4 and 3/4 of the span as well as at the mid-span. Figure 3.4 shows the unit load influence lines for the moment at 1/4 and 3/4 of the span from the left support, as well as at the mid-span. These influence lines were later used to determine the influence associated with the design trucks from CSA S6-14, CSA S6-66 and AASHTO LRFD-12.

![Influence Line for Reaction at Left Support](image)

*Figure 3.2 - Unit load influence lines for the reaction at left support*
Figure 3.3 - Unit load influence lines for the shear at the 1/4 and 3/4 span, as well as the mid-span

Influence Line for Moment at 1/4 Span from Left Support

Influence Line for Moment at 1/2 Span from Left Support
Figure 3.4 Unit load influence lines for the moment at the 1/4 and 3/4 span, as well as the mid-span

In addition to the above influence lines, the shear and moment envelope for a moving unit load was also developed to depict the maximum shear and moment values as the load moves along the bridge deck, which are shown in Figure 3.5
3.4 Code Prescribed Truck Loads for Design

For bridge deck design load determination, the design code has a specified truck to be used as the baseline loads that will be applied to the bridge deck. After the truck loads are calculated, the influence lines developed in Section 3.3 can then be employed to obtain the critical location and associated loads for the design of bridge deck and girders. Figure 3.6 to Figure 3.8 present the design truck load in CSA S6-14, ASSHTO LRFD-14 and CSA S6-66, respectively.

Figure 3.5 – Maximum Shear and Moment Envelope for Unit Load Moving along the Bridge
Figure 3.6 - The CL-W design truck as specified in CSA S6-14 [4]

Figure 3.7 - The standard design truck and loads as specified in AASHTO LRFD-14 [5]
For the purposes of determining the shear envelope with respect to the CSA S6-66 code and the AASHTO LRFD-14 code, the H-S truck was used since its heavier than the standard H truck and was therefore deemed to be more critical.

3.5 Shear and Moment Design Envelopes

The shear and moment design envelopes can be developed when apply the design truck loads, which are specified in CSA S6-14, CSA S6-66 and AASHTO LRFD-14, into the previous developed unit shear and moment envelops. Since the design truck specified in AASHTO LRFD-14 and CSA S6-66 are the same in size and weight, only one analysis was carried out for these codes together. The calculated shear and moment design envelops for the two design trucks are plotted in the same graphs to enable direct comparison. Table 3.1 presents the key points on the envelopes, and the resultant design envelopes are shown in Figure 3.9 and Figure 3.10, for shear and moment respectively.
Table 3.2 Tabulated Design Value for Shear and Moment

<table>
<thead>
<tr>
<th>Location from the left support</th>
<th>CSA S6-14</th>
<th>AASHTO LRFD-14 &amp; CSA S6-66*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Shear [kN]</td>
<td>Maximum Moment [kNm]</td>
</tr>
<tr>
<td>0</td>
<td>404.8</td>
<td>0</td>
</tr>
<tr>
<td>6.25</td>
<td>248.55</td>
<td>1853.32</td>
</tr>
<tr>
<td>12.5</td>
<td>116.3</td>
<td>2226.25</td>
</tr>
<tr>
<td>18.75</td>
<td>248.55</td>
<td>1853.32</td>
</tr>
<tr>
<td>25</td>
<td>404.8</td>
<td>0</td>
</tr>
</tbody>
</table>

Shear Design Envelope

![Shear Design Envelope](image)
3.6 Conclusion

In general, influence lines are a useful analysis tool that could be used to develop design shear and moment envelopes, which can be used as a quick reference to determine the maximum shear and moment that each section of a structural element (i.e. bridge structure) undertook as the loads move across its length. The influence lines and design envelopes developed for the design truck in CSA S6-14, CSA S6-66 and AASHTO LRFD-14 are presented in this chapter and will be used in subsequent calculations and the rest of the design process, as a reference to base the design on.
Reference:


Chapter 4 Design Loads

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4.1 Introduction

In this section, all the design loads are defined based on three difference codes: CSA S6-14, AASHTO LRFD-14, and CSA S6-66. In this case, only dead loads and live loads are defined calculated, whereas other loads such as snow loads, wind loads and seismic loads are excluded. In this design, the dead loads include the self-weight of prestressed I girders, concrete deck and pavement material. Live loads include truck load and lane load.

4.2 I Girder Specification

In order to calculate the self-weight of I girder, the cross-section geometry must be determined. Based on Figure 4.1, the span to depth ratio is defined to be: $\frac{l}{h} = 18$ for I girder section. For the design of a 25-meter span simply supported bridge, the required I girder depth, h, is calculated as follows:

$$h = \frac{l}{18} = 1.389 \text{ m}$$

Thus, AASHTO Type IV I girder, as shown in Figure 4.1, has been chosen as an educated estimate of the actual cross-section for the purpose of self-weight calculation.

![Figure 4.1 Type IV I-Girder [1]](image)
Table 4.1 Basic Geometry Information about the Type IV I-Girder [1]

<table>
<thead>
<tr>
<th>Component</th>
<th>Height</th>
<th>Area</th>
<th>Moment of Inertia</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height</td>
<td>1.372 m</td>
<td>0.509 m²</td>
<td>0.1085 m⁴</td>
</tr>
<tr>
<td>Area</td>
<td>54 inch</td>
<td>789 in²</td>
<td>260,730 in⁴</td>
</tr>
<tr>
<td>y (bottom)</td>
<td>24.73 inch</td>
<td>24.73</td>
<td></td>
</tr>
</tbody>
</table>

4.3 CSA S6-14 Design Loads [2]

This section specifically focus on defining any design loads based on CSA S6-14. It consists of dead loads, live loads and corresponding distribution factors, and dynamic impact loads.

4.3.1 Dead Load Analysis

The dead loads are calculated based on self-weight of assumed Type IV prestressed I-girder, the 200-mm reinforced concrete deck, and the 3 inch (76.2 mm) bituminous pavement. The spacing of interior girders is designed as 2.7 m, which is used as the tributary area calculation. The unit weight of each type of material is listed in Table 4.2, and the overall calculation results are summarized in Table 4.3.

<table>
<thead>
<tr>
<th>Material</th>
<th>CSA S6-14 (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous Wearing Surface</td>
<td>23.5</td>
</tr>
<tr>
<td>Plain Concrete</td>
<td>23.5</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>24.5</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>24.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>Component Area (m²)</th>
<th>Unit Weight (kN/m³)</th>
<th>Dead Load (kN/m)</th>
<th>ULS Max Load Factor</th>
<th>Factored Dead Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>0.51</td>
<td>24.50</td>
<td>12.47</td>
<td>1.10</td>
<td>13.72</td>
</tr>
<tr>
<td>Deck</td>
<td>0.54</td>
<td>24.00</td>
<td>12.96</td>
<td>1.20</td>
<td>15.55</td>
</tr>
<tr>
<td>Pavement</td>
<td>0.21</td>
<td>23.50</td>
<td>4.83</td>
<td>1.50</td>
<td>7.25</td>
</tr>
<tr>
<td><strong>Total Unfactored DL</strong></td>
<td><strong>30.27</strong></td>
<td><strong>Total Factored DL</strong></td>
<td><strong>36.52</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Once the dead loads are determined, the corresponding moment and shear force are calculated and summarized in Table 4.4.

Table 4.4 Moment and Shear Force due to Dead Loads

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (kNm)</td>
<td>0.00</td>
<td>851.21</td>
<td>1513.27</td>
<td>1986.17</td>
<td>2269.90</td>
<td>2364.48</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>378.32</td>
<td>302.65</td>
<td>226.99</td>
<td>151.33</td>
<td>75.66</td>
<td>0.00</td>
</tr>
</tbody>
</table>

4.3.2 Live Load Analysis

The actual live load is determined based on the following steps:

1. Determine the moment and shear force caused by CL-W truck loads and CL-W lane loads on a set of points along the span. The truck loads are calculated based on influence line analysis from Chapter 3, and the results are summarized in Table 4.5. The lane loads are calculated by adding 80% of truck loads with an UDL of 9 kN/m, and the results are summarized in Table 4.6.

Table 4.5 Undistributed Moment and Shear Force due to Truck Loads

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0</th>
<th>2.5</th>
<th>5</th>
<th>7.5</th>
<th>10</th>
<th>12.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>362</td>
<td>812</td>
<td>1426</td>
<td>2093.2</td>
<td>2203.4</td>
<td>2226.25</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>404.8</td>
<td>342.3</td>
<td>285.2</td>
<td>225.3</td>
<td>167.1</td>
<td>116.3</td>
</tr>
</tbody>
</table>

Table 4.6 Undistributed Moment and Shear Force due to Lane Loads

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0</th>
<th>2.5</th>
<th>5</th>
<th>7.5</th>
<th>10</th>
<th>12.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>436.34</td>
<td>902.725</td>
<td>1590.8</td>
<td>2265.185</td>
<td>2437.72</td>
<td>2484.125</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>436.34</td>
<td>363.84</td>
<td>295.66</td>
<td>225.24</td>
<td>156.18</td>
<td>93.04</td>
</tr>
</tbody>
</table>

2. Calculate n as the number design lanes as well as its width.

\[ W_e = \frac{W_c}{n} = \frac{10}{2} = 5 \text{ m} \]
3. Determine \( \mu \) as the lane modification factor based on §5.6.4.4.

\[
\mu = \frac{W_e - 3.3}{0.6} = 2.83 \text{ which is not } \leq 1.0 \rightarrow \mu = 1.0
\]

4. Determine the distribution factors of moment and shear force respectively based on §5.6.4.3.

In this case, assume a type A highway and \( Le = 25m \). Based on Chapter 2, let \( S \), center to center spacing, equals to 2.7 m. Thus:

\[
\text{Moment: } F_T = \frac{s}{D_T\gamma_c(1+\mu\lambda)} = \frac{2.7}{3.63\times1.0\times(1+2.83\times0.09)} = 0.5928 \geq 1.05 \frac{nR_L}{N} = 0.47
\]

\[
\text{Shear: } F_T = \frac{s}{D_T\gamma_c(1+\mu\lambda)} = \frac{2.7}{3.40\times1.0\times(1+2.83\times0.0)} = 0.7941 \geq 1.05 \frac{nR_L}{N} = 0.47
\]

Where \( D, \gamma, \lambda \) can be found in Table 5.3 and 5.6.

\[
D_T = 4.60 - \frac{5.30}{\sqrt{L_e + 5}} = 3.63 \geq 2.80 \text{ for moment}
\]

\[
\gamma_c = 1.0 \text{ for both moment and shear, with } s \geq 2.0
\]

\[
\lambda = 0.10 - \frac{0.25}{L_e} = 0.09 \geq 2.80 \text{ for moment}
\]

5. Determine the unfactored moment and shear force along the girder for CL-W truck load and lane load:

\[
M_L = F_T F_S M_T
\]

\[
V_L = F_T F_S V_T
\]
Using $FT = 0.5928$ (for moment), $FT = 0.7941$ (for shear force), $FS = 1.0$

Dynamic load allowance = 0.25 from §3.8.4.5.3c

The distributed moment and shear force due to truck loads and lane loads are summarized in Table 4.7 and 4.8 respectively, and the result has shown that the truck loads will govern. Hence, the result calculated from truck loads will be carried on for any further calculations.

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>M. L (kNm)</td>
<td>0.00</td>
<td>601.69</td>
<td>1056.67</td>
<td>1551.06</td>
<td>1632.72</td>
<td>1649.65</td>
</tr>
<tr>
<td>VL (kN)</td>
<td>401.81</td>
<td>339.78</td>
<td>283.10</td>
<td>223.64</td>
<td>165.87</td>
<td>115.44</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>M. L (kNm)</td>
<td>0.00</td>
<td>535.14</td>
<td>943.03</td>
<td>1342.80</td>
<td>1445.08</td>
<td>1472.59</td>
</tr>
<tr>
<td>VL (kN)</td>
<td>346.50</td>
<td>288.93</td>
<td>234.78</td>
<td>178.86</td>
<td>124.02</td>
<td>73.88</td>
</tr>
</tbody>
</table>

### 4.3.3 Load Combinations

In this case, two particular load combinations that represents Serviceability Limit State and Ultimate Limit State are summarized in Table 4.9 and 4.10 respectively.

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>M. L (kNm)</td>
<td>0.00</td>
<td>1392.74</td>
<td>2464.27</td>
<td>3382.12</td>
<td>3739.35</td>
<td>3849.17</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>739.95</td>
<td>608.45</td>
<td>481.78</td>
<td>352.60</td>
<td>224.94</td>
<td>103.90</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>M. L (kNm)</td>
<td>0.00</td>
<td>2050.05</td>
<td>3622.43</td>
<td>5033.55</td>
<td>5514.76</td>
<td>5657.68</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>1139.61</td>
<td>942.84</td>
<td>755.18</td>
<td>562.79</td>
<td>373.28</td>
<td>196.25</td>
</tr>
</tbody>
</table>
4.4 AASHTO LRFD 2012 Design Loads [3]

Similar to the previous section, this section specifically focuses on defining any design loads based on AASHTO LRFD-14. It consists of dead loads, live loads, and corresponding distribution factors, and dynamic impact loads.

4.4.1 Dead Load Analysis

The dead loads are calculated based on self-weight of assumed Type IV prestressed I-girder, the 200-mm reinforced concrete deck, and the 3 inch (76.2 mm) bituminous pavement. The spacing of interior girders is designed as 2.7 m, which is used as the tributary area calculation. The unit weight of each type of material is listed in Table 4.11.

Table 4.11 Material Unit Weight from AASHTO LRFD-14

<table>
<thead>
<tr>
<th>Material</th>
<th>AASHTO LRFD-14 (kcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous Wearing Surface</td>
<td>0.140</td>
</tr>
<tr>
<td>Normal Weight Concrete with $f'c \leq 5.0 \text{ ksi}$</td>
<td>0.145</td>
</tr>
<tr>
<td>Normal Weight Concrete with $5.0 &lt; f'c \leq 15.0 \text{ ksi}$</td>
<td>$0.140 + 0.001 f'c$</td>
</tr>
</tbody>
</table>

Unlike the CSA S6-14, which defines the material unit weight based on its reinforcement, the AASHTO defines the unit weight based on its strength. In this case, 50 MPa (7.252 ksi) concrete is used for prestressed girder, and 40 MPa (5.802 ksi) concrete is used for reinforced concrete deck. Thus, the estimated unit weight of each structural component is calculated as follows:

**Unit weight of concrete girder:**

$$W_c = 0.140 + 0.001 \cdot f'c = 0.140 + 0.001 \cdot 7.252 = 0.1473 \text{ kcf}$$

**Unit weight of concrete deck:**

$$W_c = 0.140 + 0.001 \cdot f'c = 0.140 + 0.001 \cdot 5.802 = 0.1458 \text{ kcf}$$
Unit weight of wearing surface:

\[ W_w = 0.140 \text{ kcf} \]

Once the unit weight of each structural component is determined, the result of dead loads calculation is presented and summarized in Table 4.12.

