PRESTRESSED CONCRETE BRIDGE DESIGN

CIV 498 - Department of Civil and Mineral Engineering
Professor H. Abrishami, Ph. D., P. Eng

KORAL SELCUK EREN - 1001677296
BIKRAMJEET SINGH TOOR - 1002055955

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K & B DESIGN TEAM

Koral is a civil engineer from University of Toronto. He brings experience from his independent work and past work experience. He re-analysed the loads of a 25-Storey reinforced concrete building to evaluate its condition and to evaluate how well it will perform in an upcoming earthquake. This building was once the tallest residential building in Istanbul, Turkey. He also brings experience from Yapi Akademisi Engineering, a famous structural engineering company in Turkey known for large scale projects. In addition, he also worked on site as an intern site engineer for Emsiar Inc, the contractor for a 5-star Mariott Hotel in Izmir, Turkey.

Koral's personal website : www.koraleren.com

Bikram is a 1T9 + PEY, Bachelor of Applied Science in Civil Engineering candidate, finishing up his degree this year. He brings experience working at EllisDon Corporation on the VivaNext BRT for his PEY. In addition, he brings some project management experience from a four month summer work term, working for Discovery General Contracting (formerly CH2M), working as part of the Civil Field team for the West Yas Development in Abu Dhabi, U.A.E.

K & B

EXECUTIVE SUMMARY

This report was made for academic purposes in University of Toronto. Professor Homayoun Abrishami, Ph.D., P.Eng. was the supervisor of this report. The title of this report is “Design of Reinforced Concrete Bridges”. The report is divided into two parts: Part A and Part B. Part A, presents a complete qualitative description of the typical prestressed or reinforced concrete bridge design process. Part B provides an actual quantitative detailed design. The design is intended to be a replacement for the De La Concorde Overpass that collapsed in 2006. The replacement bridge is designed in 3 design codes: Current Canadian Highway Bridge Design Code, CSA S6-14 rev. 17; Canadian Highway Bridge Design Code of 1966, CSA S6-66; and American Bridge Design Specifications, AASHTO LRFD 2014-17.

The first chapter after the introduction of part A goes over three past bridge collapses and determines lessons learned from them. The following chapter focuses on different types of bridges as well as section types, geometric properties and materials used in bridge construction. In addition, for each bridge type discussed, some advantages and disadvantages are listed. Chapter 4 discusses three different bridge design codes: Current Canadian Highway Bridge Design Code, CSA S6-14 rev. 17; Canadian Highway Bridge Design Code of 1966, CSA S6-66; and American Bridge Design Specifications, AASHTO LRFD 2014-17. American equivalent of the current Canadian code is chosen to compare the current codes and the old code is chosen because De La Concorde Overpass was designed based on that code. Moreover, after discussing the codes, required designed inputs and several feasible conceptual designs are outlined. Then, software used in structural analysis are introduced with their main features. Later on, different constructability issues together with potential site problems that will affect the integrity of the structure are examined. Lastly, plant life management and aging management programs are discussed and presented at the end of part A of the report.

Part B of this report proposes a replacement design structure in place of the De La Concorde Overpass that collapsed in 2006. The first section is the problem statement in which, there is all the information that was initially available to us. Using that information, we designed a bridge based on three codes given above. Firstly, live and dead loads are determined. Then, these loads are distributed and factored to be used in the design. After, girders are designed and later on the slab is designed. Finally, a concrete mix is proposed for a durable design. During the process, hand calculations as well as computer programs like EXCEL and MATLAB are frequently used. Two commercially available structural analysis software are also used but only for result verification purposes.

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Overall Table of Contents

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Chapter 1 – Introduction to Part A

Part A of this report highlights the overall design process of reinforced and prestressed concrete bridges. It also contains information about past failures and lessons learned from them. Part A also contains essential information about the design process that is intended to be a reference to a design team or engineer during design process for a safe and durable design.

The part A of the report consists of eleven chapters with each chapter having a unique purpose. Chapter 2 focuses on past bridge collapses and lessons learned for the following three bridges: De la Concorde Overpass, Florida International University Pedestrian Bridge and Morandi Bridge. Chapter 3 presents different types of bridges as well as section types, geometric properties and materials used in bridge construction. In addition, for each bridge type discussed, some advantages and disadvantages are given. Chapter 4 introduces and compares three different bridge design codes: Current Canadian Highway Bridge Design Code, CSA S6-14 rev. 17; Canadian Highway Bridge Design Code of 1966, CSA S6-66; and American Bridge Design Specifications, AASHTO LRFD 2014-17. Each design code has differences in various ways. Some of the most notable differences are in how design loads are calculated and how prestressing force is calculated.

Chapter 5 discusses all the necessary design inputs such as client requirements, site location, site conditions, government regulations and construction process. All of them are discussed thoroughly in detail. Chapter 6 presents three conceptual designs for the collapsed portion of the De La Concorde overpass: Precast post-tensioned box girder, extradosed bridge and New England Bulb Tee (NEBT) girder. Chapter 7 discusses three different structural analysis software that helps adequately designing the bridge structure in detail: Risa, S-Frame and CSI Bridge. Chapter 8 and 9 discusses the structural and durability design with respect to the three design codes listed above. Chapter 10 discusses different construction issues such as: construction safety, scheduling, on site quality control and budget issues. Lastly, in chapter 11, plant life management (PLiM) and aging management program (AMP) are discussed in order to explain the strength, serviceability and durability requirements of a structure throughout its life span.

To summarize, this section of the report provides a detailed overview of the design process of a typical reinforced or prestressed concrete bridge. It also provides additional considerations that a design team must consider in order to design a successful structure.

Chapter 2 – Bridge Failures and Lessons Learned

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2.1 Introduction

Bridge failures and collapses can have catastrophic consequences such as loss of human life and serious property damage. It is very important that all the people involved in bridge construction to follow strictly the current design codes and standards, construct according to the design requirements, maintain and inspect the bridge on a regular basis to keep the bridge in a good shape which will result in prevention of future collapses. In order to prevent future collapses, it is important to study and review the past collapses. This will us learn from our mistakes and prevent same things happening again by designing accordingly.

This chapter presents 3 case studies of past bridge failures and lessons learned from each failure. The first case study is De La Concorde Overpass, a bridge that was crossing the highway 19 located at north of Montreal. The bridge collapsed due to many reasons. The second one is Florida International University Pedestrian Bridge, collapsed due to design mistakes and the last one is Morandi Bridge located in Genoa, Italy. It was a viaduct crossing the Polcavera river. It also collapsed due to poor design.

2.2 De la Concorde Overpass Collapse

2.2.1 Background of the Bridge

The overpass on the Boulevard de la Concorde was a bridge located near Montreal crossing over Highway (19) Papineau. Its coordinates were 45°35′0.6″N 73°40′30.94″W. It was built in 1970 with a life expectancy of 70 years but because of the unfortunate series of events the bride collapsed after 36 years in 2006. A new steel - concrete bridge is constructed in place of the collapsed bridge that has a concrete pier and abutments and steel girder and secondary beams.

The 1970 overpass was designed to not block the visibility of the highway underneath, make future excavation easier and pass the opening with near constant depth. It was an elegant, innovative structure at its time but of course it had flaws that will be later discussed.[1][2]

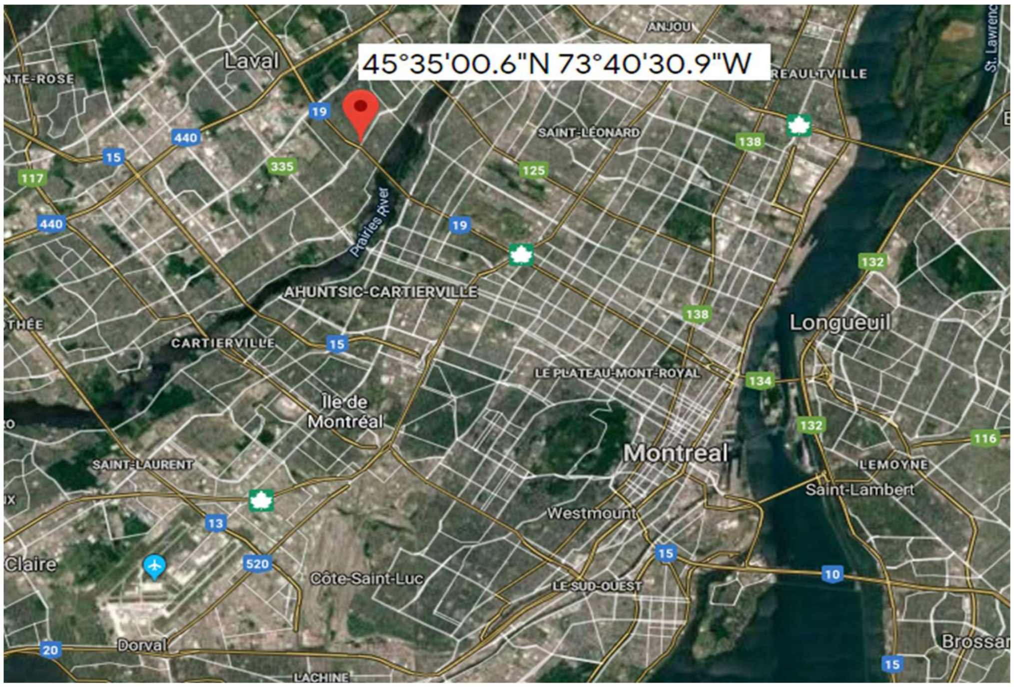

Figure 2.2.1.1 - Location of the collapsed overpass

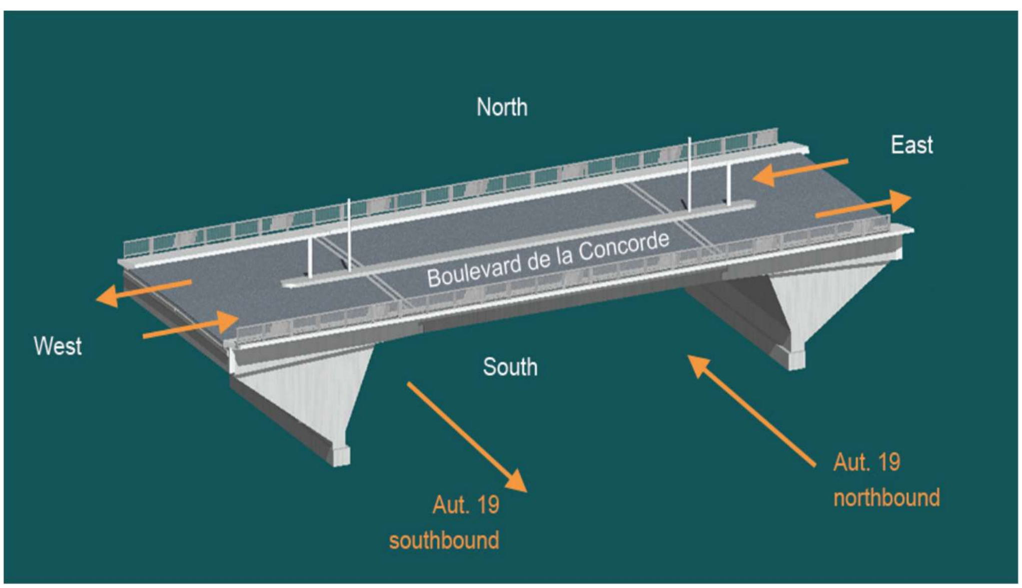

Figure 2.2.1.2 - General Perspective of the collapsed overpass [1]

Figure 2.2.1.3 - New modern overpass constructed in place of 1970 overpass

2.2.2 Bridge Type and Structure Detail

The overpass of 1970 was a pre-tensioned and post-tensioned concrete bridge. The bridge had two abutments. Ten precast box girders on each side were supporting the deck loading. Transverse post-tensioning was applied after the girders got placed in place. This allowed the overpass to cross the 6 (3-3) lane highway without a pier. It had a total unsupported length of 35.35 m (Not a huge unsupported length with modern materials available today.).
This kind of span was impressive at the time. It had 3.96 m cantilever portions at both sides. Used concretes specified cylindrical compressive strength for abutments was around 30 MPa (f'_c = 30). Nowadays we use minimum 35 MPa concrete for bridges.

The cantilever section of the overpass designed and what was actually built had slight differences, the images below shows the differences visually [1]:

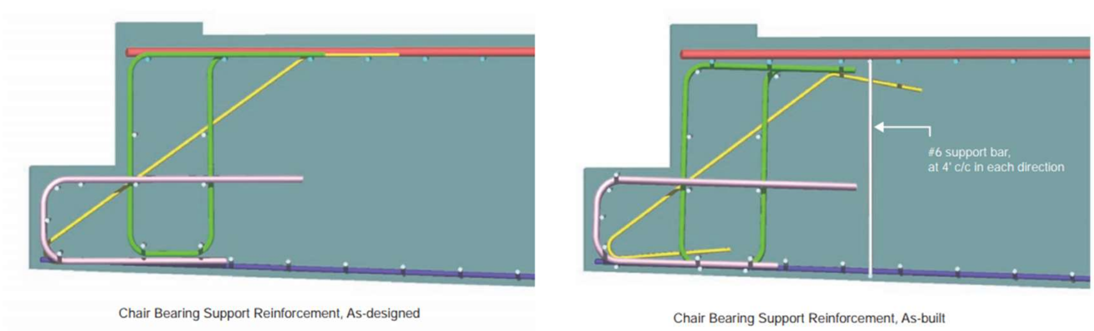

Figure 2.2.2.1 - Chair Bearing Support Reinforcement

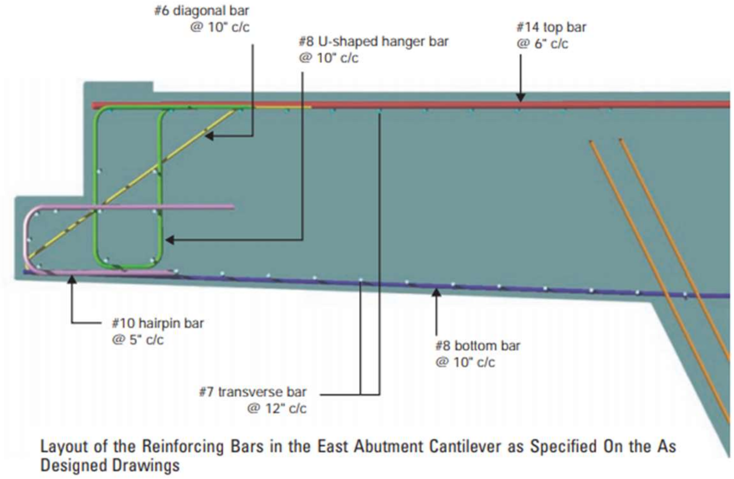

Figure 2.2.2.2 - Layout of the Reinforcing Bars in the East Abutment Cantilever

Differences in the structure as built at the cantilever section:

The number 8 bars (green) and diagonal reinforcing bars (yellow) were constructed under the no 14 bars (red). The no 8 bars were designed to transfer the load to the no 14 bars and the diagonal reinforcement bars were installed for crack prevention, interception of the tensile region formed and also transfer the loads to the top. Contractor also added vertical no 6 bars and some horizontal bars to support the upper bars.

