Chapter 1 – Introduction to Part B

The part A of this report focuses on conceptual design process. This part of the report, part B, is focused on the detailed design of a prestressed concrete simply supported bridge. The bridge designed in this report is meant to be a replacement for the De La Concorde Overpass that collapsed September 30, 2006. The purpose of this report is to compare the old code used to design the collapsed overpass, CSA S6-66, with the modern two codes, CSA S6-14 rev.17 and AASHTO LRFD 2014-17 by designing a replacement bridge in 3 different codes and comparing them.

The design process begins with chapter 2 where initial information about materials, specifications, and dimensions are given. These include the preliminary design of the bridge cross-section, stress-strain curves of materials and concrete properties.

Following this chapter, each chapter has a specific section for each design code. Calculations are done for 3 design codes and at the end of each chapter, results obtained are compared.

In chapter 3, some information about influence lines are given as an introduction to live load analysis. Then, truck loads are introduced and the process of determining undistributed unfactored truck loading is explained in detail with images and subtexts. At the end, our MATLAB codes for determining truck loading for each code is published. They are intended to visualize what happens as the truck moves and calculate loading.

Moving on to chapter 4, dead load concept is also introduced and calculated. Together with dead loads, superimposed dead loads and live loads are combined based on each design code and distributed on 1 interior girder. The exterior girder distributions and design is not presented in this report.

In chapter 5, the type of girder that will carry the slab is chosen and designed for the 3 design codes based on the loads obtained from chapter 3 and chapter 4. Both service conditions and ultimate conditions for flexure are analyzed using hand calculations and computer programs. Most calculations are done using EXCEL spreadsheets and MATLAB.

Part B will conclude with chapter 6 and 7, in which bridge deck is designed using a standard deck design procedure and concrete characteristics for a durable design is provided.

At the very end, in the main appendix, designed drawings produced by AutoCAD are published.

Chapter 2 - Project Statement

Table of Contents

2.1 Introduction

The proposed bridge design will explain the details of the girder and the reinforced concrete bridge deck. This chapter gives a brief introduction to the geometric and material properties of the proposed bridge. Most of the values provided in this section will be used throughout the bridge design process as a reference.

2.2 Geometric Properties

>The cross-section below is the cross-section of the bridge to be designed somewhere near midspan.

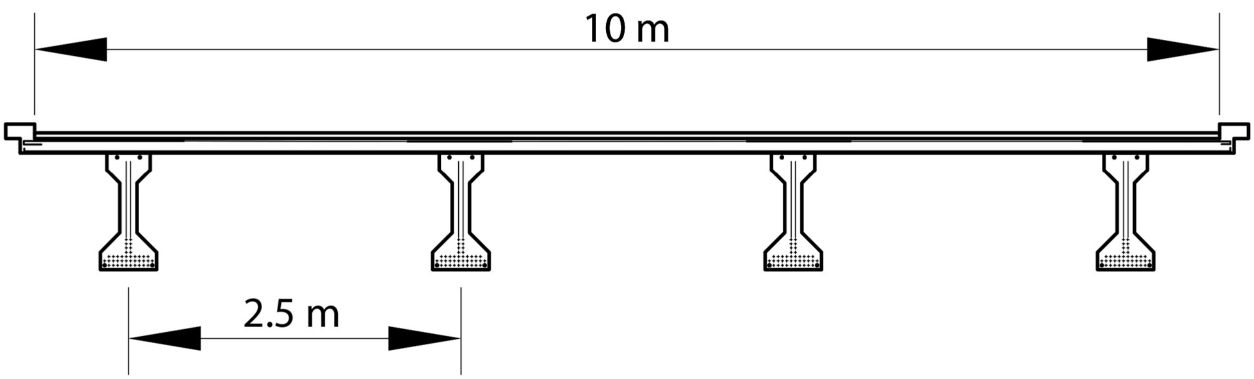

Figure 2.1.1 - Sample cross-section of the bridge to be designed

The span of the bridge is to be 26 meters and simply supported at the ends. Expansion joints must be installed at both ends to prevent cracking. Bridge deck is to be around 200 mm with approximately 65 mm of asphalt and waterproofing on top for durability.

2.3 Material Properties

Precast girders: f'c 40 MPa concrete*

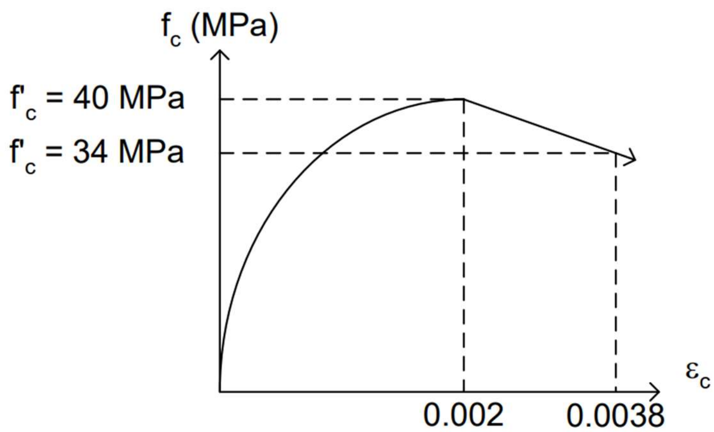

Figure 2.2.1 Girder concrete stress-strain relationship

*A minimum strength of 35 MPa is required at transfer

Girder concrete modulus of elasticity to be calculated as secant-modulus to the curve above (Modified Hognestad's Parabola) at 0.45 x f'c or 0.4 x f'c (depends on the design code) or using the empirical design code equations.

Deck: f'c 35 MPa concrete

Deck concrete modulus of elasticity can be calculated using the empirical equations given in the design codes.

Remainder of the reinforced concrete: f'c 40 MPa concrete

These will depend on the design code, however, below is suggested preliminary for 100 Year Life.

Clear cover deck top: 70 +- 10 mm
Clear cover bottom of the deck: 50 +- 10 mm
Clear cover bottom of the girder: 40 +- 10 mm
Clear cover remainder: 70 +- 10 mm

Reinforcement: Standard fy= 400 MPa Canadian Reinforcement

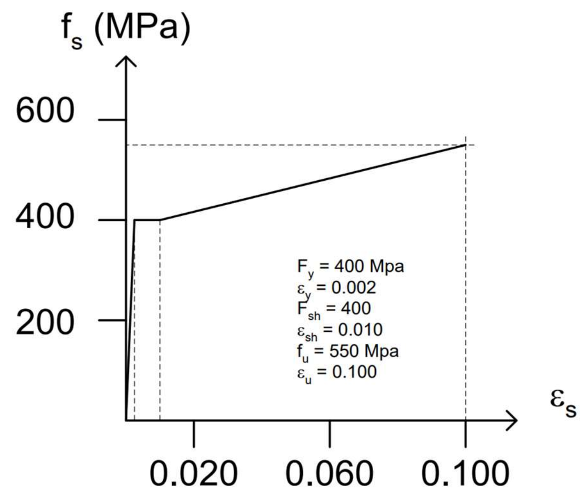

Figure 2.2.2 Reinforcing steel stress-strain relationship

<u>Prestressing strands in girders:</u> Low-Relaxation 7 wire: f_u = 1860 MPa

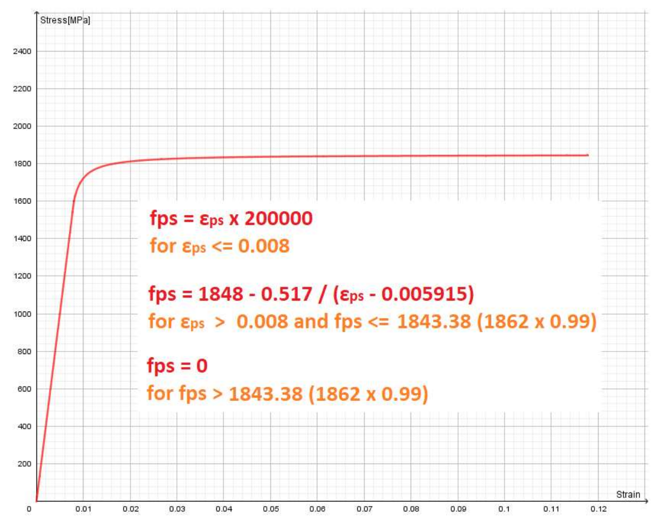

Figure 2.2.3 Prestressing steel stress-strain relationship

Force per strand after losses should be at least 100 kN at any cross section

2.4 Conclusion

The geometric and material properties and assumptions provided in this chapter will be used throughout the bridge design process and will also help in the decision-making process. The design in the following chapters will provide comparisons between the bridge design codes: CSA S6-14, AASHTO 2014 and S6-66.

Chapter 3 - Influence Lines - Truck Load Analysis & Design Envelopes

Table of Contents

3.1 Introduction

In this chapter, influence lines and truck load analysis concepts adopted by popular bridge design codes will be analyzed. The hand explanations showing the procedure and the MATLAB code (entirely written by us) used to produce the graphs presented in this chapter can be found at the appendix section of part B.

Influence lines are graphs that show the variation of shear and moment forces that a structural member experiences under unit load at a given location as load moves from one end of the member to another. They help us identify the approximate location of live load. Influence Lines don't have to be "lines" for all members. In fact, the name is only valid for determinate members [2].

Truck load analysis is based on moving a code defined design truck from one end of the bridge to other both ways and recording the maximum absolute values obtained as the truck moves. This can be done in various ways and will be discussed in section 3.4.

We will be doing a truck load analysis based on the design truck of the following 3 codes in this chapter:

3.2 Influence Line Methods

There are three main methods for generating the influence line graphs. The first one is called tabulated values procedure, the second one is called influence line equations and the last one is called qualitative influence line method (also called Muller Breslau's Principle for influence lines).

3.2.1 Tabulated Values Procedure

In tabulated procedure, a unit load is placed on the member at a distance x from left support and statics is then used to calculate reactions, moment and shear diagrams due to that load. Then, the location of the load is changed and again the same values are calculated. This process is repeated until a pattern is seen and the points that form the maximums for any point are connected to create the influence line graphs. If the member is statically determinate and simply supported, analyzing 1 point is enough to generate the influence lines if the point is smartly chosen [2].

3.2.2 Influence-Line Equations

It is possible to get an equation of the influence line graphs by choosing enough points using the tabulated procedure above and write the equation of the curve that passes from those points. How many points required for this procedure depends on the degree of indeterminacy of the member [3].

3.2.3 Qualitative Influence Lines - Muller Breslau's Principle

Qualitative influence lines are influence lines produced graphically using the principle of virtual work. The influence line for any action (reactions, internal shear forces, internal moment forces) is equivalent to the deflection curve when the action is removed and replaced with a corresponding unit displacement or rotation [1]. To get the influence line, the ability of the member to resist the corresponding action in the direction of the action should be removed. Then, the member should be allowed to deform 1 unit at that location keeping the member rigid (infinite stiffness) and obeying the internal force directions. The deflected shape will then become the influence line for that member at that location.

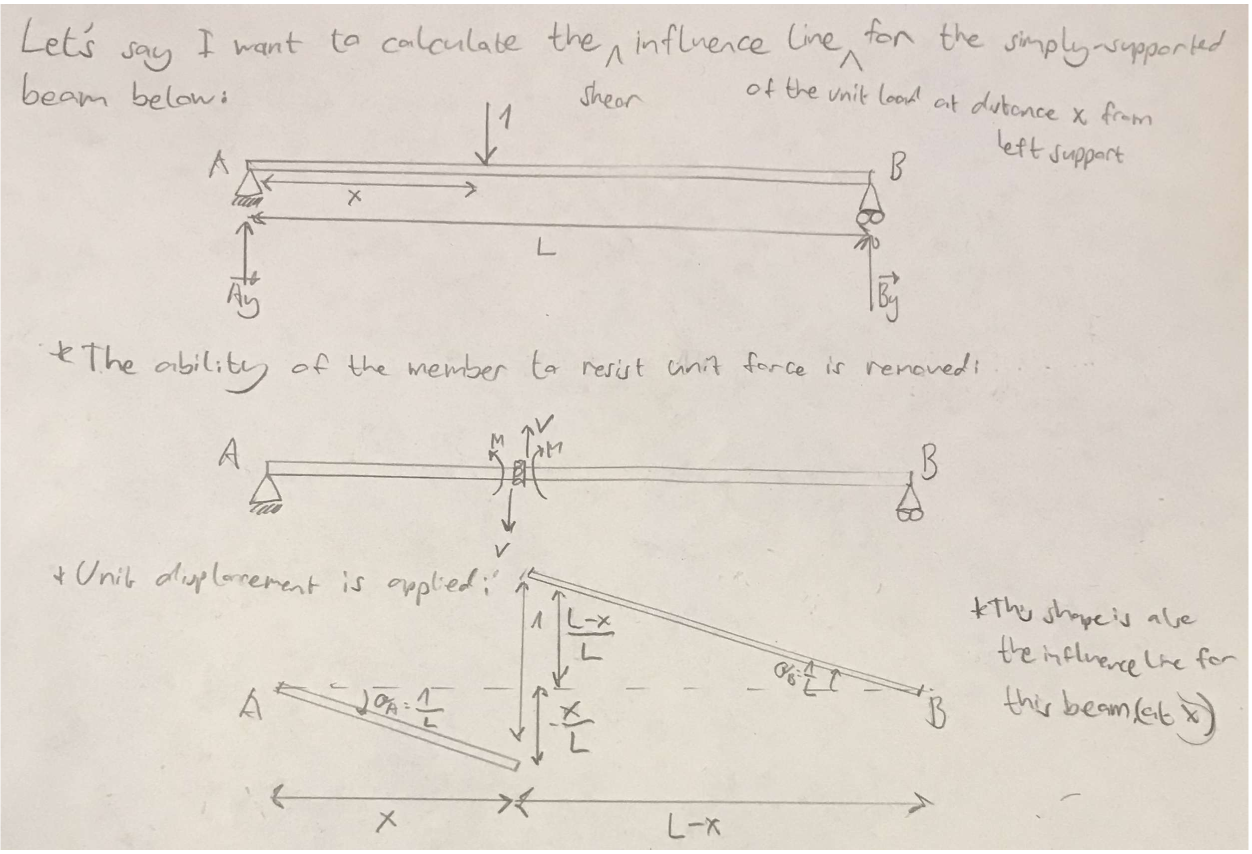

Figure 3.2.3.1: Qualitative Influence Lines

3.3 Influence Lines for our Bridge

For the purposes of this chapter, we considered our bridge as a simply supported rigid beam with a span length of 26 meters. Influence lines are plotted every 2 meters for shear and moment. Then, envelopes producing those are presented using computer graphing tools. A unit moving load of 1kN is used throughout the analysis.

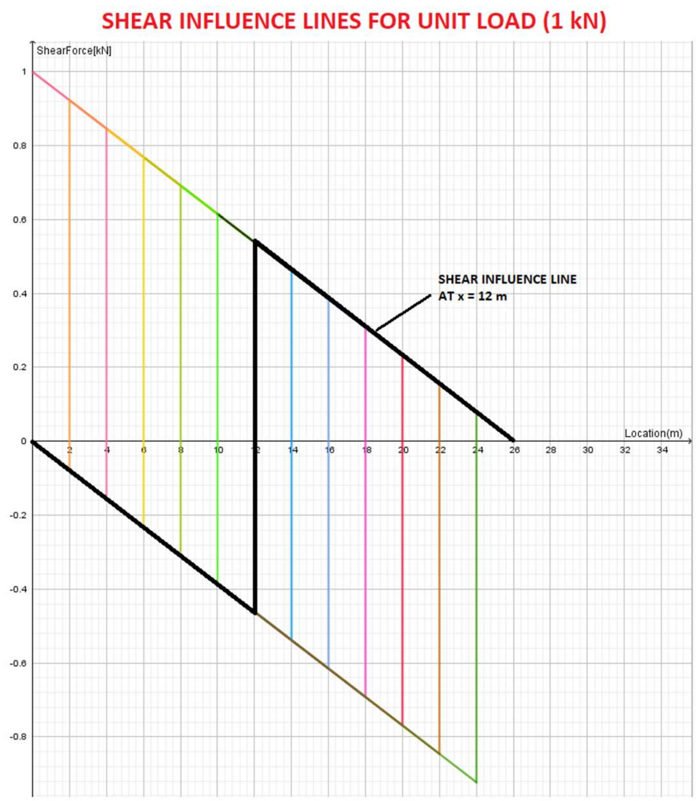

Figure 3.3.1

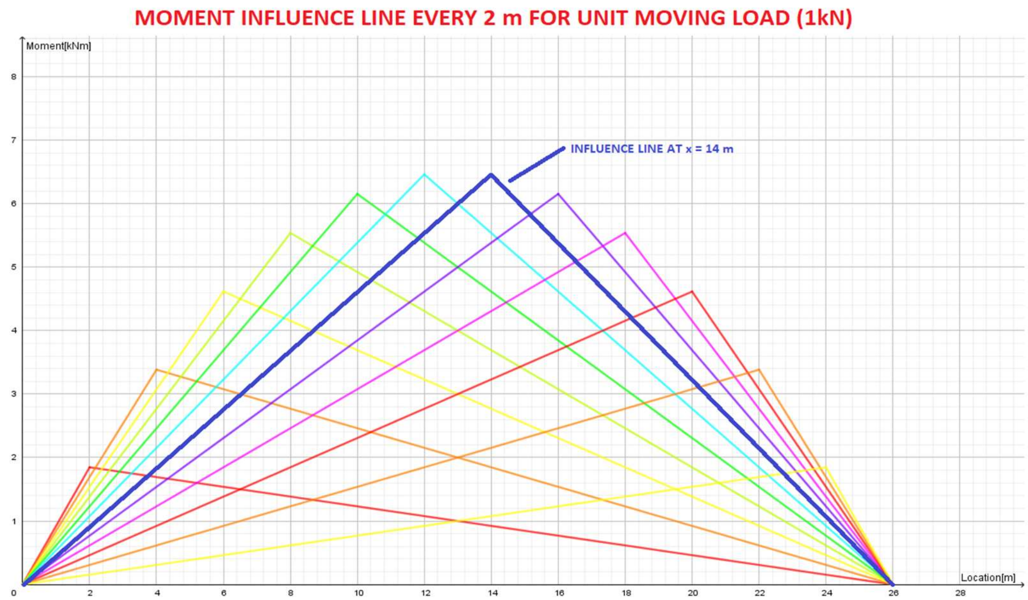

Figure 3.3.2

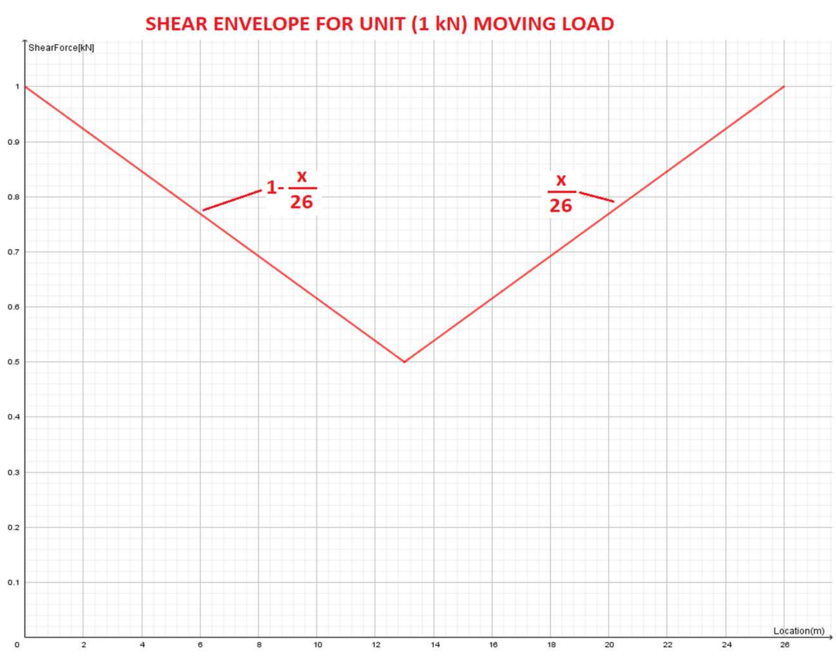

Figure 3.3.3

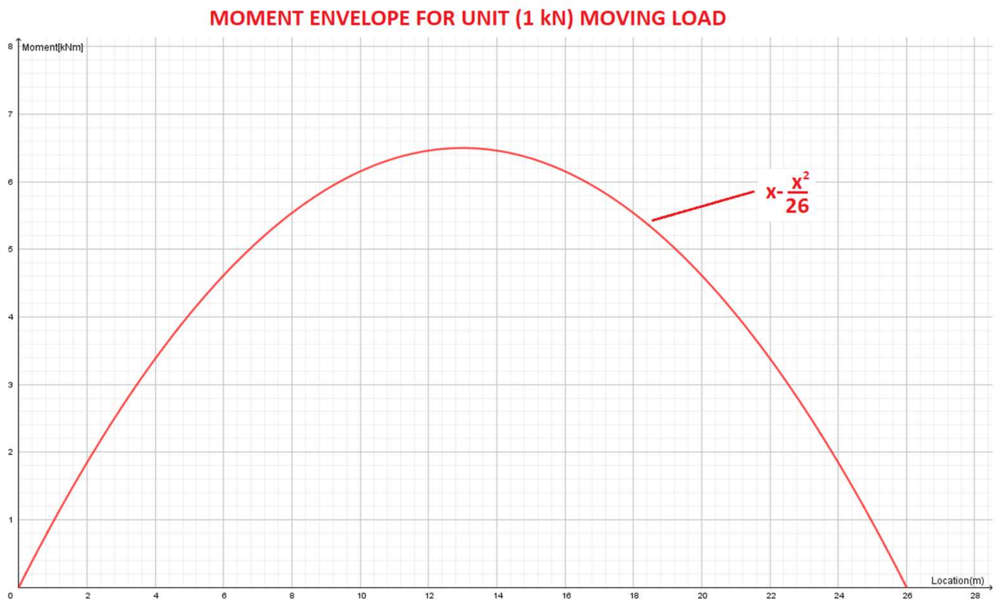

Figure 3.3.4

3.4 Truck Load Analysis

In this section, the design truck for CSA S6-14, AASHTO LRFD 2014-17 and CSA S6-66 is moved from left to right and then from right to left on a simply supported rigid beam with span length of 26 meters representing our bridge.

For the CL-625 Truck (Design tandem used by CSA S6-14), as the trucks front wheels enter on the bridge, we start drawing shear and moment diagrams for every cm truck travels. Then, we take the maximum value for diagrams again for every cm on the beam and store that information. As the truck moves, we keep updating the maximum values for moment and shear. When the rear axle of the truck exits the bridge, we stop the process and at that time we have the moment and shear envelopes for one-way travel. Then we make the truck travel the other way since the diagram obtained is not symmetrical. Then we take the maximum of each way for each of the 2600 locations and draw our both-way envelopes.

AASHTO and CSA S6-66 use the same design truck and the data collection process is the same with CSA design truck. However, AASHTO and CSA S6-66 trucks have a variable rear axle spacing. To account for that, we start with a spacing of 4.3 m and make the spacing larger by 0.94 m at each time until we reach to 9 m. We check moment and shear envelope values for each 2600 locations for each rear axle spacing, determine which axle spacing is producing more force for each case and add that to our final design envelope.

The design Truck for CSA S6-14, AASHTO and CSA S6-66 are given below:

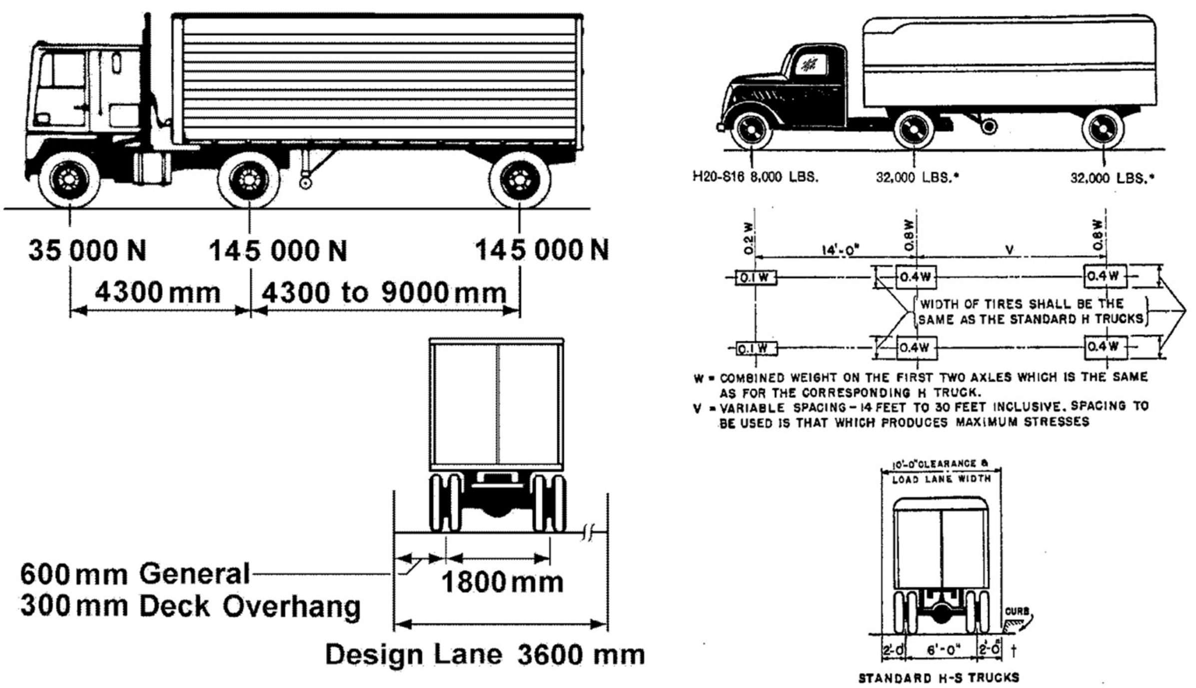

Figure 3.4.1 Design Truck for CSA S6-66 and AASHTO LRFD 2014-17 [4][6]

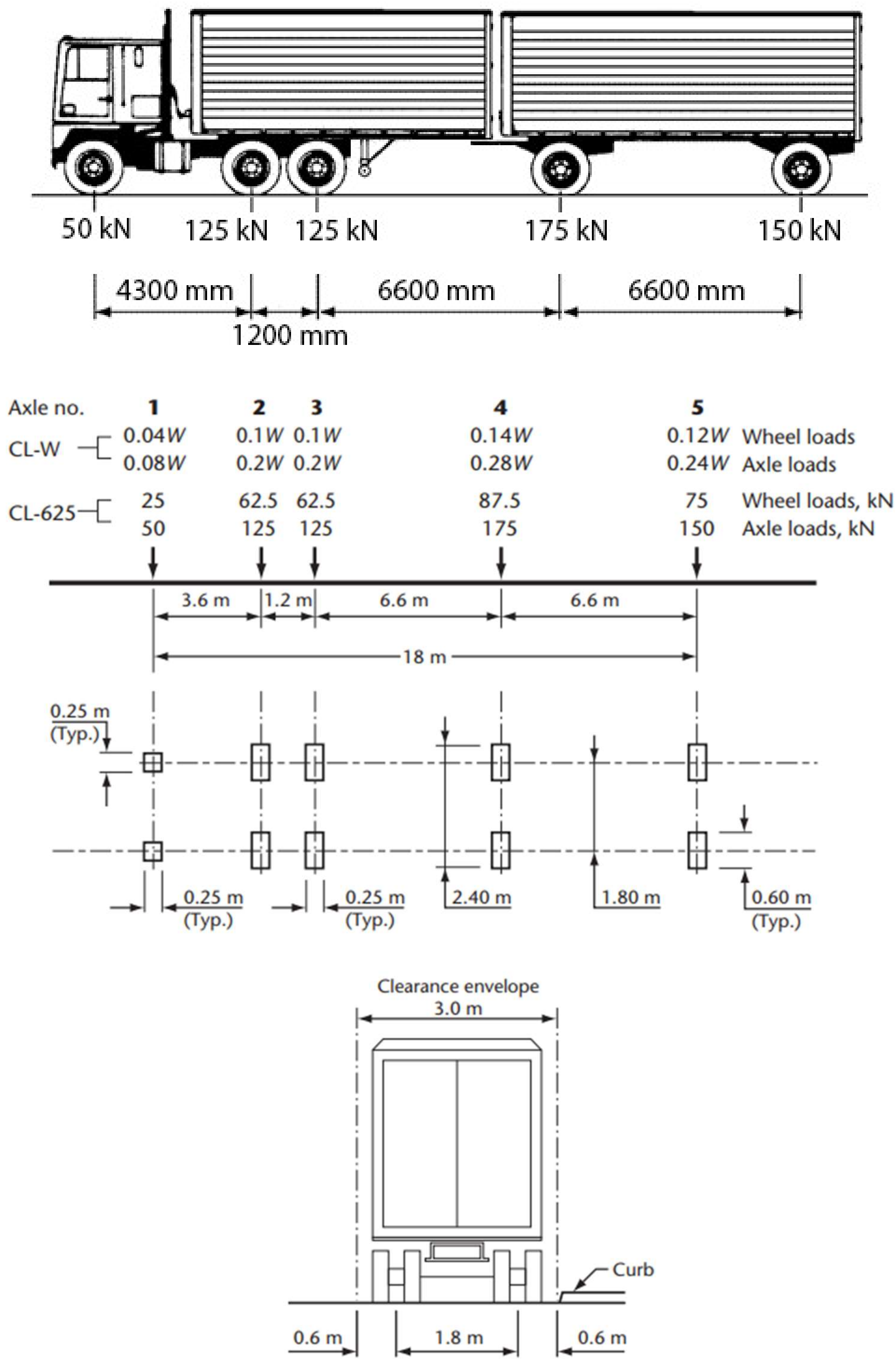

Figure 3.4.2 Design Truck For CSA S6-14 [5]

3.4.1 Shear and Moment Design Envelopes

In this section, shear and moment envelopes calculated are presented both with a numerical result table and graphically.

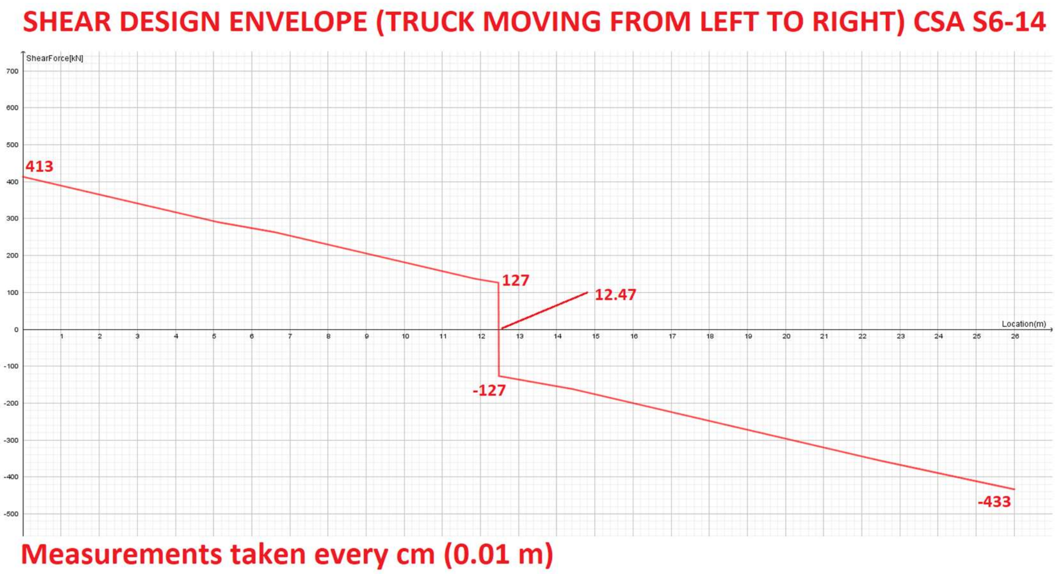

Figure 3.4.1.1

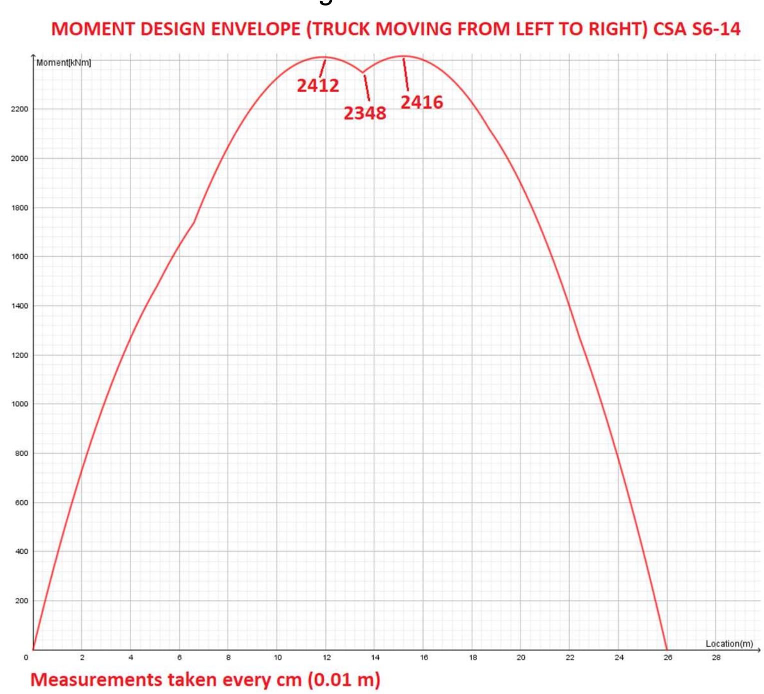

Figure 3.4.1.2

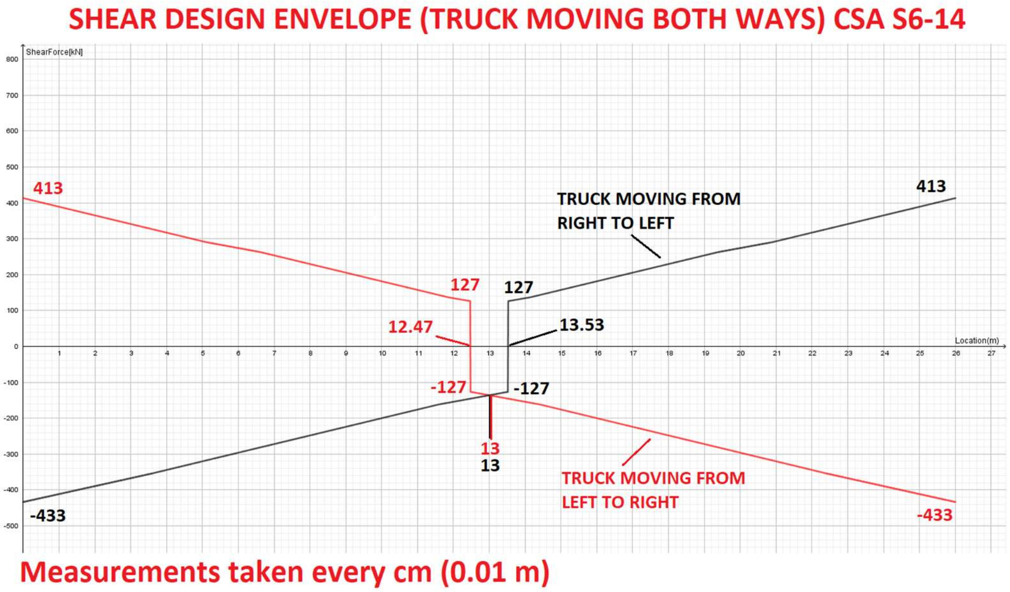

Figure 3.4.1.3

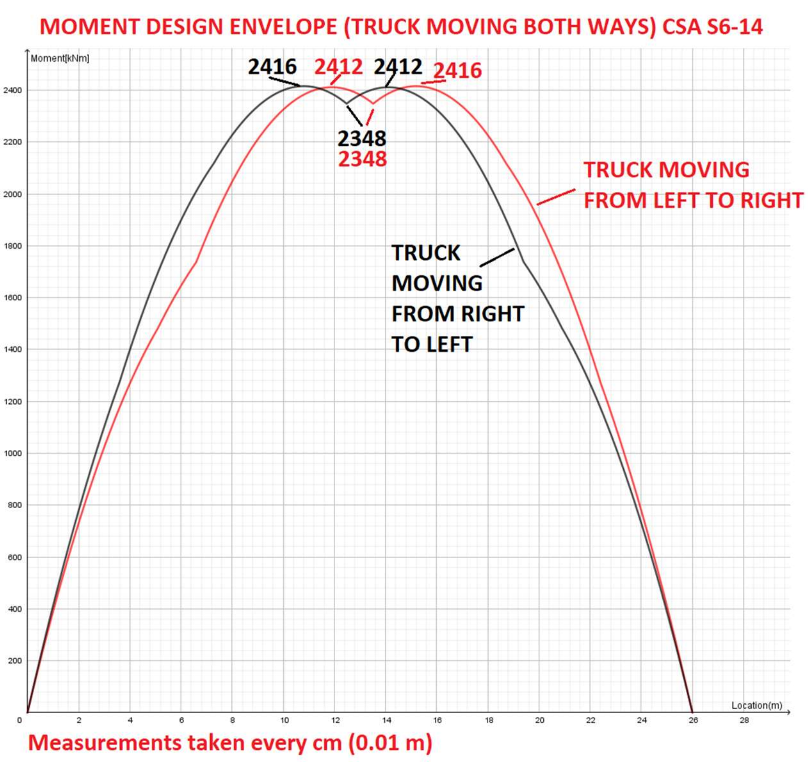

Figure 3.4.1.4

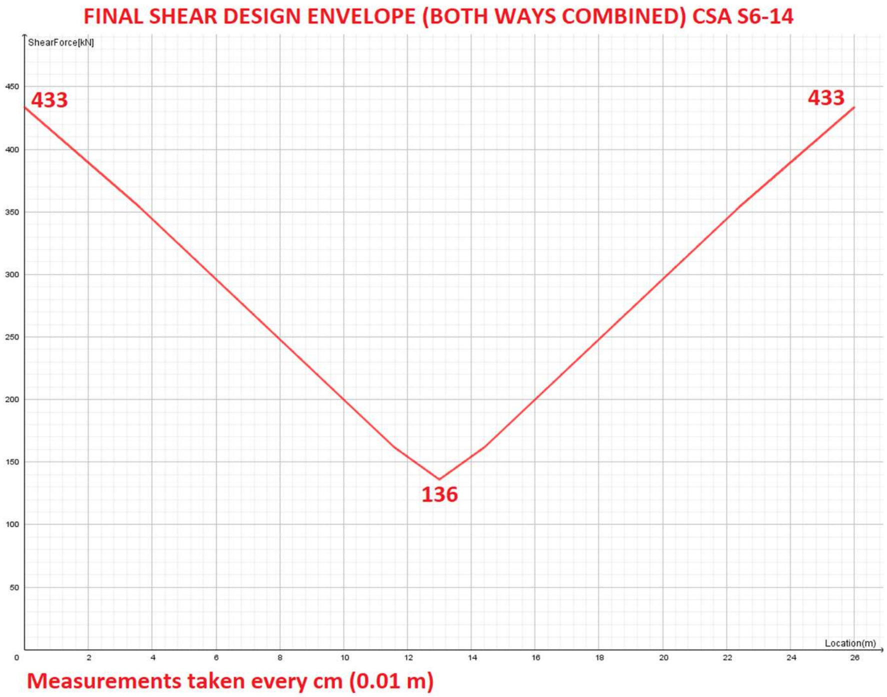

Figure 3.4.1.5

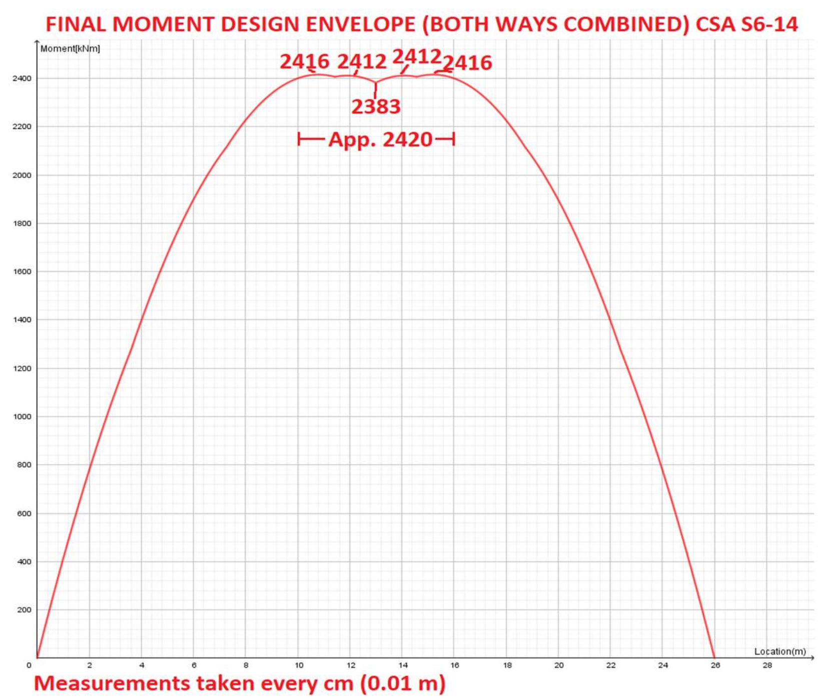

Figure 3.4.1.6

FINAL SHEAR DESIGN ENVELOPES BASED ON DIFFERENT AXLE SPACING AASHTO AND CSA S6-66

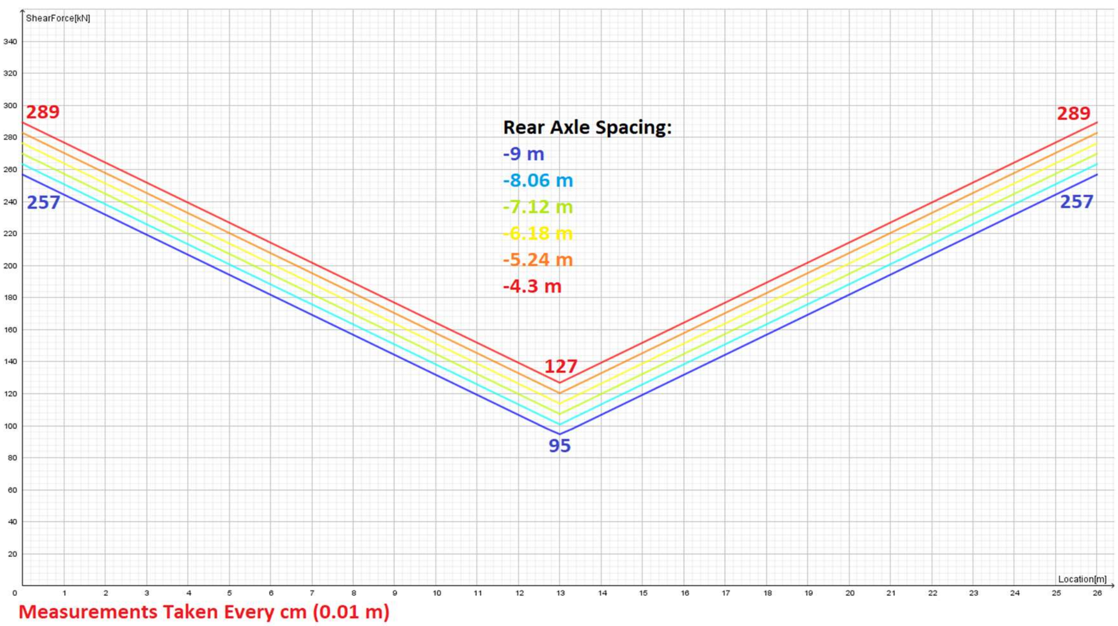

Figure 3.4.1.7

FINAL MOMENT DESIGN ENVELOPES BASED ON DIFFERENT AXLE SPACING
AASHTO & CSA S6-66

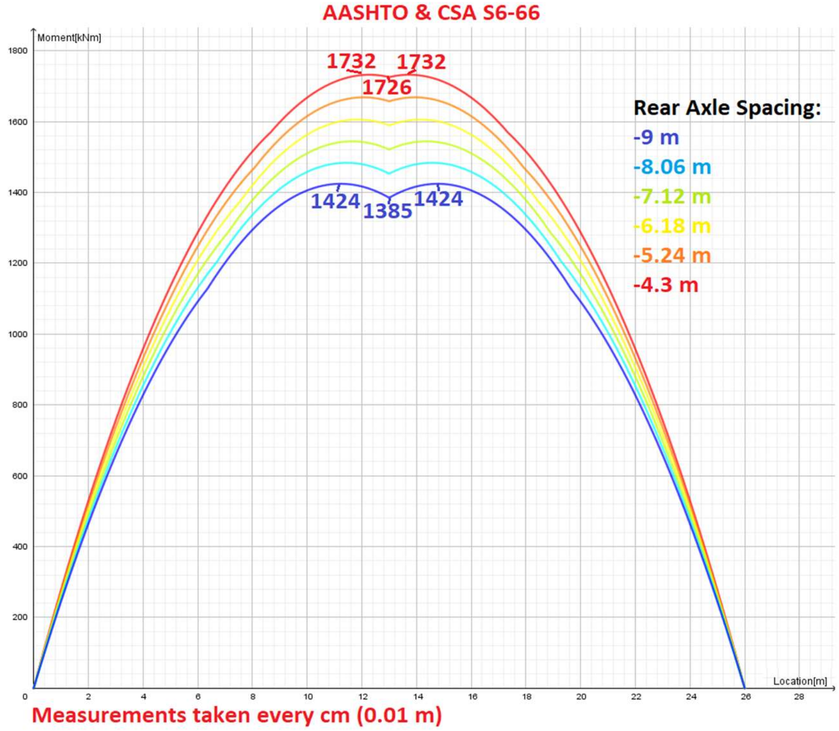

Figure 3.4.1.8

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Figure 3.4.1.9

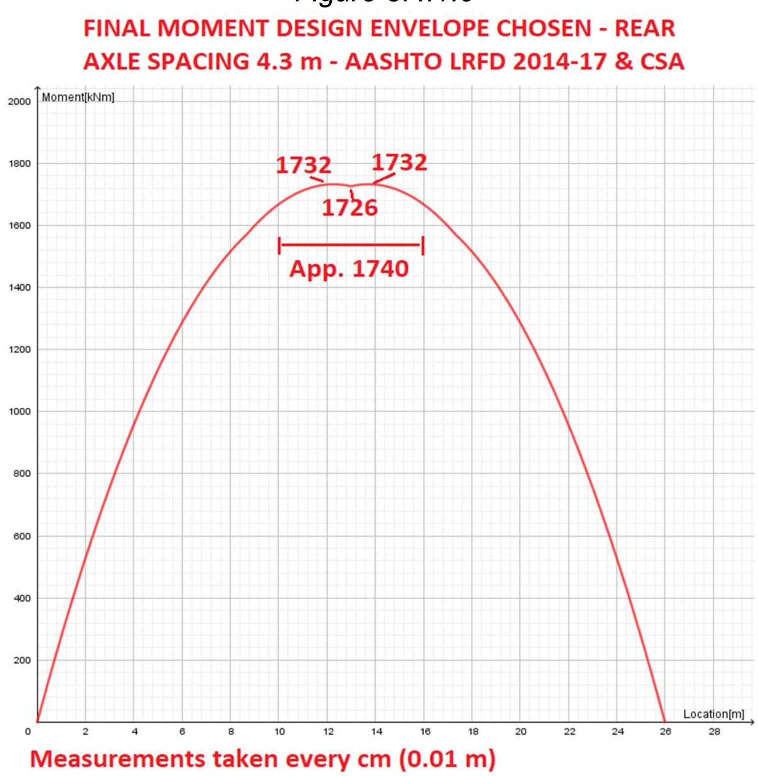

Figure 3.4.1.10

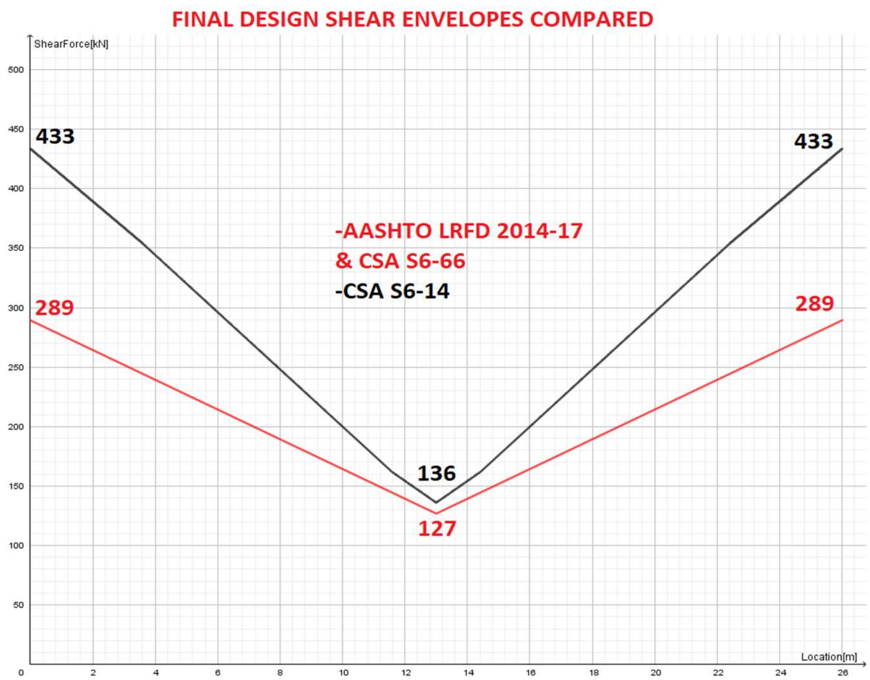

Figure 3.4.1.11

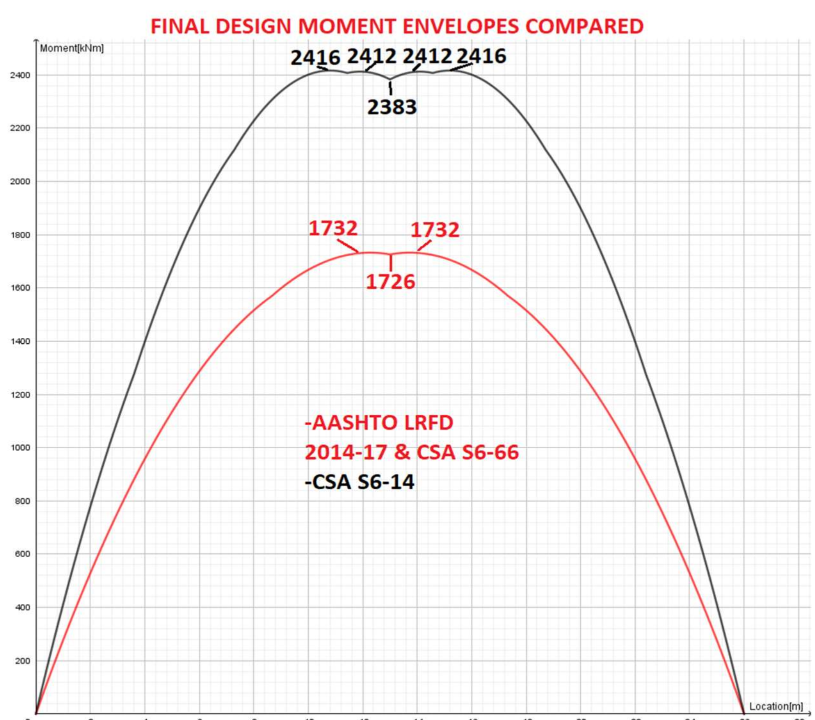

Figure 3.4.1.12

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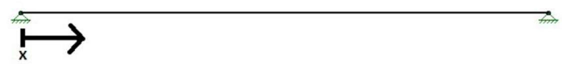
Table 3.4.1.1 - Numerical Results



	AASHTO LRFD 2014-17 & CSA S6-66		CSA S6-14	
Location from the left support x (m)	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	289.32	0	433.43	0
0.5	283.07	141.60	422.38	211.30
1	276.82	276.94	411.32	411.54
1.5	270.57	406.04	400.26	600.72
2	264.32	528.88	389.20	778.85
2.5	258.07	645.48	378.14	945.91
3	251.82	755.83	367.09	1101.92
3.5	245.57	859.92	356.03	1246.88
4	239.32	957.77	344.18	1397.69
4.5	233.07	1049.37	332.16	1540.82
5	226.82	1134.71	320.14	1671.92
5.5	220.57	1213.81	308.13	1791.01
6	214.32	1286.65	296.11	1898.08
6.5	208.07	1353.25	284.09	1993.13
7	201.82	1413.60	272.07	2076.15
7.5	195.57	1467.69	260.05	2153.51
8	189.32	1515.54	248.03	2226.92
8.5	183.07	1557.13	236.01	2288.32
9	176.82	1598.27	223.99	2337.69
9.5	170.57	1636.05	211.97	2375.05
10	164.32	1667.58	199.95	2400.38
10.5	158.07	1692.86	187.93	2413.70
11	151.82	1711.88	175.91	2415.00
11.5	145.57	1724.66	163.89	2408.03
12	139.32	1731.19	154.24	2411.54
12.5	133.07	1731.47	145.11	2403.03
13	126.82	1725.50	135.97	2382.50
13.5	133.07	1731.47	145.11	2403.03
14	139.32	1731.19	154.24	2411.54
14.5	145.57	1724.66	163.89	2408.03
15	151.82	1711.88	175.91	2415.00
15.5	158.07	1692.86	187.93	2413.70
16	164.32	1667.58	199.95	2400.38
16.5	170.57	1636.05	211.97	2375.05
17	176.82	1598.27	223.99	2337.69
17.5	183.07	1557.13	236.01	2288.32
18	189.32	1515.54	248.03	2226.92
18.5	195.57	1467.69	260.05	2153.51
19	201.82	1413.60	272.07	2076.15
19.5	208.07	1353.25	284.09	1993.13
20	214.32	1286.65	296.11	1898.08
20.5	220.57	1213.81	308.13	1791.01
21	226.82	1134.71	320.14	1671.92
21.5	233.07	1049.37	332.16	1540.82
22	239.32	957.77	344.18	1397.69
22.5	245.57	859.92	356.03	1246.88
23	251.82	755.83	367.09	1101.92
23.5	258.07	645.48	378.14	945.91
24	264.32	528.88	389.20	778.85
24.5	270.57	406.04	400.26	600.72
25	276.82	276.94	411.32	411.54
25.5	283.07	141.60	422.38	211.30
26	289.32	0	433.43	0
Critical Values:			**Critical Values:**	
0	289.32	-	433.43	-
13	126.82	-	135.97	-
0	289.32	-	433.43	-
10.8	-	-	-	2415.92
11.43	-	-	-	2406.58
11.9	-	-	-	2411.80
12.27	-	1732.12	-	-
13	-	1725.50	-	2382.50
13.73	-	1732.12	-	-
14.1	-	-	-	2411.80
14.57	-	-	-	2406.58
15.2	-	-	-	2415.92

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3.6 Conclusion

In conclusion, we can see that CSA S6-14 produces more shear and moment than AASHTO & CSA S6-66 design trucks. This is normal due to their difference in weight. CSA S6-14 truck weighs 1.92 times more than AASHTO & CSA S6-66 design trucks but this doesn't translate to 1.92 times more shear and moment. We obtained 1.5 times more critical maximum shear and 1.4 times more critical maximum moment.

3.7 References

[1] AMIN GHALI, STRUCTURAL ANALYSIS: a unified classical and matrix approach. S.I.: CRC PRESS, 2018.

[2] M. J. Ryall, G. A. R. Parke, and J. E. Harding, The manual of bridge engineering. London: Thomas Telford, 2003

[3] Hibbeler, R.C. Structural Analysis (Seventh Edition). Pearson Prentice Hall, New Jersey, 2009.

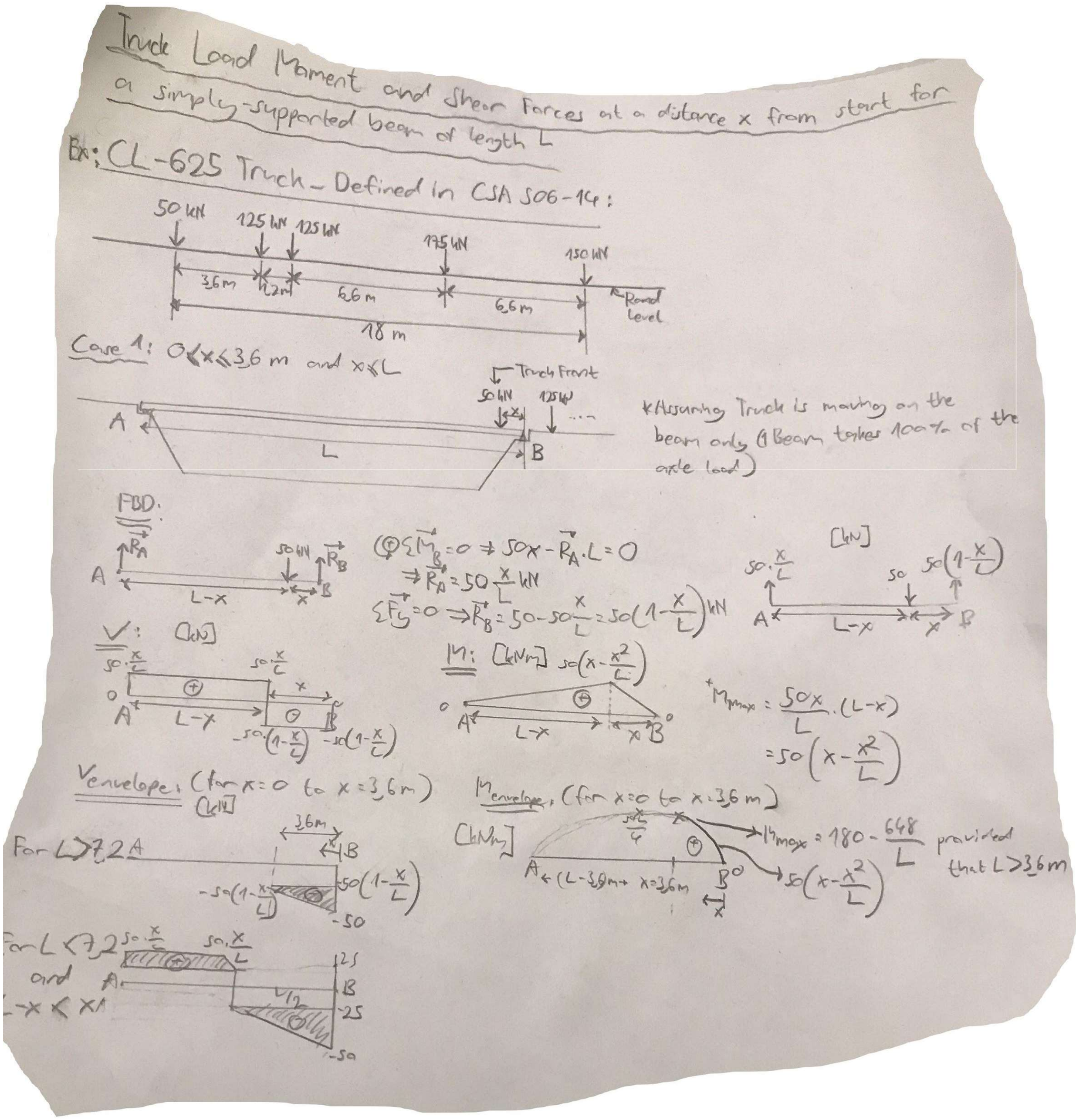
[4] AASHTO LRFD Bridge Design Specifications: American Association of State Highway and Transportation Officials, 2014, 8th Edition - Revision 2017

[5] CSA S6-14 Highway Bridge Design Code: Canadian Standards Association, 2014, Revision 2017

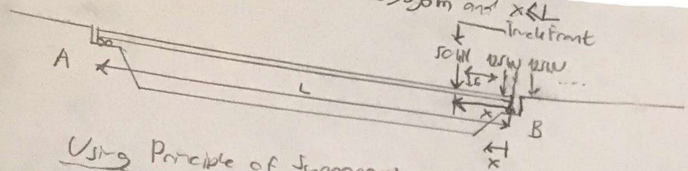
[6] CSA S6-66 Design of Highway Bridges: Canadian Standards Association, 1966

3.8 Appendix

3.8.1 Sample hand calculations for showing procedure



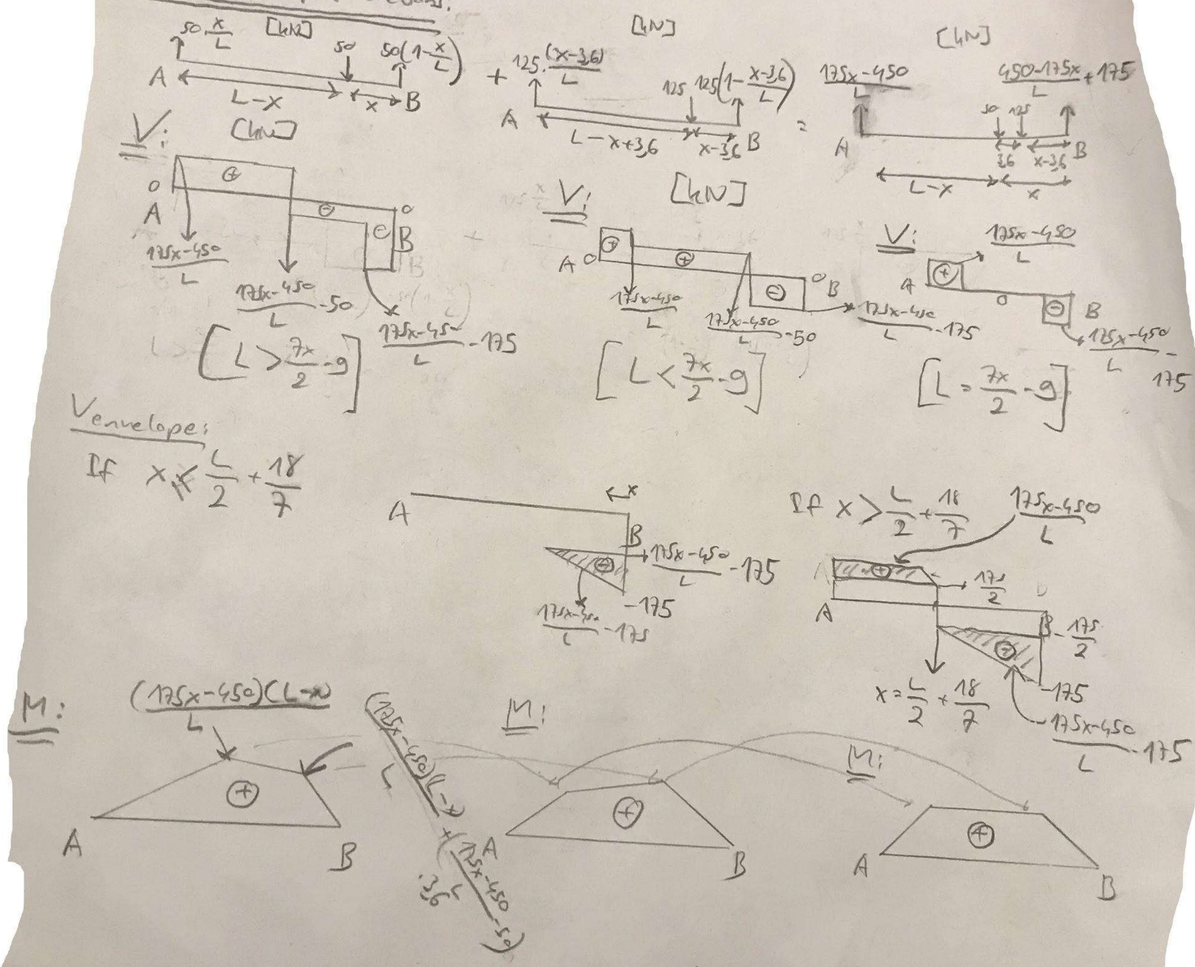
Case 2: $3.6 < x \leq 4.8$ m and $L > 3.6$ m and $x \leq L$



* Same Assumptions with Case 1

Using Principle of Superposition of Linear Systems:

FBD and support reactions:



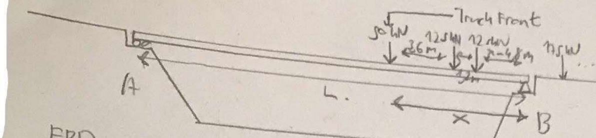
* From now on, I will proceed with $L > 18$ m since:

a) I showed the calculation procedure

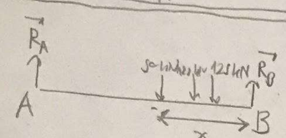
b) Our design length is 26 m.

c) Calculations get complicated.

Case 3: $3{,}8\,m < x \leq 11{,}4\,m \quad L > 18\,m$



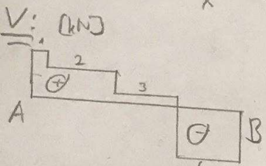
FBD and support reaction using superposition:



$$\vec{R_A} = 50\cdot\frac{x}{L} + 125\cdot\frac{(x-3{,}6)}{L} + 125\cdot\frac{(x-4{,}8)}{L} = \frac{300x-1050}{L}$$

$$\vec{R_B} = 50\left(1-\frac{x}{L}\right) + 125\left(1-\frac{(x-3{,}6)}{L}\right) + 125\left(1-\frac{(x-4{,}8)}{L}\right) = \frac{1050-300x}{L} + 300$$

V: [kN]



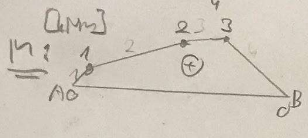
$$1 = \frac{300x-1050}{L}$$

$$2 = \frac{300x-1050}{L} - 50$$

$$3 = \text{〃} - 175$$

$$4 = \text{〃} - 300$$

M: [kNm]



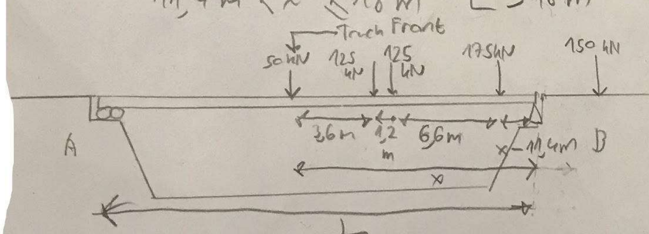
$$1 = \frac{300x-1050}{L}(L-x)$$

$$2 = \frac{300x-1050}{L}(L-x) + \left(\frac{300x-1050}{L} - 50\right)(3{,}6)$$

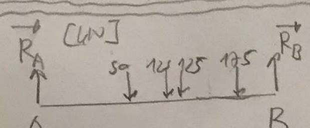
$$3 = \frac{300x-1050}{L}(L-x) + \left(\frac{300x-1050}{L} - 50\right)(3{,}6) + \left(\frac{300x-1050}{L} - 175\right)(1{,}2)$$

Case 4:

$11{,}4\,m < x \leq 18\,m \quad L > 18\,m$



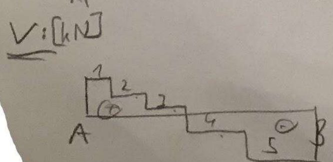
FBD and support reaction using superposition;



$$\vec{R_A} = \frac{300x-1050}{L} + 175\cdot\frac{(x-11{,}4)}{L} = \frac{475x-3045}{L}$$

$$\vec{R_B} = \frac{1050-300x}{L} + 300 + 175\left(1-\frac{(x-11{,}4)}{L}\right) = \frac{3045-475x}{L} + 475$$

V: [kN]



$$1 = \frac{475x-3045}{L}$$

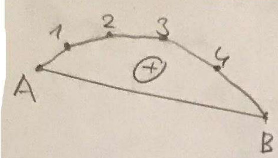
$$2 = \text{〃} - 50$$

$$3 = \text{〃} - 175$$

$$4 = \text{〃} - 300$$

$$5 = \text{〃} - 475$$

M: [kNm]



$1 = \frac{475x-3045}{L}\cdot(L-x) = a$

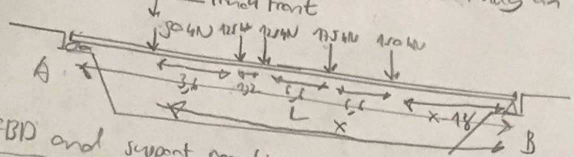
$2 = a + \frac{475x-3045}{L}\cdot(3.6) = b$

$3 = b + \frac{475x-3045}{L}\cdot(1.2) = c$

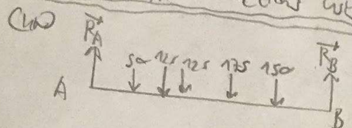
$4 = c + \frac{475x-3045}{L}\cdot(6.6)$

Case 5:

$18 < x < L$ (Truck Fully on Beam)



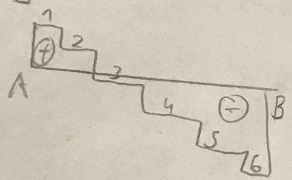
FBD and support reactions using superposition:



$\vec{R_A} = \frac{475x-3045}{L} + 150\cdot\frac{(x-18)}{L} = \frac{625x-5745}{L}$

$\vec{R_B} = \frac{5745-625x}{L} + 625$

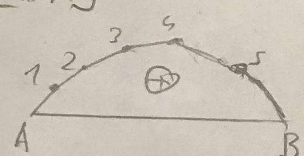
V: [kN]



$1 = \frac{625x-5745}{L} = a$ 　 $3 = a - 175$ 　 $5 = a - 475$

$2 = a - 50$ 　 $4 = a - 300$ 　 $6 = a - 625$

M: [kNm]



$1 = \frac{625x-5745}{L}\cdot(L-x) = b$

$2 = b + \left(\frac{625x-5745}{L} - 50\right)\cdot(3.6) = c$

$3 = c + \left(\frac{625x-5745}{L} - 175\right)\cdot(1.2) = d$

$4 = d + \left(\frac{625x-5745}{L} - 300\right)\cdot(6.6) = e$

$5 = e + \left(\frac{625x-5745}{L} - 475\right)\cdot(6.6)$

* After truck front will pass A, $x > L$ but same pattern applies.

Now this is simulated in MATLAB.