Table 4.12 Dead Load Calculations

<table>
<thead>
<tr>
<th>Component</th>
<th>Component Area (m^2)</th>
<th>Dead Load (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>0.509</td>
<td>11.78</td>
</tr>
<tr>
<td>Deck</td>
<td>0.54</td>
<td>12.36</td>
</tr>
<tr>
<td>Pavement</td>
<td>0.20574</td>
<td>4.52</td>
</tr>
</tbody>
</table>

4.4.2 Live Load

The actual live load is determined based on the following steps:

1. Determine the number of design lanes based on §3.6.1.1.1:

\[ n = \frac{w}{12.0} = \frac{25}{12} = 2.083 \rightarrow n = 2.0 \text{ design lanes} \]

2. Determine the Young’s Modulus of concrete used in the girder and the deck based on §5.4.2.4:

**For Girder:**

\[ E_B = E_D = 33,000K_1W_C^{1.5}\sqrt{f'_c} = 33,000 \times 1 \times 0.1473^{1.5} \times \sqrt{7.252} = 5024 \text{ ksi} \]

**For Deck:**

\[ E_B = E_D = 33,000K_1W_C^{1.5}\sqrt{f'_c} = 33,000 \times 1 \times 0.1458^{1.5} \times \sqrt{5.802} = 4425 \text{ ksi} \]

3. Determine the type of cross section used in the design, which in this case is a k-type section.
4. Determine the longitudinal stiffness parameters based on §4.6.2.2.1-1:

\[ K_g = n(I + Ae_g^2) \]

Where

\[ n = \frac{E_B}{E_D} = 1.135 \]

\[ K_g = 1.135 \times (260,730 + 789 \times 33.2) = 1,283,833 \text{ in}^4 \]

5. Determine the live load moment distribution factors for the interior girder based on §4.6.2.2.2b:

For two or more design lanes loaded:

\[ DFM = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12.0 \times L \times t_s^3} \right)^{0.1} \]

For one design lane loaded:

\[ DFM = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{12.0 \times L \times t_s^3} \right)^{0.1} \]

Table 4.13 Bridge Geometry and Applicability Check

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
<th>Requirement</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>S – girder spacing, ft</td>
<td>8.86</td>
<td>3.5 ≤ S ≤ 16</td>
<td>OK</td>
</tr>
<tr>
<td>L – span length, ft</td>
<td>82</td>
<td>20 ≤ L ≤ 240</td>
<td>OK</td>
</tr>
<tr>
<td>ts – slab thickness, in.</td>
<td>7.87</td>
<td>4.5 ≤ ts ≤ 12</td>
<td>OK</td>
</tr>
<tr>
<td>Nb – number of beams</td>
<td>4</td>
<td>4 ≤ Nb</td>
<td>OK</td>
</tr>
</tbody>
</table>
For two or more design lanes loaded:

\[
DFM = 0.075 + \left( \frac{8.86}{9.5} \right)^{0.6} \left( \frac{8.86}{82} \right)^{0.2} \left( \frac{1,283,833}{12.0 \times 82 \times 7.87^3} \right)^{0.1} = 0.753 \to \text{Govern}
\]

For one design lane loaded:

\[
DFM = 0.06 + \left( \frac{8.86}{14} \right)^{0.4} \left( \frac{8.86}{82} \right)^{0.3} \left( \frac{1,283,833}{12.0 \times 82 \times 7.87^3} \right)^{0.1} = 0.531
\]

6. Determine live load shear force distribution factors for interior girder based on §4.6.2.2.3a:

For two or more design lanes loaded:

\[
DVF = 0.2 + \frac{S}{12} - \left( \frac{S}{35} \right)^{2.0} = 0.2 + \frac{8.86}{12} - \left( \frac{8.86}{35} \right)^{2.0} = 0.874 \to \text{Govern}
\]

For one design lane loaded:

\[
DVF = 0.36 + \frac{S}{25} = 0.36 + \frac{8.86}{25} = 0.714
\]

7. Determine the dynamic load allowance for truck load only based on §3.6.2:

\[
IM = 33\%
\]

Therefore, static load shall be increased by \((1 + \frac{IM}{100})\).
8. Determine the distributed shear force and moment due to live load including impact loading:

\[ M_{LT} = (\text{live load bending moment})(DFM)(1 + \frac{IM}{100}) \]

\[ V_{LT} = (\text{live load shear force})(DFM)(1 + \frac{IM}{100}) \]

The summarized result is shown in Table 4.14.

Table 4.14 Distributed Moment and Shear Force due to Truck load

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>731.51</td>
<td>1276.92</td>
<td>1619.48</td>
<td>1844.64</td>
<td>1911.31</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>213.63</td>
<td>189.53</td>
<td>164.61</td>
<td>141.34</td>
<td>117.24</td>
<td>92.92</td>
</tr>
</tbody>
</table>

9. Determine the design lane load based on §3.6.1.2.4:

The design lane load is determined by applying an UDL of 0.64 k/ft (9.34 kN/m) in the longitudinal direction with no dynamic load allowance. Shear force and bending moment along the span of the bridge are estimated using:

\[ M_{LL} = (\text{moment per lane due to lane load})(DFM) \]

\[ V_{LT} = (\text{shear per lane due to lane load})(DFV) \]

The result is summarized in Table 4.15.

Table 4.15 Distributed Moment and Shear Force due to Lane load

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>197.75</td>
<td>351.56</td>
<td>461.42</td>
<td>527.34</td>
<td>549.31</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>102.01</td>
<td>81.61</td>
<td>61.21</td>
<td>40.81</td>
<td>20.40</td>
<td>0.00</td>
</tr>
</tbody>
</table>
10. The total live load is calculated by adding the land load and truck load with dynamic loading included, and the result is summarized in the Table 4.16.

Table 4.16 Distributed Moment and Shear Force due to Total Live Load (LL+IM)

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>929.26</td>
<td>1628.48</td>
<td>2080.91</td>
<td>2371.99</td>
<td>2460.62</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>315.64</td>
<td>271.14</td>
<td>225.81</td>
<td>182.14</td>
<td>137.64</td>
<td>92.92</td>
</tr>
</tbody>
</table>

4.4.3 Load Combinations

In this design, the following three load combinations were determined based on §3.4.1:

**Strength I:** \( \gamma (DC) + \gamma_p(DW) + 1.75(LL + IM) \)

\[ \gamma_p \text{ for } DC = 1.25 \]

\[ \gamma_p \text{ for } DW = 1.50 \]

**Service I:** \( 1.00(DC + DW) + 1.00(LL + IM) \)

**Service III:** \( 1.00(DC + DW) + 0.80(LL + IM) \)

The final results are summarized in Table 4.17, 4.18 and 4.19. Each of the load combination serves a purpose in further design.

Table 4.17 Distributed Moment and Shear Force under Strength I Condition

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (kNm)</td>
<td>0.00</td>
<td>2665.57</td>
<td>4697.59</td>
<td>6066.76</td>
<td>6922.60</td>
<td>7193.20</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>1014.31</td>
<td>844.05</td>
<td>672.34</td>
<td>503.53</td>
<td>333.27</td>
<td>162.61</td>
</tr>
</tbody>
</table>
Table 4.18 Distributed Moment and Shear Force under Service I Condition

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (kNm)</td>
<td>0.00</td>
<td>1735.33</td>
<td>3061.48</td>
<td>3961.72</td>
<td>4521.49</td>
<td>4699.69</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>673.89</td>
<td>557.74</td>
<td>440.76</td>
<td>325.44</td>
<td>209.29</td>
<td>92.92</td>
</tr>
</tbody>
</table>

Table 4.19 Distributed Moment and Shear Force under Service III Condition

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (kNm)</td>
<td>0.00</td>
<td>1549.47</td>
<td>2735.78</td>
<td>3545.54</td>
<td>4047.09</td>
<td>4207.56</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>610.76</td>
<td>503.51</td>
<td>395.60</td>
<td>289.01</td>
<td>181.77</td>
<td>74.34</td>
</tr>
</tbody>
</table>


This section presents the calculations of the design loads based on the requirements stipulated by the Canadian Bridge Design code CSA S6-66 [M1]. It consists of the calculation of the dead loads, the live loads and their corresponding distribution factors to determine the loads transmitted to the pre-stressed concrete I-girders.

4.5.1 Dead Loads

The dead loads from the self-weight of the pre-stressed concrete I-girder, reinforced concrete slab and covering materials specified in CSA S6-66 are the same as the new CSA S6-14 provision, despite the unit difference. Table 4.4.1 presents the calculated uniformly distributed dead load for main structural components with units in metric.

Table 4.20 Unit Weights, Cross-sectional Areas and Corresponding Dead Loads

<table>
<thead>
<tr>
<th>Component</th>
<th>Component Area (m²)</th>
<th>Unit Weight (kN/m³)</th>
<th>Dead Load (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I Girder</td>
<td>0.509</td>
<td>24.5</td>
<td>12.4705</td>
</tr>
<tr>
<td>Deck</td>
<td>0.54</td>
<td>24</td>
<td>12.96</td>
</tr>
<tr>
<td>Pavement</td>
<td>0.20574</td>
<td>23.5</td>
<td>4.83489</td>
</tr>
</tbody>
</table>

The total unfactored dead load is then obtained by summing all the component dead loads in Table 4.20 up:

Total Unfactored Dead Load = 30.27 kN/m
Once the dead loads are determined, the corresponding moment and shear force are calculated and summarized in Table 4.4.2.

Table 4.21 Moment and Shear Force due to Dead Loads

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (kNm)</td>
<td>0.00</td>
<td>851.21</td>
<td>1513.27</td>
<td>1986.17</td>
<td>2269.90</td>
<td>2364.48</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>378.32</td>
<td>302.65</td>
<td>226.99</td>
<td>151.33</td>
<td>75.66</td>
<td>0.00</td>
</tr>
</tbody>
</table>

4.5.2 Determination of Live Loads

This section calculates the design live loads to be applied to the bridge and the corresponding distribution factors of load transmission to an internal girder based on CSA S6-66. These live loads are imposed onto the bridge as either a truck load or a lane load, whichever one governs. The following procedures are used to determine the live loads.

1. Determination of Lane Width (CSA S6-66 §5.1.6.1):

\[ W = W_c / N \]

Where,

\[ W_c = 28.8 \text{ (Roadway width between curbs, assume curbs 2ft each)} \]

\[ N = 2 \text{ (Table 1)} \]

\[ W = 28.8 / 2 = 14.4 \text{ ft > 10 ft \rightarrow OK} \]

2. Determination of Moment Distribution Factor:

According to CSA S6-66 §5.2.1.2, the live load of an interior girder is determined by applying a fraction of the wheel load based on the conditions illustrated in Table 4. Described in Table 4 of CSA S6-66, a pre-stressed I-girders with a spacing of \( S = 8.86 \text{ ft} \) and with bridge designed for two or more traffic lanes, the moment distribution factor, MDF, is calculated as follows:
\[ MDF = 8.86 \, ft / 5.5 = 1.61 \]

This factor is to be applied to the individual wheel loads of the design truck, however, since axle loads were calculated in Ch. 3: Influence lines, the fraction of axle load that is transferred to each girder is determined by dividing the MDF by 2:

\[ MDF = 1.61/2 = 0.805 \]

Therefore, a distribution factor of 0.805 will be applied to the moments produced by the live loads in order to determine the loads per girder.

3. Determination of the Shear Distribution Factor:

According to CSA S6-66 §5.2.1.1 the distribution of shear is determined by the method prescribed for the moment distribution.

\[ SDF = 8.86 \, ft / 5.5 = 1.61 \]

It is applied to the shear and moment envelope values as:

\[ SDF = 1.61/2 = 0.805 \]

4. Determination of Impact Factor for Truck Loads

According to CSA S6-66 §5.1.11, live loads produced by HS design trucks need to be increased by an impact factor to account for dynamic, vibratory and impact effects. For a span length of \( L = 25m = 82.02 \, ft \)

\[
\text{Impact Factor} = \frac{50}{L + 125} = \frac{50}{82.02 + 125} = 0.242 < 0.30 \quad \text{OK}
\]

Therefore, the design truck loads are increased by 24.2% to account for impact loads.
5. Determination of Live Loads to be used for design

i) The live loads due to the HS design truck are determined based on the shear and moment design envelopes calculated in Ch. 3: Influence Lines for various locations along the span of the bridge. These shear or moment values are multiplied by the distribution factor of 0.805 and increased by the impact factor of 24.2%. These loads are shown in the table below.

Table 4.22 Distributed Unfactored Moment and Shear Force

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>629.30</td>
<td>1098.50</td>
<td>1393.20</td>
<td>1586.90</td>
<td>1644.25</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>283.70</td>
<td>251.70</td>
<td>218.60</td>
<td>187.70</td>
<td>155.70</td>
<td>123.40</td>
</tr>
</tbody>
</table>

Table 4.23 Distributed Factored Moment and Shear Force

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>629.18</td>
<td>1098.29</td>
<td>1392.94</td>
<td>1586.60</td>
<td>1643.94</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>283.65</td>
<td>251.65</td>
<td>218.56</td>
<td>187.66</td>
<td>155.67</td>
<td>123.38</td>
</tr>
</tbody>
</table>

ii) Lane loads are also considered and are calculated based on CSA S6-66 Figure 2. This stipulates that a uniformly distributed load of 9.34 kN/m should be applied to the span with a 115.7 kN concentrated load (for shear calculations) or a 80.1 kN concentrated load (for moment calculations) should be applied as well at a location that yields the most stress. These loads were calculated and their respective shear and moments.

Table 4.24 Distributed Unfactored Moment and Shear Force

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>362.75</td>
<td>667.13</td>
<td>913.16</td>
<td>1100.82</td>
<td>1230.13</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>174.57</td>
<td>151.23</td>
<td>127.88</td>
<td>104.54</td>
<td>81.19</td>
<td>57.85</td>
</tr>
</tbody>
</table>

4-16
Table 4.25 Distributed Factored Moment and Shear Force

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>292.01</td>
<td>537.04</td>
<td>735.09</td>
<td>886.16</td>
<td>990.25</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>152.57</td>
<td>132.17</td>
<td>111.77</td>
<td>91.37</td>
<td>70.96</td>
<td>50.56</td>
</tr>
</tbody>
</table>

iii) Once the truck loads and lane loads were determined, the larger set of values between the two were selected to be used in subsequent calculations. This is based on CSA S6-66 §5.1.8.2, which states that the loads that produce maximum stress shall be used in the design as the live load component. It was determined that the shear and moment produced by the HS truck governed

Table 4.26 Governing Distributed Moment and Shear Force

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfactored Moment (kNm)</td>
<td>0.00</td>
<td>629.18</td>
<td>1098.29</td>
<td>1392.94</td>
<td>1586.60</td>
<td>1643.94</td>
</tr>
<tr>
<td>Unfactored Shear (kN)</td>
<td>283.65</td>
<td>251.65</td>
<td>218.56</td>
<td>187.66</td>
<td>155.67</td>
<td>123.38</td>
</tr>
</tbody>
</table>

4.5.3 Determination of Total Design Loads

The total design loads are determined by taking the sum of the Live Loads, Impact Loads and the Dead Loads. According to CSA S6-66 §9.3.1.5 the design load for pre-stressed concrete should be determined using the following load combination:

**Ultimate Loads**: \( 1.5D + 2.5(L + I) \)

**Working Loads**: \( D + (L + I) \)

The design loads as per CSA S6-66 are presented in the tables below, for both Service Limit States (SLS) and Ultimate Limit States (ULS):

Table 4.27 Distributed Moment and Shear Force Under SLS

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (kNm)</td>
<td>0.00</td>
<td>1480.39</td>
<td>2611.56</td>
<td>3379.10</td>
<td>3856.50</td>
<td>4008.42</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>661.96</td>
<td>554.31</td>
<td>445.55</td>
<td>338.99</td>
<td>231.33</td>
<td>123.38</td>
</tr>
</tbody>
</table>
4.6 Summary of Design Loads

Table below summarizes the design loads for all three codes, which are subsequently used in the following chapters of this report.