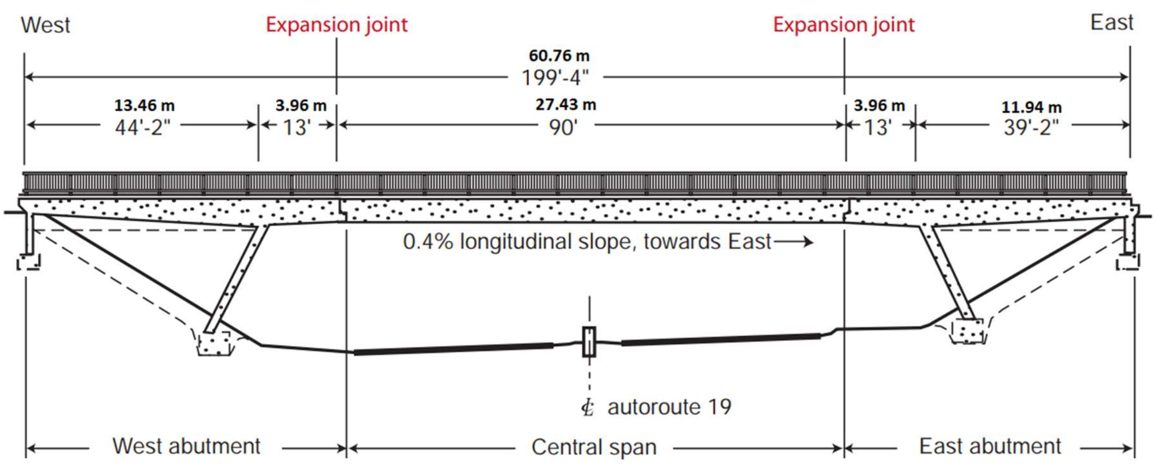

Figure 2.2.2.3 - Elevation View of the de la Concorde Overpass from Autoroute (Highway) 19 [3]

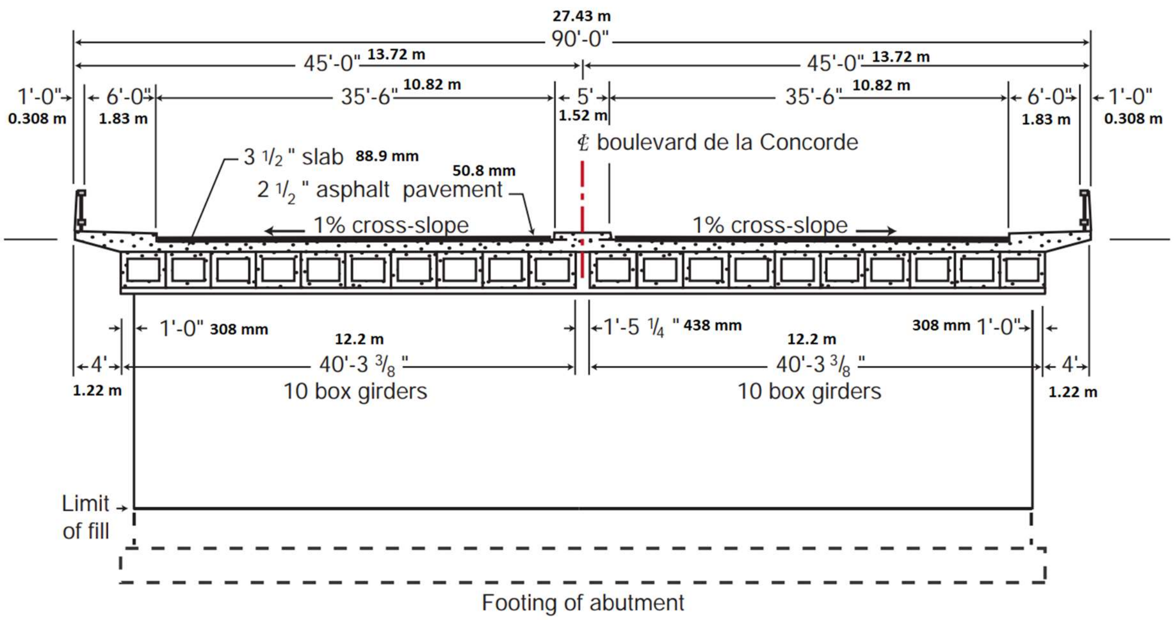

Figure 2.2.2.4 - Cross-section of the de la Concorde Overpass from Autoroute (Highway) 19 [3]

2.2.3 Description and Reasons for Failure

At the time the bridge was designed, shear reinforcement wasn't required for thick slabs. The cantilever portion of the bridge developed cracks due to a concentrated shear plane at the top of the longitudinal bars. The cracks happened due to deterioration of the concrete over time with salts and freeze thaw cycles. The concrete used also wasn't suitable for freeze thaw cycles. The expansion joints were impossible to examine. Once it got damaged, salty water went in and deteriorated the concrete.

During a repair work done in 1992, waterproofing was removed. As a result of this, salty water went in already formed cracks increasing deterioration in concrete.

Construction wasn't the same as the drawings. This created a weak plane where cracks occurred.

Low quality concrete was used for the structure. Water cement ratio was incorrect and therefore the strength of the concrete wasn't suitable for the structure.

2.2.4 Lessons Learned

-The presence of shear reinforcement can prevent shear cracks and add ductility. It cannot be relied solely on the shear resistance provided by the concrete.

-Design codes must be made more carefully and must include more detail.
-Inspection and manuals are important and they need more attention.
-Keeping a record of the structures aging in a database is recommended.
-Surveillance of the work done by designers and contractors must be increased.
-The damages observed in structures must be more deeply analyzed.

2.3 Florida International University (FIU) Pedestrian Bridge Collapse

2.3.1 Background of the Bridge

The FIU pedestrian bridge was located just west of Tamiami Trail and Southwest 109th Avenue in West Miami [1]. The bridge was planned to connect the university to the housing neighborhoods in the city of Sweetwater [1]. The bridge was designed to improve pedestrian safety for a busy area that witnessed the death of a student that got struck by a car while crossing the busy intersection [2]. It was a $14.2 million project funded by the US department of transportation and its construction began in spring 2017 and was expected to finish early 2019 [3]. Apart from the bridge, the project also included the construction of new sidewalks and a plaza. The construction of the bridge was a joint venture between Munilla Construction Management and FIGG Bridge Engineers [4]. The bridge was constructed along the roadway in a construction yard and upon completion, was shifted in place by two self-propelled modular transporters. This type of construction is called accelerated bridge construction method which causes minimal disruption to ongoing traffic and was overlooked by the FIU who are known for their bridge works [6].

Figure 2.3.1.1 - Location of FIU Pedestrian Bridge [1]

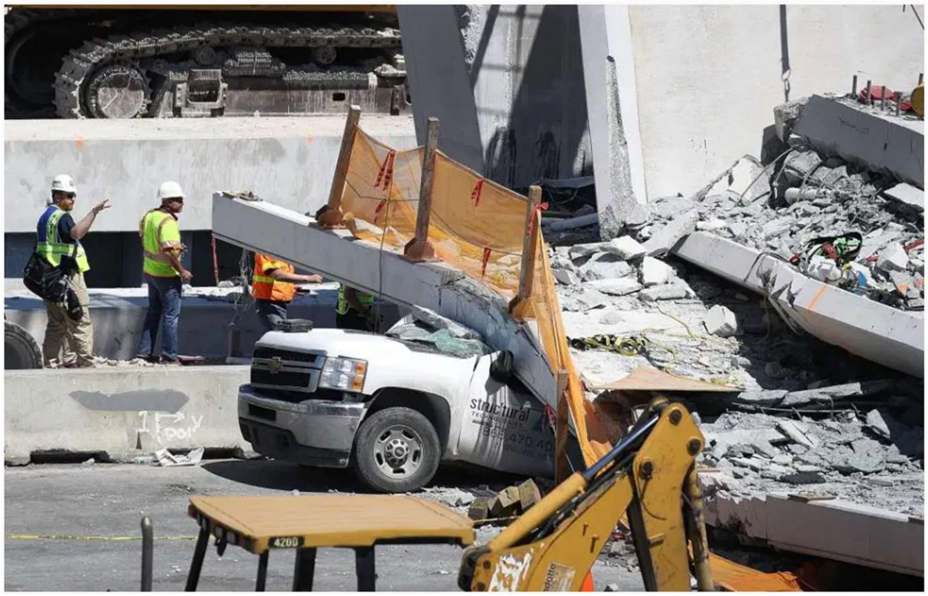

Figure 2.3.1.2 - Ongoing Investigation after Bridge Collapse [4]

2.3.2 Bridge Type and Structure Detail

The post-tensioned concrete bridge was 98 m long aiming to provide pedestrians a safe pathway to cross both a busy road and a water canal [5]. The length of the bridge over the roadway was 54m and 30m over the water canal [5]. The bridge had elevators and a staircase at both ends which was 14m in length [5]. The roadway underneath the bridge has 4 EB lanes, 3 WB lanes and a turning lane [6]. The bridge had a life expectancy of about 100 years and was designed to withstand a Category 5 Hurricane [7]. The bridge appeared to be a unique cable stayed bridge with a center tower and high cables. But they were essentially for aesthetics and the bridge was a truss bridge. The span section of the bridge was unsupported. The bridge span along the centerline consisted of triangular shaped concrete diagonal struts at different angles in order to align with cabled steel pipes [8]. The deck and the canopy of the bridge were the bottom and top flange of the wide I-beam respectively, both made out of concrete. The deck carried the tension loads and the canopy carried the compressive loads of the structure. The angles of the diagonal struts determined whether they carried tensile or compressive loads.

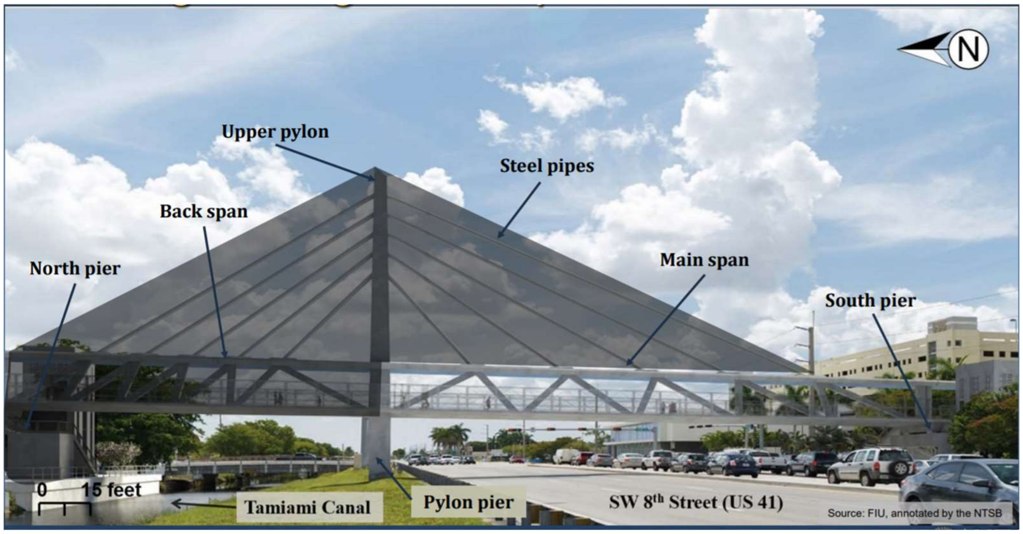

Figure 2.3.2.1 - Bridge Design Concept [5]

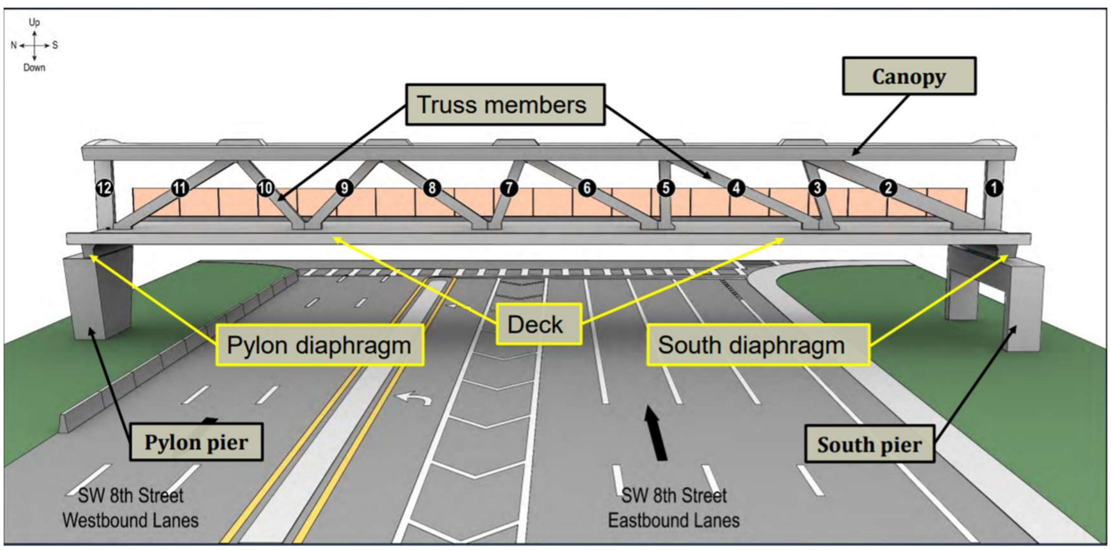

Figure 2.3.2.2 - Pedestrian Bridge Main Span [5]

2.3.3 Description and Reasons for Failure

The bridge that collapsed during its construction stage weighed 950 tons [9]. The diagonal truss members were numbered from 1 to 12 from south to north end of the bridge as seen in figure 2.4. Normally, a truss bridge would have two rows of truss members, so that if one face of the bridge fails, the whole bridge won't collapse. In this case, the bridge design was non-redundant, which means if one of the truss members fails, the whole bridge will collapse [9]. Apart from this, there were two main theories that stood out after the investigation process.

-Redundancy: Prestressed concrete truss that was used was a determinate truss. If a plastic hinge is formed in one of the members of a determinate truss by a crack, it completely collapses. However, if the truss is indeterminate, the loads will go to another member after the hinge is formed so instead of immediate failure, the structure fails gradually or survives. We prefer the second one as we don't want immediate failure.

-Construction Joint Inadequate Surface: If two concrete is cast at different times, because of the strength difference between the two, interface shear calculation should be done according to AASHTO, the design code the bridge was designed according to. The interface shear calculation requires the estimation of the shear friction. AASHTO requires the interface between the two different pours to be roughened for increased shear transfer. The company FIGG did not design a rough surface. Even the surface was roughened, due to the lack of sufficient shear reinforcement, the demand was above capacity [10]. FIGG decided to add additional tension to the cables in beam 11 after seeing growing cracks. This created more shear at the connection which caused failure right after [10].

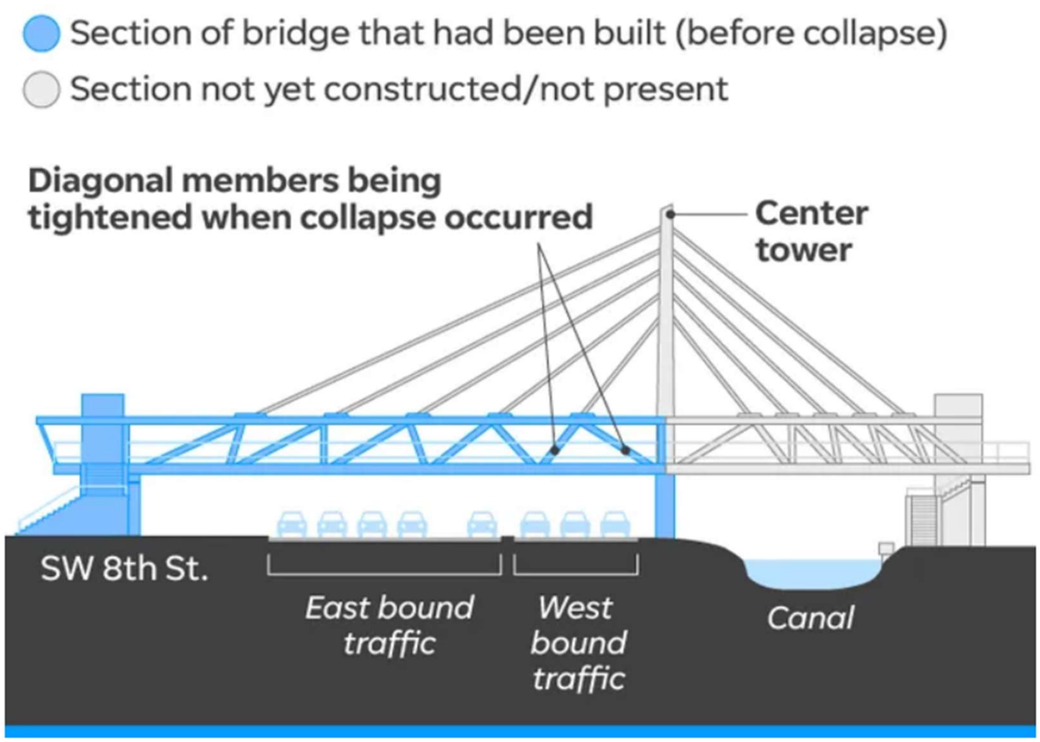

Figure 2.3.3.1 - Profile of bridge before it collapsed [7]

2.3.4 Lessons Learned
Below are the lessons learned from the collapse of the FIU pedestrian bridge:

-A qualified independent firm should be hired at all times to conduct review of bridge plan and design work.
-Work must be suspended when a damage is observed, and the damage should be thoroughly analyzed before proceeding with construction.
-Code procedures of design must be followed carefully.
-Safety must be prioritized over aesthethics.