3.8.2 Our MATLAB code for CSA S6-14 CL-625 design truck

```
%%%%% CL-625 TRUCK ON SINGLE BEAM L>18, Lmax = 100 m %%%%%

close all;
clear all;
clc;

L = 26;

Reactions = zeros(10000, 2);
Moment = zeros(10000, 6);
Shear = zeros(10000, 7);

%%%%% INDEXING %%%%%

for i = 2:10000
      Moment(i, 6) = Moment(i - 1, 6) + 0.01;
      Shear(i, 7) = Shear(i - 1, 7) + 0.01;
end

%%%%% PEAK VALUES %%%%%

i = 1;
for x = 0:0.01:(L + 18)
      if (x > 0 && x <= 3.6)
      Reactions(i, 1) = 50 * x / L;
      Reactions(i, 2) = 50 * (1 - x / L);
      Shear(i, 1) = Reactions(i, 1);
      Shear(i, 2) = Reactions(i, 1) - 50;
      Moment(i, 1) = Shear(i, 1) * (L - x);
      elseif (x > 3.6 && x <= 4.8)
      Reactions(i, 1) = (175 * x - 450) / L;
      Reactions(i, 2) = (450 - 175 * x) / L + 175;
      Shear(i, 1) = Reactions(i, 1);
      Shear(i, 2) = Reactions(i, 1) - 50;
      Shear(i, 3) = Reactions(i, 1) - 175;
      Moment(i, 1) = Shear(i, 1) * (L - x);
      Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 3.6;
      elseif (x > 4.8 && x <= 11.4)
      Reactions(i, 1) = (300 * x - 1050) / L;
      Reactions(i, 2) = (1050 - 300 * x) / L + 300;
      Shear(i, 1) = Reactions(i, 1);
```

```
Shear(i, 2) = Reactions(i, 1) - 50;
Shear(i, 3) = Reactions(i, 1) - 175;
Shear(i, 4) = Reactions(i, 1) - 300;
Moment(i, 1) = Shear(i, 1) * (L - x);
Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 3.6;
Moment(i, 3) = Moment(i, 2) + Shear(i, 3) * 1.2;
elseif (x > 11.4 && x <= 18)
Reactions(i, 1) = (475 * x - 3045) / L;
Reactions(i, 2) = (3045 - 475 * x) / L + 475;
Shear(i, 1) = Reactions(i, 1);
Shear(i, 2) = Reactions(i, 1) - 50;
Shear(i, 3) = Reactions(i, 1) - 175;
Shear(i, 4) = Reactions(i, 1) - 300;
Shear(i, 5) = Reactions(i, 1) - 475;
Moment(i, 1) = Shear(i, 1) * (L - x);
Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 3.6;
Moment(i, 3) = Moment(i, 2) + Shear(i, 3) * 1.2;
Moment(i, 4) = Moment(i, 3) + Shear(i, 4) * 6.6;
elseif (x > 18 && x < L)
Reactions(i, 1) = (625 * x - 5745) / L;
Reactions(i, 2) = (5745 - 625 * x) / L + 625;
Shear(i, 1) = Reactions(i, 1);
Shear(i, 2) = Reactions(i, 1) - 50;
Shear(i, 3) = Reactions(i, 1) - 175;
Shear(i, 4) = Reactions(i, 1) - 300;
Shear(i, 5) = Reactions(i, 1) - 475;
Shear(i, 6) = Reactions(i, 1) - 625;
Moment(i, 1) = Shear(i, 1) * (L - x);
Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 3.6;
Moment(i, 3) = Moment(i, 2) + Shear(i, 3) * 1.2;
Moment(i, 4) = Moment(i, 3) + Shear(i, 4) * 6.6;
Moment(i, 5) = Moment(i, 4) + Shear(i, 5) * 6.6;
elseif (x >= L && x < (L + 3.6))
Reactions(i, 1) = (575 * x - 5745) / L;
Reactions(i, 2) = (5745 - 575 * x) / L + 575;
Shear(i, 1) = Reactions(i, 1);
Shear(i, 2) = Reactions(i, 1) - 125;
Shear(i, 3) = Reactions(i, 1) - 250;
Shear(i, 4) = Reactions(i, 1) - 425;
Shear(i, 5) = Reactions(i, 1) - 575;
Moment(i, 1) = Shear(i, 1) * (3.6 - (x - L));
Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 1.2;
Moment(i, 3) = Moment(i, 2) + Shear(i, 3) * 6.6;
Moment(i, 4) = Moment(i, 3) + Shear(i, 4) * 6.6;
```

```
        elseif (x >= (L + 3.6) && x < (L + 4.8))
        Reactions(i, 1) = (450 * x - 5295) / L;
        Reactions(i, 2) = (5295 - 450 * x) / L + 450;
        Shear(i, 1) = Reactions(i, 1);
        Shear(i, 2) = Reactions(i, 1) - 125;
        Shear(i, 3) = Reactions(i, 1) - 300;
        Shear(i, 4) = Reactions(i, 1) - 450;
        Moment(i, 1) = Shear(i, 1) * (L - x + 4.8);
        Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 6.6;
        Moment(i, 3) = Moment(i, 2) + Shear(i, 3) * 6.6;
        elseif (x >= (L + 4.8) && x < (L + 11.4))
        Reactions(i, 1) = (325 * x - 4695) / L;
        Reactions(i, 2) = (4695 - 325 * x) / L + 325;
        Shear(i, 1) = Reactions(i, 1);
        Shear(i, 2) = Reactions(i, 1) - 175;
        Shear(i, 3) = Reactions(i, 1) - 325;
        Moment(i, 1) = Shear(i, 1) * (L - x + 11.4);
        Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 6.6;
        elseif (x >= (L + 11.4) && x < (L + 18))
        Reactions(i, 1) = (150 * x - 2700) / L;
        Reactions(i, 2) = (2700 - 150 * x) / L + 150;
        Shear(i, 1) = Reactions(i, 1);
        Shear(i, 2) = Reactions(i, 1) - 150;
        Moment(i, 1) = Shear(i, 1) * (L - x + 18);
        end
        i = i + 1;
end

v = zeros(L / 0.01 + 1, 1);
M = zeros(L / 0.01 + 1, 1);
y = 0:0.01:L;

Ve = zeros(L / 0.01 + 1, 1);
Me = zeros(L / 0.01 + 1, 1);

%%%%%%%%%%%% SHEAR %%%%%%%%%%%%%

i = 1;
for x = 0:0.01:0
        j = 1;
        for a = 0:0.01:L
        v(j) = 0;
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
```

```
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = 0.01:0.01:3.6
        j = 1;
        for a = 0:0.01:L
        if (a <= x)
              v(j) = - Shear(i, 2);
        else
              v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
              Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = 3.61:0.01:4.8
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 3.6)
              v(j) = - Shear(i, 3);
        elseif (a <= x)
              v(j) = - Shear(i, 2);
        else
```

K & B

```
            v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = 4.81:0.01:11.4
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 4.8)
                v(j) = - Shear(i, 4);
        elseif (a <= x - 3.6)
                v(j) = - Shear(i, 3);
        elseif (a <= x)
                v(j) = - Shear(i, 2);
        else
                v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
```

```
for x = 11.41:0.01:18
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 11.4)
            v(j) = - Shear(i, 5);
      elseif (a <= x - 4.8)
            v(j) = - Shear(i, 4);
      elseif (a <= x - 3.6)
            v(j) = - Shear(i, 3);
      elseif (a <= x)
            v(j) = - Shear(i, 2);
      else
            v(j) = - Shear(i, 1);
      end
      if (abs(v(j)) > abs(Ve(j)))
            Ve(j) = v(j);
      end
      j = j + 1;
      end
      plot(y, v);
      xlim([0 L]);
      ylim([- 625 625]);
      axh = gca; % use current axes
      color = 'k'; % black, or [0 0 0]
      linestyle = '-'; % solid
      line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
      pause(0.01);
      i = i + 1;
end
for x = 18.01:0.01:(L - 0.01)
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 18)
            v(j) = - Shear(i, 6);
      elseif (a <= x - 11.4)
            v(j) = - Shear(i, 5);
      elseif (a <= x - 4.8)
           v(j) = - Shear(i, 4);
      elseif (a <= x - 3.6)
            v(j) = - Shear(i, 3);
      elseif (a <= x)
            v(j) = - Shear(i, 2);
      else
            v(j) = - Shear(i, 1);
```

```
        end
        if (abs(v(j)) > abs(Ve(j)))
               Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = L:0.01:(L + 3.59)
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 18)
              v(j) = - Shear(i, 5);
        elseif (a <= x - 11.4)
              v(j) = - Shear(i, 4);
        elseif (a <= x - 4.8)
              v(j) = - Shear(i, 3);
        elseif (a <= x - 3.6)
              v(j) = - Shear(i, 2);
        else
              v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
              Ve(j) = v(j);
          end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
```

```
end
for x = (L + 3.6):0.01:(L + 4.79)
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 18)
            v(j) = - Shear(i, 4);
      elseif (a <= x - 11.4)
            v(j) = - Shear(i, 3);
          elseif (a <= x - 4.8)
            v(j) = - Shear(i, 2);
      else
            v(j) = - Shear(i, 1);
      end
      if (abs(v(j)) > abs(Ve(j)))
            Ve(j) = v(j);
      end
      j = j + 1;
      end
      plot(y, v);
      xlim([0 L]);
    ylim([- 625 625]);
      axh = gca; % use current axes
      color = 'k'; % black, or [0 0 0]
      linestyle = '-'; % solid
      line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
      pause(0.01);
      i = i + 1;
end
for x = (L + 4.8):0.01:(L + 11.39)
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 18)
            v(j) = - Shear(i, 3);
      elseif (a <= x - 11.4)
            v(j) = - Shear(i, 2);
      else
            v(j) = - Shear(i, 1);
      end
      if (abs(v(j)) > abs(Ve(j)))
            Ve(j) = v(j);
      end
      j = j + 1;
      end
      plot(y, v);
```

```
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = (L + 11.4):0.01:(L + 17.99)
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 18)
                v(j) = - Shear(i, 2);
        else
                v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
      linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = (L + 18):0.01:(L + 18)
        j = 1;
        for a = 0:0.01:L
        v(j) = 0;
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 625 625]);
```

```
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end

%%%%%%%%%%%%%%% MOMENT %%%%%%%%%%%%%

i = 1;
for x = 0:0.01:0
        j = 1;
        for a = 0:0.01:L
        M(j) = 0;
        if (M(j) > Me(j))
                Me(j) = M(j);
        end
          j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 2500]);
        pause(0.01);
        i = i + 1;
end
for x = 0.01:0.01:3.6
        j = 1;
        for a = 0:0.01:L
        if (a <= x)
                M(j) = (a / x) * Moment(i, 1);
        else
                M(j) = (L - a) / (L - x) * Moment(i, 1);
        end
        if (M(j) > Me(j))
                Me(j) = M(j);
        end
        j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 2500]);
        pause(0.01);
        i = i + 1;
```

```
end
for x = 3.61:0.01:4.8
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 3.6)
            M(j) = (a / (x - 3.6)) * Moment(i, 2);
      elseif (a <= x)
            if (Moment(i, 2) > Moment(i, 1))
                  M(j) = ((x - a) / 3.6) * (Moment(i, 2) - Moment(i, 1)) +
Moment(i, 1);
            else
                  M(j) = ((a - (x - 3.6)) / 3.6) * (Moment(i, 1) - Moment(i,
2)) + Moment(i, 2);
            end
      else
            M(j) = (L - a) / (L - x) * Moment(i, 1);
      end
      if (M(j) > Me(j))
         Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
      xlim([0 L]);
      ylim([0 2500]);
      pause(0.01);
      i = i + 1;
end
for x = 4.81:0.01:11.4
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 4.8)
            M(j) = (a / (x - 4.8)) * Moment(i, 3);
      elseif (a <= x - 3.6)
            if (Moment(i, 3) > Moment(i, 2))
                  M(j) = (1.2 - (a - (x - 4.8))) / 1.2 * (Moment(i, 3) -
Moment(i, 2)) + Moment(i, 2);
            else
                  M(j) = (a - (x - 4.8)) / 1.2 * (Moment(i, 2) - Moment(i,
3)) + Moment(i, 3);
            end
      elseif (a <= x)
            if (Moment(i, 2) > Moment(i, 1))
```

```
                M(j) = (3.6 - (a - (x - 4.8 + 1.2))) / 3.6 * (Moment(i, 2)
- Moment(i, 1)) + Moment(i, 1);
            else
                M(j) = (a - (x - 4.8 + 1.2)) / 3.6 * (Moment(i, 1) -
Moment(i, 2)) + Moment(i, 2);
            end
        else
            M(j) = (L - a) / (L - x) * Moment(i, 1);
        end
        if (M(j) > Me(j))
            Me(j) = M(j);
          end
        j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 2500]);
        pause(0.01);
        i = i + 1;
end
for x = 11.41:0.01:18
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 11.4)
            M(j) = (a / (x - 11.4)) * Moment(i, 4);
        elseif (a <= x - 4.8)
            if (Moment(i, 4) > Moment(i, 3))
                M(j) = (6.6 - (a - (x - 11.4))) / 6.6 * (Moment(i, 4) -
Moment(i, 3)) + Moment(i, 3);
            else
                M(j) = (a - (x - 11.4)) / 6.6 * (Moment(i, 3) - Moment(i,
4)) + Moment(i, 4);
            end
        elseif (a <= x - 3.6)
            if (Moment(i, 3) > Moment(i, 2))
                M(j) = (1.2 - (a - (x - 11.4 + 6.6))) / 1.2 * (Moment(i, 3)
- Moment(i, 2)) + Moment(i, 2);
             else
                M(j) = (a - (x - 11.4 + 6.6)) / 1.2 * (Moment(i, 2) -
Moment(i, 3)) + Moment(i, 3);
            end
        elseif (a <= x)
            if (Moment(i, 2) > Moment(i, 1))
```

```
                  M(j) = (3.6 - (a - (x - 11.4 + 1.2 + 6.6))) / 3.6 *
(Moment(i, 2) - Moment(i, 1)) + Moment(i, 1);
            else
                  M(j) = (a - (x - 11.4 + 1.2 + 6.6)) / 3.6 * (Moment(i, 1) -
Moment(i, 2)) + Moment(i, 2);
            end
      else
            M(j) = (L - a) / (L - x) * Moment(i, 1);
      end
      if (M(j) > Me(j))
            Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
      xlim([0 L]);
      ylim([0 2500]);
      pause(0.01);
      i = i + 1;
end
for x = 18.01:0.01:(L - 0.01)
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 18)
            M(j) = (a / (x - 18)) * Moment(i, 5);
      elseif (a <= x - 11.4)
            if (Moment(i, 5) > Moment(i, 4))
                  M(j) = (6.6 - (a - (x - 18))) / 6.6 * (Moment(i, 5) -
Moment(i, 4)) + Moment(i, 4);
            else
                  M(j) = (a - (x - 18)) / 6.6 * (Moment(i, 4) - Moment(i, 5))
+ Moment(i, 5);
            end
      elseif (a <= x - 4.8)
            if (Moment(i, 4) > Moment(i, 3))
                  M(j) = (6.6 - (a - (x - 18 + 6.6))) / 6.6 * (Moment(i, 4) -
Moment(i, 3)) + Moment(i, 3);
            else
                  M(j) = (a - (x - 18 + 6.6)) / 6.6 * (Moment(i, 3) -
Moment(i, 4)) + Moment(i, 4);
            end
      elseif (a <= x - 3.6)
            if (Moment(i, 3) > Moment(i, 2))
```

```
                M(j) = (1.2 - (a - (x - 18 + 6.6 + 6.6))) / 1.2 *
(Moment(i, 3) - Moment(i, 2)) + Moment(i, 2);
            else
                M(j) = (a - (x - 18 + 6.6 + 6.6)) / 1.2 * (Moment(i, 2) -
Moment(i, 3)) + Moment(i, 3);
           end
        elseif (a <= x)
            if (Moment(i, 2) > Moment(i, 1))
                M(j) = (3.6 - (a - (x - 18 + 1.2 + 6.6 + 6.6))) / 3.6 *
(Moment(i, 2) - Moment(i, 1)) + Moment(i, 1);
            else
                M(j) = (a - (x - 18 + 1.2 + 6.6 + 6.6)) / 3.6 * (Moment(i,
1) - Moment(i, 2)) + Moment(i, 2);
            end
        else
            M(j) = (L - a) / (L - x) * Moment(i, 1);
        end
        if (M(j) > Me(j))
            Me(j) = M(j);
        end
        j = j + 1;
        end
      plot(y, M);
        xlim([0 L]);
        ylim([0 2500]);
        pause(0.01);
        i = i + 1;
end
for x = L:0.01:(L + 3.59)
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 18)
            M(j) = (a / (x - 18)) * Moment(i, 4);
        elseif (a <= x - 11.4)
             if (Moment(i, 4) > Moment(i, 3))
                M(j) = (6.6 - (a - (x - 18))) / 6.6 * (Moment(i, 4) -
Moment(i, 3)) + Moment(i, 3);
            else
                M(j) = (a - (x - 18)) / 6.6 * (Moment(i, 3) - Moment(i, 4))
+ Moment(i, 4);
            end
        elseif (a <= x - 4.8)
            if (Moment(i, 3) > Moment(i, 2))
```

```
                  M(j) = (6.6 - (a - (x - 18 + 6.6))) / 6.6 * (Moment(i, 3) -
Moment(i, 2)) + Moment(i, 2);
            else
                  M(j) = (a - (x - 18 + 6.6)) / 6.6 * (Moment(i, 2) -
Moment(i, 3)) + Moment(i, 3);
            end
      elseif (a <= x - 3.6)
            if (Moment(i, 2) > Moment(i, 1))
                  M(j) = (1.2 - (a - (x - 18 + 6.6 + 6.6))) / 1.2 *
(Moment(i, 2) - Moment(i, 1)) + Moment(i, 1);
            else
                  M(j) = (a - (x - 18 + 6.6 + 6.6)) / 1.2 * (Moment(i, 1) -
Moment(i, 2)) + Moment(i, 2);
            end
      else
            M(j) = ((L - (14.4 + (x - 18))) - (a - (14.4 + (x - 18)))) / (L -
(14.4 + (x - 18))) * Moment(i, 1);
      end
      if (M(j) > Me(j))
            Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
      xlim([0 L]);
      ylim([0 2500]);
      pause(0.01);
      i = i + 1;
end
for x = (L + 3.6):0.01:(L + 4.79)
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 18)
            M(j) = (a / (x - 18)) * Moment(i, 3);
      elseif (a <= x - 11.4)
            if (Moment(i, 3) > Moment(i, 2))
                  M(j) = (6.6 - (a - (x - 18))) / 6.6 * (Moment(i, 3) -
Moment(i, 2)) + Moment(i, 2);
            else
                  M(j) = (a - (x - 18)) / 6.6 * (Moment(i, 2) - Moment(i, 3))
+ Moment(i, 3);
            end
      elseif (a <= x - 4.8)
            if (Moment(i, 2) > Moment(i, 1))
```

```
                  M(j) = (6.6 - (a - (x - 18 + 6.6))) / 6.6 * (Moment(i, 2) -
Moment(i, 1)) + Moment(i, 1);
            else
                  M(j) = (a - (x - 18 + 6.6)) / 6.6 * (Moment(i, 1) -
Moment(i, 2)) + Moment(i, 2);
            end
      else
            M(j) = ((L - (13.2 + (x - 18))) - (a - (13.2 + (x - 18)))) / (L -
(13.2 + (x - 18))) * Moment(i, 1);
      end
      if (M(j) > Me(j))
            Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
      xlim([0 L]);
      ylim([0 2500]);
      pause(0.01);
      i = i + 1;
end
for x = (L + 4.8):0.01:(L + 11.39)
      j = 1;
      for a = 0:0.01:L
      if (a <= x - 18)
            M(j) = (a / (x - 18)) * Moment(i, 2);
      elseif (a <= x - 11.4)
            if (Moment(i, 2) > Moment(i, 1))
                  M(j) = (6.6 - (a - (x - 18))) / 6.6 * (Moment(i, 2) -
Moment(i, 1)) + Moment(i, 1);
            else
                  M(j) = (a - (x - 18)) / 6.6 * (Moment(i, 1) - Moment(i, 2))
+ Moment(i, 2);
            end
      else
            M(j) = ((L - (6.6 + (x - 18))) - (a - (6.6 + (x - 18)))) / (L -
(6.6 + (x - 18))) * Moment(i, 1);
      end
      if (M(j) > Me(j))
            Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
```

```
        xlim([0 L]);
        ylim([0 2500]);
        pause(0.01);
        i = i + 1;
end
for x = (L + 11.4):0.01:(L + 17.99)
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 18)
              M(j) = (a / (x - 18)) * Moment(i, 1);
        else
              M(j) = ((L - (x - 18)) - (a - (x - 18))) / (L - (x - 18)) *
Moment(i, 1);
        end
        if (M(j) > Me(j))
              Me(j) = M(j);
        end
        j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 2500]);
        pause(0.01);
        i = i + 1;
end
for x = (L + 18):0.01:(L + 18)
        j = 1;
        for a = 0:0.01:L
        M(j) = 0;
        if (M(j) > Me(j))
              Me(j) = M(j);
        end
        j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 2500]);
        pause(0.01);
        i = i + 1;
end

subplot(2, 1, 1);
plot(y, Ve);
axh = gca; % use current axes
```

```
color = 'k'; % black, or [0 0 0]
linestyle = '-'; % solid
line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);

subplot(2, 1, 2);
plot(y, Me);
pause(5);

%%%%% TRUCK MOVING FROM RIGHT TO LEFT %%%%%

temp1 = zeros((L / 0.01 + 1), 1);
temp2 = zeros((L / 0.01 + 1), 1);

for i = 1:(L / 0.01 + 1)
      temp1(i) = Ve(i);
      temp2(i) = Me(i);
end
for i = 1:(L / 0.01 + 1)
      Ve(i) = temp1((L / 0.01 + 1) - i + 1);
      Me(i) = temp2((L / 0.01 + 1) - i + 1);
end

subplot(2, 1, 1);
plot(y, Ve);
axh = gca; % use current axes
color = 'k'; % black, or [0 0 0]
linestyle = '-'; % solid
line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);

subplot(2, 1, 2);
plot(y, Me);
pause(5);

%%%%% COMBINED %%%%%

for i = 1:(L / 0.01 + 1)
      if (abs(temp1(i)) > abs(Ve(i)))
      Ve(i) = abs(temp1(i));
      else
      Ve(i) = abs(Ve(i));
      end
      if (abs(temp2(i)) > abs(Me(i)))
      Me(i) = abs(temp2(i));
      else
```

```
        Me(i) = abs(Me(i));
        end
end

subplot(2, 1, 1);
plot(y, Ve);

subplot(2, 1, 2);
plot(y, Me);
pause(5);

%%%%% FOR GEOGEBRA PLOTTING %%%%%

%{
outputfile = fopen('output_CSA_S6_14.txt', 'wt');

j = 1;

for i = 0:0.01:L

      if i == 0
          fprintf(outputfile, '{(%f,%f),', i, Me(j));
      j = j + 1;
      continue;
      end

      if i == L
      fprintf(outputfile, '(%f,%f)}', i, Me(j));
      break;
      end

      fprintf(outputfile, '(%f,%f),', i, Me(j));

      j = j + 1;

end

fclose(outputfile);
%}
```

3.8.3 Our MATLAB code for AASHTO & CSA S6-66 CL-325 design truck

```
%%%%% CL-325 TRUCK (AASHTO) ON SINGLE BEAM L>(4.3+RearAxleSpacing or Rs),
Lmax = 100 m %%%%%
```

```
close all;
clear all;
clc;

L = 26;
Rs = 4.3; %RearAxleSpacing between 4.3 m and 9 m

Reactions = zeros(10000, 2);
Moment = zeros(10000, 4);
Shear = zeros(10000, 5);

%%%%% INDEXING %%%%%

for i = 2:10000
      Moment(i, 4) = Moment(i - 1, 4) + 0.01;
      Shear(i, 5) = Shear(i - 1, 5) + 0.01;
end

%%%%% PEAK VALUES %%%%%

i = 1;
for x = 0:0.01:(L + 4.3 + Rs + 0.001)
      if (x > 0 && x <= 4.3)
      Reactions(i, 1) = 35 * x / L;
      Reactions(i, 2) = 35 - Reactions(i, 1);
      Shear(i, 1) = Reactions(i, 1);
      Shear(i, 2) = Reactions(i, 1) - 35;
      Moment(i, 1) = Shear(i, 1) * (L - x);
      elseif (x > 4.3 && x < (4.3 + Rs + 0.001))
      Reactions(i, 1) = 35 * x / L + 145 * (x - 4.3) / L;
      Reactions(i, 2) = 180 - Reactions(i, 1);
      Shear(i, 1) = Reactions(i, 1);
      Shear(i, 2) = Reactions(i, 1) - 35;
      Shear(i, 3) = Reactions(i, 1) - 180;
      Moment(i, 1) = Shear(i, 1) * (L - x);
      Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 4.3;
      elseif (x > (Rs + 4.3) && x < L)
      Reactions(i, 1) = 35 * x / L + 145 * (x - 4.3) / L + 145 * (x - (4.3 +
Rs)) / L;
      Reactions(i, 2) = 325 - Reactions(i, 1);
      Shear(i, 1) = Reactions(i, 1);
      Shear(i, 2) = Reactions(i, 1) - 35;
      Shear(i, 3) = Reactions(i, 1) - 180;
```

```
        Shear(i, 4) = Reactions(i, 1) - 325;
        Moment(i, 1) = Shear(i, 1) * (L - x);
        Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * 4.3;
        Moment(i, 3) = Moment(i, 2) + Shear(i, 3) * Rs;
        elseif (x >= L && x < (L + 4.3))
        Reactions(i, 1) = 145 * (x - 4.3) / L + 145 * (x - (4.3 + Rs)) / L;
        Reactions(i, 2) = 290 - Reactions(i, 1);
        Shear(i, 1) = Reactions(i, 1);
        Shear(i, 2) = Reactions(i, 1) - 145;
        Shear(i, 3) = Reactions(i, 1) - 290;
        Moment(i, 1) = Shear(i, 1) * (L - x + 4.3);
        Moment(i, 2) = Moment(i, 1) + Shear(i, 2) * Rs;
        elseif (x >= (L + 4.3) && x < (L + 4.3 + Rs))
        Reactions(i, 1) = 145 * (x - (4.3 + Rs)) / L;
        Reactions(i, 2) = 145 - Reactions(i, 1);
        Shear(i, 1) = Reactions(i, 1);
        Shear(i, 2) = Reactions(i, 1) - 145;
        Moment(i, 1) = Shear(i, 1) * (L - x + 4.3 + Rs);
        end
        i = i + 1;
end

v = zeros(L / 0.01 + 1, 1);
M = zeros(L / 0.01 + 1, 1);
y = 0:0.01:L;

Ve = zeros(L / 0.01 + 1, 1);
Me = zeros(L / 0.01 + 1, 1);

%%%%%%%%%%%%% SHEAR %%%%%%%%%%%%%%

i = 1;
for x = 0:0.01:0
        j = 1;
        for a = 0:0.01:L
        v(j) = 0;
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 350 350]);
```

```
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = 0.01:0.01:4.3
        j = 1;
        for a = 0:0.01:L
        if (a <= x)
                v(j) = - Shear(i, 2);
        else
                v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 350 350]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = 4.31:0.01:(4.3 + Rs + 0.001)
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 4.3)
                v(j) = - Shear(i, 3);
        elseif (a <= x)
                v(j) = - Shear(i, 2);
        else
                v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
```

```
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 350 350]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = (4.3 + Rs + 0.01):0.01:(L - 0.01 + 0.001)
        j = 1;
        for a = 0:0.01:L
        if (a < x - (4.3 + Rs + 0.001))
                v(j) = - Shear(i, 4);
        elseif (a <= x - 4.3)
                v(j) = - Shear(i, 3);
        elseif (a <= x)
                v(j) = - Shear(i, 2);
        else
                v(j) = - Shear(i, 1);
        end
        if (abs(v(j)) > abs(Ve(j)))
                Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 350 350]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end
for x = L:0.01:(L + 4.29)
        j = 1;
        for a = 0:0.01:L
        if (a < x - (4.3 + Rs + 0.001))
                v(j) = - Shear(i, 3);
        elseif (a <= x - 4.3)
```

```
            v(j) = - Shear(i, 2);
      else
            v(j) = - Shear(i, 1);
      end
      if (abs(v(j)) > abs(Ve(j)))
            Ve(j) = v(j);
      end
      j = j + 1;
      end
      plot(y, v);
      xlim([0 L]);
      ylim([- 350 350]);
      axh = gca; % use current axes
      color = 'k'; % black, or [0 0 0]
      linestyle = '-'; % solid
      line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
      pause(0.01);
      i = i + 1;
end
for x = (L + 4.3):0.01:(L + 4.3 + Rs - 0.01 + 0.001)
      j = 1;
      for a = 0:0.01:L
      if (a < x - (4.3 + Rs + 0.001))
            v(j) = - Shear(i, 2);
      else
            v(j) = - Shear(i, 1);
      end
      if (abs(v(j)) > abs(Ve(j)))
            Ve(j) = v(j);
      end
      j = j + 1;
    end
      plot(y, v);
      xlim([0 L]);
      ylim([- 350 350]);
      axh = gca; % use current axes
      color = 'k'; % black, or [0 0 0]
      linestyle = '-'; % solid
      line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
      pause(0.01);
    i = i + 1;
end
for x = (L + 4.3 + Rs):0.01:(L + 4.3 + Rs)
      j = 1;
```

```
        for a = 0:0.01:L
        v(j) = 0;
        if (abs(v(j)) > abs(Ve(j)))
              Ve(j) = v(j);
        end
        j = j + 1;
        end
        plot(y, v);
        xlim([0 L]);
        ylim([- 350 350]);
        axh = gca; % use current axes
        color = 'k'; % black, or [0 0 0]
        linestyle = '-'; % solid
        line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);
        pause(0.01);
        i = i + 1;
end

%%%%%%%%%%%%%%% MOMENT %%%%%%%%%%%%%

i = 1;
for x = 0:0.01:0
        j = 1;
        for a = 0:0.01:L
        M(j) = 0;
        if (M(j) > Me(j))
              Me(j) = M(j);
        end
        j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 1750]);
        pause(0.01);
        i = i + 1;
end
for x = 0.01:0.01:4.3
        j = 1;
        for a = 0:0.01:L
        if (a <= x)
              M(j) = (a / x) * Moment(i, 1);
        else
              M(j) = (L - a) / (L - x) * Moment(i, 1);
        end
```

```
        if (M(j) > Me(j))
              Me(j) = M(j);
            end
        j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 1750]);
        pause(0.01);
        i = i + 1;
end
for x = 4.31:0.01:(4.3 + Rs)
        j = 1;
        for a = 0:0.01:L
        if (a <= x - 4.3)
              M(j) = (a / (x - 4.3)) * Moment(i, 2);
        elseif (a <= x)
              if (Moment(i, 2) > Moment(i, 1))
                    M(j) = ((x - a) / 4.3) * (Moment(i, 2) - Moment(i, 1)) +
Moment(i, 1);
              else
                    M(j) = ((a - (x - 4.3)) / 4.3) * (Moment(i, 1) - Moment(i,
2)) + Moment(i, 2);
              end
        else
              M(j) = (L - a) / (L - x) * Moment(i, 1);
        end
        if (M(j) > Me(j))
              Me(j) = M(j);
        end
        j = j + 1;
        end
        plot(y, M);
        xlim([0 L]);
        ylim([0 1750]);
        pause(0.01);
        i = i + 1;
end
for x = (4.3 + Rs + 0.01):0.01:(L - 0.01)
        j = 1;
        for a = 0:0.01:L
        if (a < x - (4.3 + Rs + 0.001))
              M(j) = (a / (x - (4.3 + Rs))) * Moment(i, 3);
        elseif (a <= x - 4.3)
```

```
            if (Moment(i, 3) > Moment(i, 2))
                  M(j) = (Rs - (a - (x - (4.3 + Rs)))) / Rs * (Moment(i, 3) -
Moment(i, 2)) + Moment(i, 2);
            else
                  M(j) = (a - (x - (4.3 + Rs))) / Rs * (Moment(i, 2) -
Moment(i, 3)) + Moment(i, 3);
            end
      elseif (a <= x)
            if (Moment(i, 2) > Moment(i, 1))
                  M(j) = (4.3 - (a - (x - (4.3 + Rs) + Rs))) / 4.3 *
(Moment(i, 2) - Moment(i, 1)) + Moment(i, 1);
            else
                  M(j) = (a - (x - (4.3 + Rs) + Rs)) / 4.3 * (Moment(i, 1) -
Moment(i, 2)) + Moment(i, 2);
            end
      else
            M(j) = (L - a) / (L - x) * Moment(i, 1);
      end
      if (M(j) > Me(j))
            Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
      xlim([0 L]);
      ylim([0 1750]);
      pause(0.01);
      i = i + 1;
end
for x = L:0.01:(L + 4.29)
      j = 1;
      for a = 0:0.01:L
      if (a < x - (4.3 + Rs + 0.001))
            M(j) = (a / (x - (4.3 + Rs))) * Moment(i, 2);
      elseif (a <= x - 4.3)
            if (Moment(i, 2) > Moment(i, 1))
                  M(j) = (Rs - (a - (x - (4.3 + Rs)))) / Rs * (Moment(i, 2) -
Moment(i, 1)) + Moment(i, 1);
            else
                  M(j) = (a - (x - (4.3 + Rs))) / Rs * (Moment(i, 1) -
Moment(i, 2)) + Moment(i, 2);
            end
      else
```

```
            M(j) = ((L - (Rs + (x - (4.3 + Rs)))) - (a - (Rs + (x - (4.3 +
Rs))))) / (L - (Rs + (x - (4.3 + Rs)))) * Moment(i, 1);
      end
      if (M(j) > Me(j))
            Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
      xlim([0 L]);
      ylim([0 1750]);
      pause(0.01);
      i = i + 1;
end
for x = (L + 4.3):0.01:(L + 4.3 + Rs - 0.01)
      j = 1;
      for a = 0:0.01:L
      if (a < x - (4.3 + Rs + 0.001))
            M(j) = (a / (x - (4.3 + Rs))) * Moment(i, 1);
      else
            M(j) = ((L - (x - (4.3 + Rs))) - (a - (x - (4.3 + Rs)))) / (L -
(x - (4.3 + Rs))) * Moment(i, 1);
      end
      if (M(j) > Me(j))
            Me(j) = M(j);
        end
      j = j + 1;
      end
      plot(y, M);
      xlim([0 L]);
      ylim([0 1750]);
      pause(0.01);
      i = i + 1;
end
for x = (L + 4.3 + Rs):0.01:(L + 4.3 + Rs)
      j = 1;
      for a = 0:0.01:L
      M(j) = 0;
      if (M(j) > Me(j))
              Me(j) = M(j);
      end
      j = j + 1;
      end
      plot(y, M);
```

```
      xlim([0 L]);
      ylim([0 1750]);
      pause(0.01);
      i = i + 1;
end

subplot(2, 1, 1);
plot(y, Ve);
axh = gca; % use current axes
color = 'k'; % black, or [0 0 0]
linestyle = '-'; % solid
line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);

subplot(2, 1, 2);
plot(y, Me);
pause(5);

%%%%% TRUCK MOVING FROM RIGHT TO LEFT %%%%%

temp1 = zeros((L / 0.01 + 1), 1);
temp2 = zeros((L / 0.01 + 1), 1);

for i = 1:(L / 0.01 + 1)
      temp1(i) = Ve(i);
      temp2(i) = Me(i);
end
for i = 1:(L / 0.01 + 1)
      Ve(i) = temp1((L / 0.01 + 1) - i + 1);
      Me(i) = temp2((L / 0.01 + 1) - i + 1);
end

subplot(2, 1, 1);
plot(y, Ve);
axh = gca; % use current axes
color = 'k'; % black, or [0 0 0]
linestyle = '-'; % solid
line(get(axh, 'XLim'), [0 0], 'Color', color, 'LineStyle', linestyle);

subplot(2, 1, 2);
plot(y, Me);
pause(5);

%%%%% COMBINED %%%%%
```

```
for i = 1:(L / 0.01 + 1)
      if (abs(temp1(i)) > abs(Ve(i)))
      Ve(i) = abs(temp1(i));
      else
      Ve(i) = abs(Ve(i));
      end
      if (abs(temp2(i)) > abs(Me(i)))
      Me(i) = abs(temp2(i));
      else
      Me(i) = abs(Me(i));
      end
end

subplot(2, 1, 1);
plot(y, Ve);

subplot(2, 1, 2);
plot(y, Me);
pause(5);

%%%%% FOR GEOGEBRA PLOTTING %%%%%

%{
outputfile = fopen('output_AASHTO_2014_17.txt', 'wt');
 j = 1;

for i = 0:0.01:L
      if i == 0
      fprintf(outputfile, '{(%f,%f),', i, Me(j));
      j = j + 1;
       continue;
      end
 if i == L
      fprintf(outputfile, '(%f,%f)}', i, Me(j));
      break;
      end
      fprintf(outputfile, '(%f,%f),', i, Me(j));
      j = j + 1;
end

fclose(outputfile);
%}
```

UNIVERSITY OF TORONTO

Chapter 4 - Design Loads

Table of Contents

K & B

4.1 Introduction

This chapter of the report shows the detailed design loads for our bridge. Design Loads presented here are for the three codes this report considers: CSA S6-14, AASHTO LRFD 2014-17 and CSA S6-66. The design loads considered here are the dead load (self-weight of the structural elements), superimposed dead load (load that comes from asphalt and waterproofing, barrier walls), live load (truck load + lane load). Our bridge deck is determined to be 200 mm in thickness and 65 mm of asphalt and waterproofing is used. For live load calculations, truck loads are obtained from data presented in the previous chapter. However, that data is multiplied with certain factors to best replicate the load each girder is taking in the real world. Lane loading is calculated in this chapter and is necessary to represent the high traffic volume situations. To be able to start doing load assessment, the geometry of girders used is determined. For this, the equations from design codes as well as from the book named "Prestressed Concrete Structures" written by Michael P. Collins D. Mitchell is used. Girder section properties is essential in the following steps of the design.

4.2 Prestressed Girder Geometry

According to Michael P. Collins and Dennis Mitchell's prestressed concrete textbook, depth-to-span ratio for an I-Girder is given by the following equation [1]:

$$\frac{L}{h} \leq 18$$

$where:$

$L = Unsupported\ Span\ Length\ [m]$

$h = Girder\ Height\ [m]$

Our unsupported span length is 26 m. According to this equation, we need at least 1.44 m girder height (depth).

Prestressed Concrete Institution (PCI) provides the following preliminary design chart which includes several type of girders:

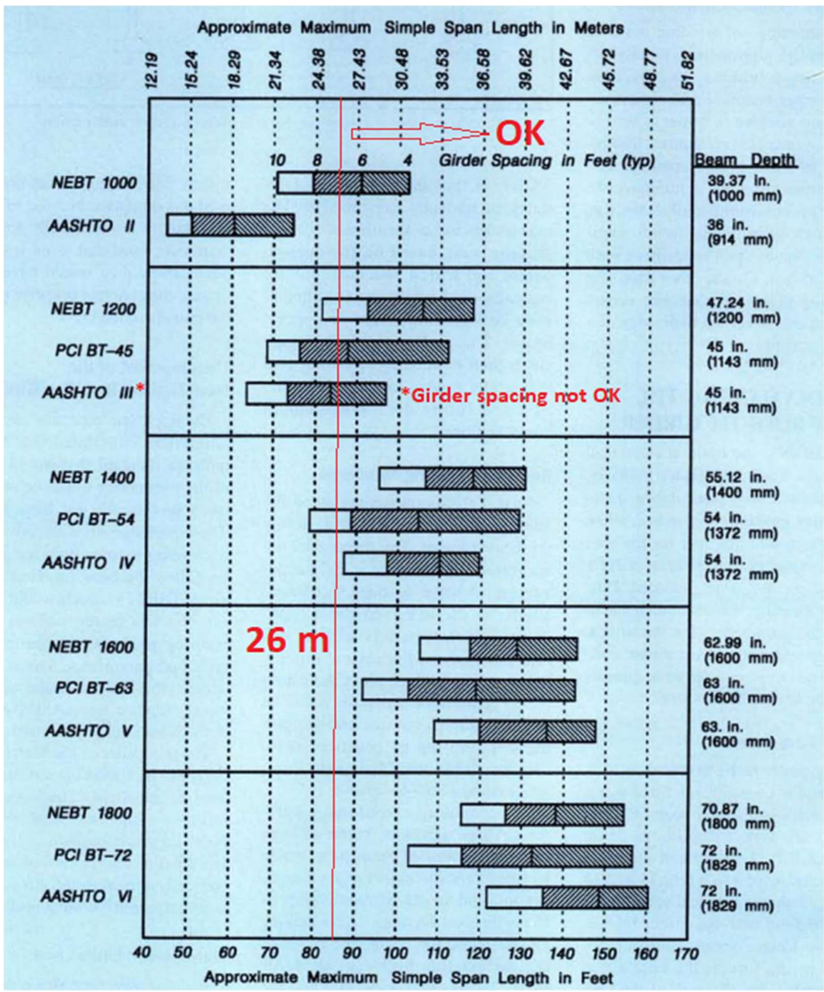


Figure 4.2.1 - Approximate Maximum Simple Span Length of Different Girders [2]

AASHTO provides a span-to-depth ratio of 0.045 in table 2.5.2.6.3-1 for prestressed I Girders. Our Span is 26 m so we need at least 1.17 m depth.

Looking at these three sources, the section chosen is “AASHTO Type IV” girder:

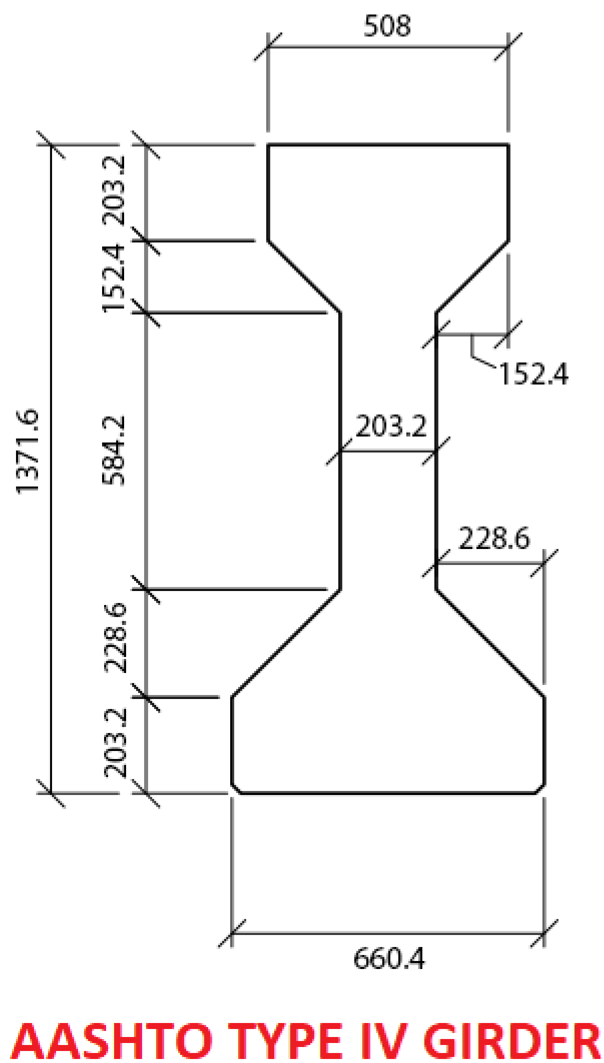


Figure 4.2.2 - AASHTO Type IV Girder Dimensions in mm

Table 4.2.1 - Section Properties for AASHTO Type IV Girder

Height [mm]	1371.6
Gross Area [mm2]	509031
c top [mm]	743.36
c bottom [mm]	628.24
Moment of Inertia (I) [mm4]	108530000000
s top [mm3]	145999148
s bottom [mm3]	172752589

4.3 Design Loads - CSA S6-14

4.3.1 Dead Load

Table 4.3.1.1 - Unit Weights for materials of Interest as given in CSA S6-14 rev. 17 [3]

Component	Unit Weight (kN/m3)
Asphalt and Waterproofing (Biteminous Wearing Surfaces)	23.5
Deck (Reinforced Concrete)	24
AASHTO Girders (Prestressed Concrete)	24.5

While doing dead load calculations, weight of barrier walls is ignored, and it is assumed that each beam takes ¼ of the load coming from 200 mm deck and 65 mm asphalt and waterproofing material on top. Secondary beams are also ignored. A tributary unit length of 1 m into the paper is assumed in calculations so at the end, results obtained have the unit kN/m.

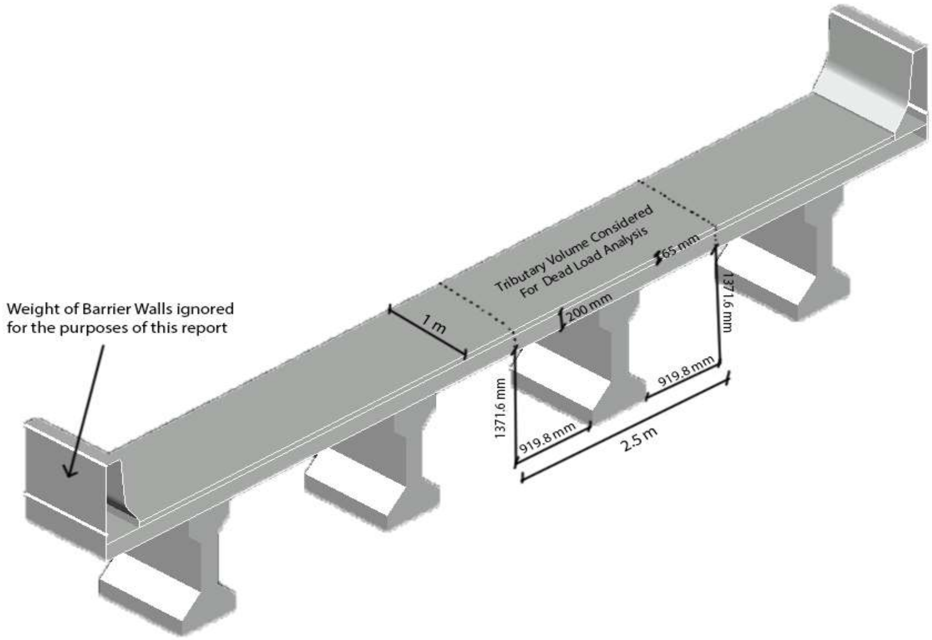


Figure 4.3.1.1 - Dead Load Analysis Model

Therefore, Dead Load (DL) per girder is sum of these:

$$DL_{Deck} = \frac{1\,m \times 10\,m \times 0.2\,m}{4 \times 1\,m} \times \frac{24\,kN}{m^3} = 12\,\frac{kN}{m}$$

$$DL_{Girder} = \frac{1\,m \times 0.50903124\,m^2}{1\,m} \times \frac{24.5\,kN}{m^3} = 12.47\,\frac{kN}{m}$$

$$DL_{Asph.+WProof.} = \frac{1\,m \times 0.065\,m \times 10\,m}{4 \times 1\,m} \times \frac{23.5\,kN}{m^3} = 3.82\,\frac{kN}{m}$$

$$DL_{per\ Girder} = 12 + 12.47 + 3.82 = 28.29\,\frac{kN}{m}$$

Table 4.3.1.2 - Unfactored Absolute Moment and Shear Values due to Dead Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	367.77	0
0.5	353.63	180.35
1	339.48	353.63
1.5	325.34	519.83
2	311.19	678.96
2.5	297.05	831.02
3	282.90	976.01
3.5	268.76	1113.92
4	254.61	1244.76
4.5	240.47	1368.53
5	226.32	1485.23
5.5	212.18	1594.85
6	198.03	1697.40
6.5	183.89	1792.88
7	169.74	1881.29
7.5	155.60	1962.62
8	141.45	2036.88
8.5	127.31	2104.07
9	113.16	2164.19
9.5	99.02	2217.23
10	84.87	2263.20
10.5	70.73	2302.10
11	56.58	2333.93
11.5	42.44	2358.68
12	28.29	2376.36
12.5	14.15	2386.97
13	0.00	2390.51
13.5	14.15	2386.97
14	28.29	2376.36
14.5	42.44	2358.68
15	56.58	2333.93
15.5	70.73	2302.10
16	84.87	2263.20
16.5	99.02	2217.23
17	113.16	2164.19
17.5	127.31	2104.07
18	141.45	2036.88
18.5	155.60	1962.62
19	169.74	1881.29
19.5	183.89	1792.88
20	198.03	1697.40
20.5	212.18	1594.85
21	226.32	1485.23
21.5	240.47	1368.53
22	254.61	1244.76
22.5	268.76	1113.92
23	282.90	976.01
23.5	297.05	831.02
24	311.19	678.96
24.5	325.34	519.83
25	339.48	353.63
25.5	353.63	180.35
26	367.77	0

4.3.2 Live Load

In this section, live load (truck and lane loading) will be analysed according to CSA S6-14.

> Truck Loading:
Unfactored and undistributed truck loads are obtained from chapter 3.

> Lane Loading:
Lane load is calculated by superimposing 0.8 times truck load and a uniform distributed load of 9 kN/m acting on the 26-meter bridge.

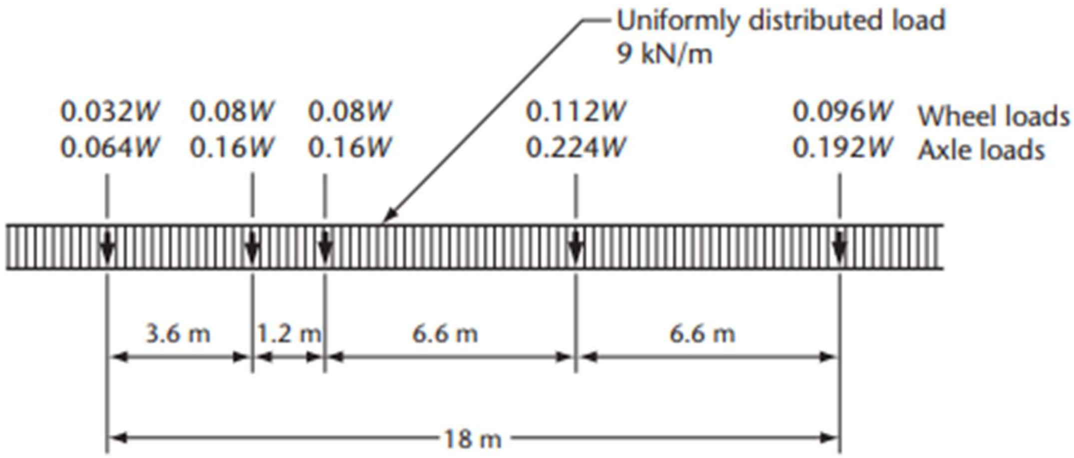


Figure 4.3.2.1 - CL-W Lane Load loading details [3]

Table 4.3.2.1 (left) - Undistributed Absolute Maximum Moment and Shear values for Truck Load (obtained from chapter 3)

Table 4.3.2.1 (right) - Undistributed Absolute Max. Moment and Shear values for Lane Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	433.43	0
0.5	422.38	211.30
1	411.32	411.54
1.5	400.26	600.72
2	389.20	778.85
2.5	378.14	945.91
3	367.09	1101.92
3.5	356.03	1246.88
4	344.18	1397.69
4.5	332.16	1540.82
5	320.14	1671.92
5.5	308.13	1791.01
6	296.11	1898.08
6.5	284.09	1993.13
7	272.07	2076.15
7.5	260.05	2153.51
8	248.03	2226.92
8.5	236.01	2288.32
9	223.99	2337.69
9.5	211.97	2375.05
10	199.95	2400.38
10.5	187.93	2413.70
11	175.91	2415.00
11.5	163.89	2408.03
12	154.24	2411.54
12.5	145.11	2403.03
13	135.97	2382.50
13.5	145.11	2403.03
14	154.24	2411.54
14.5	163.89	2408.03
15	175.91	2415.00
15.5	187.93	2413.70
16	199.95	2400.38
16.5	211.97	2375.05
17	223.99	2337.69
17.5	236.01	2288.32
18	248.03	2226.92
18.5	260.05	2153.51
19	272.07	2076.15
19.5	284.09	1993.13
20	296.11	1898.08
20.5	308.13	1791.01
21	320.14	1671.92
21.5	332.16	1540.82
22	344.18	1397.69
22.5	356.03	1246.88
23	367.09	1101.92
23.5	378.14	945.91
24	389.20	778.85
24.5	400.26	600.72
25	411.32	411.54
25.5	422.38	211.30
26	433.43	0

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	463.75	0
0.5	450.40	226.41
1	437.05	441.73
1.5	423.71	645.95
2	410.36	839.08
2.5	397.02	1021.11
3	383.67	1192.04
3.5	370.32	1351.88
4	356.35	1514.15
4.5	342.23	1668.03
5	328.12	1810.04
5.5	314.00	1940.18
6	299.88	2058.46
6.5	285.77	2164.88
7	271.65	2259.42
7.5	257.54	2347.18
8	243.42	2429.54
8.5	229.31	2500.03
9	215.19	2558.65
9.5	201.08	2605.41
10	186.96	2640.31
10.5	172.85	2663.34
11	158.73	2674.50
11.5	144.62	2676.80
12	132.39	2685.23
12.5	120.58	2681.80
13	108.78	2666.50
13.5	120.58	2681.80
14	132.39	2685.23
14.5	144.62	2676.80
15	158.73	2674.50
15.5	172.85	2663.34
16	186.96	2640.31
16.5	201.08	2605.41
17	215.19	2558.65
17.5	229.31	2500.03
18	243.42	2429.54
18.5	257.54	2347.18
19	271.65	2259.42
19.5	285.77	2164.88
20	299.88	2058.46
20.5	314.00	1940.18
21	328.12	1810.04
21.5	342.23	1668.03
22	356.35	1514.15
22.5	370.32	1351.88
23	383.67	1192.04
23.5	397.02	1021.11
24	410.36	839.08
24.5	423.71	645.95
25	437.05	441.73
25.5	450.40	226.41
26	463.75	0

> Using the simplified longitudinal design method:
Distribution of the live load per girder can be calculated using the equations in clause 5.6.4 [3]

$$V_L = F_T F_S V_T$$

$$M_L = F_T F_S M_T$$

$V_L = Longitudinal\ shear\ per\ girder$

$M_L = Longitudinal\ moment\ per\ girder$

$F_T = Truck\ Load\ Fraction$

$F_S = Skew\ factor\ = 1\ (non-skewed\ bridge)$

$V_T = Logitudinal\ shear\ generated\ by\ one\ design\ lane\ of\ loading$

$M_T = Logitudinal\ moment\ generated\ by\ one\ design\ lane\ of\ loading$

>Known Data:
Girder spacing for the bridge is 2.5 m.
Number of girders supporting the bridge is 4.
Type A highway with unsupported span length of 26 m.
Bridge deck width is 10 m.

Number of design lanes can be obtained from table 3.5 [3]:

Table 3.5
Number of design lanes
(See Clause 3.8.2.)

Deck width, W_c, m	n
6.0 or less	1
Over 6.0 to 10.0	2
Over 10.0 to 13.5	2 or 3*
Over 13.5 to 17.0	4
Over 17.0 to 20.5	5
Over 20.5 to 24.0	6
Over 24.0 to 27.5	7
Over 27.5	8

*Both should be checked.

Since the bridge deck width is 10 m, the number of design lanes is **2**.

$$Lane\ Width\ W_e = \frac{Deck\ Width\ W_c}{No.\ of\ lanes\ n} = \frac{10}{2} = 5$$

(Clause 3.8.2 [3])

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Lane Modification factor $\mu = \frac{W_e - 3.3}{0.6} = \frac{17}{6}$. However this value has to be ≤ 1.

Therefore it is 1.

(Clause 5.6.4.4 [3])

Modification factor for multi lane loading can be obtained from table 3.6 [3]:

Table 3.6
Modification factor for multi-lane loading
(See Clause 3.8.4.2.)

Number of loaded design lanes	Modification factor
1	1.00
2	0.90
3	0.80
4	0.70
5	0.60
6 or more	0.55

2 design lanes, therefore a modification factor of **0.9** is applicable.

(These factors are there since the probability of having multiple lanes loaded fully is no t probable.)

DT_V, DT_M, λ_V, λ_M and γ_M can be calculated or obtained using table 5.3 [3]:

Table 5.3
Factors D_T, λ, γ_c, and γ_e for slab-on-girder bridges for Class A and B highway
(See Clauses 5.6.6.1 and 5.6.7.1.)

Condition	Load effect	n	D_T	λ	γ_c	γ_e
ULS and SLS	Moment interior	≥ 2	$4.60 - \frac{5.30}{\sqrt{L_e + 5}} \geq 2.80$	$0.10 - \frac{0.25}{L_e}$	1.0	Not applicable
	Shear	≥ 2	3.40	0.0	See Table 5.6	Not applicable

DT_V (Truck load distribution width for shear) = **3.40**

$$DT\_M \ (Truck\ load\ distribution\ width\ for\ moment) = 4.6 - \frac{5.3}{\sqrt{26 + 5}} = \mathbf{3.65}.$$

This value has to be ≥ 2.8 **OK**

λ_V (Lane width parameter for shear) = **0**

K & B

$$\lambda\_M\ (Lane\ width\ parameter\ for\ moment) = 0.1 - \frac{0.25}{26} = \mathbf{0.0904}$$

γ_c_M (Truck modification factor for moment) = **1**

γ_c_V can be obtained from table 5.6:

Table 5.6
Factor γ_c for interior and exterior girders of slab-on-girder bridges for shear
(See Clauses 5.6.7.1 and Table 5.3.)

Condition	n	S (m)	γ_c
ULS, SLS, and FLS	All	$S \geq 2.0$	1.0

γ_c_V (Truck modification factor for shear) = **1**

Truck load fraction for shear and moment can be calculated from the following equation which can be found in clause 5.6.4.3:

$$F_T = \frac{S}{D_T\ \gamma_c (1+\mu\lambda)} \geq 1.05 \frac{n\ R_L}{N} \quad \text{for ULS and SLS}$$

$$F_{T}\_V = \frac{2.5}{3.4 \times 1 \times (1 + 1 \times 0)} = \mathbf{0.735}$$

$$F_{T}\_M = \frac{2.5}{3.65 \times 1 \times (1 + 1 \times 0.0904)} = \mathbf{0.628}$$

$$F_{T}\_V\ ,\ F_{T}\_M \geq 1.05 \times \frac{2 \times 0.9}{4}$$

$$F_{T}\_V\ ,\ F_{T}\_M \geq 0.473 - \mathbf{OK}$$

Table 4.3.2.2 - Distribution Variables and Factors summary

Variable	Symbol	Equation	Value
Girder spacing	S	Not Applicable	2.5
Number of Girders	N	Not Applicable	4
Number of Lanes	n	Not Applicable	2
Lane Width	We	W_c/n	5
Lane Modification Factor	μ	$\mu = \frac{W_e - 3.3}{0.6} \leq 1.0$	1
Modification Factor for Multi Lane Loading	R_L	Not Applicable	0.9
Truck Load Distribution Width for Shear	D_T_V	Not Applicable	3.4
Truck Load Distribution Width for Moment	D_T_M	$4.60 - \frac{5.30}{\sqrt{L_e + 5}} \geq 2.80$	3.648092
Lane Width Parameter for Shear	λ_V	Not Applicable	0
Lane Width Parameter for Moment	λ_M	$0.10 - \frac{0.25}{L_e}$	0.090385
Truck Load Modification Factor for Shear	γ_V	Not Applicable	1
Truck Load Modification Factor for Moment	γ_M	Not Applicable	1
Truck Load Fraction for Shear	F_T_V	$F_T = \frac{S}{D_T \gamma_c (1+\mu\lambda)} \geq 1.05 \frac{n R_L}{N}$	0.735294
Truck Load Fraction for Moment	F_T_M	$F_T = \frac{S}{D_T \gamma_c (1+\mu\lambda)} \geq 1.05 \frac{n R_L}{N}$	0.628484

According to clause 3.8.4.5.3, a dynamic load allowance of 0.25 is chosen for truck loading.

Final modification factors are shown below:

Table 4.3.2.3 - Modification Factors

	Truck Loading	Lane Loading
Shear	0.919118	0.735294
Moment	0.785606	0.628484

Table 4.3.2.4 (left) - Distributed Absolute Maximum Moment and Shear values for Truck Load (obtained from chapter 3)

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	398.38	0
0.5	388.21	166.00
1	378.05	323.31
1.5	367.89	471.93
2	357.72	611.87
2.5	347.56	743.11
3	337.40	865.68
3.5	327.23	979.55
4	316.34	1098.03
4.5	305.30	1210.47
5	294.25	1313.47
5.5	283.20	1407.03
6	272.16	1491.14
6.5	261.11	1565.81
7	250.06	1631.04
7.5	239.01	1691.81
8	227.97	1749.48
8.5	216.92	1797.71
9	205.87	1836.50
9.5	194.83	1865.85
10	183.78	1885.76
10.5	172.73	1896.22
11	161.69	1897.24
11.5	150.64	1891.76
12	141.77	1894.52
12.5	133.37	1887.83
13	124.97	1871.71
13.5	133.37	1887.83
14	141.77	1894.52
14.5	150.64	1891.76
15	161.69	1897.24
15.5	172.73	1896.22
16	183.78	1885.76
16.5	194.83	1865.85
17	205.87	1836.50
17.5	216.92	1797.71
18	227.97	1749.48
18.5	239.01	1691.81
19	250.06	1631.04
19.5	261.11	1565.81
20	272.16	1491.14
20.5	283.20	1407.03
21	294.25	1313.47
21.5	305.30	1210.47
22	316.34	1098.03
22.5	327.23	979.55
23	337.40	865.68
23.5	347.56	743.11
24	357.72	611.87
24.5	367.89	471.93
25	378.05	323.31
25.5	388.21	166.00
26	398.38	0

Table 4.3.2.4 (right) - Distributed Absolute Max. Moment and Shear values for Lane Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	340.99	0
0.5	331.18	142.30
1	321.36	277.62
1.5	311.55	405.97
2	301.74	527.35
2.5	291.92	641.75
3	282.11	749.18
3.5	272.30	849.63
4	262.02	951.62
4.5	251.64	1048.33
5	241.26	1137.58
5.5	230.88	1219.37
6	220.50	1293.71
6.5	210.12	1360.59
7	199.75	1420.01
7.5	189.37	1475.17
8	178.99	1526.93
8.5	168.61	1571.23
9	158.23	1608.07
9.5	147.85	1637.46
10	137.47	1659.39
10.5	127.09	1673.87
11	116.71	1680.88
11.5	106.33	1682.33
12	97.35	1687.63
12.5	88.67	1685.47
13	79.98	1675.85
13.5	88.67	1685.47
14	97.35	1687.63
14.5	106.33	1682.33
15	116.71	1680.88
15.5	127.09	1673.87
16	137.47	1659.39
16.5	147.85	1637.46
17	158.23	1608.07
17.5	168.61	1571.23
18	178.99	1526.93
18.5	189.37	1475.17
19	199.75	1420.01
19.5	210.12	1360.59
20	220.50	1293.71
20.5	230.88	1219.37
21	241.26	1137.58
21.5	251.64	1048.33
22	262.02	951.62
22.5	272.30	849.63
23	282.11	749.18
23.5	291.92	641.75
24	301.74	527.35
24.5	311.55	405.97
25	321.36	277.62
25.5	331.18	142.30
26	340.99	0

Truck loads dominate at every location so for further calculations of load combinations, truck loads will be used as live load.

4.3.3 Load Combinations

Table 4.3.3.1 (left) - Final Design Loads for Serviceability Limit State (1 x Dead Load + 0.9 x Live Load (Truck))

Table 4.3.3.1 (right) - Final Design Loads for Ultimate Limit State (1.2 x Girder Load + 1.2 x Deck Load + 1.5 x Asphalt and Waterproofing Load + 1.7 x Live Load (Truck))

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	726.31	0
0.5	703.02	329.75
1	679.72	644.60
1.5	656.43	944.57
2	633.14	1229.64
2.5	609.85	1499.82
3	586.56	1755.11
3.5	563.26	1995.52
4	539.32	2232.99
4.5	515.23	2457.96
5	491.15	2667.35
5.5	467.06	2861.17
6	442.97	3039.43
6.5	418.88	3202.11
7	394.80	3349.22
7.5	370.71	3485.25
8	346.62	3611.42
8.5	322.53	3722.01
9	298.45	3817.04
9.5	274.36	3896.50
10	250.27	3960.38
10.5	226.18	4008.70
11	202.10	4041.44
11.5	178.01	4061.26
12	155.88	4081.43
12.5	134.18	4086.02
13	112.48	4075.04
13.5	134.18	4086.02
14	155.88	4081.43
14.5	178.01	4061.26
15	202.10	4041.44
15.5	226.18	4008.70
16	250.27	3960.38
16.5	274.36	3896.50
17	298.45	3817.04
17.5	322.53	3722.01
18	346.62	3611.42
18.5	370.71	3485.25
19	394.80	3349.22
19.5	418.88	3202.11
20	442.97	3039.43
20.5	467.06	2861.17
21	491.15	2667.35
21.5	515.23	2457.96
22	539.32	2232.99
22.5	563.26	1995.52
23	586.56	1755.11
23.5	609.85	1499.82
24	633.14	1229.64
24.5	656.43	944.57
25	679.72	644.60
25.5	703.02	329.75
26	726.31	0

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	1133.46	0
0.5	1098.63	505.92
1	1063.81	988.29
1.5	1028.98	1447.13
2	994.16	1882.42
2.5	959.33	2294.17
3	924.51	2682.38
3.5	889.68	3047.05
4	853.63	3410.78
4.5	817.30	3755.46
5	780.97	4075.32
5.5	744.65	4370.35
6	708.32	4640.56
6.5	671.99	4885.94
7	635.67	5106.49
7.5	599.34	5310.70
8	563.01	5500.86
8.5	526.69	5666.20
9	490.36	5806.72
9.5	454.03	5922.41
10	417.71	6013.28
10.5	381.38	6079.32
11	345.05	6120.53
11.5	308.73	6141.93
12	276.09	6168.55
12.5	244.27	6170.34
13	212.45	6147.31
13.5	244.27	6170.34
14	276.09	6168.55
14.5	308.73	6141.93
15	345.05	6120.53
15.5	381.38	6079.32
16	417.71	6013.28
16.5	454.03	5922.41
17	490.36	5806.72
17.5	526.69	5666.20
18	563.01	5500.86
18.5	599.34	5310.70
19	635.67	5106.49
19.5	671.99	4885.94
20	708.32	4640.56
20.5	744.65	4370.35
21	780.97	4075.32
21.5	817.30	3755.46
22	853.63	3410.78
22.5	889.68	3047.05
23	924.51	2682.38
23.5	959.33	2294.17
24	994.16	1882.42
24.5	1028.98	1447.13
25	1063.81	988.29
25.5	1098.63	505.92
26	1133.46	0

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4.4 Design Loads - AASHTO LRFD 2014-17

4.4.1 Dead Load

Dead load calculation procedure is explained in previous section and it is similar in this case. The main difference here is that AASHTO choses to represent unit weight of concrete as a function of maximum cylindrical compressive strength of concrete (f’c) for f’c bigger or equal to 35 MPa up to 105 MPa. Our deck f’c is 35 MPa, and Girder f’c is 40 MPa so the formula given in table 3.5.1-1 for those is used.

Table 4.4.1.1 - Unit Weights for materials of Interest as given in AASHTO LRFD 2014-17 [4]

Component	Unit Weight (kcf)	Unit Weight (kN/m3)	Unit Weight (kN/m3)
Asphalt and Waterproofing (Biteminous Wearing Surfaces)	0.14	21.99	21.99
Deck (Reinforced Concrete)	0.14 + 0.001 x f'c	21.99 + 0.02278 x f'c	22.79
AASHTO Girders (Prestressed Concrete)	0.14 + 0.001 x f'c	21.99 + 0.02278 x f'c	22.90

$$DL_{Deck} = \frac{1\,m \times 10\,m \times 0.2\,m}{4 \times 1\,m} \times \frac{22.79\,kN}{m^3} = \mathbf{11.39}\,kN/m$$

$$DL_{Girder} = \frac{1\,m \times 0.50903124\,m^2}{1\,m} \times \frac{22.9\,kN}{m^3} = \mathbf{11.66}\,kN/m$$

$$DL_{Asph.\,+\,WProof.} = \frac{1\,m \times 0.065\,m \times 10\,m}{4 \times 1\,m} \times \frac{21.99\,kN}{m^3} = \mathbf{3.57}\,kN/m$$

$$DL_{per\ Girder} = 11.39 + 11.66 + 3.57 = \mathbf{26.63}\,kN/m$$

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Table 4.4.1.2 (Left)- Unfactored Absolute Moment and Shear Values due to Dead Load
Table 4.4.1.2 (Right)- Unfactored Absolute Moment and Shear Values due to Girder + Deck

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	346.15	0
0.5	332.84	169.75
1	319.53	332.84
1.5	306.21	489.28
2	292.90	639.05
2.5	279.59	782.17
3	266.27	918.64
3.5	252.96	1048.45
4	239.64	1171.60
4.5	226.33	1288.09
5	213.02	1397.93
5.5	199.70	1501.11
6	186.39	1597.63
6.5	173.08	1687.50
7	159.76	1770.71
7.5	146.45	1847.26
8	133.14	1917.16
8.5	119.82	1980.40
9	106.51	2036.98
9.5	93.20	2086.91
10	79.88	2130.18
10.5	66.57	2166.79
11	53.25	2196.75
11.5	39.94	2220.04
12	26.63	2236.69
12.5	13.31	2246.67
13	0.00	2250.00
13.5	13.31	2246.67
14	26.63	2236.69
14.5	39.94	2220.04
15	53.25	2196.75
15.5	66.57	2166.79
16	79.88	2130.18
16.5	93.20	2086.91
17	106.51	2036.98
17.5	119.82	1980.40
18	133.14	1917.16
18.5	146.45	1847.26
19	159.76	1770.71
19.5	173.08	1687.50
20	186.39	1597.63
20.5	199.70	1501.11
21	213.02	1397.93
21.5	226.33	1288.09
22	239.64	1171.60
22.5	252.96	1048.45
23	266.27	918.64
23.5	279.59	782.17
24	292.90	639.05
24.5	306.21	489.28
25	319.53	332.84
25.5	332.84	169.75
26	346.15	0

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	299.70	0
0.5	288.17	146.97
1	276.64	288.17
1.5	265.11	423.61
2	253.59	553.28
2.5	242.06	677.20
3	230.53	795.34
3.5	219.01	907.73
4	207.48	1014.35
4.5	195.95	1115.21
5	184.43	1210.31
5.5	172.90	1299.64
6	161.37	1383.21
6.5	149.85	1461.01
7	138.32	1533.06
7.5	126.79	1599.34
8	115.27	1659.85
8.5	103.74	1714.60
9	92.21	1763.59
9.5	80.69	1806.82
10	69.16	1844.28
10.5	57.63	1875.98
11	46.11	1901.91
11.5	34.58	1922.08
12	23.05	1936.49
12.5	11.53	1945.14
13	0.00	1948.02
13.5	11.53	1945.14
14	23.05	1936.49
14.5	34.58	1922.08
15	46.11	1901.91
15.5	57.63	1875.98
16	69.16	1844.28
16.5	80.69	1806.82
17	92.21	1763.59
17.5	103.74	1714.60
18	115.27	1659.85
18.5	126.79	1599.34
19	138.32	1533.06
19.5	149.85	1461.01
20	161.37	1383.21
20.5	172.90	1299.64
21	184.43	1210.31
21.5	195.95	1115.21
22	207.48	1014.35
22.5	219.01	907.73
23	230.53	795.34
23.5	242.06	677.20
24	253.59	553.28
24.5	265.11	423.61
25	276.64	288.17
25.5	288.17	146.97
26	299.70	0

Table 4.4.1.3 - Unfactored Absolute Moment and Shear Values due to Asphalt & Waterproofing

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	46.46	0
0.5	44.67	22.78
1	42.88	44.67
1.5	41.10	65.67
2	39.31	85.77
2.5	37.52	104.98
3	35.74	123.29
3.5	33.95	140.72
4	32.16	157.24
4.5	30.38	172.88
5	28.59	187.62
5.5	26.80	201.47
6	25.02	214.42
6.5	23.23	226.49
7	21.44	237.65
7.5	19.66	247.93
8	17.87	257.31
8.5	16.08	265.80
9	14.29	273.39
9.5	12.51	280.09
10	10.72	285.90
10.5	8.93	290.81
11	7.15	294.83
11.5	5.36	297.96
12	3.57	300.19
12.5	1.79	301.53
13	0.00	301.98
13.5	1.79	301.53
14	3.57	300.19
14.5	5.36	297.96
15	7.15	294.83
15.5	8.93	290.81
16	10.72	285.90
16.5	12.51	280.09
17	14.29	273.39
17.5	16.08	265.80
18	17.87	257.31
18.5	19.66	247.93
19	21.44	237.65
19.5	23.23	226.49
20	25.02	214.42
20.5	26.80	201.47
21	28.59	187.62
21.5	30.38	172.88
22	32.16	157.24
22.5	33.95	140.72
23	35.74	123.29
23.5	37.52	104.98
24	39.31	85.77
24.5	41.10	65.67
25	42.88	44.67
25.5	44.67	22.78
26	46.46	0

The dead load is separated because they will be factored differently at the upcoming sections.