Table 4.29 Summary of Design Moment and Shear Force

<table>
<thead>
<tr>
<th>Distance from Support (m)</th>
<th>0.00</th>
<th>2.50</th>
<th>5.00</th>
<th>7.50</th>
<th>10.00</th>
<th>12.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (kNm)</td>
<td>0.00</td>
<td>2849.77</td>
<td>5015.63</td>
<td>6461.59</td>
<td>7371.35</td>
<td>7656.57</td>
</tr>
<tr>
<td>Shear (kN)</td>
<td>1276.59</td>
<td>1083.11</td>
<td>886.88</td>
<td>696.15</td>
<td>502.67</td>
<td>308.44</td>
</tr>
</tbody>
</table>

4.7 Conclusion

In this chapter, all the required design loads are determined according to three design codes: CSA S6-14, AASHTO LRFD-14, and CSA S6-66. For the scope of this project, only dead loads and live loads are calculated, assuming other loads such as snow loads, wind loads and seismic loads are trivial for the design. The dead loads considered in this chapter include the self-weight of prestressed I girders, concrete deck and pavement material, and the live loads include truck load and lane load.
Reference:


# Chapter 5 Design of Prestressed Concrete Girder

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5.1 Introduction

In this chapter, detailed designs of the interior prestressed girder will be performed following three different codes: CSA S6-14, AASHTO LRFD-14 and CSA S6-66. Each design includes the design of the cross-section geometry, tendon profile, check of concrete stress at critical stages, flexural design, shear design, and deflection check.

5.2 Cross-Section Geometry

As stated in the previous chapter, AASHTO Type IV girder would be used in this design. Thus, the span to depth ratio would be around 18 in this case, which is an ideal situation. The detailed geometry is shown in Figure 5.1, and the cross-section property is shown in Table 5.1.

![Figure 5.1 Geometry of Type IV Girder](image)

Table 5.1 Basic Geometry Property of Type IV I-Girder [1]

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height</td>
<td>1.372 m</td>
<td>54 inch</td>
</tr>
<tr>
<td>Area</td>
<td>0.509 m²</td>
<td>789 in²</td>
</tr>
<tr>
<td>y (bottom)</td>
<td>0.628 m</td>
<td>24.73 inch</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>0.1085 m⁴</td>
<td>260,730 in⁴</td>
</tr>
</tbody>
</table>
5.3 CSA S6-14 [2]

In this section, the interior girder will be designed using CSA S6-14. In particular, the tendon profile as well as the transverse reinforcement will be determined so that the flexural capacity, shear capacity and deflection will satisfy CSA S6-14.

5.3.1 Prestressing Design

Assuming that the centroid of the tendons is located 100 mm above the bottom face of the girder, which means:

\[ e_g = 628 - 100 = 528 \text{ mm} \]

At ULS, the maximum tensile stress allowed at the bottom girder is:

\[ f_{bg} = 0.4\sqrt{f'_c} = 2.83 \text{ MPa} \]

The stress in the bottom of the girder is:

\[
\begin{align*}
f_{bg} &= -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}} + \frac{M_{da} + M_l}{S_{bc}} \\
3.54 \text{ MPa} &\geq -\frac{P_f}{509 \times 10^3 \text{ mm}^2} - \frac{P_f \times 528 \text{ mm}}{172.8 \times 10^6 \text{ mm}^3} + \frac{974.3 \text{ kN} \cdot \text{m} + 1012.5 \text{ kN} \cdot \text{m}}{172.8 \times 10^6 \text{ mm}^3} \\
&\quad + \frac{195.3 \text{ kN} \cdot \text{m} + 1406.3 \text{ kN} \cdot \text{m}}{258.4 \times 10^6 \text{ mm}^3}
\end{align*}
\]

Thus, \( P_f \) is calculated to be at least 3481 kN.

Assuming that the low-relaxation strand has strength of 1080 MPa (\( f_{pf} \)) considering all the long-term losses. Therefore:

\[ A_{ps} \geq \frac{3481 \text{ kN}}{1080 \text{ MPa}} = 3223 \text{ mm}^2 \]

Using 24 size 15 strand, which provides an area of 3360 mm\(^2\).
5.3.2 Choose the Tendon Profile
In this case, 10 of 24 strands will be draped to the third-points of the span.

5.3.3 Check the Concrete Stresses at Service Load

<table>
<thead>
<tr>
<th></th>
<th>Stress Limit at Transfer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tensile Limit (MPa)</td>
</tr>
<tr>
<td><strong>Initial Condition</strong></td>
<td>3.162</td>
</tr>
<tr>
<td><strong>Concrete Placing</strong></td>
<td>3.536</td>
</tr>
<tr>
<td><strong>Final Condition</strong></td>
<td>3.536</td>
</tr>
</tbody>
</table>

5.3.3.1 Initial Condition at Transfer
Assume the low-relaxation strands has an initial strength of 1290 MPa ($f_{p,bed}$).
Thus, $P_i = 1290 \text{ MPa} \times 3360 \text{ mm}^2 = 4334.4 \text{ kN}$

\[
f_t = -\frac{P_i}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ds}}{S_{tg}}
\]

\[
f_b = -\frac{P_i}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}}
\]
5.3.3.2 Conditions at Time of Placing Wet Concrete on Girder

At this stage, the concrete in the girder has reached its normal capacity of $f'_c$, and prestressing force will be taken as $P_f$ to be conservative.

\[
f_t = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ls}}{S_{tg}}
\]

\[
f_b = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ls}}{S_{bg}}
\]

5.3.3.3 Final Conditions

\[
f_{tg} = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ls}}{S_{tg}} + \frac{M_{da} + M_l}{S_{tc}}
\]

\[
f_{bg} = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ls}}{S_{bg}} + \frac{M_{da} + M_l}{S_{bc}}
\]

Table 5.2 Summary of Flexural Design Calculation

<table>
<thead>
<tr>
<th>Distance from Support</th>
<th>0.1L</th>
<th>0.2L</th>
<th>0.3L</th>
<th>0.4L</th>
<th>0.5L</th>
</tr>
</thead>
<tbody>
<tr>
<td>dv 1150</td>
<td>2500</td>
<td>5000</td>
<td>7500</td>
<td>10000</td>
<td>12500</td>
</tr>
</tbody>
</table>

a) Initial Conditions at Transfer

| Pi (kN) | 4334.4 | 4334.4 | 4334.4 | 4334.4 | 4334.4 | 4334.4 |
| eg (mm) | 300    | 336    | 403    | 470    | 470    | 470    |
| Mdg (kNm) | 171.02 | 350.73 | 623.53 | 818.38 | 935.29 | 974.26 |
| ft (MPa) | -0.82  | -0.98  | -0.86  | -0.21  | -1.01  | -1.27  |
| fb (MPa) | -15.05 | -14.91 | -15.02 | -15.57 | -14.89 | -14.67 |

b) Condition at Time of Placing Wet Concrete on Girder

| Pi (kN) | 3628.8 | 3628.8 | 3628.8 | 3628.8 | 3628.8 | 3628.8 |
| Mds (kNm) | 177.73 | 364.50 | 648.00 | 850.50 | 972.00 | 1012.50 |
| Mdg + Mds (kNm) | 348.75 | 715.23 | 1271.53 | 1668.88 | 1907.29 | 1986.76 |
| ft (MPa) | -2.09  | -3.69  | -5.83  | -6.88  | -8.50  | -9.05  |
| fb (MPa) | -11.41 | -10.05 | -8.23  | -7.34  | -5.96  | -5.50  |

c) Final Conditions

| Mda (kNm) | 66.31 | 135.98 | 241.75 | 317.30 | 362.63 | 377.73 |
| Ml (kNm)  | 333.24 | 683.44 | 1215.00 | 1594.69 | 1822.50 | 1898.44 |
| Mda + Ml (kNm) | 399.55 | 819.42 | 1456.75 | 1911.98 | 2185.13 | 2276.17 |
| fts (MPa) | -1.05  | -2.15  | -3.83  | -5.03  | -5.75  | -5.99  |
| ftg (MPa) | -2.77  | -5.09  | -8.31  | -10.13 | -12.22 | -12.92 |
| fbg (Mpa) | -10.73 | -8.65  | -5.75  | -4.09  | -2.24  | -1.63  |
5.3.4 Check the Flexural Capacity

Under the ULS loading, the $M_u$ at mid span is 6079.4 kNm.

$$f_{ps} = f_{pu} \left(1 - \frac{\gamma_p}{\beta_1} \frac{f_{pu}}{f'_c}\right) = 1860 \text{ MPa} \times \left(1 - \frac{0.28}{0.65} \times 0.00097 \times \frac{1860 \text{ MPa}}{50 \text{ MPa}}\right) = 1844 \text{ MPa}$$

$$a = \frac{A_{ps}f_{ps}}{0.85f'_c b} = \frac{3920 \text{ mm}^2 \times 1844 \text{ MPa}}{0.85 \times 50 \text{ MPa} \times 2336.8 \text{ mm}} = 72.8 \text{ mm}$$

The design flexural strength is determined as:

$$\phi M_n = \phi A_{ps}f_{ps} \left(d_p - \frac{a}{2}\right) = 0.95 \times 3920 \text{ mm}^2 \times 1844 \text{ MPa} \times \left(1283 - \frac{72.8}{2}\right) = 8561.7 \text{ kNm}$$

5.3.5 Check the Reserve Strength after Cracking

Cracking stress:

$$f_{cr} = 0.4\sqrt{f'_c} = 0.4\sqrt{50} = 2.83 \text{ MPa}$$

Additional tensile stress required to crack:

$$f_{cr} - f_{bg} = 2.83 \text{ MPa} - 5.61 \text{ MPa} = 8.44 \text{ MPa}$$

Additional moment caused by this stress:

$$s_{bc} \times 8.44 \text{ MPa} = 2180.7 \text{ kNm}$$

Therefore the cracking moment is determined as:

$$M_{cr} = 1986.8 \text{ kNm} + 1406.2 \text{ kNm} + 2180.7 \text{ kNm} = 5573.7 \text{ kNm}$$

Hence

$$\frac{\phi M_n}{M_{cr}} = \frac{8561.7}{5573.7} = 1.54 \geq 1.2$$
5.3.6 Check the Deflections

As shown in the previous calculation, the girder will remain uncracked under service load. Therefore, an elastic and uncracked response is assumed in the deflection calculation. Both short-term and long-term deflection will be determined.

a) Immediate deflection due to short-term live loading,
   - Total live load is determined as:

\[
P = DFV \times Truck \ Load \times IM = 0.682 \times 620 \, kN \times 1.25 = 528.6 \, kN
\]

Thus,

\[
\Delta_L = \frac{PL^3}{48EI} = \frac{528.6 \, kN \times (25 \, m)^3}{48 \times 28000 \, MPa \times 0.2463 \, m^4} = 24.9 \, mm
\]

Which is less than the limit of \( l/1000 \).

b) Deflections at the Erections,
   - The elastic deflection due to girder self-weight is

\[
\Delta_{DL} = \frac{5wL^4}{384EI} = \frac{5 \times 12.47 \, kN/m \times (25 \, m)^4}{384 \times 28000 \, MPa \times 0.2463 \, m^4} = 20.9 \, mm
\]

- The elastic deflection due to concrete deck is

\[
\Delta_{SL} = \frac{5wL^4}{384EI} = \frac{5 \times 12.96 \, kN/m \times (25 \, m)^4}{384 \times 28000 \, MPa \times 0.2463 \, m^4} = 21.7 \, mm
\]

- The elastic deflection due to asphalt pavement is

\[
\Delta_{PL} = \frac{5wL^4}{384EI} = \frac{5 \times 2.5 \, kN/m \times (25 \, m)^4}{384 \times 28000 \, MPa \times 0.2463 \, m^4} = 1.84 \, mm
\]

- The upward elastic camper due to prestressing is
\[ \Delta_c = -\frac{14 PeL^2}{8EI} - \left[ \frac{e_c - \beta^2}{6} (e_c - e_e) \right] \frac{10 PL^2}{EI} = -46.0 \text{ mm} \]

- Total deflection at erection is calculated as

\[ 1.85 \times \Delta_{DL} + 1.8 \times \Delta_c = 1.85 \times 20.9 + 1.8 \times (-46.0) = -44.1 \text{ mm} \]

c) Long-term deflection,

\[ 2.4 \times \Delta_{DL} + 2.2 \times \Delta_c + 2.3 \times \Delta_{SL} + 3.0 \times \Delta_{PL} = 4.43 \text{ mm} \]

### 5.3.7 Design for Shear Reinforcement

Assuming that the #10M rebar is used with \( A_v = 200 \text{ mm}^2 \), a sample calculation is provided at the location of \( d_v \):

1. Determine \( d_v \) as the first critical location that needs to be designed for:

\[ d_v = \max(0.9d, 0.72h) = 1150 \text{ mm} \]

2. Determine the vertical component of the prestressing force \( V_p \):

\[ V_p = A_{ps} (0.6f_{pu}) \left( \frac{e_c - e_e}{\text{harping distance}} \right) = 41.8 \text{ kN} \]

3. Equivalent crack spacing parameter \( S_{ze} = 300 \text{ mm} \) in this case since at least minimum shear reinforcement will be provided.