2.4 Morandi Bridge

2.4.1 Background of the Bridge
Morandi Bridge was a 50 year old bridge over the river Polcevera connecting Sampierdarena and Cornigliano districts in Genoa, Italy. The construction of the bridge took four years to complete between 1963 and 1967. It was one of the largest concrete bridges in the 1960s. It was also a part of a critical roadway connecting Italy to France. The bridge was 1182 m long crossing over a valley, river, railway track, some houses and some factories [12]. A 210 m long portion of the bridge collapsed during a severe rainstorm on August 14, 2018 killing 43 people [13]. The whole bridge was fully demolished in June 2019.

2.4.2 Bridge Type and Structure Detail
Morandi Bridge was a cable stayed bridge with a total length of 1102 m [14][15]. The piers, pylons, deck, stays were all made out of prestressed concrete. The longest span of the bridge was approximately 210 m [14]. Cable stayed spans were only supported by 2 stays in each side. This was quite unusual from a regular cable stayed bridge design [14]. The stays were made out of steel cables with a layer of prestressed concrete shells on top of it. The concrete piers supporting the arches of the bridge were 90 m in height [14][15].

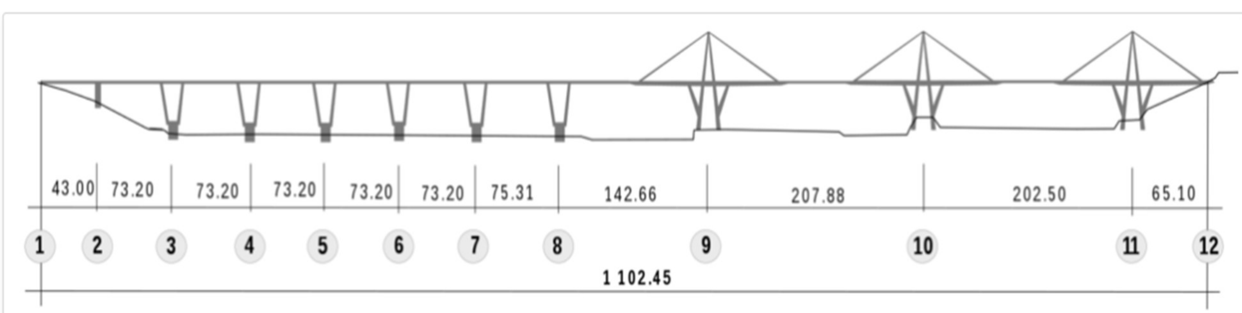

Figure 2.4.2.1 - Perspective Drawing of Morandi Bridge [16]

Portion of the bridge between 1 and 8 in figure 2.4.2.1 was the trestle system portion (A trestle system consists of short spans which are supported by frames like tripods.). In this portion, piers had 2 reinforced concrete inclined beam-columns, attached to each other by a double cantilever girder at the top and supported by a raft foundation at the bottom which lies on drilled piles [16].

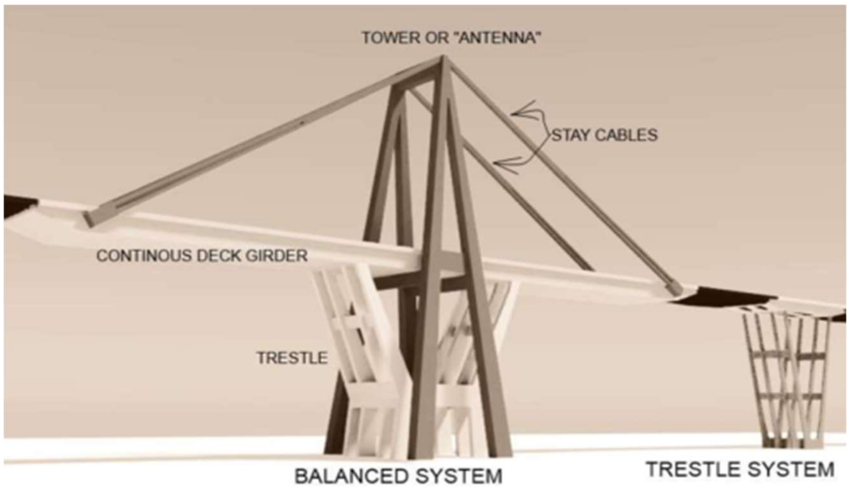

Figure 2.4.2.2 - Trestle and Balanced System [16]

Portion of the bridge between 8 and 12 in figure 2.4.2.1 was the balanced system portion. The portion that collapsed extended from the end of pier 8 to the point of cable connection of pier 10. There were 3 balanced systems in total and each balanced system contained a ribbed foundation made out of reinforced concrete sitting on drilled piles, a reinforced concrete trestle made up of 4 side by side H elements which provided elastic support to the continuous deck girder [16] and a suspension tower, also called the antenna, that provided the main frame which was made up of 4 inclined legs [16].

Prestressed concrete continuous deck girder consisted of a top slab, a bottom slab and 6 longitudinal ribs banking on the trestle [16].

2.4.3 Description and Reasons for Failure

The collapse of the portion extended from the end of pier 8 to the point of cable connection of pier 10 took place on August 14, 2018. Below were the reasons of failure:

-The corrosion of the prestressed stay cables due to salty air coming from Mediterranean winds and chemicals coming from the steel mill close by is thought to be the main reason for the collapse. According to the measurements done, only 10 MPa of prestressing is left at one of the corroded cables coming from stack 9 [16]. This resulted in slipping of one of the cables from its tie rod. Due to the way this section of the bridge is designed, it couldn't carry the demand once a cable failed and therefore it collapsed.

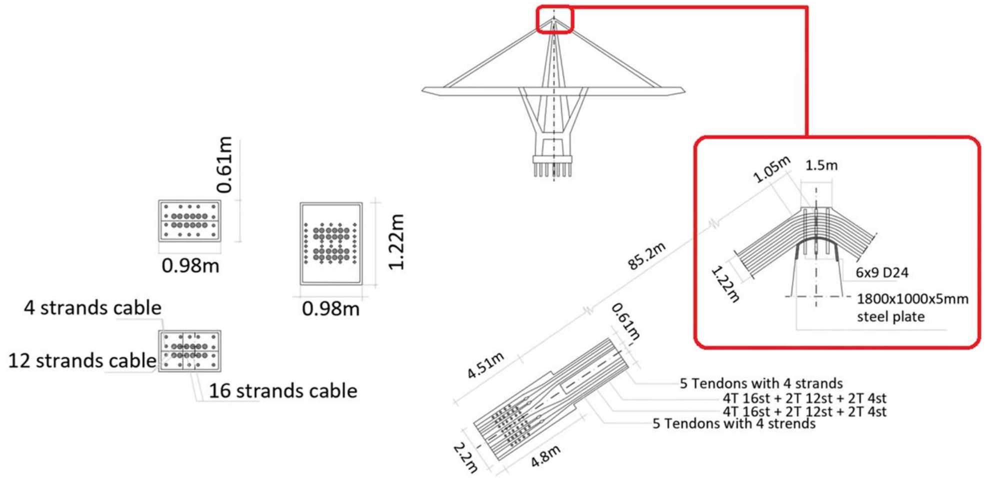

Figure 2.4.3.1 - View of the stay cable system [17]

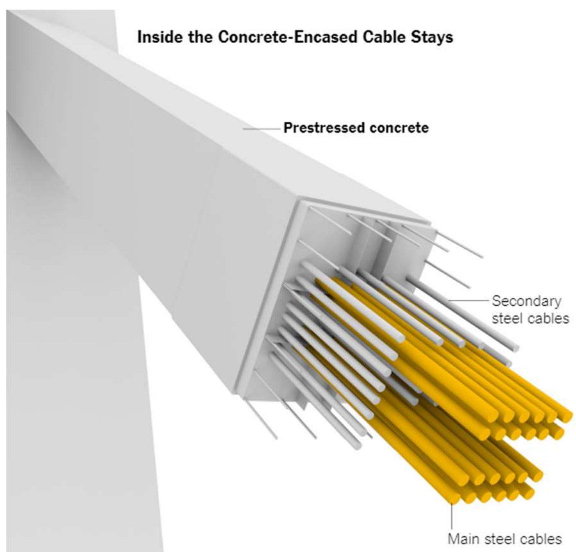

Figure 2.4.3.2 - 3D View of the stay cable system [18]

-There was a major error in the design calculation. The inaccurate calculation about how prestressing tendons will perform over time with usual load cycles was considered one of the key reasons for failure. The loss of stress due to this effect is called creep loss and it may be a significant loss between other losses.

-Bad weather (winds and intense rain) on the day of the collapse also played a role in the collapse. Rain added additional load on the slab and winds created vibrations. Vibration is a big problem for corroded cables.

Figure 2.4.3.3 - Photo of corroded steel and prestressing steel of the collapsed section [17]

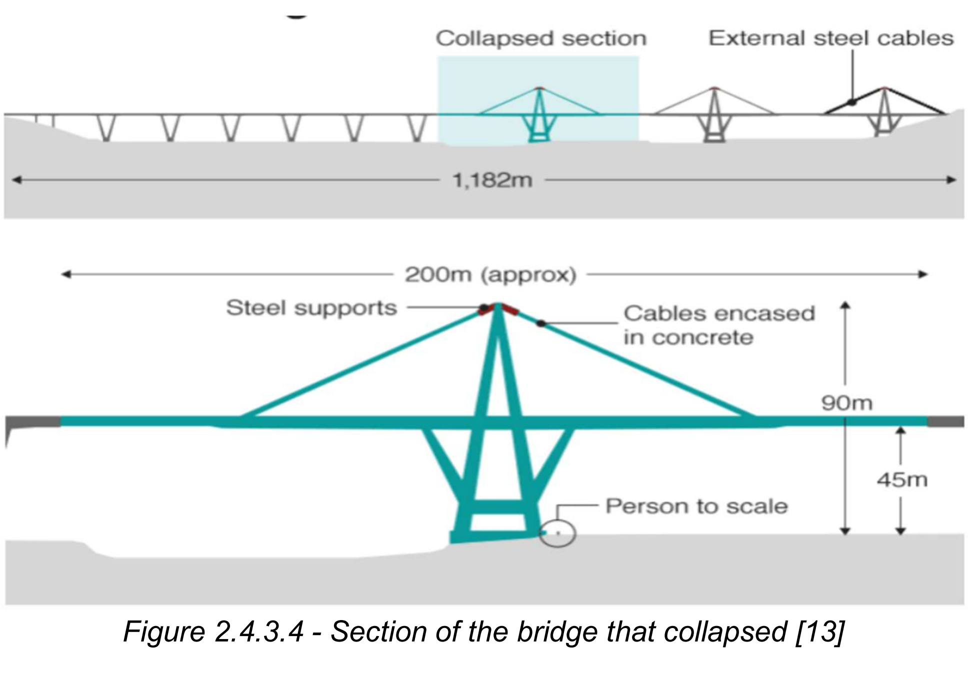

Figure 2.4.3.4 - Section of the bridge that collapsed [13]

2.4.4 Lessons Learned

-Cover requirements: In exposed areas, if concrete is going to be used in a structural element like the cable stays in Morandi Bridge, adequate cover must be designed to prevent similar disasters.

-Redundancy – More cable stays: Designs must have a "just in case" option. For cable stayed bridges, use of more cable stays is required. Morandi Bridge only had 2 in each side[19]. Adding more cable stays helps prevent the domino effect and therefore adds structural redundancy [19].

-Age & Maintenance: Bridges built in the 1960s are reaching their useful life and regular maintenance and bridge enhancements need to be done. If not done properly, this may lead to stressing the bridge beyond its limits causing total or partial collapse.

-Privatization & Inspection: The bridge was maintained and supervised by a private company and not by a municipal government agency. Regular inspection needs to be done by a third party or by the municipal government if the owners are modifying the bridge according to the new guidelines and standards, considering the current demand.

2.5 Conclusion

Failure of bridges have occurred from the time we started constructing bridges at different locations and for different reasons. It is difficult to generalize the causes of failure as each bridge has different characteristics and properties. From the above three discussed case studies, we can note a few common causes of bridge failure: design errors, wrong construction practices, failure to inspect and maintain the bridge. Lessons learned provided for each case study helps prevent future bridge collapses that are similar and avoidable.

2.6 References

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Chapter 3 – Types of Reinforced and Prestressed Concrete Bridges

Table of Contents

3.1 Introduction

There are many different bridges with different geometric and material properties. Many factors such as geographical constraints, financial constraints and time constraints affect the choice of these properties. Almost all of the bridges have common elements like a slab and a foundation. The designer needs to acknowledge different components and types of bridges to be able to choose an appropriate bridge for his/her design. This chapter introduces different types of bridges, the geometry of their main structural elements and materials used in those bridges in detail.

3.2 Material Types

Bridges can be classified based on different types of materials used for their construction. This section introduces bridges built with masonry, timber, steel and concrete. Reinforced concrete and prestressed concrete concepts are explained together with their differences. Each material has different characteristics such as strength, durability, workability, weight and resistance against corrosion. Each material also varies in their structure, texture and color. In order to construct a bridge with the correct material type, designer must know their properties.

3.2.1 Masonry

Masonry is a type of construction material used to build structures using bricks or stones. Mortar is used to interlock the individual brick or stone units. Masonry bridges are usually designed in an arched shape with massive supports as demonstrated in Figure 3.2.1.1. The load bearing capacity of the bridge is made of bricks or stone in an arched shape. Masonry bridges are one of the oldest types of bridges and they are still in existence because of their high ductility and aesthetics [1]. Masonry bridges usually have a span ranging between 70 m to 120 m [2]. The span length is not large mainly due to Masonries low bearing capacity and high labor intensity required to put it together [3]. Masonry bridges require little to no maintenance compared to other bridge types [1]. Nowadays, masonry is used in the construction of low-rise buildings and homes.

Figure 3.2.1.1 - Roman stone arch bridge in Spain [1]

3.2.2 Timber

Timber is one of the oldest materials used for bridge construction. The bridges started getting constructed with steel and concrete in the 20th century [4]. Timber is still used to build bridges that are of short and medium spans as it is abundant, versatile and easily obtainable everywhere [4]. Timber has great strength; it is light weight and it has energy absorbing properties which makes it ideal for bridge construction. Timber is able to support short term overloads without any adverse effects [4]. Timber bridge construction is suitable for all weather conditions as it doesn't get affected by continuous freezing and thawing and it copes well with de-icing agents. Also, life cycle cost of timber provides gives it an advantage over other bridge materials [4]. The timber construction process doesn't require any special equipment, or any type of skilled labor [4]. The duration of timber bridge construction is fast compared to other materials due to its high-speed erection and installation times [5]. It provides a very natural appearance and beautiful structures with innumerable environmental benefits [5]. The members used in timber construction are also available in many sizes like other materials [5]. However, timber is prone to decay due to insect attack and it is also sensitive to moisture. The bridge deterioration can be prevented by using preservative chemicals that enhance the bridge life by 50 years or more. Timber is not good for long spans due to deflections it experiences under load [4].

Figure 3.2.1.1 – Timber Bridge in Scandinavian [5]

3.2.3 Steel

Steel is a combination of iron and carbon and sometimes a mix of other alloying elements [6]. Steel is very well known for its versatility, cost effectiveness, longevity and sustainability, making it a crucial component in constructing different types and sizes of bridges [7]. It is not just an attractive option, but it is also stronger, safer and faster to build. It doesn't require maintenance often and it has high flexibility, which helps it resist well against natural disasters [7]. The high strength to weight ratio, minimizes the weight of the structure and reduces the cost of the substructure that is used to carry the steel structure [7]. This comes handy when the surface is unstable and uneven like riverbeds and canyons [7]. Steel can be molded into any shape by bending and twisting giving it an aesthetically appealing look. Steel has the capability to carry all kinds of loads such as shear, tension and compression making it ideal for different types of bridges [7]. It is used in beam bridges, box girder bridges, truss bridges, arch bridges, cable stayed bridges and suspension bridges. Steel comes in different grades, sizes and shapes. Most of the steel that is used for bridge construction is prefabricated in a controlled environment. Therefore, it is consistent, and it has high quality. The different processes for bridge steelwork are molding, cutting, drilling, assembling and welding [7].