4.4.2 Live Load

Live load consists of Truck Load (calculated in chapter 3) and lane load which is a uniformly distributed load of 9.34 kN/m (0.64 kilo pound-force per linear foot).

Table 4.4.2.1 (Left)- Undistributed Absolute Maximum Moment and Shear values for Truck Load (obtained from chapter 3)

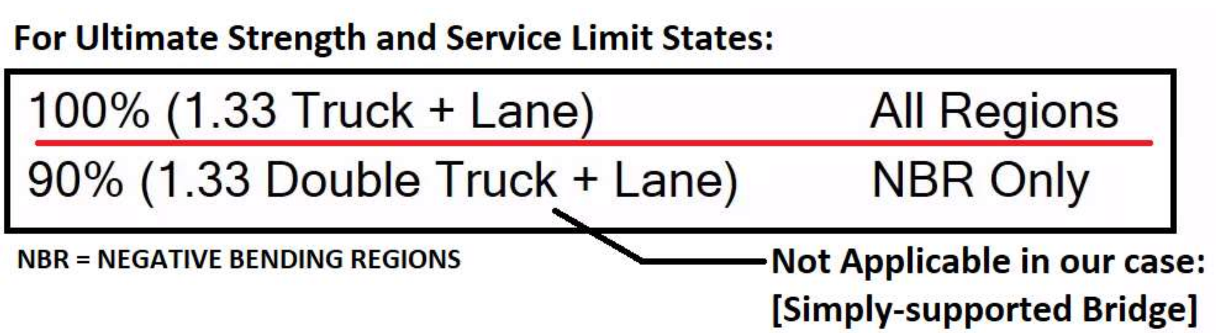
Table 4.4.2.1 (Right)- Undistributed Absolute Shear and Moment values for Lane Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	289.32	0
0.5	283.07	141.60
1	276.82	276.94
1.5	270.57	406.04
2	264.32	528.88
2.5	258.07	645.48
3	251.82	755.83
3.5	245.57	859.92
4	239.32	957.77
4.5	233.07	1049.37
5	226.82	1134.71
5.5	220.57	1213.81
6	214.32	1286.65
6.5	208.07	1353.25
7	201.82	1413.60
7.5	195.57	1467.69
8	189.32	1515.54
8.5	183.07	1557.13
9	176.82	1598.27
9.5	170.57	1636.05
10	164.32	1667.58
10.5	158.07	1692.86
11	151.82	1711.88
11.5	145.57	1724.66
12	139.32	1731.19
12.5	133.07	1731.47
13	126.82	1725.50
13.5	133.07	1731.47
14	139.32	1731.19
14.5	145.57	1724.66
15	151.82	1711.88
15.5	158.07	1692.86
16	164.32	1667.58
16.5	170.57	1636.05
17	176.82	1598.27
17.5	183.07	1557.13
18	189.32	1515.54
18.5	195.57	1467.69
19	201.82	1413.60
19.5	208.07	1353.25
20	214.32	1286.65
20.5	220.57	1213.81
21	226.82	1134.71
21.5	233.07	1049.37
22	239.32	957.77
22.5	245.57	859.92
23	251.82	755.83
23.5	258.07	645.48
24	264.32	528.88
24.5	270.57	406.04
25	276.82	276.94
25.5	283.07	141.60
26	289.32	0

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	121.42	0
0.5	116.75	59.54
1	112.08	116.75
1.5	107.41	171.62
2	102.74	224.16
2.5	98.07	274.37
3	93.40	322.23
3.5	88.73	367.77
4	84.06	410.96
4.5	79.39	451.83
5	74.72	490.36
5.5	70.05	526.55
6	65.38	560.41
6.5	60.71	591.93
7	56.04	621.12
7.5	51.37	647.97
8	46.70	672.49
8.5	42.03	694.67
9	37.36	714.52
9.5	32.69	732.03
10	28.02	747.21
10.5	23.35	760.05
11	18.68	770.56
11.5	14.01	778.73
12	9.34	784.57
12.5	4.67	788.07
13	0.00	789.24
13.5	4.67	788.07
14	9.34	784.57
14.5	14.01	778.73
15	18.68	770.56
15.5	23.35	760.05
16	28.02	747.21
16.5	32.69	732.03
17	37.36	714.52
17.5	42.03	694.67
18	46.70	672.49
18.5	51.37	647.97
19	56.04	621.12
19.5	60.71	591.93
20	65.38	560.41
20.5	70.05	526.55
21	74.72	490.36
21.5	79.39	451.83
22	84.06	410.96
22.5	88.73	367.77
23	93.40	322.23
23.5	98.07	274.37
24	102.74	224.16
24.5	107.41	171.62
25	112.08	116.75
25.5	116.75	59.54
26	121.42	0

AASHTO requires the amplification of the truck loads by a dynamic amplification factor to account for imperfections on pavement. An example is a truck passing from potholes. As it passes, it bounces up and down causing vibrations.

For ultimate strength and service limit states, AASHTO provides a dynamic amplification factor of 1.33 (Article 3.6.2.1) and provides the two Truck cases below [4]:



Only the first case is applicable to the bridge considered in this report since only positive moments are encountered in a simply supported bridge in longitudinal direction.

UNIVERSITY OF TORONTO

Table 4.4.2.2 – 100% of (1.33 x Truck Load + 1 x Lane Load)
(Unfactored and Undistributed Live Load Moment and Shear)

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	506.21	0
0.5	493.23	247.87
1	480.25	485.08
1.5	467.27	711.66
2	454.28	927.58
2.5	441.30	1132.85
3	428.32	1327.48
3.5	415.34	1511.46
4	402.35	1684.80
4.5	389.37	1847.48
5	376.39	1999.52
5.5	363.41	2140.91
6	350.42	2271.66
6.5	337.44	2391.75
7	324.46	2501.20
7.5	311.48	2600.00
8	298.49	2688.15
8.5	285.51	2765.66
9	272.53	2840.22
9.5	259.54	2907.97
10	246.56	2965.09
10.5	233.58	3011.55
11	220.60	3047.36
11.5	207.61	3072.53
12	194.63	3087.05
12.5	181.65	3090.93
13	168.67	3084.15
13.5	181.65	3090.93
14	194.63	3087.05
14.5	207.61	3072.53
15	220.60	3047.36
15.5	233.58	3011.55
16	246.56	2965.09
16.5	259.54	2907.97
17	272.53	2840.22
17.5	285.51	2765.66
18	298.49	2688.15
18.5	311.48	2600.00
19	324.46	2501.20
19.5	337.44	2391.75
20	350.42	2271.66
20.5	363.41	2140.91
21	376.39	1999.52
21.5	389.37	1847.48
22	402.35	1684.80
22.5	415.34	1511.46
23	428.32	1327.48
23.5	441.30	1132.85
24	454.28	927.58
24.5	467.27	711.66
25	480.25	485.08
25.5	493.23	247.87
26	506.21	0

K & B

4.4.3 Load Distribution

To convert the moments and shears obtained by a 2D longitudinal analysis to real life moment, forces need to be distributed. AASHTO provides an empirical equation based on finite element analysis and experiments to account for longitudinal stiffness differences before distributing forces (Article 4.6.2.2.1-1) [4].

$$K_g = n\left(I + Ae_g{}^2\right)$$

$$n = \frac{E_B}{E_D}$$

where:

E_B = modulus of elasticity of beam material (ksi)
E_D = modulus of elasticity of deck material (ksi)
I = moment of inertia of beam (in.^{4})
e_g = distance between the centers of gravity of the basic beam and deck (in.)

where:

E_B = modulus of elasticity of beam material (MPa)
E_D = modulus of elasticity of deck material (MPa)
I = moment of inertia of beam (mm^4)
e_g = distance between the centers of gravity of the basic beam and deck (mm)

Kg is called the "longitudinal stiffness parameter" and has the units of mm^4 (for SI Units)

"n" is called the modular ratio and it is the ratio between modulus of elasticity of beam and deck material. This converts temporarily beam material to deck material to prevent working with apples and oranges. For concrete beam and deck, this ratio is expected to be close to 1. This gains importance when steel girders and a concrete deck is used.

"A" is the cross-sectional area of the girder in mm^2 (for SI Units).

Modulus of Elasticity of concrete for deck and Girder can be calculated from (Clause 5.4.2.4-1)[4]:

$$E_c = 120{,}000 K_1 w_c^{2.0} f_c'^{0.33}$$

where:

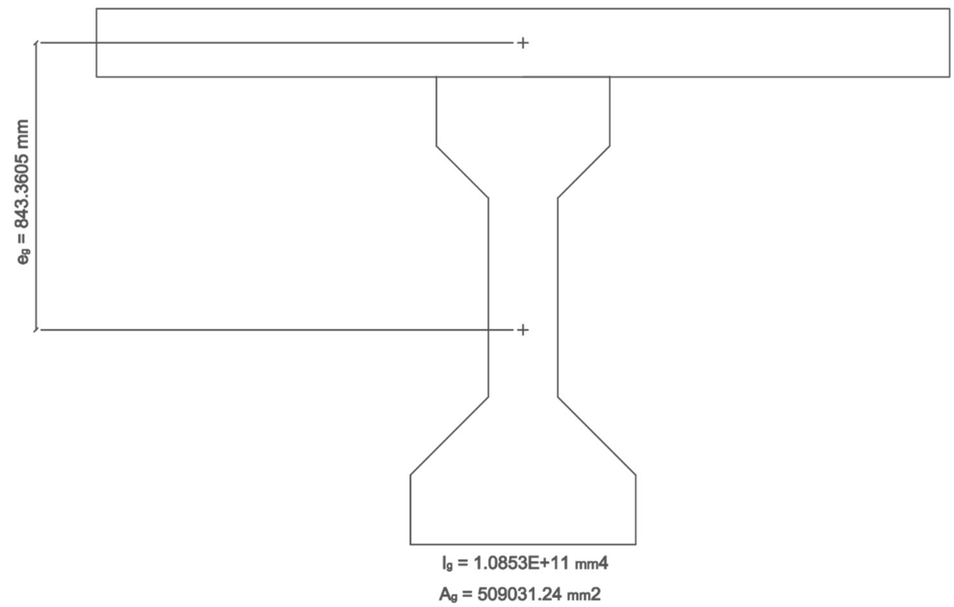
K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the owner
w_c = unit weight of concrete (kcf)
f'_c = compressive strength of concrete for use in design (ksi)

$$E_c \quad 0.043\, K_1\, \gamma_c^{1.5} \sqrt{f_c'}$$

where:

K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the owner
w_c = unit weight of concrete (kg/m^3)
f'_c = compressive strength of concrete for use in design (MPa)

Figure 4.4.3.1 – Composite Section Parameters



$$E_D = 0.043 \times 1 \times 2323^{1.5} \times \sqrt{35} = 28484\,MPa$$

$$E_B = 0.043 \times 1 \times 2334^{1.5} \times \sqrt{40} = 30451\,MPa$$

$$n = \frac{30451}{28484} = 1.069$$

$$Kg = 1.069 \times (1.0853 \times 10^{11} + 509031.24 \times 843.3605^2) = \mathbf{5.031 \times 10^{11}}\ mm^4$$

After calculating Kg, now it is time to calculate distribution factors. In the calculation of these factors, only interior girder will be shown in here. In order to be able to calculate these factors, several important parameters about the bridge is given here:

Table 4.4.3.1 – Bridge Parameters

Bridge Parameters	Value
Spacing of Beams [m]	2.5
Girder span length [m]	26
Slab Depth [mm]	200
Number of Girders	4

The bridge deck is reinforced concrete and supported with prestressed concrete girders.[4] Therefore, in table 4.6.2.2.2b-1, the following distribution factor equations for **moment** is given:

One Design Lane Loaded:

$$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.1}$$

Two or More Design Lanes Loaded:

$$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^{\ 3}}\right)^{0.1}$$

$3.5 \le S \le 16.0$
$4.5 \le t_s \le 12.0$
$20 \le L \le 240$
$N_b \ge 4$
$10{,}000 \le K_g \le 7{,}000{,}000$

S = spacing of beams or webs (ft)
L = span of beam (ft)
K_g = longitudinal stiffness parameter (in.^{4})
t_s = depth of concrete slab (in.)
N_b = number of beams, stringers or girders

One Design Lane Loaded:

$$0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^{\ 3}}\right)^{0.1}$$

Two or More Design Lanes Loaded:

$$0.075+\left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^{\ 3}}\right)^{0.1}$$

$1100 \le S \le 4900$
$110 \le t_s \le 300$
$6000 \le L \le 73\,000$
$N_b \ge 4$
$4\times10^9 \le K_g \le 3\times10^{12}$

S = spacing of beams or webs (mm)
t_s = depth of concrete slab (mm)
L = span of beam (mm)
K_g = longitudinal stiffness parameter (mm^4)
N_b = number of beams, stringers or girders

At this point, the number of design lanes should be determined. Using the information in article 3.6.1.1.1[4]:

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The number of design lanes should be determined by taking the integer part of the ratio $w/12.0$.
where w:
is the clear roadway width in feet between curbs, barriers, or both

The number of design lanes should be determined by taking the integer part of the ratio $w/3600$
where w:
is the clear roadway width in mm between curbs, barriers, or both

The bridge has **2** design lanes.

Table 4.4.3.2 – Moment Distribution Parameters and Criteria Check

Parameter	Value	Criteria	OK or ERR
S	2500	$1100 \le S \le 4900$	OK
t_s	200	$110 \le t_s \le 300$	OK
L	26000	$6000 \le L \le 73\,000$	OK
Nb	4	$N_b \ge 4$	OK
Kg	5.0307E+11	$4\times10^9 \le K_g \le 3\times10^{12}$	OK

$$DF_{1\_M}\ (One\ lane\ loaded) = 0.06 + \left(\frac{2500}{4300}\right)^{0.4} \times \left(\frac{2500}{26000}\right)^{0.3} \times \left(\frac{5.031 \times 10^{11}}{26000 \times 200^3}\right)^{0.1}$$

$$= 0.496$$

$$DF_{2\_M}\ (Two\ lanes\ loaded) = 0.075 + \left(\frac{2500}{2900}\right)^{0.6} \times \left(\frac{2500}{26000}\right)^{0.2} \times \left(\frac{5.031 \times 10^{11}}{26000 \times 200^3}\right)^{0.1}$$

$$= 0.701$$

$$max\,(DF_{1\_M}, DF_{2\_M}) = 0.701$$

In table 4.6.2.2.3a-1, the following distribution factor equations for **shear** is given [4]:

K & B

One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$

One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
$0.36+\frac{S}{7600}$	$0.2+\frac{S}{3600}-\left(\frac{S}{10\,700}\right)^{2.0}$	$1100 \le S \le 4900$ $6000 \le L \le 73\,000$ $110 \le t_s \le 300$ $N_b \ge 4$

Table 4.4.3.3 – Shear Distribution Parameters and Criteria Check

Parameter	Value	Criteria	OK or ERR
S	2500	$1100 \le S \le 4900$	OK
L	26000	$6000 \le L \le 73\,000$	OK
ts	200	$110 \le t_s \le 300$	OK
Nb	4	$N_b \ge 4$	OK

$$DF_{1\_V}\ (One\ lane\ loaded) = 0.36 + \left(\frac{2500}{7600}\right)$$

$$= 0.689$$

$$DF_{2\_V}\ (Two\ lanes\ loaded) = 0.2 + \left(\frac{2500}{3600}\right) - \left(\frac{2500}{10700}\right)^2$$

$$= 0.84$$

$$max\,(DF_{1\_V}, DF_{2\_V}) = 0.84$$

Table 4.4.3.3 – Final Distribution Factors

Distribution Factors	Value
Shear	0.839855
Moment	0.700569

Table 4.4.3.4 – Final Unfactored, Distributed Moment and Shear Values due to Live Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	425.15	0
0.5	414.24	173.65
1	403.34	339.84
1.5	392.44	498.56
2	381.53	649.83
2.5	370.63	793.64
3	359.72	929.99
3.5	348.82	1058.89
4	337.92	1180.32
4.5	327.01	1294.29
5	316.11	1400.80
5.5	305.21	1499.86
6	294.30	1591.45
6.5	283.40	1675.59
7	272.50	1752.26
7.5	261.59	1821.48
8	250.69	1883.24
8.5	239.79	1937.54
9	228.88	1989.77
9.5	217.98	2037.24
10	207.08	2077.25
10.5	196.17	2109.80
11	185.27	2134.89
11.5	174.37	2152.52
12	163.46	2162.69
12.5	152.56	2165.41
13	141.66	2160.66
13.5	152.56	2165.41
14	163.46	2162.69
14.5	174.37	2152.52
15	185.27	2134.89
15.5	196.17	2109.80
16	207.08	2077.25
16.5	217.98	2037.24
17	228.88	1989.77
17.5	239.79	1937.54
18	250.69	1883.24
18.5	261.59	1821.48
19	272.50	1752.26
19.5	283.40	1675.59
20	294.30	1591.45
20.5	305.21	1499.86
21	316.11	1400.80
21.5	327.01	1294.29
22	337.92	1180.32
22.5	348.82	1058.89
23	359.72	929.99
23.5	370.63	793.64
24	381.53	649.83
24.5	392.44	498.56
25	403.34	339.84
25.5	414.24	173.65
26	425.15	0

4.4.4 Load Combinations

There are strength, extreme event, service and fatigue limit state load combinations in AASHTO. They can be found in table 3.4.1-1. However, each combination is given in the code is there for a specific purpose. In this report, many of the loads like wind loads and earthquake loads are not considered. Therefore, most of these load combinations are not applicable to this design. Plugging in the numbers and evaluating the results based on the biggest might therefore not be the best approach here unless the load combination used serves the design purpose.

The load combinations that apply to this bridge are the following:

>***Service I***
>***Service III***
>***Strength I***

Each of these combinations will be used at the further calculations in this report. There is a factor called "γ_p" related with permanent loads (Table 3.4.1-2). Only permanent loads considered in this report is self-weight of the concrete portion of the bridge (DC) and asphalt and waterproofing (DW). They are separated because DW is much more variable then DC and that must be accounted with a different factor. The minimum factors specified for these factors is for uplift effect for continuous multi-span bridges and not applicable in a simply supported bridge in any way.

Table 4.4.4.1 – Load Combination Factors [4]

Load Combination Limit State	*DC* *DW*	*LL* *IM*
Strength I	γ_p	1.75
Service I	1.00	1.00
Service III	1.00	γ_{LL}

Table 4.4.4.2 – γ_p Values [4]

Type of Load, Foundation Type, and Method Used to Calculate Downdrag	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DW: Wearing Surfaces and Utilities	1.50	0.65

Table 4.4.4.3 – γ_{LL} Values [4]

Component	γ_{LL}
Prestressed concrete components designed using the refined estimates of time-dependent losses as specified in Article 5.9.5.4 in conjunction with taking advantage of the elastic gain	1.0
All other prestressed concrete components	0.8

Final load combinations to be used:

>***Service I: 1 x Dead Load (DC + DW) + 1 x Live Load (LL + IM)***
>***Service III: 1 x Dead Load (DC + DW) + 0.8 x Live Load (LL + IM)***
>***Strength I: 1.25 x (Deck Self-Weight Load + Girder Self-Weight Load) (DC) + 1.5 x Asphalt and Waterproofing Load (DW) + 1.75 x Live Load (LL + IM)***

Table 4.4.4.4 (left) - Final Design Loads for Service I Limit State
Table 4.4.4.4 (right) - Final Design Loads for Service III Limit State

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	771.30	0
0.5	747.08	343.40
1	722.87	672.68
1.5	698.65	987.84
2	674.43	1288.89
2.5	650.21	1575.82
3	626.00	1848.63
3.5	601.78	2107.33
4	577.56	2351.91
4.5	553.35	2582.38
5	529.13	2798.73
5.5	504.91	3000.97
6	480.69	3189.08
6.5	456.48	3363.09
7	432.26	3522.97
7.5	408.04	3668.74
8	383.83	3800.40
8.5	359.61	3917.93
9	335.39	4026.75
9.5	311.18	4124.15
10	286.96	4207.42
10.5	262.74	4276.59
11	238.52	4331.64
11.5	214.31	4372.57
12	190.09	4399.38
12.5	165.87	4412.08
13	141.66	4410.66
13.5	165.87	4412.08
14	190.09	4399.38
14.5	214.31	4372.57
15	238.52	4331.64
15.5	262.74	4276.59
16	286.96	4207.42
16.5	311.18	4124.15
17	335.39	4026.75
17.5	359.61	3917.93
18	383.83	3800.40
18.5	408.04	3668.74
19	432.26	3522.97
19.5	456.48	3363.09
20	480.69	3189.08
20.5	504.91	3000.97
21	529.13	2798.73
21.5	553.35	2582.38
22	577.56	2351.91
22.5	601.78	2107.33
23	626.00	1848.63
23.5	650.21	1575.82
24	674.43	1288.89
24.5	698.65	987.84
25	722.87	672.68
25.5	747.08	343.40
26	771.30	0

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	686.27	0
0.5	664.23	308.67
1	642.20	604.71
1.5	620.16	888.13
2	598.12	1158.92
2.5	576.09	1417.09
3	554.05	1662.63
3.5	532.02	1895.55
4	509.98	2115.85
4.5	487.94	2323.52
5	465.91	2518.57
5.5	443.87	2701.00
6	421.83	2870.79
6.5	399.80	3027.97
7	377.76	3172.52
7.5	355.72	3304.45
8	333.69	3423.75
8.5	311.65	3530.43
9	289.62	3628.80
9.5	267.58	3716.70
10	245.54	3791.98
10.5	223.51	3854.63
11	201.47	3904.66
11.5	179.43	3942.06
12	157.40	3966.84
12.5	135.36	3979.00
13	113.32	3978.53
13.5	135.36	3979.00
14	157.40	3966.84
14.5	179.43	3942.06
15	201.47	3904.66
15.5	223.51	3854.63
16	245.54	3791.98
16.5	267.58	3716.70
17	289.62	3628.80
17.5	311.65	3530.43
18	333.69	3423.75
18.5	355.72	3304.45
19	377.76	3172.52
19.5	399.80	3027.97
20	421.83	2870.79
20.5	443.87	2701.00
21	465.91	2518.57
21.5	487.94	2323.52
22	509.98	2115.85
22.5	532.02	1895.55
23	554.05	1662.63
23.5	576.09	1417.09
24	598.12	1158.92
24.5	620.16	888.13
25	642.20	604.71
25.5	664.23	308.67
26	686.27	0

Table 4.4.4.5 - Final Design Loads for Strength I Limit State

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	1188.31	0
0.5	1152.14	521.76
1	1115.97	1021.93
1.5	1079.80	1500.50
2	1043.63	1957.47
2.5	1007.46	2392.84
3	971.29	2806.61
3.5	935.12	3198.79
4	898.95	3569.36
4.5	862.78	3918.34
5	826.61	4245.72
5.5	790.44	4551.50
6	754.27	4835.69
6.5	718.10	5098.27
7	681.93	5339.26
7.5	645.77	5558.65
8	609.60	5756.44
8.5	573.43	5932.64
9	537.26	6096.67
9.5	501.09	6243.82
10	464.92	6369.38
10.5	428.75	6473.34
11	392.58	6555.70
11.5	356.41	6616.46
12	320.24	6655.62
12.5	284.07	6673.19
13	247.90	6669.15
13.5	284.07	6673.19
14	320.24	6655.62
14.5	356.41	6616.46
15	392.58	6555.70
15.5	428.75	6473.34
16	464.92	6369.38
16.5	501.09	6243.82
17	537.26	6096.67
17.5	573.43	5932.64
18	609.60	5756.44
18.5	645.77	5558.65
19	681.93	5339.26
19.5	718.10	5098.27
20	754.27	4835.69
20.5	790.44	4551.50
21	826.61	4245.72
21.5	862.78	3918.34
22	898.95	3569.36
22.5	935.12	3198.79
23	971.29	2806.61
23.5	1007.46	2392.84
24	1043.63	1957.47
24.5	1079.80	1500.50
25	1115.97	1021.93
25.5	1152.14	521.76
26	1188.31	0

4.5 Design Loads - CSA S6-66

4.5.1 Dead Load

CSA S6-66 and CSA S6-14 rev. 17 uses the same unit weights for materials that are used for dead load calculations, therefore the calculations will be similar.

Table 4.5.1.1 - Unit Weights for materials of Interest as given in CSA S6-66 [5]

Component	Unit Weight (kN/m3)
Asphalt and Waterproofing (Biteminous Wearing Surfaces)	23.5
Deck (Reinforced Concrete)	24
AASHTO Girders (Prestressed Concrete)	24.5

Therefore, Dead Load (DL) per girder is sum of these:

$$DL_{Deck} = \frac{1\,m \times 10\,m \times 0.2\,m}{4 \times 1\,m} \times \frac{24\,kN}{m^3} = 12\,\frac{kN}{m}$$

$$DL_{Girder} = \frac{1\,m \times 0.50903124\,m^2}{1\,m} \times \frac{24.5\,kN}{m^3} = 12.47\,\frac{kN}{m}$$

$$DL_{Asph.+WProof.} = \frac{1\,m \times 0.065\,m \times 10\,m}{4 \times 1\,m} \times \frac{23.5\,kN}{m^3} = 3.82\,\frac{kN}{m}$$

$$DL_{per\ Girder} = 12 + 12.47 + 3.82 = 28.29\,\frac{kN}{m}$$

Table 4.5.1.2 - Unfactored Absolute Moment and Shear Values due to Dead Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	367.77	0
0.5	353.63	180.35
1	339.48	353.63
1.5	325.34	519.83
2	311.19	678.96
2.5	297.05	831.02
3	282.90	976.01
3.5	268.76	1113.92
4	254.61	1244.76
4.5	240.47	1368.53
5	226.32	1485.23
5.5	212.18	1594.85
6	198.03	1697.40
6.5	183.89	1792.88
7	169.74	1881.29
7.5	155.60	1962.62
8	141.45	2036.88
8.5	127.31	2104.07
9	113.16	2164.19
9.5	99.02	2217.23
10	84.87	2263.20
10.5	70.73	2302.10
11	56.58	2333.93
11.5	42.44	2358.68
12	28.29	2376.36
12.5	14.15	2386.97
13	0.00	2390.51
13.5	14.15	2386.97
14	28.29	2376.36
14.5	42.44	2358.68
15	56.58	2333.93
15.5	70.73	2302.10
16	84.87	2263.20
16.5	99.02	2217.23
17	113.16	2164.19
17.5	127.31	2104.07
18	141.45	2036.88
18.5	155.60	1962.62
19	169.74	1881.29
19.5	183.89	1792.88
20	198.03	1697.40
20.5	212.18	1594.85
21	226.32	1485.23
21.5	240.47	1368.53
22	254.61	1244.76
22.5	268.76	1113.92
23	282.90	976.01
23.5	297.05	831.02
24	311.19	678.96
24.5	325.34	519.83
25	339.48	353.63
25.5	353.63	180.35
26	367.77	0

4.5.2 Live Load

In this section, live load (truck and lane loading) will be analyzed according to CSA S6-66.

> Truck Loading:
Unfactored and undistributed truck loads are obtained from chapter 3.

> Lane Loading:
Unfactored, undistributed lane loads are calculated by superimposing a point load and a uniformly distributed load of 9.34 kN/m acting on the 26-meter bridge. Point load location should be selected and changed according to distance from left support, to create the most shear and moment possible at every location throughout the bridge (Figure 2 CSA S6-66) [5].

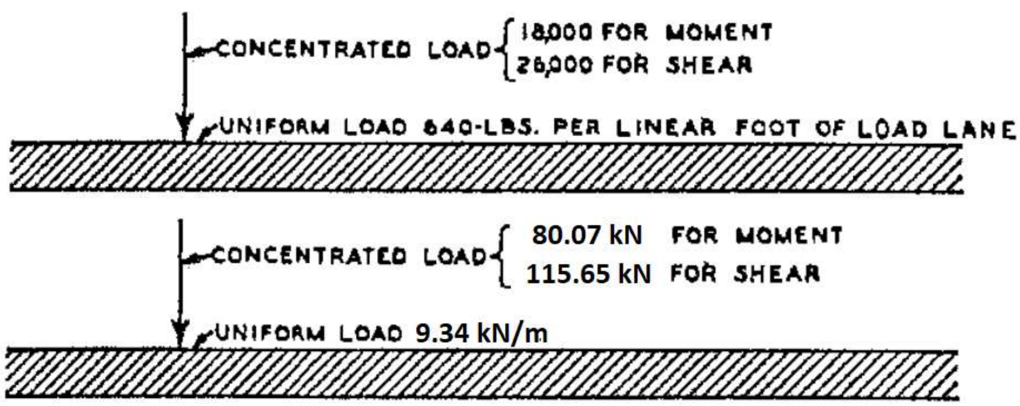

Table 4.5.2.1 (Left)- Undistributed Absolute Maximum Moment and Shear values for Truck Load (obtained from chapter 3)

Table 4.5.2.1 (Right)- Undistributed Absolute Shear and Moment values for Lane Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	289.32	0
0.5	283.07	141.60
1	276.82	276.94
1.5	270.57	406.04
2	264.32	528.88
2.5	258.07	645.48
3	251.82	755.83
3.5	245.57	859.92
4	239.32	957.77
4.5	233.07	1049.37
5	226.82	1134.71
5.5	220.57	1213.81
6	214.32	1286.65
6.5	208.07	1353.25
7	201.82	1413.60
7.5	195.57	1467.69
8	189.32	1515.54
8.5	183.07	1557.13
9	176.82	1598.27
9.5	170.57	1636.05
10	164.32	1667.58
10.5	158.07	1692.86
11	151.82	1711.88
11.5	145.57	1724.66
12	139.32	1731.19
12.5	133.07	1731.47
13	126.82	1725.50
13.5	133.07	1731.47
14	139.32	1731.19
14.5	145.57	1724.66
15	151.82	1711.88
15.5	158.07	1692.86
16	164.32	1667.58
16.5	170.57	1636.05
17	176.82	1598.27
17.5	183.07	1557.13
18	189.32	1515.54
18.5	195.57	1467.69
19	201.82	1413.60
19.5	208.07	1353.25
20	214.32	1286.65
20.5	220.57	1213.81
21	226.82	1134.71
21.5	233.07	1049.37
22	239.32	957.77
22.5	245.57	859.92
23	251.82	755.83
23.5	258.07	645.48
24	264.32	528.88
24.5	270.57	406.04
25	276.82	276.94
25.5	283.07	141.60
26	289.32	0

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	237.07	0
0.5	230.18	98.81
1	223.28	193.74
1.5	216.39	284.80
2	209.49	371.98
2.5	202.60	455.29
3	195.71	534.72
3.5	188.81	610.28
4	181.92	681.97
4.5	175.02	749.78
5	168.13	813.71
5.5	161.24	873.77
6	154.34	929.95
6.5	147.45	982.26
7	140.55	1030.70
7.5	133.66	1075.26
8	126.77	1115.94
8.5	119.87	1152.76
9	112.98	1185.69
9.5	106.08	1214.75
10	99.19	1239.94
10.5	92.30	1261.25
11	85.40	1278.69
11.5	78.51	1292.25
12	71.61	1301.94
12.5	64.72	1307.75
13	57.83	1309.69
13.5	64.72	1307.75
14	71.61	1301.94
14.5	78.51	1292.25
15	85.40	1278.69
15.5	92.30	1261.25
16	99.19	1239.94
16.5	106.08	1214.75
17	112.98	1185.69
17.5	119.87	1152.76
18	126.77	1115.94
18.5	133.66	1075.26
19	140.55	1030.70
19.5	147.45	982.26
20	154.34	929.95
20.5	161.24	873.77
21	168.13	813.71
21.5	175.02	749.78
22	181.92	681.97
22.5	188.81	610.28
23	195.71	534.72
23.5	202.60	455.29
24	209.49	371.98
24.5	216.39	284.80
25	223.28	193.74
25.5	230.18	98.81
26	237.07	0

From the tables above, truck load dominates lane loading so lane loading is ignored.

Now distribution factors need to be calculated.

From clause 5.1.6.1, design lane width and number of design lanes determined [5]

$$W = \frac{Wc}{N}$$

where
Wc = roadway width between curbs exclusive of median strip,
W = width of design traffic lane.

N =

Wc Feet	N
20 to 30 inclusive	2

$$W = \frac{Wc}{N}$$

where
Wc = roadway width between curbs exclusive of median strip,
W = width of design traffic lane.

N =

Wc (meters)	N
6.1 to 9.14 inclusive	2

Assuming 600 mm curbs in each side (typical),

$$W_c = 8.8\, m$$

$$N = 2$$

$$W = \frac{8.8}{2} = 4.4\, m$$

Moment and shear distribution factor for interior girders can be obtained from table 4 [5]

BENDING MOMENTS FOR INTERIOR STRINGERS

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or More Traffic Lanes
Concrete: On steel I-beam stringers and prestressed concrete girders	S/7.0 If S exceeds 10 feet use footnote 2	S/5.5 If S exceeds 14 feet use footnote 2

BENDING MOMENTS FOR INTERIOR STRINGERS

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or More Traffic Lanes
Concrete: On steel I-beam stringers and prestressed concrete girders	S/2.134 If S exceeds 3.05 m use footnote 2	S/1.676 If S exceeds 4.27 m use footnote 2

The bridge has 2 design lanes and it is designed for 2 lanes so, the distribution factor is

$$\frac{2.5}{1.676} = 1.491$$

However, this factor is per wheel. The half of it is the axle load.

= **0.746**

Impact factors for truck loads are calculated using the impact formula in clause 5.1.11.1 [5]

Impact Formula	Impact Formula
$I = \frac{50}{L+125} <= 0.3$	$I = \frac{50}{3.28 \times L + 125} <= 0.3$
L = Span length in feet	L = Span length in meters

$$I = \frac{50}{3.28 \times 26 + 125} = 0.238 \leq 0.3\ OK$$

Final Distribution Factor (1.238) (0.746) = **0.923**

Table 4.5.5.2 – Final Unfactored, Distributed Moment and Shear Values due to Live Load

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	267.02	0
0.5	261.25	130.68
1	255.48	255.60
1.5	249.71	374.74
2	243.95	488.12
2.5	238.18	595.73
3	232.41	697.57
3.5	226.64	793.64
4	220.87	883.95
4.5	215.10	968.49
5	209.34	1047.25
5.5	203.57	1120.25
6	197.80	1187.49
6.5	192.03	1248.95
7	186.26	1304.64
7.5	180.49	1354.57
8	174.73	1398.73
8.5	168.96	1437.12
9	163.19	1475.08
9.5	157.42	1509.95
10	151.65	1539.05
10.5	145.88	1562.38
11	140.12	1579.94
11.5	134.35	1591.74
12	128.58	1597.76
12.5	122.81	1598.02
13	117.04	1592.51
13.5	122.81	1598.02
14	128.58	1597.76
14.5	134.35	1591.74
15	140.12	1579.94
15.5	145.88	1562.38
16	151.65	1539.05
16.5	157.42	1509.95
17	163.19	1475.08
17.5	168.96	1437.12
18	174.73	1398.73
18.5	180.49	1354.57
19	186.26	1304.64
19.5	192.03	1248.95
20	197.80	1187.49
20.5	203.57	1120.25
21	209.34	1047.25
21.5	215.10	968.49
22	220.87	883.95
22.5	226.64	793.64
23	232.41	697.57
23.5	238.18	595.73
24	243.95	488.12
24.5	249.71	374.74
25	255.48	255.60
25.5	261.25	130.68
26	267.02	0

4.5.3 Load Combinations

Working load combinations (Service combinations) are obtained from the table in clause 5.2.6. Most of the parameters given here are not required or designed for in this report. Therefore, they will be ignored. The remaining combinations after eliminating parameters are given in blue below (Live load includes impact in the loads calculated in previous section).

WORKING LOAD COMBINATIONS

Group	Combination	Percentage of unit stress
Group I	= D+L+I+E+B+SF	100 = x1
Group II	= D+E+B+SF+W	125 =x1.25
Group III	= Group I+LF+F+CF+30 per cent W+WL	125
Group IV	= Group I+R+S+T	125
Group V	= Group II+R+S+T	140
Group VI	= Group III+R+S+T	140
Group VII	= D+E+B+SF+EQ	133⅓
Group VIII	= Group I+ICE	140
Group IX	= Group II+ICE	150

D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy
W = Wind Load on Structure
WL = Wind Load on Live Load — 100 pounds per linear foot
LF = Longitudinal Force from Live Load
CF = Centrifugal Force
F = Longitudinal Force Due to Friction and Elastomeric Bearing Pads
R = Rib Shortening
S = Shrinkage
T = Temperature
EQ = Earthquake
SF = Stream Flow Pressure
ICE = Ice Pressure

Combinations:

1.25 x Dead Load

1 x Dead Load + 1 x Live Load

: Not of Interest

According to clause 9.3.1.5, for ultimate design, the load combination must be the following at minimum:

1.5 x Dead Load + 2.5 x Live Load (includes impact)

So, the final load combinations are the following:

1.25 x Dead Load

1 x Dead Load + 1 x Live Load (includes impact)

1.5 x Dead Load + 2.5 x Live Load (includes impact)

Table 4.5.3.1 (left) - Final Design Loads for 1.25 x Dead Load (Working Limit State)
Table 4.5.3.1 (right) - Final Design Loads for 1xDead Load+1xLive Load (Working Limit State)

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	459.71	0
0.5	442.03	225.44
1	424.35	442.03
1.5	406.67	649.79
2	388.99	848.70
2.5	371.31	1038.77
3	353.63	1220.01
3.5	335.94	1392.40
4	318.26	1555.95
4.5	300.58	1710.66
5	282.90	1856.53
5.5	265.22	1993.56
6	247.54	2121.75
6.5	229.86	2241.10
7	212.18	2351.61
7.5	194.49	2453.27
8	176.81	2546.10
8.5	159.13	2630.09
9	141.45	2705.23
9.5	123.77	2771.54
10	106.09	2829.00
10.5	88.41	2877.63
11	70.73	2917.41
11.5	53.04	2948.35
12	35.36	2970.45
12.5	17.68	2983.71
13	0.00	2988.13
13.5	17.68	2983.71
14	35.36	2970.45
14.5	53.04	2948.35
15	70.73	2917.41
15.5	88.41	2877.63
16	106.09	2829.00
16.5	123.77	2771.54
17	141.45	2705.23
17.5	159.13	2630.09
18	176.81	2546.10
18.5	194.49	2453.27
19	212.18	2351.61
19.5	229.86	2241.10
20	247.54	2121.75
20.5	265.22	1993.56
21	282.90	1856.53
21.5	300.58	1710.66
22	318.26	1555.95
22.5	335.94	1392.40
23	353.63	1220.01
23.5	371.31	1038.77
24	388.99	848.70
24.5	406.67	649.79
25	424.35	442.03
25.5	442.03	225.44
26	459.71	0

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	634.79	0
0.5	614.88	311.03
1	594.96	609.22
1.5	575.05	894.57
2	555.14	1167.08
2.5	535.22	1426.75
3	515.31	1673.58
3.5	495.40	1907.56
4	475.48	2128.71
4.5	455.57	2337.02
5	435.66	2532.48
5.5	415.74	2715.10
6	395.83	2884.89
6.5	375.92	3041.83
7	356.00	3185.93
7.5	336.09	3317.19
8	316.18	3435.61
8.5	296.26	3541.19
9	276.35	3639.27
9.5	256.44	3727.18
10	236.52	3802.25
10.5	216.61	3864.48
11	196.70	3913.87
11.5	176.78	3950.42
12	156.87	3974.12
12.5	136.96	3984.99
13	117.04	3983.01
13.5	136.96	3984.99
14	156.87	3974.12
14.5	176.78	3950.42
15	196.70	3913.87
15.5	216.61	3864.48
16	236.52	3802.25
16.5	256.44	3727.18
17	276.35	3639.27
17.5	296.26	3541.19
18	316.18	3435.61
18.5	336.09	3317.19
19	356.00	3185.93
19.5	375.92	3041.83
20	395.83	2884.89
20.5	415.74	2715.10
21	435.66	2532.48
21.5	455.57	2337.02
22	475.48	2128.71
22.5	495.40	1907.56
23	515.31	1673.58
23.5	535.22	1426.75
24	555.14	1167.08
24.5	575.05	894.57
25	594.96	609.22
25.5	614.88	311.03
26	634.79	0

Table 4.5.3.2 - Final Design Loads for 1.5 x Dead Load + 2.5 x Live Load (Ultimate Limit State)

Distance From Left Support	Maximum Absolute Shear (kN)	Maximum Absolute Moment (kNm)
0	1219.20	0
0.5	1183.56	597.23
1	1147.92	1169.43
1.5	1112.29	1716.60
2	1076.65	2238.74
2.5	1041.01	2735.86
3	1005.37	3207.94
3.5	969.73	3654.99
4	934.10	4077.01
4.5	898.46	4474.01
5	862.82	4845.97
5.5	827.18	5192.91
6	791.54	5514.81
6.5	755.90	5811.69
7	720.27	6083.54
7.5	684.63	6330.36
8	648.99	6552.14
8.5	613.35	6748.90
9	577.71	6933.99
9.5	542.07	7100.72
10	506.44	7242.42
10.5	470.80	7359.10
11	435.16	7450.74
11.5	399.52	7517.36
12	363.88	7558.94
12.5	328.25	7575.50
13	292.61	7567.03
13.5	328.25	7575.50
14	363.88	7558.94
14.5	399.52	7517.36
15	435.16	7450.74
15.5	470.80	7359.10
16	506.44	7242.42
16.5	542.07	7100.72
17	577.71	6933.99
17.5	613.35	6748.90
18	648.99	6552.14
18.5	684.63	6330.36
19	720.27	6083.54
19.5	755.90	5811.69
20	791.54	5514.81
20.5	827.18	5192.91
21	862.82	4845.97
21.5	898.46	4474.01
22	934.10	4077.01
22.5	969.73	3654.99
23	1005.37	3207.94
23.5	1041.01	2735.86
24	1076.65	2238.74
24.5	1112.29	1716.60
25	1147.92	1169.43
25.5	1183.56	597.23
26	1219.20	0

4.6 Summary of Design Loads

There are 2 combinations chosen from AASHTO LFRD for service limit state and 2 combinations chosen for working load limit state from CSA S6-66. In this report, only the combination that produces the most loads will be selected in between those which produces conservative loads which is fine for the purposes of this project/report.

Loads per interior girder is summarized below graphically and numerically

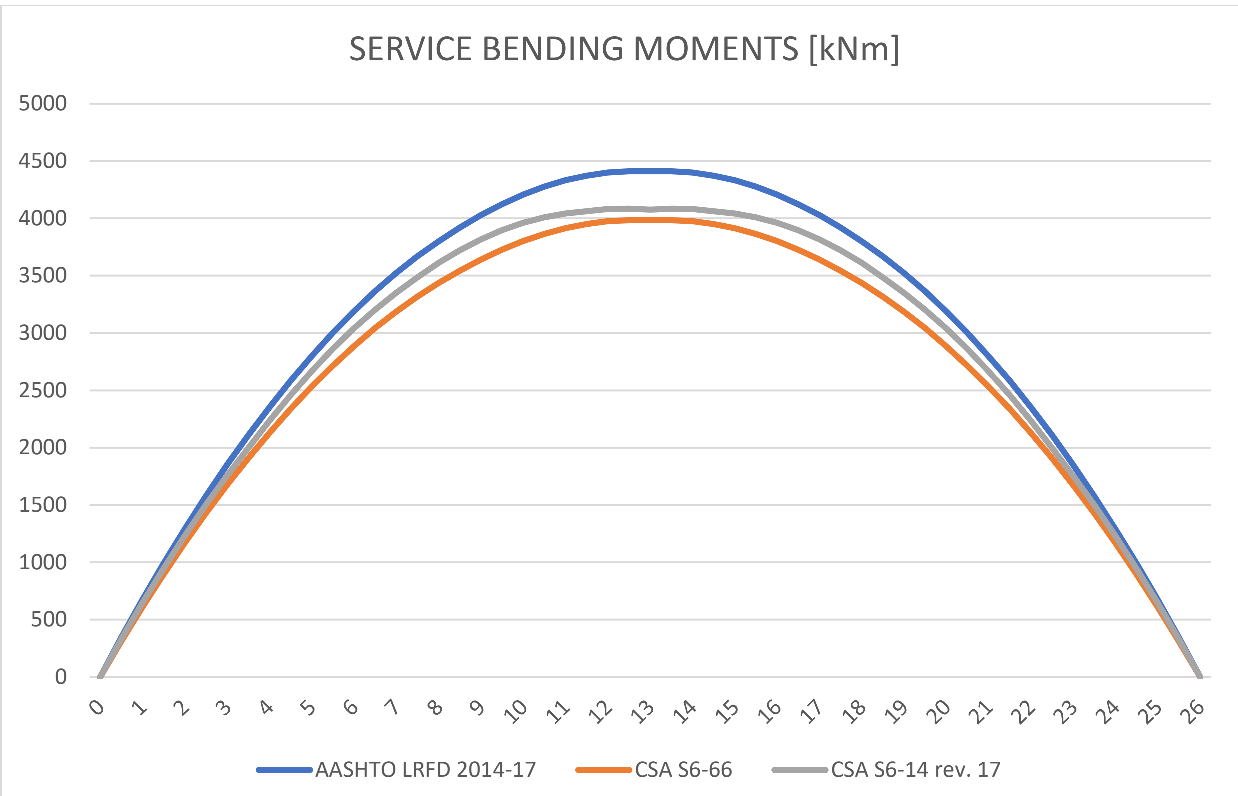

Figure 4.6.1 – Service Bending Moments – Graphical Results

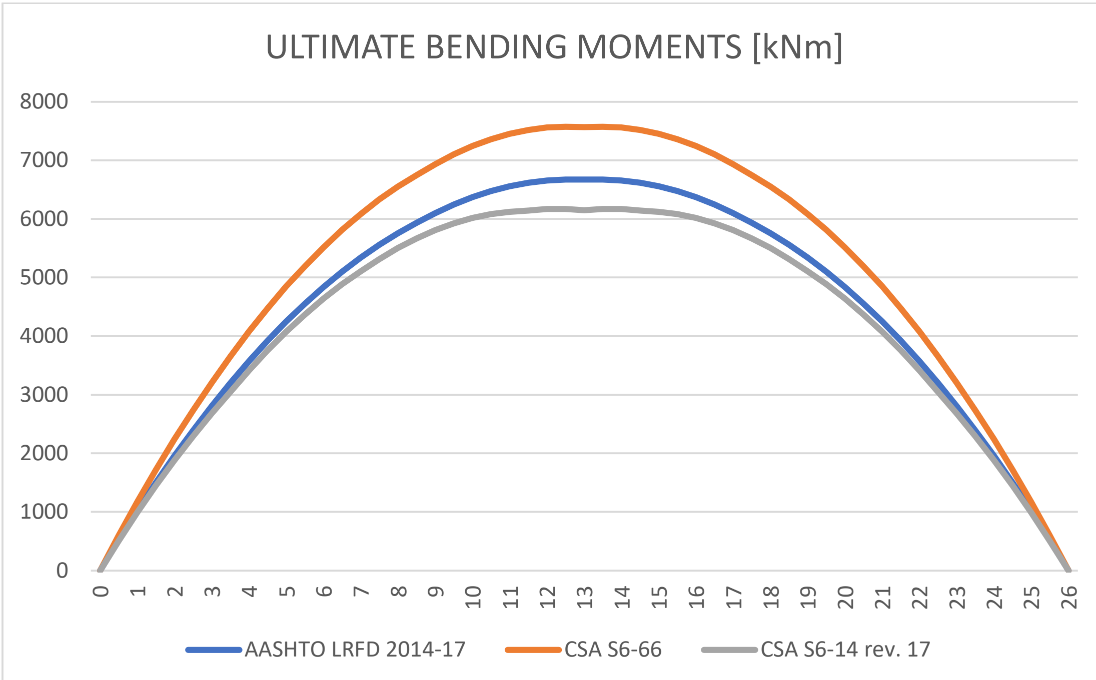

Figure 4.6.1 – Ultimate Bending Moments – Graphical Results

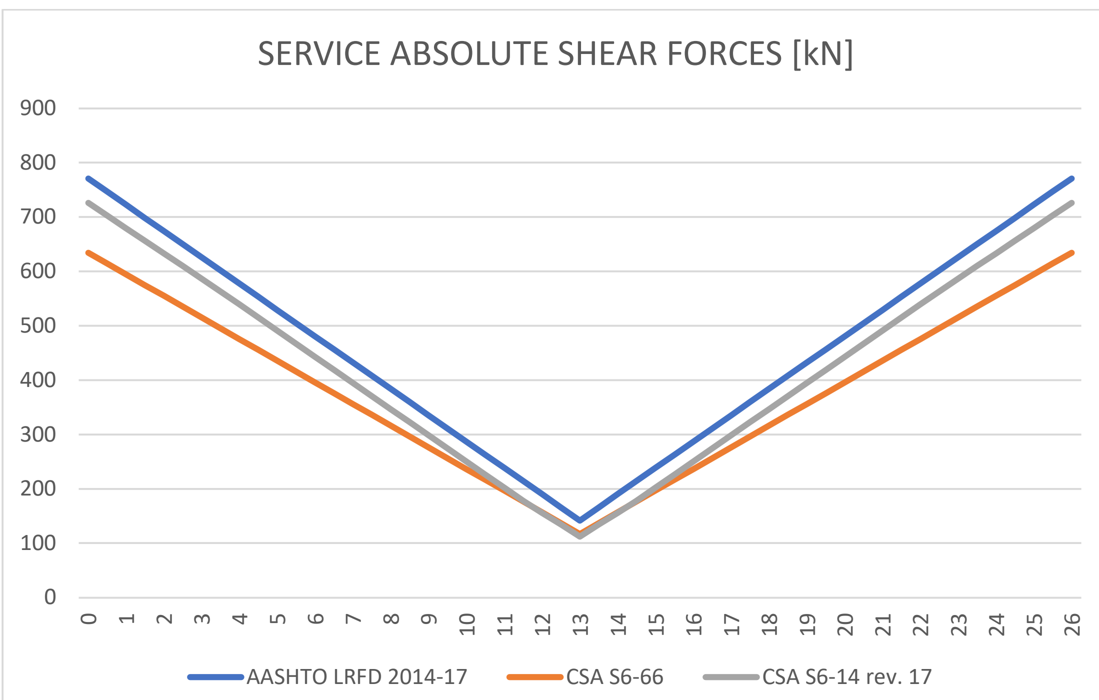

Figure 4.6.3 – Service Absolute Shear Forces – Graphical Results

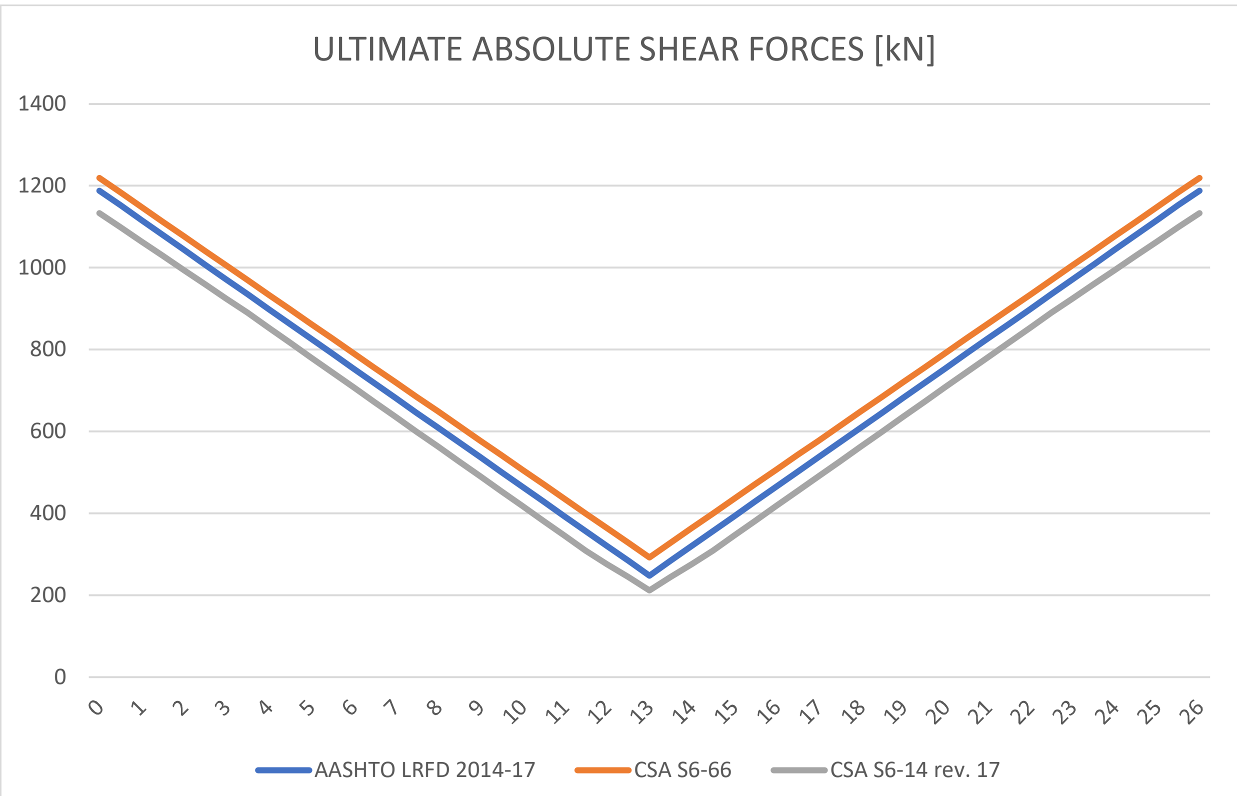

Figure 4.6.4 – Ultimate Absolute Shear Forces – Graphical Results

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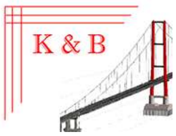
Table 4.6.1 – Final Design Loads – Numerical Results

Distance From Left Support	SERVICE LOADS						ULTIMATE LOADS					
	Maximum Absolute Moment (kNm)			Maximum Absolute Shear (kN)			Maximum Absolute Moment (kNm)			Maximum Absolute Shear (kN)		
	AASHTO LRFD 2014-17 2014-17	CSA S6-66	CSA S6-14 rev.17	AASHTO LRFD 2014-17 2014-17	CSA S6-66	CSA S6-14 rev.17	AASHTO LRFD 2014-17 2014-17	CSA S6-66	CSA S6-14 rev.17	AASHTO LRFD 2014-17 2014-17	CSA S6-66	CSA S6-14 rev.17
0	0	0	0	771.30	634.79	726.31	0	0	0	1188.31	1219.20	1133.46
0.5	343.40	311.03	329.75	747.08	614.88	703.02	521.76	597.23	505.92	1152.14	1183.56	1098.63
1	672.68	609.22	644.60	722.87	594.96	679.72	1021.93	1169.43	988.29	1115.97	1147.92	1063.81
1.5	987.84	894.57	944.57	698.65	575.05	656.43	1500.50	1716.60	1447.13	1079.80	1112.29	1028.98
2	1288.89	1167.08	1229.64	674.43	555.14	633.14	1957.47	2238.74	1882.42	1043.63	1076.65	994.16
2.5	1575.82	1426.75	1499.82	650.21	535.22	609.85	2392.84	2735.86	2294.17	1007.46	1041.01	959.33
3	1848.63	1673.58	1755.11	626.00	515.31	586.56	2806.61	3207.94	2682.38	971.29	1005.37	924.51
3.5	2107.33	1907.56	1995.52	601.78	495.40	563.26	3198.79	3654.99	3047.05	935.12	969.73	889.68
4	2351.91	2128.71	2232.99	577.56	475.48	539.32	3569.36	4077.01	3410.78	898.95	934.10	853.63
4.5	2582.38	2337.02	2457.96	553.35	455.57	515.23	3918.34	4474.01	3755.46	862.78	898.46	817.30
5	2798.73	2532.48	2667.35	529.13	435.66	491.15	4245.72	4845.97	4075.32	826.61	862.82	780.97
5.5	3000.97	2715.10	2861.17	504.91	415.74	467.06	4551.50	5192.91	4370.35	790.44	827.18	744.65
6	3189.08	2884.89	3039.43	480.69	395.83	442.97	4835.69	5514.81	4640.56	754.27	791.54	708.32
6.5	3363.09	3041.83	3202.11	456.48	375.92	418.88	5098.27	5811.69	4885.94	718.10	755.90	671.99
7	3522.97	3185.93	3349.22	432.26	356.00	394.80	5339.26	6083.54	5106.49	681.93	720.27	635.67
7.5	3668.74	3317.19	3485.25	408.04	336.09	370.71	5558.65	6330.36	5310.70	645.77	684.63	599.34
8	3800.40	3435.61	3611.42	383.83	316.18	346.62	5756.44	6552.14	5500.86	609.60	648.99	563.01
8.5	3917.93	3541.19	3722.01	359.61	296.26	322.53	5932.64	6748.90	5666.20	573.43	613.35	526.69
9	4026.75	3639.27	3817.04	335.39	276.35	298.45	6096.67	6933.99	5806.72	537.26	577.71	490.36
9.5	4124.15	3727.18	3896.50	311.18	256.44	274.36	6243.82	7100.72	5922.41	501.09	542.07	454.03
10	4207.42	3802.25	3960.38	286.96	236.52	250.27	6369.38	7242.42	6013.28	464.92	506.44	417.71
10.5	4276.59	3864.48	4008.70	262.74	216.61	226.18	6473.34	7359.10	6079.32	428.75	470.80	381.38
11	4331.64	3913.87	4041.44	238.52	196.70	202.10	6555.70	7450.74	6120.53	392.58	435.16	345.05
11.5	4372.57	3950.42	4061.26	214.31	176.78	178.01	6616.46	7517.36	6141.93	356.41	399.52	308.73
12	4399.38	3974.12	4081.43	190.09	156.87	155.88	6655.62	7558.94	6168.55	320.24	363.88	276.09
12.5	4412.08	3984.99	4086.02	165.87	136.96	134.18	6673.19	7575.50	6170.34	284.07	328.25	244.27
13	4410.66	3983.01	4075.04	141.66	117.04	112.48	6669.15	7567.03	6147.31	247.90	292.61	212.45
13.5	4412.08	3984.99	4086.02	165.87	136.96	134.18	6673.19	7575.50	6170.34	284.07	328.25	244.27
14	4399.38	3974.12	4081.43	190.09	156.87	155.88	6655.62	7558.94	6168.55	320.24	363.88	276.09
14.5	4372.57	3950.42	4061.26	214.31	176.78	178.01	6616.46	7517.36	6141.93	356.41	399.52	308.73
15	4331.64	3913.87	4041.44	238.52	196.70	202.10	6555.70	7450.74	6120.53	392.58	435.16	345.05
15.5	4276.59	3864.48	4008.70	262.74	216.61	226.18	6473.34	7359.10	6079.32	428.75	470.80	381.38
16	4207.42	3802.25	3960.38	286.96	236.52	250.27	6369.38	7242.42	6013.28	464.92	506.44	417.71
16.5	4124.15	3727.18	3896.50	311.18	256.44	274.36	6243.82	7100.72	5922.41	501.09	542.07	454.03
17	4026.75	3639.27	3817.04	335.39	276.35	298.45	6096.67	6933.99	5806.72	537.26	577.71	490.36
17.5	3917.93	3541.19	3722.01	359.61	296.26	322.53	5932.64	6748.90	5666.20	573.43	613.35	526.69
18	3800.40	3435.61	3611.42	383.83	316.18	346.62	5756.44	6552.14	5500.86	609.60	648.99	563.01
18.5	3668.74	3317.19	3485.25	408.04	336.09	370.71	5558.65	6330.36	5310.70	645.77	684.63	599.34
19	3522.97	3185.93	3349.22	432.26	356.00	394.80	5339.26	6083.54	5106.49	681.93	720.27	635.67
19.5	3363.09	3041.83	3202.11	456.48	375.92	418.88	5098.27	5811.69	4885.94	718.10	755.90	671.99
20	3189.08	2884.89	3039.43	480.69	395.83	442.97	4835.69	5514.81	4640.56	754.27	791.54	708.32
20.5	3000.97	2715.10	2861.17	504.91	415.74	467.06	4551.50	5192.91	4370.35	790.44	827.18	744.65
21	2798.73	2532.48	2667.35	529.13	435.66	491.15	4245.72	4845.97	4075.32	826.61	862.82	780.97
21.5	2582.38	2337.02	2457.96	553.35	455.57	515.23	3918.34	4474.01	3755.46	862.78	898.46	817.30
22	2351.91	2128.71	2232.99	577.56	475.48	539.32	3569.36	4077.01	3410.78	898.95	934.10	853.63
22.5	2107.33	1907.56	1995.52	601.78	495.40	563.26	3198.79	3654.99	3047.05	935.12	969.73	889.68
23	1848.63	1673.58	1755.11	626.00	515.31	586.56	2806.61	3207.94	2682.38	971.29	1005.37	924.51
23.5	1575.82	1426.75	1499.82	650.21	535.22	609.85	2392.84	2735.86	2294.17	1007.46	1041.01	959.33
24	1288.89	1167.08	1229.64	674.43	555.14	633.14	1957.47	2238.74	1882.42	1043.63	1076.65	994.16
24.5	987.84	894.57	944.57	698.65	575.05	656.43	1500.50	1716.60	1447.13	1079.80	1112.29	1028.98
25	672.68	609.22	644.60	722.87	594.96	679.72	1021.93	1169.43	988.29	1115.97	1147.92	1063.81
25.5	343.40	311.03	329.75	747.08	614.88	703.02	521.76	597.23	505.92	1152.14	1183.56	1098.63
26	0	0	0	771.30	634.79	726.31	0	0	0	1188.31	1219.20	1133.46



4.7 Conclusion

The design loads presented here are just the fundamental loads that every bridge designer should consider. In real world applications, the consideration for transversal and longitudinal skews, the lever rule explained in AASHTO for exterior girders, extreme event loads such as earthquake loads and flood loads, snow loading, wind loads and many other design parameters must be calculated in order to have a publicly safe bridge.

4.8 References

[1] Michael P. Collins and Denis Mitchell, Prestressed Concrete Structures, 1991, ISBN: 9780136916352

[2] Alexandre K. Bardow, Rita L. Seraderian and Michael P. Culmo, comparisons of spans for New England bulb-tee girder with PCI bulb-tee girder and AASHTO I-Girder, Prestressed Concrete Institution Archive, 1997

[3] CSA S6-14 Highway Bridge Design Code: Canadian Standards Association, 2014, Revision 2017

[4] AASHTO LRFD Bridge Design Specifications: American Association of State Highway and Transportation Officials, 2014, 8th Edition - Revision 2017

[5] CSA S6-66 Design of Highway Bridges: Canadian Standards Association, 1966

Chapter 5 – Design of Interior Prestressed Concrete Girder

Table of Contents

5.1 Introduction

This chapter will present the design of an interior girder based on the loads calculated in chapter 4. Three different designs are done based on the equations given in CSA S6-14 rev.17, AASHTO LRFD 2014-17 and CSA S6-66 respectively. The designs are then checked with strain compatibility analysis using computer program MATLAB for every cm.