4. Determine the longitudinal strain at the centroid axis:

\[ \varepsilon_x = \frac{M_f/d_v + V_f - V_p + 0.5N_f - A_{ps}A_{po}}{2(E_sA_s + E_pA_{ps})} = 0 \]
5. Determine the value $\beta$:

$$\beta = \left[ \frac{0.4}{(1 + 1500\varepsilon_x)} \right] \left[ \frac{1300}{(1000 + S_{ze})} \right] = 0.4$$

6. Determine the angle of inclination $\theta$:

$$\theta = (29 + 7000\varepsilon_x) \left( 0.88 + \frac{S_{ze}}{2500} \right) = 29$$

7. Determine the shear resistance contributed by concrete component $V_c$:

$$V_c = \beta \phi \sqrt{f'_c b_v d_v} = 495.2 \text{ kN}$$

8. Determine the shear resistance that needs to be provided by shear reinforcement $V_s$

$$V_{sreq} = V_f - V_p - V_c = 512.0 \text{ kN}$$

9. Determine the required spacing $s_{req}$:

$$s_{req} = \frac{\phi_s A_v f_y d_v \cot \theta}{V_s} = 291.7 \text{ mm}$$

10. Check the maximum spacing $s_{max}$:

$$s_{max} = \min(0.75d_v, 600) = 600 \text{ mm}$$

11. Check if the shear reinforcement meets the minimum requirement:

$$A_{vmin} = \frac{0.06 \sqrt{f'_c b_v s}}{f_y} = 53.8 \text{ mm}^2$$
12. Check if the longitudinal tendons have exceeded the capacity:

\[ F_{lt} = M_f/d_v + 0.5N_f + (V_f - V_p - V_s)\cot\theta = 2159.7 \text{ kN} \]

\[ F_p = \phi_{ps}f_{ps}A_{ps} = 6777.7 \text{ kN} \]

Table 5.3 Summary of Shear Design Calculation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Distance from the Support, m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.15</td>
</tr>
<tr>
<td>Vf, kN</td>
<td>1049.09</td>
</tr>
<tr>
<td>Mt, kNm</td>
<td>1013.78</td>
</tr>
<tr>
<td>Vp, kN</td>
<td>35.85</td>
</tr>
<tr>
<td>Ex</td>
<td>0.00</td>
</tr>
<tr>
<td>(\theta)</td>
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</tr>
<tr>
<td>(\beta)</td>
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<tr>
<td>Vc, kN</td>
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<tr>
<td>Vs req, kN</td>
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<tr>
<td>Sreq, mm</td>
<td>288.36</td>
</tr>
<tr>
<td>Smax, mm</td>
<td>600.00</td>
</tr>
<tr>
<td>Sdesign, mm</td>
<td>250.00</td>
</tr>
<tr>
<td>Vs design, kN</td>
<td>597.50</td>
</tr>
<tr>
<td>Amin, mm²</td>
<td>53.83</td>
</tr>
<tr>
<td>Vc+Vs design, kN</td>
<td>1092.72</td>
</tr>
<tr>
<td>Flt, kN</td>
<td>2170.52</td>
</tr>
<tr>
<td>Fp, kN</td>
<td>5809.44</td>
</tr>
</tbody>
</table>

Table 5.4 Summary of Stirrup Design

<table>
<thead>
<tr>
<th>Distance Spacing</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.15 - 2.4</td>
<td>250 mm</td>
</tr>
<tr>
<td>2.4 - 5.2</td>
<td>350 mm</td>
</tr>
<tr>
<td>5.2 - 12.5</td>
<td>600 mm</td>
</tr>
</tbody>
</table>

5.3.8 Design for Shrinkage and Temperature Reinforcement

CSA S6-14 requires the design to have reinforcement provided normal to the principal reinforcement for the purpose of shrinkage and temperature control. The minimum
requirement for these shrinkage and temperature reinforcement is 500 $mm^2/m$, with rebar spacing no more than 300 $mm$.

- For reinforcements parallel to the span:

$$A_{s\text{req}} = \frac{500 \, mm^2}{m} \times 1.372 \, m = 686 \, mm^2$$

Provide 7 #10M bars with $A_s = 700 \, mm^2$.

$$s = \frac{1372 \, mm}{7} = 196 \, mm \rightarrow use \ 200 \, mm$$

- For reinforcements perpendicular to the span:

$$A_{s\text{req}} = \frac{500 \, mm^2}{m} \times 1 \, m = 50 \, mm^2$$

Provide 5 #10M bars with $A_s = 500 \, mm^2$.

$$s = \frac{1000 \, mm}{5} = 200 \, mm \rightarrow use \ 200 \, mm$$

### 5.4 AASHTO LRFD-14 [3]

In this section, the interior girder will be designed using AASHTO. In particular, the tendon profile as well as the transverse reinforcement will be determined so that the flexural capacity, shear capacity and deflection will satisfy AASHTO LRFD-14.

#### 5.4.1 Prestressing Design

Assuming that the centroid of the tendons is located 100 mm above the bottom face of the girder, which means:
\[ e_g = 628 - 100 = 528 \text{ mm} \]

At ULS, the maximum tensile stress allowed at the bottom girder is:

\[ f_{bg} = 6\sqrt{f'_c} = 511 \text{ psi} = 3.52 \text{ MPa} \]

The stress in the bottom of the girder is:

\[
f_{bg} = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}} + \frac{M_{da} + M_t}{S_{bc}}
\]

\[
3.52 \text{ MPa} \geq -\frac{P_f}{509 \times 10^3 \text{ mm}^2} - \frac{P_f \times 528 \text{ mm}}{172.8 \times 10^6 \text{ mm}^3} + \frac{974.3 \text{ kNm} + 1012.5 \text{ kNm}}{172.8 \times 10^6 \text{ mm}^3} + \frac{195.3 \text{ kNm} + 1406.3 \text{ kNm}}{258.4 \times 10^6 \text{ mm}^3}
\]

Thus, \( P_f \) is calculated to be at least 3432 kN.

Assuming that the low-relaxation strand has strength of 1080 MPa (\( f_{pf} \)) considering all the long-term losses.

Therefore:

\[
A_{ps} \geq \frac{3432 \text{ kN}}{1080 \text{ MPa}} = 3178 \text{ mm}^2
\]

Using 24 size 15 strand, which provides an area of 3360 mm².

5.4.2 Choose the Tendon Profile

In this case, 10 of 24 strands will be draped to the third-points of the span.
5.4.3 Check the Concrete Stresses at Service Load

Table 5.5 Stress Limit for ASSHTO LRFD-14

<table>
<thead>
<tr>
<th></th>
<th>Stress Limit at Transfer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tensile Limit (MPa)</td>
</tr>
<tr>
<td>Initial Condition</td>
<td>3.155</td>
</tr>
<tr>
<td>Concrete Placing</td>
<td>3.528</td>
</tr>
<tr>
<td>Final Condition</td>
<td>3.528</td>
</tr>
</tbody>
</table>

5.4.3.1 Initial Condition at Transfer

Assume the low-relaxation strands has an initial strength of 1290 MPa ($f_{p,bed}$).

Thus, $P_t = 1290 \text{ MPa} \times 3360 \text{ mm}^2 = 4334.4 \text{ kN}$

$$f_t = -\frac{P_i}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ds}}{S_{tg}}$$

$$f_b = -\frac{P_i}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}}$$
5.4.3.2 Conditions at Time of Placing Wet Concrete on Girder

At this stage, the concrete in the girder has reached its normal capacity of $f'c$, and prestressing force will be taken as $P_f$ to be conservative.

$$f_t = \frac{P_f}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ds}}{S_{tg}}$$

$$f_b = \frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}}$$

5.4.3.3 Final Conditions

$$f_{tg} = \frac{P_f}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ds}}{S_{tg}} + \frac{M_{da} + M_l}{S_{tc}}$$

$$f_{bg} = \frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}} + \frac{M_{da} + M_l}{S_{bc}}$$
5.4.4 Check the Flexural Capacity

Under the ULS loading, the $M_u$ at mid span is 6079.4 kNm.

$$f_{ps} = f_{pu} \left( 1 - \frac{\gamma_p}{\beta_1} \frac{f_{pu}}{f'_c} \right) = 1860 \text{ MPa} \times \left( 1 - \frac{0.28}{0.65} \times 0.00097 \times \frac{1860 \text{ MPa}}{50 \text{ MPa}} \right) = 1831 \text{ MPa}$$

$$a = \frac{A_{ps}f_{ps}}{0.85f'_c b} = \frac{3360 \text{ mm}^2 \times 1831 \text{ MPa}}{0.85 \times 50 \text{ MPa} \times 2336.8} = 62.0 \text{ mm}$$

The design flexural strength is determined as:

$$\phi M_n = \phi A_{ps}f_{ps} \left( d_p - \frac{a}{2} \right) = 0.95 \times 3360 \text{ mm}^2 \times 1831 \text{ MPa} \times \left( 1283 - \frac{62}{2} \right) \text{ mm} = 8451.3 \text{ kNm}$$

Which is greater than the factored moment ($M_f = 6730 \text{ kNm}$) this specimen will experience.
5.4.5 Check the Reserve Strength after Cracking

Cracking stress:

\[ f_{cr} = 7.5\sqrt{f'_c} = 7.5\sqrt{7252} = 639 \text{ psi} = 4.40 \text{ MPa} \]

Additional tensile stress required to crack:

\[ f_{cr} - f_{bg} = 4.40 \text{ MPa} - (-1.57 \text{ MPa}) = 5.97 \text{ MPa} \]

Additional caused by this stress:

\[ s_{bc} \times 5.97 \text{ MPa} = 1543.5 \text{ kNm} \]

Therefore the cracking moment is determined as:

\[ M_{cr} = 1885.9 \text{ kNm} + 2187.5 \text{ kNm} + 1543.5 \text{ kNm} = 5617 \text{ kNm} \]

Hence

\[ \frac{\phi M_n}{M_{cr}} = \frac{8451.3 \text{ kNm}}{5617 \text{ kNm}} = 1.50 \geq 1.2 \]

5.4.6 Check the Deflections

As shown in the previous calculation, the girder will remain uncracked under service load. Therefore, an elastic and uncracked response is assumed in the deflection calculation. Both short-term and long-term deflection will be determined.

d) Immediate deflection due to short-term live loading,

Total live load is determined as:

\[ P = DFV \times Truck \ Load \times IM = 0.753 \times 325 \text{ kN} \times 1.33 = 325.5 \text{ kN} \]
Thus,

\[
\Delta_L = \frac{PL^3}{48EI} = \frac{325.5 \text{ kN} \times (25 \text{ m})^3}{48 \times 28000 \text{ MPa} \times 0.2463 \text{ m}^4} = 15.4 \text{ mm}
\]

Which is less than the limit of \(l/1000\).

e) Deflections at the Erections,

- The elastic deflection due to girder self-weight is

\[
\Delta_{DL} = \frac{5wl^4}{384EI} = \frac{5 \times 11.78 \text{ kN/m} \times (25 \text{ m})^4}{384 \times 28000 \text{ MPa} \times 0.1085 \text{ m}^4} = 19.72 \text{ mm}
\]

- The elastic deflection due to concrete deck is

\[
\Delta_{SL} = \frac{5wl^4}{384EI} = \frac{5 \times 12.36 \text{ kN/m} \times (25 \text{ m})^4}{384 \times 28000 \text{ MPa} \times 0.1085 \text{ m}^4} = 20.69 \text{ mm}
\]

- The elastic deflection due to asphalt pavement is

\[
\Delta_{PL} = \frac{5wl^4}{384EI} = \frac{5 \times 4.52 \text{ kN/m} \times (25 \text{ m})^4}{384 \times 28000 \text{ MPa} \times 0.2463 \text{ m}^4} = 3.33 \text{ mm}
\]

- The upward elastic camper due to prestressing is

\[
\Delta_c = -\frac{14}{24} \frac{P_e l^2}{8EI} - \left[ \frac{\beta^2}{6} (e_c - e_e) \right] \frac{10}{24} \frac{PL^2}{EI} = -39.39 \text{ mm}
\]

- Total deflection at erection is calculated as

\[
1.85 \times \Delta_{DL} + 1.8 \times \Delta_c = -34.42 \text{ mm}
\]
f) Long-term deflection,

\[ 2.4 \times \Delta_{DL} + 2.2 \times \Delta_{c} + 2.3 \times \Delta_{SL} + 3.0 \times \Delta_{PL} = -18.32 \text{ mm} \]

5.4.7 Design for Shear Reinforcement

Assuming that the #10M rebar is used with \( A_v = 200 \text{ mm}^2 \), a sample calculation is provided at the location of \( d_v \):

1. Determine \( d_v \) as the first critical location that needs to be designed for:

\[ d_v = \max(0.9d, 0.72h) = 1.15 \text{ m} \]

2. Determine the vertical component of the prestressing force \( V_p \):

\[ V_p = A_{ps}(0.6f_{pu})\Bigg(\frac{e_c-e_e}{\text{harping distance}}\Bigg) = 41.83 \text{ kN} \]

3. Determine the longitudinal strain at the centroid axis:

\[ \varepsilon_s = \frac{\left| \frac{M_f}{d_v} \right| + 0.5N_f + \left| V_f - V_p \right| - A_{ps}A_{po}}{E_sA_s + E_pA_{ps}} \]

\[ = 0 \]

4. Determine the value \( \beta \):

\[ \beta = \frac{4.8}{(1 + 750\varepsilon_s)} = 4.8 \]

5. Determine the angle of inclination \( \theta \):

\[ \theta = 29 + 3500\varepsilon_s = 29 \]

6. Determine the shear resistance contributed by concrete component \( V_c \):
\[ V_c = 0.0316 \sqrt{f'_c b_v d_v} = 657.5 \text{ kN} \]

7. Determine the shear resistance that needs to be provided by shear reinforcement \( V_s \):

\[ V_s = \frac{V_u}{\phi} - V_p - V_c = 414.0 \text{ kN} \]

8. Determine the required spacing \( s_{req} \):

\[ s_{req} = \frac{A_v f_y d_v \cot \theta}{V_s} = 401 \text{ mm} \]

9. Check the maximum spacing \( s_{max} \):

\[ s_{max} = \min(0.8d_v, 24) = 24 \text{ inch} = 600 \text{ mm} \]

10. Check if the shear reinforcement meets the minimum requirement:

\[ A_{v_{min}} = \frac{0.0316 \sqrt{f'_c b_v s}}{f_y} = 119.1 \text{ mm}^2 \]

11. Check if the longitudinal tendons have exceeded the capacity:

\[ F_{lt} = \left| \frac{M_u}{d_v \phi_f} \right| + 0.5N_f + \left( \left| \frac{V_u}{\phi_v} - V_p \right| - 0.5V_s \right) \cot \theta = 2925 \text{ kN} \]

\[ F_p = f_{ps} A_{ps} = 6115 \text{ kN} \]
Table 5.7 Summary of Shear Design Calculation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1.15</th>
<th>2.5</th>
<th>5</th>
<th>7.5</th>
<th>10</th>
<th>12.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_t$, kN</td>
<td>1113.25</td>
<td>1024.39</td>
<td>828.96</td>
<td>638.01</td>
<td>444.82</td>
<td>251.02</td>
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<td>$M_t$, kNm</td>
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<td>2488.34</td>
<td>4388.22</td>
<td>5674.40</td>
<td>6475.68</td>
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<tr>
<td>$V_p$, kN</td>
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<td>41.83</td>
<td>41.83</td>
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<td>$\varepsilon_x$</td>
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<td>0.00034</td>
<td>0.00172</td>
<td>0.00253</td>
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<td>$\theta$</td>
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<td>29</td>
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<td>37.86</td>
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</tr>
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<td>$\beta$</td>
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<td>4.8</td>
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<tr>
<td>$u_u$, MPa</td>
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<td>3.57</td>
<td>2.71</td>
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<tr>
<td>$V_c$, kN</td>
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<td>657.46</td>
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<td>287.12</td>
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<td>224.45</td>
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<td>309.07</td>
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<tr>
<td>$s_{\text{req}}$, mm</td>
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<td>600.94</td>
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<td>0</td>
<td>0</td>
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<tr>
<td>$s_{\text{max}}$, mm</td>
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<td>600</td>
<td>600</td>
<td>600</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>$s_{\text{design}}$, mm</td>
<td>400</td>
<td>500</td>
<td>600</td>
<td>600</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>$V_{s\text{ design}}$, kN</td>
<td>373.44</td>
<td>298.75</td>
<td>237.21</td>
<td>196.94</td>
<td>177.52</td>
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</tr>
<tr>
<td>$A_{v_{\text{min}}}$, mm$^2$</td>
<td>119.10</td>
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<td>178.66</td>
<td>178.66</td>
<td>178.66</td>
<td>178.66</td>
</tr>
<tr>
<td>$F_{\text{lt}}$, kN</td>
<td>2925.13</td>
<td>4112.63</td>
<td>5547.31</td>
<td>6293.97</td>
<td>6778.32</td>
<td>6746.49</td>
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<tr>
<td>$F_{\text{p}}$, kN</td>
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<td>6115.2</td>
<td>6115.2</td>
<td>6115.2</td>
<td>6115.2</td>
<td>6115.2</td>
</tr>
</tbody>
</table>