The few disadvantages of steel are:

- They have high upfront cost and high maintenance cost [8]
- They are not good in resisting fire with comparison to concrete as steel buckles with heat [8].
- The likelihood of brittle fracture increases as steel starts to lose its ductility strength. Cold steel may become very brittle. [8]

Table 3.2.3.1 – Benefits of Steel

Economic Benefits	Environmental Benefits
Rapid construction decreases interruption in traffic and business nearby [7]	Reduces environmental impact as components of steel can be reused and recycled [7]
The components of steel require less maintenance, therefore don't have to be replaced frequently [7]	Steel can be used to construct longer spans reducing the impact on the habitat below the bridge [7]
Lightweight of steel helps lower the construction cost due to the less costly machinery that is required to lift the steel [7]	Less energy consumption because of steel being lightweight [7]

Figure 3.2.3.1 – Galvanized Steel Suspension Bridge in China [9]

3.2.4 Concrete

<u>Constituent materials:</u>

Concrete is a material generally composed of Portland Cement, water, aggregates (general name for sand, gravel, crushed stone or slag used in concrete), supplementary cementing materials and admixtures. The first four of these ingredients are the main ones. For many applications, using just the first four will suffice. Portland cement and water when mixed together forms a chemical reaction that creates bonds in between and becomes a hard material. The advantage of concrete is that before it hardens it is a liquid like mixture. This allows it to be formed into different shapes. Generating custom shapes is way harder with other types of non-composite materials. Cement is an expensive material and concrete composed with just cement and water will be expensive and unnecessary for most applications. We add nature abundant aggregates to reduce the cost and sometimes to achieve the required properties for the application. Water cement ratio and admixtures used inside the concrete determine the strength of the concrete.

Figure 3.2.4.1 – (Left) Fresh Concrete and (Right) Colored Concrete due to admixtures (Photo from Prof Ahmet Topcu - Eskisehir University Lecture Notes)

Figure 3.2.4.2 – (Left): C30 Concrete: 310 kg Cement, 1.2% Grace Ex 3282, 0.03% Daravair1000 (Air-entraining admixture), 7.2% Air and (Right): C40 Concrete: 370 kg Cement, 1.1% Grace Ex 3282, 0.03% Daravair1000 (Air-entraining admixture), 7.2% Air

Properties of Hardened Concrete:

Some of the values and formulas given below are used for building construction only and
might differ in bridge construction. They are given as a general idea of material

The hardened concrete is much better in compression then it is in tension. Therefore, we want to benefit from the compressive properties of the concrete.

a) Tensile behavior of concrete:

Concrete has some but little tensile strength. According to CSA A23.3-04 concrete has a trustable tensile strength of $\boldsymbol{0.6 \times f'_c{}^{0.5}}$ assuming normal density and normal strength concrete up to 50 MPa (C50 concrete European). This formula is empirical and based on tests done on various samples. In reality, the behavior is very similar in tension to this.

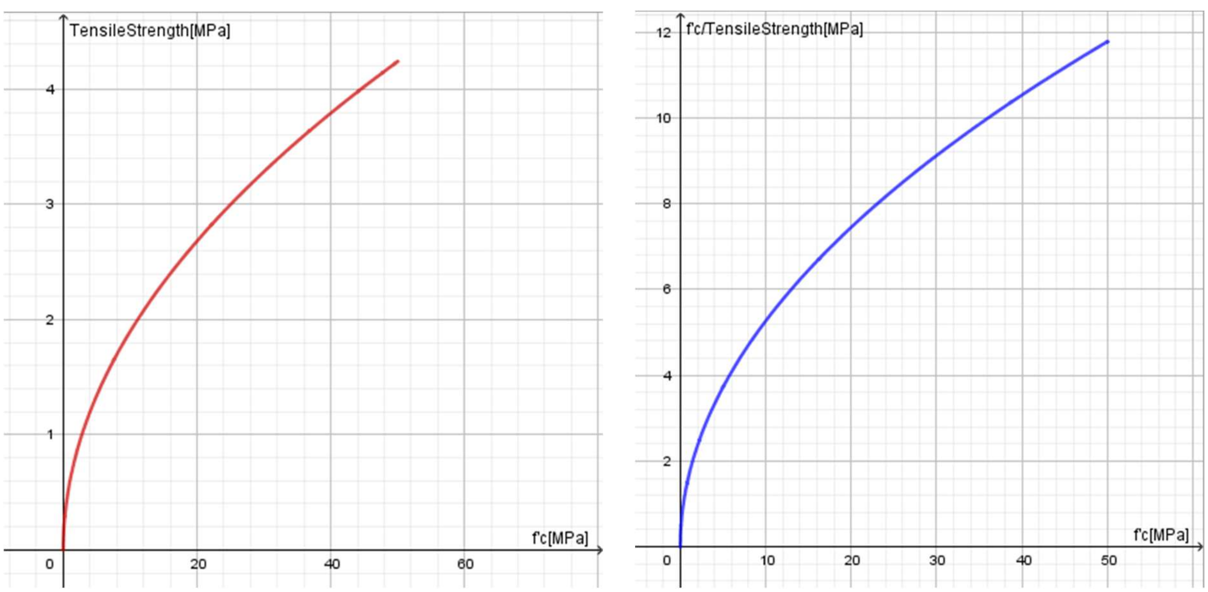


Figure 3.2.4.3 – (Left) Tensile strength of concrete versus its maximum cylindrical compressive strength and (Right) Concrete maximum cylindrical compressive strength / tensile strength of concrete

b) Compressive behavior of concrete:

Based on various tests done on different samples of concrete, concrete shows a parabolic (non-linear) behavior until its peak compressive strength. This theory was proposed by Hognestad in 1951. To simplify some stuff, up to 40% (Turkish BC) to 45% (CSA) of f'c of the stress-strain curve of concrete is assumed to be linear (although concrete is a highly non-linear material). Assuming linear material allows us to use Hooke's spring Law (***F = k x x***) for design and calculation purposes up to a point. This eliminates the need to take numerical integrals or iterations in case if a finite element program is used (Updating stiffness matrix each time is not required for linear materials therefore saves computational time). Modulus of Elasticity of concrete is taken as the secant modulus (CSA) or tangent modulus (Turkish BC, Eurocode) if the concrete stress-strain curve is known. There are also empirical formulas to determine the modulus of elasticity (Ec) of concrete:

Turkish BC: ***12680 + 460 x (f'$_c$)***
CSA: ***4500 x sqrt(f'$_c$) or (3000 x sqrt(f'$_c$) + 6900) x (DensityOfConcrete/2300)$^{1.5}$***

DensityOfConcrete: [kg/m^3]
f'$_c$: [MPa]

The strain at the point where ultimate compressive stress happens is ***2 x f'$_c$/E$_c$*** or ***1.8 x f'$_c$/E$_c$*** or simply ***0.002*** depending on the code used.

Hognestad's Parabola:

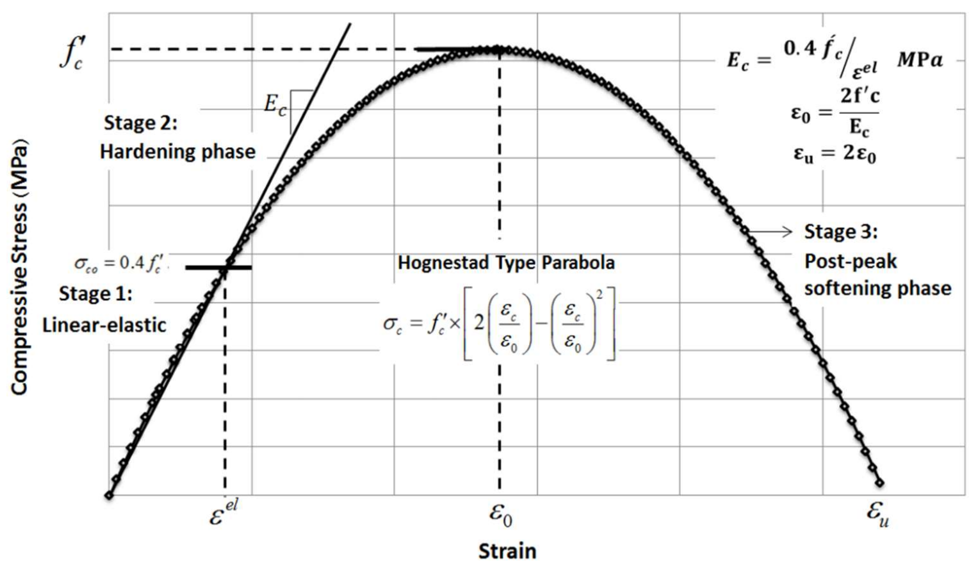
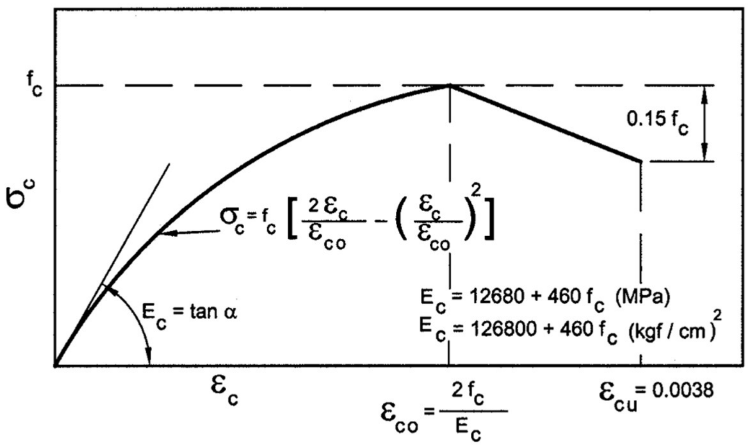


Figure 3.2.4.4 – Concrete Uniaxial Stress-Strain Diagram (Theoretical) [10]

The Hognestad's Parabola Theory is very good and accurate but after the peak, different samples behave differently according to many tests done. So, most of the codes simplify that part. Instead of continuing the parabola, a line is drawn between the peak point (described before) and the point at 0.0038 strain and 0.85 of the peak strength of the concrete.

There is also an important point called ultimate. Ultimate is the value on the line that corresponds to (0.003) strain in most codes. However, CSA takes ultimate as (0.0035).



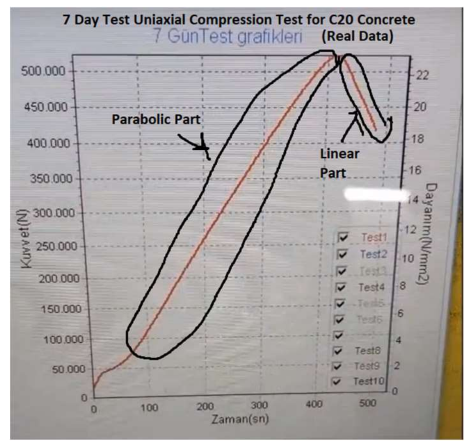
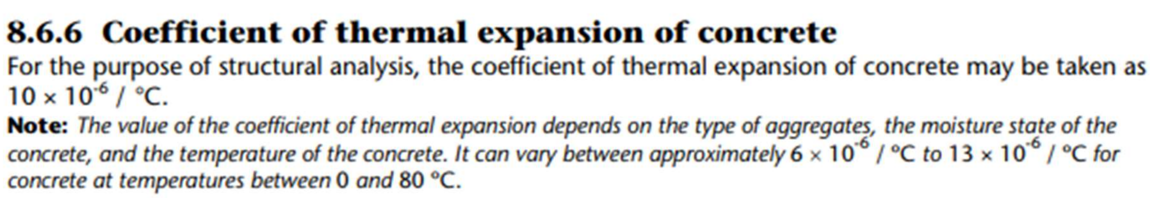


Figure 3.2.4.5 – (Left): An example of Modified Hognestad Parabola From Turkish BC 2018 and (Right) Real test data from a test done by an assistant in Turkey

c) Temperature related behavior of concrete:

Like all other materials concrete contracts and expands with temperature change. These deformations are said to be volumetric. (In x, y and z directions). CSA A23.3-04 provides a simple formula for shrinkage of concrete. According to CSA, concrete expands/contracts 10^{-5} per degree C.



8.6.6 Coefficient of thermal expansion of concrete

For the purpose of structural analysis, the coefficient of thermal expansion of concrete may be taken as 10×10^{-6} / °C.

Note: *The value of the coefficient of thermal expansion depends on the type of aggregates, the moisture state of the concrete, and the temperature of the concrete. It can vary between approximately* 6×10^{-6} / °C *to* 13×10^{-6} / °C *for concrete at temperatures between* 0 *and* 80 °C.

Figure 3.2.4.6 – Coefficient of Thermal Expansion of Concrete

Cement and water reaction produce heat which causes strains in concrete in cold regions. For most applications in warm countries this is neglected but depending on the construction climate, this might be significant.

d) Shrinkage of concrete:

Concrete will shrink due to water evaporating or from hydration reaction. We don't want concrete to lose water so it can react as much as it can with cement. ACI provides some calculations for shrinkage but they aren't necessary to be explained here. For normal strength concrete, at infinity, shrinkage strain values change between 0.0002 and 0.0003.

3.2.5 Reinforced Concrete

Reinforced concrete is a composite material with reinforcing steel and concrete. The purpose of using such material is to increase the tensile capabilities of concrete and to make it actually usable knowing that steel is good in tension.

Some of the values and formulas given below are used for building construction only and
might differ in bridge construction. They are given as a general idea of material
CSI ETABS software mentioned below is suitable for tall building construction

<u>Reinforcing Steel:</u>

The properties of reinforcing steel depend on the country and codes used. In Canada, the standard steel used have a stress-strain profile like this having a yield stress of 400 MPa:

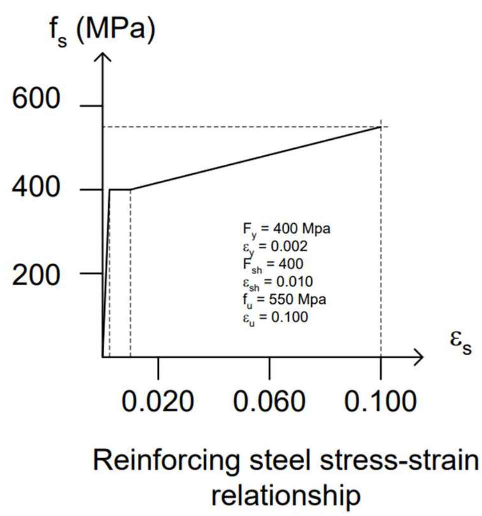


Figure 3.2.5.1 – Reinforcing Steel-Strain Relationship

In Turkey, Turkish BC 2018 prohibits the usage of any steel having a yield stress below 420 MPa and allows usage of S420 by imposing some conditions. Recommended steel has a yield strength of 500 MPa for most applications.

Steel bars produced also are available in different sizes depending on country:

Diamater

Donatı çubukları enkesit alanları

ϕ mm	g kg/m	Çubuk Sayısı 1	2	3	4	5	6	7	8	9	10
8	0.395	0.50	1.01	1.51	2.01	2.51	3.01	3.52	4.02	4.52	5.03
10	0.617	0.79	1.57	2.36	3.14	3.93	4.71	5.50	6.28	7.07	7.85
12	0.888	1.13	2.26	3.39	4.52	5.65	6.79	7.92	9.05	10.18	11.31
14	1.21	1.54	3.08	4.62	6.16	7.70	9.24	10.78	12.32	13.85	15.39
16	1.58	2.01	4.02	6.03	8.04	10.05	12.06	14.07	16.08	18.10	20.11
18	2.00	1.54	5.09	7.63	10.18	12.72	15.26	17.81	20.36	22.90	25.45
20	2.47	3.14	6.28	9.42	12.57	15.71	18.84	21.99	25.14	28.28	31.42
22	2.98	3.80	7.60	11.40	15.27	19.01	22.81	26.61	30.41	34.21	38.01
24	3.55	4.52	9.05	13.57	18.10	22.62	27.14	31.67	36.19	40.72	45.24
26	4.17	5.31	10.62	15.93	21.24	26.55	31.86	37.17	42.47	47.78	53.09
28	4.83	6.16	12.31	18.47	24.63	30.79	36.94	43.10	49.26	55.42	61.58
30	5.55	7.07	14.14	21.21	28.27	35.34	42.41	49.48	56.55	63.62	70.69
32	6.31	8.04	16.06	24.13	32.17	40.21	48.26	56.30	64.34	72.38	80.42
34	7.13	9.08	18.16	27.24	36.32	45.40	54.48	63.56	72.63	81.71	90.79
36	7.99	10.18	20.36	30.54	40.72	50.90	61.07	71.25	81.43	91.61	101.7
38	8.90	11.34	22.68	34.02	45.36	56.70	68.04	79.38	90.73	102.0	113.4
40	9.87	12.57	25.13	37.70	50.26	62.83	75.40	87.96	100.5	113.1	125.6
45	12.48	15.90	31.81	47.71	63.62	79.52	95.43	111.3	12723	143.1	159.0
50	15.41	19.64	39.27	58.91	78.54	98.15	117.8	137.4	157.0	176.7	196.3

Metric bar size	Linear Mass Density (kg/m)	Nominal diameter (mm)	Cross-sectional Area (mm²)
10M	0.785	11.3	100
15M	1.570	16.0	200
20M	2.355	19.5	300
25M	3.925	25.2	500
30M	5.495	29.9	700
35M	7.850	35.7	1000
45M	11.775	43.7	1500
55M	19.625	56.4	2500

Figure 3.2.5.2 – (Left): Turkish rebar sizes: www.sanalsantiye.com and (Right) Canadian rebar sizes: en.wikipedia.org/wiki/Rebar

Comparison between Reinforced Concrete and Just Concrete

This can be explained by giving an example in a pure bending simply supported beam with distributed load on it:

We will look into how adding 1 25M bar changes the moment resistance of this section. When calculations are done with CSI ETABS software using C40 Concrete with Modified Hognestad Model, an ultimate moment resistance of 28 kNm is obtained without a reinforcement bar.