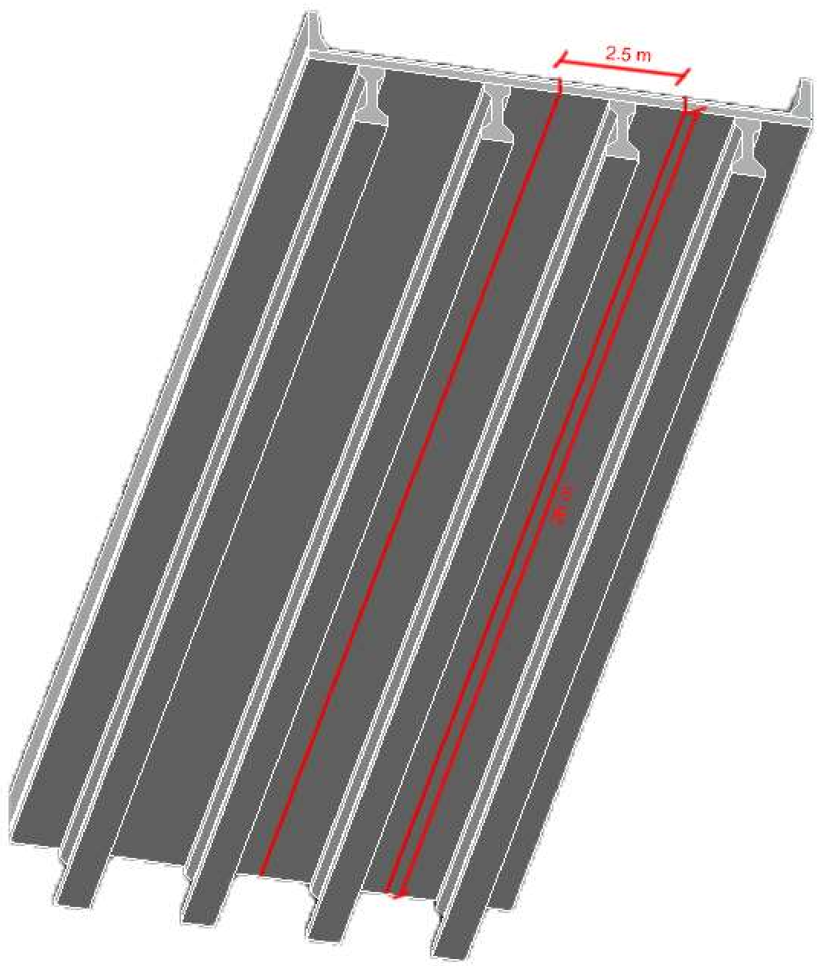


Figure 5.1.1 – 3D view of interior girder designed (girder between red longitudinal lines)

5.2 Composite Cross-Section Geometry and Material Properties

Although the materials used are listed in chapter 2 of this report, they are summarized below for convenience and easy access. Additionally, more detailed properties added:

Deck, asphalt and waterproofing

Cast in place deck: Thickness, $t_s = 200\ mm$

Deck 28 Day concrete strength, $f'_c = 35\ MPa$

Thickness of asphalt and waterproofing, $t_w = 65\ mm$

Precast beam: AASHTO Type-IV

Concrete strength at transfer, $f'_{ci} = 35\ MPa$

28 Day concrete strength, $f'_c = 40\ MPa$

Span Length, $L = 26\ m$

Pretensioning Strands

12.7 *mm diameter*, 7 *wire low relaxation strands*

Area of one strand $= 98.7\ mm^2$

Ultimate Stress, $f_{pu} = 1860\ MPa$

Yield Stress, $f_{py} = 0.9 \times f_{pu} = 1674\ MPa$

Stress limit at transfer $= f_{pi} \leq 0.75 \times f_{pu}$ $<=>$ $f_{pi} \leq 1395\ MPa$

for AASHTO and CSA S6– 66 design [*AASHTO table* 5.9.2.2– 1]

Stress limit at transfer $= f_{pi} \leq 0.74 \times f_{pu}$ $<=>$ $f_{pi} \leq 1377\ MPa$

for CSA S6– 14 design [CSA S6– 14 *table* 8.2]

Stress limit after all losses $= f_{pe} \leq 0.80 \times f_{pu}$ $<=>$ $f_{pe} \leq 1488\ MPa$

for AASHTO and CSA S6– 66 design [*AASHTO table* 5.9.2.2– 1]

Stress limit after all losses $= f_{pe} \leq 0.78 \times f_{pu}$ $<=>$ $f_{pe} \leq 1451\ MPa$

for CSA S6– 14 design [CSA S6– 14 *table* 8.2]

Modulus of Elasticity of prestressing steel, $E_p = 200000\ MPa$

Standard Reinforcement (non-prestressed)

Yield Stress, $f_y = 400\ MPa$

strain at yield, $\varepsilon_s = 0.002$

Modulus of Elasticity of reinforcing steel, $E_s = 200000\ MPa$

Ultimate Stress, $f_u = 550\ MPa$

strain at ultimate, $\varepsilon_u = 0.1$

Modulus of Elasticity equation given in CSA S6 – 14 rev. 17 $= \left(3000 \times \sqrt{f'_c} + 6900\right) \times \left(\frac{\gamma_c}{2300}\right)^{1.5}$

where:

$f'_c =$ *Maximum cylindrical compressive strength of concrete* [MPa]

$\gamma_c =$ *Concrete Density* [kg/m^3]

Modulus of Elasticity of deck concrete (CSA S6 – 14), $\mathbf{E_c}$ ***for deck*** $= \left(3000 \times \sqrt{35} + 6900\right) \times \left(\frac{2450}{2300}\right)^{1.5} = \mathbf{27100\ MPa}$

Modulus of Elasticity of girder concrete (CSA S6 – 14), $\mathbf{E_c}$ ***for girder*** $= \left(3000 \times \sqrt{40} + 6900\right) \times \left(\frac{2500}{2300}\right)^{1.5} = \mathbf{29320\ MPa}$

Modulus of Elasticity equation given in AASHTO LRFD 2014 – 17 $= 0.043 \times \gamma_c^{1.5} \times \sqrt{f'_c}$

where:

$f'_c =$ *Maximum cylindrical compressive strength of concrete* [MPa]

$\gamma_c =$ *Concrete Density* [kg/m^3]

Modulus of Elasticity of deck concrete (AASHTO LRFD 2014 – 17),

$\mathbf{E_c}$ ***for deck*** $= 0.043 \times 2450^{1.5} \times \sqrt{35} = \mathbf{30850\ MPa}$

Modulus of Elasticity of girder concrete (AASHTO LRFD 2014 – 17),

$\mathbf{E_c}$ ***for girder*** $= 0.043 \times 2500^{1.5} \times \sqrt{40} = \mathbf{33994\ MPa}$

Modulus of Elasticity equation given in CSA S6 – 66 $= 5000 \times \sqrt{f'_c}$

where:

$f'_c =$ *Maximum cylindrical compressive strength of concrete* [MPa]

Modulus of Elasticity of deck concrete (CSA S6 – 66), $\mathbf{E_c}$ ***for deck*** $= 5000 \times \sqrt{35} = \mathbf{29580\ MPa}$

Modulus of Elasticity of girder concrete (CSA S6 – 66), $\mathbf{E_c}$ ***for girder*** $= 5000 \times \sqrt{40} = \mathbf{31623\ MPa}$

Section Properties of the composite section and the girder

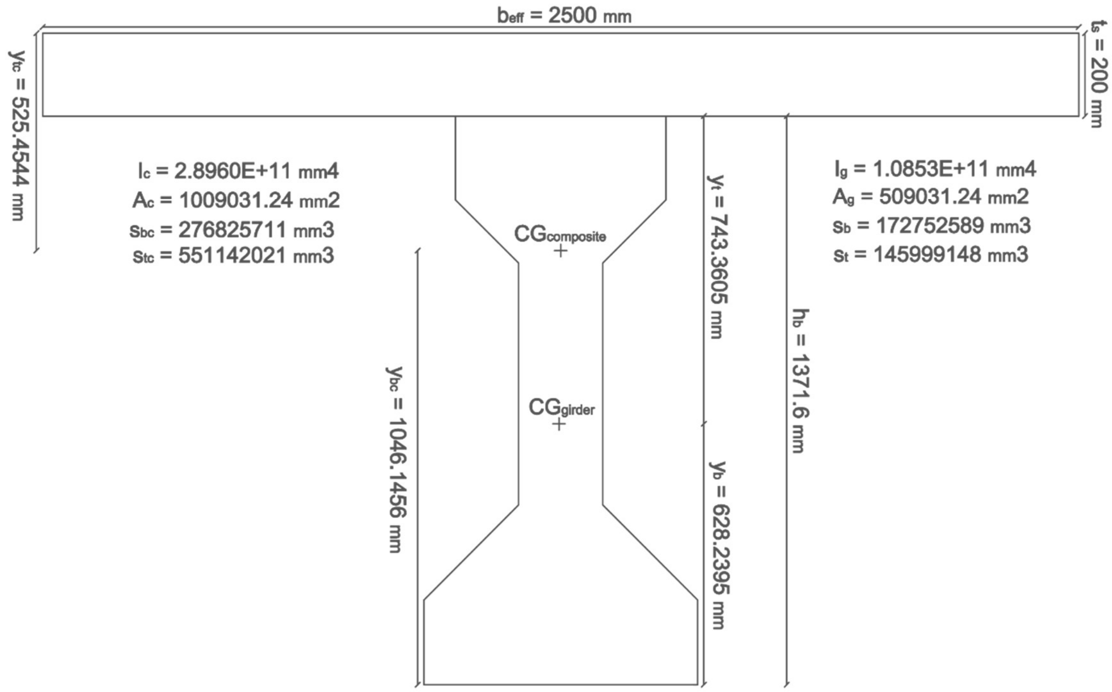


Figure 5.2.1 – Composite Girder Parameters

b_{eff} = *Effective flange width*

t_s = *Deck thickness*

h_b = *Girder height*

CG = *Center of gravity*

y_t = *Distance from extreme top fiber of non – composite precast girder to CG of girder*

y_b = *Distance from extreme bottom fiber of non – composite precast girder to CG of girder*

y_{tc} = *Distance from extreme top fiber of the composite section to CG of the composite section*

y_{bc} = *Distance from extreme bottom fiber of the composite section to CG of the composite section*

I_g = *Moment of inertia of girder*

A_g = *Cross – sectional area of girder*

s_t = *Section modulus from top of girder*

s_b = *Section modulus from bottom of girder*

I_c = *Moment of inertia of composite section*

A_c = *Cross − sectional area of composite section*

s_{tc} = *Section modulus from top of composite section*

s_{bc} = *Section modulus from bottom of composite section*

5.3 CSA S6-14 rev. 17

5.3.1 Estimation of Required Prestress and Initial Strand Pattern

Bottom tensile stress at midspan during service according to service combination in CSA S6 − 14:

$$fb = \frac{M_G + M_S}{s_b} + \frac{M_{SDL} + 0.9 \times M_{LL}}{s_{bc}}$$

$$\boldsymbol{fb} = \frac{(1053.82 + 1014) \times 10^6}{1.7275 \times 10^8} + \frac{(322.68 + 0.9 \times 1897.24) \times 10^6}{2.7683 \times 10^8} = \mathbf{19.3037\ \boldsymbol{MPa}}$$

M_G = *Moment due to self − weight of girder at midspan*

M_S = *Moment due to self − weight of deck at midspan*

M_{SDL} = *Moment due to self − weight of asphalt and waterproofing at midspan*

M_{LL} = *Moment due to live load at midspan*

At service loading conditions, allowable tensile stress according to CSA S6 − 14 rev. 17 is:

$$\boldsymbol{F_b} = 0.4 \times \sqrt{f'_c \text{ for girder}} = 0.4 \times \sqrt{40} = \mathbf{2.53\ \boldsymbol{MPa}}$$

Required Number of Strands:

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress − allowable tensile stress at final $= f_b - F_b$

$$\boldsymbol{f_{pb}} = \mathbf{19.3037 - 2.53 = 16.7739\ \boldsymbol{MPa}}$$

Assuming the distance from center of graavity of strands to the bototm fiber of the beam is equal to $\mathbf{y_{bs} = 100\ \boldsymbol{mm}}$

Strand eccentricity at midspan:

$$\boldsymbol{e_c} = y_b - y_{bs} = 628.2395 - 100 = \mathbf{528.2395\ \boldsymbol{mm}}$$

Bottom fiber stress due to prestress after losses:

$$f_{b\_prestress} = \frac{P_{pe}}{A_g} + \frac{P_{pe} \times e_c}{s_b} \quad \textit{where } P_{pe} = \textit{Effective prestressing force after all losses}$$

$$16.7739 = \frac{P_{pe} \times 10^3}{5.0903 \times 10^5} + \frac{P_{pe} \times 528.2395 \times 10^3}{1.7275 \times 10^8}$$

solving this for P_{pe}, $\boldsymbol{P_{pe} = 3339.88\ kN}$

Assuming final losses is 20% *of* f_{pi} *(for now)*

Assumed final losses $= 0.2 \times 1377 = \boldsymbol{275.28\ MPa}$

The prestress force per strand after losses $=$ *cross* $-$ *sectional area of one strand* $\times$ $(f_{pi} - \textit{losses})$

$= 98.7 \times (1377 - 275.28) \times 10^{-3} = \boldsymbol{108.6805\ kN}$

Number of Strands required $= 3.3399 \times 10^3 \ / \ 108.6805 = \boldsymbol{30.7312}$

Try **32 strands** *as an initial trial:*

Effective strand eccentricity at midspan after strand arrangement

$$\boldsymbol{e_c} = 628.2395 - \frac{12 \times (50 + 100) + 8 \times 150}{32} = \boldsymbol{534.4895\ mm}$$

$$\boldsymbol{P_{pe}} = 32 \times 108.6805 = \boldsymbol{3477.8\ kN}$$

$$\boldsymbol{f_b} = \frac{3477.8 \times 10^3}{5.0903 \times 10^5} + \frac{534.4895 \times 3477.8 \times 10^3}{1.7275 \times 10^8} = \boldsymbol{17.5923\ MPa}$$

$\boldsymbol{17.5923\ MPa > 16.7739\ MPa}$ ***therefore OK***

Trying **30 strands** *hoping to use less steel if possible (Iteration # 2):*

Effective strand eccentricity at midspan after strand arrangement

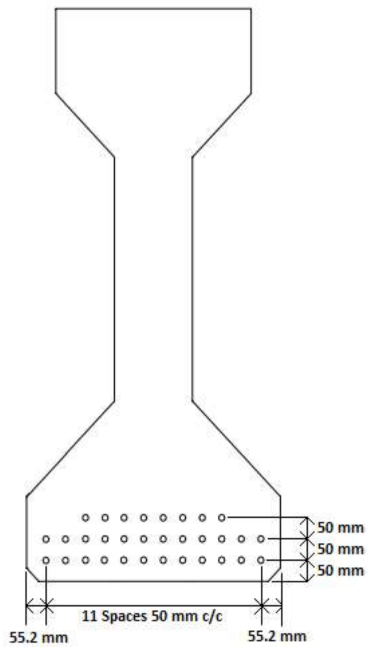
$$\boldsymbol{e_c} = 628.2395 - \frac{12 \times (50 + 100) + 6 \times 150}{30} = \boldsymbol{538.2395\ mm}$$

$$\boldsymbol{P_{pe}} = 30 \times 108.6805 = \boldsymbol{3260.4\ kN}$$

$$\boldsymbol{f_b} = \frac{3260.4 \times 10^3}{5.0903 \times 10^5} + \frac{538.2395 \times 3260.4 \times 10^3}{1.7275 \times 10^8} = \boldsymbol{17.5923\ MPa}$$

$\boldsymbol{16.5635\ MPa < 16.7739\ MPa}$ ***therefore NOT OK***

Therefore use **32 strands**



Initial Strand Pattern

Figure 5.3.1.1 – Initial Strand Pattern

5.3.2 Prestressing Losses

Total prestress loss:

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$

where

Δf_{pES} = *Loss of prestress due to elastic shortening*

Δf_{pSR} = *Loss of prestress due to concrete shrinkage*

Δf_{pCR} = *Loss of prestress due to creep of concrete*

Δf_{pR2} = *Loss of prestress due to relaxation of steel after transfer*

Elastic Shortening:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} \times f_{cir}$$

where:

f_{cir} = *Sum of concrete stresses at the center of gravity of prestressing steel due to moment and axial force caused by the prestressing force and due to the moment caused by self – weight of the girder*

$$f_{cir} = \frac{P_i}{A_{gt}} + \frac{P_i \times e_c^{\,2}}{I_{gt}} - \frac{M_G \times e_c}{I_{gt}}$$

where:

P_i = *Pretensioning force after allowing for initial losses*

A_{gt} = *Transformed area of the girder*

M_G = *Moment caused by the self – weight of the girder*

e_c = *Distance from the CG of the prestessing steel to CG of girder*

With the absence of more information, a 8% loss from maximum allowed initial stress at transfer is assumed.

$$\boldsymbol{Pi} = 32\ strands \times 98.7\ mm^2 \times 0.92^{*} \times 1376.4\ MPa^{**} \times 10^{-3} = \mathbf{3999.4\ kN}$$

$$^{*}\ \frac{(100\% - 8\%)}{100}\ ,\ ^{**}f_{pi}$$

Using transformed properties is more common these days with the advancement of computer technology.

Using gross area here also gives acceptable results.

$$\boldsymbol{A_{gt}} = \mathbf{528487.6385}\ \boldsymbol{mm^2}$$

I_{gt} = *Transformed moment of Inertia*

$$\boldsymbol{I_{gt}} = \mathbf{1.1391 \times 10^{11}}\ \boldsymbol{mm^4}$$

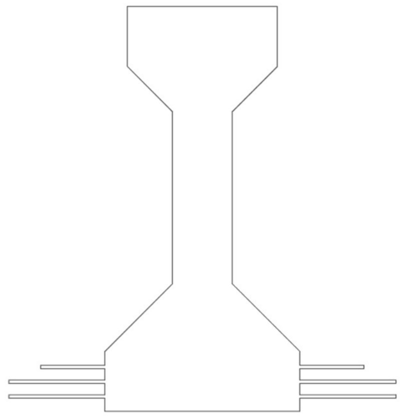


Figure 5.3.2.1 – Transformed Girder

$e_c = 534.4895\ mm$

$M_G = 1053.82\ kNm$

$$fcir = \frac{3999.4 \times 10^3}{528488} + \frac{3999.4 \times 10^3 \times 534.4895^2}{1.1391 \times 10^{11}} - \frac{1053.82 \times 10^6 \times 534.4895}{1.1391 \times 10^{11}}$$

$fcir = 12.6533\ MPa$

$Eci = 27932\ MPa$

$Ep = 200000\ MPa$

$$n = \frac{\mathrm{Ep}}{\mathrm{Eci}} = \frac{200000}{27932} = 7.1602$$

$\Delta f_{pES} = n \times fcir = 7.1602 \times 12.6533 = \mathbf{90.6\ MPa}$

Losses due to Shrinkage of Concrete:

$\Delta f_{pSR} = 117 - 1.05 \times RH$

where:

RH = *Relative humidity of surrounding air* (***Assumed* 60%**)

$\Delta f_{pSR} = 117 - 1.05 \times 60 = \mathbf{54\ MPa}$

***Losses due to Creep*:**

$$\Delta f_{pCR} = [1.37 - 0.77 \times (0.01 \times RH)^2] \times K_{cr} \times n \times (f_{cir} - f_{cds})$$

***where*:**

$RH =$ *Relative humidity of surrounding air* **(*Assumed* 60%)**

$K_{cr} =$ *Creep coefficient* **(2 *for pretensioned*)**

$f_{cds} =$ *Change of stress at the center of gravity of the prestressing steel due to moment caused by the self − weight of the deck and moment caused by the asphalt and waterproofing*.

$$n = \frac{E_p}{E_c} = \frac{200000}{29321} = \mathbf{6.8211}$$

$$I_{ct} = 3.0694 \times 10^{11}\ mm^4$$

$$A_{ct} = 1028487.6385\ mm^2$$

$$\Delta f_{pCR} = [1.37 - 0.77 \times (0.01 \times 60)^2] \times 2 \times 6.8211 \times (12.6533 - f_{cds})$$

$$f_{cds} = \frac{M_s \times e_c}{I_{gt}} + \frac{M_{SDL} \times (y_{bc} - (y_b - e_c))}{I_{ct}}$$

$$f_{cds} = \frac{1014 \times 534.4895 \times 10^6}{1.1391 \times 10^{11}} + \frac{322.68 \times 10^6 \times (1046.1 - (628.2395 - 534.4895))}{3.0694 \times 10^{11}} = \mathbf{5.7591\ MPa}$$

$$\Delta f_{pCR} = [1.37 - 0.77 \times (0.01 \times 60)^2] \times 2 \times 6.8211 \times (12.6533 - 5.7591) = \mathbf{102.7794\ MPa}$$

***Losses due to the relaxation of prestressing strands*:**

Initial loss before transfer is accounted in the girder fabrication process therefore not calculated here or taken as 0.

$$\Delta f_{pR2} = \left(\frac{f_{pi}}{f_{pu}} - 0.55\right) \times \left(0.34 - \frac{\Delta f_{pCR} + \Delta f_{pSR}}{1.25 \times f_{pu}}\right) \times \frac{f_{pu}}{3} \geq 0.002 \times f_{pu}$$

$$\Delta f_{pR2} = \left(\frac{1376.4}{1860} - 0.55\right) \times \left(0.34 - \frac{102.7794 + 54}{1.25 \times 1860}\right) \times \frac{1860}{3} \geq 0.002 \times 1860 = \mathbf{32.1085\ MPa}$$

***Total Prestressing losses*:** $32.1085 + 102.7794 + 54 + 90.6004 = \mathbf{279.4883\ MPa}$

$$\%\ Loss = \frac{279.4883}{f_{pi}} \times 100 = \frac{279.4883}{1376.4} \times 100 = \mathbf{20.3057\ \%}$$

For safety reasons however, 50% *of total relaxation losses will be counted in initial prestressing loss*

Initial Prestressing losses: Losses due to Elastic Shortening + 50% *of Total Relaxation Losses*

$$\textit{Initial Prestressing Loss} = \frac{90.6004 + 0.5 \times 32.1085}{1376.4} \times 100 = \mathbf{7.7488\ \%}$$

$7.7488\ \%$ ***is approximately equal to*** 8% *so* ***no need to iterate for now***. *If this wasn't a close value, iteration assuming this as initial loss would be required.*

Total final loss $= 279.4883\ MPa$

Total initial loss $= 106.6547\ MPa$

Final effective prestress, $f_{pe} = f_{pi} - \Delta f_{pT} = 1376.4 - 279.4883 = \mathbf{1096.9\ MPa}$

At service, $f_{pe} \leq 1450.8\ MPa\ OK$

Total prestressing force after all losses, $P_{pe} = 32 \times 1096.9 \times 98.7 \times 10^{-3} = \mathbf{3464.5\ kN}$

Final stress in the bottom fiber at midspan:

$$f_b = \frac{P_{pe}}{A_g} + \frac{P_{pe} \times e_c}{s_b} = \frac{3464.5 \times 10^3}{5.0903 \times 10^5} + \frac{3464.5 \times 10^3 \times 534.4895}{1.7275 \times 10^8} = 17.5250\ MPa > 16.7739\ MPa\ OK$$

5.3.3 Concrete stress limits at top and bottom

5.3.3.1 Stress limits at transfer and Strand Pattern

Midspan:

At transfer, the compressive stress in the top fiber cannot exceed:

$$f_{ti} = 0.6 \times 35 = 21\ MPa$$

$$f_{ti} \geq \frac{P_i}{A_g} - \frac{P_i \times e_c}{s_t} + \frac{M_G}{s_t}$$

$$P_i = 32 \times 98.7 \times (1376.4 - 106.6547) \times 10^{-3} = \mathbf{4010.4\ kN}$$

$$f_{ti} = \frac{4010.4 \times 10^3}{5.0903 \times 10^5} - \frac{4010.4 \times 10^3 \times 534.4895}{1.46 \times 10^8} + \frac{1053.82 \times 10^6}{1.46 \times 10^8} = 0.4149\ MPa\ OK$$

At transfer, the compressive stress in the bottom fiber cannot exceed:

$$f_{bi} = 0.6 \times 35 = 21\ MPa$$

$$f_{bi} \geq + \frac{P_i}{A_g} + \frac{P_i \times e_c}{s_b} - \frac{M_G}{s_b}$$

$$f_{bi} = \frac{4010.4 \times 10^3}{5.0903 \times 10^5} + \frac{4010.4 \times 10^3 \times 534.4895}{1.7275 \times 10^8} - \frac{1053.82 \times 10^6}{1.7275 \times 10^8} = 14.1861\ MPa\ OK$$

This same procedure is done for every 0.5 m of span and limits of eccentricities are determined using excel. This will serve to determine the optimal hold down points for harped strands.

The beam is divided into 53 pieces in longitudinal direction. Every cross-section of these 52 pieces is divided into 1372 pieces resulting in 72716 elements. For all small elements, stresses are calculated as if the strands weren't harped. Prestressing losses are calculated using

MATLAB using the procedure shown above. The MATLAB code is available in the appendix of this chapter.

For straight strands, entirety of the beam was within limits of compression allowed at transfer. However, as expected, the top ends of the beam exceeded the tensile stress limit allowed by CSA S6-14 rev. 17.

At transfer, the tensile stress in concrete cannot exceed:

$$f_{\text{tensile allowed}} = 0.25 \times \sqrt{35} = 1.479\ MPa$$

Figure below shows in red where tensile stress exceeds 1.479 MPa. The green elements are within limits of stress.

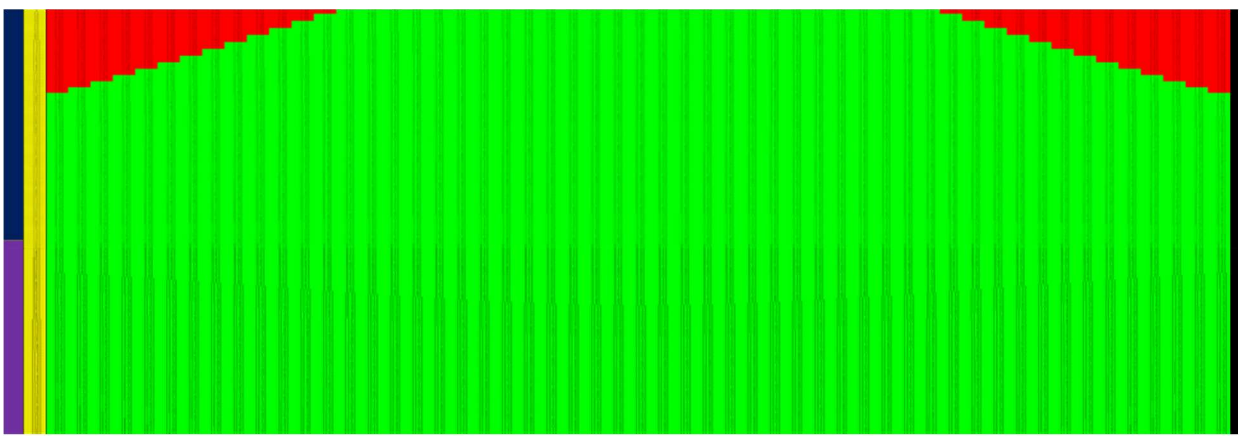


Figure 5.3.3.1.1 – Straight Strands – Stresses experienced

Looking at the stress values, optimal hold down points determined to be x = 8.5 m and x = 17.5 m from left support.

The strand profile below is determined to give the best stress results (32 12.7 mm strands with the arrangement and pattern below):

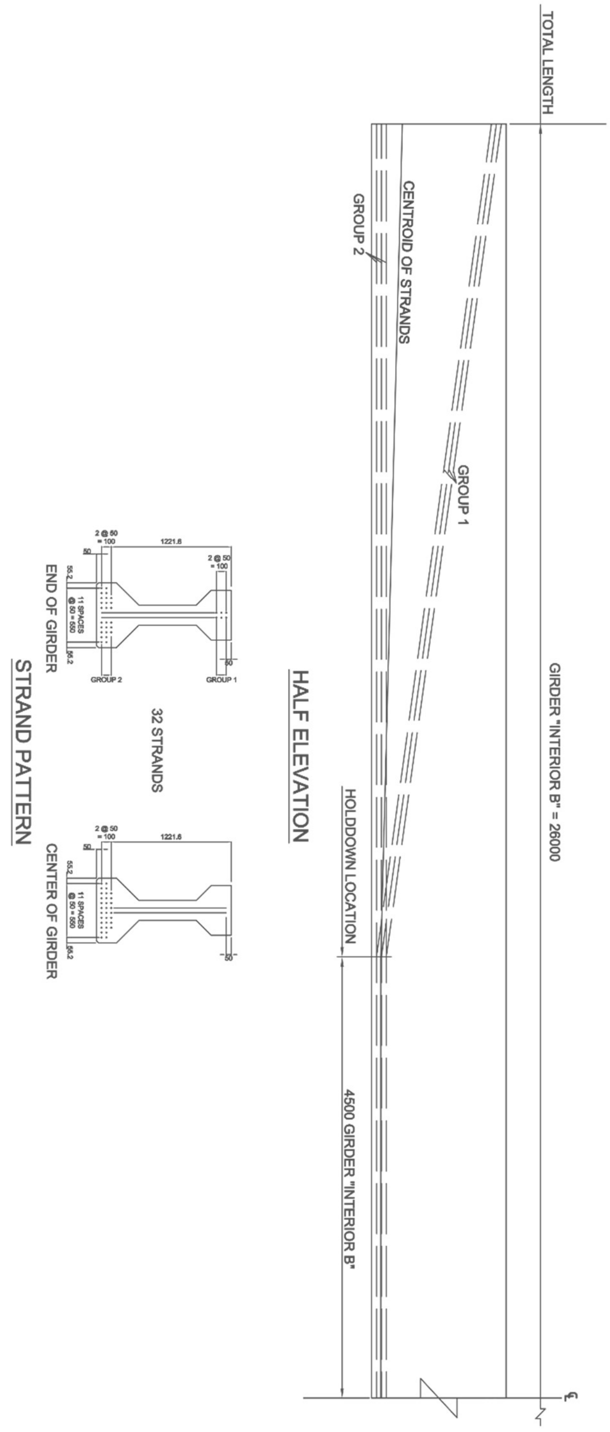


Figure 5.3.3.1.2 – Strand Pattern

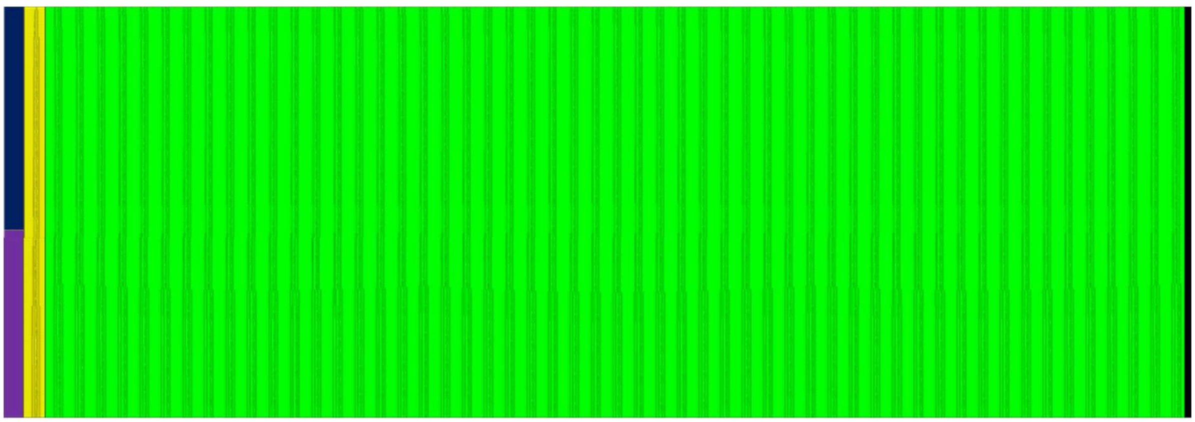


Figure 5.3.3.1.3 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced

Maximum stresses recorded at transfer are:

$0.769\ MPa\ for\ tension < 0.25 \times \sqrt{35}\ (1.479\ MPa)\ OK$

$15.18\ MPa\ for\ compression < 0.6 \times 35\ (21\ MPa)\ OK$

Table 5.3.3.1.1 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced [-Compression, +Tension]

Distance From Left Support	Maximum Top Stress (MPa)	Maximum Bottom Stress (MPa)
0	0.77	-15.18
0.5	0.58	-15.02
1	0.41	-14.88
1.5	0.26	-14.75
2	0.14	-14.65
2.5	0.03	-14.56
3	-0.05	-14.49
3.5	-0.11	-14.44
4	-0.15	-14.40
4.5	-0.17	-14.39
5	-0.17	-14.39
5.5	-0.14	-14.41
6	-0.10	-14.45
6.5	-0.03	-14.50
7	0.06	-14.58
7.5	0.17	-14.67
8	0.30	-14.78
8.5	0.45	-14.91
9	0.27	-14.76
9.5	0.11	-14.62
10	-0.03	-14.50
10.5	-0.15	-14.41
11	-0.24	-14.32
11.5	-0.32	-14.26
12	-0.37	-14.22
12.5	-0.40	-14.19
13	-0.41	-14.18
13.5	-0.40	-14.19
14	-0.37	-14.22
14.5	-0.32	-14.26
15	-0.24	-14.32
15.5	-0.15	-14.41
16	-0.03	-14.50
16.5	0.11	-14.62
17	0.27	-14.76
17.5	0.45	-14.91
18	0.30	-14.78
18.5	0.17	-14.67
19	0.06	-14.58
19.5	-0.03	-14.50
20	-0.10	-14.45
20.5	-0.14	-14.41
21	-0.17	-14.39
21.5	-0.17	-14.39
22	-0.15	-14.40
22.5	-0.11	-14.44
23	-0.05	-14.49
23.5	0.03	-14.56
24	0.14	-14.65
24.5	0.26	-14.75
25	0.41	-14.88
25.5	0.58	-15.02
26	0.77	-15.18

MAXIMUM TENSION	=	0.77
MAXIMUM COMPRESSION	=	-15.18

5.3.3.2 Service conditions

Midspan:

At service, the compressive stress in top fiber cannot exceed:

$$f_{ts} = 0.45 \times 40 = \mathbf{18}\ \boldsymbol{MPa}$$

$$P_{pe}\ @\ midspan = 3464.5\ kN$$

$$f_{ts} \geq \frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c}{s_t} + \frac{M_G + M_S}{s_t} + \frac{M_{SDL} + 0.9 \times M_{LL}}{\frac{I_c}{(y_{tc} - 200)}}$$

$$f_{ts} = \frac{3464.5 \times 10^3}{5.0903 \times 10^5} - \frac{3464.5 \times 10^3 \times 534.4895}{1.46 \times 10^8} + \frac{(1053.82 + 1014) \times 10^6}{1.46 \times 10^8}$$

$$+ \frac{(322.68 + 0.9 \times 1871.71) \times 10^6}{\frac{2.896 \times 10^{11}}{(525.4544 - 200)}} = \mathbf{10.5482}\ \boldsymbol{MPa\ OK}$$

At service, the tensile stress in the bottom fiber connot exceed:

$$f_{bs} = 0.50 \times \sqrt{40} = \mathbf{3.162}\ \boldsymbol{MPa}$$

$$f_{bs} \geq -\frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c}{s_b} + \frac{M_G + M_S}{s_b} + \frac{M_{SDL} + 0.9 \times M_{LL}}{s_{bc}}$$

$$f_{bs} = -\frac{3464.5 \times 10^3}{5.0903 \times 10^5} - \frac{3464.5 \times 10^3 \times 534.4895}{1.7275 \times 10^8} + \frac{(1053.82 + 1014) \times 10^6}{1.7275 \times 10^8}$$

$$+ \frac{(322.68 + 0.9 \times 1871.71) \times 10^6}{2.7683 \times 10^8} = \mathbf{1.6957}\ \boldsymbol{MPa\ OK}$$

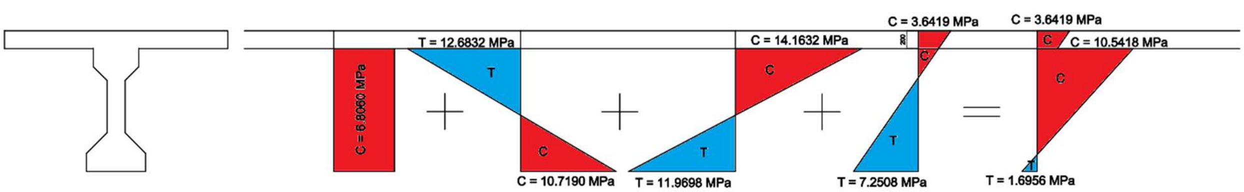


Figure 5.3.3.2.1 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced visualized at midspan

This same procedure is done for every 0.5 m of span and top and bottom stresses are determined using excel. This will serve to verify the safety of stresses experienced when the strand pattern in figure 5.3.3.1.2 is used. Although unnecessary at this point, for every small 72176 element, stresses are also calculated.

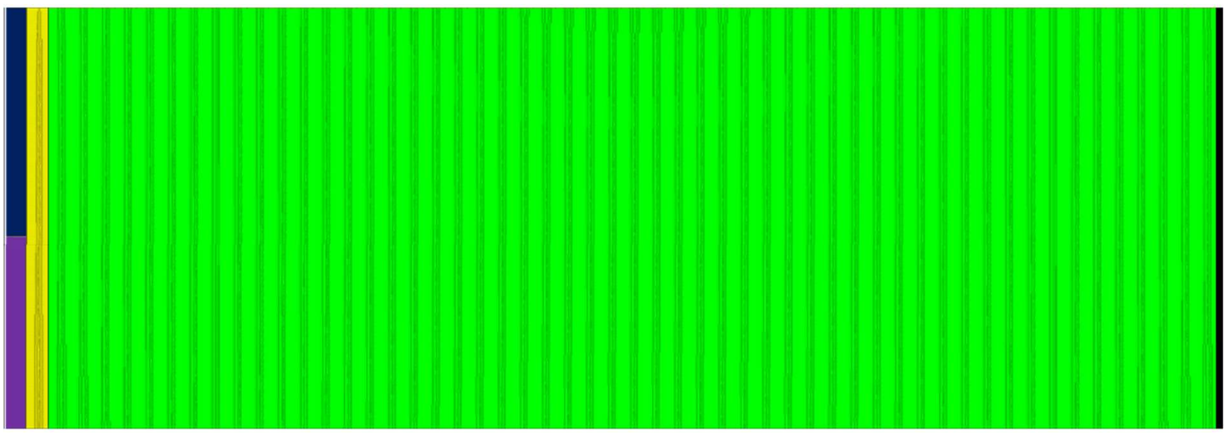


Figure 5.3.3.2.2 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced at service conditions

Maximum stresses recorded at service are:

$1.72\,MPa$ *for tension* $< 0.5 \times \sqrt{40}\,(3.162\,MPa)$ *OK*

$13.11\,MPa$ *for compression* $< 0.45 \times 40\,(18\,MPa)$ *OK*

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Table 5.3.3.2.1 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced by the girder during service [-Compression, +Tension]

Distance From Left Support	Maximum Top Stress (MPa)	Maximum Bottom Stress (MPa)
0	0.66	-13.11
0.5	-0.29	-11.84
1	-1.20	-10.64
1.5	-2.05	-9.50
2	-2.85	-8.43
2.5	-3.60	-7.43
3	-4.30	-6.50
3.5	-4.95	-5.63
4	-5.56	-4.78
4.5	-6.12	-4.00
5	-6.62	-3.28
5.5	-7.08	-2.63
6	-6.31	-0.89
6.5	-7.83	-1.55
7	-8.12	-1.11
7.5	-8.37	-0.73
8	-8.58	-0.39
8.5	-8.73	-0.12
9	-9.13	0.33
9.5	-9.48	0.72
10	-9.78	1.04
10.5	-10.03	1.28
11	-10.22	1.46
11.5	-10.37	1.58
12	-10.48	1.69
12.5	-10.54	1.72
13	-10.54	1.69
13.5	-10.54	1.72
14	-10.48	1.69
14.5	-10.37	1.58
15	-10.22	1.46
15.5	-10.03	1.28
16	-9.78	1.04
16.5	-9.48	0.72
17	-9.13	0.33
17.5	-8.73	-0.12
18	-8.58	-0.39
18.5	-8.37	-0.73
19	-8.12	-1.11
19.5	-7.83	-1.55
20	-7.48	-2.06
20.5	-7.08	-2.63
21	-6.62	-3.28
21.5	-6.12	-4.00
22	-5.56	-4.78
22.5	-4.95	-5.63
23	-4.30	-6.50
23.5	-3.60	-7.43
24	-2.85	-8.43
24.5	-2.05	-9.50
25	-1.20	-10.64
25.5	-0.29	-11.84
26	0.66	-13.11

MAXIMUM TENSION	=	1.72
MAXIMUM COMPRESSION	=	-13.11

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5.3.4 Ultimate Flexural Capacity

Ultimate flexural capacity of the composite section can be calculated in two ways.

The first and most commonly used method that works for every section is strain compatibility analysis. In this method, the section is divided into small rectangles and stresses are assumed constant throughout the small rectangle. Each of the rectangle will have a resultant force. The moment caused by all resultant forces are assembled into 1 compressive force with a certain distance from the centroid. Equating tensile force at the level of center of gravity of steel with this compressive force gives the magnitude of the compressive force. Ultimate moment capacity (Mr) is then determined by multiplying tensile or compressive force by the moment arm.

The concrete stress-strain curve used for the strain compatibility analysis presented in this report is based on the Hognestad's Modified Parabola. The prestressing steel and concrete stress-strain curve is given in the chapter 2 of this report.

Another way that is simpler and gives good enough results for most sections is assuming a rectangular stress pattern (Whitney's Stress Block). However, in CSA S6-14 rev. 17, this rectangular block parameters are different then the Whitney's Stress Block, but the concept is similar. It is still required to iterate to find for the location of compressive force with this method if the centroid of compressive forces is not in a rectangular section.

So, both ways, the usage of a computer program is very helpful.

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5.3.4.1 Rectangular Stress Block Assumption

RECTANGULAR SECTION ASSUMPTION AT MIDSPAN

→ Flexural demand at midspan, $M_f = 6147,31$ kNm (From chapter 3)

According to CSA S6-14 rev. 17:

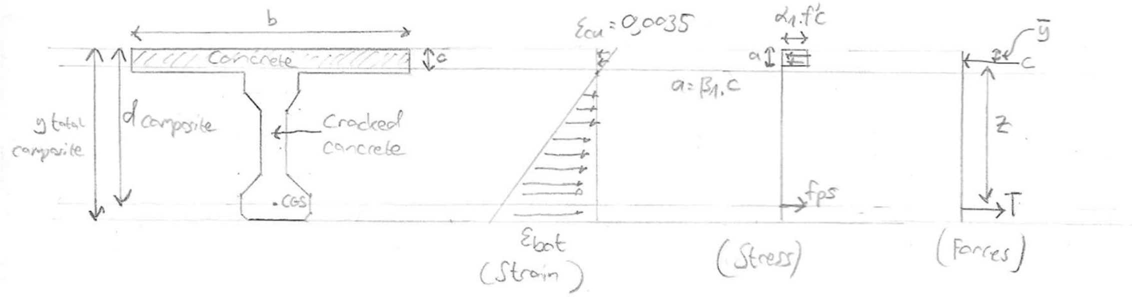
Stress block parameters:

$\alpha_1 = 0,85 - 0,0015 \cdot f'c = 0,79$

$\beta_1 = 0,97 - 0,0025 \cdot f'c = 0,87$

$\phi_p = 0,95$ $\quad f_{pu} = 1860$ MPa

$\phi_c = 0,75$ $\quad f'c = 40$ MPa



a) Without reduction factors ϕ_p, ϕ_c:

$$a = \frac{A_{ps} \cdot f_{ps}}{\alpha_1 \cdot f'c \cdot b} = \frac{98,7 \cdot 32 \cdot f_{ps}}{0,79 \cdot 40 \cdot 2500} = 0,04 \cdot f_{ps}$$

$$\frac{c_u}{d_p} = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot \beta_1 \cdot f'c \cdot b \cdot d} = \frac{98,7 \cdot 32 \cdot 1860}{0,79 \cdot 0,87 \cdot 40 \cdot 2500 \cdot 1477,8} = 0,05784$$

$$k_p = 3\left(1 - \underbrace{\frac{f_{py}}{f_{pu}}}_{0,9}\right) = 3 \cdot (1 - 0,9) = 0,3 \text{ or } 0,28 \text{ for low relaxation}$$

both OK

$$f_{ps} = f_{pu}\left(1 - k_p \cdot \frac{c_u}{d_p}\right) = 1860 \cdot (1 - 0,28 \cdot 0,05784) = 1830 \text{ MPa}$$

$$a = 0,04 \cdot f_{ps} = 0,04 \cdot 1830 = 73,2 \text{ mm}$$

$a < t_s$ (200 mm) ∴ Rectangular assumption can be used.

$$M_n = \underbrace{A_{ps} \cdot f_{ps}}_{T} \cdot \underbrace{\left(d - \frac{a}{2}\right)}_{z} = 98,7 \cdot 32 \cdot \left(1477,8 - \frac{73,2}{2}\right) \cdot 1830 \cdot 10^{-6}$$

$M_n = 8330$ kNm > 6147,31 kNm OK

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b) Using reduction factors ϕ_p, ϕ_c :

$$a = \frac{A_{ps} \cdot f_{ps} \cdot \phi_p}{\alpha_1 \cdot f'_c \cdot b \cdot \phi_c} = \frac{98{,}7 \cdot 32 \cdot 0{,}95 \cdot f_{ps}}{0{,}79 \cdot 40 \cdot 2500 \cdot 0{,}75} = 0{,}05064 \cdot f_{ps}$$

$$\frac{c_u}{d_p} = \frac{A_{ps} \cdot f_{pu} \cdot \phi_p}{\alpha_1 \cdot \beta_1 \cdot f'_c \cdot b \cdot d \cdot \phi_c} = \frac{98{,}7 \cdot 32 \cdot 1860 \cdot 0{,}95}{0{,}79 \cdot 0{,}87 \cdot 40 \cdot 2500 \cdot 1477{,}8 \cdot 0{,}75} = 0{,}07326$$

$k_p = 0{,}28$ for low-relaxation strands.

$$f_{ps} = f_{pu} \cdot \left(1 - k_p \cdot \frac{c_u}{d_p}\right) = 1860 \cdot \left(1 - 0{,}28 \cdot 0{,}07326\right) = 1822 \text{ MPa}$$

$a = 0{,}05064 \cdot f_{ps} = 92{,}3$ mm $a < t_s$ (200 mm) ∴ Rectangular Assumption can be used

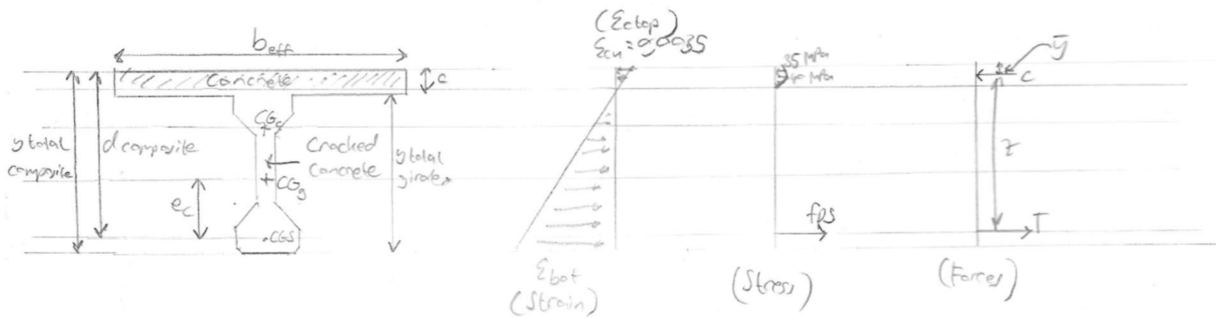
$$M_r = A_{ps} \cdot f_{ps} \cdot \left(d - \frac{a}{2}\right) \cdot \phi_p = 98{,}7 \cdot 32 \cdot 1822 \cdot \left(1477{,}8 - \frac{92{,}3}{2}\right) \cdot 0{,}95 \cdot 10^{-6}$$

$M_r = 7827$ kNm $> 6147{,}31$ kNm Ok

5.3.4.2 Strain-compatibility analysis

STRAIN - COMPATIBILITY ANALYSIS AT MIDSPAN

→ Flexural demand at midspan, $M_f = 6147{,}31$ kNm (from chapter 3)



$E_{ps} = 200000$ MPa

$E_c = (3000 \cdot \sqrt{40} + 6900)\left(\frac{2500}{2300}\right)^{1.5} = 29321$ MPa

$A_{ps} = 98{,}7 \cdot 32 = 3158{,}4\ mm^2$

$A_g = 509031{,}24\ mm^2$

A_{gt} (Transformed area of the prestressing girder) $= 528487{,}6385\ mm^2$

y total $= 1371{,}6\ mm^2$

y total composite $= 1571{,}6\ mm^2$

$d = 1277{,}8$ mm

d composite $= 1477{,}8$ mm

$e_c = 534{,}4895$ mm

$y_t = 743{,}3605$ mm

$y_b = 628{,}2395$ mm

$I_g = 1{,}0853 \cdot 10^{11}\ mm^4$

$I_{gt} = 1{,}1391 \cdot 10^{11}\ mm^4$

$P_{pe} = 3464{,}5$ kN

$\varepsilon_{ctop} = \varepsilon_{cu} = 0{,}0035$

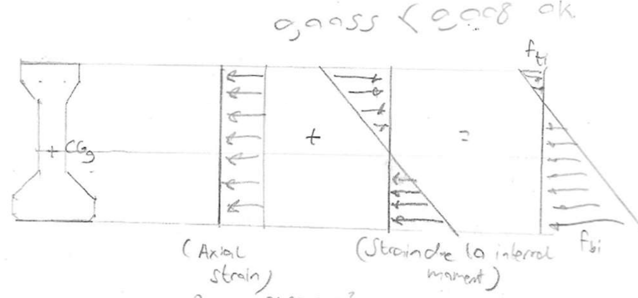
$\varepsilon_o = 0{,}002$

$f'_c = 40$ MPa

Initial prestressing strain in the steel due to P_{pe}.

$$\varepsilon_{si} = \frac{P_{pe}}{A_{ps} \cdot E_{ps}} = \frac{3464{,}5 \cdot 10^3}{3158{,}4 \cdot 200000} = 0{,}0055$$

$0{,}0055 < 0{,}008$ ok



$$\frac{P_{pe}}{A_{gt} \cdot E_{ps}} = \frac{3464{,}5 \cdot 10^3}{528487{,}6385 \cdot 200000} = -3{,}2777 \cdot 10^{-5}$$

$$\varepsilon_{ti_p} = -\frac{P_{pe}}{A_{gt} \cdot E_{ps}} + \frac{P_{pe} \cdot e_c \cdot y_t}{I_{gt} \cdot E_{ps}} = -3{,}2777 \cdot 10^{-5} + \frac{3464{,}5 \cdot 10^3 \cdot 534{,}4895 \cdot 743{,}3605}{1{,}1391 \cdot 10^{11} \cdot 200000} = 2{,}7643 \cdot 10^{-5}$$

$$\varepsilon_{bi_p} = -\frac{P_{pe}}{A_{gt} \cdot E_{ps}} - \frac{P_{pe} \cdot e_c \cdot y_b}{I_{gt} \cdot E_{ps}} = -3{,}2777 \cdot 10^{-5} - \frac{3464{,}5 \cdot 10^3 \cdot 534{,}4895 \cdot 628{,}2395}{1{,}1391 \cdot 10^{11} \cdot 200000} = -8{,}3841 \cdot 10^{-5}$$

$$\varepsilon_{cip} = \varepsilon_{tip} + (\varepsilon_{bip} - \varepsilon_{tip}) \cdot d \cdot \frac{1}{y_{total}} = 2{,}7643 \cdot 10^{-5} + (-8{,}3841 \cdot 10^{-5} - 2{,}7643 \cdot 10^{-5}) \cdot 1277{,}8 \cdot \frac{1}{1371{,}6} = -7{,}6221 \cdot 10^{-5}$$

Assuming unshored construction

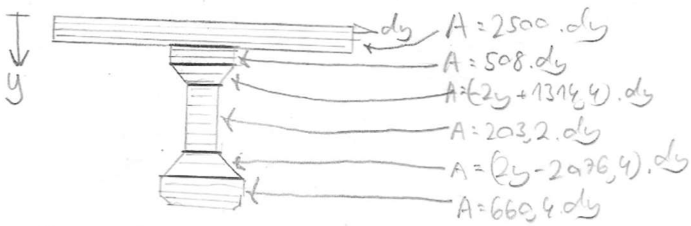
$$\varepsilon_{ti_{DL}} = -\frac{(M_G + M_S) \cdot y_t}{I_{gt} \cdot E_{ps}} = -\frac{(1053{,}82 + 1014) \cdot 10^6 \cdot 743{,}3605}{1{,}1391 \cdot 10^{11} \cdot 200000} = -6{,}7472 \cdot 10^{-5}$$

$$\varepsilon_{bi_{DL}} = \frac{(M_G + M_S) \cdot y_b}{I_{gt} \cdot E_{ps}} = \frac{(1053{,}82 + 1014) \cdot 10^6 \cdot 628{,}2395}{1{,}1391 \cdot 10^{11} \cdot 200000} = 5{,}7023 \cdot 10^{-5}$$

$$\varepsilon_{ci_{DL}} = \frac{\overbrace{d - y_t}^{e_c}}{y_b} \cdot \varepsilon_{bi_{DL}} = \frac{534{,}4895}{628{,}2395} \cdot 5{,}7023 \cdot 10^{-5} = 4{,}8113 \cdot 10^{-5}$$

$$\varepsilon_{ci} = |\varepsilon_{cip}| + \varepsilon_{c_{DL}} = 1{,}2473 \cdot 10^{-4}$$

* Area is divided into rectangles with height 0,01 m or 1cm ($dy = 0{,}01$ for numerical integration)



$$\varepsilon_s = \varepsilon_{si} + \varepsilon_{ci} + \varepsilon_{ctop} \cdot \frac{d_{composite} - c}{c}$$

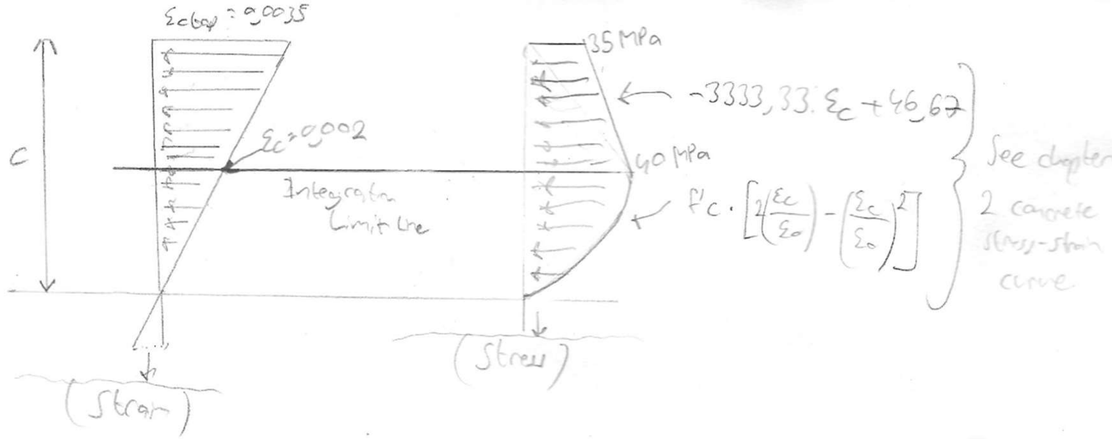
$$\varepsilon_s = 0{,}0055 + 1{,}2473 \cdot 10^{-4} + 0{,}0035 \cdot \frac{1477{,}8 - c}{c}$$

$$f_{ps} = E_{ps} \cdot \varepsilon_s = 200000 \cdot \varepsilon_s \quad \text{for } \varepsilon_s \leq 0{,}008$$

$$f_{ps} = 1848 - 0{,}517 \cdot \frac{1}{\varepsilon_s - 0{,}005915} \quad \text{for } \varepsilon_s > 0{,}008$$

$$f_{ps} = \alpha \quad \text{for } f_{ps} > 0{,}95 \cdot f_{pu}$$

* See chapter 2 Prestressing Steel Stress-Strain curve



$$\text{Integration Limit} = -(0{,}002 - \varepsilon_{ctop}) \cdot \frac{c}{\varepsilon_{ctop}} = -(0{,}002 - 0{,}0035) \cdot \frac{c}{0{,}0035}$$

$$= 0{,}429\,c \text{ from top}$$

$$\varepsilon_c = \varepsilon_{ctop} - \varepsilon_{ctop} \cdot \frac{y}{c}$$

$T = f_{ps} \cdot A_{ps}$

$$C = \int_0^{0{,}429c} (-3333{,}33 \cdot \varepsilon_c + 46{,}67) \cdot A + \int_{0{,}429c}^{c} 40 \cdot \left[2 \cdot \left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right] \cdot A$$

* Solving $T = C$ numerically using MATLAB, $c = 74{,}26$ mm (code available in the appendix section)

Let $c = 74{,}26$ mm

$$\varepsilon_s = 0{,}0055 + 1{,}2473 \cdot 10^{-4} + 0{,}0035 \cdot \frac{1477{,}8 - 74{,}26}{74{,}26} = 0{,}0718$$

$$\varepsilon_s > 0{,}008$$

$$\therefore f_{ps} = 1848 - 0{,}517 \cdot \frac{1}{0{,}0718 - 0{,}005915} = 1840{,}1 \text{ MPa}$$

$$1840{,}1 \text{ MPa} < 0{,}99 \cdot f_{pu} \text{ OK}$$
$$(1841{,}4)$$

$$\text{Integration Limit} = 0{,}429\,c = 31{,}83 \text{ mm}$$

$$T \cong C = f_{ps} \cdot A_{ps} = 1840{,}1 \cdot 3158{,}4 = 5{,}812 \cdot 10^6 \text{ N}$$

$$\bar{y} = \frac{\int_0^{0,429c}(-3333,33 \cdot \varepsilon_c + 46,67) \cdot A \cdot y + \int_{0,429c}^{c} 40 \cdot \left[2 \cdot \left(\frac{\varepsilon_c}{0,002}\right) - \left(\frac{\varepsilon_c}{0,002}\right)^2\right] \cdot A \cdot y}{T}$$

$$\varepsilon_c = 0,0035 - 0,0035 \cdot \frac{y}{74,26}$$

$$\bar{y} = \frac{\left(\int_0^{31,83}\left(-3333,33 \cdot \left(0,0035 - 0,0035 \cdot \frac{y}{74,26}\right) + 46,67\right) \cdot 2500 \cdot y \cdot dy + \int_{31,83}^{74,26} 40 \cdot \left[2 \cdot \left(\frac{\left(0,0035 - 0,0035 \cdot \frac{y}{74,26}\right)}{0,002}\right) - \left(\frac{\left(0,0035 - 0,0035 \cdot \frac{y}{74,26}\right)}{0,002}\right)^2\right] \cdot 2500 \cdot y \cdot dy\right)}{(5,812 \cdot 10^6)}$$

$$\bar{y} = \frac{1,8358 \cdot 10^8}{5,812 \cdot 10^6} = 31,5861 \text{ mm}$$

$$z = d_{composite} - \bar{y} = 1477,9 - 31,5861 = 1446,3 \text{ mm}$$

$$M_r = z \cdot T \cdot 10^{-6} = 1446,3 \cdot 5,812 = 8406 \text{ kNm} \pm 0,5 \text{ kNm}$$

$$\boxed{8406 \text{ kNm} > 6147,31 \text{ kNm} \quad \text{OK}}$$

5.3.5 Reserve capacity

Moment resistance of the section at ultimate must be at least 1.2 times more than the cracking moment of the section. The reserve capacity check requirement can be waived if it is proven that the section has 1.33 times more moment resistance than the factored demand at ultimate.

The maximum moment experienced at ultimate is at 12.5 *m and* 13.5 *m from left support. It is equal to* 6170.34 *kNm*.

The moment reistance obtained by stain compatibility is 8406 *kNm*.

$$1.33 \times 6170.34 = \mathbf{8227.12}\ kNm < 8406\ kNm$$

Therefore this requirement can be waived.

However, for the purposes of this report, it will be checked:

At service and at midspan:

$$f_b = -\frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c \times y_b}{I_g} + \frac{(M_g + M_s) \times y_b}{I_g} + \frac{(M_{SDL} + 0.9 \times M_{LL}) \times y_{bc}}{I_c}$$

$$f_b = -\frac{3464.5 \times 10^3}{5.0903 \times 10^5} - \frac{3464.5 \times 10^3 \times 534.4895 \times 628.2395}{1.0853 \times 10^{11}}$$

$$+\frac{(1053.82 + 1014) \times 10^6 \times 628.2395}{1.0853 \times 10^{11}} + \frac{(322.68 + 0.9 \times 1684.53) \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = \mathbf{1.6956}\ MPa\ T$$

At cracking, the bottom stress $= 0.4 \times \sqrt{f'_c} = 0.4 \times \sqrt{40} = \mathbf{2.5298}\ MPa\ T$

The additional moment must create a bottom stress of $2.5298 - 1.6956 = \mathbf{0.8342}\ MPa\ T$

$$\frac{M_{add} \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = 0.8342,\ \textit{solving for}\ M_{add},\ M_{add} = \mathbf{230.92}\ kNm$$

Therefore, $M_{cr} = 230.92 + (322.68 + 0.9 \times 1684.53) + (1053.82 + 1014) = \mathbf{4306}\ kNm$

$$1.2 \times 4306\ kNm = 5167.1\ kNm < 8406\ kNm\ OK$$

5.3.6 Deflection limits check

During service and initial stage, the beam is under linear stresses with respect to the strains experienced. Therefore, most of the equations given here are for first order linear-elastic analysis.

The deflections experienced in ultimate stage is not the main concern of the design since the bridge is expected to never reach ultimate loading unless some extraordinary, extreme event happens. Nevertheless, the deflection is checked using stain-compatibility together with finite-element analysis. The ultimate deflections will not be presented in this report.

Deflections due to shear deformations are ignored in this report.

SIMPLY SUPPORTED BRIDGE DEFLECTION AND MAXIMUM DEFLECTION

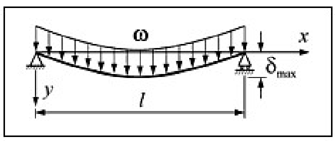
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	$y = \frac{\omega x}{24EI}\left(l^3 - 2lx^2 + x^3\right)$	$\delta_{max} = \frac{5\omega l^4}{384EI}$

Figure 5.3.6.1 – Deflection equations for UDL on a simply supported beam

SIMPLY SUPPORTED DEFLECTION AND MAXIMUM DEFLECTION

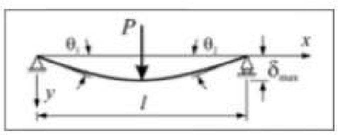
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	$y = \frac{Px}{12EI}\left(\frac{3l^2}{4} - x^2\right)$ for $0 < x < \frac{l}{2}$	$\delta_{max} = \frac{Pl^3}{48EI}$

Figure 5.3.6.2 – Deflection equations for point load at midspan on a simply supported beam

Immediate deflection due to live load:

Immediate deflection due to live load can be calculated from the deflection occuring when applying the truck load at the midspan of the interior girder as a single point load. This simple method will give conservative results. If the deflection obtained is within the critical range, then the distribution and impact factors can be taken into account.

$$\Delta_L = \frac{P \times L^3}{48 \times E_c \times I_c} = \frac{625000 \times 26000^3}{48 \times 27098 \times 2.8960 \times 10^{11}} = 27\ mm\ downwards$$

Erection deflections:

Elastic Deflection due to girder self – weight:

$$\Delta_{DL} = \frac{5 \times w_G \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 12.47 \times 26000^4}{384 \times 27098 \times 1.0853 \times 10^{11}} = 23\ mm\ downwards$$

Elastic Deflection due to deck:

$$\Delta_{SL} = \frac{5 \times w_S \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 12 \times 26000^4}{384 \times 27098 \times 1.0853 \times 10^{11}} = 22\ mm\ downwards$$

Elastic Deflection due to asphalt and waterproofing:

$$\Delta_{PL} = \frac{5 \times w_{SDL} \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 3.82 \times 26000^4}{384 \times 27098 \times 1.0853 \times 10^{11}} = 7\ mm\ downwards$$

Upward Elastic Deflection due to Camber:

There are many different methods to calculate Camber. Camber calculations can be done using the " Hyperbolic Functions Method" proposed by Sinno Rauf and Howard L Furr (1970) or using the PCI s equations. However, in this report, camber is calulated using the approximate equations proposed by Collins and Mitchell.

$$\Delta_c = \left(\frac{e_c}{8} - \beta^2 \times \frac{(e_c - e_e)}{6}\right) \times P_{pi} \times \frac{L^2}{(E_c \times I_g)}$$

where:

β *= Ratio of harping length at one end with respect to total length*

e_e *= Average eccentricity at girder ends*

$$\beta = \frac{8.5}{26} = 0.327$$

Between 0 and 8.5 m from left support, the center of gravity of steel is given by this equation:

$$Distance\ from\ bottom\ to\ CGS\ [mm] = -\frac{313.425 - 93.75}{8500} \times Dist\ from\ left\ supp. + 313.425$$

Therefore CGS @ 0 m = 313.425 mm from bottom

$e_e = 628.2395 - 313.425 = 314.8145\ mm$

$$\Delta_c = \left(\frac{534.4895}{8} - 0.327^2 \times \frac{(534.4895 - 314.814)}{6}\right) \times 4010.4 \times 10^3 \times \frac{26000^2}{(27098 \times 1.0853 \times 10^{11})}$$

$\Delta_c = 58\ mm\ upwards$

Total deflection at erection $= 1.85 \times \Delta_{DL} + 1.8 \times \Delta_c = 1.85 \times 23 + 1.8 \times -58 =$ **62 mm upwards**

Total long term deflection $= 2.4 \times \Delta_{DL} + 2.2 \times \Delta_c + 2.3 \times \Delta_{SL} + 3 \times \Delta_{PL}$

Total long term deflection $= 2.4 \times 23 + 2.2 \times -58 + 2.3 \times 22 + 3 \times 7 =$ **1 mm upwards**

All deflections are within limit of $\frac{1}{1000}$ *so this design is safe.*

FLEXURAL DESIGN NOW COMPLETE

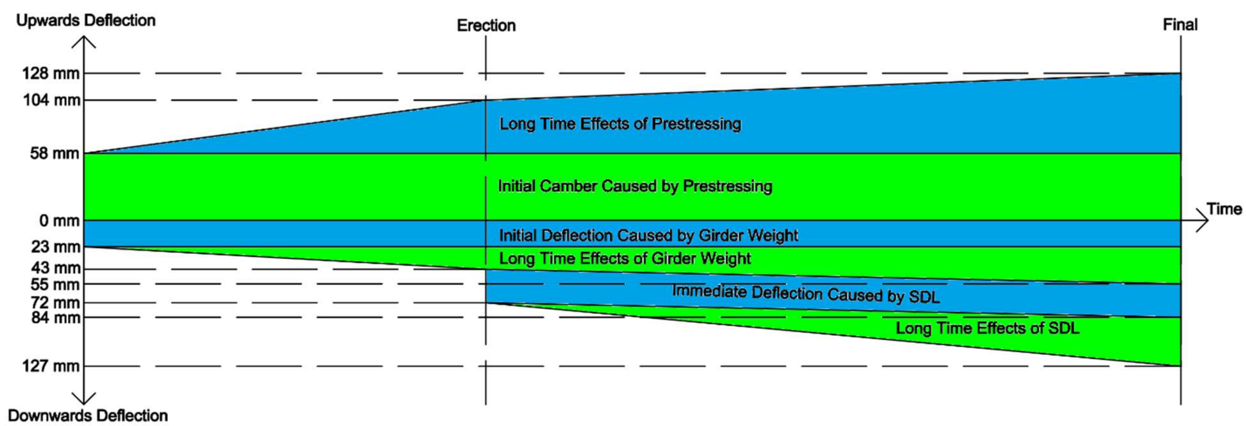


Figure 5.3.6.3 – Visual Representation of the Deflections Experienced

5.3.7 Design for shear and anchorage zone

For shear design, 15 M Canadian reinforcement bars with 16 mm diameter will be used. Each bar will therefore have an area of 200 mm^2, and 400 mm^2 when bent to be double legged. Ultimate shear values from chapter 4 must be used for the shear design

<u>Determination of V_p:</u>

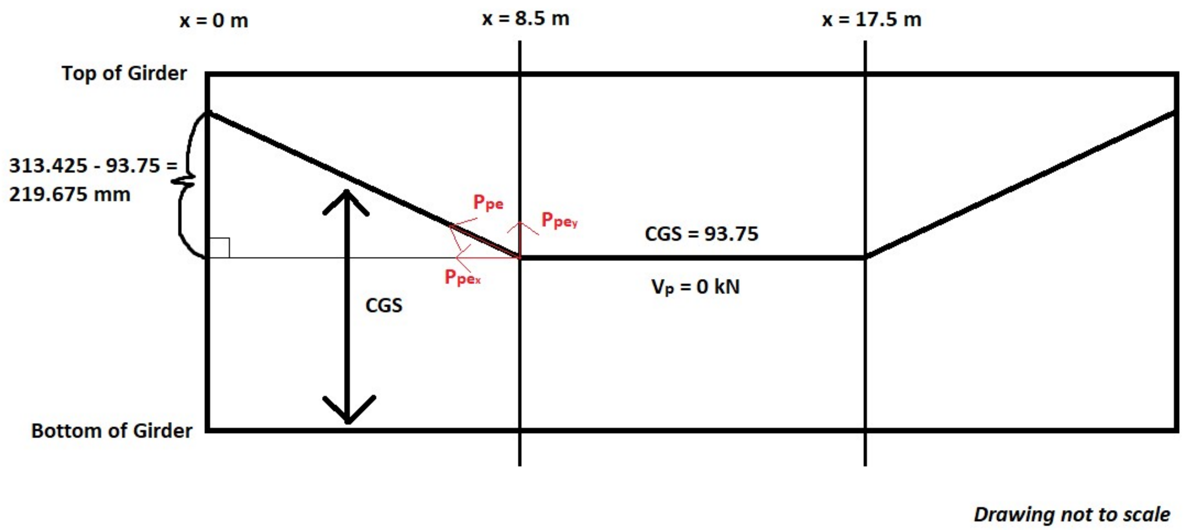


Figure 5.3.7.1 – Calculation of V_p

Determination of V_p (Component of effective prestressing force after all losses in the direction of applied shear. Positive if resisting shear, negative if adding to shear experienced):

$\boldsymbol{V_p}$ ***from x = 0 m to x = 8.5 m and x = 17.5 m to x = 26 m***

From figure above, P_{pe_y} is equal to V_p. Using the triangle from figure above

$$V_p = P_{pe} \times \sin\left(\arctan\left(\frac{219.675}{8500}\right)\right) = \mathbf{89.5\ kN}\ (P_{pe}\ \text{was}\ 3464.5\ kN)$$

Determination of equivalent cracking parameter S_{ze}:

S_{ze} *can be taken as* 300 *mm as long as minimum shear reinforcement is provided.*

$$\boldsymbol{S_{ze} = 300\ mm}$$

Determination of the longitudinal strain at the centroidal axis of the critical section ε_x:

$$\varepsilon_x = \frac{\dfrac{M_f}{d_v} + V_f - V_p + 0.5 \times N_f - A_{ps} \times f_{po}}{2 \times \left(E_s \times A_s + E_p \times A_{ps}\right)}$$

where:

$$M_f \geq \left(V_f - V_p\right) \times d_v$$

$$f_{po} = 0.7 \times f_{pu}$$

$$-0.0002\ (\textit{Conditionally}) \leq \varepsilon_x \leq 0.003$$

At midspan:

$$\boldsymbol{\varepsilon_x = 0.001144}$$

Determination of the angle of inclination of the compressive stresses and value of beta:

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

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

At midspan:

$$\boldsymbol{\theta} = (29 + 7000 \times 0.001144) \times \left(0.88 + \frac{300}{2500}\right) = \mathbf{37.01}\ \textbf{\textit{degrees}}$$

$$\boldsymbol{\beta} = \frac{0.4}{(1 + 1500 \times 0.001144)} \times \frac{1300}{(1000 + 300)} = \mathbf{0.15}$$

Determination of the shear stress that can be resisted by concrete alone:

$$V_c = 2.5 \times \beta \times \Phi_c \times f_{cr} \times b_v \times d_v$$

At midspan:

$$\boldsymbol{V_c} = 2.5 \times 0.15 \times 0.75 \times 0.4 \times \sqrt{40} \times 203.2 \times 1150.07 \times 10^{-3} = \mathbf{163.25\ kN}$$

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Determination of the shear stress that must be resisted by the reinforcement:

$$V_s = V_f - V_p - V_c \geq 0$$

At midspan:

$$\mathbf{V_s} = 212.45 - 0 - 163.25 = \mathbf{49.21\ kN}$$

Determination of the shear reinforcement spacing if $V_s > 0$:

$$s_{required} = \frac{\Phi_s \times A_v \times f_y \times d_v \times cot(\theta)}{V_s}$$

At midspan:

$$\mathbf{s_{required}} = \frac{0.9 \times 400 \times 400 \times 1150.07 \times cot(37.01)}{49.21 \times 10^3} = \mathbf{4464.75\ mm}$$

Determination of maximum shear reinforcement spacing:

$$If\ V_f \leq (0.1 \times \Phi_c \times f'_c \times b_v \times d_v + V_p)$$

$$-s_{max} = Lesser\ of\ 600\ mm\ or\ (0.75 \times d_v)$$

$$If\ V_f > (0.1 \times \Phi_c \times f'_c \times b_v \times d_v + V_p)$$

$$-s_{max} = Lesser\ of\ 300\ mm\ or\ (0.33 \times d_v)$$

At midspan:

$$V_f = 212.45\ kN$$

$$212.45 < 701.08\ (0.1 \times 0.75 \times 40 \times 203.2 \times 1150.07 \times 10^{-3})$$

$$-s_{max} = Lesser\ of\ 600\ mm\ or\ 862.5\ (0.75 \times 1150.07)$$

Therefore maximum spacing can be 600 mm

Determination of minimum shear reinforcement area:

$$A_{v,min} = \frac{0.15 \times f_{cr} \times b_v \times s}{f_y}$$

At midspan:

$$A_{v,min} = \frac{0.15 \times 0.4 \times \sqrt{40} \times 203.2 \times 600}{400} = 115.66\ mm^2$$

Provide minimum 1 double legged 15 M bar with $A_s = 400\ mm^2$ per spacing

K & B

Determination of anchorage zone reinforcement design for pretensioned members:

$-Stirrups\ with\ a\ total\ area\ of\ at\ least\ \dfrac{0.08 \times F_p}{\Phi_s \times f_y}\ must\ be\ distributed\ over\ a\ distance\ of\ 0.25 \times h$

$F_p = 5811.93\ kN\ from\ strain-compatibility\ analysis.\ Therefore\ an\ area\ of\ at\ least\ \dfrac{0.08 \times 5811.93 \times 10^3}{0.9 \times 400}$

$is\ required\ within\ a\ distance\ of\ 0.25 \times 1371.6.$

$A_{vrequired} = 1292\ mm^2\ over\ a\ distance\ of\ 343\ m\ from\ left\ support.$

Leave 50 mm for cover requirements. ***Provide stirrups every 70 mm up to 400 mm.***

$\mathbf{A_{vprovided} = 2000\ mm^2 > 1292\ mm^2\ OK.}$

(Note: Extra 1 stirrup is provided to meet with shear demand together with anchorage zone requirements. Also another extra is provided for making spacing equal to equally distribute the stresses.)

–There must also be a stirrup every 150 mm up to a distance of h. The bottom end of these stirrups must go around the strands and cover them. Minimum 10 M bars are required for the bottom and this can be different then the top part. Therefore, reinforcement type in anchorage zone can be different then regular stirrups.

$h = 1317.6\ mm$

Provide stirrups every 150 mm from 400 mm to 1450 mm from left support.

Design spacing to accommodate for shear:

A spacing of 300 mm is required up to 5500 mm from left support (See tables below). Based on this requirement, from ***1450 mm to 5650 mm, stirrups provided every 300 mm.*** *From 5650 mm to midspan requirement is 600 mm. However, having a nice number spacing adds value to constructibility so* ***stirrups provided every 525 mm from 5650 mm to 20350 mm.*** *Since the demand is symmetric, both ends will have similar reinforcement.*

Shear strength provided by the shear reinforcement:

$$V_{s,design} = \frac{\Phi_s \times A_v \times f_y \times d_v \times cot(\theta)}{s_{design}}$$

At midspan:

$$V_{s,design} = \frac{0.9 \times 400 \times 400 \times 1150.07 \times cot(37.01)}{525} \times 10^{-3} = 418.48\ kN$$

Determination of V_c required and checking it against V_c available:

$$V_{c,needed} = V_f - V_s - V_p \leq V_{c,available}$$

$$V_{c,needed} \geq 0$$

At midspan:

$$V_{c,needed} = 212.45 - 418.48 - 0 = -206.03\ kN < 0\ therefore\ \mathbf{0\ kN}$$

$$\mathbf{V_{c,available} = 163.25\ kN > 0\ kN\ therefore\ OK}$$

Forces in strands compared with force at ultimate design for flexure:

$$F_{lt} = \frac{M_f}{d_v} + 0.5 \times N_f + (V_f - 0.5 \times V_s - V_p) \times cot(\theta) < F_p$$

$$F_p = 5811.93\ kN\ from\ strain - compatiblity\ analysis$$

At midspan:

$$\mathbf{F_{lt} = 5349.45 < 5811.93\ kN\ OK}$$

Note on b_v and d_v Values: Those values are calculated to be conservative. In every section throughout the span, concrete shear resistance is greater than what actually calculated.