Table 5.8 Summary of Stirrup Design

<table>
<thead>
<tr>
<th>Distance</th>
<th>Spacing</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.15 - 2.75</td>
<td>400 mm</td>
<td>4 # 10M</td>
</tr>
<tr>
<td>2.75 - 5.25</td>
<td>500 mm</td>
<td>5 # 10M</td>
</tr>
<tr>
<td>5.25 - 12.5</td>
<td>600 mm</td>
<td>12 # 10M</td>
</tr>
</tbody>
</table>

5.4.8 Design for Shrinkage and Temperature Reinforcement

AASHTO LRFD-14 requires: “Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not exceed 1.5 times the thickness of the member or 18.0 in. The maximum spacing of spirals, ties, and temperature shrinkage reinforcement shall be as specified”

CSA S6-14 requires the design to have reinforcement provided normal to the principal reinforcement for the purpose of shrinkage and temperature control. The minimum requirement for these shrinkage and temperature reinforcement is 500 mm$^2$/m, with rebar spacing no more than 300 mm.
• For reinforcements parallel to the span:

\[ A_{sreq} = \frac{500 \, mm^2}{m} \times 1.372 \, m = 686 \, mm^2 \]

Provide 7 #10M bars with \( A_s = 700 \, mm^2 \).

\[ s = \frac{1372 \, mm}{7} = 196 \, mm \to use \ 200 \, mm \]

• For reinforcements perpendicular to the span:

\[ A_{sreq} = \frac{500 \, mm^2}{m} \times 1 \, m = 50 \, mm^2 \]

Provide 5 #10M bars with \( A_s = 500 \, mm^2 \).

\[ s = \frac{1000 \, mm}{5} = 200 \, mm \to use \ 200 \, mm \]

5.5 CSA S6-66 [4]
In this section, the interior girder will be designed using CSA S6-66. In particular, the tendon profile as well as the transverse reinforcement will be determined so that the flexural capacity, shear capacity and deflection will satisfy CSA S6-66.

5.5.1 Prestressing Design
Assuming that the centroid of the tendons is located 100 mm above the bottom face of the girder, which means:

\[ e_g = 628 - 100 = 528 \, mm \]

At ULS, the maximum tensile stress allowed at the bottom girder is
The stress in the bottom of the girder is:

\[ f_{bg} = -\frac{P_f}{A_g} - \frac{P_f e_d}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}} + \frac{M_{da} + M_l}{S_{bc}} \]

\[ 1.76 \text{ MPa} \geq -\frac{P_f}{509 \times 10^3 \text{ mm}^2} - \frac{P_f \times 528 \text{ mm}}{172.8 \times 10^6 \text{ mm}^3} + \frac{974.3 \text{ kNm} + 1012.5 \text{ kNm}}{172.8 \times 10^6 \text{ mm}^3} \]

\[ + \frac{377.7 \text{ kNm} + 1643 \text{ kNm}}{258.4 \times 10^6 \text{ mm}^3} \]

Thus, \( P_f \) is calculated to be at least 3497 kN.

Assuming that the low-relaxation strand has strength of 1080 MPa \( (f_{pf}) \) considering all the long-term losses.

Therefore: \( A_{ps} \geq \frac{3497 \text{ kN}}{1080 \text{ MPa}} = 3238 \text{ mm}^2 \)

Using 24 size 15 strand, which provides an area of 3360 mm².

**5.5.2 Choose the Tendon Profile**

In this case, 10 of 24 strands will be draped to the third-points of the span.
5.5.3 Check the Concrete Stresses at Service Load

5.5.3.1 Initial Condition at Transfer

Assume the low-relaxation strands has an initial strength of 1290 MPa \( (f_{p,bed}) \).
Thus, \( P_i = 1290 \text{ MPa} \times 3360 \text{ mm}^2 = 4334.4 \text{ kN} \)

\[
f_t = -\frac{P_i}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ds}}{S_{tg}}
\]

\[
f_b = -\frac{P_i}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}}
\]

5.5.3.2 Conditions at Time of Placing Wet Concrete on Girder

At this stage, the concrete in the girder has reached its normal capacity of \( f'c \), and prestressing force will be taken as \( P_f \) to be conservative.

\[
f_t = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ds}}{S_{tg}}
\]

\[
f_b = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}}
\]

5.5.3.3 Final Conditions

\[
f_{tg} = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{tg}} + \frac{M_{dg} + M_{ds}}{S_{tg}} + \frac{M_{da} + M_l}{S_{tc}}
\]

\[
f_{bg} = -\frac{P_f}{A_g} - \frac{P_f e_g}{S_{bg}} + \frac{M_{dg} + M_{ds}}{S_{bg}} + \frac{M_{da} + M_l}{S_{bc}}
\]
Table 5.9 Summary of Flexural Design Calculation

<table>
<thead>
<tr>
<th>Distance from Support</th>
<th>0.1L</th>
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<th>0.3L</th>
<th>0.4L</th>
<th>0.5L</th>
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</thead>
<tbody>
<tr>
<td>dv</td>
<td>1150</td>
<td>2500</td>
<td>5000</td>
<td>7500</td>
<td>10000</td>
</tr>
</tbody>
</table>

a) Initial Conditions at Transfer
- $f_{su} = 1860 \text{ MPa} \times \left( 1 - 0.5 \times 0.001 \times \frac{1860 \text{ MPa}}{50 \text{ MPa}} \right) = 1665.1 \text{ MPa}$
- $A_{sf} = 0.85f'_{c}(b - b') \frac{t}{f_{su}} = 0.85 \times 50 \text{ MPa} \times \frac{203 \text{ mm}}{1665.1 \text{ MPa}} = 1580.3 \text{ mm}^2$
- $A_{sr} = A_s - A_{sf} = 3640 - 1580.3 = 2059.7 \text{ mm}^2$

The design flexural strength is determined as:

$$M_u = A_{sr}d f_{su} \left( 1 - 0.6 \frac{A_{sr}f_{su}}{b'df''_c} \right) = 7989.7 \text{ kNm}$$

$$M_f = 7656 \text{ kNm}$$
5.5.5 Check the Deflections

As shown in the previous calculation, the girder will remain uncracked under service load. Therefore, an elastic and uncracked response is assumed in the deflection calculation. Both short-term and long-term deflection will be determined.

\[ E_c = 33w^{1.5} \sqrt{f'_c} = 33830.5 \text{ MPa} \]

a) Immediate deflection due to short-term live loading,

Total live load is determined as:

\[ P = DFV \times Truck \ Load \times IM = 0.805 \times 320 \text{ kN} \times 1.224 = 315.3 \text{ kN} \]

Thus,

\[ \Delta_L = \frac{PL^3}{48EI} = \frac{315.3 \text{ kN} \times (25 \text{ m})^3}{48 \times 28000 \text{ MPa} \times 0.2463 \text{ m}^4} = 12.3 \text{ mm} \]

\[ \Delta_{LT} = 2 \times \Delta_L = 24.6 \text{ mm} \]

Which is less than the Limit \( \frac{l}{600} = 41.7 \text{ mm} \).

5.5.6 Design for Shear Reinforcement

Assuming that the #10M rebar is used with \( A_v = 200 \text{ mm}^2 \), a sample calculation is provided at the location of \( d_v \):

1. Determine \( d_v \) as the first critical location that needs to be designed for:

\[ d_v = \max(0.9d, 0.72h) = 1150 \text{ mm} \]
2. Determine the concrete stress limit $v_c$:

$$v_c = 4\sqrt{f'_c} = 4 \times \sqrt{7252} = 340.6 \text{ psi} = 2.35 \text{ MPa}$$

3. Determine the shear resistance contributed by concrete component $V_c$:

$$V_c = v_c b_v h = 2.35 \text{ MPa} \times 203 \text{ mm} \times 1372 \text{ mm} = 654.1 \text{ kN}$$

4. Determine the shear resistance that needs to be provided by shear reinforcement $V_s$

$$V_s = V_f - V_c = 1187.6 \text{ kN} - 654.1 \text{ kN} = 533.5 \text{ kN}$$

5. Determine the required spacing $s_{req}$:

$$s_{req} = \frac{d_f A_v}{V_s} = \frac{1272 \text{ mm} \times 400 \text{ MPa} \times 200 \text{ mm}^2}{533.5 \text{ kN}} = 190.8 \text{ mm}$$

6. Check the maximum spacing $s_{max}$:

$$s_{max} = 0.5 d = 0.5 \times 1272 \text{ mm} = 636 \text{ mm}$$

7. Check if the shear reinforcement meets the minimum requirement:

$$A_{v_{min}} = \frac{A_{sf}}{80} \frac{s}{f_y} \sqrt{\frac{1}{b'd}} = 69.6 \text{ mm}$$
5.5.7 Design for Shrinkage and Temperature Reinforcement

- For reinforcements parallel to the span:

\[ A_{s\text{req}} = 0.2 \text{in}^2/ft \times 1.372 \text{ m} = 580 \text{ mm}^2 \]

Provide 6 #10M bars with \( A_s = 600 \text{ mm}^2 \).

- For reinforcements perpendicular to the span:

\[ A_{s\text{req}} = \frac{500 \text{ mm}^2}{m} \times 1 \text{ m} = 50 \text{ mm}^2 \]

Provide 5 #10M bars with \( A_s = 500 \text{ mm}^2 \) in every meter.
5.6 Summary

The table below summarizes the prestressing and reinforcing steel design for the prestressed concrete I girder as well as the designed long-term midspan deflection.

Table 5.12 Summary of Prestressed I Girder

<table>
<thead>
<tr>
<th>Summary</th>
<th>CSA S6-14</th>
<th>ASSHTO LRFD-14</th>
<th>CSA S6-66</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressing Tendons</td>
<td>24 x 15 mm Strands: 10 Harped; 14 Straight</td>
<td>24 x 15 mm Strands: 10 Harped; 14 Straight</td>
<td>24 x 15 mm Strands: 10 Harped; 14 Straight</td>
</tr>
<tr>
<td>Shear Reinforcement</td>
<td>Total of 52 # 10M double-legged stirrups</td>
<td>Total of 42 # 10M double-legged stirrups</td>
<td>Total of 52 # 10M double-legged stirrups</td>
</tr>
<tr>
<td>Longitudinal (Shrinkage and Temperature) Reinforcement</td>
<td>Longitudinal: 7 #10M bars Transvers: #10M @ 200 mm</td>
<td>Longitudinal: 14 #10M bars Transvers: #10M @ 500 mm</td>
<td>Longitudinal: 30 #10M bars Transvers: #10M @ 500 mm</td>
</tr>
<tr>
<td>Long-Term Midspan Deflection</td>
<td>4.43 mm</td>
<td>18.26 mm</td>
<td>24.64 mm</td>
</tr>
</tbody>
</table>
Reference:


Chapter 6 Reinforced Concrete Deck Design

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6.1 Introduction

This chapter presents the design and calculation of the bridge’s reinforced concrete deck. Three provisions are used to develop the required deck design, CSA S6-14, ASSHO LRFD-12 and CSA S6-66. The difference and similarity of each of the three designs are summarized and compared at the end of this chapter.

6.2 CSA S6-14 [1]

This section summaries the bridge reinforced concrete deck design based on the provisions of the Canadian Highway Bridge Design Code CSA S6-14. The dead and the live loads are determined and used to calculate the factored design loads. Based on the calculated design loads, the size and spacing of the transverse reinforcement is selected with the consideration of the maximum and minimum spacing requirements and crack control. In addition, top and bottom longitudinal reinforcement layout are also designed based on the code requirement.

6.2.1 Design Input

Deck slab thickness: 200 mm
Girder Spacing: 2.7 m
Top cover: 50 mm (§8.11.2.2, Table 8.5 exposure to de-icing chemicals)
Bottom cover: 30 mm (§8.11.2.2, Table 8.5 no exposure de-icing chemicals)
Concrete compressive strength: 40 Mpa
Concrete unit weight: 23.5 kN/m^3
Wearing surface thickness: 50 mm
Wearing surface unit weight 23.5 kN/m^3
Determine $\alpha_1$ and $\beta_1$

$\alpha_1 = 0.85 - 0.0015 f'c = 0.79$
$\beta_1 = 0.97 - 0.0025 f'c = 0.87$
6.2.2 Design Loads

6.2.2.1 Dead Loads

Unfactored moments due to the dead load of the concrete slab deck and wearing surface per unit width can be approximated using the continuous beam equations:

\[
M_D^- = \frac{wl^2}{11}
\]

\[
M_D^+ = \frac{wl^2}{16}
\]

Deck Load:

\[
w_D = 0.200 \, m \times 1.0 \, m \times 23.5 \, kN/m^3 = \frac{4.70 \, kN}{m}
\]

\[
M_D^- = \frac{wl^2}{11} = \frac{4.70 \times 2.7^2}{11} = 3.115
\]

\[
M_D^+ = \frac{wl^2}{16} = \frac{4.70 \times 2.7^2}{16} = 2.141
\]

Wearing Surface:

\[
w_{ws} = 0.05 \, m \times 1.0 \, m \times 23.5 \, kN/m^3 = \frac{1.175 \, kN}{m}
\]

\[
M_D^- = \frac{wl^2}{11} = \frac{1.175 \times 2.7^2}{11} = 0.779
\]

\[
M_D^+ = \frac{wl^2}{16} = \frac{1.175 \times 2.7^2}{16} = 0.535
\]

6.2.2.2 Live Loads

Concrete deck slabs that are supported by longitudinal girders may be analyzed for transverse bending using the simplified elastic method in which the maximum transverse moment intensity is given by §5.7.1.2.