When a single 25 M Canadian 400 MPa steel is added 100 mm above the bottom an ultimate stress of 52.5 kNm is obtained from CSI ETABS software.

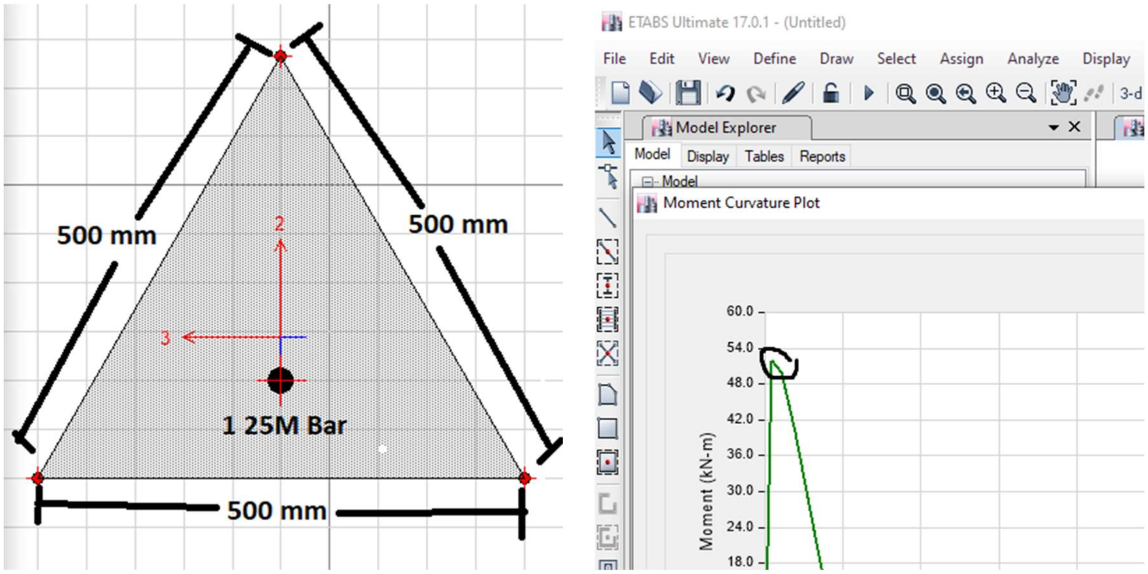


Figure 3.2.5.3 – CSI ETABS software

This value can be verified by strain compatibility or by Whitney Stress Block by hand. The moment capacity almost doubled with the addition of 1 single bar of reinforcement to this unusual section.

3.2.6 Prestressed Concrete

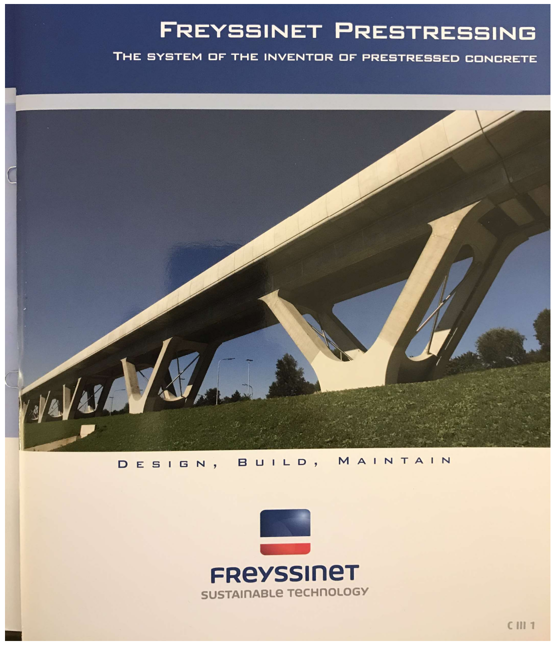
<u>Historical Background of Prestressed Concrete:</u>

Prestressing attempts started back in 1872 when an engineer from California, US got a patent for a simple prestressing system to construct beams or arches part by part. [11]. In 1888, an engineer from Germany named Doehring got a patent for prestressing slabs using some sort of metal wires [11]. Both of these attempts in prestressing were not successful because the high strength steel used in prestressing wasn’t available at that time and after the huge losses coming from prestressing due to lower quality concrete, it was not feasible to use the system [11]. A French scientist Eugene Freysinnet developed a system between 1926 and 1928 to overcome prestressing losses with the use of high strength steel [11]. He started a company for prestressing after several years and the company still exists today as the largest in Europe. After world war 2, various other engineers contributed to the prestressing systems [11]. Today Frayssinet equipment is still used mostly in the post-tensioning area.


Figure 3.2.6.1 – (Left) Eugene Freyssinet [12] and (Right) Modern Freyssinet post-tensioning equipment from construction of North Marmara Highway, Istanbul, Turkey (Feb 18, 2016)

Prestressing Steel:

Prestressing steel is formed by heating base steel material to 800 C to form a homogenous material. After it is cold drawn to form a wire. This results in further increase in its strength. Later on it is heated up to 350 C to homogenize the material again. Then strands are formed consisting of different amount of wires (Ex: 7 Wire Strand)



GES C AND F

1/ Prestressing strands

The table below gives the main characteristics of the most common strands, useable with the Freyssinet prestressing system:

CHARACTERISTICS OF STRANDS AS PER PREN10138-3

Standard	Grade MPa	Nominal diameter (mm)	Nominal reinforcement cross-section (mm²)	Nominal weight (kg/m)	Guaranteed breaking load (Fpk kN)	Elastic limit (Fp0.1 kN)
pr EN 10138-3	1,770	12.5	93	0.73	165	145
		12.9	100	0.78	177	156
		15.3	140	1.09	248	218
		15.7	150	1.18	265	234
	1,860	12.5	93	0.73	173	152
		12.9	100	0.78	186	164
		15.3	140	1.09	260	229
		15.7	150	1.18	279	246

- *Typical elongation under maximum load for all strands is ≥ 3.5%,*
- *maximum relaxation at 1,000 hours under 0.7 fpk for all strands is ≤ 2.5%.*

Figure 3.2.6.2 – Modern Prestressing strands used in practice [14]

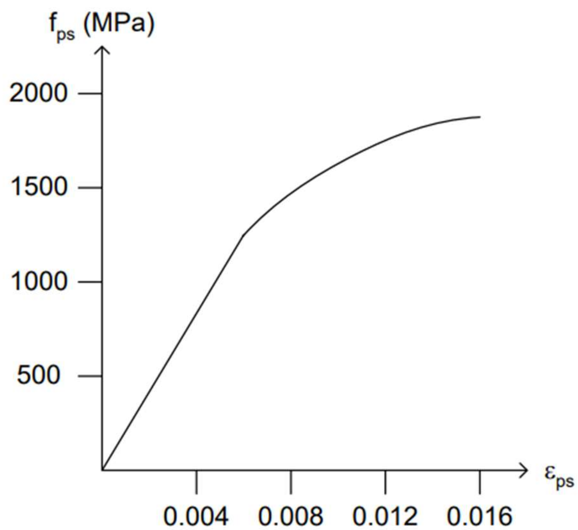


Figure 3.2.6.3 – Stress-Strain Curve of a sample prestressing steel

Comparison between Reinforced Concrete and Prestressed Concrete

Usage of prestressed concrete allows higher span to depth ratios. This means longer spans can be achieved with less depth and shear problem.

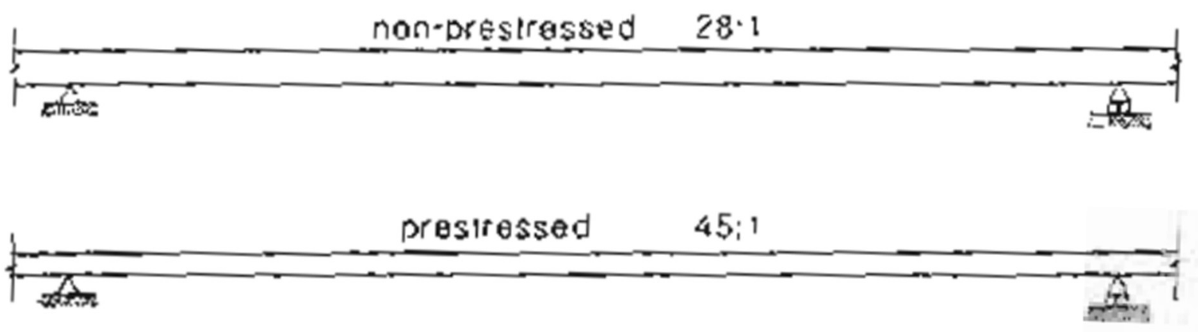


Figure 3.2.6.4 – Typical One-Way Span to depth ratios of beams [15]

Prestressed structures are usually designed to have minimum to no cracks. However, in reinforced concrete structures we design for ultimate. Below are the images showing different stress diagrams observed during different stages of loading.

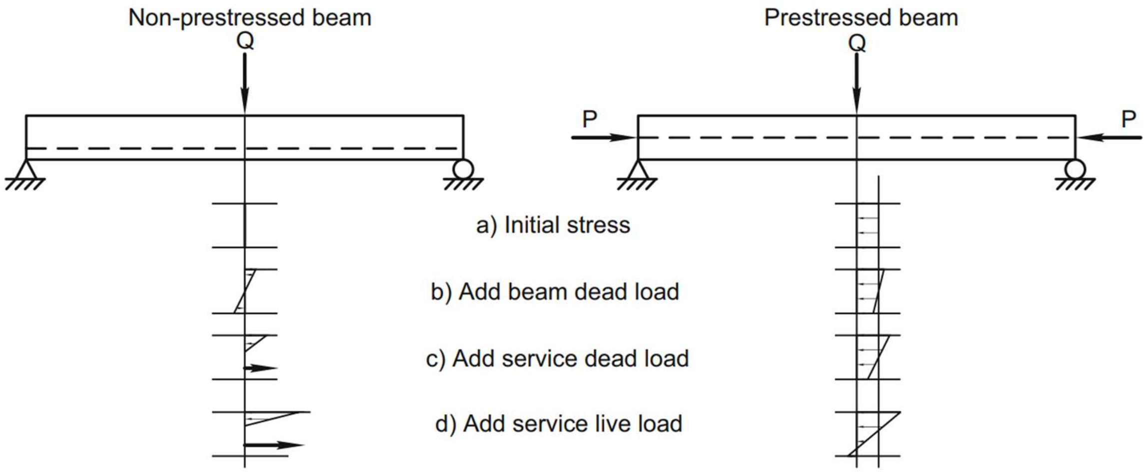


Figure 3.2.6.5 – Stress profiles encountered at beams during different stages (Reinforced Concrete to the left, Prestressed Concrete to the right) [16]

Prestressed concrete structures have much higher moment capacity compared to reinforced concrete structures.

As an example to this, we used the triangle section in 3.1.5 and added 5 grade 1860 strands each having a total area of 100 mm^2 totaling 500 mm^2 (Same as the reinforced concrete area). We made sure that the center of gravity of steel becomes the same as the reinforced concrete one. We made sure that effective prestressing force is 700 kN (after losses).

Using a MATLAB script written by us, the following results were obtained:
-Prestressed Concrete at ultimate:
Moment Resistance: ***155 kNm***
-Prestressed Concrete just before cracking:
Moment Resistance: ***111 kNm***
-Reinforced Concrete at ultimate:
Moment Resistance: ***53 kNm***

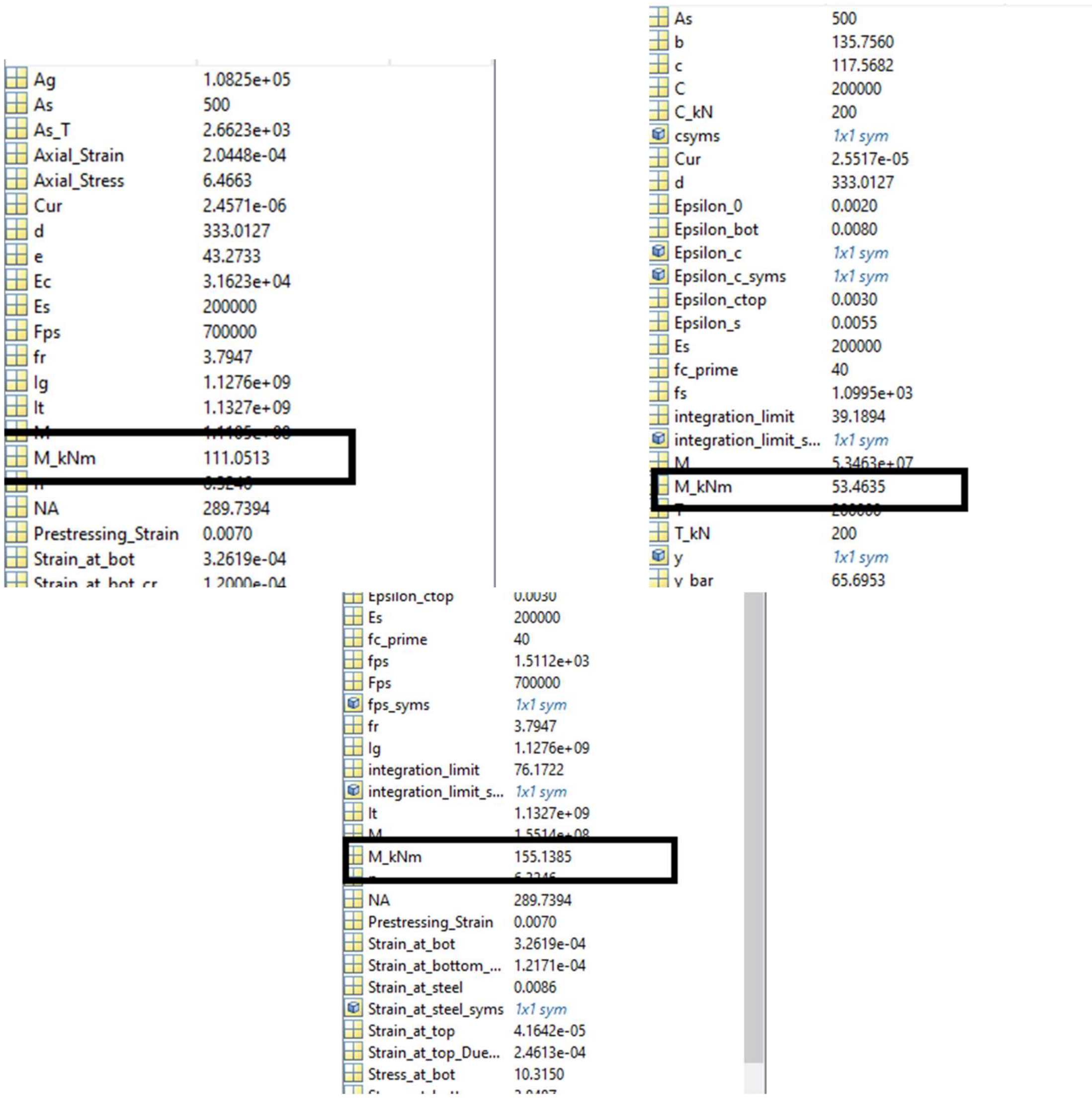


Figure 3.2.6.6 – (Left): Prestressed concrete just before cracking
(Right): Reinforced concrete at ultimate (strain at top compression fiber = 0.003)
(Bottom): Prestressed concrete at ultimate

So, the moment capacity of a given section for prestressed concrete is much more than just reinforced concrete.

Flexibility and rigidity of a structure can be controlled in a post-tensioned system whereas in reinforced concrete this is not possible. By changing the prestressing force, a structure can be made more rigid if there is too much vibration [11].