Table 5.3.7.1 – Final Shear Reinforcement Layout

From 50 mm to 400 mm	5 spacing @ 70 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1
From 400 mm to 1450 mm	7 spacing @ 150 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1
From 1450 mm to 5650 mm	14 spacing @ 300 mm c/c	15 M Double-Legged	Type 2
From 5650 mm to 20350 mm	28 spacing @ 525 mm c/c	15 M Double-Legged	Type 2
From 20350 mm to 24550 mm	14 spacing @ 300 mm c/c	15 M Double-Legged	Type 2
From 24550 mm to 25600 mm	7 spacing @ 150 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1
From 25600 mm to 25950 mm	5 spacing @ 70 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1

Type 1 and Type 2 reinforcement drawings can be found at the appendix section of part B (at the very end of this report together with design drawings)

Table 5.3.7.2.a – Shear Design Calculations

x [m]	M_f [kNm]	V_f [kN]	CGS [mm]	d [mm]	d_v [mm]	V_p [kN]	ε_x initial	ε_x modified	θ [degrees]	β
0	0	1133.46	313.43	1058.18	987.55	89.55	-0.002429	0	29	0.4
0.5	505.92	1098.63	300.50	1071.10	987.55	89.55	-0.002051	0	29	0.4
1	988.29	1063.81	287.58	1084.02	987.55	89.55	-0.001692	0	29	0.4
1.5	1447.13	1028.98	274.66	1096.94	987.55	89.55	-0.001352	0	29	0.4
2	1882.42	994.16	261.74	1109.86	998.88	89.55	-0.001047	0	29	0.4
2.5	2294.17	959.33	248.81	1122.79	1010.51	89.55	-0.000769	0	29	0.4
3	2682.38	924.51	235.89	1135.71	1022.14	89.55	-0.000517	0	29	0.4
3.5	3047.05	889.68	222.97	1148.63	1033.77	89.55	-0.000289	0	29	0.4
4	3410.78	853.63	210.05	1161.55	1045.40	89.55	-0.000068	0	29	0.4
4.5	3755.46	817.30	197.13	1174.47	1057.03	89.55	0.000133	0.000133	29.93	0.33
5	4075.32	780.97	184.20	1187.40	1068.66	89.55	0.000311	0.000311	31.18	0.27
5.5	4370.35	744.65	171.28	1200.32	1080.29	89.55	0.000466	0.000466	32.26	0.24
6	4640.56	708.32	158.36	1213.24	1091.92	89.55	0.000599	0.000599	33.19	0.21
6.5	4885.94	671.99	145.44	1226.16	1103.55	89.55	0.000711	0.000711	33.97	0.19
7	5106.49	635.67	132.52	1239.08	1115.18	89.55	0.000802	0.000802	34.61	0.18
7.5	5310.70	599.34	119.59	1252.01	1126.81	89.55	0.000879	0.000879	35.15	0.17
8	5500.86	563.01	106.67	1264.93	1138.44	89.55	0.000944	0.000944	35.61	0.17
8.5	5666.20	526.69	93.75	1277.85	1150.07	0	0.001062	0.001062	36.43	0.15
9	5806.72	490.36	93.75	1277.85	1150.07	0	0.001130	0.001130	36.91	0.15
9.5	5922.41	454.03	93.75	1277.85	1150.07	0	0.001181	0.001181	37.26	0.14
10	6013.28	417.71	93.75	1277.85	1150.07	0	0.001214	0.001214	37.50	0.14
10.5	6079.32	381.38	93.75	1277.85	1150.07	0	0.001231	0.001231	37.62	0.14
11	6120.53	345.05	93.75	1277.85	1150.07	0	0.001231	0.001231	37.61	0.14
11.5	6141.93	308.73	93.75	1277.85	1150.07	0	0.001217	0.001217	37.52	0.14
12	6168.55	276.09	93.75	1277.85	1150.07	0	0.001209	0.001209	37.46	0.14
12.5	6170.34	244.27	93.75	1277.85	1150.07	0	0.001185	0.001185	37.30	0.14
13	6147.31	212.45	93.75	1277.85	1150.07	0	0.001144	0.001144	37.01	0.15
13.5	6170.34	244.27	93.75	1277.85	1150.07	0	0.001185	0.001185	37.30	0.14
14	6168.55	276.09	93.75	1277.85	1150.07	0	0.001209	0.001209	37.46	0.14
14.5	6141.93	308.73	93.75	1277.85	1150.07	0	0.001217	0.001217	37.52	0.14
15	6120.53	345.05	93.75	1277.85	1150.07	0	0.001231	0.001231	37.61	0.14
15.5	6079.32	381.38	93.75	1277.85	1150.07	0	0.001231	0.001231	37.62	0.14
16	6013.28	417.71	93.75	1277.85	1150.07	0	0.001214	0.001214	37.50	0.14
16.5	5922.41	454.03	93.75	1277.85	1150.07	0	0.001181	0.001181	37.26	0.14
17	5806.72	490.36	93.75	1277.85	1150.07	0	0.001130	0.001130	36.91	0.15
17.5	5666.20	526.69	93.75	1277.85	1150.07	0	0.001062	0.001062	36.43	0.15
18	5500.86	563.01	106.67	1264.93	1138.44	89.55	0.000944	0.000944	35.61	0.17
18.5	5310.70	599.34	119.59	1252.01	1126.81	89.55	0.000879	0.000879	35.15	0.17
19	5106.49	635.67	132.52	1239.08	1115.18	89.55	0.000802	0.000802	34.61	0.18
19.5	4885.94	671.99	145.44	1226.16	1103.55	89.55	0.000711	0.000711	33.97	0.19
20	4640.56	708.32	158.36	1213.24	1091.92	89.55	0.000599	0.000599	33.19	0.21
20.5	4370.35	744.65	171.28	1200.32	1080.29	89.55	0.000466	0.000466	32.26	0.24
21	4075.32	780.97	184.20	1187.40	1068.66	89.55	0.000311	0.000311	31.18	0.27
21.5	3755.46	817.30	197.13	1174.47	1057.03	89.55	0.000133	0.000133	29.93	0.33
22	3410.78	853.63	210.05	1161.55	1045.40	89.55	-0.000068	0	29	0.4
22.5	3047.05	889.68	222.97	1148.63	1033.77	89.55	-0.000289	0	29	0.4
23	2682.38	924.51	235.89	1135.71	1022.14	89.55	-0.000517	0	29	0.4
23.5	2294.17	959.33	248.81	1122.79	1010.51	89.55	-0.000769	0	29	0.4
24	1882.42	994.16	261.74	1109.86	998.88	89.55	-0.001047	0	29	0.4
24.5	1447.13	1028.98	274.66	1096.94	987.55	89.55	-0.001352	0	29	0.4
25	988.29	1063.81	287.58	1084.02	987.55	89.55	-0.001692	0	29	0.4
25.5	505.92	1098.63	300.50	1071.10	987.55	89.55	-0.002051	0	29	0.4
26	0	1133.46	313.43	1058.18	987.55	89.55	-0.002429	0	29	0.4

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Table 5.3.7.2.b – Shear Design Calculations

x [m]	V_c [kN]	V_s [kN]	$s_{required}$ [mm]	s_{max} [mm]	$A_{v,min}$ [mm2]	s_{design} [mm]	Governed By	$V_{s,design}$ [kN]	$V_s + V_p$	$V_{c,\ needed}$	$V_{c,\ provided}$ OK?	F_{lt} [kN]	F_p [kN]
0	380.75	663.16	386.86	300	57.83	70	Anchorage Zone	3664.99	3754.53	0	OK	0	5811.93
0.5	380.75	628.34	408.30	300	57.83	150	Anchorage Zone	1710.33	1799.87	0	OK	789.98	5811.93
1	380.75	593.51	432.25	300	57.83	150	Anchorage Zone	1710.33	1799.87	0	OK	1215.61	5811.93
1.5	380.75	558.69	459.20	300	57.83	300	Smax	855.16	944.71	84.27	OK	2388.78	5811.93
2	385.11	519.50	499.50	300	57.83	300	Smax	864.97	954.52	39.64	OK	2736.27	5811.93
2.5	389.60	480.19	546.68	300	57.83	300	Smax	875.04	964.59	0	OK	3050.15	5811.93
3	394.08	440.88	602.28	300	57.83	300	Smax	885.11	974.66	0	OK	3332.21	5811.93
3.5	398.56	401.57	668.75	300	57.83	300	Smax	895.18	984.73	0	OK	3583.53	5811.93
4	403.05	361.03	752.22	300	57.83	300	Smax	905.25	994.80	0	OK	3824.55	5811.93
4.5	339.63	388.12	681.11	300	57.83	300	Smax	881.18	970.72	0	OK	4051.59	5811.93
5	281.00	410.43	619.69	300	57.83	300	Smax	847.80	937.34	0	OK	4255.66	5811.93
5.5	245.20	409.90	601.24	600	115.66	300	Smax	821.51	911.05	0	OK	4432.66	5811.93
6	221.79	396.99	605.46	600	115.66	525	Constructibilility	457.83	547.38	160.94	OK	4845.88	5811.93
6.5	205.95	376.49	626.37	600	115.66	525	Constructibilility	449.19	538.74	133.26	OK	4958.55	5811.93
7	195.19	350.93	663.01	600	115.66	525	Constructibilility	443.18	532.73	102.94	OK	5049.30	5811.93
7.5	187.37	322.43	714.62	600	115.66	525	Constructibilility	438.88	528.43	70.91	OK	5125.36	5811.93
8	181.62	291.85	784.27	600	115.66	525	Constructibilility	435.98	525.52	37.49	OK	5188.65	5811.93
8.5	171.03	355.66	630.85	600	115.66	525	Constructibilility	427.36	427.36	99.32	OK	5350.91	5811.93
9	164.56	325.80	676.83	600	115.66	525	Constructibilility	420.02	420.02	70.34	OK	5422.32	5811.93
9.5	160.03	294.01	740.39	600	115.66	525	Constructibilility	414.63	414.63	39.40	OK	5473.92	5811.93
10	157.15	260.55	828.34	600	115.66	525	Constructibilility	411.10	411.10	6.61	OK	5505.13	5811.93
10.5	155.77	225.61	952.60	600	115.66	525	Constructibilility	409.36	409.36	0	OK	5515.37	5811.93
11	155.80	189.25	1135.73	600	115.66	525	Constructibilility	409.40	409.40	0	OK	5504.05	5811.93
11.5	156.96	151.76	1421.30	600	115.66	525	Constructibilility	410.86	410.86	0	OK	5475.04	5811.93
12	157.59	118.50	1823.67	600	115.66	525	Constructibilility	411.64	411.64	0	OK	5455.35	5811.93
12.5	159.63	84.65	2568.66	600	115.66	525	Constructibilility	414.14	414.14	0	OK	5414.05	5811.93
13	163.25	49.21	4464.75	600	115.66	525	Constructibilility	418.48	418.48	0	OK	5349.45	5811.93
13.5	159.63	84.65	2568.66	600	115.66	525	Constructibilility	414.14	414.14	0	OK	5414.05	5811.93
14	157.59	118.50	1823.67	600	115.66	525	Constructibilility	411.64	411.64	0	OK	5455.35	5811.93
14.5	156.96	151.76	1421.30	600	115.66	525	Constructibilility	410.86	410.86	0	OK	5475.04	5811.93
15	155.80	189.25	1135.73	600	115.66	525	Constructibilility	409.40	409.40	0	OK	5504.05	5811.93
15.5	155.77	225.61	952.60	600	115.66	525	Constructibilility	409.36	409.36	0	OK	5515.37	5811.93
16	157.15	260.55	828.34	600	115.66	525	Constructibilility	411.10	411.10	6.61	OK	5505.13	5811.93
16.5	160.03	294.01	740.39	600	115.66	525	Constructibilility	414.63	414.63	39.40	OK	5473.92	5811.93
17	164.56	325.80	676.83	600	115.66	525	Constructibilility	420.02	420.02	70.34	OK	5422.32	5811.93
17.5	171.03	355.66	630.85	600	115.66	525	Constructibilility	427.36	427.36	99.32	OK	5350.91	5811.93
18	181.62	291.85	784.27	600	115.66	525	Constructibilility	435.98	525.52	37.49	OK	5188.65	5811.93
18.5	187.37	322.43	714.62	600	115.66	525	Constructibilility	438.88	528.43	70.91	OK	5125.36	5811.93
19	195.19	350.93	663.01	600	115.66	525	Constructibilility	443.18	532.73	102.94	OK	5049.30	5811.93
19.5	205.95	376.49	626.37	600	115.66	525	Constructibilility	449.19	538.74	133.26	OK	4958.55	5811.93
20	221.79	396.99	605.46	600	115.66	525	Constructibilility	457.83	547.38	160.94	OK	4845.88	5811.93
20.5	245.20	409.90	601.24	600	115.66	300	Smax	821.51	911.05	0	OK	4432.66	5811.93
21	281.00	410.43	619.69	300	57.83	300	Smax	847.80	937.34	0	OK	4255.66	5811.93
21.5	339.63	388.12	681.11	300	57.83	300	Smax	881.18	970.72	0	OK	4051.59	5811.93
22	403.05	361.03	752.22	300	57.83	300	Smax	905.25	994.80	0	OK	3824.55	5811.93
22.5	398.56	401.57	668.75	300	57.83	300	Smax	895.18	984.73	0	OK	3583.53	5811.93
23	394.08	440.88	602.28	300	57.83	300	Smax	885.11	974.66	0	OK	3332.21	5811.93
23.5	389.60	480.19	546.68	300	57.83	300	Smax	875.04	964.59	0	OK	3050.15	5811.93
24	385.11	519.50	499.50	300	57.83	300	Smax	864.97	954.52	39.64	OK	2736.27	5811.93
24.5	380.75	558.69	459.20	300	57.83	300	Smax	855.16	944.71	84.27	OK	2388.78	5811.93
25	380.75	593.51	432.25	300	57.83	150	Anchorage Zone	1710.33	1799.87	0	OK	1215.61	5811.93
25.5	380.75	628.34	408.30	300	57.83	150	Anchorage Zone	1710.33	1799.87	0	OK	789.98	5811.93
26	380.75	663.16	386.86	300	57.83	70	Anchorage Zone	3664.99	3754.53	0	OK	0	5811.93

SHEAR DESIGN NOW COMPLETE

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5.3.8 Design for shrinkage and temperature variation

According to CSA S6 − 14, reinforcement is required in both direction.

Requirement:

$-500\ mm^2$ *of standard reinforcement per each meter*

$-s_{max} = 300\ mm$

$A_{s,required}$ ***in the direction parallel to span*** $= 1.3716\ m \times 500 = \mathbf{685.8\ mm^2}$

$A_{s,required}$ ***in the direction transverse to span*** $= 0.6604\ m \times 500 = \mathbf{330.2\ mm^2}$

Provide:

4 − 15 *M bars* @ 300 *in the direction parallel to span.* $A_s = 800\ mm^2$

4 − 10 *M bars* @ 130 *in the direction transverse to span* $A_s = 400\ mm^2$

130, 300 *mm* ≤ 300 *mm OK*

5.4 AASHTO LRFD 2014-17

5.4.1 Estimation of Required Prestress and Initial Strand Pattern

Bottom tensile stress at midspan during service according to service combination in AASHTO LFRD 2014 − 17:

$$f_b = \frac{M_G + M_S}{s_b} + \frac{M_{SDL} + M_{LL}}{s_{bc}}$$

$$\boldsymbol{f_b} = \frac{(985.16 + 962.86) \times 10^6}{1.7275 \times 10^8} + \frac{(301.98 + 2165.41) \times 10^6}{2.7683 \times 10^8} = \mathbf{20.1895\ MPa}$$

$M_G =$ *Moment due to self − weight of girder at midspan*

$M_S =$ *Moment due to self − weight of deck at midspan*

$M_{SDL} =$ *Moment due to self − weight of asphalt and waterproofing at midspan*

$M_{LL} =$ *Moment due to live load at midpsan*

At service loading conditions, allowable tensile stress according to AASHTO LRFD 2014 − 17 is:

$$\boldsymbol{F_b} = 0.5 \times \sqrt{f'_c \ for\ girder} = 0.5 \times \sqrt{40} = \mathbf{3.162\ MPa}$$

Required Number of Strands:

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress − allowable tensile stress at final $= f_b - F_b$

$$\boldsymbol{f_{pb}} = 20.1895 - 3.162 = \mathbf{17.0272\ MPa}$$

Assuming the distance from center of gravity of strands to the bottom fiber of the beam is equal to

$$\boldsymbol{y_{bs}} = \mathbf{100\ mm}$$

Strand eccentricity at mispan:

$$\boldsymbol{e_c} = y_b - y_{bs} = 628.2395 - 100 = \mathbf{528.2395\ mm}$$

Bottom fiber stress due to prestress after losses:

$$f_{b\_prestress} = \frac{P_{pe}}{A_g} + \frac{P_{pe} \times e_c}{s_b} \quad where\ P_{pe} = Effective\ prestressing\ force\ alfter\ all\ losses$$

$$17.0272 = \frac{P_{pe} \times 10^3}{5.0903 \times 10^5} + \frac{P_{pe} \times 528.2395 \times 10^3}{1.7275 \times 10^8}$$

solving this for P_{pe}, $\boldsymbol{P_{pe}} = \mathbf{3390.33\ kN}$

Assuming final losses is 20% of f_{pi} (for now)

Assumed final losses = $0.2 \times 1395 = \mathbf{279\ MPa}$

The prestress force per strand after losses = *Cross − sectional area of one strand* $\times (f_{pi} - losses)$

$$= 98.7 \times (1395 - 279) \times 10^{-3} = \mathbf{110.1492\ kN}$$

Try **32** *Strands as an initial trial:*

Effective strand eccentricity at midspan after strand arrangement

$$e_c = 628.2395 - \frac{12 \times (50 + 100) + 8 \times 150}{32} = \mathbf{534.4895\ mm}$$

$$P_{pe} = 32 \times 110.1492 = \mathbf{3524.8\ kN}$$

$$f_b = \frac{3524.8 \times 10^3}{5.0903 \times 10^5} + \frac{534.4895 \times 3524.8 \times 10^3}{1.7275 \times 10^8} = \mathbf{17.83\ MPa}$$

17.83 *MPa* > 17.0272 *MPa therefore OK*

Trying **30 *strands*** *hoping to use less steel if possible (Iteration # 2):*

$$e_c = 628.2395 - \frac{12 \times (50 + 100) + 6 \times 150}{30} = \mathbf{528.2395\ mm}$$

$$P_{pe} = 30 \times 110.1492 = \mathbf{3304.5\ kN}$$

$$f_b = \frac{3304.5 \times 10^3}{5.0903 \times 10^5} + \frac{528.2395 \times 3304.5 \times 10^3}{1.7275 \times 10^8} = \mathbf{16.7873\ MPa}$$

16.7873 *MPa* < 17.0272 *MPa therefore NOT OK*

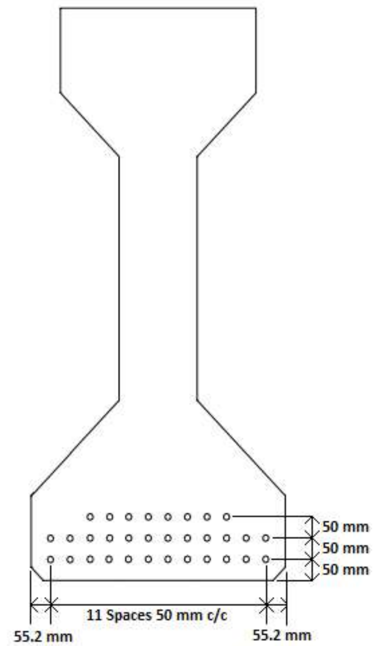

Figure 5.4.1.1 – Initial Strand Pattern

5.4.2 Prestressing Losses

Total prestress loss:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

where

$\Delta f_{pES} =$ *Loss of prestress due to elastic shortening*

$\Delta f_{pSR} =$ *Loss of prestress due to concrete shrinkage*

$\Delta f_{pCR} =$ *Loss of prestress due to creep of concrete*

$\Delta f_{pR2} =$ *Loss of prestress due to relaxation of steel after transfer*

Elastic Shortening:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} \times f_{cgs}$$

where:

f_{cgs} = *Sum of concrete stresses at the center of gravity of prestressing steel due to moment and axial force caused by the prestressing force and due to the moment caused by the self – weight of the girder*

$$f_{cgs} = \frac{P_i}{A_{gt}} + \frac{P_i \times e_c^{\,2}}{I_{gt}} - \frac{M_G \times e_c}{I_{gt}}$$

where:

P_i = *Pretensioning force after allowing for initial losses*

A_{gt} = *Transformed area of the girder*

M_G = *Moment caused by the self – weight of the girder*

e_c = *Distance from CG of the prestressing steel to CG of the girder*

With the absence of more information, a 8% loss from maximum allowed initial stress at transfer is assumed.

$$\boldsymbol{P_i} = 32 \times 98.7\,mm^2 \times 0.92^{*} \times 1395^{**} \times 10^{-3} = \mathbf{4053.5\,kN}$$

$$*\frac{(100\% - 8\%)}{100},\ **f_{pi}$$

Using transformed area properties in here is more common these days with the advancement of computer technology. Using gross area also gives acceptable results.

$$\boldsymbol{A_{gt}} = \mathbf{528487.6385\,mm^2}$$

I_{gt} = *Transformed moment of Inertia of the girder*

$$\boldsymbol{I_{gt}} = 1.1391 \times 10^{11}\,\boldsymbol{mm^4}$$

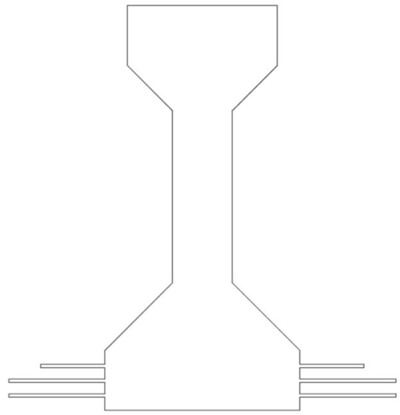

Figure 5.4.2.1 – Transformed Girder

$$e_c = 534.4895\ mm$$

$$M_G = 985.16\ kNm$$

$$f_{cgs} = \frac{4053.5 \times 10^3}{528488} + \frac{4053.5 \times 10^3 \times 534.4895^2}{1.1391 \times 10^{11}} - \frac{985.16 \times 10^6 \times 534.4895}{1.1391 \times 10^{11}} = 13.2133\ MPa$$

$$E_{ci} = 31800\ MPa$$

$$E_p = 200000\ MPa$$

$$n = \frac{200000}{31800} = 6.2895$$

$$\Delta f_{pES} = n \times f_{cgs} = 6.2895 \times 13.2133 = 83.1055\ MPa$$

Long term losses (Approximate Formula):

AASHTO LRFD 2014 – 17 proposes an approximate formula for estimating long term losses. This formula is suitable for I girders but for more custom sections, refined procedure of calculating losses must be followed.

$$\Delta f_{pLT} = \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

$$\Delta f_{pLT} = 10 \times \frac{f_{pi} \times A_{ps}}{A_g} \times \gamma_h \times \gamma_{st} + 83 \times \gamma_h \times \gamma_{st} + \Delta f_{pR2}$$

$$\Delta f_{pR1} = \Delta f_{pR2} = 17\ MPa \text{ for low} - \text{relaxation steel}$$

$$\gamma_h = 1.7 - 0.01 \times H \text{ where } H \text{ is the relative humidity of the surrounding air in \% (assumed 60\%)}$$

$$\gamma_h = 1.7 - 0.01 \times 60 = 1.1$$

$$f_{pi} = 1395\ MPa$$

$$A_{ps} = 98.7 \times 32 = 3158.4\ mm^2$$

$$A_g = 509031.24\ mm^2$$

$$\gamma_{st} = \frac{35}{\left(7 + f'_{ci}\right)} = \mathbf{0.833}$$

$$\Delta f_{pLT} = 10 \times \frac{1395 \times 3158.4}{509031} \times 1.1 \times 0.833 + 83 \times 1.1 \times 0.833 + 17 = \mathbf{172.4263\ MPa}$$

Total Prestressing Losses: $172.4263 + 83.1055 + 17 = \mathbf{272.5317\ MPa}$

$$\%\ Loss = \frac{272.5317}{f_{pi}} \times 100 = \frac{272.5317}{1395} \times 100 = \mathbf{19.5363\ \%}$$

For safety reasons, 17 MPa of total relaxation loss will be counted in initial prestressing loss

Initial prestressing losses = Losses due to Elastic Shortening + 17

Initial Prestressing Loss $= 83.1055 + 17 = \mathbf{100.1055\ MPa}$

$$\%\ Initial\ Loss = \frac{100.1055}{f_{pi}} \times 100 = \frac{100.1055}{1395} \times 100 = \mathbf{7.1760\ \%}$$

7.1760 % < 8% and is close to 8% so no iteration is required

Final effective prestress, $f_{pe} = f_{pi} - \Delta f_{pT} = 1395 - 272.5317 = \mathbf{1122.5\ MPa}$

At service, $f_{pe} \leq 1488\ MPa\ OK\ \left(0.8 \times f_{pu}\right)$

Total prestressing force after all losses, $P_{pe} = 32 \times 98.7 \times 1122.5 \times 10^{-3} = \mathbf{3545.2\ kN}$

Initial prestressing force after initial losses, $P_i = 32 \times 98.7 \times (1395 - 100.1055) \times 10^{-3} = \mathbf{4089.8\ kN}$

Final stress at the bottom fiber in midspan:

$$f_b = \frac{P_{pe}}{A_g} + \frac{P_{pe} \times e_c}{s_b} = \frac{3545.2 \times 10^3}{509031} - \frac{3545.2 \times 10^3 \times 534.4895}{1.7275 \times 10^8} = \mathbf{17.9333\ MPa > 17.0272\ OK}$$

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5.4.3 Concrete stress limits at top and bottom

5.4.3.1 Stress limits at transfer and Strand Pattern

At Midspan:

At transfer, the tensile stress in the top fiber connot exceed:

$$f_{ti} = 0.25 \times \sqrt{35} = 1.479\ MPa$$

$$f_{ti} \geq \frac{P_i}{A_g} - \frac{P_i \times e_c}{s_t} + \frac{M_G}{s_t}$$

$$f_{ti} = -\frac{4089.8 \times 10^3}{509031} + \frac{4089 \times 10^3}{1.46 \times 10^8} - \frac{985.16 \times 10^6}{1.46 \times 10^8} = 0.1912\ MPa\ OK$$

At transfer, the compressive stress in the bottom fiber cannot exceed:

$$f_{bi} = 0.6 \times 35 = 21\ MPa$$

$$f_{bi} \geq \frac{P_i}{A_g} + \frac{P_i \times e_c}{s_b} - \frac{M_G}{s_b}$$

$$f_{bi} = \frac{4089.8 \times 10^3}{509031} + \frac{4089.8 \times 10^3}{1.7275 \times 10^8} - \frac{985.16 \times 10^6}{1.7275 \times 10^8} = 14.9854\ MPa\ OK$$

This same procedure is done for every 0.5 m of span and limits of eccentricities are determined using excel. This will serve to determine the optimal hold down points for harped strands.

The beam is divided into 53 pieces in longitudinal direction. Every cross-section of these 52 pieces is divided into 1372 pieces resulting in 72716 elements. For all small elements, stresses are calculated as if the strands weren’t harped. Prestressing losses are calculated using MATLAB using the procedure shown above. The MATLAB code is available in the appendix of this chapter.

For straight strands, entirety of the beam was within limits of compression allowed at transfer. However, as expected, the top ends of the beam exceeded the tensile stress limit allowed by AASHTO LRFD 2014-17.

At transfer, the tensile stress in concrete cannot exceed:

$$f_{\text{tensile allowed}} = 0.25 \times \sqrt{35} = 1.479\ MPa$$

Figure below shows in red where tensile stress exceeds 1.479 MPa. The green elements are within limits of stress.

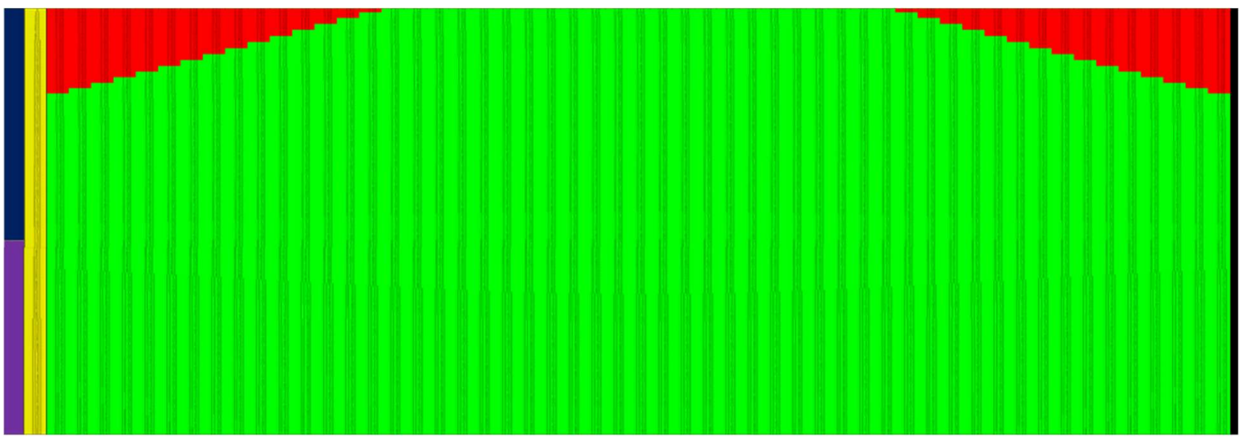


Figure 5.4.3.1.1 – Straight Strands – Stresses experienced

Looking at the stress values, optimal hold down points determined to be x = 8.5 m and x = 17.5 m from left support.

The strand profile below is determined to give the best stress results (32 12.7 mm strands with the arrangement and pattern below):

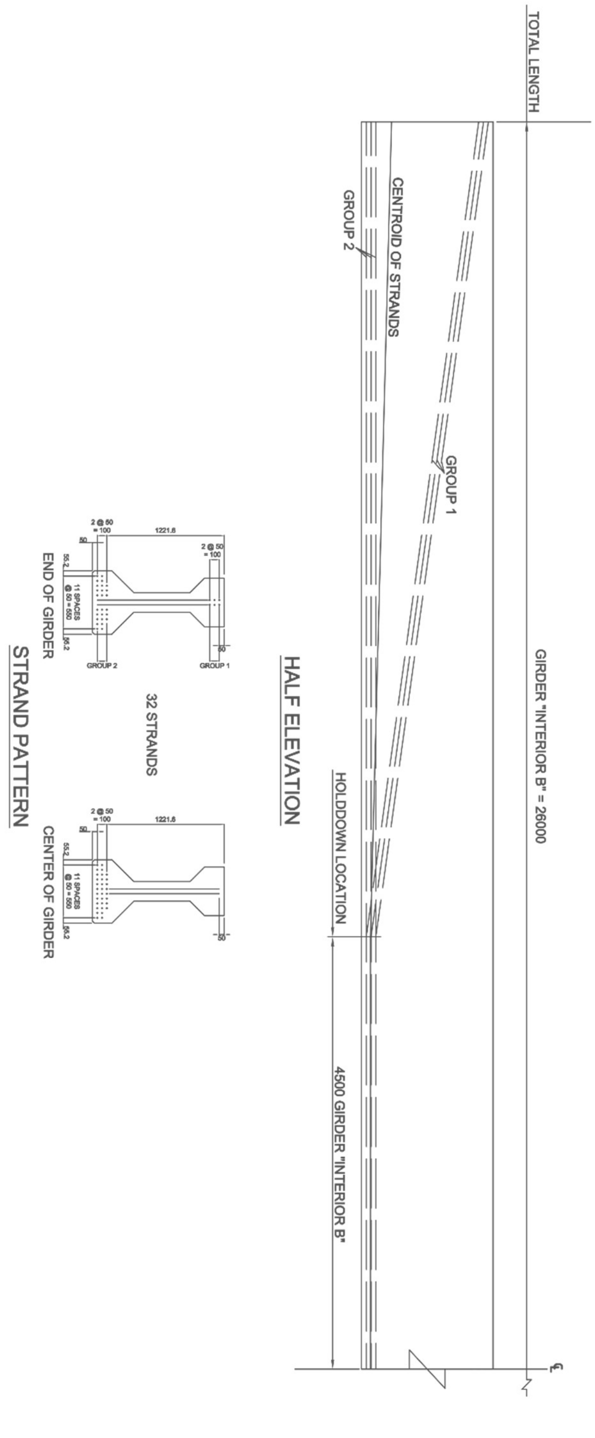


Figure 5.4.3.1.2 – Strand Pattern

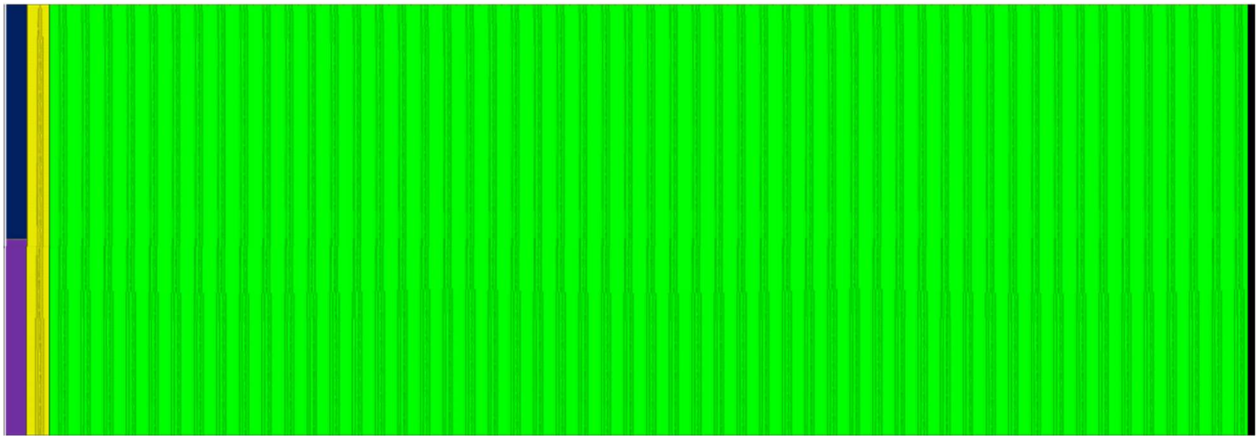


Figure 5.4.3.1.3 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced

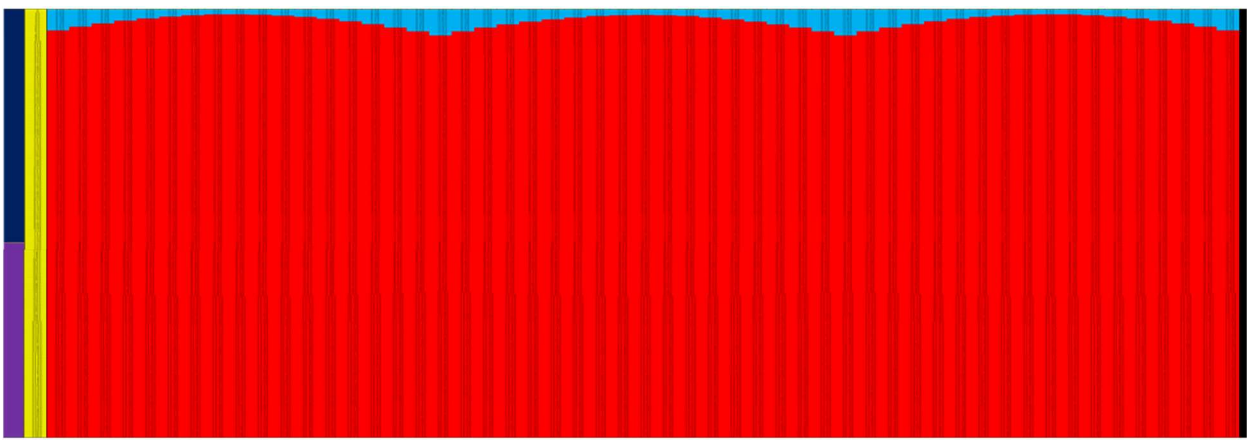


Figure 5.4.3.1.4 – Harped Strands with groups as in figure 5.4.3.1.2 – Elements in tension Blue, Elements in compression Red

Maximum stresses recorded at transfer are:

$0.999\,MPa \text{ for tension } < 0.25 \times \sqrt{35}\ (1.479\,MPa)\ OK$

$15.66\,MPa \text{ for compression } < 0.6 \times 35\ (21\,MPa)\ OK$

Table 5.4.3.1.1 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced [-Compression, +Tension]

Distance From Left Support	Maximum Top Stress (MPa)	Maximum Bottom Stress (MPa)
0	0.78	-15.48
0.5	0.64	-15.36
1	0.51	-15.25
1.5	0.40	-15.16
2	0.32	-15.08
2.5	0.25	-15.03
3	0.20	-14.99
3.5	0.17	-14.96
4	0.17	-14.96
4.5	0.18	-14.97
5	0.21	-15.00
5.5	0.26	-15.04
6	0.34	-15.10
6.5	0.43	-15.18
7	0.54	-15.28
7.5	0.67	-15.39
8	0.83	-15.52
8.5	1.00	-15.66
9	0.83	-15.52
9.5	0.68	-15.39
10	0.55	-15.28
10.5	0.44	-15.19
11	0.35	-15.11
11.5	0.28	-15.05
12	0.23	-15.01
12.5	0.20	-14.99
13	0.19	-14.98
13.5	0.20	-14.99
14	0.23	-15.01
14.5	0.28	-15.05
15	0.35	-15.11
15.5	0.44	-15.19
16	0.55	-15.28
16.5	0.68	-15.39
17	0.83	-15.52
17.5	1.00	-15.66
18	0.83	-15.52
18.5	0.67	-15.39
19	0.54	-15.28
19.5	0.43	-15.18
20	0.34	-15.10
20.5	0.26	-15.04
21	0.21	-15.00
21.5	0.18	-14.97
22	0.17	-14.96
22.5	0.17	-14.96
23	0.20	-14.99
23.5	0.25	-15.03
24	0.32	-15.08
24.5	0.40	-15.16
25	0.51	-15.25
25.5	0.64	-15.36
26	0.78	-15.48

MAXIMUM TENSION	=	***1.00***
MAXIMUM COMPRESSION	=	***-15.66***

5.4.3.2 Service conditions

At Midspan:

At service, the compressive stress in top fiber cannot exceed:

$$f_{ts} = 0.45 \times 40 = \mathbf{18\,MPa}$$

$$P_{pe}\ @\ midspan = \mathbf{3545.2\,kN}$$

$$f_{ts} \geq \frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c}{s_t} + \frac{M_G + M_S}{s_t} + \frac{M_{SDL} + M_{LL}}{\dfrac{I_c}{(y_{tc} - 200)}}$$

$$f_{ts} = \frac{3545.2 \times 10^3}{509031} - \frac{3545.2 \times 10^3 \times 534.4895}{1.46 \times 10^8} + \frac{(985.16 + 962.86) \times 10^6}{1.46 \times 10^8}$$

$$+ \frac{(301.98 + 2160.66) \times 10^6}{\dfrac{2.896 \times 10^{11}}{(525.4544 - 200)}} = \mathbf{10.0962\,MPa\ OK}$$

At service, the tensile stress in the bottom fiber cannot exceed:

$$f_{bs} = 0.5 \times \sqrt{40} = \mathbf{3.162\,MPa}$$

$$f_{bs} \geq -\frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c}{s_b} + \frac{M_G + M_S}{s_b} + \frac{M_{SDL} + M_{LL}}{s_{bc}}$$

$$f_{bs} = -\frac{3545.2 \times 10^3}{509031} - \frac{3545.2 \times 10^3 \times 534.4895}{1.7275 \times 10^8} + \frac{(985.16 + 962.86) \times 10^6}{1.7275 \times 10^8}$$

$$+ \frac{(301.98 + 2160.66) \times 10^6}{2.7683 \times 10^8} = \mathbf{2.239\,MPa\ OK}$$

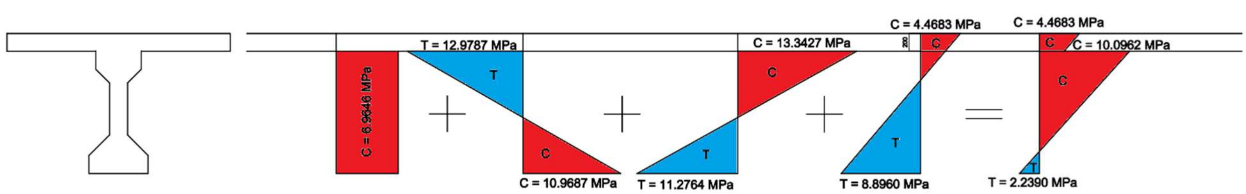


Figure 5.4.3.2.1 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced visualized at midspan

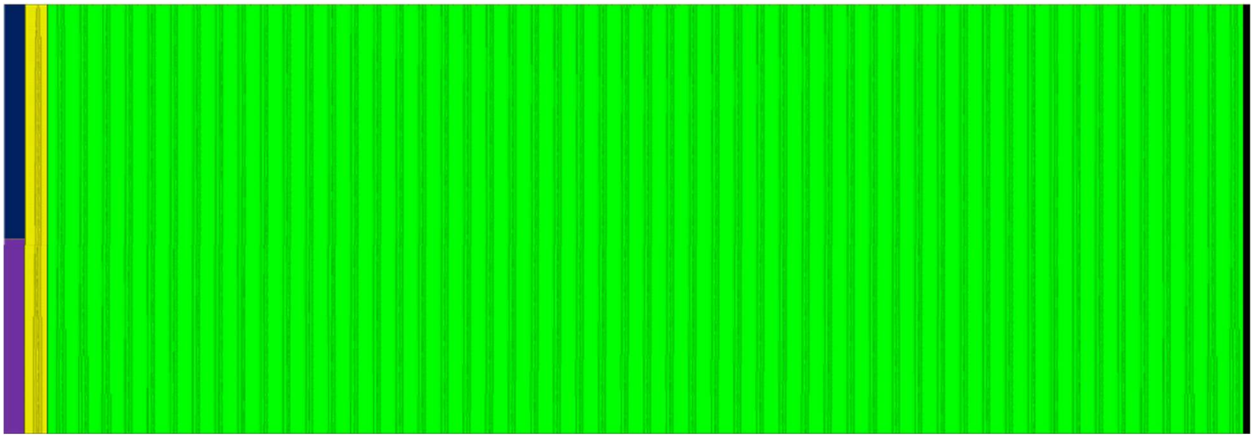


Figure 5.4.3.2.2 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced at service conditions

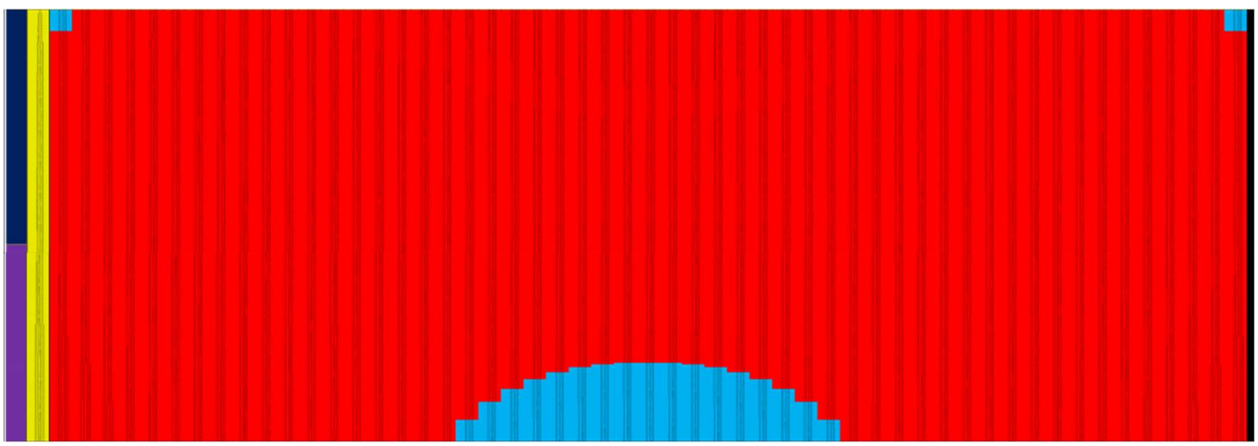


Figure 5.4.3.2.3 – Harped Strands with groups as in figure 5.4.3.1.2 – Elements in tension Blue, Elements in compression Red – Service Conditions

Maximum stresses recorded at service are:

$2.23\ MPa\ for\ tension < 0.5 \times \sqrt{40}\ (3.162\ MPa)\ OK$

$13.42\ MPa\ for\ compression < 0.45 \times 40\ (18\ MPa)\ OK$

Table 5.4.3.2.1 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced by the girder during service [-Compression, +Tension]

Distance From Left Support	Maximum Top Stress (MPa)	Maximum Bottom Stress (MPa)
0	0.68	-13.42
0.5	-0.23	-12.12
1	-1.10	-10.89
1.5	-1.91	-9.73
2	-2.68	-8.62
2.5	-3.40	-7.58
3	-4.07	-6.61
3.5	-4.69	-5.69
4	-5.26	-4.84
4.5	-5.78	-4.06
5	-6.26	-3.33
5.5	-6.68	-2.68
6	-5.95	-0.97
6.5	-7.39	-1.55
7	-7.66	-1.08
7.5	-7.89	-0.67
8	-8.07	-0.33
8.5	-8.21	-0.05
9	-8.61	0.45
9.5	-8.97	0.89
10	-9.27	1.27
10.5	-9.53	1.59
11	-9.74	1.85
11.5	-9.90	2.04
12	-10.02	2.17
12.5	-10.08	2.23
13	-10.10	2.23
13.5	-10.08	2.23
14	-10.02	2.17
14.5	-9.90	2.04
15	-9.74	1.85
15.5	-9.53	1.59
16	-9.27	1.27
16.5	-8.97	0.89
17	-8.61	0.45
17.5	-8.21	-0.05
18	-8.07	-0.33
18.5	-7.89	-0.67
19	-7.66	-1.08
19.5	-7.39	-1.55
20	-7.06	-2.08
20.5	-6.68	-2.68
21	-6.26	-3.33
21.5	-5.78	-4.06
22	-5.26	-4.84
22.5	-4.69	-5.69
23	-4.07	-6.61
23.5	-3.40	-7.58
24	-2.68	-8.62
24.5	-1.91	-9.73
25	-1.10	-10.89
25.5	-0.23	-12.12
26	0.68	-13.42

MAXIMUM TENSION	=	2.23
MAXIMUM COMPRESSION	=	-13.42

5.4.4 Ultimate Flexural Capacity

Ultimate flexural capacity of the composite section can be calculated in two ways.

The first and most commonly used method that works for every section is strain compatibility analysis. In this method, the section is divided into small rectangles and stresses are assumed constant throughout the small rectangle. Each of the rectangle will have a resultant force. The moment caused by all resultant forces are assembled into 1 compressive force with a certain distance from the centroid. Equating tensile force at the level of center of gravity of steel with this compressive force gives the magnitude of the compressive force. Ultimate moment capacity (Mr) is then determined by multiplying tensile or compressive force by the moment arm.

The concrete stress-strain curve used for the strain compatibility analysis presented in this report is based on the Hognestad's Modified Parabola. The prestressing steel and concrete stress-strain curve is given in the chapter 2 of this report.

Another way that is simpler and gives good enough results for most sections is assuming a rectangular stress pattern (Whitney's Stress Block). Whitney's Stress Block parameters are used by AASHTO LRFD 2014-17. It is still required to iterate to find for the location of compressive force with this method if the centroid of compressive forces is not in a rectangular section.

So, both ways, the usage of a computer program is very helpful.

5.4.4.1 Rectangular Stress Block Assumption

RECTANGULAR SECTION ASSUMPTION AT MIDSPAN

→ Flexural demand at midspan, $M_f = 6669{,}16$ kNm (From chapter 4)

According to AASHTO LRFD 2014-17

Strength Limit state:

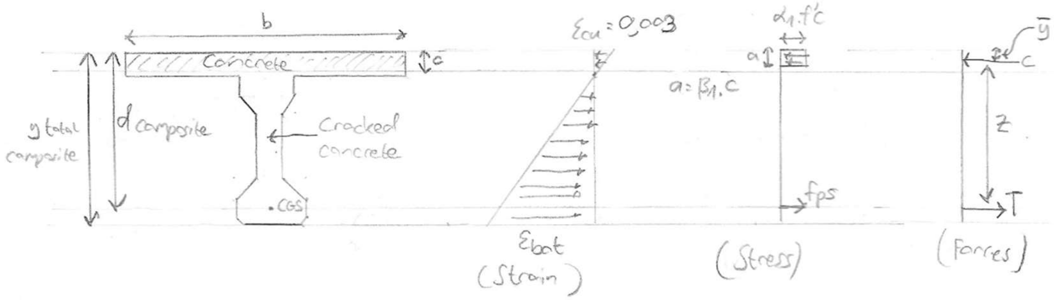
$\phi_p = 1$ $\quad f_{pu} = 1860$ MPa

$\phi_c = 1$ $\quad f'_c = 40$ MPa

Stress block parameters:

$\alpha_1 = 0{,}85$

$\beta_1 = 0{,}85$



a) Without reduction factors ϕ_p, ϕ_c - they are 1 in AASHTO anyways -

$$a = \frac{A_{ps} \cdot f_{ps}}{\alpha_1 \cdot f'_c \cdot b} = \frac{98{,}7 \cdot 32 \cdot f_{ps}}{0{,}85 \cdot 40 \cdot 2500} = 0{,}0372 \cdot f_{ps}$$

$$\frac{c_u}{d_p} = \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot \beta_1 \cdot f'_c \cdot b \cdot d} = \frac{98{,}7 \cdot 32 \cdot 1860}{0{,}85 \cdot 0{,}85 \cdot 40 \cdot 2500 \cdot 1477{,}8} = 0{,}055$$

$$k_p = 3\left(1 - \underbrace{\frac{f_{po}}{f_{pu}}}_{0{,}9}\right) = 3 \cdot (1 - 0{,}9) = 0{,}3 \text{ or } 0{,}28 \text{ for low relaxation}$$

both OK

$$f_{ps} = f_{pu}\left(1 - k_p \cdot \frac{c_u}{d_p}\right) = 1860 \cdot (1 - 0{,}28 \cdot 0{,}055) = 1831 \text{ MPa}$$

$$a = 0{,}0372 \cdot f_{ps} = 0{,}0372 \cdot 1831 = 68{,}05 \text{ mm}$$

$a < t_s$ (200 mm) ∴ Rectangular assumption can be used.

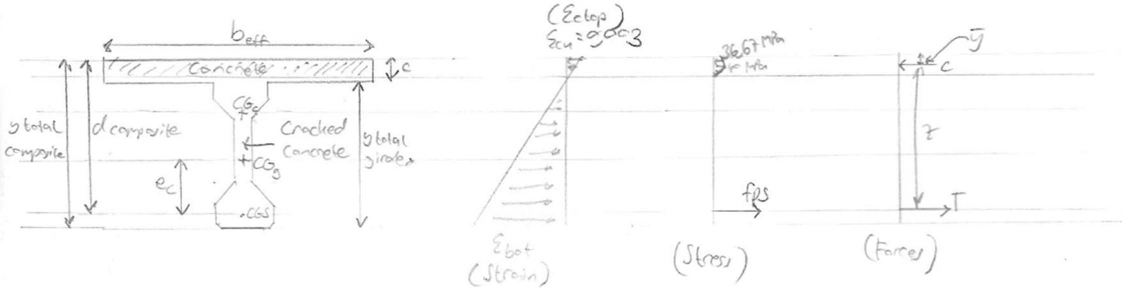
$$M_n = \underbrace{A_{ps} \cdot f_{ps}}_{T} \cdot \underbrace{\left(d - \frac{a}{2}\right)}_{Z} = 98{,}7 \cdot 32 \cdot \left(1477{,}8 - \frac{68{,}05}{2}\right) \cdot 1831 \cdot 10^{-6}$$

$M_n = 8351$ kNm > 6669,16 kNm ok

5.4.4.2 Strain-compatibility analysis

STRAIN - COMPATIBILITY ANALYSIS AT MIDSPAN

→ Flexural demand at midspan, $M_f = 6669,16$ kNm (from chapter 4)



$E_{ps} = 200000$ MPa

$E_c = 0,043 \cdot 2500^{1.5} \cdot \sqrt{40} = 33994$ MPa

$A_{ps} = 98,7 \cdot 32 = 3158,4\ mm^2$

$A_g = 509031,24\ mm^2$

A_{gt} (Transformed area of the prestressing girder) = $528487,6385\ mm^2$

y total = 1371,6 mm^2

y total composite = 1571,6 mm^2

$d = 1277,8$ mm

d composite = 1477,8 mm

$e_c = 534,4895$ mm

$y_t = 743,3605$ mm

$y_b = 628,2395$ mm

$I_g = 1,0853 \cdot 10^{11}\ mm^4$

$I_{gt} = 1,1391 \cdot 10^{11}\ mm^4$

$P_{pe} = 3545,2$ kN

$\varepsilon_{ctop} = \varepsilon_{cu} = 0,003$

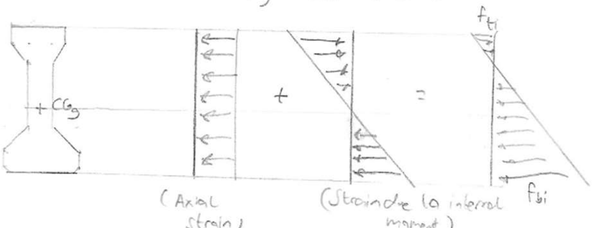
$\varepsilon_o = 0,002$

$f'_c = 40$ MPa

Initial prestressing strain in the steel due to P_{pe}

$$\varepsilon_{si} = \frac{P_{pe}}{A_{ps} \cdot E_{ps}} = \frac{3545,2 \cdot 10^3}{3158,4 \cdot 200000} = 0,0056$$

$0,0056 < 0,008$ ok



$$-\frac{P_{pe}}{A_{gt} \cdot E_{ps}} = \frac{-3545,2 \cdot 10^3}{528487,6385 \cdot 200000} = -3,3541 \cdot 10^{-5}$$

$$\varepsilon_{ti_p} = -\frac{P_{pe}}{A_{gt}\cdot E_{ps}} + \frac{P_{pe}\cdot e_c\cdot y_t}{I_{gt}\cdot E_{ps}} = -3,3541\cdot10^{-5} + \frac{35452\cdot10^{3}\cdot534,4895\cdot743,3605}{1,1391\cdot10^{11}\cdot200000} = 2,8287\cdot10^{-5}$$

$$\varepsilon_{bi_p} = -\frac{P_{pe}}{A_{gt}\cdot E_{ps}} - \frac{P_{pe}\cdot e_c\cdot y_b}{I_{gt}\cdot E_{ps}} = -3,3541\cdot10^{-5} - \frac{35452\cdot10^{3}\cdot534,4895\cdot628,2395}{1,1391\cdot10^{11}\cdot200000} = -8,5794\cdot10^{-5}$$

$$\varepsilon_{cip} = \varepsilon_{tip} + (\varepsilon_{bip} - \varepsilon_{tip})\cdot d\cdot\frac{1}{y_{total}} = 2,8287\cdot10^{-5} + (-8,5794\cdot10^{-5} - 2,8287\cdot10^{-5})\cdot1277,8\cdot\frac{1}{1371,6} = -7,7997\cdot10^{-5}$$

Assuming unshored construction

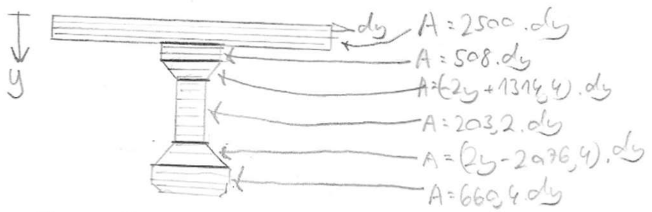
$$\varepsilon_{ti_{DL}} = -\frac{(M_G+M_S)\cdot y_t}{I_{gt}\cdot E_{ps}} = -\frac{(985,16+982,86)\cdot10^{6}\cdot743,3605}{1,1391\cdot10^{11}\cdot200000} = -6,3562\cdot10^{-5}$$

$$\varepsilon_{bi_{DL}} = \frac{(M_G+M_S)\cdot y_b}{I_{gt}\cdot E_{ps}} = \frac{(985,16+982,86)\cdot10^{6}\cdot628,2395}{1,1391\cdot10^{11}\cdot200000} = 5,3179\cdot10^{-5}$$

$$\varepsilon_{ci_{DL}} = \frac{\overset{e_c}{d-y_t}}{y_b}\cdot\varepsilon_{bi_{DL}} = \frac{534,4895}{628,2395}\cdot5,3179\cdot10^{-5} = 4,5703\cdot10^{-5}$$

$$\varepsilon_{ci} = |\varepsilon_{cip}| + \varepsilon_{c_{DL}} = 1,2370\cdot10^{-4}$$

* Area is divided into rectangles with height 0,01 m or 1cm ($dy = 0,01$ for numerical integration)



$$\varepsilon_s = \varepsilon_{si} + \varepsilon_{ci} + \varepsilon_{ctop}\cdot\frac{d_{composite} - c}{c}$$

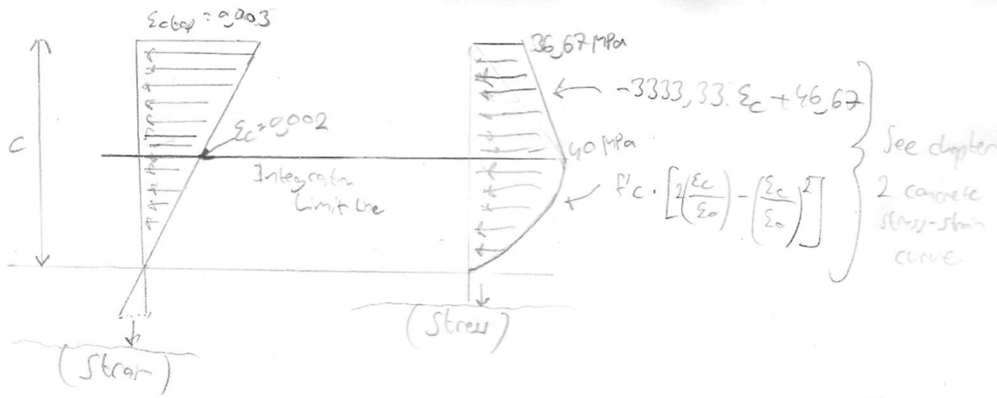
$$\varepsilon_s = 0,0056 + 1,2370\cdot10^{-4} + 0,003\cdot\frac{1477,8 - c}{c}$$

$$f_{ps} = E_{ps}\cdot\varepsilon_s = 200000\cdot\varepsilon_s \quad \text{for } \varepsilon_s \leq 0,008$$

$$f_{ps} = 1848 - 0,517\cdot\frac{1}{\varepsilon_s - 0,005915} \quad \text{for } \varepsilon_s > 0,008$$

$$f_{ps} = 0 \quad \text{for } f_{ps} > 0,99\cdot f_{pu}$$

*See chapter 2 Prestressing Steel Stress-Strain curve



$$\text{Integration Limit} = -(0{,}002 - \varepsilon_{ctop})\cdot\frac{c}{\varepsilon_{ctop}} = -(0{,}002 - 0{,}003)\cdot\frac{c}{0{,}003}$$

$$= 0{,}333\,c \ \text{from top}$$

$$\varepsilon_c = \varepsilon_{ctop} - \varepsilon_{ctop}\cdot\frac{y}{c}$$

$$T = f_{ps}\cdot A_{ps}$$

$$C = \int_0^{0{,}333c} (-3333{,}33\cdot\varepsilon_c + 46{,}67)\cdot A + \int_{0{,}333c}^{c} 40\cdot\left[2\cdot\left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right]\cdot A$$

*Solving T = C numerically using MATLAB, c = 76,02 mm (code available in the appendix section)

Let c = 76,02 mm

$$\varepsilon_s = 0{,}0056 + 1{,}2370\cdot10^{-4} + 0{,}003\cdot\frac{1477{,}8 - 76{,}02}{76{,}02} = 0{,}0611$$

$$\varepsilon_s > 0{,}008$$

$$\therefore f_{ps} = 1848 - 0{,}517\cdot\frac{1}{0{,}0611 - 0{,}005915} = 1838{,}61 \text{ MPa}$$

$$1838{,}61 \text{ MPa} < 0{,}99\cdot f_{pu} \ \text{OK} \ (1841{,}4)$$

$$\text{Integration Limit} = 0{,}333\,c = 25{,}34 \text{ mm}$$

$$T \cong C = f_{ps}\cdot A_{ps} = 1838{,}6\cdot 3158{,}4 = 5{,}807\cdot10^6 \text{ N}$$

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$$\bar{y} = \frac{\int_0^{0{,}333c} (-3333{,}33 \cdot \varepsilon_c + 46{,}67) \cdot A \cdot y + \int_{0{,}333c}^{c} 40 \cdot \left[2 \cdot \left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right] \cdot A \cdot y}{T}$$

$$\varepsilon_c = 0{,}003 - 0{,}003 \cdot \frac{y}{76{,}02}$$

$$\bar{y} = \frac{\left(\int_0^{25{,}34} \left(-3333{,}33 \cdot \left(0{,}003 - 0{,}003 \frac{y}{76{,}02}\right) + 46{,}67\right) \cdot 2500 \cdot y \cdot dy + \int_{25{,}34}^{76{,}02} 40 \cdot \left[2 \cdot \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}02}}{0{,}002}\right) - \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}02}}{0{,}002}\right)^2\right] \cdot 2500 \cdot y \cdot dy \right)}{(5{,}807 \cdot 10^6)}$$

$$\bar{y} = \frac{1{,}8104 \cdot 10^8}{5{,}807 \cdot 10^6} = 31{,}1783 \text{ mm}$$

$$z = d_{composite} - \bar{y} = 1477{,}9 - 31{,}1783 = 1446{,}7 \text{ mm}$$

$$M_r = z \cdot T \cdot 10^{-6} = 1446{,}7 \cdot 5{,}807 = 8401 \text{ kNm} \pm 0{,}5 \text{ kNm}$$

$$\boxed{8401 \text{ kNm} > 6669{,}16 \text{ kNm} \quad \text{OK}}$$

K & B

5.4.5 Reserve capacity

Moment resistance of the section at ultimate must be at least 1.2 times more than the cracking moment of the section. The reserve capacity check requirement can be waived if it is proven that the section has 1.33 times more moment resistance than the factored demand at ultimate.

The maximum moment experienced at ultimate is at $12.5\ m$ *and* $13.5\ m$ *from left support. It is equal to* $6673.18\ kNm$.

The moment resistance obtained by strain – compatibility is $8401\ kNm$.

$$1.33 \times 6673.18 = \mathbf{8875.34\ kNm > 8401\ kNm}$$

Therefore reserve capacity must be checked.

At service and at midspan:

$$f_b = -\frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c \times y_b}{I_g} + \frac{(M_g + M_s) \times y_b}{I_g} + \frac{(M_{SDL} + M_{LL}) \times y_{bc}}{I_c}$$

$$f_b = -\frac{3545.2 \times 10^3}{509031} - \frac{3545.2 \times 10^3 \times 534.4895 \times 628.2395}{1.0853 \times 10^{11}} + \frac{(985.16 + 962.86) \times 10^6 \times 628.2395}{1.0853 \times 10^{11}}$$

$$+ \frac{(301.98 + 2165.4) \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = \mathbf{2.2562\ MPa\ T}$$

At cracking, the bottom stress $= 0.6 \times \sqrt{f'_c} = 0.6 \times \sqrt{40} = \mathbf{3.7947\ MPa\ T}$

The additional moment must create a bottom stress of $3.7947 - 2.2562 = \mathbf{1.5386\ MPa\ T}$

$$\frac{M_{add} \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = 1.5386,\ \textit{solving for } M_{add},\ \mathbf{M_{add} = 425.9120\ kNm}$$

Therefore, $M_{cr} = 425.9120 + 985.16 + 962.86 + 301.98 + 2165.4 = \mathbf{4841.3\ kNm}$

$\mathbf{1.2 \times 4841.3\ kNm = 5809.6\ kNm < 8401\ kNm\ OK}$

5.4.6 Deflection limits check

During service and initial stage, the beam is under linear stresses with respect to the strains experienced. Therefore, most of the equations given here are for first order linear-elastic analysis.

The deflections experienced in ultimate stage is not the main concern of the design since the bridge is expected to never reach ultimate loading unless some extraordinary, extreme event happens. Nevertheless, the deflection is checked using stain-compatibility together with finite-element analysis. The ultimate deflections will not be presented in this report.

Deflections due to shear deformations are ignored in this report.

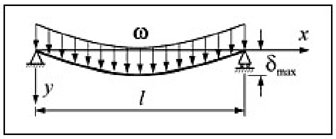
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED BRIDGE DEFLECTION AND MAXIMUM DEFLECTION	
	$y=\frac{\omega x}{24EI}\left(l^3-2lx^2+x^3\right)$	$\delta_{max}=\frac{5\omega l^4}{384EI}$

Figure 5.4.6.1 – Deflection equations for UDL on a simply supported beam

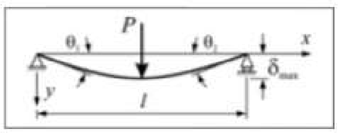
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED DEFLECTION AND MAXIMUM DEFLECTION	
	$y=\frac{Px}{12EI}\left(\frac{3l^2}{4}-x^2\right)$ for $0<x<\frac{l}{2}$	$\delta_{max}=\frac{Pl^3}{48EI}$

Figure 5.4.6.2 – Deflection equations for point load at midspan on a simply supported beam

Immediate deflection due to live load:

Immediate deflection due to live load can be calculated from the deflection occuring when applying the truck load at the midspan of the interior girder as a single point load. This simple method will give conservative results. If the deflection obtained is within the critical range, then the distribution and impact factors can be taken into account.

$$\Delta_L = \frac{P \times L^3}{48 \times E_c \times I_c} = \frac{325000 \times 26000^3}{48 \times 33994 \times 2.8960 \times 10^{11}} = 12\ mm\ downwards$$

Erection deflections:

Elastic Deflection due to girder self – weight:

$$\Delta_{DL} = \frac{5 \times w_G \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 11.66 \times 26000^4}{384 \times 33994 \times 1.0853 \times 10^{11}} = 19\ mm\ downwards$$

Elastic Deflection due to deck:

$$\Delta_{SL} = \frac{5 \times w_S \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 11.39 \times 26000^4}{384 \times 33994 \times 1.0853 \times 10^{11}} = 18\ mm\ downwards$$

Elastic Deflection due to asphalt and waterproofing:

$$\Delta_{PL} = \frac{5 \times w_{SDL} \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 3.57 \times 26000^4}{384 \times 33994 \times 1.0853 \times 10^{11}} = 6\ mm\ downwards$$

Upward Elastic Deflection due to Camber:

There are many different methods to calculate Camber. Camber calculations can be done using the " Hyperbolic Functions Method" proposed by Sinno Rauf and Howard L Furr (1970) or using the PCI s equations. However, in this report, camber is calulated using the approximate equations proposed by Collins and Mitchell.

$$\Delta_c = \left(\frac{e_c}{8} - \beta^2 \times \frac{(e_c - e_e)}{6}\right) \times P_{pi} \times \frac{L^2}{(E_c \times I_g)}$$

where:

β = *Ratio of harping length at one end with respect to total length*

e_e = *Average eccentricity at girder ends*

$$\beta = \frac{8.5}{26} = 0.327$$

Between 0 and 8.5 m from left support, the center of gravity of steel is given by this equation:

$$Distance\ from\ bottom\ to\ CGS\ [mm] = -\frac{313.425 - 93.75}{8500} \times Dist\ from\ left\ supp. + 313.425$$

Therefore CGS @ 0 m = 313.425 mm from bottom

$$e_c = 628.2395 - 313.425 = 314.8145\ mm$$

$$\Delta_c = \left(\frac{534.4895}{8} - 0.327^2 \times \frac{(534.4895 - 314.815)}{6}\right) \times 4089.8 \times 10^3 \times \frac{26000^2}{(33994 \times 1.0853 \times 10^{11})}$$

$$\boldsymbol{\Delta_c = 47\ mm\ upwards}$$

$$\textbf{Total deflection at erection} = 1.85 \times \Delta_{DL} + 1.8 \times \Delta_c = 1.85 \times 19 + 1.8 \times -47 = \mathbf{49\ mm\ upwards}$$

$$Total\ long\ term\ deflection = 2.4 \times \Delta_{DL} + 2.2 \times \Delta_c + 2.3 \times \Delta_{SL} + 3 \times \Delta_{PL}$$

$$\textbf{Total long term deflection} = 2.4 \times 19 + 2.2 \times -47 + 2.3 \times 18 + 3 \times 6 = \mathbf{2\ mm\ downwards}$$

$$\textbf{All deflections are within limit of } \frac{1}{800} \textit{ so this design is safe}$$

FLEXURAL DESIGN NOW COMPLETE

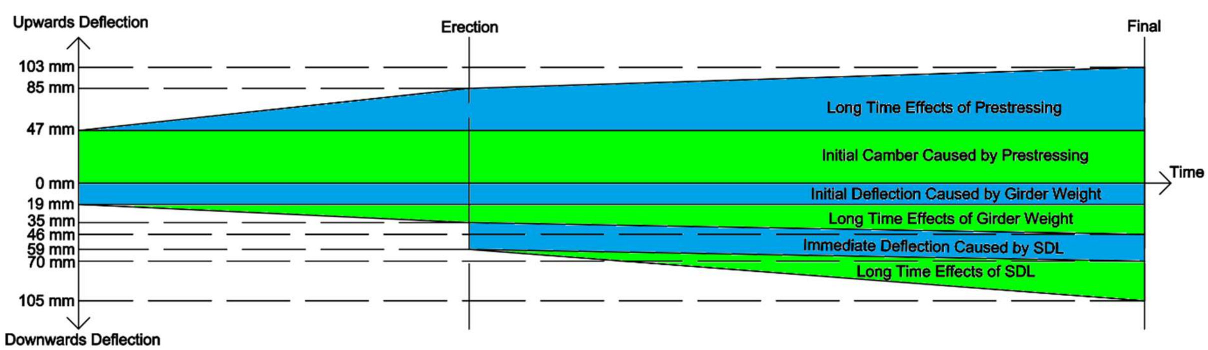


Figure 5.4.6.3 – Visual Representation of the Deflections Experienced

5.4.7 Design for shear and anchorage zone

For shear design, 15 M Canadian reinforcement bars with 16 mm diameter will be used. Each bar will therefore have an area of 200 mm^2, and 400 mm^2 when bent to be double legged. Ultimate shear values from chapter 4 must be used for the shear design

<u>Determination of V_p:</u>

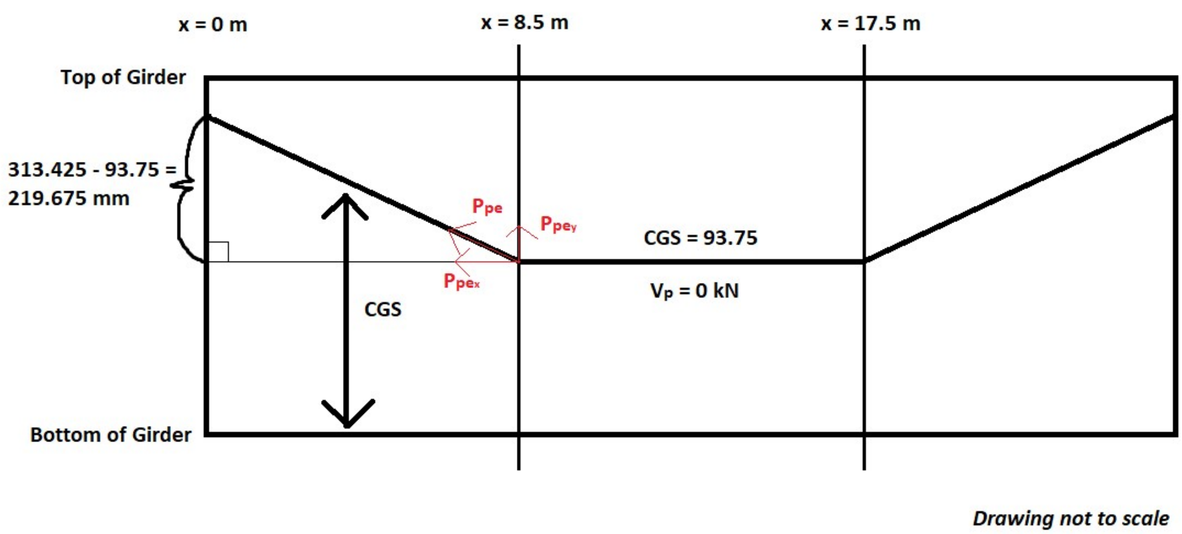


Figure 5.3.7.1 – Calculation of V_p

Determination of V_p (Component of effective prestressing force after all losses in the direction of applied shear. Positive if resisting shear, negative if adding to shear experienced):

V_p from $x = 0\,m$ to $x = 8.5\,m$ and $x = 17.5\,m$ to $x = 26\,m$

From figure above, P_{pe_y} is equal to V_p. Using the triangle from figure above

$$V_p = P_{pe} \times \sin\left(\arctan\left(\frac{219.675}{8500}\right)\right) = \mathbf{91.6\,kN}\ (P_{pe}\ was\ 3545.2\,kN)$$

Shear design equations changed in AASHTO in 2008 revisions. Before 2008, according to the commentary written about the shear section, there were tables that the designer has to choose values from for β and θ values. Therefore, a value was assumed and then updated until a safe value is obtained. So, shear design was an iterative process.