From Figure 6.1, for the proposed bridge girder spacing:

\[
S_e = 2.700 - 0.924 = 1.776 \, m
\]
Dynamic load allowance (DLA) for single axle = 0.4 (§3.8.4.5.3)

Transverse live load moment intensity:

\[ M_{TL} = (S_e + 0.6) \frac{P}{10} DLA = \frac{(1.776 + 0.6) \times 87.5}{10} \times 1.4 = 29.106 \text{ kNm/m} \]

For slabs supported over more than 3 supports the transverse live moment intensity may be reduced to 80% of the max bending moment for that determined for a simple span.

\[ M_{TL} = 0.8 \times 29.106 \times 23.2848 \text{ kNm/m} \]

Longitudinal live load moment intensity:

\[ M_{LL} = \frac{120}{S_e^{0.5}} M_{TL} = \frac{120}{1.776^{0.5}} M_{TL} = 90.05\% M_{TL} \geq 67\% M_{TL} \]

\[ M_{LL} = 67\% M_{TL} = 0.67 \times 23.2848 = 15.60 \text{ kNm/m} \]

![Figure 6.1 Graphic Illustration of Se](Image)

6.2.3 Factored Design Moments

ULS combination 1:

\[ \alpha_D (DD) + \alpha_D (DW) + 1.7(LL) \]

Where \( \alpha_D (DD) = 1.2, \alpha_D (DW) = 1.5 \)

The ULS combination is therefore: \( 1.2 (DD) + 1.5 (DW) + 1.7(LL) \)

Negative Transverse Factored Moment:

\[ M_{support}^- = 1.2 \times 3.115 + 1.5 \times 0.779 + 1.7 \times 23.2848 = 44.49 \text{ kNm/m} \]

Positive Transverse Factored Moment:
Positive Longitudinal Factored Moment:

\[ M_{\text{support}}^+ = 1.2 \times 2.141 + 1.5 \times 0.535 + 1.7 \times 23.2848 = 42.96 \text{ kNm/m} \]

6.2.4 Negative Transverse Moment Flexure Design

Assuming 15M-bar

Bar area = 200 mm\(^2\)
Bar diameter = 16 mm

Effective depth:

\[ d = 200 - \frac{16}{2} - 50 = 142 \text{ mm} \]
\[ K_r = \frac{M_f}{bd^2} = \frac{44.49 \times 10^6}{1000 \times 142^2} = 2.206 \]
\[ A_s\text{required} = \frac{\alpha_1 \phi_c f'_c(b)}{\phi_{sf} f_y} \times (d - \sqrt{d^2 - \frac{2M_r}{\alpha_1 \phi_c f'_c(b)}}) \]
\[ A_s\text{required} = 1038.224 \text{ mm}^2 \]

Assuming 6x15M bars with \(A_s\text{ design} = 1200 \text{ mm}^2\), the required spacing:

\[ s = \frac{1000}{A_s\text{ design}/Ab} = \frac{1000}{1200/200} = 167 \text{ mm/m} \]

To satisfy crack control use 15M @ 150mm.

\[ \rho_{\text{provided}} = \frac{1333.3}{1000 \times 142} = 0.94\% \geq 0.3\% \]

Crack Control:

\[ Z = f_s (d_c A)^{\frac{1}{3}} \]

Where \( d_c = \text{cover} + \frac{db}{2} = 50 + 8 = 58 \text{ mm} \]
6.2.5 Positive Transverse Moment Flexure Design

Assuming 15M-bar
Bar area = 200 mm²
Bar diameter = 16 mm

Effective depth:

\[ d = 200 - \frac{16}{2} - 30 = 162 \text{ mm} \]

\[ K_r = \frac{M_f}{bd^2} = \frac{42.96 \times 10^6}{1000 \times 142^2} = 2.1305 \]

\[ A_{s\text{required}} = \frac{\alpha_1 \phi_c f'_c(b)}{\phi_s f_y} \times \left( d - \frac{d^2 - \frac{2M_r}{\alpha_1 \phi_c f'_c(b)}}{\sqrt{d^2 - \frac{2M_r}{\alpha_1 \phi_c f'_c(b)}}} \right) \]

\[ A_{s\text{required}} = 864.6528 \text{ mm}^2 \]

Assuming 5x15M bars with \( A_{\text{design}} = 1000 \text{ mm}^2 \), the required spacing:

\[ s = \frac{1000}{A_{\text{design}}/Ab} = \frac{1000}{1000/200} = 200 \text{ mm/m} \]

Crack Control:

\[ Z = f_s (d_c A)^{\frac{1}{3}} \]

Where \( d_c = \text{cover} + \frac{d_b}{2} = 30 + 8 = 38 \text{ mm} \)

\[ g = h - d = 200 - 162 = 38 \text{ mm} \]

\[ A = 2gb/n = 2(38)(1000)/(1000/200) = 15200 \text{ mm}^2 \]

\[ F_s = 0.6 f_y = 240 \text{ MPa} \]
\[ Z = f_s(d_c A)^{\frac{1}{3}} = 19987.3 \text{ N/mm} \leq 25000 \text{ N/mm} \]

6.2.6 Bottom Distribution Reinforcement

Amount of bottom slab reinforcement as a percentage of the primary reinforcement (§8.18.7):

\[ \% = \frac{120}{\sqrt{5}} \leq 67\% \]
\[ \% = \frac{120}{\sqrt{2.7}} = 73.0\% \leq 67\% \]

Therefore use 67% of transverse reinforcement

Design transverse reinforcement for positive flexure: 15M at 200 mm spacing with

\[ A_s \text{design} = 1000 \text{ mm}^2/m \]

Design longitudinal reinforcement:

\[ A_s \text{required} = 0.67 A_s \text{transverse} = 1000 \times 0.67 = 670 \text{ mm}^2/m \]
\[ A_s \text{design} = 600 \text{ mm}^2/m \]

Required spacing using 10M bars

\[ s = \frac{100 \text{mm}}{600} \times 1000 = 167 \text{ mm/m} \]

⇒ Use 165 mm/m

Crack Control:

\[ Z = f_s(d_c A)^{\frac{1}{3}} \]

Where \( d_c = 51 \text{ mm} \)
\[ A = 2gb/n = 2(51)(1000)/(1000/165) = 16830 \text{ mm}^2 \]
\[ F_s = 0.6 \text{ fy} = 240 \text{ MPa} \]

\[ Z = f_s(d_c A)^{\frac{1}{3}} = 22808.5 \text{ N/mm} \leq 25000 \text{ N/mm} \]
6.2.7 Top of Slab Shrinkage and Temperature Reinforcement

Minimum amount of temperature and shrinkage reinforcement (§8.12.6):

\[ As = \frac{500 \text{mm}^2}{m} \text{ and } s \leq 300 \text{mm} \]

Use 5x10M bars with \( A_v = 500 \text{ mm}^2 \). Required spacing:

\[ s = \frac{1000 \text{mm}}{500/100} = 200 \text{ mm} \leq 300 \text{mm} \rightarrow \text{ok} \]

6.3 ASSHTO LRFD-14 [2]

This section demonstrates the design of the reinforced concrete deck that sits above the prestressed girders according to AASHTO LRFD 14. The arrangement of flexural reinforcement designed based on the corresponding dead loads and live loads that need to be resisted. For typical deck slabs, shear reinforcement does not have to be investigated according to the code, therefore, it is not discussed in this section.

6.3.1 Design Input

Deck slab thickness: 8 in. (200 mm)
Girder Spacing: 8 ft. -10 in. (2.7m)
Top cover: 2.5 in. (Table 5.12.3-1 exposure to de-icing salts [3]) (64 mm)
Bottom cover: 1.0 in (Table 5.12.3-1) (25 mm)
Concrete compressive strength: 5801.508 psi (40 Mpa)
Concrete unit weight: 0.1458 kcf (22.9 kN/m^3)
Wearing surface thickness: 3 in. (76.2 mm)
Wearing surface unit weight: 0.140 kcf (Table 3.5.1-1) (22.0 kN/m)
6.3.2 Dead Loads

Assume that the reinforced concrete deck has a similar behavior to the continuous beam. Thus, bending moment generated by the self-weight of concrete deck and wearing surface of a 1 m section are estimated as follow:

Deck Self-weight:

\[ w_D = 0.200 \, m \times 1.0 \, m \times 22.9 \, \frac{kN}{m^3} = 4.58 \, kN/m \]

\[ M_D^- = \frac{wL^2}{11} = \frac{4.58 \times 2.7^2}{11} = 3.035 \, kN \cdot m \]

\[ M_D^+ = \frac{wL^2}{16} = \frac{4.58 \times 2.7^2}{16} = 2.087 \, kN \cdot m \]

Wearing Surface Self-weight:

\[ w_{ws} = 0.05 \, m \times 1.0 \, m \times 22.0 \, \frac{kN}{m^3} = 1.10 \, kN/m \]

\[ M_{Dws}^- = \frac{w_{ws}L^2}{11} = \frac{1.10 \times 2.7^2}{11} = 0.729 \, kN \cdot m \]

\[ M_{Dws}^+ = \frac{w_{ws}L^2}{16} = \frac{1.10 \times 2.7^2}{16} = 0.501 \, kN \cdot m \]

6.3.3 Live Loads

The live load is calculated based on the equivalent strip method conditions, which relies on a few assumptions and limitations. In this case, the main bars are aligned transverse to the traffic, and the span is 106 in. Thus, the moment caused by live load with dynamic loads included is calculated as:

\[ M_L^+ = 6.19 \, \text{kip} \cdot \text{ft/ft} = 27.53 \, \text{kN} \cdot \text{m/m} \]

\[ M_L^- = 6.76 \, \text{kip} \cdot \text{ft/ft} = 30.07 \, \text{kN} \cdot \text{m/m} \]
6.3.4 Factored Design Moment

Design for Strength I condition:

**Strength I:** \( \gamma_p(DC) + \gamma_p(DW) + 1.75(LL + IM) \)

- \( \gamma_p \) for \( DC \) = 1.25
- \( \gamma_p \) for \( DW \) = 1.50

Negative moment at the support:

\[
M_{support}^- = 1.25 \times 3.035 + 1.50 \times 0.729 + 1.75 \times 30.07 = 57.51 \text{ kNm/m}
\]

Positive moment at the midspan:

\[
M_{midspan}^+ = 1.25 \times 2.087 + 1.50 \times 0.501 + 1.75 \times 27.53 = 51.54 \text{ kNm/m}
\]

6.3.5 Flexural Design at Midspan

Assume using #5-bar (15M bar)

Bar area = 0.31 in\(^2\) = 200 mm\(^2\)

Bar diameter = 0.625 in. = 16 mm

**Effective depth:**

\[
d = 200 - \frac{16}{2} - 25 = 167 \text{ mm}
\]

\[
k' = \frac{M_f}{\phi b d^2} = \frac{51.54 \times 10^6}{0.9 \times 1000 \times 167^2} = 2.053
\]

\[
\rho = 0.85 \left( \frac{f_c'}{f_y} \right) \left( 1 - \frac{2k'}{0.85f_c'} \right) = 0.0053
\]

**As\text{\_required} = \rho \times b \times d = 0.0053 \times 1000 \times 167 = 884.7 \text{ mm}^2**

Assuming 5x #5 bars with \textit{As\_design} = 1000 mm\(^2\), the required spacing:

\[
s = \frac{1000}{As\_design/Ab} = \frac{1000}{1000/200} = 200 \text{ mm/m}
\]

Therefore use 15M @ 200mm.

**Crack Control:**

\[
s \leq \frac{700 \gamma_e}{\beta s f_{ss}} - 2d_c
\]
\[ \beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{1.3}{0.7(7.87 - 1.3)} = 1.283 \]

Where \( d_c = \text{cover} + \frac{d_b}{2} = 1 + 0.3 = 1.3 \text{ in.} = 33 \text{ mm} \)

\[ h = 7.87 \text{ in.} = 200 \text{ mm} \]

\[ \gamma_e = 0.75 \text{ for class 2 condition} \]

\[ f_{ss} = 0.6 \text{ fy} = 34.8 \text{ ksi} = 240 \text{ MPa} \]

\[ s \leq \frac{700 \times 0.75}{1.283 \times 34.8} - 2 \times 1.3 = 9.16 \text{ in.} = 232.6 \text{ mm} \]

### 6.3.6 Flexural Design at Support

Assume using #5-bar (15M bar)

Bar area = 0.31 in\(^2\) = 200 mm\(^2\)

Bar diameter = 0.625 in. = 16 mm

\[ d = 200 - \frac{16}{2} - 64 = 128 \text{ mm} \]

\[ k' = \frac{M_f}{\phi b d^2} = \frac{57.51 \times 10^6}{0.9 \times 1000 \times 128^2} = 3.900 \]

\[ \rho = 0.85 \left( \frac{f_{c'}'}{f_y} \right) \left( 1 - \sqrt{1 - \frac{2k'}{0.85f_{c'}'}} \right) = 0.01038 \]

\[ A_{\text{required}} = \rho \times b \times d = 0.01038 \times 1000 \times 128 = 1329.2 \text{ mm}^2 \]

Assuming 7x #5 bars with \( A_{\text{design}} = 1400 \text{ mm}^2\), the required spacing:

\[ s = \frac{1000}{A_{\text{design}}/Ab} = \frac{1000}{1400/200} = 143 \text{ mm/m} \]

Therefore, use 15M @ 140 mm.
6.3.7 Bottom Reinforcement Perpendicular Direction

Amount of bottom slab longitudinal reinforcement as a percentage of the primary reinforcement is calculated (§9.7.3.2):

\[
\frac{120}{\sqrt{5}} = \frac{120}{\sqrt{8.83}} = 40.4\% \leq 67\%
\]

Therefore use 40.4% of transverse reinforcement. Transverse reinforcement for positive flexure is designed as: 15M at 200 mm spacing with:

\[A_{s\,\text{design}} = 1000 \text{ mm}^2/\text{m}\]

Design longitudinal reinforcement:

\[A_{s\,\text{required}} = 0.404 \times A_{s\,\text{transverse}} = 1000 \times 0.404 = 404 \text{ mm}^2/\text{m}\]

Assume using 10M bars

\[s = \frac{1000 \text{ mm}}{404/100} = 247.5 \text{ mm/m}\]

Therefore use 10M @ 240 mm

\[A_{s\,\text{design}} = 417 \text{ mm}^2/\text{m}\]

6.3.8 Shrinkage and Temperature Reinforcement

Additional reinforcement need to be applied at top of the deck in longitudinal directions, which accounts for creep, shrinkage and temperature change. According to §5.10.8:

\[A_s \leq \frac{1.30bh}{2(b+h)f_y} = \frac{1.30 \times 39.37 \text{ in} \times 7.87 \text{ in.}}{2 \times (39.37 \text{ in} + 7.87 \text{ in}) \times 58 \text{ ksi}} = 0.0735 \text{ in}^2/\text{ft}\]

\[0.11 \text{ in}^2/\text{ft} \leq A_s \leq 0.60 \text{ in}^2/\text{ft}\]

Therefore use \(A_s = 0.11 \text{ in}^2/\text{ft} = 230 \text{ mm}^2/\text{m}\)
Use 3x10M bars with $A_v = 300 \text{ mm}^2$. Required spacing:

$$s = \frac{1000\text{mm}}{300/100} = 333 \text{ mm/m}$$

Therefore use 10M @ 330 mm.

6.4 CSA S6-66 [3]

This section outlines the bridge reinforced concrete deck design based on the provisions of the Canadian Design of highway Bridges CSA S6-66 [2]. Similar to the previous two sections, the dead and the live loads are determined and used to calculate the factored design loads. Based on the calculated design loads, the size and spacing of the transverse reinforcement is selected with the consideration of the maximum and minimum spacing requirements and crack control. In addition, top and bottom longitudinal reinforcement layout are also designed based on the code requirement.