3.3 Section Types

3.3.1 Box Girder Section

Box Girder consists of top deck, vertical web, and bottom slab as depicted in Figure 3.19. The box girder can support both positive and negative bending moments, as both the top and bottom flanges can withstand stress [17]. The web is comparatively thin to reduce the deadweight of the section. The box girder section can be made out of different types of materials such as steel, prestressed concrete or reinforced concrete. It is widely used in the construction of cantilever, continuous and cable stayed bridges. The biggest advantage of using box girder section is that it can be used for long spans keeping the self-weight of the web to a minimal [18]. It has greater aerodynamic stability than an I girder [18]. The maintenance cost of the protective coating is less as the exposed surface is small [18]. The box girder is usually prefabricated in a controlled environment and then hauled to the site [18].

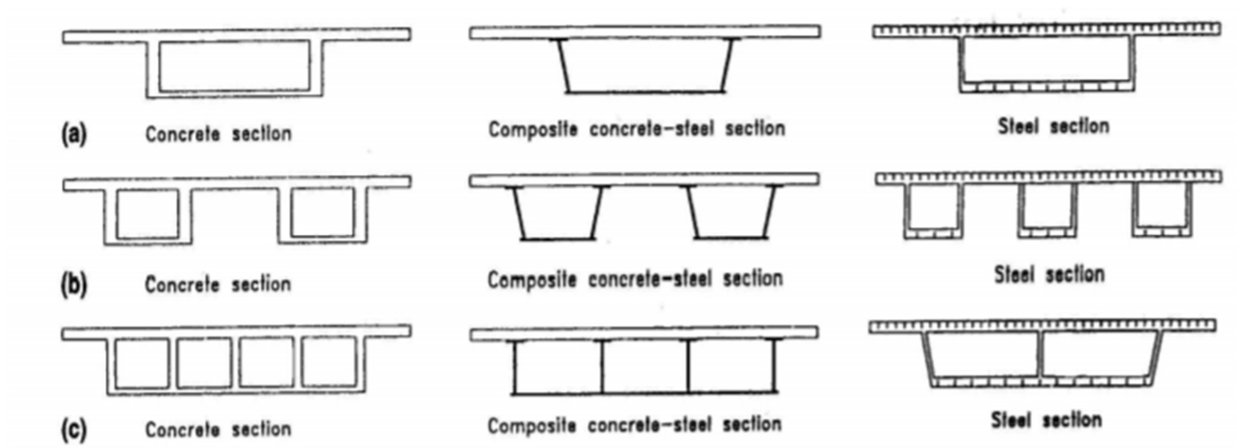


Figure 3.3.1.1 – Different types of cross sections for a box girder bridge [19]

3.3.2 I-Section

I-Section consists of a top flange, a bottom flange and a web connecting the two flanges. The top flange connects the girder to the deck above, the bottom flange is used to provide strength where tensile stresses are significant, and the web provides bending resistance [20]. The two flanges do not twist or tilt easily because both of them have the capacity to cope with high bending and shearing stress [20]. The web also resists shear forces [21]. I-Section is used for girders in bridge construction, supporting trusses in buildings and also used as rails for railway tracks [21]. I-Section is weak in resisting torsion and also weak in the transverse direction [21]. Figure 3.20 depicts a typical stainless steel I-Section beam.

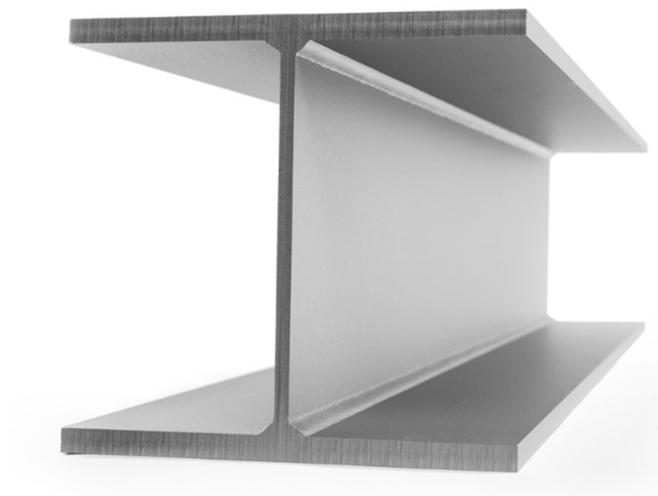


Figure 3.3.2.1 – Stainless Steel I-Beam [22]

3.3.3 T-Beam

A T-beam can be the load bearing component of bridges made out of reinforced concrete, wood or metal. It has a T-shaped cross section as depicted in Figure C. The top flange of the T-beam is called the compression flange because it resists compressive stresses [23]. The portion below the flange is called the web or rib of the beam. It resists shear stress and also bending [23]. The T-beam has no bottom flange making it difficult to cope with tensile forces but saves the use of extra material [24].

Figure 3.3.3.1 – Double T-Beam Cross Section [25]

3.3.4 Slab Section

Slab sections are the easiest and cheapest sections that can be used in bridges. Unfortunately, due to material strength limitations, they cannot be used for long bridges. If the slab is steel, this type of section can be used up to 25 meters in length. For concrete, slabs must get thicker quickly with length and the problem of shear becomes significant together with weight. At a certain length, the required reinforcement spacing will be less than the minimum spacing allowed by the construction code not allowing the usage of the section.

Figure 3.3.4.1 – Slab Bridge: [26]

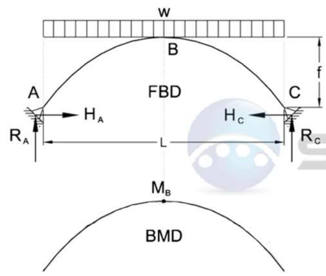
3.4 Geometry Types and Application

3.4.1 Arch Bridges

3.4.1.1 General info and characteristics of Arch Bridges

The first construction of Arch Bridges happened around 4000 B.C. in Mesopotamia according to our current knowledge (Arch Bridges, Anna Sipoli, University of Architecture, Venice). The people of that time didn’t know and have any construction material that can resist tension well. Most of the construction was done putting together stones or some sort of natural material. They discovered if they do a construction in a shape close to a parabola, the materials tend to resist forces more and they can actually construct a bridge.

Nowadays with our current knowledge, we know that if we can make a perfect parabolic arch, the bending moments in each section becomes zero given a distributed load is applied. For point loads and other types of loading this changes a bit. So, with today's knowledge we choose an arch as close as we can get to a parabola. It happens so that sometimes circular and parabolic arch differences become negligible (large length compared to depth).



Support Reactions

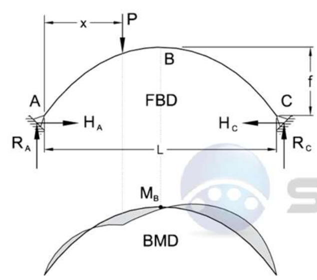
$$R_A = R_C \ldots\ldots = \frac{wL}{2}$$

$$H_A = H_C \ldots\ldots = \frac{wL^2}{8f}$$

Bending Moments

$$M_B \ldots\ldots = 0$$

Figure 3.4.1.1.1 – Parabolic two hinged arch with distributed load



Support Reactions

$$R_A \ldots\ldots = P\frac{L-x}{L}$$

$$R_C \ldots\ldots = P\frac{x}{L}$$

$$H_A = H_C \ldots\ldots = \frac{5PL}{8f}\left(\left(\frac{x}{L}\right) - 2\left(\frac{x}{L}\right)^3 + \left(\frac{x}{L}\right)^4\right)$$

Bending Moments

$$M_B \ldots\ldots = \frac{PL}{8}\left(4\left(\frac{x}{L}\right) - 5\left(\left(\frac{x}{L}\right) - 2\left(\frac{x}{L}\right)^3 + \left(\frac{x}{L}\right)^4\right)\right)$$

Figure 3.4.1.1.2 – Parabolic two hinged arch with point load at any point

Using the property of superposition, and the diagrams above, a linear elastic calculation can be done for static loads. For dynamic loading, a finite element analysis is usually required since it will be very difficult (almost impossible) by hand to perform those calculations.

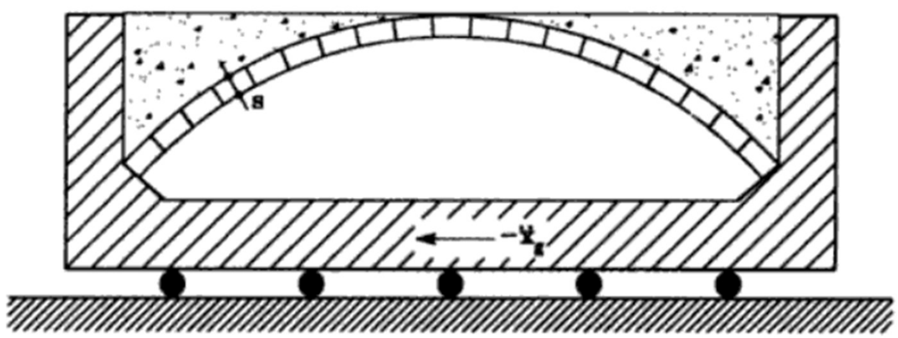


Figure 3.4.1.1.3 – Structural Model of a stone arch under horizontal ground motion loading [27] (Arch Bridges, Anna Sipoli, University of Architecture, Venice).

3.4.1.2 Construction Methods for Arch Bridges

Falsework centering:

A falsework is a temporary construction to support the arch until it is fully constructed. Since most of the ancient bridges were made out of stone and without mortar some sort of carrying work had to be provided. Most of the ancient bridges were constructed using this method. Some current concrete bridges also use this method (if underneath surface permits and if economical).

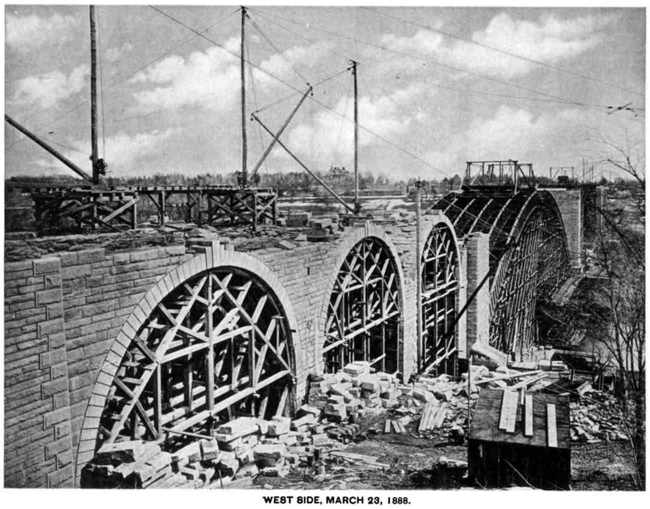


Figure 3.4.1.2.1 – (Left) Washington Bridge west side 1888 and (Right) A small arch model bridge made by a person by putting stones together by hand [28]

Figure 3.4.1.2.2 – Modern use of examples of falsework Galena Bridge 2012 Nevada [29]

<u>Travelling formwork method for concrete:</u>

It is not always possible or economical to construct a falsework. Instead a formwork with certain curvature is sometimes prepared (concrete bridges). Then starting from abutments, the concrete is cast. Once the concrete hardens, the formwork is moved one step ahead. After concrete is cast again. At some point, due to the cantilever behavior of two sides, supporting cables are required. At the end, two sides meet in the middle. After the middle concrete hardens, cables are removed, and arch behavior is established.

Figure 3.4.1.2.3 – Photo of construction of an arch bridge with travelling formwork method [30]

Segmental Construction:

Bridge is constructed segment by segment and put in place piece by piece starting from abutments towards the middle. Used in modern prestressed bridge applications.

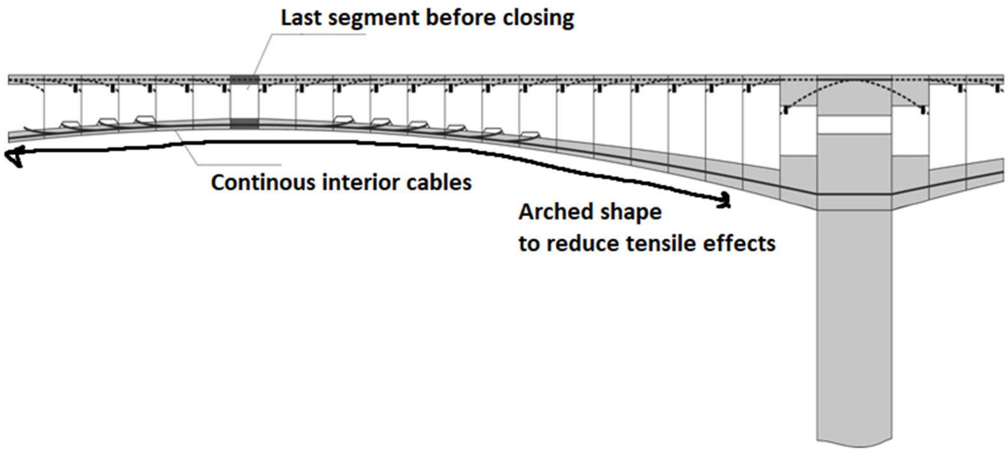


Figure 3.4.1.2.4 – Possible cable layout for segmental construction drawing of Prof Erhan Karaesmen Middle East Technical University, Turkey [31]

Figure 3.4.1.2.5 – Stolmadunsed Bridge, Norway [32]

Lowering Method:

A method developed in Italy around the 1950s. Two sides of arches are constructed vertically on both sides. Then they are lowered and put in place with the help of cables.

Figure 3.4.1.2.6 – Construction of an Arch Bridge by Lowering Method [33]

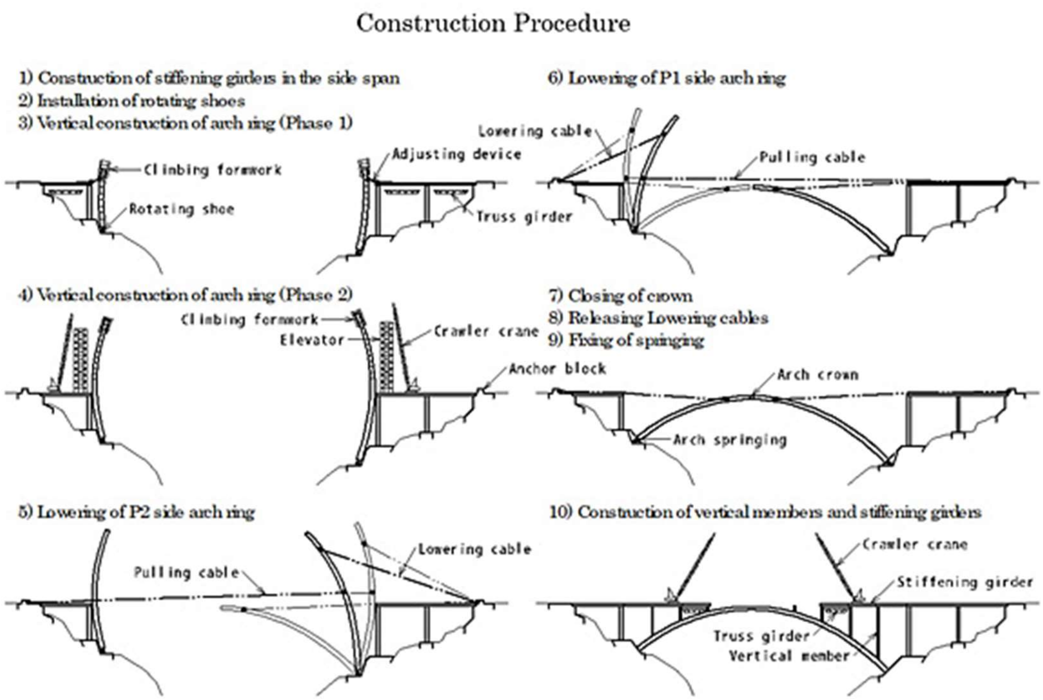


Figure 3.4.1.2.7 – Construction Procedure [34]

3.4.2 Movable Bridges

A movable bridge is a bridge that changes its position or shape to allow both vehicular traffic over the deck of the bridge and river traffic underneath the bridge. This type of bridge has low construction cost as the use of high piers and long approaches are not required for its construction. Only one type of traffic is allowed to pass at once (ex: Boat vs vehicular traffic) [35]. Usually, this type of bridge is used when the difference between the road and the water level is small and both types of traffic movements need to be accommodated. The operation and maintenance cost of movable bridges are more expensive than the stationary bridges, and hence are rarely constructed in present times unless absolutely required [38].

3.4.2.1 General info and characteristics of Movable Bridges

The first movable bridge was constructed in the 2nd millennium BC in Egypt followed by Chaldea in the middle east in 6th century BC [35]. There are several different types of movable bridges, below are a few movable bridges that are still in use and desirable.