The procedure shown here is mostly similar to CSA S6-14 rev. 17 shear design since AASHTO LRFD 2014-17 uses mostly the same equations.

Determination of equivalent cracking parameter S_{ze}:

S_{ze} *can be taken as 300 mm as long as minimum shear reinforcement is provided.*

$$S_{ze} = 300\ mm$$

Determination of the longitudinal strain at the centroidal axis of the critical section ε_x:

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + V_u - V_p + 0.5 \times N_u - A_{ps} \times f_{po}}{2 \times (E_s \times A_s + E_p \times A_{ps})}$$

where:

$$M_u \geq (V_u - V_p) \times d_v$$

$$f_{po} = 0.7 \times f_{pu}$$

$$-0.0002\ (Conditionally) \leq \varepsilon_x \leq 0.003$$

At midspan:

$$\varepsilon_x = 0.001531$$

– The denominator of the equation above must be changed if $\varepsilon_x < 0$ *by* $2 \times (E_s \times A_s + E_p \times A_{ps} + E_c \times A_c)$ *and* ε_x *must be recalculated.*

Determination of the angle of inclination of the compressive stresses and value of beta:

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

$$\beta = \frac{0.4}{(1 + 1500 \times \varepsilon_x)} \times \frac{1300}{(1000 + S_{ze})}$$

$$At\ midspan:$$

$$\theta = (29 + 7000 \times 0.001531) \times \left(0.88 + \frac{300}{2500}\right) = \mathbf{39.72\ degrees}$$

$$\beta = \frac{0.4}{(1 + 1500 \times 0.001531)} \times \left(\frac{1300}{1000 + 300}\right) = \mathbf{0.12}$$

<u>Determination of the shear stress that can be resisted by concrete alone:</u>

$$V_c = 1.25 \times \beta \times \Phi_c \times f_{cr} \times b_v \times d_v$$

$$At\ midspan:$$

$$V_c = 1.25 \times 0.12 \times 1 \times 0.6 \times \sqrt{40} \times 203.2 \times 1150.07 \times 10^{-3} = \mathbf{134.49\ kN}$$

<u>Determination of the shear stress that must be resisted by the reinforcement:</u>

$$V_s = V_u - V_p - V_c \geq 0$$

$$At\ midspan:$$

$$V_s = 247.90 - 0 - 134.49 = \mathbf{113.41\ kN}$$

<u>Determination of the shear reinforcement spacing if $V_s > 0$:</u>

$$s_{required} = \frac{\Phi_s \times A_v \times f_y \times d_v \times cot(\theta)}{V_s}$$

$$At\ midspan:$$

$$s_{required} = \frac{0.9 \times 400 \times 400 \times 1150.07 \times cot(39.72)}{113.41 \times 10^3} = \mathbf{1757.71\ mm}$$

Determination of the shear stress on concrete v_u:

$$v_u = \frac{V_u - \Phi_s \times V_p}{\Phi_s \times b_v \times d_v}$$

At midspan:

$$v_u = \frac{247.90 \times 10^3 - 0.9 \times 0}{0.9 \times 203.2 \times 1150.07} = 1.18\,MPa$$

Determination of maximum shear reinforcement spacing:

$$\text{If } v_u \geq 0.125 \times f'_c$$

$$\text{then, } s_{max} = \text{Lesser of } 0.4 \times d_v \text{ or } 300\,mm$$

$$\text{If } v_u \geq 0.5 \times \sqrt{f'_c}$$

$$\text{then, } s_{max} = \text{Lesser of } 0.4 \times d_v \text{ or } 450\,mm$$

$$\text{If } v_u < 0.125 \times f'_c$$

$$\text{then, } s_{max} = \text{Lesser of } 0.8 \times d_v \text{ or } 600\,mm$$

At midspan:

$$v_u\,(1.18\,MPa) < 0.125 \times f'c\ (5\,MPa)$$

$$\text{therefore } s_{max} = \text{Lesser of } 920 \text{ and } 600\,mm$$

$$\text{therefore } s_{max} = 600\,mm$$

Determination of minimum shear reinforcement area:

$$A_{v,min} = 0.083 \times \sqrt{f'_c} \times \frac{b_v \times s}{f_y}$$

At midspan:

$$A_{v,min} = 0.083 \times \sqrt{40} \times \frac{203.2 \times 600}{400} = 160\,mm^2$$

Determination of anchorage zone reinforcement design for pretensioned members:

$P_r = f_s \times A_s$ *where*

$f_s = Min\ 140\ MPa\ (take\ 140\ MPa)$

$P_r = 0.04 \times P_i$ *where* $P_i = 4089.8\ kN$ *in this design*

Therefore $P_r = 163.6\ kN$

solving this for A_s

$$A_{s,required} = \frac{163.6 \times 10^3}{140} = 1170\ mm^2$$

This area must be distributed over a distance of $0.25 \times h = 343\ mm$ *from left support*

Leave 50 *mm for cover requirements.* ***Provide stirrups every 90 mm up to 410 mm.***

$\mathbf{A_{s,provided} = 1600\ mm^2 > 1170\ mm^2\ OK.}$

(Note: Extra 1 stirrup is provided to meet with shear demand together with anchorage zone requirements. Also, another extra isprovided for making spacing equal to equally distribute the stresses.)

– There must also be a stirrup every 150 *mm up to a distance of* $1.5 \times d_v$. *The cottom end of these stirrups must goaround the strands and cover them. Minimum 10 M bars are required for the bottom and this can be different from the top part. Therefore, reinforcement type in anchorage zone will be different from the regular stirrups.*

$1.5 \times d_v = 1.5 \times 987.55$ *(not at midspan, this value is at the ends of girder)* $= 1481\ mm$

Provide stirrups every 150 mm from 410 mm to 1610 mm from left support.

Design spacing to accommodate for shear:

From 1610 mm to 3110 mm from left support, provide stirrups every 300 mm based on s_{max} ***requirement.***

From 3110 mm to 6310 mm from left support, provide stirrups every 400 mm based on s_{max} ***requirement.***

From 6310 mm to 10150 mm from left support, provide stirrups every 480 mm based on $s_{required}$ ***and constructibility requirement.***

From 10150 mm to 15850 mm from left support, provide stirrups every 570 mm based on s_{max} ***requirement.***

Shear strength provided by the shear reinforcement:

$$V_{s,design} = \frac{\Phi_s \times A_v \times f_y \times d_v \times cot(\theta)}{s_{design}}$$

At midspan:

$$V_{s,design} = \frac{0.9 \times 400 \times 400 \times 1150.07 \times cot(39.72)}{570} \times 10^{-3} = \mathbf{349.72\ kN}$$

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Determination of V_c required and checking it against V_c available:

$$V_{c,\,needed} = V_u - V_s - V_p \leq V_{c,\,available}$$

$$V_{c,\,needed} \geq 0$$

At midspan:

$$V_{c,\,needed} = 247.90 - 349.72 - 0 = -101.82\,kN < 0\ therefore\ 0\,kN$$

$$V_{c,\,available} = 134.49\,kN > 0\,kN\ therefore\ OK$$

Forces in strands compared with force at ultimate design for flexure:

$$F_{lt} = \frac{M_u}{d_v} + 0.5 \times N_u + (V_u - 0.5 \times V_s - V_p) \times cot(\theta) < F_p$$

$$F_p = 5807.11\,kN\ from\ strain - compatibility\ anaylsis$$

At midspan:

$$d_v = 1446.67\ from\ strain - compatibility\ analysis\,.\ *$$

** The value code assumes is conservative. Designer has the option to do stain − compatibility analysis at every 10% at least to override the code value if the design is at the limits.*

$$F_{lt} = 4697 < 5807.11\,kN\ OK$$

Note on b_v and d_v Values: Those values are calculated to be conservative. In every section throughout the span, concrete shear resistance is greater than what actually calculated.

Table 5.4.7.1 – Final Shear Reinforcement Layout

From 50 mm to 410 mm	4 spacing @ 90 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1
From 410 mm to 1610 mm	8 spacing @ 150 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1
From 1610 mm to 3110 mm	5 spacing @ 300 mm c/c	15 M Double-Legged	Type 2
From 3110 mm to 6310 mm	8 spacing @ 400 mm c/c	15 M Double-Legged	Type 2
From 6310 mm to 10510 mm	8 spacing @ 480 mm c/c	15 M Double-Legged	Type 2
From 10150 mm to 15850 mm	10 spacing @ 570 mm c/c	15 M Double-Legged	Type 2
From 15850 mm to 19690 mm	8 spacing @ 480 mm c/c	15 M Double-Legged	Type 2
From 19690 mm to 22890 mm	8 spacing @ 400 mm c/c	15 M Double-Legged	Type 2
From 22890 mm to 24390 mm	5 spacing @ 300 mm c/c	15 M Double-Legged	Type 2
From 24390 mm to 25590 mm	8 spacing @ 150 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1
From 25590 mm to 25950 mm	4 spacing @ 90 mm c/c	15 M Double-Legged, Bottom closing 10 M	Type 1

Type 1 and Type 2 reinforcement drawings can be found at the appendix section of part B (at the very end of this report together with design drawings)

K & B

Table 5.4.7.2.a – Shear Design Calculations

x [m]	M_u [kNm]	V_u [kN]	CGS [mm]	d [mm]	d_v [mm]	V_p [kN]	ε_x initial	ε_x modified	θ [degrees]	β
0	0	1188.31	313.43	1058.18	987.55	91.63	-0.002387	-0.000043	28.70	0.43
0.5	521.76	1152.14	300.50	1071.10	987.55	91.63	-0.001997	-0.000036	28.75	0.42
1	1021.93	1115.97	287.58	1084.02	987.55	91.63	-0.001625	-0.000029	28.79	0.42
1.5	1500.50	1079.80	274.66	1096.94	987.55	91.63	-0.001270	-0.000023	28.84	0.41
2	1957.47	1043.63	261.74	1109.86	998.88	91.63	-0.000950	-0.000017	28.88	0.41
2.5	2392.84	1007.46	248.81	1122.79	1010.51	91.63	-0.000656	-0.000012	28.92	0.41
3	2806.61	971.29	235.89	1135.71	1022.14	91.63	-0.000385	-0.000007	28.95	0.40
3.5	3198.79	935.12	222.97	1148.63	1033.77	91.63	-0.000138	-0.000002	28.98	0.40
4	3569.36	898.95	210.05	1161.55	1045.40	91.63	0.000087	0.000087	29.61	0.35
4.5	3918.34	862.78	197.13	1174.47	1057.03	91.63	0.000290	0.000290	31.03	0.28
5	4245.72	826.61	184.20	1187.40	1068.66	91.63	0.000472	0.000472	32.30	0.23
5.5	4551.50	790.44	171.28	1200.32	1080.29	91.63	0.000633	0.000633	33.43	0.21
6	4835.69	754.27	158.36	1213.24	1091.92	91.63	0.000775	0.000775	34.42	0.18
6.5	5098.27	718.10	145.44	1226.16	1103.55	91.63	0.000898	0.000898	35.28	0.17
7	5339.26	681.93	132.52	1239.08	1115.18	91.63	0.001002	0.001002	36.01	0.16
7.5	5558.65	645.77	119.59	1252.01	1126.81	91.63	0.001088	0.001088	36.62	0.15
8	5756.44	609.60	106.67	1264.93	1138.44	91.63	0.001157	0.001157	37.10	0.15
8.5	5932.64	573.43	93.75	1277.85	1150.07	0	0.001282	0.001282	37.97	0.14
9	6096.67	537.26	93.75	1277.85	1150.07	0	0.001366	0.001366	38.56	0.13
9.5	6243.82	501.09	93.75	1277.85	1150.07	0	0.001439	0.001439	39.07	0.13
10	6369.38	464.92	93.75	1277.85	1150.07	0	0.001497	0.001497	39.48	0.12
10.5	6473.34	428.75	93.75	1277.85	1150.07	0	0.001540	0.001540	39.78	0.12
11	6555.70	392.58	93.75	1277.85	1150.07	0	0.001568	0.001568	39.97	0.12
11.5	6616.46	356.41	93.75	1277.85	1150.07	0	0.001581	0.001581	40.07	0.12
12	6655.62	320.24	93.75	1277.85	1150.07	0	0.001579	0.001579	40.05	0.12
12.5	6673.19	284.07	93.75	1277.85	1150.07	0	0.001563	0.001563	39.94	0.12
13	6669.15	247.90	93.75	1277.85	1150.07	0	0.001531	0.001531	39.72	0.12
13.5	6673.19	284.07	93.75	1277.85	1150.07	0	0.001563	0.001563	39.94	0.12
14	6655.62	320.24	93.75	1277.85	1150.07	0	0.001579	0.001579	40.05	0.12
14.5	6616.46	356.41	93.75	1277.85	1150.07	0	0.001581	0.001581	40.07	0.12
15	6555.70	392.58	93.75	1277.85	1150.07	0	0.001568	0.001568	39.97	0.12
15.5	6473.34	428.75	93.75	1277.85	1150.07	0	0.001540	0.001540	39.78	0.12
16	6369.38	464.92	93.75	1277.85	1150.07	0	0.001497	0.001497	39.48	0.12
16.5	6243.82	501.09	93.75	1277.85	1150.07	0	0.001439	0.001439	39.07	0.13
17	6096.67	537.26	93.75	1277.85	1150.07	0	0.001366	0.001366	38.56	0.13
17.5	5932.64	573.43	93.75	1277.85	1150.07	0	0.001282	0.001282	37.97	0.14
18	5756.44	609.60	106.67	1264.93	1138.44	91.63	0.001157	0.001157	37.10	0.15
18.5	5558.65	645.77	119.59	1252.01	1126.81	91.63	0.001088	0.001088	36.62	0.15
19	5339.26	681.93	132.52	1239.08	1115.18	91.63	0.001002	0.001002	36.01	0.16
19.5	5098.27	718.10	145.44	1226.16	1103.55	91.63	0.000898	0.000898	35.28	0.17
20	4835.69	754.27	158.36	1213.24	1091.92	91.63	0.000775	0.000775	34.42	0.18
20.5	4551.50	790.44	171.28	1200.32	1080.29	91.63	0.000633	0.000633	33.43	0.21
21	4245.72	826.61	184.20	1187.40	1068.66	91.63	0.000472	0.000472	32.30	0.23
21.5	3918.34	862.78	197.13	1174.47	1057.03	91.63	0.000290	0.000290	31.03	0.28
22	3569.36	898.95	210.05	1161.55	1045.40	91.63	0.000087	0.000087	29.61	0.35
22.5	3198.79	935.12	222.97	1148.63	1033.77	91.63	-0.000138	-0.000002	28.98	0.40
23	2806.61	971.29	235.89	1135.71	1022.14	91.63	-0.000385	-0.000007	28.95	0.40
23.5	2392.84	1007.46	248.81	1122.79	1010.51	91.63	-0.000656	-0.000012	28.92	0.41
24	1957.47	1043.63	261.74	1109.86	998.88	91.63	-0.000950	-0.000017	28.88	0.41
24.5	1500.50	1079.80	274.66	1096.94	987.55	91.63	-0.001270	-0.000023	28.84	0.41
25	1021.93	1115.97	287.58	1084.02	987.55	91.63	-0.001625	-0.000029	28.79	0.42
25.5	521.76	1152.14	300.50	1071.10	987.55	91.63	-0.001997	-0.000036	28.75	0.42
26	0	1188.31	313.43	1058.18	987.55	91.63	-0.002387	-0.000043	28.70	0.43

Table 5.4.7.2.b – Shear Design Calculations

x [m]	V_c [kN]	V_s [kN]	$s_{required}$ [mm]	v_u [MPa]	s_{max} [mm]	$s_{required}$ [mm]	$A_{v,min}$ [mm2]	s_{design} [mm]	Governed By	$V_{s,design}$ [kN]	$V_s + V_p$	$V_{c,needed}$	$V_{c,provided}$ OK?	F_{lt} [kN]	F_p [kN]
0	407.10	689.58	376.71	6.12	300	300	80.00	90	Anchorage Zone	2886.34	2977.97	0	OK	0	5807.11
0.5	402.55	657.95	394.01	5.92	300	300	80.00	150	Anchorage Zone	1728.27	1819.90	0	OK	886.33	5807.11
1	398.30	626.04	413.29	5.72	300	300	80.00	150	Anchorage Zone	1724.90	1816.54	0	OK	1329.35	5807.11
1.5	394.33	593.84	434.89	5.52	300	300	80.00	150	Anchorage Zone	1721.70	1813.34	0	OK	1750.63	5807.11
2	395.30	556.70	468.44	5.26	300	300	80.00	300	s_{max}	869.27	960.90	82.73	OK	2897.66	5807.11
2.5	396.65	519.18	507.36	5.01	300	300	80.00	300	s_{max}	878.04	969.67	37.79	OK	3231.09	5807.11
3	398.24	481.42	552.67	4.75	408.85	408.85	109.03	300	s_{max}	886.89	978.52	0	OK	3534.36	5807.11
3.5	400.06	443.43	606.07	4.51	413.51	413.51	110.27	400	s_{max}	671.87	763.50	171.62	OK	3627.15	5807.11
4	356.70	450.63	587.90	4.27	418.16	418.16	111.51	400	s_{max}	662.31	753.94	145.01	OK	3845.03	5807.11
4.5	284.11	487.04	519.57	4.04	422.81	422.81	112.75	400	s_{max}	632.63	724.26	138.52	OK	4037.86	5807.11
5	241.33	493.65	493.10	3.81	427.46	427.46	113.99	400	s_{max}	608.55	700.18	126.44	OK	4210.57	5807.11
5.5	213.63	485.18	485.67	3.58	432.11	432.11	115.23	400	s_{max}	589.10	680.73	109.72	OK	4364.63	5807.11
6	194.68	467.96	490.27	3.36	436.77	436.77	116.47	400	s_{max}	573.56	665.20	89.08	OK	4371.10	5807.11
6.5	181.31	445.16	504.47	3.15	600	504.47	160.00	480	$s_{required}$	467.85	559.49	158.62	OK	4458.47	5807.11
7	171.77	418.53	527.83	2.94	600	527.83	160.00	480	$s_{required}$	460.24	551.87	130.07	OK	4552.69	5807.11
7.5	165.02	389.11	561.12	2.73	600	561.12	160.00	480	$s_{required}$	454.87	546.50	99.27	OK	4631.65	5807.11
8	160.42	357.54	606.22	2.53	600	600.00	160.00	480	Constructibilility	451.56	543.19	66.40	OK	4695.03	5807.11
8.5	151.69	421.74	503.08	2.73	600	503.08	160.00	480	$s_{required}$	442.01	442.01	131.41	OK	4552.38	5807.11
9	145.40	391.85	530.10	2.55	600	530.10	160.00	480	$s_{required}$	432.75	432.75	104.50	OK	4616.75	5807.11
9.5	140.38	360.70	565.51	2.38	600	565.51	160.00	480	$s_{required}$	424.96	424.96	76.13	OK	4671.47	5807.11
10	136.64	328.28	612.47	2.21	600	600.00	160.00	480	Constructibilility	418.88	418.88	46.04	OK	4712.95	5807.11
10.5	133.98	294.77	674.86	2.04	600	600.00	160.00	570	s_{max}	348.99	348.99	79.75	OK	4780.04	5807.11
11	132.29	260.28	758.97	1.87	600	600.00	160.00	570	s_{max}	346.57	346.57	46.00	OK	4793.15	5807.11
11.5	131.52	224.89	875.55	1.69	600	600.00	160.00	570	s_{max}	345.44	345.44	10.97	OK	4791.97	5807.11
12	131.62	188.62	1044.33	1.52	600	600.00	160.00	570	s_{max}	345.58	345.58	0	OK	4776.02	5807.11
12.5	132.59	151.47	1305.78	1.35	600	600.00	160.00	570	s_{max}	347.00	347.00	0	OK	4744.84	5807.11
13	134.49	113.41	1757.71	1.18	600	600.00	160.00	570	s_{max}	349.72	349.72	0	OK	4697.91	5807.11
13.5	132.59	151.47	1305.78	1.35	600	600.00	160.00	570	s_{max}	347.00	347.00	0	OK	4744.84	5807.11
14	131.62	188.62	1044.33	1.52	600	600.00	160.00	570	s_{max}	345.58	345.58	0	OK	4776.02	5807.11
14.5	131.52	224.89	875.55	1.69	600	600.00	160.00	570	s_{max}	345.44	345.44	10.97	OK	4791.97	5807.11
15	132.29	260.28	758.97	1.87	600	600.00	160.00	570	s_{max}	346.57	346.57	46.00	OK	4793.15	5807.11
15.5	133.98	294.77	674.86	2.04	600	600.00	160.00	570	s_{max}	348.99	348.99	79.75	OK	4780.04	5807.11
16	136.64	328.28	612.47	2.21	600	600.00	160.00	480	Constructibilility	418.88	418.88	46.04	OK	4712.95	5807.11
16.5	140.38	360.70	565.51	2.38	600	565.51	160.00	480	$s_{required}$	424.96	424.96	76.13	OK	4671.47	5807.11
17	145.40	391.85	530.10	2.55	600	530.10	160.00	480	$s_{required}$	432.75	432.75	104.50	OK	4616.75	5807.11
17.5	151.69	421.74	503.08	2.73	600	503.08	160.00	480	$s_{required}$	442.01	442.01	131.41	OK	4552.38	5807.11
18	160.42	357.54	606.22	2.53	600	600.00	160.00	480	Constructibilility	451.56	543.19	66.40	OK	4695.03	5807.11
18.5	165.02	389.11	561.12	2.73	600	561.12	160.00	480	$s_{required}$	454.87	546.50	99.27	OK	4631.65	5807.11
19	171.77	418.53	527.83	2.94	600	527.83	160.00	480	$s_{required}$	460.24	551.87	130.07	OK	4552.69	5807.11
19.5	181.31	445.16	504.47	3.15	600	504.47	160.00	480	$s_{required}$	467.85	559.49	158.62	OK	4458.47	5807.11
20	194.68	467.96	490.27	3.36	436.77	436.77	116.47	400	s_{max}	573.56	665.20	89.08	OK	4371.10	5807.11
20.5	213.63	485.18	485.67	3.58	432.11	432.11	115.23	400	s_{max}	589.10	680.73	109.72	OK	4364.63	5807.11
21	241.33	493.65	493.10	3.81	427.46	427.46	113.99	400	s_{max}	608.55	700.18	126.44	OK	4210.57	5807.11
21.5	284.11	487.04	519.57	4.04	422.81	422.81	112.75	400	s_{max}	632.63	724.26	138.52	OK	4037.86	5807.11
22	356.70	450.63	587.90	4.27	418.16	418.16	111.51	400	s_{max}	662.31	753.94	145.01	OK	3845.03	5807.11
22.5	400.06	443.43	606.07	4.51	413.51	413.51	110.27	400	s_{max}	671.87	763.50	171.62	OK	3627.15	5807.11
23	398.24	481.42	552.67	4.75	408.85	408.85	109.03	300	s_{max}	886.89	978.52	0	OK	3534.36	5807.11
23.5	396.65	519.18	507.36	5.01	300	300	80.00	300	s_{max}	878.04	969.67	37.79	OK	3231.09	5807.11
24	395.30	556.70	468.44	5.26	300	300	80.00	300	s_{max}	869.27	960.90	82.73	OK	2897.66	5807.11
24.5	394.33	593.84	434.89	5.52	300	300	80.00	150	Anchorage Zone	1721.70	1813.34	0	OK	1750.63	5807.11
25	398.30	626.04	413.29	5.72	300	300	80.00	150	Anchorage Zone	1724.90	1816.54	0	OK	1329.35	5807.11
25.5	402.55	657.95	394.01	5.92	300	300	80.00	150	Anchorage Zone	1728.27	1819.90	0	OK	886.33	5807.11
26	407.10	689.58	376.71	6.12	300	300	80.00	90	Anchorage Zone	2886.34	2977.97	0	OK	0	5807.11

SHEAR DESIGN NOW COMPLETE

5.4.8 Design for shrinkage and temperature variation

$$A_s > \frac{0.0018 \times 415 \times b \times h}{2 \times (b + h) \times f_y} \text{ where } A_s \text{ must be between } 233\ mm^2 \text{ per } m \text{ and } 1270\ mm^2 \text{ per } m$$

$$A_s > \frac{0.0018 \times 415 \times 203.2 \times 1371.6}{2 \times (1317.6 + 203.2) \times 400} \times 10^3$$

$$A_s > 171\ mm^2 \text{ per } m$$

Therefore $A_s > 233\ mm^2$ *per m in each direction.*

Max spacing for shrinkage and temperature reinforcement is 450 mm.

Provide:

4 − 15M bars @ 300 in the direction parallel to span. $A_s = 800\ mm^2\ (667\ mm^2/m)$

3 − 10M bars @ 260 in the direction transverse to span $A_s = 300\ mm^2\ (577\ mm^2/m)$

5.5 CSA S6-66

5.5.1 Estimation of Required Prestress and Initial Strand Pattern

Bottom tensile stress at midspan during service according to service combination in CSA S6 − 66:

$$f_b = \frac{M_G + M_S}{s_b} + \frac{M_{SDL} + M_{LL}}{s_{bc}}$$

$$f_b = \frac{(1053.82 + 1014) \times 10^6}{1.7275 \times 10^8} + \frac{(322.68 + 1598.02) \times 10^6}{2.7683 \times 10^8} = \mathbf{18.9082\ MPa}$$

M_G = *Moment due to self − weight of girder at midspan*

M_S = *Moment due to self − weight of deck at midspan*

M_{SDL} = *Moment due to self − weight of asphalt and waterproofing at midspan*

M_{LL} = *Moment due to live load at midpsan*

At service loading conditions, allowable tensile stress according to CSA S6 − 66 is:

$$F_b = 0.5 \times \sqrt{f'_c \ for\ girder} = 0.5 \times \sqrt{40} = \mathbf{3.162\ MPa}$$

Required Number of Strands:

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress − allowable tensile stress at final $= f_b - F_b$

$$f_{pb} = 18.9082 - 3.162 = \mathbf{15.7459\ MPa}$$

Assuming the distance from center of gravity of strands to the bottom fiber of the beam is equal to

$$y_{bs} = \mathbf{100\ mm}$$

Strand eccentricity at mispan:

$$e_c = y_b - y_{bs} = 628.2395 - 100 = \mathbf{528.2395\ mm}$$

Bottom fiber stress due to prestress after losses:

$$f_{b\_prestress} = \frac{P_{pe}}{A_g} + \frac{P_{pe} \times e_c}{s_b} \quad \text{where } P_{pe} = \text{Effective prestressing force alfter all losses}$$

$$15.7459 = \frac{P_{pe} \times 10^3}{5.0903 \times 10^5} + \frac{P_{pe} \times 528.2395 \times 10^3}{1.7275 \times 10^8}$$

solving this for P_{pe}, $\mathbf{P_{pe} = 3135.17\ kN}$

Assuming final losses is 20% *of* f_{pi} *(for now)*

Assumed final losses $= 0.2 \times 1395 = \mathbf{279\ MPa}$

The prestress force per strand after losses $=$ *Cross − sectional area of one strand* $\times$ $(f_{pi}\ -\ losses)$

$= 98.7 \times (1395 - 279) \times 10^{-3} = \mathbf{110.1492\ kN}$

Try **32** *Strands as an initial trial*:

Effective strand eccentricity at midspan after strand arrangement

$$e_c = 628.2395 - \frac{12 \times (50 + 100) + 8 \times 150}{32} = \mathbf{534.4895\ mm}$$

$$P_{pe} = 32 \times 110.1492 = \mathbf{3524.8\ kN}$$

$$f_b = \frac{3524.8 \times 10^3}{5.0903 \times 10^5} + \frac{534.4895 \times 3524.8 \times 10^3}{1.7275 \times 10^8} = \mathbf{17.83\ MPa}$$

17.83 *MPa* > 15.7459 *MPa therefore OK*

Therefore use **32** ***strands***

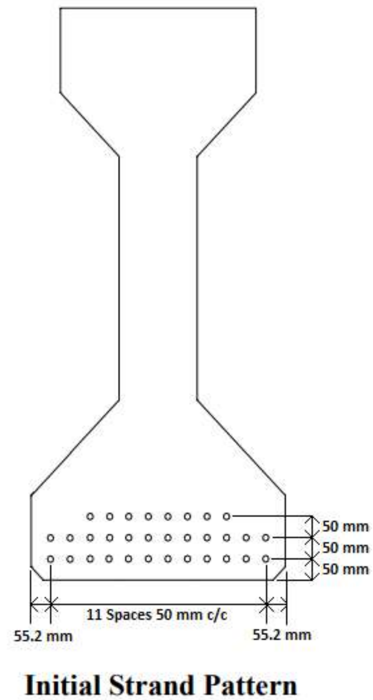


Figure 5.5.1.1 – Initial Strand Pattern

5.5.2 Prestressing Losses

In CSA S6 − 66, prestressing losses for pretensioned are assumed to be 240 MPa (35000 psi)

Initial losses are assumed to be 105 MPa (15000 psi)

$$\% \, Loss = \frac{240}{f_{pi}} \times 100 = \frac{240}{1395} \times 100 = 17.2043 \, \%$$

$$\% \, Initial \; Loss = \frac{105}{f_{pi}} \times 100 = \frac{105}{1395} \times 100 = 7.5269 \, \%$$

$$Final \; effective \; prestress, \; f_{pe} = f_{pi} - \Delta f_{pT} = 1395 - 240 = 1155 \, MPa$$

$$At \; service, \; f_{pe} \leq 1488 \, MPa \; OK \; (0.8 \times f_{pu})$$

$$Total \; prestressing \; force \; after \; all \; losses, \; P_{pe} = 32 \times 98.7 \times 1155 \times 10^{-3} = 3647.95 \, kN$$

$$Initial \; prestressing \; force \; after \; initial \; losses, \; P_{i} = 32 \times 98.7 \times (1395 - 105) \times 10^{-3} = 4074.34 \, kN$$

Final stress at the bottom fiber in midspan:

$$f_b = \frac{P_{pe}}{A_g} + \frac{P_{pe} \times e_c}{s_b} = \frac{3647.95 \times 10^3}{509031} - \frac{3647.95 \times 10^3 \times 534.4895}{1.7275 \times 10^8} = 18.4531\ MPa > 15.7459\ OK$$

5.5.3 Concrete stress limits at top and bottom

5.5.3.1 Stress limits at transfer and Strand Pattern

At Midspan:

At transfer, the compressive stress in the top fiber cannot exceed:

$$f_{ti} = 0.6 \times 35 = 21\ MPa$$

$$f_{ti} \geq \frac{P_i}{A_g} - \frac{P_i \times e_c}{s_t} + \frac{M_G}{s_t}$$

$$f_{ti} = \frac{4074.3 \times 10^3}{509031} - \frac{4074.3 \times 10^3}{1.46 \times 10^8} + \frac{1053.82 \times 10^6}{1.46 \times 10^8} = 0.3063\ MPa\ OK$$

At transfer, the compressive stress in the bottom fiber cannot exceed:

$$f_{bi} = 0.6 \times 35 = 21\ MPa$$

$$f_{bi} \geq \frac{P_i}{A_g} + \frac{P_i \times e_c}{s_b} - \frac{M_G}{s_b}$$

$$f_{bi} = \frac{4074.3 \times 10^3}{509031} + \frac{4074.3 \times 10^3}{1.7275 \times 10^8} - \frac{1053.82 \times 10^6}{1.7275 \times 10^8} = 14.5097\ MPa\ OK$$

This same procedure is done for every 0.5 m of span and limits of eccentricities are determined using excel. This will serve to determine the optimal hold down points for harped strands.

The beam is divided into 53 pieces in longitudinal direction. Every cross-section of these 52 pieces is divided into 1372 pieces resulting in 72716 elements. For all small elements, stresses are calculated as if the strands weren't harped.

For straight strands, entirety of the beam was within limits of compression allowed at transfer. However, as expected, the top ends of the beam exceeded the tensile stress limit allowed by CSA S6-66.

At transfer, the tensile stress in concrete cannot exceed:

$$f_{\text{tensile allowed}} = 0.25 \times \sqrt{35} = 1.479\ MPa$$

Figure below shows in red where tensile stress exceeds 1.479 MPa. The green elements are within limits of stress.

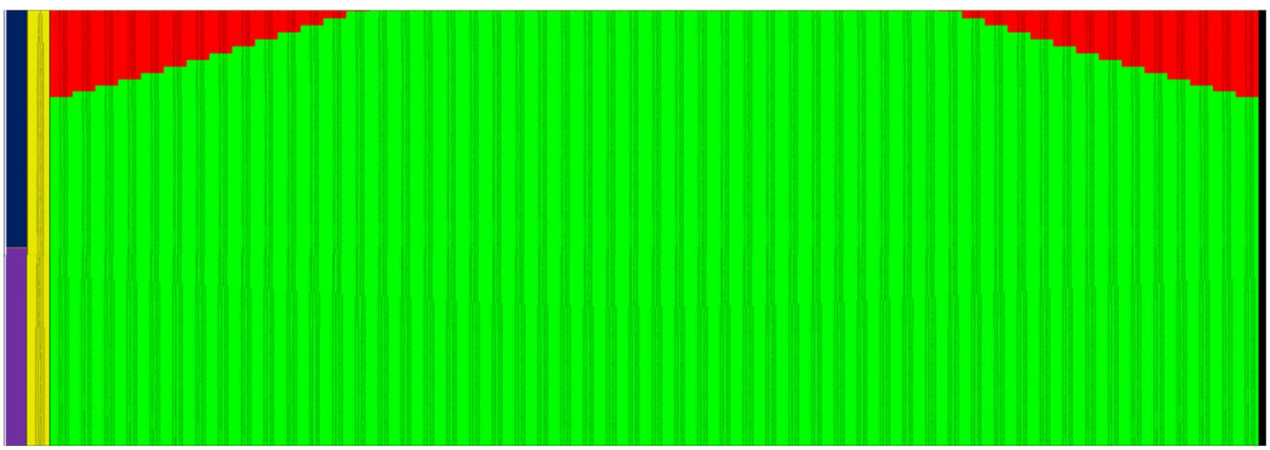


Figure 5.5.3.1.1 – Straight Strands – Stresses experienced

Looking at the stress values, optimal hold down points determined to be x = 8.5 m and x = 17.5 m from left support.

The strand profile below is determined to give the best stress results (32 12.7 mm strands with the arrangement and pattern below):

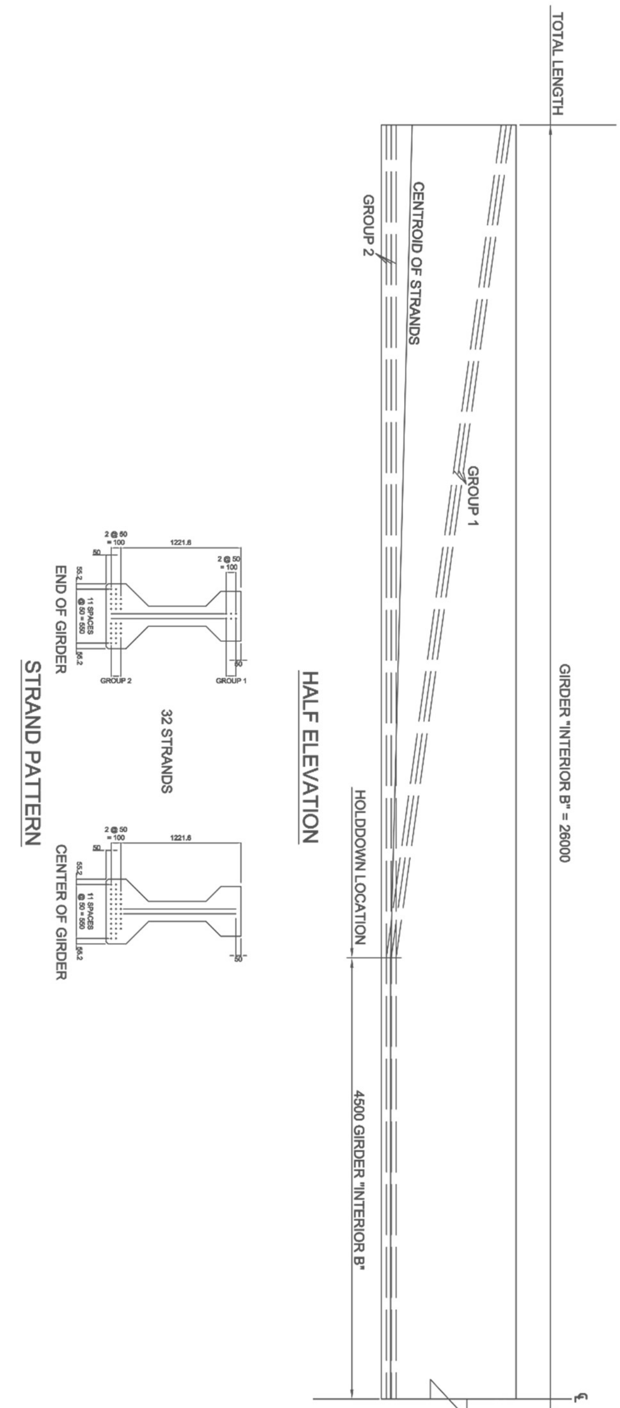


Figure 5.5.3.1.2 – Strand Pattern

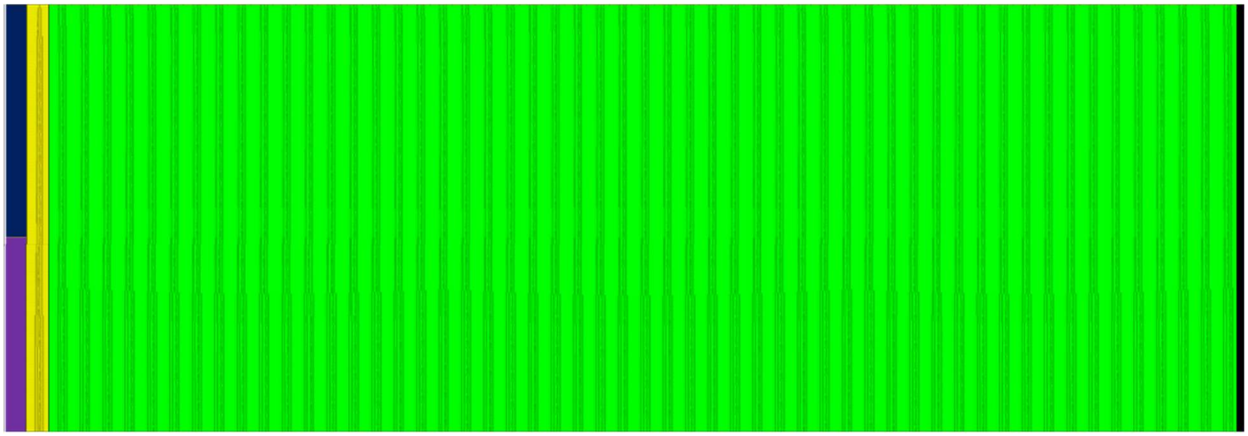


Figure 5.5.3.1.3 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced

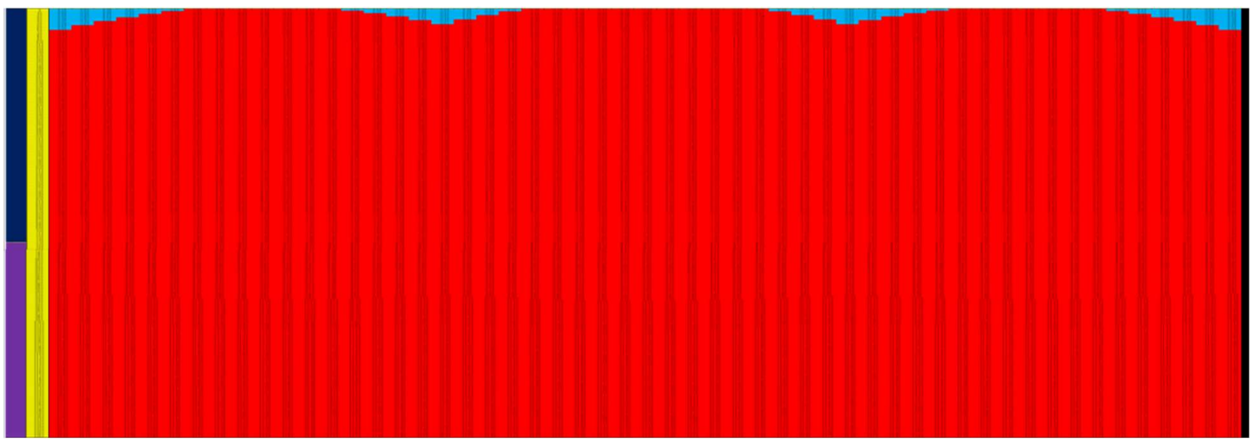


Figure 5.5.3.1.4 – Harped Strands with groups as in figure 5.5.3.1.2 – Elements in tension Blue, Elements in compression Red

$$Maximum\ stresses\ recorded\ at\ transfer\ are:$$

$$0.78\,MPa\ for\ tension < 0.25 \times \sqrt{35}\ (1.479\,MPa)\ OK$$

$$15.42\,MPa\ for\ compression < 0.6 \times 35\ (21\,MPa)\ OK$$

Table 5.5.3.1.1 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced [-Compression, +Tension]

Distance From Left Support	Maximum Top Stress (MPa)	Maximum Bottom Stress (MPa)
0	0.78	-15.42
0.5	0.60	-15.27
1	0.43	-15.13
1.5	0.29	-15.01
2	0.17	-14.91
2.5	0.08	-14.83
3	0.00	-14.76
3.5	-0.06	-14.71
4	-0.09	-14.68
4.5	-0.11	-14.67
5	-0.10	-14.68
5.5	-0.07	-14.71
6	-0.02	-14.75
6.5	0.06	-14.81
7	0.15	-14.89
7.5	0.26	-14.99
8	0.40	-15.10
8.5	0.56	-15.23
9	0.38	-15.08
9.5	0.22	-14.95
10	0.08	-14.83
10.5	-0.04	-14.73
11	-0.14	-14.65
11.5	-0.21	-14.58
12	-0.26	-14.54
12.5	-0.30	-14.51
13	-0.31	-14.50
13.5	-0.30	-14.51
14	-0.26	-14.54
14.5	-0.21	-14.58
15	-0.14	-14.65
15.5	-0.04	-14.73
16	0.08	-14.83
16.5	0.22	-14.95
17	0.38	-15.08
17.5	0.56	-15.23
18	0.40	-15.10
18.5	0.26	-14.99
19	0.15	-14.89
19.5	0.06	-14.81
20	-0.02	-14.75
20.5	-0.07	-14.71
21	-0.10	-14.68
21.5	-0.11	-14.67
22	-0.09	-14.68
22.5	-0.06	-14.71
23	0.00	-14.76
23.5	0.08	-14.83
24	0.17	-14.91
24.5	0.29	-15.01
25	0.43	-15.13
25.5	0.60	-15.27
26	0.78	-15.42

MAXIMUM TENSION	=	0.78
MAXIMUM COMPRESSION	=	-15.42

5.4.3.2 Service conditions

At Midspan:

At service, the compressive stress in top fiber cannot exceed:

$$f_{ts} = 0.45 \times 40 = 18\,MPa$$

$$P_{pe}\ @\ midspan = 3648\,kN$$

$$f_{ts} \geq \frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c}{s_t} + \frac{M_G + M_S}{s_t} + \frac{M_{SDL} + M_{LL}}{\dfrac{I_c}{(y_{tc} - 200)}}$$

$$f_{ts} = \frac{3648 \times 10^3}{509031} - \frac{3648 \times 10^3 \times 534.4895}{1.46 \times 10^8} + \frac{(1053.82 + 1014) \times 10^6}{1.46 \times 10^8}$$

$$+ \frac{(322.68 + 1598.02) \times 10^6}{\dfrac{2.896 \times 10^{11}}{(525.4544 - 200)}} = 10.1272\,MPa\ OK$$

At service, the tensile stress in the bottom fiber cannot exceed:

$$f_{bs} = 0.5 \times \sqrt{40} = 3.162\,MPa$$

$$f_{bs} \geq -\frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c}{s_b} + \frac{M_G + M_S}{s_b} + \frac{M_{SDL} + M_{LL}}{s_{bc}}$$

$$f_{bs} = -\frac{3648 \times 10^3}{509031} - \frac{3648 \times 10^3 \times 534.4895}{1.7275 \times 10^8} + \frac{(1053.82 + 1014) \times 10^6}{1.7275 \times 10^8}$$

$$+ \frac{(322.68 + 1598.02) \times 10^6}{2.7683 \times 10^8} = 0.4352\,MPa\ OK$$

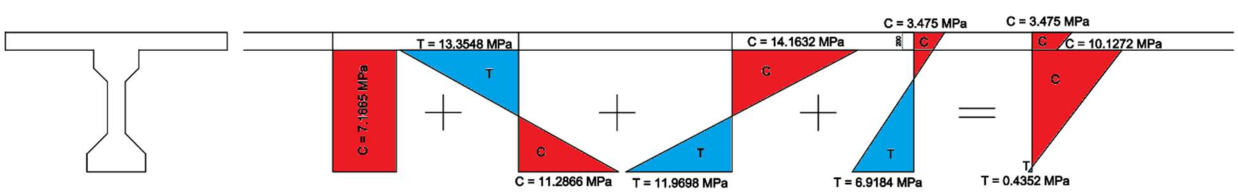


Figure 5.5.3.2.1 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced visualized at midspan

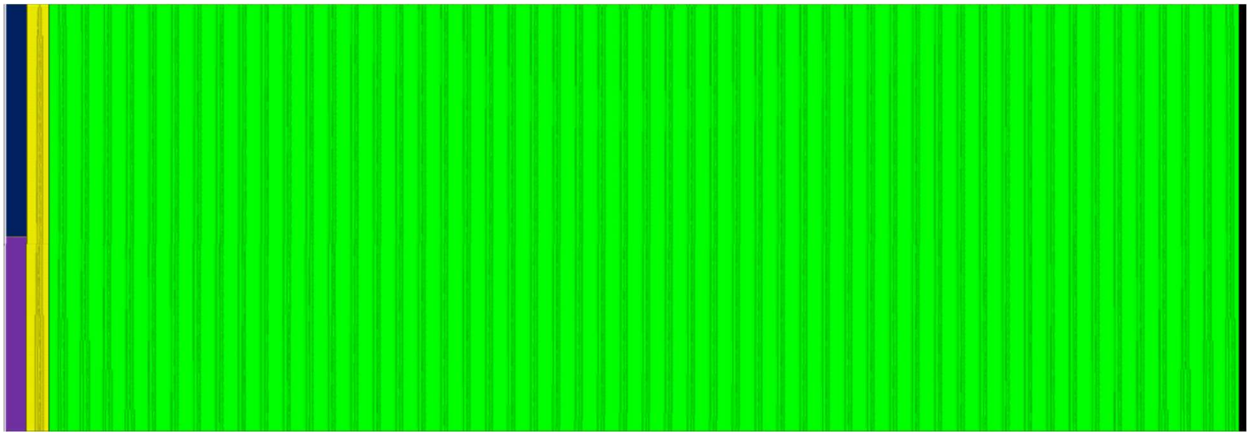


Figure 5.5.3.2.2 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced at service conditions

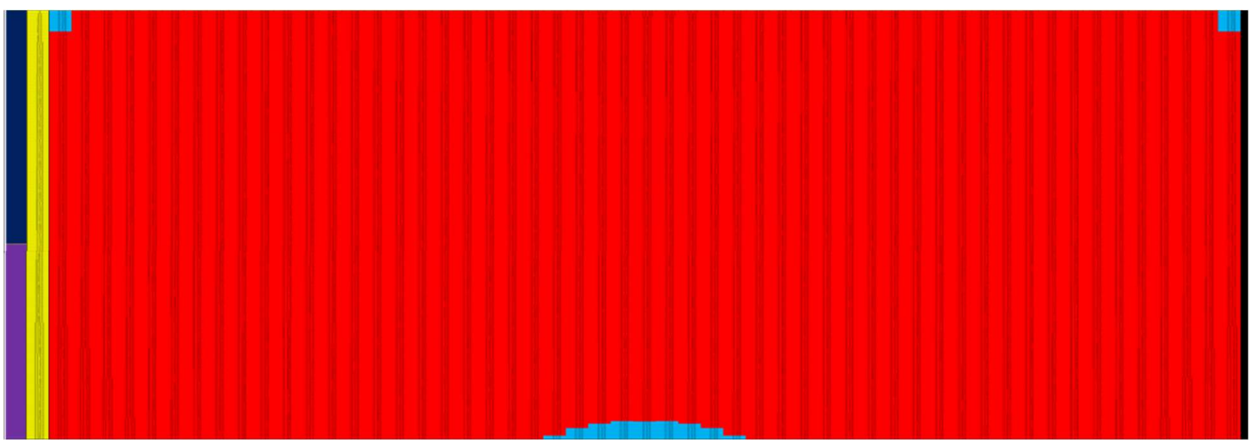


Figure 5.5.3.2.3 – Harped Strands with groups as in figure 5.5.3.1.2 – Elements in tension Blue, Elements in compression Red – Service Conditions

Maximum stresses recorded at service are:

$0.70\,MPa$ *for tension* $< 0.5 \times \sqrt{40}\ (3.162\,MPa)$ *OK*

$13.81\,MPa$ *for compression* $< 0.45 \times 40\ (18\text{ MPa})$ *OK*

Table 5.5.3.2.1 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced by the girder during service [-Compression, +Tension]

Distance From Left Support	Maximum Top Stress (MPa)	Maximum Bottom Stress (MPa)
0	0.70	-13.81
0.5	-0.22	-12.62
1	-1.09	-11.49
1.5	-1.91	-10.42
2	-2.68	-9.41
2.5	-3.41	-8.46
3	-4.08	-7.57
3.5	-4.70	-6.74
4	-5.27	-5.96
4.5	-5.80	-5.25
5	-6.27	-4.60
5.5	-6.70	-4.01
6	-5.91	-2.31
6.5	-7.40	-3.00
7	-7.68	-2.59
7.5	-7.91	-2.23
8	-8.08	-1.94
8.5	-8.21	-1.70
9	-8.62	-1.24
9.5	-8.98	-0.82
10	-9.29	-0.46
10.5	-9.56	-0.16
11	-9.77	0.07
11.5	-9.93	0.25
12	-10.05	0.37
12.5	-10.11	0.43
13	-10.13	0.43
13.5	-10.11	0.43
14	-10.05	0.37
14.5	-9.93	0.25
15	-9.77	0.07
15.5	-9.56	-0.16
16	-9.29	-0.46
16.5	-8.98	-0.82
17	-8.62	-1.24
17.5	-8.21	-1.70
18	-8.08	-1.94
18.5	-7.91	-2.23
19	-7.68	-2.59
19.5	-7.40	-3.00
20	-7.07	-3.47
20.5	-6.70	-4.01
21	-6.27	-4.60
21.5	-5.80	-5.25
22	-5.27	-5.96
22.5	-4.70	-6.74
23	-4.08	-7.57
23.5	-3.41	-8.46
24	-2.68	-9.41
24.5	-1.91	-10.42
25	-1.09	-11.49
25.5	-0.22	-12.62
26	0.70	-13.81

MAXIMUM TENSION	=	***0.70***
MAXIMUM COMPRESSION	=	***-13.81***

5.5.4 Ultimate Flexural Capacity

Ultimate flexural capacity of the composite section can be calculated in two ways.

The first and most commonly used method that works for every section is strain compatibility analysis. In this method, the section is divided into small rectangles and stresses are assumed constant throughout the small rectangle. Each of the rectangle will have a resultant force. The moment caused by all resultant forces are assembled into 1 compressive force with a certain distance from the centroid. Equating tensile force at the level of center of gravity of steel with this compressive force gives the magnitude of the compressive force. Ultimate moment capacity (Mr) is then determined by multiplying tensile or compressive force by the moment arm.

The concrete stress-strain curve used for the strain compatibility analysis presented in this report is based on the Hognestad's Modified Parabola. The prestressing steel and concrete stress-strain curve is given in the chapter 2 of this report.

Another way that is simpler and gives good enough results for most sections is assuming a rectangular stress pattern (Whitney's Stress Block). Whitney's Stress Block parameters are used by CSA S6-66. It is still required to iterate to find for the location of compressive force with this method if the centroid of compressive forces is not in a rectangular section.

So, both ways, the usage of a computer program is very helpful.

5.4.4.1 Rectangular Stress Block Assumption

RECTANGULAR SECTION ASSUMPTION AT MIDSPAN

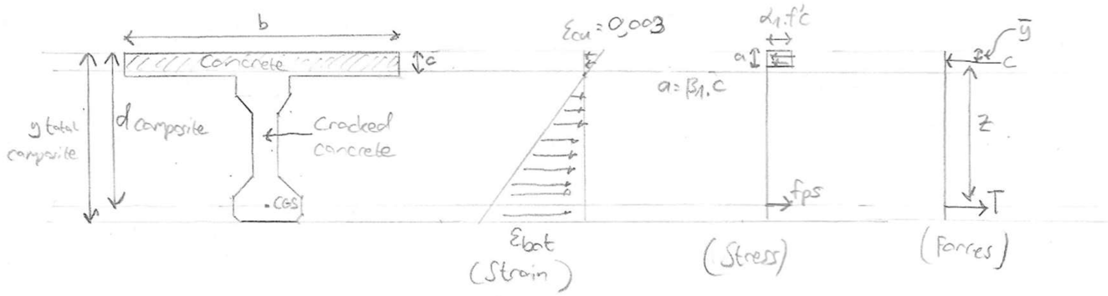
→ Flexural demand at midspan, $M_f = 7567{,}03$ kNm (From chapter 4)

According to CSA S6-66

Stress block parameters:

$\alpha_1 = 0{,}85$ $\phi_p = 1$ $f_{pu} = 1860$ MPa

$\beta_1 = 0{,}85$ $\phi_c = 1$ $f'_c = 40$ MPa



a) Without reduction factors ϕ_p, ϕ_c – they are assumed 1

$$a = \frac{A_{ps} . f_{ps}}{\alpha_1 . f'_c . b} = \frac{98{,}7 . 32 . f_{ps}}{0{,}85 . 40 . 2500} = 0{,}0372 . f_{ps}$$

$$\frac{c_u}{d_p} = \frac{A_{ps} . f_{pu}}{\alpha_1 . \beta_1 . f'_c . b . d} = \frac{98{,}7 . 32 . 1860}{0{,}85 . 0{,}85 . 40 . 2500 . 1477{,}8} = 0{,}055$$

$$k_p = 3\left(1 - \frac{f_{py}}{f_{pu}}\right) = 3 . (1 - 0{,}9) = 0{,}3 \text{ or } 0{,}28 \text{ for low relaxation}$$

($\frac{f_{py}}{f_{pu}} = 0{,}9$) both OK

$$f_{ps} = f_{pu}\left(1 - k_p . \frac{c_u}{d_p}\right) = 1860 . (1 - 0{,}28 . 0{,}055) = 1831 \text{ MPa}$$

$a = 0{,}0372 . f_{ps} = 0{,}0372 \; 1831 = 68{,}05$ mm $\quad a < t_s$ (200 mm) ∴ Rectangular assumption can be used.

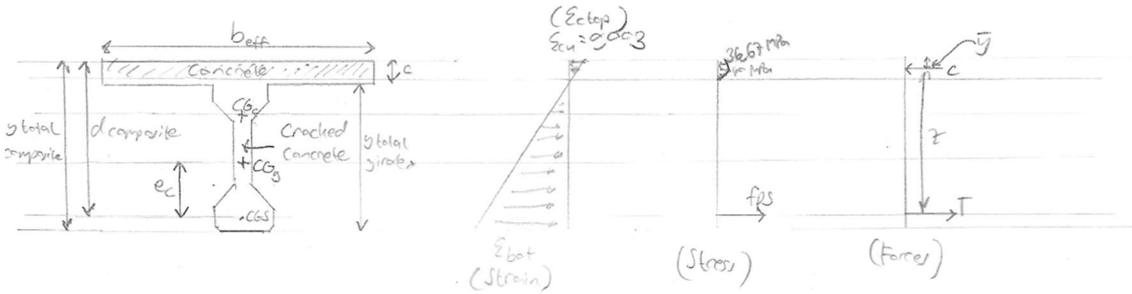
$$M_r = \underbrace{A_{ps} . f_{ps}}_{T} . \underbrace{\left(d - \frac{a}{2}\right)}_{z} = 98{,}7 . 32 . \left(1477{,}8 - \frac{68{,}05}{2}\right) . 1831 . 10^{-6}$$

$M_r = 8351$ kNm > 7567,03 kNm ok

5.5.4.2 Strain-compatibility analysis

STRAIN - COMPATIBILITY ANALYSIS AT MIDSPAN

→ Flexural demand at midspan, $M_f = 7567{,}03$ kNm (from chapter 4)



$E_{ps} = 200000$ MPa

$E_c = 5000 . \sqrt{40} = 31623$ MPa

$A_{ps} = 98{,}7 . 32 = 3158{,}4$ mm²

$A_g = 509031{,}24$ mm²

A_{gt} (Transformed area of the prestressing girder) = 528487,6385 mm²

y total = 1371,6 mm²

y total composite = 1571,6 mm²

d = 1277,8 mm

d composite = 1477,8 mm

$e_c = 534{,}4895$ mm

$y_t = 743{,}3605$ mm

$y_b = 628{,}2395$ mm

$I_g = 1{,}0853 . 10^{11}$ mm⁴

$I_{gt} = 1{,}1391 . 10^{11}$ mm⁴

$P_{pe} = 3648$ kN

$\varepsilon_{ctop} = \varepsilon_{cu} = 0{,}003$

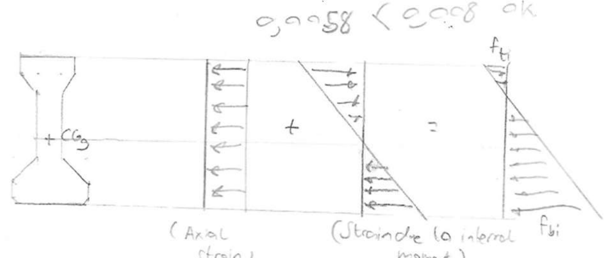
$\varepsilon_o = 0{,}002$

$f'_c = 40$ MPa

Initial prestressing strain in the steel due to P_{pe}.

$$\varepsilon_{si} = \frac{P_{pe}}{A_{ps} . E_{ps}} = \frac{3648 . 10^3}{3158{,}4 . 200000} = 0{,}0058$$

0,0058 < 0,008 ok



$$-\frac{P_{pe}}{A_{gt} . E_{ps}} = \frac{-3648 . 10^3}{528487{,}6385 . 200000} = -3{,}4513 . 10^{-5}$$

$$\varepsilon_{ti_p} = -\frac{P_{pe}}{A_{gt} \cdot E_{ps}} + \frac{P_{pe} \cdot e_c \cdot y_t}{I_{gt} \cdot E_{ps}} = -3{,}4513 \cdot 10^{-5} + \frac{3648 \cdot 10^3 \cdot 534{,}4895 \cdot 743{,}3605}{1{,}1391 \cdot 10^{11} \cdot 200000} = 2{,}9107 \cdot 10^{-5}$$

$$\varepsilon_{bi_p} = -\frac{P_{pe}}{A_{gt} \cdot E_{ps}} - \frac{P_{pe} \cdot e_c \cdot y_b}{I_{gt} \cdot E_{ps}} = -3{,}4513 \cdot 10^{-5} - \frac{3648 \cdot 10^3 \cdot 534{,}4895 \cdot 628{,}2395}{1{,}1391 \cdot 10^{11} \cdot 200000} = -8{,}8281 \cdot 10^{-5}$$

$$\varepsilon_{cip} = \varepsilon_{tip} + (\varepsilon_{bip} - \varepsilon_{tip}) \cdot d \cdot \frac{1}{y_{total}} = 2{,}9107 \cdot 10^{-5} + (-8{,}8281 \cdot 10^{-5} - 2{,}9107 \cdot 10^{-5}) \cdot 1277{,}8 \cdot \frac{1}{1371{,}6} = -8{,}0257 \cdot 10^{-5}$$

Assuming unshored construction

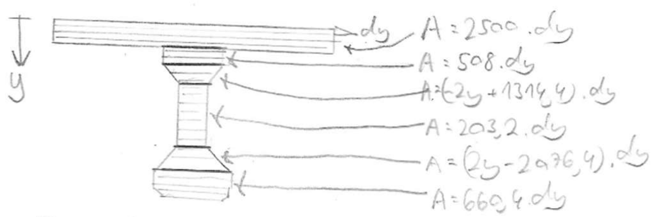
$$\varepsilon_{ti_{DL}} = -\frac{(M_G + M_S) \cdot y_t}{I_{gt} \cdot E_{ps}} = -\frac{(1053{,}82 + 1014) \cdot 10^6 \cdot 743{,}3605}{1{,}1391 \cdot 10^{11} \cdot 200000} = -6{,}7472 \cdot 10^{-5}$$

$$\varepsilon_{bi_{DL}} = \frac{(M_G + M_S) \cdot y_b}{I_{gt} \cdot E_{ps}} = \frac{(1053{,}82 + 1014) \cdot 10^6 \cdot 628{,}2395}{1{,}1391 \cdot 10^{11} \cdot 200000} = 5{,}7023 \cdot 10^{-5}$$

$$\varepsilon_{ci_{DL}} = \frac{\overset{e_c}{d - y_t}}{y_b} \cdot \varepsilon_{bi_{DL}} = \frac{534{,}4895}{628{,}2395} \cdot 5{,}7023 \cdot 10^{-5} = 4{,}8513 \cdot 10^{-5}$$

$$\varepsilon_{ci} = |\varepsilon_{cip}| + \varepsilon_{ci_{DL}} = 1{,}2877 \cdot 10^{-4}$$

* Area is divided into rectangles with height 0,01 m or 1cm ($dy = 0{,}01$ for numerical integration)



$$\varepsilon_s = \varepsilon_{si} + \varepsilon_{ci} + \varepsilon_{ctop} \cdot \frac{d_{composite} - c}{c}$$

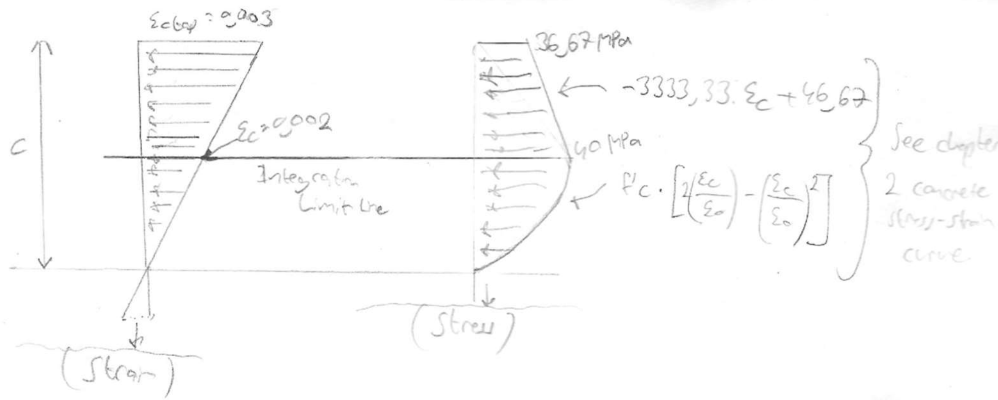
$$\varepsilon_s = 0{,}0058 + 1{,}2877 \cdot 10^{-4} + 0{,}003 \cdot \frac{1477{,}8 - c}{c}$$

$f_{ps} = E_{ps} \cdot \varepsilon_s = 200000 \cdot \varepsilon_s$ for $\varepsilon_s \leq 0{,}008$

$f_{ps} = 1848 - 0{,}517 \cdot \frac{1}{\varepsilon_s - 0{,}005915}$ for $\varepsilon_s > 0{,}008$

$f_{ps} = \alpha$ for $f_{ps} > 0{,}99 \cdot f_{pu}$

*See chapter 2 Prestressing Steel Stress-Strain curve



$$\text{Integration Limit} = -(0{,}002 - \varepsilon_{ctop})\cdot\frac{c}{\varepsilon_{ctop}} = -(0{,}002 - 0{,}003)\cdot\frac{c}{0{,}003}$$

$$= 0{,}333c \text{ from top}$$

$$\varepsilon_c = \varepsilon_{ctop} - \varepsilon_{ctop}\cdot\frac{y}{c}$$

$T = f_{ps}\cdot A_{ps}$

$$C = \int_{0}^{0{,}333c}(-3333{,}33\cdot\varepsilon_c + 46{,}67)\cdot A + \int_{0{,}333c}^{c} 40\cdot\left[2\cdot\left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right]\cdot A$$

*Solving $T = C$ numerically using MATLAB, $c = 76{,}03$ mm (code available in the appendix section)

Let $c = 76{,}03$ mm

$$\varepsilon_s = 0{,}0058 + 1{,}2877\cdot10^{-4} + 0{,}003\cdot\frac{1477{,}8 - 76{,}03}{76{,}03} = 0{,}0612$$

$$\varepsilon_s > 0{,}008$$

$$\therefore f_{ps} = 1848 - 0{,}517\cdot\frac{1}{0{,}0612 - 0{,}005915} = 1838{,}7 \text{ MPa}$$

$$1838{,}7 \text{ MPa} < 0{,}99\cdot f_{pu} \quad \text{OK}$$
$(1841{,}4)$

$$\text{Integration Limit} = 0{,}333c = 25{,}34 \text{ mm}$$

$$T \cong C = f_{ps}\cdot A_{ps} = 1838{,}7\cdot3158{,}4 = 5{,}807\cdot10^6 \text{ N}$$

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$$\bar{y} = \frac{\int_0^{0{,}333c}(-3333{,}33 \cdot \varepsilon_c + 46{,}67) \cdot A \cdot y + \int_{0{,}333c}^{c} 40 \cdot \left[2 \cdot \left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right] \cdot A \cdot y}{T}$$

$$\varepsilon_c = 0{,}003 - 0{,}003 \cdot \frac{y}{76{,}03}$$

$$\bar{y} = \frac{\int_0^{25{,}34}\left(-3333{,}33 \cdot \left(0{,}003 - 0{,}003 \frac{y}{76{,}03}\right) + 46{,}67\right) \cdot 2500 \cdot y \cdot dy + \int_{25{,}34}^{76{,}02} 40 \cdot \left[2 \cdot \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}03}}{0{,}002}\right) - \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}03}}{0{,}002}\right)^2\right] \cdot 2500 \cdot y \cdot dy}{(5{,}807 \cdot 10^6)}$$

$$\bar{y} = \frac{1{,}8109 \cdot 10^8}{5{,}807 \cdot 10^6} = 31{,}1824 \text{ mm}$$

$$z = d \text{ composite} - \bar{y} = 1477{,}9 - 31{,}1824 = 1446{,}7 \text{ mm}$$

$$M_r = z \cdot T \cdot 10^{-6} = 1446{,}7 \cdot 5{,}807 = 8401 \text{ kNm} \pm 0{,}5 \text{ kNm.}$$

8401 kNm > 7567,03 kNm OK

K & B

5.4.5 Reserve capacity

Moment resistance of the section at ultimate must be at least 1.2 times more than the cracking moment of the section. The reserve capacity check requirement can be waived if it is proven that the section has 1.33 times more moment resistance than the factored demand at ultimate.

The maximum moment experienced at ultimate is at 12.5 m and 13.5 m from left support. It is equal to 7575.5 kNm.

The moment resistance obtained by strain – compatibility is 8401 kNm.

$$1.33 \times 7575.5 = \mathbf{10100}\ kNm > \mathbf{8401}\ kNm$$

Therefore reserve capacity must be checked.