6.4.1 Design Input

Deck slab thickness: 200 mm
Girder Spacing: 2.7 m
Top cover: 50 mm (§8.6.2 [2])
Bottom cover: 25 mm (§8.6.2 [2])
Concrete compressive strength: 40 Mpa
Concrete unit weight: 23.5 kN/m^3
Wearing surface thickness: 50 mm
Wearing surface unit weight 23.5 kN/m^3

6.4.2 Dead Load Effects

Unfactored moments due to the dead load of the concrete slab deck and wearing surface per unit width can be approximated using the continuous beam equations:

$$M_D = \frac{wL^2}{11}$$
Displacement:

\[ M_D = \frac{wL^2}{16} \]

Deck Load:

\[ w_D = 0.200 \, m \times 1.0 \, m \times \frac{kN}{m^3} = \frac{4.70 \, kN}{m} \]

\[ M_D^- = \frac{wL^2}{11} = \frac{4.70 \times 2.7^2}{11} = 3.115 \]

\[ M_D^+ = \frac{wL^2}{16} = \frac{wL^2}{16} = \frac{4.70 \times 2.7^2}{16} = 2.141 \]

Wearing Surface:

\[ w_{ws} = 0.05 \, m \times 1.0 \, m \times \frac{kN}{m^3} = \frac{1.175 \, kN}{m} \]

\[ M_D^- = \frac{wL^2}{11} = \frac{1.175 \times 2.7^2}{11} = 0.779 \]

\[ M_D^+ = \frac{wL^2}{16} = \frac{wL^2}{16} = \frac{1.175 \times 2.7^2}{16} = 0.535 \]

6.4.3 Live Loads

Concrete deck slabs that are supported by longitudinal girders may be analyzed for transverse bending in which the maximum transverse moment intensity is given by §5.2.2.3 Case A – Main Reinforcement Perpendicular to Traffic (Spans 2 to 24 feet, inclusive)

Transverse live load moment intensity:

\[ M_{TL} = \frac{(S + 2)}{32} \cdot P = \frac{(6.5 + 2)}{32} \cdot 16000 = 18.89 \, kN/m \]

where S = Effective span length, in feet, as defined under “Span Lengths” (§ 5.2.2.1)
E = Width of slab in feet over which a wheel is distributed
P = Load on one rear wheel of truck = 16,000 pounds for H20 or H20-S16 loads

From § 5.2.2.1, for simple spans the span length S is taken as the minimum of the distance centre-to-centre of supports and the clear span plus thickness of slab.
Therefore,
\[ S = 1.976 \, \text{m} = 6.5 \, \text{ft} \]

For slabs supported over three or more supports the transverse live moment intensity shall be reduced to 80% of the max bending moment for both positive and negative moment.

\[ M_{TL} = 0.8 \times 18.89 = 15.11 \, \text{kNm/m} \]

From § 5.1.11.1, the impact formula to determine the impact factor is as follows:

\[ I = \frac{50}{L + 125} = \frac{50}{8.86 + 125} 0.374 \geq 0.3 \rightarrow I = 0.3 \]

\[ M_I = 0.3 \times 15.11 = 4.53 \, \text{kNm/m} \]

**6.4.4 Factored Design Moments**

The load combination is expressed as the following for Group 1 condition:

**Group 1**: \( D + L + I \)

Negative Transverse Factored Moment:

\[ M^-_{support} = 3.115 + 0.779 + 15.11 + 4.53 = 23.53 \, \text{kNm/m} \]

Positive Transverse Factored Moment:

\[ M^+_{support} = 2.141 + 0.535 + 15.11 + 4.53 = 22.32 \, \text{kNm/m} \]

**6.4.5 Negative Transverse Moment Flexure Design**

Assuming 15M-bar

Bar area = 200 mm\(^2\)

Bar diameter = 16 mm

Effective depth:

\[ d = 200 - 5 - 50 = 145 \, \text{mm} \]

\[ K_r = \frac{M_f}{bd^2} = \frac{23.53 \times 10^6}{1000 \times 145^2} = 1.119 \]
Assuming 3x15M bars with $As_{design} = 600 \text{ mm}^2$, the required spacing:

$$s = \frac{1000}{As_{design}/Ab} = \frac{1000}{600/200} = 334 \text{ mm/m}$$

Crack Control:

$$Z = f_s(d_cA)^{\frac{1}{3}}$$

Where $d_c = \text{cover} + \frac{d_b}{2} = 50 + 8 = 58$ mm

\[A = 2gb/n = 2(58)(1000)/(1000/334) = 38744 \text{ mm}^2\]

\[Fs = 0.6 f_y = 240 \text{ MPa}\]

\[Z = f_s(d_c A)^{\frac{1}{3}} = 31436 N/mm \geq 25000 N/mm\]

Therefore, $s$ is adjusted to 167 mm/m to satisfy crack control. The corresponding $Z$ is presented as follows:

$$Z = f_s(d_c A)^{\frac{1}{3}} = 24850 N/mm \leq 25000 N/mm$$

### 6.4.6 Positive Transverse Moment Flexure Design

Assuming 15M-bar

Bar area = 200 mm$^2$

Bar diameter = 16 mm

Effective depth:

$$d = 200 - 5 - 25 = 170 \text{ mm}$$

$$K_r = \frac{M_f}{bd^2} = \frac{22.32 \times 10^6}{1000 \times 170^2} = 0.772$$

$$As_{required} = \frac{\alpha_1 \phi f_c'(b)}{\phi sfy} \times (d - \sqrt{d^2 - \frac{2M_r}{\alpha_1 \phi f_c'(b)}})$$

$$As_{required} = 419 \text{ mm}^2$$
Assuming 3x15M bars with $As\ design = 600\ mm^2$, the required spacing:

$$s = \frac{1000}{As\ design/Ab} = \frac{1000}{600/200} = 333.3\ mm/m$$

Crack Control:

$$Z = f_s(d_cA)^{\frac{1}{3}}$$

Where

$$d_c = \text{cover} + \frac{db}{2} = 30 + 8 = 38\ mm$$

$$g = h - d = 200 - 162 = 38\ mm$$

$$A = 2gb/n = 2(38)(1000)/(600/200) = 25333\ mm^2$$

$$Fs = 0.6\ fy = 240\ Mpa$$

$$Z = f_s(d_cA)^{\frac{1}{3}} = 23698\ \frac{N}{mm} \leq 25000\ \frac{N}{mm} \rightarrow ok$$

**6.4.7 Bottom Distribution Reinforcement**

From § 5.2.2.5, Amount of bottom slab reinforcement as a percentage of the primary reinforcement:

$$\% = \frac{220}{\sqrt{S}} \leq 67\%$$

$$\% = \frac{220}{\sqrt{S}} = 73.9\% \rightarrow \% = 67\%$$

where $S = \text{the effective span length in feet} = 8.86$ feet

Therefore use 67% of transverse reinforcement

Design transverse reinforcement for positive flexure: 15M at 333 mm spacing with

$$As\ design = 600\ mm^2/m$$

Design longitudinal reinforcement:

$$As\ required = 0.67As\ transverse = 600 \times 0.67 = 402\ mm^2/m$$

$$As\ design = 500\ mm^2/m$$
Required spacing using 10M bars

\[ s = \frac{100\text{mm}}{500} \times 1000 = 200 \text{ mm/m} \]

### 6.4.8 Top of Slab Shrinkage and Temperature Reinforcement

Minimum amount of temperature and shrinkage reinforcement is limited by minimum area

\[ A_s = \frac{400 \text{mm}^2}{m} \text{ and } s \leq 300 \text{mm} \]

Use 4x10M bars with \( A_v = 400 \text{ mm}^2 \). Required spacing:

\[ s = \frac{1000\text{mm}}{400/100} = 250 \frac{\text{mm}}{m} \]

→ Use 10M bars @ 250mm
6.5 Conclusion

In summary, Table 6.1 summarizes the reinforcement design for the bridge slab. The most conservative transverse reinforcement design was developed based on the AASHTO LRFD-14 design standard, while the S6-66 design was the least conservative due to the use of smaller bars and larger spacing in some cases. For the longitudinal reinforcement, the S6-66 provision results in the most conservative design while the AASHTO LRFD-14 yields the least conservative one.

Table 6.1 Summary of Slab Reinforcement Design

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>CSA S6-14</th>
<th>AASHTO LRFD-14</th>
<th>CSA S6-66</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Transverse:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive moment reinforcement</td>
<td>15M@200 mm</td>
<td>15M@200 mm</td>
<td>15M@330 mm</td>
</tr>
<tr>
<td>Negative moment reinforcement</td>
<td>15M@150 mm</td>
<td>15M@140 mm</td>
<td>15M@165 mm</td>
</tr>
<tr>
<td><strong>Longitudinal:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom distribution reinforcement</td>
<td>10M@165 mm</td>
<td>10M@240 mm</td>
<td>10M@150 mm</td>
</tr>
<tr>
<td>Top shrinkage and temperature reinforcement</td>
<td>10M@200 mm</td>
<td>10M@330 mm</td>
<td>10M@250 mm</td>
</tr>
</tbody>
</table>
Reference:


# Chapter 7 Durability Design

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7.1 Introduction

Material's durability significantly influences the performance of the structure. In order to achieve the strength requirement in the structural design part, concrete mix must be appropriately designed as different mix will result in different chemical and physical properties. In this chapter, concrete mix design will be specified. The general design method is based on Designing and Proportioning Normal Concrete Mixtures in Design and Control of Concrete Mixtures.

7.2 Concrete Exposure Condition

According to Table 1 in CSA A23.1-14, the bridge deck is classified in C-1 and the girder is classified in A-1 [1].

The most common size of aggregate is 25 mm in CSA A23.1-14. Therefore, we will use it as our nominal aggregate size to determine the air content. Entrained air must be used in all concrete that will be exposed to freezing and thawing and deicing chemicals and can be used to improve workability even where not required. The detailed requirement for this class of concrete is listed below.

Table 7.1. Detailed Requirement for the Specific Class of Concrete [2]

<table>
<thead>
<tr>
<th>Class</th>
<th>Maximum Water-Cement Ratio</th>
<th>Minimum Specified Concrete Strength (MPa)</th>
<th>Air Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-1 (Deck)</td>
<td>0.4</td>
<td>35</td>
<td>4-7%</td>
</tr>
<tr>
<td>A-1 (Girder)</td>
<td>0.4</td>
<td>35</td>
<td>4-7%</td>
</tr>
</tbody>
</table>

7.3 Strength

In our design, the compressive strength (f'c) for the bridge deck is 40 MPa, and the compressive strength for the girder is 50 MPa. According to ACI 318-08, the required average compressive strength is generally higher than the design value. If the standard deviation of the concrete is not available, the following table should be used to determine
the average compressive strength [3]. Therefore, in our design, the required average compressive strength for the bridge deck is 49 MPa, and for the girder is 60 MPa.

Table 7.2. Required Average Compressive Strength

<table>
<thead>
<tr>
<th>Specified Compressive Strength (MPa)</th>
<th>Required Average Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;21</td>
<td>f’c+7.0</td>
</tr>
<tr>
<td>21 to 35</td>
<td>f’c + 8.5</td>
</tr>
<tr>
<td>&gt;35</td>
<td>1.1 f’c + 5.0</td>
</tr>
</tbody>
</table>

7.4 Water-Cement Ratio

Compressive strength is inversely related to the water-cement ratio. The tabulated values are summarized in Table 7.3 below.

Table 7.3 Relationship between Compressive Strength and W/C ratio [3]

<table>
<thead>
<tr>
<th>Compressive strength at 28 days, MPa</th>
<th>Water-cementitious materials ratio by mass</th>
<th>Non-air-entrained concrete</th>
<th>Air-entrained concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td></td>
<td>0.38</td>
<td>0.30</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td>0.42</td>
<td>0.34</td>
</tr>
<tr>
<td>35</td>
<td></td>
<td>0.47</td>
<td>0.39</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>0.54</td>
<td>0.45</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>0.61</td>
<td>0.52</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>0.69</td>
<td>0.60</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>0.79</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Strength is based on cylinders moist-cured 28 days in accordance with ASTM C 31 (AASHTO T 23). Relationship assumes nominal maximum size aggregate of about 19 to 25 mm. Adapted from ACI 211.1 and ACI 211.3.

As the concrete is exposed to extreme weather environment, i.e. need to account for freezing and thawing, air entrained concrete will be used.

By extrapolating the date listed above, for the bridge deck (f’cr = 49 MPa), the w/c ratio should be 0.26. For the girder (f’cr = 60 MPa), the w/c ratio should be 0.2.
7.5 Air Content

Entrained air must be used in all concrete that will be exposed to freezing and thawing and deicing chemicals and can be used to improve workability even where not required.

For the bridge deck, as it would be exposed to de-ice environment, this will be classified as Severe Exposure.

For the girders, they will be classified as Moderate Exposure. The Maximum Aggregate Size will be 25 mm (1 in). According to Figure 7.1, the air content for the bridge deck is 6% and the air content for the bridge girder is 4.5%. Both of them is in the range of 4-7% which is required in CSA A23.1-14.

![Figure 7.1 Relationship between Air Content and Nominal Maximum Aggregate Size [3]](image)

7.6 Slump

Concrete must always be made with a workability, consistency, and plasticity suitable for job conditions. Workability of concrete is a measure of how easy or difficult for the
concrete to be placed, consolidated and finished. Consistency the ability of freshly mixed concrete to flow. Plasticity determines concrete’s ease of molding.

The slump test is used to measure concrete consistency. For a given proportion of cement and aggregate without admixtures, the higher the slump, the wetter the mixture. Slump is indicative of workability when assessing similar mixtures. According to Table 7.4, for the bridge deck, the slump should be 75 mm and for the bridge girder, the slump should be 100 mm.

Table 7.4 Recommended Slumps for Various Types of Construction [3]

<table>
<thead>
<tr>
<th>Concrete construction</th>
<th>Slump, mm (in.)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum*</td>
<td>Minimum</td>
</tr>
<tr>
<td>Reinforced foundation walls and footings</td>
<td>75 (3)</td>
<td>25 (1)</td>
</tr>
<tr>
<td>Plain footings, caissons, and substructure walls</td>
<td>75 (3)</td>
<td>25 (1)</td>
</tr>
<tr>
<td>Beams and reinforced walls</td>
<td>100 (4)</td>
<td>25 (1)</td>
</tr>
<tr>
<td>Building columns</td>
<td>100 (4)</td>
<td>25 (1)</td>
</tr>
<tr>
<td>Pavements and slabs</td>
<td>75 (3)</td>
<td>25 (1)</td>
</tr>
<tr>
<td>Mass concrete</td>
<td>75 (3)</td>
<td>25 (1)</td>
</tr>
</tbody>
</table>

*May be increased 25 mm (1 in.) for consolidation by hand methods, such as rodding and spading. Plasticizers can safely provide higher slumps. Adapted from ACI 211.1.

7.7 Water Content

Based on Table 7.5 below, for our case, since 25 mm aggregate will be used, the water content for both deck and girders is 193 kg/m³.