-Bascule Bridge: It is fixed and supported on one end like a hinge and the other end is lifted to allow for the passage of water traffic. This bridge is also called drawbridge and was used as entrances for castles in London [35]. This bridge can be of two types, single leaf or double leaf as shown in Figure 3.4.2.1.1. These types of movable bridges are structurally sound, reliable and economically viable [36].

Figure 3.4.2.1.1 – Single and Double Leaf Bascule Bridge [36]

-Vertical Lifting Bridge: This type of bridge usually has a truss span which is lifted up and down by the two towers at each end of the span to allow for the movement of water traffic as shown in

Figure 3.4.2.1.2. The span weight is balanced by using a counterweight and the sheaves which are fixed on the tower are used to connect the span to the counterweight [37]. The details of the vertical lifting bridge are shown in Figure 3.4.2.1.3. These types of bridges have a high stability power and are therefore used where long spans are required [37].

Figure 3.4.2.1.2 – Vertical Lifting Bridge [36]

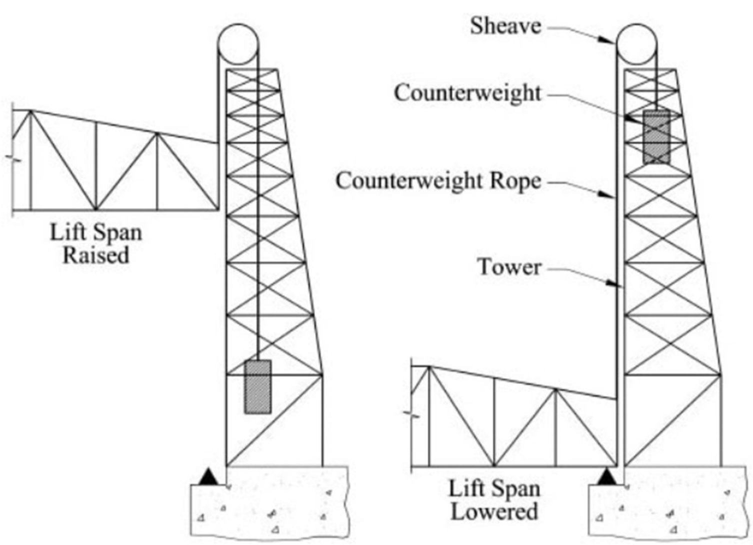


Figure 3.4.2.1.3 – Details of Vertical Lifting Bridge [37]

-Swing Bridge: It is a bridge where the deck rotates horizontally around the pivotal pier located at the center of the deck to allow for movement of water traffic as shown in Figure 3.4.2.1.4. In most situations, the span of the deck is made of plate girder as it is more economical [37]. The ends of the deck rests on the abutments when the passage for water traffic is closed and it's free supported by the pivotal pier when the water traffic is open.

Figure 3.4.2.1.4 – Swing Bridge [37]

3.4.2.2 Construction Methods for Movable Bridges

This section explains the construction process for the most common type of movable bridge, the Bascule Bridge. This is just a generic description of how a bascule bridge is constructed.

-Sample Pier Construction:

Cofferdams are used to build the foundation of the bridge in order to prevent sediments going into the water affecting the river and aquatic life [39]. Forms are utilized to shape the concrete piers along with rebars to provide reinforcement cage in the interior of the piers. A fender is constructed on all sides of the piers to protect the ships colliding during its construction process [38]. Slippery plastic is placed on the fenders to help avert minor impacts [38].

3.4.3 Truss Bridges

3.4.3.1 General info and characteristics of Truss Bridges

Truss Bridges are bridges constructed in a way so that all members become two axial force members (tension or compression). Main structural members of truss bridges have no shear, flexural or torsional forces in them. This can be achieved through geometry or by choosing appropriate connections. These types of bridges are usually constructed in steel or wood.

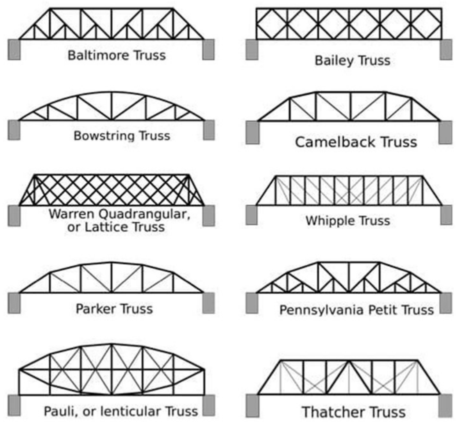


Figure 3.4.3.1.1 – Some of the different types of trusses [40]

The history of truss bridges goes before the industrial revolution. There were many wood bridges constructed as a truss at that time. After the industrial revolution, engineers and scientists started patenting different types of trusses. The usage of iron and later steel increased for construction [41].

Currently the longest span truss bridge is Ikitsuki Bridge with a 400 m span located in Japan [42]. The bridge has 2 other 200 m spans. The total length of the bridge is 960 m [43]

Figure 3.4.3.1.2 – Ikitsuki Bridge [44]

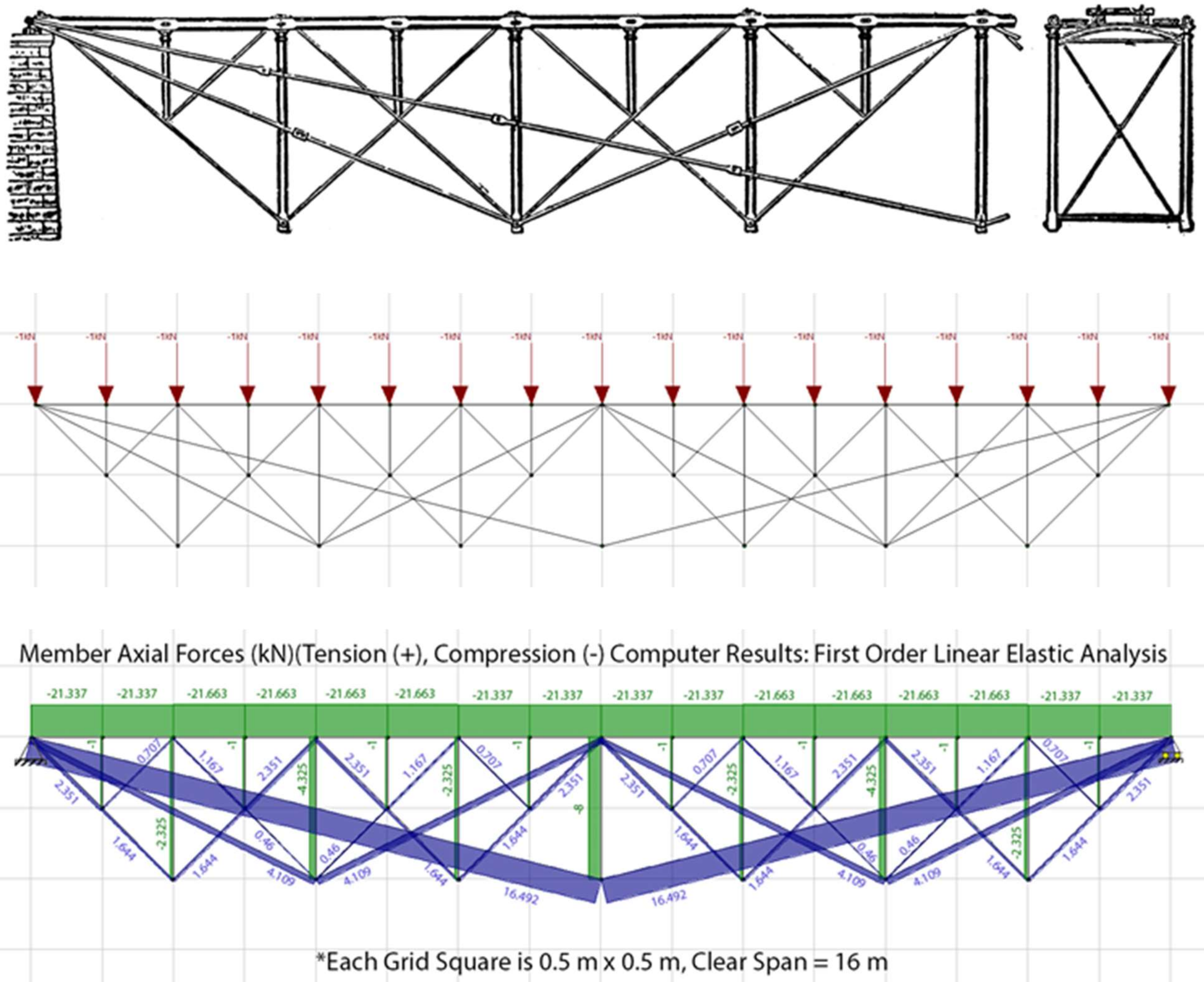


Figure 3.4.3.1.3 – Force distribution in one of the old Truss Bridge Design, Fink Truss

-Cables are modeled as tension only members (solver assigns a buckling load of 0 kN for the member making its stiffness 0 in axial direction as soon as it goes under compression)

Advantages of Truss Bridges:

-Cheap in terms of the price of materials.
-Can be constructed somewhere else and installed using incremental launch or direct placement [44].

Disadvantages of Truss Bridges:

-Heavy: Larger dead load due to lots of steel members.
-Expensive to construct since construction methods like incremental launch are expensive.
-Complicated design. Every member has to be produced at specified length. Otherwise load balances may change [44].

3.4.3.2 Construction Methods for Truss Bridges

Construction by Incremental Launching:

This method is mostly used in prestressed concrete bridge construction, but it is also used for truss bridges. It is usually preferred when fast construction is required. In this method, a bridge is partly or fully constructed on one side. That side is like the factory of bridge. Then moved slowly towards the other end. There is a supporting device usually attached to the bridge to reduce the maximum cantilever length. This method is expensive and not used if some other construction can be done if budget is the main priority.

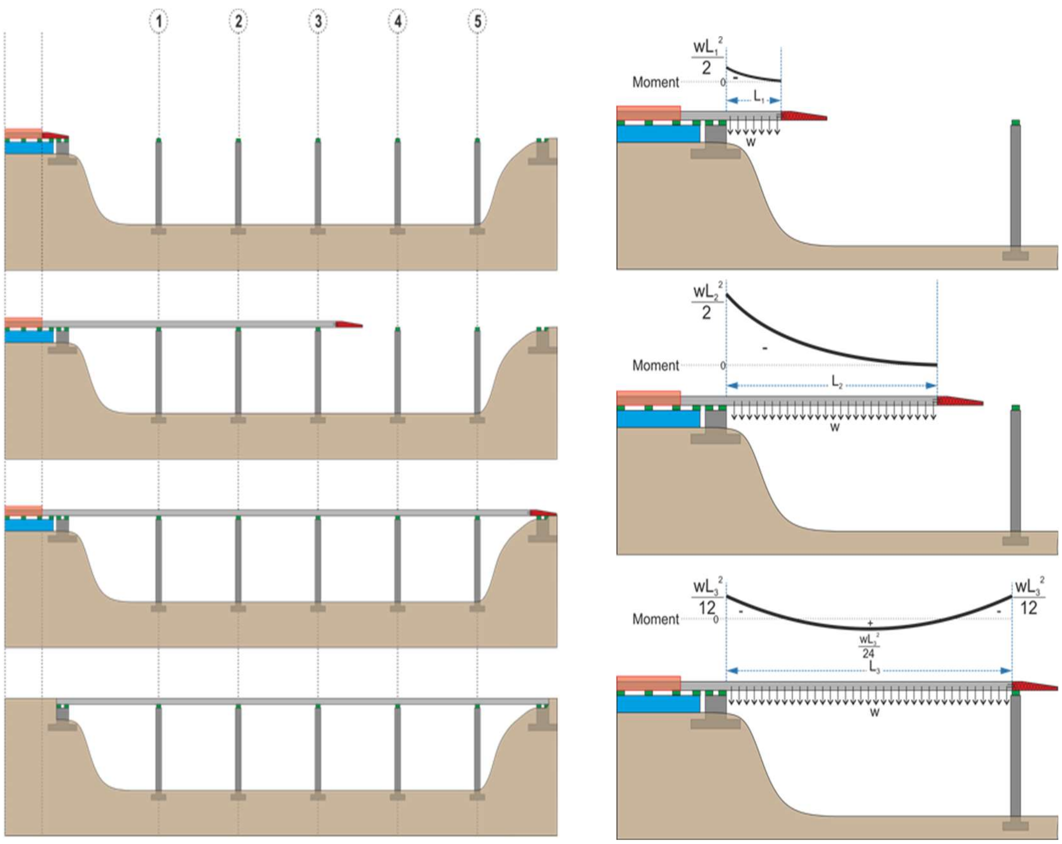


Figure 3.4.3.2.1 – Incremental Launching Method [45]

Figure 3.4.3.2.2 – Launching Nose [46]

Figure 3.4.3.2.3 – Surface with less friction removal once bridge moves over support [46]

<u>Direct Placement:</u>

Truss is fabricated somewhere and placed right into the spot by the help of two or more cranes. Usually used for short span bridges.

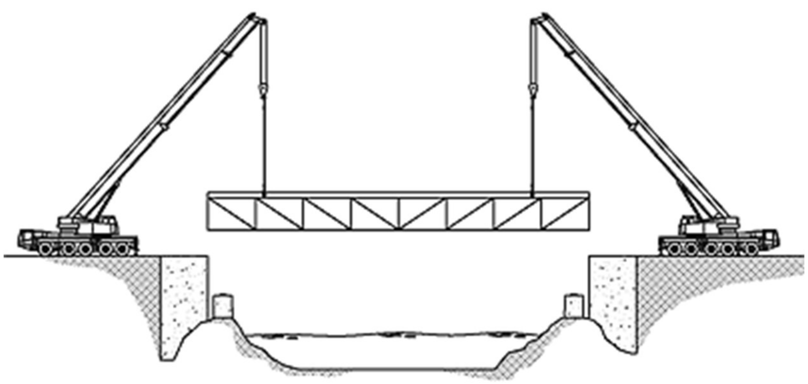


Figure 3.4.3.2.4 – Direct Placement [47]

3.4.4 Cantilever Bridge

3.4.4.1 General info and characteristics of Cantilever Bridges

A cantilever bridge is a bridge whose main components are cantilevers which are anchored at one end and projected outwards on the other end [48]. For pedestrian bridge use, cantilevers are constructed from beams, however for larger bridges that support road or rail traffic, the cantilever is constructed using trusses (structural steel) or box girder (prestressed concrete) [48]. Hassfurt Bridge crossing over the main river in Germany was the first cantilever bridge constructed in 1866 by Heinrich Gerber [48]. Currently, Quebec bridge built in 1919 is the longest span cantilever bridge in the world, with a span of 549 m [48].

A basic cantilever bridge will have two cantilever arms extending from opposite sides of an obstacle that span and converge at the center. There is a suspended span in the center portion of a cantilever bridge as shown in Figure 3.4.4.1. The one end is always fixed while the other end of the structure is open to space.

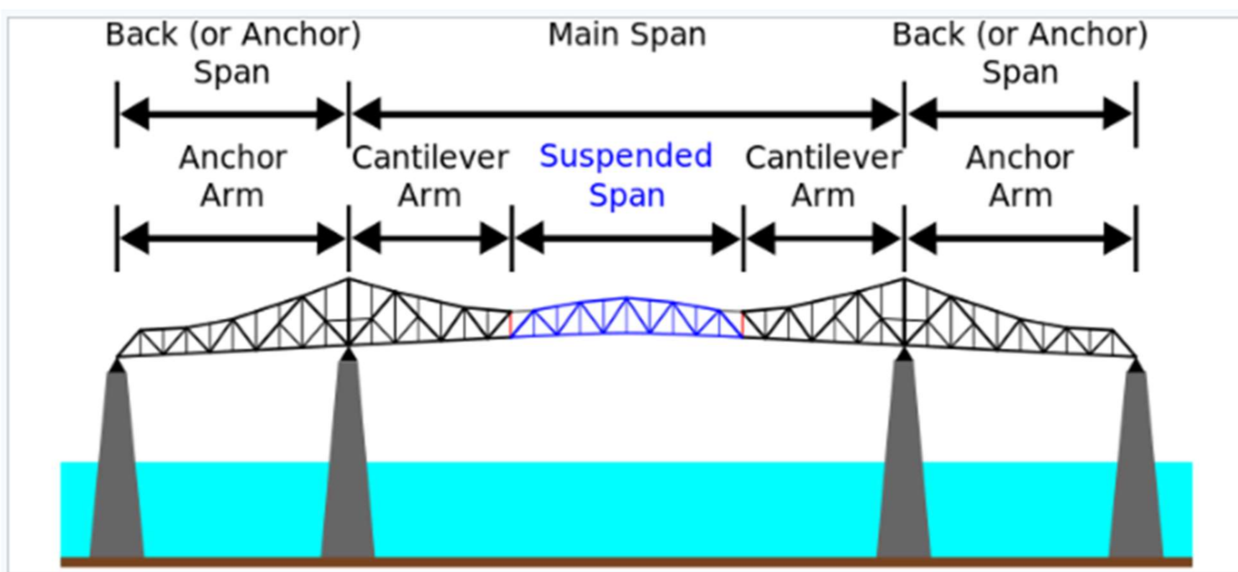


Figure 3.4.4.1.1 – Simple Cantilever Bridge [49]

Figure 3.4.4.1.2 – Quebec Bridge across the lower Saint Lawrence River [50]

Advantages:

-Each cantilever just needs to be supported only on one of its ends [51].
-Falsework for bridge work is not required except for the pier construction [51].
-The support structure for the bridge can be simple columns [51].
-Ideal for long span bridge construction [51].