At service and at midspan:

$$f_b = -\frac{P_{pe}}{A_g} - \frac{P_{pe} \times e_c \times y_b}{I_g} + \frac{\left(M_g + M_s\right) \times y_b}{I_g} + \frac{\left(M_{SDL} + M_{LL}\right) \times y_{bc}}{I_c}$$

$$f_b = -\frac{3648 \times 10^3}{509031} - \frac{3648 \times 10^3 \times 534.4895 \times 628.2395}{1.0853 \times 10^{11}} + \frac{(1053.82 + 1014) \times 10^6 \times 628.2395}{1.0853 \times 10^{11}}$$

$$+ \frac{(322.68 + 1598.02) \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = \mathbf{0.4551}\ MPa\ T$$

At cracking, the bottom stress $= 0.6 \times \sqrt{f'_c} = 0.6 \times \sqrt{40} = \mathbf{3.7947}\ MPa\ T$

The additional moment must create a bottom stress of $3.7947 - 0.4551 = \mathbf{3.3397}\ MPa\ T$

$$\frac{M_{add} \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = 3.3397,\ solving\ for\ M_{add},\ M_{add} = \mathbf{924.5014}\ kNm$$

Therefore, $M_{cr} = 924.5014 + 1053.82 + 1014 + 322.68 + 1598.02 = \mathbf{4913}\ kNm$

$$1.2 \times 4913\ kNm = 5895.6\ kNm < 8401\ kNm\ OK$$

5.5.6 Deflection limits check

During service and initial stage, the beam is under linear stresses with respect to the strains experienced. Therefore, most of the equations given here are for first order linear-elastic analysis.

The deflections experienced in ultimate stage is not the main concern of the design since the bridge is expected to never reach ultimate loading unless some extraordinary, extreme event happens. Nevertheless, the deflection is checked using stain-compatibility together with finite-element analysis. The ultimate deflections will not be presented in this report.

Deflections due to shear deformations are ignored in this report.

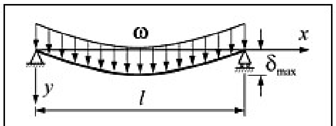
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED BRIDGE DEFLECTION AND MAXIMUM DEFLECTION	
	$y = \frac{\omega x}{24EI}\left(l^3 - 2lx^2 + x^3\right)$	$\delta_{max} = \frac{5\omega l^4}{384EI}$

Figure 5.5.6.1 – Deflection equations for UDL on a simply supported beam

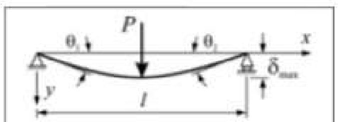
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED DEFLECTION AND MAXIMUM DEFLECTION	
	$y = \frac{Px}{12EI}\left(\frac{3l^2}{4} - x^2\right)$ for $0 < x < \frac{l}{2}$	$\delta_{max} = \frac{Pl^3}{48EI}$

Figure 5.4.6.2 – Deflection equations for point load at midspan on a simply supported beam

Immediate deflection due to live load:

Immediate deflection due to live load can be calculated from the deflection occuring when applying the truck load at the midspan of the interior girder as a single point load. This simple method will give conservative results. If the deflection obtained is within the critical range, then the distribution and impact factors can be taken into account.

$$\Delta_L = \frac{P \times L^3}{48 \times E_c \times I_c} = \frac{325000 \times 26000^3}{48 \times 31623 \times 2.8960 \times 10^{11}} = 13\ mm\ downwards$$

Erection deflections:

Elastic Deflection due to girder self – weight:

$$\Delta_{DL} = \frac{5 \times w_G \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 12.47 \times 26000^4}{384 \times 31623 \times 1.0853 \times 10^{11}} = 22\ mm\ downwards$$

Elastic Deflection due to deck:

$$\Delta_{SL} = \frac{5 \times w_S \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 12 \times 26000^4}{384 \times 31623 \times 1.0853 \times 10^{11}} = 21\ mm\ downwards$$

Elastic Deflection due to asphalt and waterproofing:

$$\Delta_{PL} = \frac{5 \times w_{SDL} \times L^4}{384 \times E_c \times I_g} = \frac{5 \times 3.82 \times 26000^4}{384 \times 31623 \times 1.0853 \times 10^{11}} = 7\ mm\ downwards$$

Upward Elastic Deflection due to Camber:

There are many different methods to calculate Camber. Camber calculations can be done using the " Hyperbolic Functions Method" proposed by Sinno Rauf and Howard L Furr (1970) or using the PCI s equations. However, in this report, camber is calulated using the approximate equations proposed by Collins and Mitchell.

$$\Delta_c = \left(\frac{e_c}{8} - \beta^2 \times \frac{(e_c - e_e)}{6}\right) \times P_{pi} \times \frac{L^2}{(E_c \times I_g)}$$

where:

β = *Ratio of harping length at one end with respect to total length*

e_e = *Average eccentricity at girder ends*

$$\beta = \frac{8.5}{26} = 0.327$$

Between 0 and 8.5 m from left support, the center of gravity of steel is given by this equation:

$$Distance\ from\ bottom\ to\ CGS\ [mm] = -\frac{313.425 - 93.75}{8500} \times Dist\ from\ left\ supp. + 313.425$$

Therefore CGS @ 0 m = 313.425 mm from bottom

$e_c = 628.2395 - 313.425 = 314.8145\ mm$

$$\Delta_c = \left(\frac{534.4895}{8} - 0.327^2 \times \frac{(534.4895 - 314.815)}{6}\right) \times 4074.3 \times 10^3 \times \frac{26000^2}{(31623 \times 1.0853 \times 10^{11})}$$

$\Delta_c = 50\ mm\ upwards$

$Total\ deflection\ at\ erection = 1.85 \times \Delta_{DL} + 1.8 \times \Delta_c = 1.85 \times 22 + 1.8 \times -49 = 49\ mm\ upwards$

$Total\ long\ term\ deflection = 2.4 \times \Delta_{DL} + 2.2 \times \Delta_c + 2.3 \times \Delta_{SL} + 3 \times \Delta_{PL}$

$Total\ long\ term\ deflection = 2.4 \times 22 + 2.2 \times -50 + 2.3 \times 21 + 3 \times 7 = 12.1\ mm\ downwards$

$All\ deflections\ are\ within\ limit\ of\ \frac{1}{800}\ so\ this\ design\ is\ safe$

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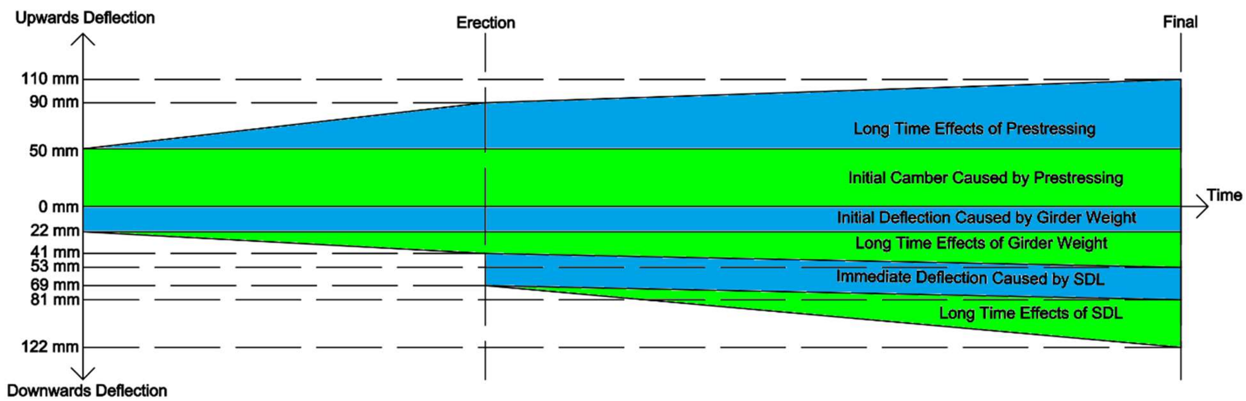

Figure 5.5.6.3 – Visual Representation of the Deflections Experienced

5.5.7 Design for shear

For shear design, 15 M Canadian reinforcement bars with 16 mm diameter will be used. Each bar will therefore have an area of 200 mm^2, and 400 mm^2 when bent to be double legged. Ultimate shear values from chapter 4 must be used for the shear design.

Determination of the stress limit in concrete to be used in concrete shear resistance calculation:

The shear stress limit in concrete can be calculated as:

$$v_c = 0.33 \times \sqrt{f'_c} = 0.33 \times \sqrt{40} = \mathbf{2.1\ MPa}$$

Determination of the shear resistance of the concrete:

Shear resistance of concrete can be calculated as:

$$V_c = v_c \times b_v \times h \text{ where } b_v = \text{Smallest width of the section } (203.2\ mm \text{ for AASHTO Type 4 Girders}).$$

$$V_c = 2.1 \times 203.2 \times 1371.6 \times 10^{-3} = \mathbf{585.46\ kN}$$

Determination of shear resistance required to be resisted by the steel $V_{s,required}$:

$$V_{s,required} = V_f - V_c \geq 0$$

At midspan:

$$V_{s,required} = 292.61 - 585.46 = \mathbf{-292.85\ kN < 0 \text{ therefore } 0\ kN}$$

Determination of required spacing based on $V_{s,required}$:

$$s_{required} = \frac{d_v \times f_y \times A_v}{V_s} \text{ where } V_s > 0 \text{ if } V_s = 0 \text{ then } 0$$

At midspan:

$$V_s = 0 \text{ so } s_{required} = \mathbf{0\ mm}$$

Determination of maximum spacing s_{max}:

$$s_{max} = 0.5 \times d_v \text{ applicable where } V_s > 0$$

At midspan:

$$s_{max} = 0.5 \times 1150.07 = \mathbf{575\ mm \text{ but not applicable}} \text{ since } V_s = 0\ kN$$

Concrete may experience micro cracking over time due to several factors. Providing at least some reinforcement with large spacing prevents the propagation of the cracks. However, since this code doesn't require a maximum, no reinforcement will be provided close to midspan.

Determination of design spacing for shear reinforcement:

At midspan:

Since at midspan $V_s = 0\,kN$*, shear reinforcement not required.*

Determination of the design shear resistance provided by the steel:

Based on design spacing, shear resistance provided can be calculated as:

$$V_s = \frac{d_v \times f_y \times A_v}{s_{design}} \text{ where } s_{design} > 0\,mm,\ 0 \text{ if } s_{design} = 0$$

At midspan:

$s_{design} = 0\,mm$ so $V_s = 0\,kN$

Checking of resistance provided against resistance available from concrete:

Check against aavailable concete resistance:

$$V_f - V_s \leq V_c$$

At midspan:

$V_f < V_c$ *so the design is OK at midspan.*

Table 5.5.7.1 – Final Shear Reinforcement Layout

From 50 mm to 2050 mm	10 spacing @ 200 mm c/c	15 M Double-Legged	Type 2
From 2050 mm to 4150 mm	7 spacing @ 300 mm c/c	15 M Double-Legged	Type 2
From 4150 mm to 9400 mm	10 spacing @ 525 mm c/c	15 M Double-Legged	Type 2
From 9400 mm to 16600 mm	-	-	-
From 16600 mm to 21850 mm	10 spacing @ 525 mm c/c	15 M Double-Legged	Type 2
From 21850 mm to 23950 mm	7 spacing @ 300 mm c/c	15 M Double-Legged	Type 2
From 23950 mm to 25950 mm	10 spacing @ 200 mm c/c	15 M Double-Legged	Type 2

Table 5.5.7.2 – Shear Design Calculations

x [m]	V_f [kN]	CGS [mm]	d [mm]	d_v [mm]	V_c [kN]	V_s [kN]	$s_{required}$ [mm]	s_{max} [mm]	s_{design} [mm]	Governed By	$V_{s,design}$ [kN]	$V_{c,needed}$	$V_{c,provided}$ OK?
0	1219.20	313.43	1058.18	987.55	585.46	633.74	249.33	493.78	200	$s_{required}$	790.04	429.16	OK
0.5	1183.56	300.50	1071.10	987.55	585.46	598.10	264.18	493.78	200	$s_{required}$	790.04	393.52	OK
1	1147.92	287.58	1084.02	987.55	585.46	562.46	280.92	493.78	200	$s_{required}$	790.04	357.88	OK
1.5	1112.29	274.66	1096.94	987.55	585.46	526.82	299.93	493.78	200	$s_{required}$	790.04	322.24	OK
2	1076.65	261.74	1109.86	998.88	585.46	491.18	325.38	499.44	200	$s_{required}$	799.10	277.55	OK
2.5	1041.01	248.81	1122.79	1010.51	585.46	455.55	354.92	505.25	300	$s_{required}$	538.94	502.07	OK
3	1005.37	235.89	1135.71	1022.14	585.46	419.91	389.47	511.07	300	$s_{required}$	545.14	460.23	OK
3.5	969.73	222.97	1148.63	1033.77	585.46	384.27	430.43	516.88	300	$s_{required}$	551.34	418.39	OK
4	934.10	210.05	1161.55	1045.40	585.46	348.63	479.77	522.70	300	$s_{required}$	557.54	376.55	OK
4.5	898.46	197.13	1174.47	1057.03	585.46	312.99	540.34	528.51	525	s_{max}	322.14	576.32	OK
5	862.82	184.20	1187.40	1068.66	585.46	277.36	616.48	534.33	525	s_{max}	325.69	537.13	OK
5.5	827.18	171.28	1200.32	1080.29	585.46	241.72	715.07	540.14	525	s_{max}	329.23	497.95	OK
6	791.54	158.36	1213.24	1091.92	585.46	206.08	847.77	545.96	525	s_{max}	332.77	458.77	OK
6.5	755.90	145.44	1226.16	1103.55	585.46	170.44	1035.95	551.77	525	s_{max}	336.32	419.59	OK
7	720.27	132.52	1239.08	1115.18	585.46	134.80	1323.63	557.59	525	s_{max}	339.86	380.40	OK
7.5	684.63	119.59	1252.01	1126.81	585.46	99.16	1818.08	563.40	525	s_{max}	343.41	341.22	OK
8	648.99	106.67	1264.93	1138.44	585.46	63.53	2867.32	569.22	525	s_{max}	346.95	302.04	OK
8.5	613.35	93.75	1277.85	1150.07	585.46	27.89	6598.24	575.03	525	s_{max}	350.50	262.86	OK
9	577.71	93.75	1277.85	1150.07	585.46	0	0	Not Required	525	Constructibilility	350.50	227.22	OK
9.5	542.07	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	542.07	OK
10	506.44	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	506.44	OK
10.5	470.80	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	470.80	OK
11	435.16	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	435.16	OK
11.5	399.52	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	399.52	OK
12	363.88	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	363.88	OK
12.5	328.25	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	328.25	OK
13	292.61	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	292.61	OK
13.5	328.25	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	328.25	OK
14	363.88	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	363.88	OK
14.5	399.52	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	399.52	OK
15	435.16	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	435.16	OK
15.5	470.80	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	470.80	OK
16	506.44	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	506.44	OK
16.5	542.07	93.75	1277.85	1150.07	585.46	0	0	Not Required	0	Not Available	0	542.07	OK
17	577.71	93.75	1277.85	1150.07	585.46	0	0	Not Required	525	Constructibilility	350.50	227.22	OK
17.5	613.35	93.75	1277.85	1150.07	585.46	27.89	6598.24	575.03	525	s_{max}	350.50	262.86	OK
18	648.99	106.67	1264.93	1138.44	585.46	63.53	2867.32	569.22	525	s_{max}	346.95	302.04	OK
18.5	684.63	119.59	1252.01	1126.81	585.46	99.16	1818.08	563.40	525	s_{max}	343.41	341.22	OK
19	720.27	132.52	1239.08	1115.18	585.46	134.80	1323.63	557.59	525	s_{max}	339.86	380.40	OK
19.5	755.90	145.44	1226.16	1103.55	585.46	170.44	1035.95	551.77	525	s_{max}	336.32	419.59	OK
20	791.54	158.36	1213.24	1091.92	585.46	206.08	847.77	545.96	525	s_{max}	332.77	458.77	OK
20.5	827.18	171.28	1200.32	1080.29	585.46	241.72	715.07	540.14	525	s_{max}	329.23	497.95	OK
21	862.82	184.20	1187.40	1068.66	585.46	277.36	616.48	534.33	525	s_{max}	325.69	537.13	OK
21.5	898.46	197.13	1174.47	1057.03	585.46	312.99	540.34	528.51	525	s_{max}	322.14	576.32	OK
22	934.10	210.05	1161.55	1045.40	585.46	348.63	479.77	522.70	300	$s_{required}$	557.54	376.55	OK
22.5	969.73	222.97	1148.63	1033.77	585.46	384.27	430.43	516.88	300	$s_{required}$	551.34	418.39	OK
23	1005.37	235.89	1135.71	1022.14	585.46	419.91	389.47	511.07	300	$s_{required}$	545.14	460.23	OK
23.5	1041.01	248.81	1122.79	1010.51	585.46	455.55	354.92	505.25	300	$s_{required}$	538.94	502.07	OK
24	1076.65	261.74	1109.86	998.88	585.46	491.18	325.38	499.44	200	$s_{required}$	799.10	277.55	OK
24.5	1112.29	274.66	1096.94	987.55	585.46	526.82	299.93	493.78	200	$s_{required}$	790.04	322.24	OK
25	1147.92	287.58	1084.02	987.55	585.46	562.46	280.92	493.78	200	$s_{required}$	790.04	357.88	OK
25.5	1183.56	300.50	1071.10	987.55	585.46	598.10	264.18	493.78	200	$s_{required}$	790.04	393.52	OK
26	1219.20	313.43	1058.18	987.55	585.46	633.74	249.33	493.78	200	$s_{required}$	790.04	429.16	OK

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5.5.8 Design for shrinkage and temperature variation

$425\ mm^2 / m$ *is required in the direction parallel to the span*

$500\ mm^2 / m$ *is required in the direction transverse to the span*

Provide:

$4-\ 15\,M$ *bars @ 300 in the direction parallel to span.* $A_s = 800\ mm^2\ (667\ mm^2/m)$

$3-\ 10\,M$ *bars @ 260 in the direction transverse to span* $A_s = 300\ mm^2\ (577\ mm^2/m)$

5.6 Design Summary for Three Codes

Design Code	Flexural Design	Shear Design	Shrinkage & Temperature Reinforcement	Long Term Deflections
CSA S6-17 rev. 17	32 Low-Relaxation 7-wire strands	26 Type 1	Vertical: 15 M @ 300 mm spacing	1 mm
	12.7 mm diameter, 98.7 mm2 Area, A_{total} = 3158.4 mm2	55 Type 2	Horizontal: 10 M @ 130 mm spacing	Upwards
AASHTO LRFD 2014-17	32 Low-Relaxation 7-wire strands	26 Type 1	Vertical: 15 M @ 300 mm spacing	2 mm
	12.7 mm diameter, 98.7 mm2 Area, A_{total} = 3158.4 mm2	51 Type 2	Horizontal: 10 M @ 260 mm spacing	Downwards
CSA S6-66	32 Low-Relaxation 7-wire strands	0 Type 1	Vertical: 15 M @ 300 mm spacing	12 mm
	12.7 mm diameter, 98.7 mm2 Area, A_{total} = 3158.4 mm2	56 Type 2	Horizontal: 10 M @ 260 mm spacing	Downwards

Type 1 and Type 2 Stirrup details can be found at the very and of this report with design drawings

5.7 References

- Michael P. Collins and Denis Mitchell, Prestressed Concrete Structures, 1991, ISBN: 9780136916352

-Edward G. Navy, Prestressed Concrete, A fundamental approach, Fifth Edition, 2009

-Charles W. Dolan, H. R. (Trey) Hamilton - Prestressed Concrete_ Building, Design, and Construction, 2019

- CSA S6-14 Highway Bridge Design Code: Canadian Standards Association, 2014, Revision 2017

- AASHTO LRFD Bridge Design Specifications: American Association of State Highway and Transportation Officials, 2014, 8th Edition, SI - Revision 2017

- CSA S6-66 Design of Highway Bridges: Canadian Standards Association, 1966

-Texas Department of Transportation and North Carolina Department of transportation: Design Examples on AASHTO Girder Design

5.8 Appendix

MATLAB Codes for CSA S6-14 rev. 17:

```
close all;
clear all;
clc;

ts = 200;
fc_deck = 35;
tw = 65;
fci_girder = 35;
fc_girder = 40;
L = 26;
A_1strand = 98.7;
fpu = 1860;
fpy = 0.9 * fpu;
Ep = 200000;
fy = 400;
epsilon_s_yield = 0.002;
Es = 200000;
fu = 550;
epsilon_s_ultimate = 0.1;
Ec_deck = (3000*sqrt(fc_deck)+6900)*(2450/2300)^1.5;
Ec_girder = (3000*sqrt(fc_girder)+6900)*(2500/2300)^1.5;
Eci_girder = (3000*sqrt(fci_girder)+6900)*(2500/2300)^1.5;
b_eff = 2500;
Ig = 1.0853*10^11;
Ag = 509031.24;
yt = 743.3605;
yb = 628.2395;
sb = Ig/yb;
st = Ig/yt;
hb = 1371.6;
Ic = 2.8960*10^11;
Ac = 1009031.24;
ytc = 525.4544;
ybc = 1046.1456;
sbc = Ic/ybc;
stc = Ic/ytc;

fb = ((1053.82192461 + 1014)*10^6)/sb +
((322.684375+0.9*1897.23737433914)*10^6)/sbc;

Fb = 0.4 * sqrt(fc_girder);

fpb = fb - Fb;
```

```
ybs = 100;

ec = yb - ybs;

Ppe = 3339.87879455;

fpi = 0.74 * fpu;

AssumedFinalLosses = 0.2 * fpi;

force_per_strand_after_losses = A_1strand *(fpi - AssumedFinalLosses)/10^3;

Number_of_strands_required = Ppe / force_per_strand_after_losses;

ec_2 = yb-(12*(50+100)+8*150)/32;

Ppe_2 = 32 * force_per_strand_after_losses;

fb_2 = (Ppe_2 * 10^3)/Ag + (Ppe_2 * ec_2 * 10^3)/sb;

ec_3 = yb-(12*(50+100)+6*150)/30;

Ppe_3 = 30 * force_per_strand_after_losses;

fb_3 = (Ppe_3 * 10^3)/Ag + (Ppe_3 * ec_3 * 10^3)/sb;

Pi = 32 * 98.7 * 0.92 * fpi * 10^-3;

n = Ep / Eci_girder;

(n-1)*98.7*8/2/(sqrt(98.7/pi)*2);
sqrt(98.7/pi);

Agt = 528487.6385;

Igt = 1.1391*10^11;

fcir = (Pi * 10^3) / Agt + ((Pi * 10^3) * (ec_2)^2) / Igt -
(1053.82192461*10^6*ec_2) / Igt;

Delta_ES = n*fcir;

Delta_SR = 117 - 1.05 * 60;

n_2 = Ep / Ec_girder;
```

```
Ict = 3.0694*10^11;

Act = 1028487.6385;

fcds = (1014*10^6*ec_2)/Igt+ (322.684375*10^6*(ybc-(yb-ec_2)))/Ict;

Delta_CR = (1.37-0.77*(0.01*60)^2)*2*n_2*(fcir-fcds);

Delta_R2 = (fpi/fpu-0.55)*(0.34-(Delta_CR+Delta_SR)/(1.25*fpu))*fpu/3;

Total_loss = (Delta_R2 + Delta_CR + Delta_ES + Delta_SR);

Total_initial_loss = (Delta_ES + 0.5*Delta_R2);

Total_loss_percent = (Delta_R2 + Delta_CR + Delta_ES + Delta_SR)/fpi*100;

Initial_loss_percent = (Delta_ES + 0.5*Delta_R2)/fpi*100;

fpe = fpi - Total_loss;

Ppe_4 = 32*fpe*98.7*10^-3;

fb_4 = (Ppe_4 * 10^3)/Ag + (Ppe_4 * ec_2 * 10^3)/sb;

Pi = 32*98.7*(fpi-Total_initial_loss)*10^-3;


fti = Pi*10^3/Ag - (Pi*10^3*ec_2)/st + 1053.82192461*10^6/st;

fbi = Pi*10^3/Ag + (Pi*10^3*ec_2)/sb - 1053.82192461*10^6/sb;

fts = Ppe_4*10^3/Ag - (Ppe_4*10^3*ec_2)/st + (1053.82192461+1014)*10^6/st +
(322.684375+1684.53467491788)*10^6/(Ic/(ytc-200));

fbs = Ppe_4*10^3/Ag + (Ppe_4*10^3*ec_2)/sb - (1053.82192461+1014)*10^6/sb -
(322.684375+1684.53467491788)*10^6/sbc;


%Rectangular Section Assumption:
alpha1 = 0.85 - 0.0015*40;
beta1 = 0.97 - 0.0025*40;

fbs_2 = -(Ppe_4*10^3)/Ag-(Ppe_4*10^3*ec_2*yb)/Ig+((1053.82+1014)*10^6*yb)/Ig
+ (322.68+0.9*1871.705194)*10^6*ybc/Ic;

additional_bottom_stress = 0.4 * sqrt(40) - fbs_2;
```

```
M_additional = additional_bottom_stress * Ic / (ybc * 10^6);

M_cr = M_additional + (1053.82+1014) + (322.68+0.9*1871.705194);

1.2*M_cr;

%Deflections:

DeltaL = 625000*26000^3/(48*Ec_girder*Ic);

DeltaDL = 5*0.50903124*24.5*26000^4/(384*Ec_girder*Ig);

DeltaSL = 5*12*26000^4/(384*Ec_girder*Ig);

DeltaPL = 5*0.065*2.5*23.5*26000^4/(384*Ec_girder*Ig);
```

```
clear all;
close all;
clc;

%Imput Parameters

Eps = 200000;
Ec = (3000*sqrt(40)+6900)*(2500/2300)^1.5;

Aps = 98.7*32;
Ag = 509031.24;
Ag_T = 528487.6385;


y_total = 1371.6;
y_total_composite = 1371.6 + 200;

d = y_total - 93.75;
d_composite = y_total_composite - 93.75;

yt = 743.3605;
yb = y_total - yt;

ec = 534.4895;

Ig = 1.0853*10^11;
Ig_T = 1.1391*10^11;

Ppe = 3464.485816857033 * 10^3;
```

```
Epsilon_ctop = 0.0035;
Epsilon_0 = 0.002;
fc_prime = 40;

%Initial Stage "Epsilon_si"

Epsilon_si = Ppe / (Aps * Eps);

%Initial Stage "Epsilon_ci"

ft = -(Ppe / Ag_T) + ( Ppe * ec * yt / Ig_T);
fb = -(Ppe / Ag_T) - ( Ppe * ec * yb / Ig_T);
Epsilon_t = ft / Eps;
Epsilon_b = fb / Eps;
Epsilon_ci_p = Epsilon_t + (Epsilon_b - Epsilon_t) * d / y_total;

Mg = 1053.82192461 * 10^6;
Ms = 1014 * 10^6;

Epsilon_t_DL = -(Mg + Ms) * yt / (Ig_T * Eps);
Epsilon_b_DL = (Mg + Ms) * yb / (Ig_T * Eps);
Epsilon_ci_DL = (ec / yb) * Epsilon_b_DL;

Epsilon_ci = abs(Epsilon_ci_p) + Epsilon_ci_DL;

%Calculation of area segments: Rectangle width 0.01 m or 1 cm%

Area = zeros(y_total_composite/0.01,1);

i=1;
for y=0.01:0.01:y_total_composite
if (y <= 200)
    Area(i)=2500*0.01;

elseif (y <= 403.2)
    Area(i)=508*0.01;

elseif (y <= 555.6)
    Area(i)=(-2*y+1314.4508)*0.01;

elseif (y <= 1139.8)
    Area(i)=203.2*0.01;

elseif (y <= 1368.4)
    Area(i)=(2*y-2076.4)*0.01;
```

```
else
    Area(i)=660.4*0.01;

end

i=i+1;
end

i=1;
j=1;

%Calculation part

for c = 0.01:0.01:y_total_composite
% Epsilon_s = Strain at prestressing steel at the level of the CGS.
Epsilon_s = Epsilon_si + Epsilon_ci + Epsilon_ctop * (d_composite-c)/c;

if (Epsilon_s <= 0.008)
fps = Eps * Epsilon_s;
elseif (Epsilon_s > 0.008)
fps = 1848 - 0.517 / (Epsilon_s - 0.005915);
end

if (fps > 1843.38)
fps = -10^15;
end

integration_limit = -(0.002-Epsilon_ctop)*c/Epsilon_ctop;

C = 0;
i = 1;

T = fps * Aps;

for y=0.01:0.01:c

    Epsilon_c = Epsilon_ctop - Epsilon_ctop*y/c;

    if (y <= integration_limit)
    C = C + (-3333.3333333333*Epsilon_c+46.6666666667)*Area(i);
    else
    C = C + fc_prime*(2*(Epsilon_c/Epsilon_0)-
(Epsilon_c/Epsilon_0)^2)*Area(i);
    end

    i = i+1;
end
```

```
if (abs(C-T) < 500)
    break;
end

j = j+1;

end

Sum = 0;
i=1;
for y=0.01:0.01:c

    Epsilon_c = Epsilon_ctop - Epsilon_ctop*y/c;

    if (y <= integration_limit)
    Cdy = (-3333.3333333333*Epsilon_c+46.6666666667)*Area(i);
    else
    Cdy = fc_prime*(2*(Epsilon_c/Epsilon_0)-(Epsilon_c/Epsilon_0)^2)*Area(i);
    end

    Sum = Sum + Cdy * y;
    i = i+1;
end

y_bar = Sum / C;

z = d_composite - y_bar;

M = z * T;

M_kNm = M * 10^-6;

fprintf('Moment Resistance of the given section: %4.0f +- 0.5 kNm\n\n',
M_kNm);
```

MATLAB Codes for AASHTO LRFD 2014-17:

```
close all;
clear all;
clc;

ts = 200;
fc_deck = 35;
tw = 65;
fci_girder = 35;
fc_girder = 40;
L = 26;
A_1strand = 98.7;
fpu = 1860;
fpy = 0.9 * fpu;
Ep = 200000;
fy = 400;
epsilon_s_yield = 0.002;
Es = 200000;
fu = 550;
epsilon_s_ultimate = 0.1;
Ec_deck = 0.043 * 2450^1.5 * sqrt(fc_deck);
Ec_girder = 0.043 * 2500^1.5 * sqrt(fc_girder);
Eci_girder = 0.043 * 2500^1.5 * sqrt(fci_girder);
b_eff = 2500;
Ig = 1.0853*10^11;
Ag = 509031.24;
yt = 743.3605;
yb = 628.2395;
sb = Ig/yb;
st = Ig/yt;
hb = 1371.6;
Ic = 2.8960*10^11;
Ac = 1009031.24;
ytc = 525.4544;
ybc = 1046.1456;
sbc = Ic/ybc;
stc = Ic/ytc;

MG = 985.155290047748;
MS = 962.863612220337;
MSDL = 301.981010999023;
MLL = 2165.40842169309;
MLL_midspan = 2160.66268043619;

fb = ((MG + MS )*10^6)/sb + ((MSDL + MLL)*10^6)/sbc;

Fb = 0.5 * sqrt(fc_girder);
```

```
fpb = fb - Fb;

ybs = 100;

ec = yb - ybs;

Ppe = 3390.326529319760;

fpi = 0.75 * fpu;

AssumedFinalLosses = 0.2 * fpi;

force_per_strand_after_losses = A_1strand *(fpi - AssumedFinalLosses)/10^3;

Number_of_strands_required = Ppe / force_per_strand_after_losses;

ec_2 = yb-(12*(50+100)+8*150)/32;

Ppe_2 = 32 * force_per_strand_after_losses;

fb_2 = (Ppe_2 * 10^3)/Ag + (Ppe_2 * ec_2 * 10^3)/sb;

ec_3 = yb-(12*(50+100)+6*150)/30;

Ppe_3 = 30 * force_per_strand_after_losses;

fb_3 = (Ppe_3 * 10^3)/Ag + (Ppe_3 * ec_3 * 10^3)/sb;

Pi = 32 * 98.7 * 0.92 * fpi * 10^-3;

n = Ep / Eci_girder;

Agt = 528487.6385;

Igt = 1.1391*10^11;

n_2 = Ep / Ec_girder;

Ict = 3.0694*10^11;

Act = 1028487.6385;

fcgs = (Pi * 10^3) / Agt + ((Pi * 10^3) * (ec_2)^2) / Igt - (MG*10^6*ec_2) /
Igt;

Delta_ES = n*fcgs;
```

UNIVERSITY OF TORONTO

```
Delta_SR_CR_REL = 10 * fpi * ((98.7 * 32)/Ag) * 1.1 * (35/42) + 83 * 1.1 *
(35/42) + 17;

Total_PSLoss = Delta_SR_CR_REL + Delta_ES + 17;

Total_PSLoss_Percent = Total_PSLoss/fpi*100;

Initial_PSLoss = 17 + Delta_ES;

Initial_PSLoss_Percent = Initial_PSLoss/fpi*100;

fpe = fpi - Total_PSLoss;

Ppe_4 = 32*fpe*98.7*10^-3;

fb_4 = (Ppe_4 * 10^3)/Ag + (Ppe_4 * ec_2 * 10^3)/sb;

Pi = 32*98.7*(fpi-Initial_PSLoss)*10^-3;


fti = -Pi*10^3/Ag + (Pi*10^3*ec_2)/st - MG*10^6/st;

fbi = Pi*10^3/Ag + (Pi*10^3*ec_2)/sb - MG*10^6/sb;

fts = Ppe_4*10^3/Ag - (Ppe_4*10^3*ec_2)/st + (MG+MS)*10^6/st +
(MSDL+MLL_midspan)*10^6/(Ic/(ytc-200));

fbs = Ppe_4*10^3/Ag + (Ppe_4*10^3*ec_2)/sb - (MG+MS)*10^6/sb -
(MSDL+MLL_midspan)*10^6/sbc;


%Cracking Moment:

fbs_2 = -(Ppe_4*10^3)/Ag-(Ppe_4*10^3*ec_2*yb)/Ig+((MG+MS)*10^6*yb)/Ig +
(MSDL+MLL)*10^6*ybc/Ic;

additional_bottom_stress = 0.6 * sqrt(40) - fbs_2;

M_additional = additional_bottom_stress * Ic / (ybc * 10^6);

M_cr = M_additional + (MG+MS) + (MSDL+MLL);

1.2*M_cr;

%Deflections:
```

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```
DeltaL = 625000*26000^3/(48*Ec_girder*Ic);

DeltaDL = 5*0.50903124*22.9035893470353*26000^4/(384*Ec_girder*Ig);

DeltaSL = 5*11.39483565*26000^4/(384*Ec_girder*Ig);

DeltaPL = 5*0.065*2.5*21.99224477007*26000^4/(384*Ec_girder*Ig);
```

```
clear all;
close all;
clc;

%Imput Parameters

Eps = 200000;
Ec = 0.043 * 2500^1.5 * sqrt(40);

Aps = 98.7*32;
Ag = 509031.24;
Ag_T = 528487.6385;


y_total = 1371.6;
y_total_composite = 1371.6 + 200;

d = y_total - 93.75;
d_composite = y_total_composite - 93.75;

yt = 743.3605;
yb = y_total - yt;

ec = 534.4895;

Ig = 1.0853*10^11;
Ig_T = 1.1391*10^11;

Ppe = 3545.203730618863 * 10^3;

Epsilon_ctop = 0.003;
Epsilon_0 = 0.002;
fc_prime = 40;

%Initial Stage "Epsilon_si"

Epsilon_si = Ppe / (Aps * Eps);
```

```
%Initial Stage "Epsilon_ci"

ft = -(Ppe / Ag_T) + ( Ppe * ec * yt / Ig_T);
fb = -(Ppe / Ag_T) - ( Ppe * ec * yb / Ig_T);
Epsilon_t = ft / Eps;
Epsilon_b = fb / Eps;
Epsilon_ci_p = Epsilon_t + (Epsilon_b - Epsilon_t) * d / y_total;

Mg = 985.1552900477480 * 10^6;
Ms = 962.8636122203370 * 10^6;

Epsilon_t_DL = -(Mg + Ms) * yt / (Ig_T * Eps);
Epsilon_b_DL = (Mg + Ms) * yb / (Ig_T * Eps);
Epsilon_ci_DL = (ec / yb) * Epsilon_b_DL;

Epsilon_ci = abs(Epsilon_ci_p) + Epsilon_ci_DL;

%Calculation of area segments: Rectangle width 0.01 m or 1 cm%

Area = zeros(y_total_composite/0.01,1);

i=1;
for y=0.01:0.01:y_total_composite
if (y <= 200)
    Area(i)=2500*0.01;

elseif (y <= 403.2)
    Area(i)=508*0.01;

elseif (y <= 555.6)
    Area(i)=(-2*y+1314.4508)*0.01;

elseif (y <= 1139.8)
    Area(i)=203.2*0.01;

elseif (y <= 1368.4)
    Area(i)=(2*y-2076.4)*0.01;

else
    Area(i)=660.4*0.01;

end

i=i+1;
end
```

```
i=1;
j=1;

%Calculation part

for c = 0.01:0.01:y_total_composite
% Epsilon_s = Strain at prestressing steel at the level of the CGS.
Epsilon_s = Epsilon_si + Epsilon_ci + Epsilon_ctop * (d_composite-c)/c;

if (Epsilon_s <= 0.008)
fps = Eps * Epsilon_s;
elseif (Epsilon_s > 0.008)
fps = 1848 - 0.517 / (Epsilon_s - 0.005915);
end

if (fps > 1843.38)
fps = -10^15;
end

integration_limit = -(0.002-Epsilon_ctop)*c/Epsilon_ctop;

C = 0;
i = 1;

T = fps * Aps;

for y=0.01:0.01:c

    Epsilon_c = Epsilon_ctop - Epsilon_ctop*y/c;

    if (y <= integration_limit)
    C = C + (-3333.3333333333*Epsilon_c+46.6666666667)*Area(i);
    else
    C = C + fc_prime*(2*(Epsilon_c/Epsilon_0)-
(Epsilon_c/Epsilon_0)^2)*Area(i);
    end

    i = i+1;
end

if (abs(C-T) < 500)
    break;
end

j = j+1;

end
```

```
Sum = 0;
i=1;
for y=0.01:0.01:c

    Epsilon_c = Epsilon_ctop - Epsilon_ctop*y/c;

    if (y <= integration_limit)
    Cdy = (-3333.3333333333*Epsilon_c+46.6666666667)*Area(i);
    else
    Cdy = fc_prime*(2*(Epsilon_c/Epsilon_0)-(Epsilon_c/Epsilon_0)^2)*Area(i);
    end

    Sum = Sum + Cdy * y;
    i = i+1;
end

y_bar = Sum / C;

z = d_composite - y_bar;

M = z * T;

M_kNm = M * 10^-6;

fprintf('Moment Resistance of the given section: %4.0f +- 0.5 kNm\n\n',
M_kNm);
```

MATLAB Codes for CSA S6-66 rev. 17:

```
close all;
clear all;
clc;

ts = 200;
fc_deck = 35;
tw = 65;
fci_girder = 35;
fc_girder = 40;
L = 26;
A_1strand = 98.7;
fpu = 1860;
fpy = 0.9 * fpu;
Ep = 200000;
fy = 400;
epsilon_s_yield = 0.002;
```

```
Es = 200000;
fu = 550;
epsilon_s_ultimate = 0.1;
Ec_deck = 5000 * sqrt(fc_deck);
Ec_girder = 5000 * sqrt(fc_girder);
Eci_girder = 5000 * sqrt(fci_girder);
b_eff = 2500;
Ig = 1.0853*10^11;
Ag = 509031.24;
yt = 743.3605;
yb = 628.2395;
sb = Ig/yb;
st = Ig/yt;
hb = 1371.6;
Ic = 2.8960*10^11;
Ac = 1009031.24;
ytc = 525.4544;
ybc = 1046.1456;
sbc = Ic/ybc;
stc = Ic/ytc;

MG = 1053.82192461;
MS = 1014;
MSDL = 322.684375;
MLL = 1598.01844605996;
MLL_midspan = 1592.50751740879;

fb = ((MG + MS )*10^6)/sb + ((MSDL + MLL)*10^6)/sbc;

Fb = 0.5 * sqrt(fc_girder);

fpb = fb - Fb;

ybs = 100;

ec = yb - ybs;

Ppe = 3135.1679430798;

fpi = 0.75 * fpu;

AssumedFinalLosses = 0.2 * fpi;

force_per_strand_after_losses = A_1strand *(fpi - AssumedFinalLosses)/10^3;

Number_of_strands_required = Ppe / force_per_strand_after_losses;
```

```
ec_2 = yb-(12*(50+100)+8*150)/32;

Ppe_2 = 32 * force_per_strand_after_losses;

fb_2 = (Ppe_2 * 10^3)/Ag + (Ppe_2 * ec_2 * 10^3)/sb;

ec_3 = yb-(12*(50+100)+6*150)/30;

Ppe_3 = 30 * force_per_strand_after_losses;

fb_3 = (Ppe_3 * 10^3)/Ag + (Ppe_3 * ec_3 * 10^3)/sb;

Pi = 32 * 98.7 * 0.92 * fpi * 10^-3;

n = Ep / Eci_girder;

Agt = 528487.6385;

Igt = 1.1391*10^11;

n_2 = Ep / Ec_girder;

Ict = 3.0694*10^11;

Act = 1028487.6385;

Total_PSLoss = 240;

Total_PSLoss_Percent = Total_PSLoss/fpi*100;

Initial_PSLoss = 105;

Initial_PSLoss_Percent = Initial_PSLoss/fpi*100;

fpe = fpi - Total_PSLoss;

Ppe_4 = 32*fpe*98.7*10^-3;

fb_4 = (Ppe_4 * 10^3)/Ag + (Ppe_4 * ec_2 * 10^3)/sb;

Pi = 32*98.7*(fpi-Initial_PSLoss)*10^-3;


fti = -Pi*10^3/Ag + (Pi*10^3*ec_2)/st - MG*10^6/st;

fbi = Pi*10^3/Ag + (Pi*10^3*ec_2)/sb - MG*10^6/sb;
```

```
fts = Ppe_4*10^3/Ag - (Ppe_4*10^3*ec_2)/st + (MG+MS)*10^6/st +
(MSDL+MLL_midspan)*10^6/(Ic/(ytc-200));

fbs = Ppe_4*10^3/Ag + (Ppe_4*10^3*ec_2)/sb - (MG+MS)*10^6/sb -
(MSDL+MLL_midspan)*10^6/sbc;


%Rectangular Section Assumption:

fbs_2 = -(Ppe_4*10^3)/Ag-(Ppe_4*10^3*ec_2*yb)/Ig+((MG+MS)*10^6*yb)/Ig +
(MSDL+MLL)*10^6*ybc/Ic;

additional_bottom_stress = 0.6 * sqrt(40) - fbs_2;

M_additional = additional_bottom_stress * Ic / (ybc * 10^6);

M_cr = M_additional + (MG+MS) + (MSDL+MLL);

1.2*M_cr;

%Deflections:

DeltaL = 325000*26000^3/(48*Ec_girder*Ic);

DeltaDL = 5*0.50903124*24.5*26000^4/(384*Ec_girder*Ig);

DeltaSL = 5*12*26000^4/(384*Ec_girder*Ig);

DeltaPL = 5*0.065*2.5*23.5*26000^4/(384*Ec_girder*Ig);
```

```
clear all;
close all;
clc;

%Imput Parameters

Eps = 200000;
Ec = 5000 * sqrt(40);

Aps = 98.7*32;
Ag = 509031.24;
Ag_T = 528487.6385;


y_total = 1371.6;
y_total_composite = 1371.6 + 200;
```

```
d = y_total - 93.75;
d_composite = y_total_composite - 93.75;

yt = 743.3605;
yb = y_total - yt;

ec = 534.4895;

Ig = 1.0853*10^11;
Ig_T = 1.1391*10^11;

Ppe = 3647.952 * 10^3;

Epsilon_ctop = 0.003;
Epsilon_0 = 0.002;
fc_prime = 40;

%Initial Stage "Epsilon_si"

Epsilon_si = Ppe / (Aps * Eps);

%Initial Stage "Epsilon_ci"

ft = -(Ppe / Ag_T) + ( Ppe * ec * yt / Ig_T);
fb = -(Ppe / Ag_T) - ( Ppe * ec * yb / Ig_T);
Epsilon_t = ft / Eps;
Epsilon_b = fb / Eps;
Epsilon_ci_p = Epsilon_t + (Epsilon_b - Epsilon_t) * d / y_total;

Mg = 1053.82192461 * 10^6;
Ms = 1014 * 10^6;

Epsilon_t_DL = -(Mg + Ms) * yt / (Ig_T * Eps);
Epsilon_b_DL = (Mg + Ms) * yb / (Ig_T * Eps);
Epsilon_ci_DL = (ec / yb) * Epsilon_b_DL;

Epsilon_ci = abs(Epsilon_ci_p) + Epsilon_ci_DL;

%Calculation of area segments: Rectangle width 0.01 m or 1 cm%

Area = zeros(y_total_composite/0.01,1);

i=1;
for y=0.01:0.01:y_total_composite
if (y <= 200)
    Area(i)=2500*0.01;
```

```
elseif (y <= 403.2)
    Area(i)=508*0.01;

elseif (y <= 555.6)
    Area(i)=(-2*y+1314.4508)*0.01;

elseif (y <= 1139.8)
    Area(i)=203.2*0.01;

elseif (y <= 1368.4)
    Area(i)=(2*y-2076.4)*0.01;

else
    Area(i)=660.4*0.01;

end

i=i+1;
end

i=1;
j=1;

%Calculation part

for c = 0.01:0.01:y_total_composite
% Epsilon_s = Strain at prestressing steel at the level of the CGS.
Epsilon_s = Epsilon_si + Epsilon_ci + Epsilon_ctop * (d_composite-c)/c;

if (Epsilon_s <= 0.008)
fps = Eps * Epsilon_s;
elseif (Epsilon_s > 0.008)
fps = 1848 - 0.517 / (Epsilon_s - 0.005915);
end

if (fps > 1843.38)
fps = -10^15;
end

integration_limit = -(0.002-Epsilon_ctop)*c/Epsilon_ctop;

C = 0;
i = 1;

T = fps * Aps;
```

```
for y=0.01:0.01:c

    Epsilon_c = Epsilon_ctop - Epsilon_ctop*y/c;

    if (y <= integration_limit)
    C = C + (-3333.3333333333*Epsilon_c+46.6666666667)*Area(i);
    else
    C = C + fc_prime*(2*(Epsilon_c/Epsilon_0)-
(Epsilon_c/Epsilon_0)^2)*Area(i);
    end

    i = i+1;
end

if (abs(C-T) < 500)
    break;
end

j = j+1;
end

Sum = 0;
i=1;
for y=0.01:0.01:c

    Epsilon_c = Epsilon_ctop - Epsilon_ctop*y/c;

    if (y <= integration_limit)
    Cdy = (-3333.3333333333*Epsilon_c+46.6666666667)*Area(i);
    else
    Cdy = fc_prime*(2*(Epsilon_c/Epsilon_0)-(Epsilon_c/Epsilon_0)^2)*Area(i);
    end

    Sum = Sum + Cdy * y;
    i = i+1;
end

y_bar = Sum / C;

z = d_composite - y_bar;

M = z * T;

M_kNm = M * 10^-6;