However, a water reducing mixture will be added to the mixture. Therefore, the ultimate required water content will be reduced by 10%, which is 193*0.9 = 174 kg/m³.
Table 7.5 Water Content for Specified Air Content and Different Slumps as well as Nominal Maximum Sizes of Aggregate [3]

<table>
<thead>
<tr>
<th>Slump, mm</th>
<th>9.5 mm</th>
<th>12.5 mm</th>
<th>19 mm</th>
<th>25 mm</th>
<th>37.5 mm</th>
<th>50 mm**</th>
<th>75 mm**</th>
<th>150 mm**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-air-entrained concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25 to 50</td>
<td>207</td>
<td>199</td>
<td>190</td>
<td>179</td>
<td>168</td>
<td>154</td>
<td>130</td>
<td>113</td>
</tr>
<tr>
<td>75 to 100</td>
<td>228</td>
<td>216</td>
<td>205</td>
<td>198</td>
<td>181</td>
<td>169</td>
<td>145</td>
<td>124</td>
</tr>
<tr>
<td>150 to 175</td>
<td>243</td>
<td>228</td>
<td>216</td>
<td>202</td>
<td>190</td>
<td>178</td>
<td>160</td>
<td>—</td>
</tr>
<tr>
<td>Approximate amount of entrapped air in non-air-entrained concrete, percent</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2.5</td>
<td>2</td>
<td>1.5</td>
<td>1</td>
<td>0.5</td>
<td>0.3</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Air-entrained concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25 to 50</td>
<td>181</td>
<td>175</td>
<td>168</td>
<td>160</td>
<td>150</td>
<td>142</td>
<td>122</td>
<td>107</td>
</tr>
<tr>
<td>75 to 100</td>
<td>202</td>
<td>193</td>
<td>184</td>
<td>175</td>
<td>165</td>
<td>157</td>
<td>133</td>
<td>119</td>
</tr>
<tr>
<td>150 to 175</td>
<td>216</td>
<td>205</td>
<td>197</td>
<td>184</td>
<td>174</td>
<td>169</td>
<td>154</td>
<td>—</td>
</tr>
<tr>
<td>Recommended average total air content, percent, for level of exposure:†</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mild exposure</td>
<td>4.5</td>
<td>4.0</td>
<td>3.5</td>
<td>3.0</td>
<td>2.5</td>
<td>2.0</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Moderate exposure</td>
<td>6.0</td>
<td>5.5</td>
<td>5.0</td>
<td>4.5</td>
<td>4.0</td>
<td>3.5</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>Severe exposure</td>
<td>7.5</td>
<td>7.0</td>
<td>6.0</td>
<td>5.0</td>
<td>5.0</td>
<td>4.0</td>
<td>4.0</td>
<td></td>
</tr>
</tbody>
</table>

* These quantities of mixing water are for use in computing cementitious material contents for trial batches. They are maximums for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications.

** The slump values for concrete containing aggregates larger than 37.5 mm are based on slump tests made after removal of particles larger than 37.5 mm by wet screening.

† The air content in job specifications should be specified to be delivered within ±2 percentage points of the table target value for moderate and severe exposures.

Adapted from ACI 211.1 and ACI 318, Hover (1995) presents this information in graphical form.

### 7.8 Cement Content

For the bridge deck, w/c ratio is 0.26, the water content = 174 kg/m$^3$, therefore, the cement content is $174/0.26 = 669$ kg/m$^3$.

For the bridge girder, w/c ratio = 0.2, the water content = 174 kg/m$^3$, therefore, the cement content is $174/0.2 = 870$ kg/m$^3$.

The proposed cement content for both the bridge deck and girder are greater than 335 kg/m$^3$, which is the minimum requirement for severe freeze-thaw, deicer environment.
Table 7.6. Minimum Requirements for Cementing Materials for Concrete [3]

<table>
<thead>
<tr>
<th>Nominal maximum size of aggregate, mm (in.)</th>
<th>Cementing materials, kg/m³ (lb/yd³)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5 (1½)</td>
<td>280 (470)</td>
</tr>
<tr>
<td>25 (1)</td>
<td>310 (520)</td>
</tr>
<tr>
<td>19 (¾)</td>
<td>320 (540)</td>
</tr>
<tr>
<td>12.5 (½)</td>
<td>350 (590)</td>
</tr>
<tr>
<td>9.5 (¾)</td>
<td>360 (610)</td>
</tr>
</tbody>
</table>

* Cementing materials quantities may need to be greater for severe exposure. For example, for deicer exposures, concrete should contain at least 335 kg/m³ (564 lb/yd³) of cementing materials. Adapted from ACI302.

7.9 Coarse Aggregate Content

According to Table 7.7, 25 mm aggregate and 2.8 fineness moduli should be used in our case. The corresponding bulk volume is 0.67. A bulk density of 1600 kg/m³ is assumed. Therefore, the dry mass of coarse aggregate for a cubic meter of concrete is 1600*0.67 = 1072 kg.

Table 7.7. Bulk Volume of Coarse Aggregate per Unit Volume of Concrete [3]
7.10 Admixture Content

For bridge deck, a 6% air content, 100 ml air entraining per 100 kg of cement material should be used. Therefore, for every cubic meter, 669 kg * 100 ml/100 kg = 669 ml air entraining admixture should be used.

For girder, 4.5% air content should also need 100 ml air entraining per 100 kg of cement. Therefore, for bridge deck, 870 ml air entraining mixture per cubic meter of concrete should be used.

Moreover, the water reducer dosage rate of 3 g per kg of cement results in 3*669 = 2007 g of water reducer per cubic meter of concrete should be used for the bridge deck. 2610 g of water reducer per cubic meter of concrete should be used for the bridge girder.

When using more than one admixture in concrete, the compatibility of intermixing admixtures should be assured by the manufacturers. They shall not react with each other, which may have negative influence on concrete. Moreover, for chloride-based admixture, the maximum dosage is restricted in Table 7.8 below to ensure the durability and reinforcement requirements.

Table 7.8. Maximum Chloride-Ion Content for Corrosion Protection

<table>
<thead>
<tr>
<th>Type of member</th>
<th>Maximum water-soluble chloride ion (Cl⁻) in concrete, percent by mass of cement*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressed concrete</td>
<td>0.06</td>
</tr>
<tr>
<td>Reinforced concrete exposed to chloride in service</td>
<td>0.15</td>
</tr>
<tr>
<td>Reinforced concrete that will be dry or protected from moisture in service</td>
<td>1.00</td>
</tr>
<tr>
<td>Other reinforced concrete construction</td>
<td>0.30</td>
</tr>
</tbody>
</table>
7.11 Fine Aggregate Content

This section presents the calculation to determine the fine aggregate content for the bridge deck and girder.

For the bridge deck, the following calculations are used to determine the estimated mass of fine aggregate:

\[
\begin{align*}
\text{Water} &= \frac{174}{1 \times 1000} = 0.174 \, m^3 \\
\text{Cement} &= \frac{669}{3000} = 0.223 \, m^3 \\
\text{Air} &= \frac{6}{100} = 0.06 \, m^3 \\
\text{Coarse Aggregate} &= \frac{1072}{2680} = 0.4 \, m^3
\end{align*}
\]

Volume of fine aggregate: 0.143 m³  
Mass of fine aggregate: 0.143 \times 2640 = 377.5 kg  
Estimated concrete density: 174 + 669 + 1072 + 377.5 = 2292.5 kg/m³

For the bridge girder: the following calculations are used to determine the estimated mass of fine aggregate:

\[
\begin{align*}
\text{Water} &= \frac{174}{1 \times 1000} = 0.174 \, m^3 \\
\text{Cement} &= \frac{870}{3000} = 0.29 \, m^3 \\
\text{Air} &= \frac{4.5}{100} = 0.045 \, m^3 \\
\text{Coarse Aggregate} &= \frac{1072}{2680} = 0.4 \, m^3
\end{align*}
\]

Volume of fine aggregate: 0.091 m³  
Mass of fine aggregate: 0.091 \times 2640 = 240 kg  
Estimated concrete density: 174 + 870 + 1072 + 240 = 2356 kg/m³
### 7.12 Moisture

In our case, coarse aggregate moisture content is 2% and fine aggregate moisture content is 6%.

Surface moisture contributed by the coarse aggregate is 2% - 0.5% =1.5%, that by the fine aggregate is 6% - 0.7% = 5.3%.

For the bridge deck:
- Coarse aggregate = 1072*1.02 = 1093.4 kg
- Fine aggregate = 377.5*1.06 = 400 kg
- Adjusted water content: 174 - (1072*0.015) - (377.5*0.053) = 138 kg

For the girder:
- Coarse aggregate = 1072*1.02 = 1093.4 kg
- Fine aggregate = 240*1.06 = 254.4 kg
- Adjusted water content: 174 - (1072*0.015) - (240*0.053) = 145 kg

### 7.13 Summary

The durability design for our previous detailed concrete bridge girder and deck are summarized in Table 7.9 and Table 7.10.

Table 7.9 Concrete Mix Design for Deck

<table>
<thead>
<tr>
<th>Concrete Mix for the Bridge Deck per Cubic Meter</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Class</strong></td>
</tr>
<tr>
<td><strong>Maximum Nomial Aggregate Size</strong></td>
</tr>
<tr>
<td><strong>Water to Cement Ratio</strong></td>
</tr>
<tr>
<td><strong>Required Average Strength</strong></td>
</tr>
<tr>
<td><strong>Air Content</strong></td>
</tr>
<tr>
<td><strong>Water Content</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>--------------------------------</td>
</tr>
<tr>
<td>Cement Content</td>
</tr>
<tr>
<td>Coarse Aggregate Mass</td>
</tr>
<tr>
<td>Fine Aggregate Mass</td>
</tr>
<tr>
<td>Admixtures</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Slump</td>
</tr>
</tbody>
</table>

Table 7.9 Concrete Mix Design for Girder

<table>
<thead>
<tr>
<th>Concrete Mix for the Girder per Cubic Meter</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Class</td>
<td>A-1</td>
</tr>
<tr>
<td>Maximum Nominal Aggregate Size</td>
<td>25 mm</td>
</tr>
<tr>
<td>Water to Cement Ratio</td>
<td>0.2</td>
</tr>
<tr>
<td>Required Average Strength</td>
<td>60 MPa</td>
</tr>
<tr>
<td>Air Content</td>
<td>4.5%</td>
</tr>
<tr>
<td>Water Content</td>
<td>145 kg</td>
</tr>
<tr>
<td>Cement Content</td>
<td>870 kg</td>
</tr>
<tr>
<td>Coarse Aggregate Mass</td>
<td>1072 kg</td>
</tr>
<tr>
<td>Fine Aggregate Mass</td>
<td>240 kg</td>
</tr>
<tr>
<td>Admixtures</td>
<td></td>
</tr>
<tr>
<td></td>
<td>669 ml Air Entraining</td>
</tr>
<tr>
<td></td>
<td>2.007 kg Water Reducer</td>
</tr>
<tr>
<td>Slump</td>
<td>100 ± 20 mm</td>
</tr>
</tbody>
</table>
Reference:

[1] Canadian Standards Association, A23.1-14/A23.2-14 Concrete materials and methods of concrete construction/Test methods and standard practices for concrete, Mississauga, Canada: Table 1.

[2] Canadian Standards Association, A23.1-14/A23.2-14 Concrete materials and methods of concrete construction/Test methods and standard practices for concrete, Mississauga, Canada: Table 2.

Appendix A: Design Drawing
Bridge Overview

Produced By:
BriD&E Design and Engineering Consulting Corp.
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SIDE 1:150

ELEVATION 1:150

ISOTROPIC 1:300

MATERIAL PROPERTIES:

CONCRETE:
\( f'c = 40 \text{ MPa} \)
\( f'c = 50 \text{ MPa (GIRDER)} \)
\( f'c = 40 \text{ MPa (DECK)} \)

ALL OTHER MATERIALS SPECIFIED ON INDIVIDUAL SHEETS

DETAIL A - GIRDER CROSS SECTION 1 : 25

OVERALL GEOMETRY

Drawing Title: 

Date: MARCH 2017

Sheet Number: 2

Drawn By: M.Z.

Designed By: BriD&E

Scale: N.T.S.

Civil Engineering
UNIVERSITY OF TORONTO

BriD&E
REINFORCEMENT LAYOUT

MATERIAL PROPERTIES:
CONCRETE:
TOP COVER (MIN) = 40 MM

PRESTRESSING TENDON:
SIZE 15 LOW RELAXATION
7 WIRE STRANDS
fu = 1860 MPa
fy = 1675 MPa
Ep = 200,000 MPa

DETAIL C
SCALE 1: 25

STIRRUP PROFILE
10M @200 MM TOP SHRINKAGE AND TEMPERATURE REINFORCEMENT

15M @200 MM TOP TRANSVERSE REINFORCEMENT

15M @150 MM BOTTOM TRANSVERSE REINFORCEMENT

10M @165 MM BOTTOM DISTRIBUTION REINFORCEMENT

MATERIAL PROPERTIES:

CONCRETE:
TOP COVER (MIN) = 40 MM
BOTTOM COVER (MIN) = 20 MM

REINFORCING STEEL:
fy = 400 MPa
Es = 200,000 MPa

Drawing Title: DECK DESIGN CSA S6-14

Date: MARCH 2017

Sheet Number: 4

Designed By: BriD&E

Scale: 1:150
ISOTROPIC VIEW 1:250

MATERIAL PROPERTIES:
CONCRETE:
TOP COVER (MIN) = 40 MM

PRESTRESSING TENDON:
SIZE 15 LOW RELAXATION
7 WIRE STRANDS
fu = 1860 MPa
fy = 1675 MPa
Ep = 200,000 MPa

STIRRUP PROFILE

12-10M @ 400 MM
5-10M @ 500 MM
4-10M @ 400 MM

Detail C
SCALE 1:25
DECK REINFORCEMENT LAYOUT

COMBINED
LONGITUDINAL
TRANSVERSE

MATERIAL PROPERTIES:

CONCRETE:
- TOP COVER (MIN) = 40 MM
- BOTTOM COVER (MIN) = 20 MM

REINFORCING STEEL:
- $f_y = 400$ MPa
- $E_s = 200,000$ MPa

Drawing Title: DECK DESIGN ASSHTO LRFD-14
Sheet Number: 6

Date: MARCH 2017
Drawn By: M.Z.
Designed By: BriD&E
Scale: 1:150
REINFORCEMENT LAYOUT

MATERIAL PROPERTIES:
CONCRETE:
TOP COVER (MIN) = 40 MM

PRESTRESSING TENDON:
SIZE 15 LOW RELAXATION
7 WIRE STRANDS
fu = 1860 MPa
fy = 1675 MPa
Ep = 200,000 MPa

DETAIL F
SCALE 1 : 25

TENDON PROFILE

STIRRUP PROFILE

6-10M @ 400 MM 12-10M @ 200 MM 8-10M @ 180 MM
COMBINED
LONGITUDINAL
TRANSVERSE
DECK REINFORCEMENT LAYOUT

10M @250 MM TOP SHRINKAGE AND TEMPERATURE REINFORCEMENT.
15M @ 330 MM TOP TRANSVERSE REINFORCEMENT
15M @165 MM BOTTOM TRANSVERSE REINFORCEMENT
10M @150 MM BOTTOM DISTRIBUTION REINFORCEMENT

MATERIAL PROPERTIES:
CONCRETE:
TOP COVER (MIN) = 40 MM
BOTTOM COVER (MIN) = 20 MM
REINFORCING STEEL:
f_y = 400 MPa
E_s = 200,000 MPa

Drawing Title: DECK DESIGN CSA S6-66
Sheet Number: 8

Date: MARCH 2017
Drawn By: M.Z.
Designed By: BriD&E
Scale: 1:150