fprintf('Moment Resistance of the given section: %4.0f +- 0.5 kNm\n\n',
M_kNm);
```

Chapter 6 – Reinforced Concrete Deck Design

Table of Contents

6.1 Introduction

The 200 mm reinforced concrete deck is designed in this chapter. The dimensions of the deck are 26000 x 10000 x 200 mm. Like the previous chapters, deck is designed in the three design codes: CSA S6-14 rev. 17, AASHTO LRFD 2014-17 and CSA S6-66. Dead loads and live loads from chapter 3 and 4 of this report is used in this chapter as well. Based on these loads, flexural and shear reinforcement is chosen. Temperature variation and shrinkage reinforcement is also added. At the end, in the summary section, the 3 different designs are compared to see the differences.

6.2 CSA S6-14 rev. 17

6.2.1 Design Inputs

Table 6.2.1.1 – Design Inputs for CSA S6-14 rev. 17 design

Design Components	Dimension
Thickness of slab	200 mm
Girder Spacing	2.5 m
Top Cover	60 +- 10 mm
Bottom Cover	40 +- 10 mm
Concrete cylindrical compressive strength	35 MPa
Reinforced Conrete Unit Weight	24 kN/m3
Asphalt and Waterproofing Thickness	65 mm
Asphalt and Waterproofing Unit Weight	23.5 kN/m3

6.2.2 Dead Loads

Unfactored moment due to dead load, self-weight of the deck slabs and the wearing surface can be approximated by the following equation:

$$M_D^{-} = \frac{w \times L^2}{11} \qquad\qquad M_D^{+} = \frac{w \times L^2}{16}$$

Note: The L in the equations above is the unsupported length between girders. However, in this report, to be conservative, L is chosen as center to center spacing between girders which is 2.5 m.

Moment caused by the self-weight of the deck slab can be determined as the following:

$$w = d \times h \times \gamma_c = 1 \times 0.2 \times 24 = 4.8 \frac{kN}{m}$$

$$M_D^- = \frac{4.8 \times (2.5)^2}{11} = 2.727 \frac{kNm}{m}$$

$$M_D^+ = \frac{4.8 \times (2.5)^2}{16} = 1.875 \frac{kNm}{m}$$

Moment caused by asphalt and waterproofing can be determined as the following:

$$w = d \times h_w \times \gamma_w = 1 \times 0.065 \times 23.5 = 1.5275 \frac{kN}{m}$$

$$M_D^- = \frac{1.5275 \times (2.5)^2}{11} = 0.868 \frac{kNm}{m}$$

$$M_D^+ = \frac{1.5275 \times (2.5)^2}{16} = 0.5967 \frac{kNm}{m}$$

6.2.3 Live Loads

The maximum transverse moment caused by CL-625 Truck on deck slabs longitudinally supported by girders can be determined by the simplified elastic method, using the following equation:

$$M_{CL-625} = \left(0.8 \times \left(S_e + 0.6\right) \times P\right) \times \frac{(1 + DLA)}{10}$$

where:

M_{CL-625} = *Negative and positive transverse bending moment, kNm/m*

S_e = *Equivalent span legnth of concrete deck in meters*

P = *Maximum wheel load of CL − 625 Truck* $= 175/2 = 87.5\ kN$

DLA = *Dynamic Load Allowance* $= 0.4$

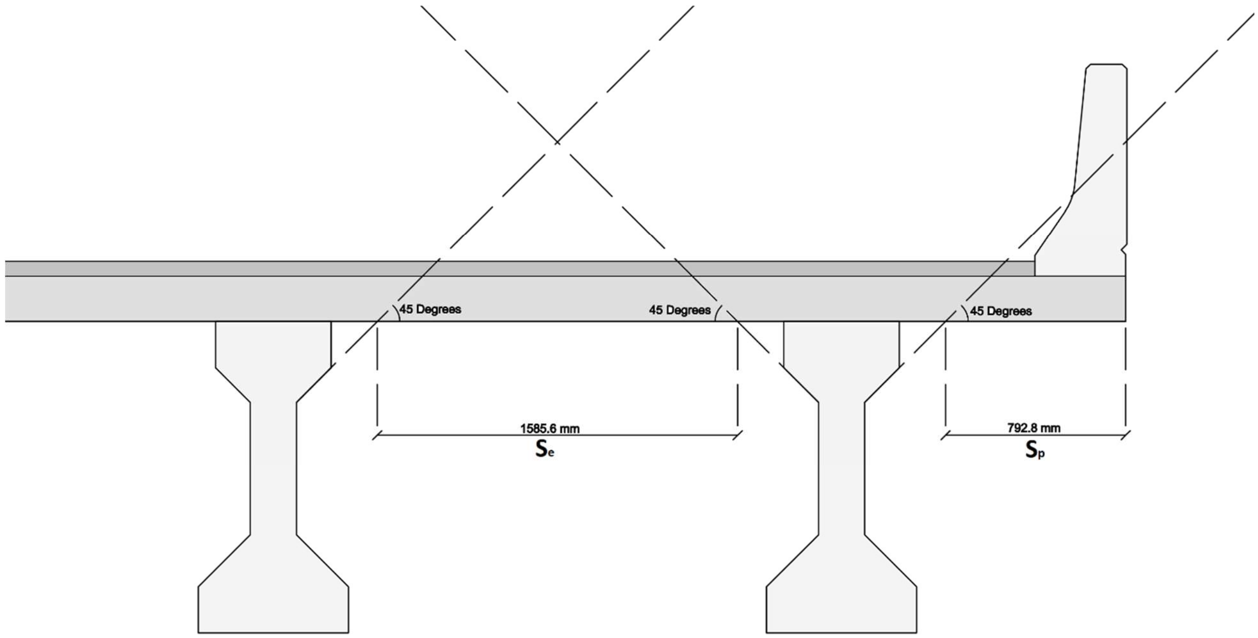


Figure 6.2.3.1 – Graphical Representation of S_e and S_p for the bridge

$$S_e = 1585.6\,mm$$

$$S_p = 792.8\,mm$$

$$M_{CL-625} = (0.8 \times (1.5856 + 0.6) \times 87.5) \times \frac{(1+0.4)}{10} = \mathbf{21.42\,kNm/m}$$

According to CSA S6-14 rev. 17, the longitudinal moment found above must be multiplied with a factor of "120/$S_e^{0.5}$ [percent]" but must not be more than 2/3 of the maximum transverse moment.

$$\frac{120}{S_e^{0.5}} = \frac{120}{1.5856^{0.5}} = \mathbf{95.3\,\% > 2/3\,(66.7\,\%)}$$

Therefore, longitudinal moment can be calculated as follows:

$$M_{CL-625} = 21.42 \times \frac{2}{3} = \mathbf{14.28\,kNm/m}$$

6.2.4 Factored Design Moment

From chapter 4, ultimate limit state load combination was the following:

1.2 x Girder Load + 1.2 x Deck Load + 1.5 x Asphalt and Waterproofing Load + 1.7 x Live Load (Truck)

For slab design, Girder load is out of interest, so this becomes:

1.2 x Deck Load + 1.5 x Asphalt and Waterproofing Load + 1.7 x Live Load (Truck)

Negative Transverse Factored Moment at Supports (At the top of slab where girders are):

$M^{-}_{support}$ = *1.2 x 2.727 + 1.5 x 0.868 + 1.7 x 21.42 =* ***40.99 kNm / m***

Positive Transverse Factored Moment at Midspan (At the bottom of slab between girders):

$M^{+}_{midspan}$ = *1.2 x 1.875 + 1.5 x 0.597 + 1.7 x 21.42 =* ***39.56 kNm / m***

Positive Longitudinal Factored Moment at Midspan (At the bottom of slab):

$M^{+}_{midspan}$ = *1.2 x 1.875 + 1.5 x 0.597 + 1.7 x 14.28 =* ***27.42 kNm / m***

6.2.5 Flexural Design

6.2.5.1 Positive Transverse Moment (Bottom of slab)

Using 15 M Canadian Reinforcement with A_s = 200 mm^2 and d_{bar} = 16 mm

Calculation of the effective depth:

$$d = t_s - cover - \frac{d_{bar}}{2}\,(radius\ of\ bar) = 200 - 40 - \sqrt{\frac{200}{\pi}} = 152.02\ mm$$

Determination of K_r:

$b = 1000\ mm$ *as unit length*

$$K_r = \frac{M_r}{\left(b \times d^2\right)} = \frac{39.56 \times 10^6}{\left(1000 \times 152.02^2\right)}$$

$$= 1.71$$

Determination of the reinforcement ratio, ρ (%):

$-f'_c = 35\ MPa$

$-K_r = 1.71$

From table 2.1 CSA – A23.3 – 14:

$\boldsymbol{\rho = 0.53\ \%}$

Therefore, $A_{s,\,required} = \rho \times b \times d = 0.0053 \times 1000 \times 152.02 = \mathbf{805.71\ mm^2}$

Assuming *5 × 15M Bars which gives* $A_s = \mathbf{1000\ mm^2}$

Minimum reinforcement check:

$A_{s,\,min} = 0.002 \times b \times t_s = 0.002 \times 1000 \times 200 = \mathbf{400\ mm^2}$

$A_s > A_{s,\,min}$ ***so 5 × 15 M OK***

Determination of the reinforcement spacing:

S = The lesser of:

- *S = The larger of:*
  - *– 200 mm*
  - *– Lesser of 1.5 × bar diameter, 1.5 × largest aggregate size (assume 20 mm), 40*
- *S = The lesser of:*
  - *– 200 mm*
  - *– Lesser of 1.5 × slab thickness, 450*

S = The lesser of:

- *S = The larger of:*
  - *– 200 mm*
  - *– Lesser of 23.94, 30, 40 = 23.94 mm*
  - ***– 200 mm chosen***
- *S = The lesser of:*
  - *– 200 mm*
  - *– Lesser of 300, 450 = 300 mm*
  - ***– 200 mm chosen***

– 200 mm chosen

Therefore it is OK to use 15 M bars with 200 mm spacing.

However, this still needs to be checked.

Check for M_r using rectangular stress block assumption:

Stress block parameter, $\alpha = 0.85 + 0.0015 \times f'_c = 0.85 + 0.0015 \times 35 = 0.9025$

$$a = \frac{A_s \times \Phi_s \times f_y}{(\alpha \times \Phi_c \times f'_c \times b)} = \frac{1000 \times 0.9 \times 400}{0.9025 \times 0.75 \times 35 \times 1000} = 15.2\,mm$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 1000 \times 0.9 \times 400 \times \left(152.02 - \frac{15.2}{2}\right) \times 10^{-6} = 52\,kNm/m$$

$M_r\,(52\,kNm/m) > M_f\,(39.56\,kNm/m)$ *OK*

Checking of design against maximum and minimum reinforcement:

Maximum reinforcement requirement $= \frac{c}{d} < 0.5$

Minimum reinforcement requirement = Adequate reinforcement must be placed so that $M_r > 1.2 \times M_{cr}$

$$c = \frac{a}{\alpha} = \frac{15.2}{0.9} = 16.84\,mm$$

$$\frac{c}{d} = \frac{16.84}{152.02} = 0.11 < 0.5\ OK - Max.\ req.\ satisified$$

$$f_{cr} = 0.4 \times \sqrt{f'_c} = 0.4 \times \sqrt{35} = 2.37\,MPa$$

$$S = \frac{b \times t_s^{\,2}}{6} = \frac{1000 \times 200^2}{6} = 6666667$$

$$M_{cr} = S \times f_{cr} = 6666667 \times 2.37 \times 10^{-6} = 15.78\,kNm/m$$

$M_r\,(52\,kNm/m) > 1.2 \times M_{cr}\,(18.93\,kNm/m)$ *OK – Min. req. satisfied*

Calculation of the crack control parameter:

$$z = f_s \times \sqrt[3]{(d_c \times A)} \quad [N/mm]$$

$$z < 25000\ N/mm \text{ for exterior exposure}$$

$$d_c = 40 + \sqrt{\frac{200}{\pi}} = 47.98\ mm$$

$$g = t_s - d = 47.98\ mm$$

$$A = \frac{2 \times g \times b}{n} = \frac{2 \times 47.98 \times 1000}{\frac{1000}{200}} = 19192\ mm^2$$

$$f_s = 0.6 \times f_y = 0.6 \times 400 = 240\ MPa$$

$$z = f_s \times \sqrt[3]{d_c \times A} = 240 \times \sqrt[3]{47.98 \times 19192} = 23348\ N/mm < 25000\ N/mm\ OK$$

Design Reinforcement:

Use 15M @ 200 mm spacing

6.2.5.2 Negative Transverse Moment (Top of slab)

Using 15 M Canadian Reinforcement with A_s = 200 mm^2 and d_{bar} = 16 mm

Calculation of the effective depth:

$$d = t_s - cover - \frac{d_{bar}}{2}\ (radius\ of\ bar) = 200 - 60 - \sqrt{\frac{200}{\pi}} = 132.02\ mm$$

Determination of K_r:

$$b = 1000\ mm \text{ as unit length}$$

$$K_r = \frac{M_r}{\left(b \times d^2\right)} = \frac{40.99 \times 10^6}{\left(1000 \times 132.02^2\right)}$$

$$= 2.35$$

Determination of reinforcement ratio ρ, (%):

$-f'_c = 35\ MPa$

$-K_r = 2.35$

From table 2.1 CSA − A23.3− 14:

By interpolation $\rho = 0.745$ %

Therefore, $A_{s,required} = \rho \times b \times d = 0.00745 \times 1000 \times 132.02 = \mathbf{983.56\ mm^2}$

Assuming $5 \times 15M$ *Bars which gives* $A_s = \mathbf{1000\ mm^2}$

Minimum reinforcement check:

$A_{s,\min} = 0.002 \times b \times t_s = 0.002 \times 1000 \times 200 = \mathbf{400\ mm^2}$

$A_s > A_{s,\min}$ *so* $\mathbf{5 \times 15\,M}$ ***OK***

Determination of the preliminary spacing:

$$s = b \times \frac{A_b}{A_s} = 1000 \times \frac{200}{1000} = \mathbf{200\ mm}$$

Determination of the reinforcement spacing:

S = The lesser of:

S = The larger of:

− 160 *mm*

− *Lesser of* 1.5 × *bar diameter*, 1.5 × *largest aggregate size* (*assume* 20 *mm*), 40

S = The lesser of:

− 160 *mm*

− *Lesser of* 1.5 × *slab thickness*, 450

$S = The\ lesser\ of:$

$S = The\ larger\ of:$

$-\ 160\ mm$

$-\ Lesser\ of\ 23.94,\ 30,\ 40\ = 23.94\ mm$

$-\ 160\ mm\ chosen$

$S = The\ lesser\ of:$

$-\ 160\ mm$

$-\ Lesser\ of\ 300,\ 450 = 300\ mm$

$-\ 160\ mm\ chosen$

$-\ 160\ mm\ chosen$

Therefore it is OK to use 15 *M bars with* 160 *mm spacing.*

Therefore update with 6 × 15 *M per meter.*

However, this still needs to be checked.

Check for M_r *using rectangular stress block assumption:*

Stress block parameter, $\alpha = 0.85 + 0.0015 \times f'_c = 0.85 + 0.0015 \times 35 = 0.9025$

$$a = \frac{A_s \times \Phi_s \times f_y}{(\alpha \times \Phi_c \times f'_c \times b)} = \frac{1200 \times 0.9 \times 400}{0.9025 \times 0.75 \times 35 \times 1000} = 18.24\ mm$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 1200 \times 0.9 \times 400 \times \left(132.02 - \frac{18.24}{2}\right) \times 10^{-6} = 53.09\ kNm/m$$

$M_r\,(53.09\ kNm/m) > M_f\,(40.99\ kNm/m)\ OK$

Checking of design against maximum and minimum reinforcement:

$$Maximum\ reinforcement\ requirement = \frac{c}{d} < 0.5$$

$$Minimum\ reinforcement\ requirement = Adequate\ reinforcement\ must\ be\ placed\ so\ that\ M_r > 1.2 \times M_{cr}$$

$$c = \frac{a}{\alpha} = \frac{18.24}{0.9} = 20.21\ mm$$

$$\frac{c}{d} = \frac{20.21}{132.02} = \mathbf{0.153 < 0.5\ OK - Max.\ req.\ satisified}$$

$$f_{cr} = 0.4 \times \sqrt{f'_c} = 0.4 \times \sqrt{35} = 2.37\ MPa$$

$$S = \frac{b \times t_s^{\,2}}{6} = \frac{1000 \times 200^2}{6} = 6666667$$

$$M_{cr} = S \times f_{cr} = 6666667 \times 2.37 \times 10^{-6} = 15.78\ kNm/m$$

$$M_r(53.09\ kNm/m) > 1.2 \times M_{cr}(18.93\ kNm/m)\ \mathbf{OK - Min.\ req.\ satisfied}$$

Calculation of the crack control parameter:

$$z = f_s \times \sqrt[3]{d_c \times A}\ [N/mm]$$

$$z < 25000\ N/mm\ for\ exterior\ exposure$$

$$d_c = 50^* + \sqrt{\frac{200}{\pi}} = 57.98\ mm$$

* "*In calculating d_c and A for crack control, the clear cover need not be taken to be greater than 50 mm*"

Therefore taken as 50 mm even though cover is 60 mm.

$$A = \frac{2 \times g \times b}{n} = \frac{2 \times 57.98 \times 1000}{\frac{1000}{160}} = 18553\ mm^2$$

$$f_s = 0.6 \times 400 = 240\ MPa$$

$$z = f_s \times \sqrt[3]{d_c \times A} = 240 \times \sqrt[3]{57.98 \times 18553} = \mathbf{24591\ N/mm < 25000\ N/mm\ OK}$$

***Design Reinforcement*:**

Use 15 M @ 160 mm spacing

6.2.5.3 Positive Longitudinal Moment (Bottom Distribution Reinforcement)

According to CSA S6-14 rev. 17, the reinforcement area per meter for longitudinal moment reinforcement must be found by multiplying transverse bottom reinforcement area per meter found above with a factor of “120/$S_e^{0.5}$ [percent]” but this factor must not be more than 200/3.

$$\frac{120}{S_e^{0.5}} = \frac{120}{1.5856^{0.5}} = 95.3 > 66.7$$

$$Therefore\ A_{s,\,required} = \frac{1000 \times 2}{3} = 666.67\ mm^2/m$$

$$A_{s,\,design} = 600\ mm^2/m$$

Use 10 M Canadian Reinforcement with $A_s = 100\ mm^2$

The required preliminary spacing therefore is:

$$s = \frac{100}{600} \times 1000 = 167\ mm/m\ \ use\ s = 140\ mm/m$$

$$A_v = \frac{100 \times 1000}{140} = 714\ mm^2$$

Calculation of the effective depth:

$$d = t_s - cover - d_{bar\ transverse} - \frac{d_{bar}}{2}\ (radius\ of\ bar) = 200 - 40 - 2 \times \sqrt{\frac{200}{\pi}} - \sqrt{\frac{100}{\pi}} = 138.4\ mm$$

Minimum Reinforcement Check:

$$A_{s,\,min} = 0.002 \times b \times t_s = 0.002 \times 1000 \times 200 = 400\ mm^2$$

$$A_s\,(714\ mm^2) > A_{s,\,min}\,(400\ mm^2)$$

Determination of the reinforcement spacing:

S = *The lesser of*:

S = *The larger of*:

− 200 *mm*

− *Lesser of* $1.5 \times$ *bar diameter*, $1.5 \times$ *largest aggregate size* (*assume* 20 *mm*), 40

S = *The lesser of*:

− 200 *mm*

− *Lesser of* $1.5 \times$ *slab thickness*, 450

S = *The lesser of*:

S = *The larger of*:

− 200 *mm*

− *Lesser of* 23.94, 30, 40 = 23.94 *mm*

− 200 *mm chosen*

S = *The lesser of*:

− 200 *mm*

− *Lesser of* 300, 450 = 300 *mm*

− 200 *mm chosen*

− 200 *mm chosen*

However, since *the design spacing was* 140 *mm* < 200 *mm spacing is chosen as* 140 *mm*. *The* 200 *mm spacing is the maximum spacing*.

Therefore it is OK to use 10 *M bars with* 140 *mm spacing.*

However, this still needs to be checked.

Check for M_r using rectangular stress block assumption:

Stress block parameter, $\alpha = 0.85 + 0.0015 \times f'_c = 0.85 + 0.0015 \times 35 = 0.9025$

$$a = \frac{A_s \times \Phi_s \times f_y}{\left(\alpha \times \Phi_c \times f'_c \times b\right)} = \frac{714 \times 0.9 \times 400}{0.9025 \times 0.75 \times 35 \times 1000} = 10.85\ mm$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 714 \times 0.9 \times 400 \times \left(138.4 - \frac{10.85}{2}\right) \times 10^{-6} = 34.19\ kNm/m$$

$M_r\,(34.19\ kNm/m) > M_f\,(27.42\ kNm/m)$ *OK*

Checking of design against maximum and minimum reinforcement:

Maximum reinforcement requirement $= \frac{c}{d} < 0.5$

Minimum reinforcement requirement = Adequate reinforcement must be placed so that $M_r > 1.2 \times M_{cr}$

$$c = \frac{a}{\alpha} = \frac{10.85}{0.9} = 12.03\ mm$$

$$\frac{c}{d} = \frac{12.03}{138.4} = 0.0869 < 0.5\ OK - Max.\ req.\ satisified$$

$$f_{cr} = 0.4 \times \sqrt{f'_c} = 0.4 \times \sqrt{35} = 2.37\ MPa$$

$$S = \frac{b \times t_s^{\,2}}{6} = \frac{1000 \times 200^2}{6} = 6666667$$

$$M_{cr} = S \times f_{cr} = 6666667 \times 2.37 \times 10^{-6} = 15.78\ kNm/m$$

$M_r\,(34.19\ kNm/m) > 1.2 \times M_{cr}\,(18.93\ kNm/m)$ *OK − Min. req. satisfied*

Calculation of the crack control parameter:

$$z = f_s \times \sqrt[3]{d_c \times A}\ [N/mm]$$

$z < 25000\ N/mm$ *for exterior exposure*

$$d_c = g = 40 + 2 \times \sqrt{\frac{200}{\pi}} + \sqrt{\frac{100}{\pi}} = 61.6\ mm$$

$$A = \frac{2 \times g \times b}{n} = \frac{2 \times 61.6 \times 1000}{\frac{1000}{140}} = 17248\ mm^2$$

$$f_s = 0.6 \times 400 = 240\ MPa$$

$$z = f_s \times \sqrt[3]{d_c \times A} = 240 \times \sqrt[3]{61.6 \times 17248} = 24490\ N/mm < 25000\ N/mm\ OK$$

Design Reinforcement:

Use 10 *M* @ 140 *mm spacing*

6.2.6 Design for Top of Slab Shrinkage and Temperature Reinforcement

The minimum amount of reinforcement for shrinkage and temperature must be:

$- 500\ mm^2/m$ *where* $s < 300\ mm$

Use $5 \times 10\ M$ *bars where* $A_v = 500\ mm^2$ *with a spacing of:*

$$s = \frac{1000}{\frac{500}{100}} = 200\ mm/m < 300\ mm/m$$

Design Reinforcement:

Use 10 *M* @ 200 *mm spacing*

6.2.7 Shear Resistance Check

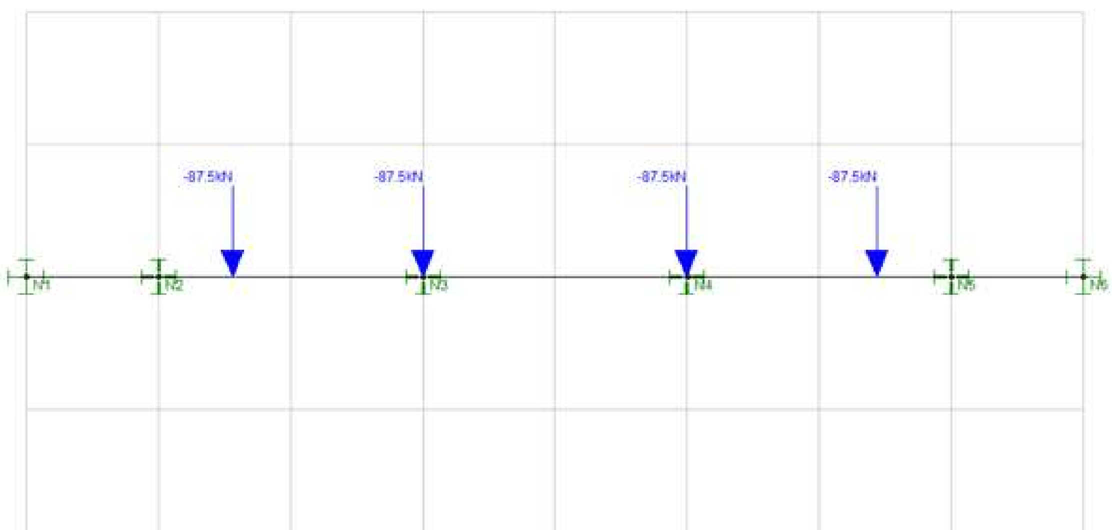
Shear generated by dead loads:

w_D = 1.2 x w_S + 1.5 x w_w
w_D = 1.2 x 4.8 + 1.5 x 1.5275 = 8.036 kN/m
l_n (unsupported length between girders) = 1.992 m
V_{Df} = w_D x l_n = 16 kN

Transverse shear generated by live load truck:

Transverse shear is obtained by arranging trucks heaviest axle wheel loads transversely on the slab. The location and arrangement that produces the largest shear force is chosen. The distance between wheels is 1.8 m and each wheel applies a downward point load of 87.5 kN.

Using a commercial structural analysis software, the arrangement below is found to create the largest transverse shear force (Trucks left wheel is placed at 1.949999 m or 6.250001 m):



MATERIAL PROPERTIES

Label	E [MPa]	G [MPa]	Nu
Concrete_CSA_C35	27098	11782	.15

MAX FORCES OBTAINED

	Axial[kN]	Loc[m]	y Shear[kN]
max	0	0	57.0307
min	0	0	-117.9693

SECTION PROPERTIES

Label	Shape	Type	Material	A [mm2]	Iyy [mm4]	Izz [mm4]	J [mm4]
Slab		None	Concrete_CSA_C35	2e+6	1e+15	6.667e+9	1e+15

-Max shear changes between 116.2 to 117.96 depending on number of stations used. Max value taken.

Figure 6.2.7.1 – Axle Loads that create the maximum shear force

V_{Truck} = 118 kN
Dynamic Load Allowance when only 1 axle used = 0.4
V_f = 16 + 1.7 x (1+0.4) x 118 = 297 kN

Shear resistance provided by concrete:

$$V_c = 2.5 \times \beta \times 0.4 \times \sqrt{f'_c} \times b_{eff} \times d_v$$

where:

$\beta = 0.18$

$f'_c = 35\ MPa$

$d_v = Greater\ of\ 0.9 \times d\ \ or\ \ 0.72 \times t_s = 144\ mm$

$b_{eff} = Effective\ width\ (Calculated\ below)$

$$b_e = \frac{1 - \left(1 - \frac{L}{15 \times b}\right)^3}{b}\ \ where\ L/b < 15,\ \ b_e = b\ for\ L/b \geq 15$$

where:

$b = Half\ of\ the\ unsupported\ distance\ between\ girders = 0.996\ m$

$L = 26\ m$

$L/b = 26.1$

$b_e = 0.996\ m$

$b_{eff} = 0.996 \times 2 + 0.508 = 2.5\ m$

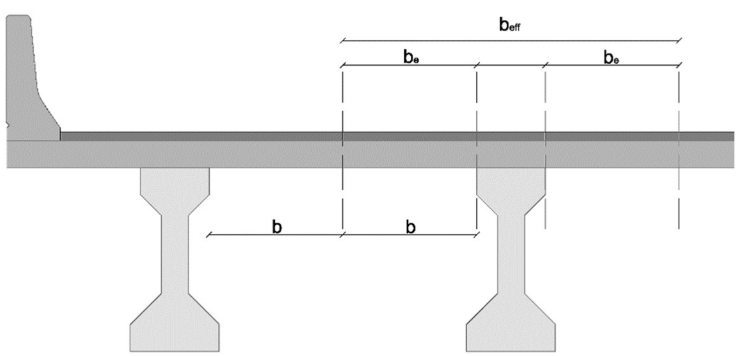


Figure 6.2.7.2 – Graphical Representation of b_{eff} for the bridge

Therefore:

$$V_c = 2.5 \times 0.18 \times 0.4 \times \sqrt{35} \times 2500 \times 144 \times 10^{-3} = 383\ kN > 297\ kN\ OK$$

No need to provide stirrups. Strirrups are usually not provided in thin slabs unless the bridge is constructed in a location where extreme events are common.

6.2.8 Instantaneous Deflection Check

$$\Delta = \frac{P \times L^3}{48 \times E_c \times I_e} \text{ where } I_e = I_{cr} + (I_g - I_{cr}) \times \left(\frac{M_{cr}}{M_a}\right)^3 \leq I_g$$

$$I_g = \frac{l_n \times t_s^3}{12} = \frac{1992 \times 200^3}{12} = 1.328 \times 10^9\ mm^2$$

Assume $I_e = 0.5 \times I_g$ *(Conservative assumption) as an initial deflection check:*

$$I_e = 1.328 \times 10^9 \times 0.5 = 6.64 \times 10^8\ mm^4$$

$$\Delta = \frac{87500 \times 1992^3}{48 \times 27098 \times 6.64 \times 10^8} = 0.8\ mm < \frac{L\ (l_n)}{1000} = \frac{1992}{1000} = 1.992\ mm \text{ therefore OK}$$

Therefore no need to do further calculation.

For the purposes of this report, here is the calculation procedure for I_e *(not required):*

$$E_c = \left(3000 \times \sqrt{f'_c} + 6900\right) \times \left(\frac{2450}{2300}\right)^{1.5} = 27098\ MPa$$

$$n = \frac{E_s}{E_c} = \frac{200000}{27098} = 7.38$$

Assume $A_s = l_n = 1992\ mm^2$

$$B = \frac{l_n}{n \times A_s} = \frac{1992}{7.38 \times 1992} = 0.135$$

$$c = \frac{\sqrt{2 \times d \times B + 1} - 1}{B} = \frac{\sqrt{2 \times 152.02 \times 0.315 + 1} - 1}{0.135} = 40.56\ mm$$

$$I_{cr} = \frac{l_n \times c^3}{3} + n \times A_s \times (d - c)^2 = \frac{1992 \times 40.56^3}{3} + 7.38 \times 1992 \times (152.02 - 40.56)^2 = 226.96 \times 10^6\ mm^4$$

$$M_{cr} = S \times f_{cr} = \frac{1992 \times 200^2}{6} \times 0.4 \times \sqrt{35} \times 10^{-6} = 31.43\ kNm$$

$$I_e = 226.96 \times 10^6 + \left(1.328 \times 10^9 - 226.96 \times 10^6\right) \times \left(\frac{31.43}{40.99}\right)^3 = 723.27 \times 10^6\ mm^4$$

K & B

6.3 AASHTO LRFD 2014-17

6.3.1 Design Inputs

Table 6.3.1.1 – Design Inputs for AASHTO LRFD 2014-17 design

Design Components	Dimension
Thickness of slab	200 mm
Girder Spacing	2.5 m
Top Cover	65 +- 10 mm
Bottom Cover	30 +- 10 mm
Concrete cylindrical compressive strength	35 MPa
Reinforced Conrete Unit Weight	22.8 kN/m3
Asphalt and Waterproofing Thickness	65 mm
Asphalt and Waterproofing Unit Weight	22 kN/m3

6.3.2 Dead Loads

Unfactored moment due to dead load, self-weight of the deck slabs and the wearing surface can be approximated by the following equation:

$$M_D^- = \frac{w \times L^2}{11} \qquad M_D^+ = \frac{w \times L^2}{16}$$

Note: The L in the equations above is the unsupported length between girders. However, in this report, to be conservative, L is chosen as center to center spacing between girders which is 2.5 m.

Moment caused by the self-weight of the deck slab can be determined as the following:

$$w = d \times h_s \times \gamma_c = 1 \times 0.2 \times 22.8 = 4.56 \frac{kN}{m}$$

$$M_D^- = \frac{4.56 \times (2.5)^2}{11} = 2.59 \frac{kNm}{m}$$

$$M_D^+ = \frac{4.56 \times (2.5)^2}{16} = 1.78 \frac{kNm}{m}$$

Moment caused by asphalt and waterproofing can be determined as the following:

$$w = d \times h_w \times \gamma_w = 1 \times 0.065 \times 22 = 1.43 \frac{kN}{m}$$

$$M_D^{-} = \frac{1.43 \times (2.5)^2}{11} = 0.813 \frac{kNm}{m}$$

$$M_D^{+} = \frac{1.43 \times (2.5)^2}{16} = 0.559 \frac{kNm}{m}$$

6.3.3 Live Loads

From table A4-1 which can be found at the end of chapter 4 of AASHTO:

Table 6.3.3.1 – Part of Table A4-1 AASHTO

S mm	Positive Moment	NEGATIVE MOMENT Distance from CL of Girder to Design Section for Negative Moment						
		0.0 mm	75 mm	150 mm 169.33	225 mm	300 mm	450 mm	600 mm
1300	21 130	11 720	10 270	8940	7950	7150	6060	5470
1400	21 010	14 140	12 210	10 340	8940	7670	5960	5120
1500	21 050	16 320	14 030	11 720	9980	8240	5820	5250
1600	21 190	18 400	15 780	13 160	11 030	8970	5910	4290
1700	21 440	20 140	17 290	14 450	12 010	9710	6060	4510
1800	21 790	21 690	18 660	15 630	12 930	10 440	6270	4790
1900	22 240	23 050	19 880	16 710	13 780	11 130	6650	5130
2000	22 780	24 260	20 960	17 670	14 550	11 770	7030	5570
2100	23 380	26 780	23 190	19 580	16 060	12 870	7410	6080
2200	24 040	27 670	24 020	20 370	16 740	13 490	7360	6730
2300	24 750	28 450	24 760	21 070	17 380	14 570	9080	8050
2400	25 500	29 140	25 420	21 700	17 980	15 410	10 870	9340
2500	26 310	29 720	25 990	22 250	18 510	16 050	12 400	10 630
2600	27 220	30 220	26 470	22 730	18 980	16 480	13 660	11 880

For a spacing of 2500 mm, positive moment is taken from the table as:
26.31 kNm / m

Article 4.6.2.1.6 says that design section spacing for I precast girders (like AASHTO Type IV) can be taken as one third of the top flange width of the girder section.

AASHTO Type IV Girders have a top flange width of 508 mm. One third of 508 is 169.33 mm.

For negative moment, values of 22.25 kNm and 18.51 kNm are taken to be interpolated for 170 mm.

$$M_{LL}^{+} = 26.31\ kNm/m$$

$$M_{LL}^{-} = 22.25 - \frac{170-150}{225-150} \times (22.25 - 18.51) = 21.25\ kNm/m$$

6.3.4 Factored Design Moment

Using Strength I factors from chapter 4:

M^{+} = 1.25 x 1.78 + 1.5 x 0.559 + 1.75 x 26.31 = 49.11 kN/m
M^{-} = 1.25 x 2.59 + 1.5 x 0.813 + 1.75 x 21.25 = 41.64 kN/m

6.3.5 Flexural Design

6.3.5.1 Steel Reinforcement at the Midspan

For the design, Canadian 15 M reinforcement bars with A_s = 200 mm^2 and d_{bar} = 16 mm will be used since this design is to be constructed in Canada.

Determination of Number of Reinforcements and Spacing:

$$d = 200\ mm - 30\ mm\ cover - \sqrt{\frac{200}{\pi}} = 168.23\ mm$$

$$k' = \frac{M_u}{\Phi \times b \times d^2} = \frac{49.11 \times 10^6}{0.9 \times 1000 \times 168.23^2} = 1.928$$

$$\rho = 0.85 \times \frac{f'_c}{f_y} \times \left(1 - \sqrt{1 - 2 \times \frac{k'}{0.85 \times f'_c}}\right)$$

$$\rho = 0.85 \times \frac{35}{400} \times \left(1 - \sqrt{1 - 2 \times \frac{1.928}{0.85 \times 35}}\right) = 0.00499$$

$$A_{s,\,required} = \rho \times b \times d = 0.00499 \times 1000 \times 168.23 = 839\ mm^2$$

$$Number\ of\ bars = \frac{839\ mm^2}{200\ mm^2} = 4.19$$

$$Spacing = \frac{1000\ mm}{4.19} = 238\ mm$$

Use design spacing 225 mm

$$A_{s,provided} = \frac{1000}{225} \times 200 = 888.89\ mm^2 / m$$

Checking against minimum reinforcement requirement:

$$M_{cr} = \gamma_3 \times \gamma_1 \times f_r \times S_c$$

$$\gamma_3 = 0.67\ for\ f_y = 400\ MPa\ reinforcing\ steel$$

$$\gamma_1 = 1.6$$

$$f_r = 0.6 \times \sqrt{f'_c}$$

$$M_{cr} = 0.67 \times 1.6 \times 0.6 \times \sqrt{35} \times \frac{1000 \times 200^2}{6} = 25.37\ kNm$$

$$M_r = \Phi \times M_n = \Phi \times \left(A_s \times f_y \times \left(d_s - \frac{A_s \times f_y}{0.85 \times 2 \times f'_c \times b}\right)\right)$$

$$M_r = 0.9 \times \left(888.89 \times 400 \times \left(168.23 - \frac{888.89 \times 400}{0.85 \times 2 \times 35 \times 1000}\right)\right) \times 10^{-6}$$

$$M_r = 51.92\ kNm / m > M_{cr}\ OK$$

6.3.5.2 Steel Reinforcement at the end supports

For the design, Canadian 15 M reinforcement bars with A_s = 200 mm^2 and d_{bar} = 16 mm will be used since this design is to be constructed in Canada.

Determination of Number of Reinforcements and Spacing:

$$d = 200\,mm - 65\,mm\ cover - \sqrt{\frac{200}{\pi}} = 127.02\,mm$$

$$k' = \frac{M_u}{\Phi \times b \times d^2} = \frac{41.64 \times 10^6}{0.9 \times 1000 \times 127.02^2} = 2.868$$

$$\rho = 0.85 \times \frac{f'_c}{f_y} \times \left(1 - \sqrt{1 - 2 \times \frac{k'}{0.85 \times f'_c}}\right)$$

$$\rho = 0.85 \times \frac{35}{400} \times \left(1 - \sqrt{1 - 2 \times \frac{2.868}{0.85 \times 35}}\right) = 0.00755$$

$$A_{s,\ required} = \rho \times b \times d = 0.00755 \times 1000 \times 127.02 = 959.43\,mm^2$$

$$Number\ of\ bars = \frac{959.43\,mm^2}{200\,mm^2} = 4.8$$

$$Spacing = \frac{1000\,mm}{4.8} = 208.45\,mm$$

Use design spacing 200 mm

$$A_{s,provided} = \frac{1000}{200} \times 200 = 1000\,mm^2 / m$$

Checking against minimum reinforcement requirement:

$$M_{cr} = \gamma_3 \times \gamma_1 \times f_r \times S_c$$

$$\gamma_3 = 0.67\ for\ f_y = 400\ MPa\ reinforcing\ steel$$

$$\gamma_1 = 1.6$$

$$f_r = 0.6 \times \sqrt{f'_c}$$

$$M_{cr} = 0.67 \times 1.6 \times 0.6 \times \sqrt{35} \times \frac{1000 \times 200^2}{6} = 25.37\,kNm$$

$$M_r = \Phi \times M_n = \Phi \times \left(A_s \times f_y \times \left(d_s - \frac{A_s \times f_y}{0.85 \times 2 \times f'_c \times b} \right) \right)$$

$$M_r = 0.9 \times \left(1000 \times 400 \times \left(127.02 - \frac{1000 \times 400}{0.85 \times 2 \times 35 \times 1000} \right) \right) \times 10^{-6}$$

$$M_r = 43.31\ kNm/m > M_{cr}\ OK$$

6.3.5.3 Crack Control

For the design, Canadian 15 M reinforcement bars with A_s = 200 mm^2 and d_{bar} = 16 mm will be used since this design is to be constructed in Canada.

Reinforcement for crack control in both directions can be determined as follows:

$$\frac{A_v}{b_w \times s_v} \geq 0.00092$$

$$\frac{200}{1000 \times s_v} \geq 0.00092 \quad Therefore\ s_v = 217\ mm$$

Positive moment Reinforcement at midspan has a spacing of 225 mm update it to 200 mm to meet this requirement.

Provide 15 M Bars @ 200 mm in longitudinal direction to meet with this requirement.

6.3.6 Additional Reinforcement

A percentage of the transverse reinforcement must be installed in the longitudinal direction. The calculations for it is as follows:

$$\frac{125}{\sqrt{S}} \leq 66.7\% \text{ where } S = \text{Distance between the faces of girder webs in meters}$$

$$\frac{125}{\sqrt{2.2968}} = 82.5 > 66.7\% \text{ take } 66.7\%$$

$$A_s = \frac{2}{3} \times 1000 = 667\ mm^2 / m$$

$$\text{Number of Bars} = \frac{667}{200} = 3.33$$

$$\text{Spacing} = \frac{1000}{3.33} = 300\ mm$$

However crack control requirement was 200 mm which passes this so use 200 mm spacing.

$15\ M$ @ $200\ mm > 575\ mm^2 / m$ OK – Minimum Requirement Satisfied for bottom.

$15\ M$ @ $200\ mm > 385\ mm^2 / m$ OK – Minimum Requirement Satisfied for top.

UNIVERSITY OF TORONTO

6.4 CSA S6-66

6.4.1 Design Inputs

Table 6.4.1.1 – Design Inputs for CSA S6-66 design

Design Components	Dimension
Thickness of slab	200 mm
Girder Spacing	2.5 m
Top Cover	50 +- 10 mm
Bottom Cover	25 +- 10 mm
Concrete cylindrical compressive strength	35 MPa
Reinforced Conrete Unit Weight	24 kN/m3
Asphalt and Waterproofing Thickness	65 mm
Asphalt and Waterproofing Unit Weight	23.5 kN/m3

6.4.2 Dead Loads

Unfactored moment due to dead load, self-weight of the deck slabs and the wearing surface can be approximated by the following equation:

$$M_D^{-} = \frac{w \times L^2}{11} \qquad M_D^{+} = \frac{w \times L^2}{16}$$

Note: The L in the equations above is the unsupported length between girders. However, in this report, to be conservative, L is chosen as center to center spacing between girders which is 2.5 m.

Moment caused by the self-weight of the deck slab can be determined as the following:

$$w = d \times h \times \gamma_c = 1 \times 0.2 \times 24 = 4.8 \frac{kN}{m}$$

$$M_D^{-} = \frac{4.8 \times (2.5)^2}{11} = 2.727 \frac{kNm}{m}$$

$$M_D^{+} = \frac{4.8 \times (2.5)^2}{16} = 1.875 \frac{kNm}{m}$$

Moment caused by asphalt and waterproofing can be determined as the following:

K & B

$$w = d \times h_w \times \gamma_w = 1 \times 0.065 \times 23.5 = 1.5275 \frac{kN}{m}$$

$$M_D^{-} = \frac{1.5275 \times (2.5)^2}{11} = 0.868 \frac{kNm}{m}$$

$$M_D^{+} = \frac{1.5275 \times (2.5)^2}{16} = 0.5967 \frac{kNm}{m}$$

6.4.3 Live Loads

Determination of Live Load moment:

$$M_{LL}{}^{*} = 0.8 \times (1+I) \times \frac{S+2}{32} \times P$$

* *This is an equation that requires imperial units*

$P = 16000\ lbs$ (*Load applied by a single rear wheel of the design truck*)

S = *Effective Span Length in feet* = *clear span between supports* + *slab thickness* $= 6.535 + 0.656 = 7.19\ ft$

$$I = \frac{50}{\text{Length between girders }(L) + 125} = \frac{50}{8.2 + 125} = 0.375 > 0.3 \text{ Therefore } I = 0.3$$

$$M_{LL} = 0.8 \times 1.3 \times \frac{7.19 + 2}{32} \times 16000 \times 10^{-3} = 4.78\ kft/ft = 21.26\ kNm/m$$

6.4.4 Factored Design Moment

Using ultimate factors from chapter 4:

M^+ = 1.5 x 1.875 + 1.5 x 0.5967 + 2.5 x 21.26 = 56.86 kN/m
M^- = 1.5 x 2.727 + 1.5 x 0.868 + 2.5 x 21.26 = 58.54 kN/m

6.4.5 Flexural Design

6.4.5.1 Negative Transverse Moment Reinforcement (Top of Slab)

For the design, Canadian 15 M reinforcement bars with A_s = 200 mm^2 and d_{bar} = 16 mm will be used since this design is to be constructed in Canada.

Calculation of the effective depth:

$$d = t_s - cover - \frac{d_{bar}}{2}\,(radius\ of\ bar) = 200 - 50 - \sqrt{\frac{200}{\pi}} = 142.02\ mm$$

Determination of K_r:

$$K_r = \frac{M_f}{b \times d^2} = \frac{58.54 \times 10^6}{1000 \times 142.02^2} = 2.9$$

The following desing process is iterative in CSA S6 – 66:

Determination of reinforcement spacing and area:

Assume 15 M Bars @ 200 mm

$$Therefore\ A_s = \frac{200 \times 1000}{200} = 1000\ mm^2$$

Check for M_r using rectangular stress block assumption:

$$a = \frac{A_s \times \Phi_s \times f_y}{(0.85 \times \Phi_c \times f'_c \times b)} = \frac{1000 \times 0.9 \times 400}{0.85 \times 0.75 \times 35 \times 1000} = 16.13\ mm$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 1000 \times 0.9 \times 400 \times \left(142.02 - \frac{16.13}{2}\right) \times 10^{-6} = 48.22\ kNm/m$$

$$M_r\,(48.22\ kNm/m) < M_f\,(58.54\ kNm/m)\ NOT\ OK$$

Therefore, in this case initial assumption of spacing must be reduced and calculations must be redone. These iterations can be done by EXCEL or MATLAB easily.

Determination of reinforcement spacing and area:

Assume 15 M Bars @ 160 mm

$$Therefore\ A_s = \frac{200 \times 1000}{160} = 1250\ mm^2$$

Check for M_r using rectangular stress block assumption:

$$a = \frac{A_s \times \Phi_s \times f_y}{(0.85 \times \Phi_c \times f'_c \times b)} = \frac{1250 \times 0.9 \times 400}{0.85 \times 0.75 \times 35 \times 1000} = 20.17\ mm$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 1250 \times 0.9 \times 400 \times \left(142.02 - \frac{20.17}{2}\right) \times 10^{-6} = 59.37\ kNm/m$$

$$M_r\,(59.37\ kNm/m) > M_f\,(58.54\ kNm/m)\ OK$$

Checking of design against minimum reinforcement requirement:

$$f_{cr} = 0.6 \times \sqrt{f'_c} = 0.6 \times \sqrt{35} = 3.55\ MPa$$

$$S = \frac{b \times t_s^2}{6} = \frac{1000 \times 200^2}{6} = 6666667$$

$$M_{cr} = S \times f_{cr} = 6666667 \times 2.37 \times 10^{-6} = 23.66\ kNm/m$$

$$M_r\,(59.37\ kNm/m) > 1.2 \times M_{cr}\,(28.4\ kNm/m)\ OK - Min.\ req.\ satisfied$$

In this case no iteration is required for this step.

Calculation of the crack control parameter:

$$z = f_s \times \sqrt[3]{d_c \times A}\ [N/mm]$$

$$z < 25000\ N/mm\ for\ exterior\ exposure$$

$$d_c = g = 50 + \sqrt{\frac{200}{\pi}} = 57.98\ mm$$

$$A = \frac{2 \times g \times b}{n} = \frac{2 \times 57.98 \times 1000}{\frac{1000}{160}} = 18553\ mm^2$$

$f_s = 0.6 \times 400 = 240\ MPa$

$z = f_s \times \sqrt[3]{d_c \times A} = 240 \times \sqrt[3]{57.98 \times 18553} = 24591\ N/mm < 25000\ N/mm\ OK$

Design Reinforcement:

Use 15 M @ 160 mm spacing

In this case no iteration is required for this step.

6.4.5.2 Steel Reinforcement at the Ends (Bottom of slab)

For the design, Canadian 15 M reinforcement bars with A_s = 200 mm^2 and d_{bar} = 16 mm will be used since this design is to be constructed in Canada.

Calculation of the effective depth:

$$d = t_s - cover - \frac{d_{bar}}{2} (radius\ of\ bar) = 200 - 25 - \sqrt{\frac{200}{\pi}} = 167.02\ mm$$

Determination of reinforcement spacing and area:

Assume 15 M Bars @ 200 mm

$$Therefore\ A_s = \frac{200 \times 1000}{200} = 1000\ mm^2$$

Check for M_r using rectangular stress block assumption:

$$a = \frac{A_s \times \Phi_s \times f_y}{(0.85 \times \Phi_c \times f'_c \times b)} = \frac{1000 \times 0.9 \times 400}{0.85 \times 0.75 \times 35 \times 1000} = 16.13\ mm$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 1000 \times 0.9 \times 400 \times \left(167.02 - \frac{16.13}{2}\right) \times 10^{-6} = 57.22\ kNm/m$$

$M_r (57.22\ kNm/m) > M_f (56.86\ kNm/m)\ OK$

Checking of design against minimum reinforcement requirement:

$$f_{cr} = 0.6 \times \sqrt{f'_c} = 0.6 \times \sqrt{35} = 3.55\ MPa$$

$$S = \frac{b \times t_s^{\ 2}}{6} = \frac{1000 \times 200^2}{6} = 6666667$$

$$M_{cr} = S \times f_{cr} = 6666667 \times 2.37 \times 10^{-6} = 23.66\ kNm/m$$

$$M_r\,(57.22\ kNm/m) > 1.2 \times M_{cr}\,(28.4\ kNm/m)\ OK - Min.\ req.\ satisfied$$

Calculation of the crack control parameter:

$$z = f_s \times \sqrt[3]{d_c \times A}\ [N/mm]$$

$$z < 25000\ N/mm\ for\ exterior\ exposure$$

$$d_c = g = 25 + \sqrt{\frac{200}{\pi}} = 32.98\ mm$$

$$A = \frac{2 \times g \times b}{n} = \frac{2 \times 32.98 \times 1000}{\frac{1000}{200}} = 13192\ mm^2$$

$$f_s = 0.6 \times 400 = 240\ MPa$$

$$z = f_s \times \sqrt[3]{d_c \times A} = 240 \times \sqrt[3]{32.98 \times 13192} = 18185\ N/mm < 25000\ N/mm\ OK$$

Design Reinforcement:

Use 15 M @ 200 mm spacing

In this case no iterations are required.

6.4.6 Bottom Distribution Reinforcement

$$\text{Bottom distribution reinforcement in } \% = \frac{125}{\sqrt{S}} < 66.7\% \quad S \text{ in meters}$$

$$\text{Bottom distribution reinforcement in } \% = 66.7\%$$

$$A_{s,bottom} = \frac{2}{3} \times \text{Area per meter of Bottom transverse reinforcement} = 667\ mm^2/m$$

$$\text{Number of Bars} = \frac{667}{200} = 3.33$$

$$\text{Spacing} = \frac{1000}{3.33} = 300\ mm$$

Design Reinforcement:

15 *M* @ 300 *mm*

6.4.7 Top Shrinkage and Temperature Reinforcement

According to CSA S6 – 66:

Minimum area required: 400 mm^2/m

Spacing required: 300 *mm*

Design Reinforcement:

Use 10 *M* @ 250 *mm*

6.5 Summary

Based on the results, AASHTO LRFD 2014-17 requires more reinforcement in longitudinal direction. For bottom primary reinforcement, all codes are similar.

Table 6.5.1 – Summary of deck reinforcement designed

	Transverse direction			Longitudinal Direction		
Reinforcement	**CSA S6-14 rev. 17**	**AASHTO LRFD 2014-17**	**CSA S6-66**	**CSA S6-14 rev. 17**	**AASHTO LRFD 2014-17**	**CSA S6-66**
Bottom of Slab	15 M @ 200 mm	15 M @ 200 mm	15 M @ 200 mm	10 M @ 140 mm	15 M @ 200 mm	15 M @ 300 mm
Top of Slab	15 M @ 160 mm	15 M @ 200 mm	15 M @ 160 mm	10 M @ 200 mm	15 M @ 200 mm	10 M @ 250 mm

6.6 References

- CSA S6-14 Highway Bridge Design Code: Canadian Standards Association, 2014, Revision 2017

- AASHTO LRFD Bridge Design Specifications: American Association of State Highway and Transportation Officials, 2014, 8th Edition, SI - Revision 2017

- CSA S6-66 Design of Highway Bridges: Canadian Standards Association, 1966

Chapter 7 – Durability Design

Table of Contents

7.1 Introduction

This final chapter includes the design of the concrete to be used in the construction. Exposure levels are introduced and identified both for the girder and the deck. Based on the conditions determined, properties of the concrete to be used are specified. Some of these properties are water to cement ratio, air entraining admixtures and required slum.

7.2 Concrete Exposure Condition

According to CSA A23.1-14:

Exposure class for girders = A-1
Exposure class for deck = C-1

Aggregate size is assumed to be 20 mm (conservative assumption).

Table 7.2.1 – Requirements for C-1 and A-1 class concrete

Class	Maximum Water to cement ratio	Minimum specified cylidrical compressive strength (MPa)	Air Content (%)
A-1	0.4	35	between 5 to 8
C-1	0.4	35	between 5 to 8

7.3 Mix Design Components

7.3.1 Strength

The design in this report uses f'_c = 35 MPa for deck and f'_c = 40 MPa for girders. These values are the minimum values. The concrete mixture should be able to provide *minimum* this, ideally higher.

According to ACI 318, the design mixture should aim these average values:

For Deck: ***f_{cr}' =1.1 x f'_c + 5 = 1.1 x 35 + 5 = 43.5 MPa so approximately 45 MPa***
For Girders***: f_{cr}' =1.1 x f'_c + 5 = 1.1 x 40 + 5 = 49 MPa so approximately 50 MPa***

7.3.2 Water to Cement Ratio

The table below shows the proposed water to cement ratio to be used in the US. For the purposes of this report, this data is used in determining the water cement ratio.

Table 7.3.2.1 – American concrete institutions water to cement ratio requirements

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

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.

The bridge is to be constructed in Canada where extreme weather conditions occur frequently. Therefore, Air-entrained concrete will be used.

Looking at the table data and curve fitting the data on EXCEL, design water to cement ratio for:
Deck = 0.3
Girders = 0.27

7.3.3 Air Content

Bridge deck is exposed to salts and chemicals every winter in Canada. Therefore, it is considered in the severe exposure category.

Girders however are protected by the bridge deck, so they are considered in moderate exposure category.

Based on the figure below, air content is chosen to be ***6.4% for deck and 5.2 % for girders***.

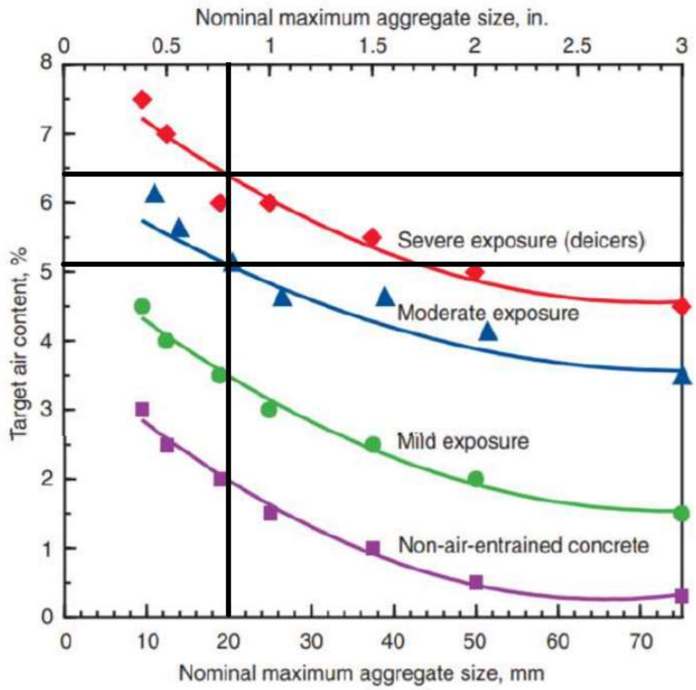


Figure 7.3.3.1 – Relationship between exposure type, aggregate size and air content

7.3.4 Slump

Slump test is a measure of concrete consistency and workability. Concrete needs to flow through the steel bars but should also not be very fluid like so that it stays where it should stay. Higher slump therefore means more water like concrete which once disturbed, recovers quickly. ACI provides a table for limits of slump. Those values are used for this design.

Table 7.3.4.1 – ACI recommended values for slump of concrete to be used in different parts of the bridge

	Slump, mm (in.)	
Concrete construction	**Maximum***	**Minimum**
Reinforced foundation walls and footings	75 (3)	25 (1)
Plain footings, caissons, and substructure walls	75 (3)	25 (1)
Beams and reinforced walls	100 (4)	25 (1)
Building columns	100 (4)	25 (1)
Pavements and slabs	75 (3)	25 (1)
Mass concrete	75 (3)	25 (1)

*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.

Based on the table above, ***slump of concrete expected to be 75+-25 mm for girders and 50+-25 for the deck.***

7.3.5 Water Content

The water content approximation by ACI that is used in this design is given below:

Table 7.3.4.1 – Approximate water content recommendations given by ACI

Slump, mm	Water, kilograms per cubic meter of concrete, for indicated sizes of aggregate*							
	9.5 mm	12.5 mm	19 mm	25 mm	37.5 mm	50 mm**	75 mm**	150 mm**
	Non-air-entrained concrete							
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	—
Approximate amount of entrapped air in non-air-entrained concrete, percent	3	2.5	2	1.5	1	0.5	0.3	0.2
	Air-entrained concrete							
25 to 50	181	175	168	160	150	142	122	107
75 to 100	202	193	184	175	165	157	133	119
150 to 175	216	205	197	184	174	166	154	—
Recommended average total air content, percent, for level of exposure:†								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
Severe exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5	4.0

* 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 −1 to +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.

Assuming an aggregate size of 20 mm, a slump of 50 mm and air-entrained concrete, for the deck, water content is chosen to be an average of 168 and 184, 176 kg/m^3. 10% of this demand will be reduced by water reducers.

Therefore, water content demand approximately equals to 160 kg/m^3.

7.3.6 Cement Content

Water cement ratio for deck = 0.3
Water cement ratio for girders = 0.27

Cement for deck = 160 / 0.3 = Approximately 540 kg/m³
Cement for girder = 160 / 0.27 = Approximately 600 kg/m³

7.3.7 Coarse Aggregate Content

According to CSA, fine aggregates must have a fineness modulus of 2.7+-0.2.

For this design, a fineness modulus of 2.8 mm and maximum aggregate size of 20 mm will be used. 2.8 mm is chosen because the value is on the table below.

Table 7.3.7.1 – Bulk volume of dry-rodded coarse aggregate per unit volume of concrete for different moduli of fine aggregate given by ACI

Nominal maximum size of aggregate, mm (in.)	Bulk volume of dry-rodded coarse aggregate per unit volume of concrete for different fineness moduli of fine aggregate*			
	2.40	2.60	2.80	3.00
9.5 (⅜)	0.50	0.48	0.46	0.44
12.5 (½)	0.59	0.57	0.55	0.53
19 (¾)	0.66	0.64	0.62	0.60
25 (1)	0.71	0.69	0.67	0.65
37.5 (1½)	0.75	0.73	0.71	0.69
50 (2)	0.78	0.76	0.74	0.72
75 (3)	0.82	0.80	0.78	0.76
150 (6)	0.87	0.85	0.83	0.81

*Bulk volumes are based on aggregates in a dry-rodded condition as described in ASTM C 29 (AASHTO T 19). Adapted from ACI 211.1.

For calculation purposes, a bulk density of 1600 kg/m³ is assumed. Interpolating between 0.62 and 0.67 from the table above, the bulk volume is found to be 0.63.

Therefore, dry mass of concrete is approximately equal to 1600 x 0.63 = 1000 kg

7.3.8 Admixture Content

Air-entraining admixtures used in Canada should meet the requirements ASTM C260 and ASTM C494.

According to Daravair, a concrete air-entraining admixture manufacturer, an average of 0.005 kg of Daravair per kg of cement is recommended for air content between 4% and 8% and water reducer to be used with their product is recommended as 0.003 kg per kg of cement:

This means, for air-entraining admixture:

For deck: 0.0005 x 540 = 0.27 kg/m^3

For girders: 0.0005 x 600 = 0.3 kg/m^3

For water reducer:

For deck: 0.003 x 540 = 1.6 kg/m^3

For girders: 0.003 x 600 = 1.8 kg/m^3

7.3.9 Fine Aggregate Content

Fine aggregate volume can be found by subtracting the values above from 1 m^3.

For deck:

Water = 160 kg = 0.16 m^3 (1 g/cm^3)
Cement = 540 kg = 0.18 m^3 (3 g/cm^3)
Air = 6.4 % = 0.064 m^3
Coarse aggregate = 1000 kg = 0.38 m^3 (2680 kg/m^3)
Volume of fine aggregate calculated = 1 – 0.16 – 0.18 – 0.064 – 0.38 = 0.216 m^3
Mass of fine aggregate = 0.216 x 2640 = 570 kg
Estimated concrete density = 160 + 540 + 1000 + 570 = 2270 kg/m^3

For girder:

Water = 160 kg = 0.16 m^3 (1 g/cm^3)
Cement = 600 kg = 0.2 m^3 (3 g/cm^3)
Air = 5.2 % = 0.052 m^3
Coarse aggregate = 1000 kg = 0.38 m^3 (2680 kg/m^3)
Volume of fine aggregate calculated = 1 – 0.16 – 0.2 – 0.052 – 0.38 = 0.208 m^3
Mass of fine aggregate = 0.208 x 2640 = 550 kg
Estimated concrete density = 160 + 600 + 1000 + 550 = 2310 kg/m^3

7.3.10 Moisture Content

Assuming that moisture content for:

Coarse aggregate = 2.5 %
Fine aggregate for deck = 5 %
Fine aggregate for girder = 5 %

Therefore, the trial batch aggregate proposition:

Coarse aggregate = 1000 x 1.025 = 1025 kg
Fine aggregate for deck = 570 x 1.05 = 600 kg
Fine aggregate for girder = 550 x 1.05 = 575 kg

Aggregates will absorb water. This needs to be accounted in the calculations. Assuming coarse aggregate will absorb 1.5 % and fine aggregate will absorb 5% of the water:

For deck:
160 - (1000 x 0.015) - 570 x 0.05 = 120 kg

For girder:
160 - (1000 x 0.015) - 550 x 0.05 = 130 kg

7.4 Summary

Table 7.4.1 – Concrete Mix Design for deck and girder for 1 m^3 of concrete

Component	Deck	Girder
Exposure Class	C-1	A-1
Maximum Nominal Aggregate size	20 mm	20 mm
Water to Cement Ratio	0.3	0.27
Required Average Cylindrical Compressive Strength	45 MPa	50 MPa
Air Content	6.40%	5.20%
Slump	50+-25	75+-25
Water to be added	120 kg	130 kg
Cement Weight	540 kg	600 kg
Mass of Coarse Aggregate (2.5% MC)	1025 kg	1025 kg
Mass of Fine Aggregate (5% MC)	600 kg	575 kg
Total	2290 kg	2330 kg
Air-entraining admixture	0.27 kg	0.3 kg
Water Reducer	1.6 kg	1.8 kg

7.5 References

-Canadian Standards Association (2014). Concrete materials and methods of concrete Construction - A23.1-2014.

-American Concrete Institute (2014). Metric Building Code Requirements for Structural Concrete & Commentary - ACI 318M-14.

-Daravair, Product Data Sheet. Available: https://gcpat.hk/sites/hk.gcpat.com/files/2017-08/Daravair.pdf

-The Constructor, "Effects of Air Entrainment on Concrete Strength," Available: https://theconstructor.org/concrete/effect-air-entrainment-concrete-strength/8427/

-S.H.Kosmatka and M.L.Wilson (2011). Design and Control of Concrete Mixtures: The guide to applications, methods, and materials - Portland Cement Association

-American Society for Testing and Materials (2014). Tolerances in Slump or Slump Flow - ASTM MNL49-2ND-EB

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APPENDIX
DESIGN DRAWINGS

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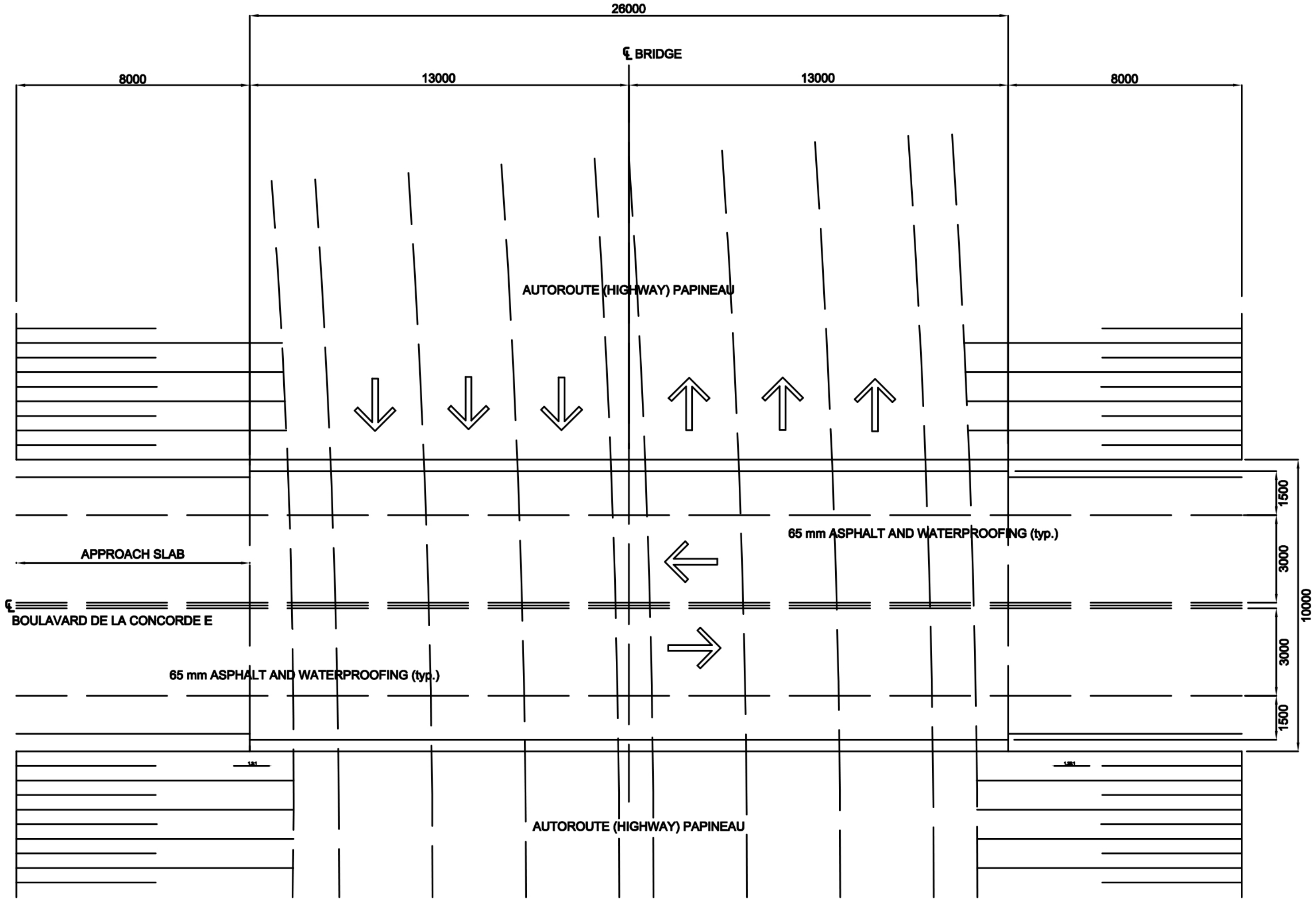
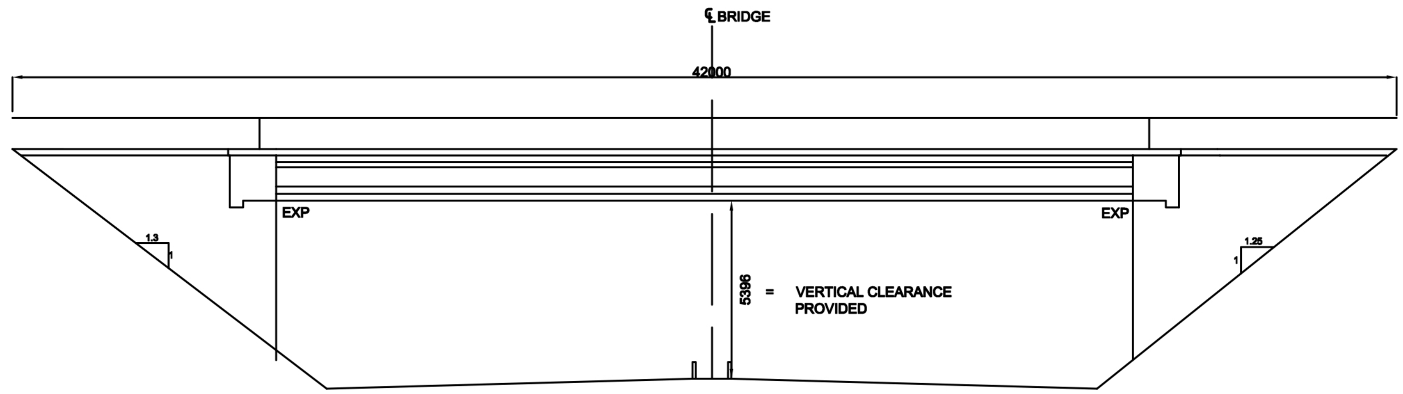
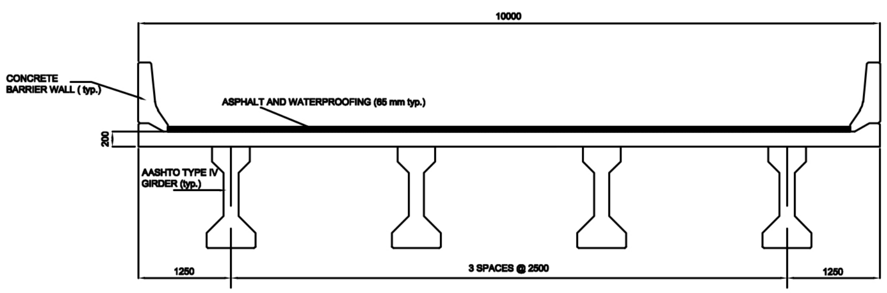


PLAN



ELEVATION



GENERAL NOTES

1. DESIGN CRITERIA
   - BRIDGE IS DESIGNED ACCORDING TO CSA S6-14 rev. 17
   - BRIDGE IS DESIGNED ACCORDING TO AASHTO LRFD 2014-17
   - BRIDGE IS DESIGNED ACCORDING TO CSA S6-66
2. CONCRETE STRENGTH AT 28 DAYS
   - PRECAST GIRDERS 40 MPa MIN
   - DECK 35 MPa MIN
   - REMAINDER REINFORCED CONCRETE 40 MPa MIN - 45 @ 56 DAYS
3. CLEAR COVER TO REINFORCING STEEL
   - DECK TOP
     60 mm +- 10 mm (CSA S6-14 rev. 17)
     65 mm +- 10 mm (AASHTO LRFD 2014-17)
     50 mm +- 10 mm (CSA S6-66)
   - DECK BOTTOM
     40 mm +- 10 mm (CSA S6-14 rev. 17)
     30 mm +- 10 mm (AASHTO LRFD 2014-17)
     25 mm +- 10 mm (CSA S6-66)
4. REINFORCING STEEL
   - REINFORCING STEEL MUST BE GRADE 400 UNLESS OTHERWISE SPECIFIED.
5. PRESTRESSING STRANDS
   - PRESTRESSING STRANDS MUST BE 7 WIRE LOW-RELAXATION TYPE AND MUST HAVE AN ULTIMATE TENSILE STRENGTH OF 1860 MPa.
6. DIMENSIONS
   - ALL DIMESIONS ARE IN MM UNLESS OTHERWISE NOTED.

Drawing title: GENERAL ARRANGEMENT	
Drawn by: Koral Eren	
Date: April 2020	
Drawing Number: 1	

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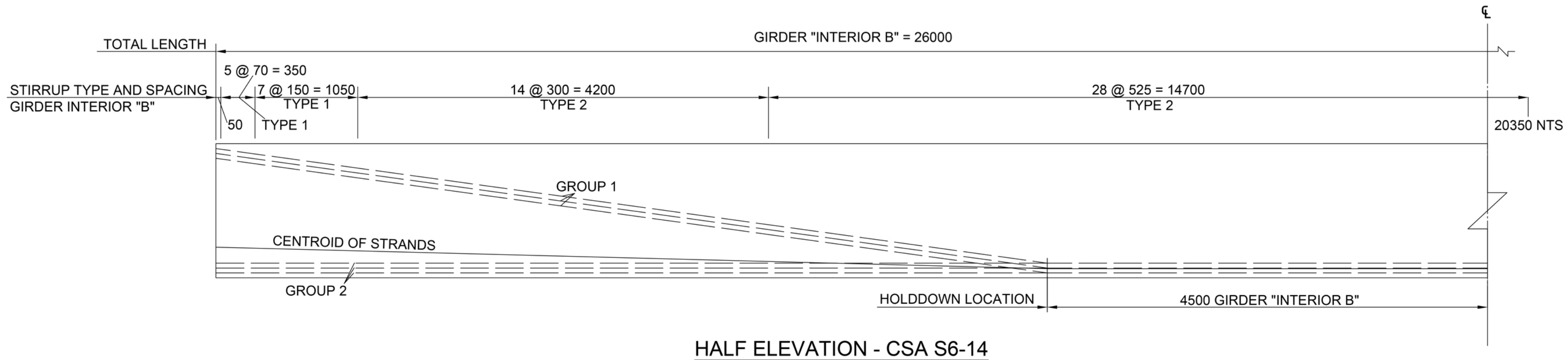
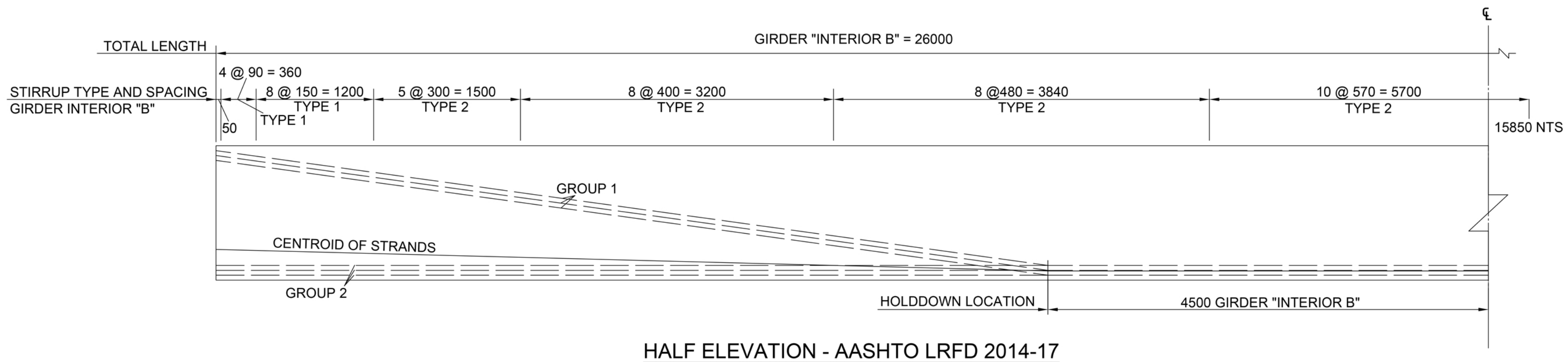
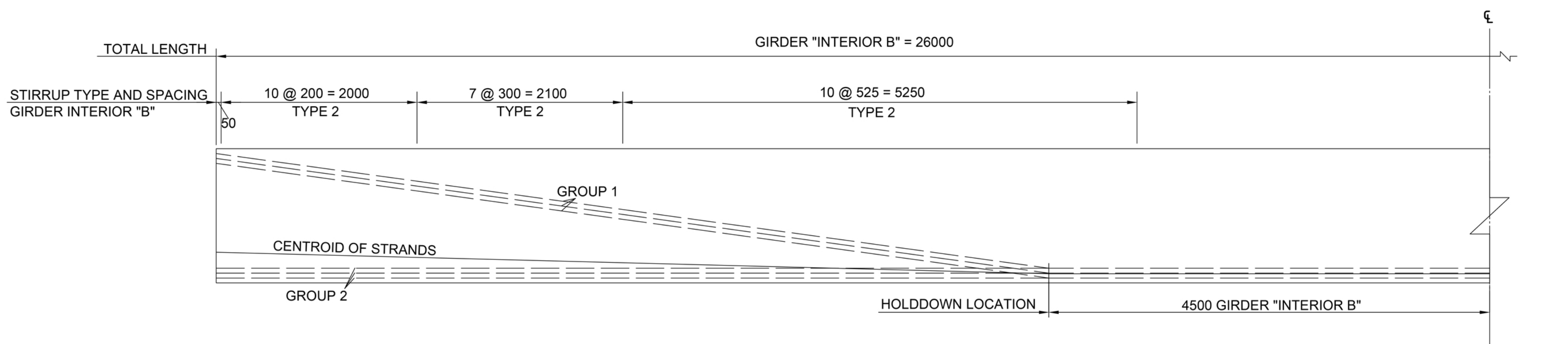


HALF ELEVATION - CSA S6-14



HALF ELEVATION - AASHTO LRFD 2014-17



HALF ELEVATION - CSA S6-66

NOTES

1. PRESTRESSING STEEL MUST BE LOW-RELAXATION 12.7 mm 7-wire GRADE 1860
2. JACKING FORCE PER STRAND = 136 kN (CSA S6-14 design value taken)
3. FORCE PER STRAND AFTER ALL LOSSES 108.5 kN (CSA S6-14 design value taken)
4. AT LEAST 16 HOURS MUST PASS BETWEEN JACKING AND TRANSFER
5. CONCRETE STRENGTH AT 28 DAYS = 40 MPa
6. CONCRETE STRENGTH AT TRANSFER = MIN 35 MPa
7. REINFORCING STEEL MUST BE IN ACCORDANCE WITH CSA S - G30.18
8. UNSHORED CONSTRUCTION IS ASSUMED DURING DESIGN PROCESS.

Drawing title: HALF ELEVATION - GIRDER
Drawn by: Koral Eren
Date: April 2020
Drawing Number: 2
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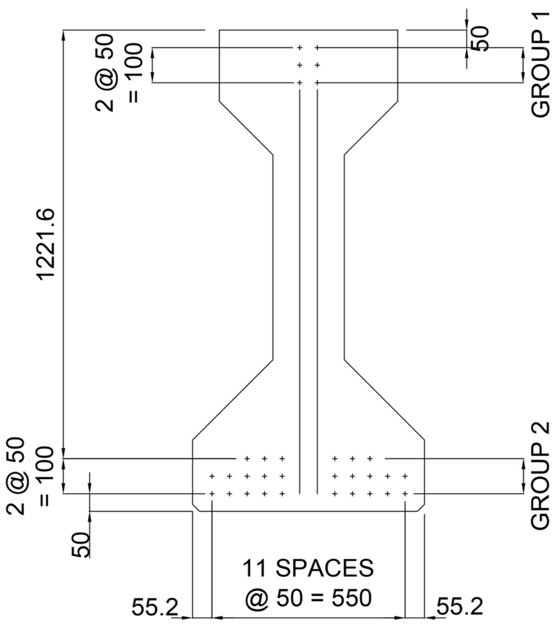
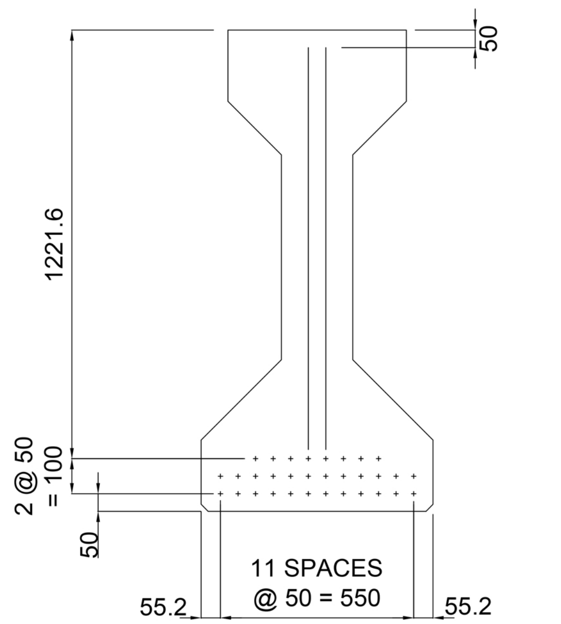


END OF GIRDER

32 STRANDS



CENTER OF GIRDER

STRAND PATTERN

NOTES

1. PRESTRESSING STEEL MUST BE LOW-RELAXATION 12.7 mm 7-wire GRADE 1860
2. JACKING FORCE PER STRAND = 136 kN (CSA S6-14 design value taken)
3. FORCE PER STRAND AFTER ALL LOSSES 108.5 kN (CSA S6-14 design value taken)
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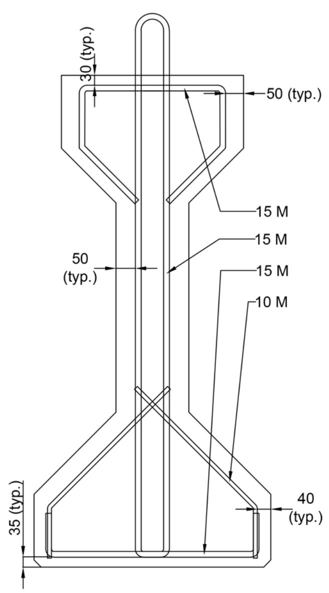
6. CONCRETE STRENGTH AT TRANSFER = MIN 35 MPa
7. REINFORCING STEEL MUST BE IN ACCORDANCE WITH CSA S - G30.18
8. UNSHORED CONSTRUCTION IS ASSUMED DURING DESIGN PROCESS.



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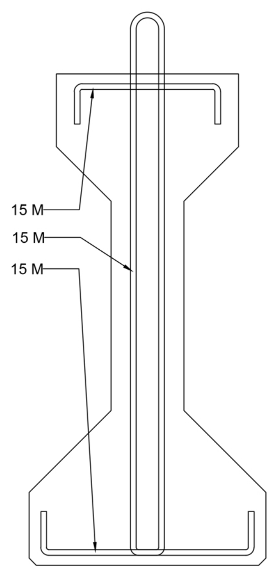

TYPE 2

STIRRUP DETAILS

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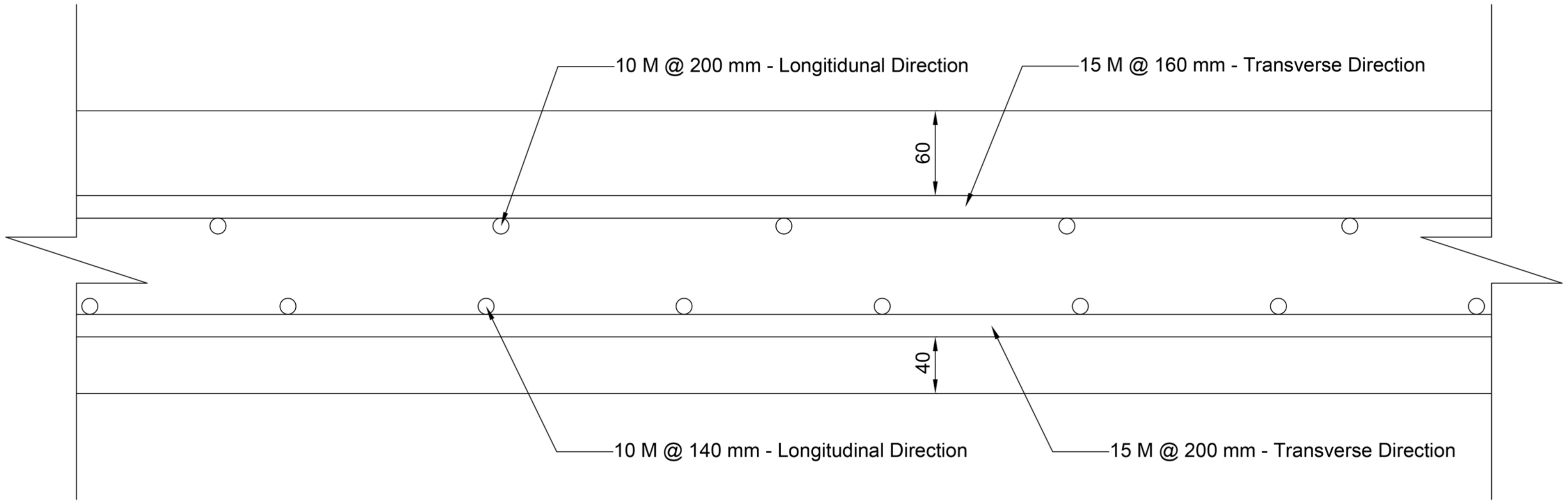
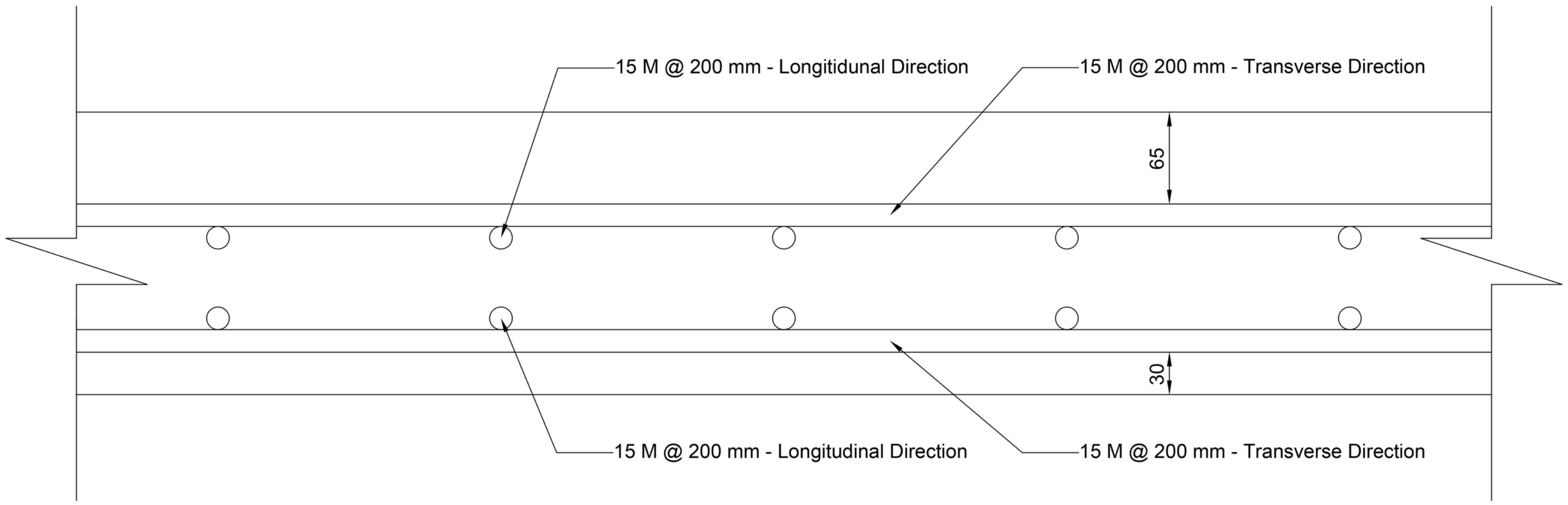
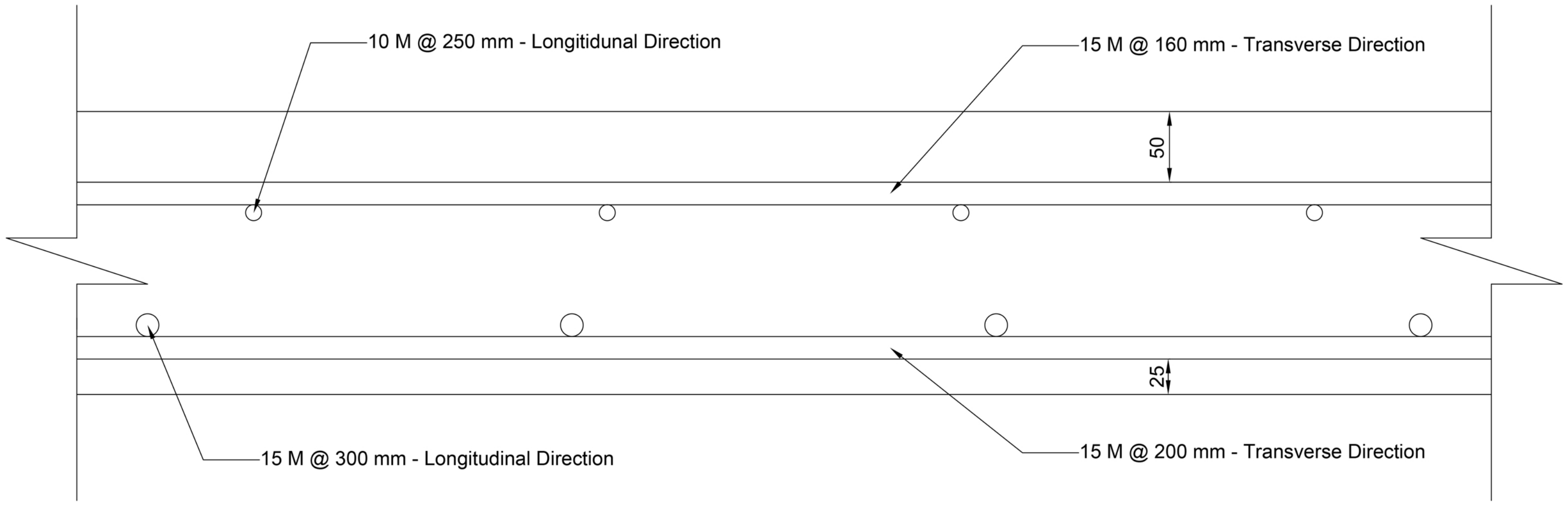
Drawn by: Koral Eren	
Date: April 2020	
Drawing Number: 3	



BRIDGE DECK CROSS-SECTION - CSA S6-14 rev.17



BRIDGE DECK CROSS-SECTION - AASHTO LRFD 2014-17



BRIDGE DECK CROSS-SECTION - CSA S6-66

Drawing title:
DECK REINFORCEMENT DETAIL

Drawn by:
Koral Eren

Date:
April 2020

Drawing Number:
4

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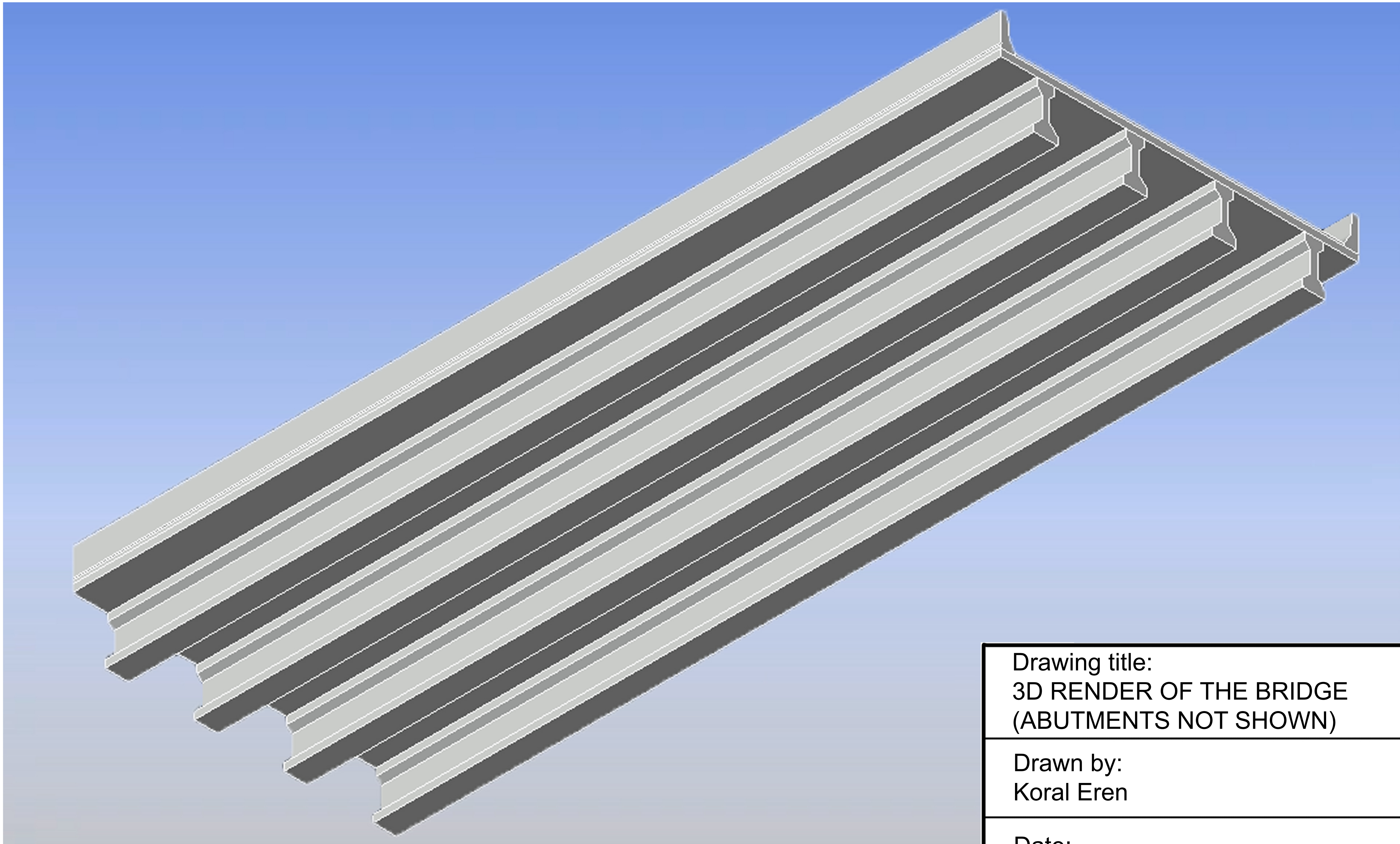

Drawing title:
3D RENDER OF THE BRIDGE
(ABUTMENTS NOT SHOWN)

Drawn by:
Koral Eren

Date:
April 2020

Drawing Number:
5

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