Disadvantages:

-The cantilever structures are massive in size, requiring complex construction techniques [51].
-Very expensive compared to the other bridge types as it consumes more material to build the required heavy [51] structure and to balance the tensile and compressive forces [51].

3.4.4.2 Construction Method for Cantilever Bridges

The construction of cantilever bridges starts from the two opposite ends and meets at the center to complete the bridge structure. The bridge construction method can be slightly different depending on the design requirement where the two cantilevered ends connect to a middle-suspended span to complete the bridge structure. The suspended span of the bridge can either be constructed off site and lifted into place by a special equipment or constructed on site using a special support machine as seen in Figure 3.4.4.2.1 [52].

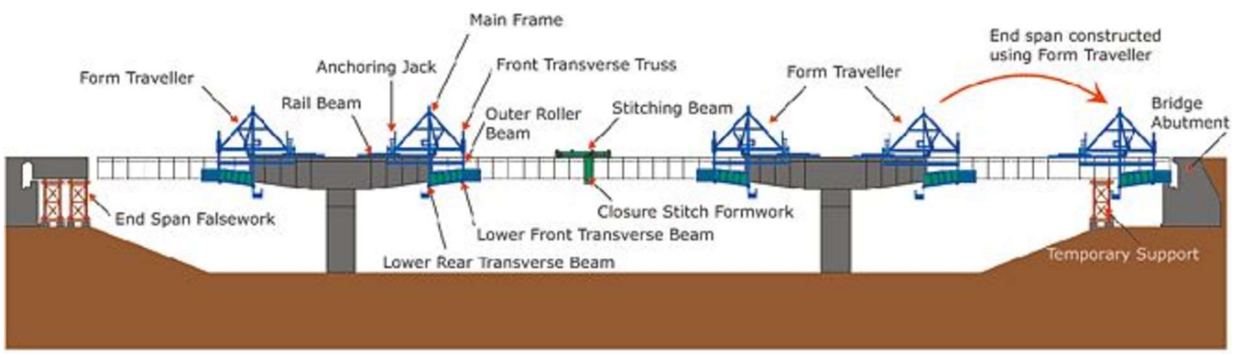


Figure 3.4.4.2.1– Construction Technique for Cantilever Bridge Construction [53]

3.4.5 Cable-Stayed Bridges

3.4.5.1 General Info and Characteristics of Cable-Stayed Bridges

A cable stayed bridge is a bridge with one or more towers from which cables supporting the bridge deck. The difference of this kind of bridge from suspension bridge is that in cable stayed bridge, every cable is connected directly to the towers from the deck whereas in a suspension bridge cables are connected to the main cable [54].

Cable-stayed bridges started appearing in the 19th century mostly in Europe and continued in the 20th century with modern bridges [55].

Figure 3.4.5.1.1– Brooklyn Bridge - 1883 [56]

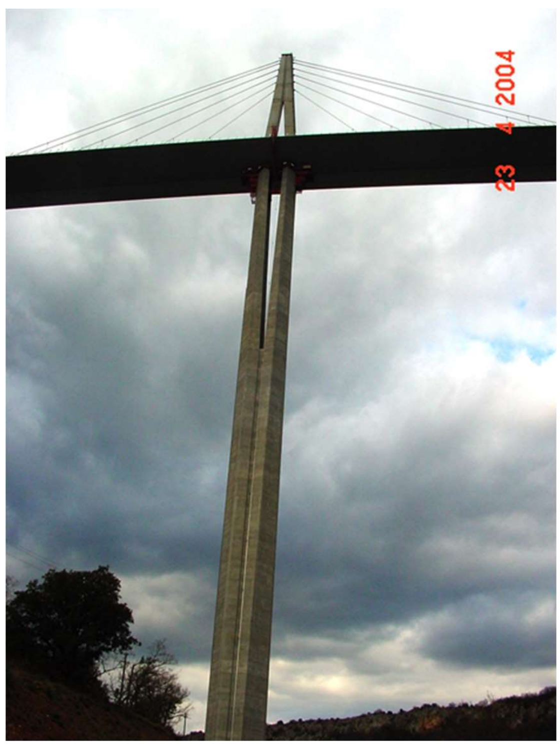


Figure 3.4.5.1.1– Milau Viaduct - Tallest Bridge of the world approx. 340 m height [57]

Cable-stayed Bridge vs Suspension Bridge:

-Cable-stayed bridges are cheaper but have shorter spans compared to a suspension bridge.
-Cable-stayed bridges can be built faster.
-They don't have anchorages.

3.4.5.2 Construction Method for Cable Stayed Bridges

Cable stay bridges are constructed usually using the cantilever method explained in section 3.4.4.2

3.4.6 Suspension Bridges

3.4.6.1 General Info and Characteristics of Suspension Bridges

A suspension bridge is a type of bridge where the deck, which is the load bearing section of the bridge, is hung below suspension cables on vertical suspenders [58]. Suspension cables are tied up at both ends of the bridge, so they carry the bulk of the load with towers in between [58]. The suspension bridge is constructed without falsework in most cases [58]. The earliest version of suspension bridge was built in the 15th century by Thangtong Gyalpo in Tibet and Bhutan [58]. Menai bridge is the first modern day suspension bridge built in 1826 between the Wales coast to the coast of Anglesey and it is supported by 16 large chains [62]. It has a span length of 175m and at a height of 30m above the water and it is still in use [62]. The modern-day suspension bridges have the following components: towers, main suspension cables, vertical suspenders, cable anchors and bridge deck.

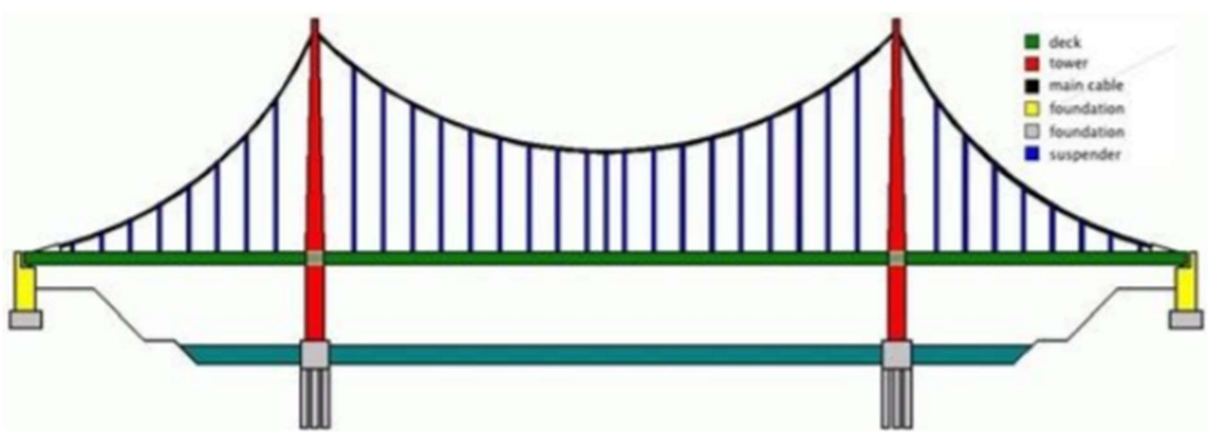


Figure 3.4.6.1.1– Labelled Suspension Bridge [59]

The towers carry the major portion of the weight, where the compression forces are transferred from the deck to the towers via cables, ropes and suspenders [60]. The towers will then dissipate the compression forces into the ground as shown in Figure 3.4.6.1.2. This is the principle of how compression forces travel in the most iconic suspension bridge “The Golden Gate Bridge” as shown in Figure 3.4.6.1.3.

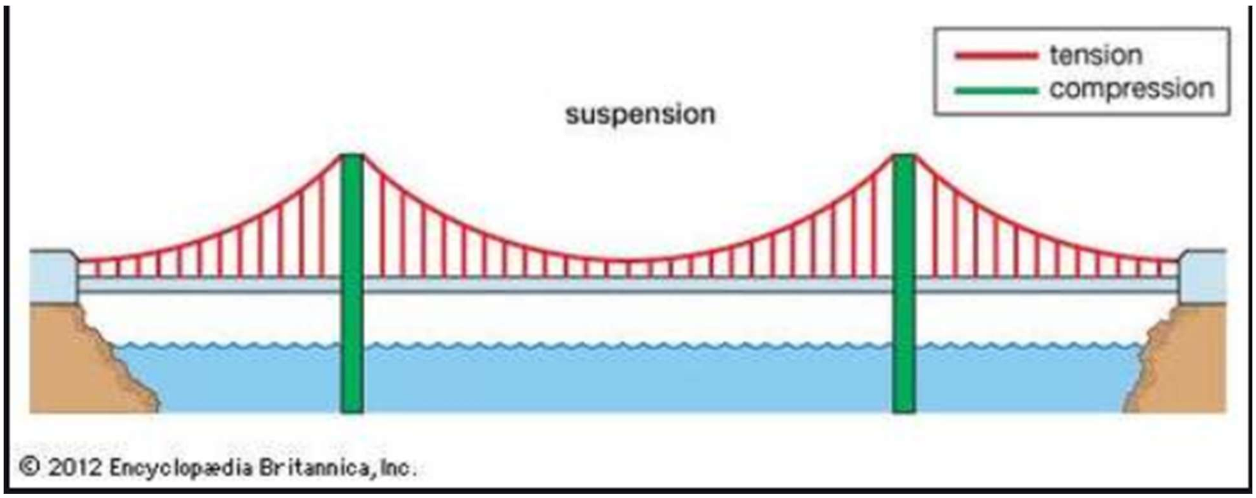


Figure 3.4.6.1.2– Compression and Tension Force in Suspension Bridge [60]

Figure 3.4.6.1.3– Golden Gate Bridge - Classic Example of Suspension Bridge [60]

The suspension bridges typically have a span length ranging between 2000ft - 7000ft [61]. Currently, Akashi Kaikyo Bridge is the longest suspension bridge across the globe with a center span length of 65227ft and the total length of the bridge is 12,828 feet as shown in Figure 3.4.6.1.4 [61].

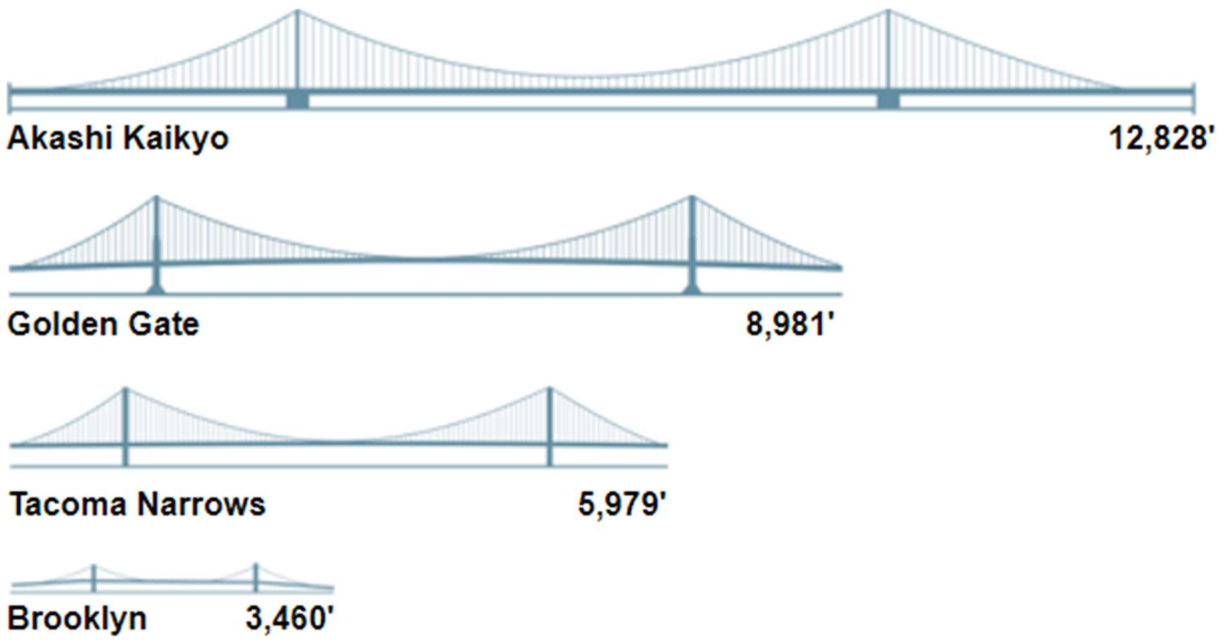


Figure 3.4.6.1.4– Akashi Kaikyo Compared to other Iconic Suspension Bridges [61]

Advantages:
-Greater span length with minimal material usage [64].
-Can be built high up to allow for water traffic [64].
-Great flexibility allows to withstand earthquakes [64].

Disadvantages:
-Cannot support heavy traffic due to low deck stiffness [64].
-Cannot be built in areas with soft ground, will require extensive foundation work [64].

3.4.6.2 Construction Method of Suspension Bridges

The construction of suspension bridges has 6 main steps: Firstly, anchors which are usually made out of concrete are fixed in the ground. They support the cables. Secondly, the foundation of towers is constructed which needs to be on a solid platform as the compressive forces are transferred through the tower into the ground. If the foundation is constructed underwater, then the water needs to be dammed and pumped out. Thirdly, towers are erected, this is an important step as higher the towers, longer the span of the bridge. Fourthly, the main suspension cable is stretched between the two anchor points and over the two towers. Twisted steel cable is used for the two main cables, the size of the cable depends on the load that the bridge can sustain and span of the bridge. Fifthly, trusses are attached to the main cable which adds to the stiffness of the cable to restrict cable movement during windy conditions. Vertical suspenders are then connected from the truss to the deck. Lastly, the deck is fixed to the vertical suspenders and finishing of the deck is completed.

<u>Load Path is as shown below:</u>

Deck → Vertical Suspenders → Main Cables → Towers → Ground

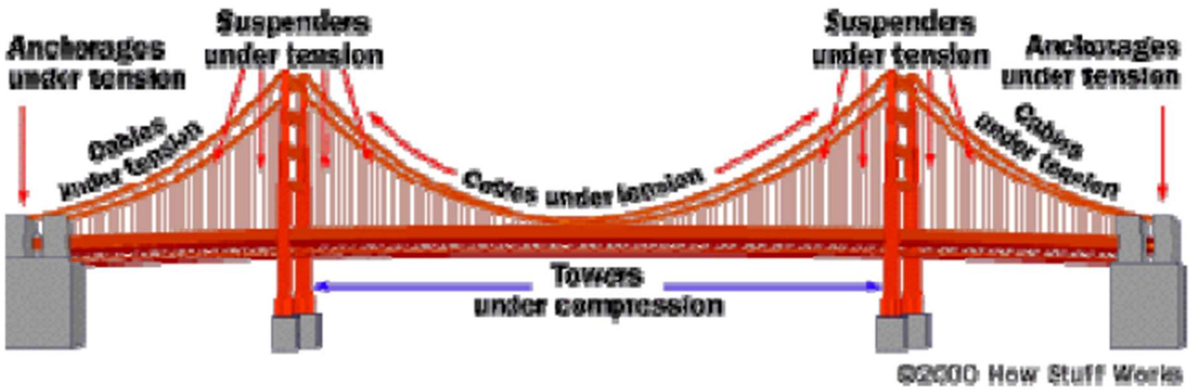


Figure 3.4.6.2.1– Typical Construction Sequence of Suspension Bridge [65]

3.5 Summary

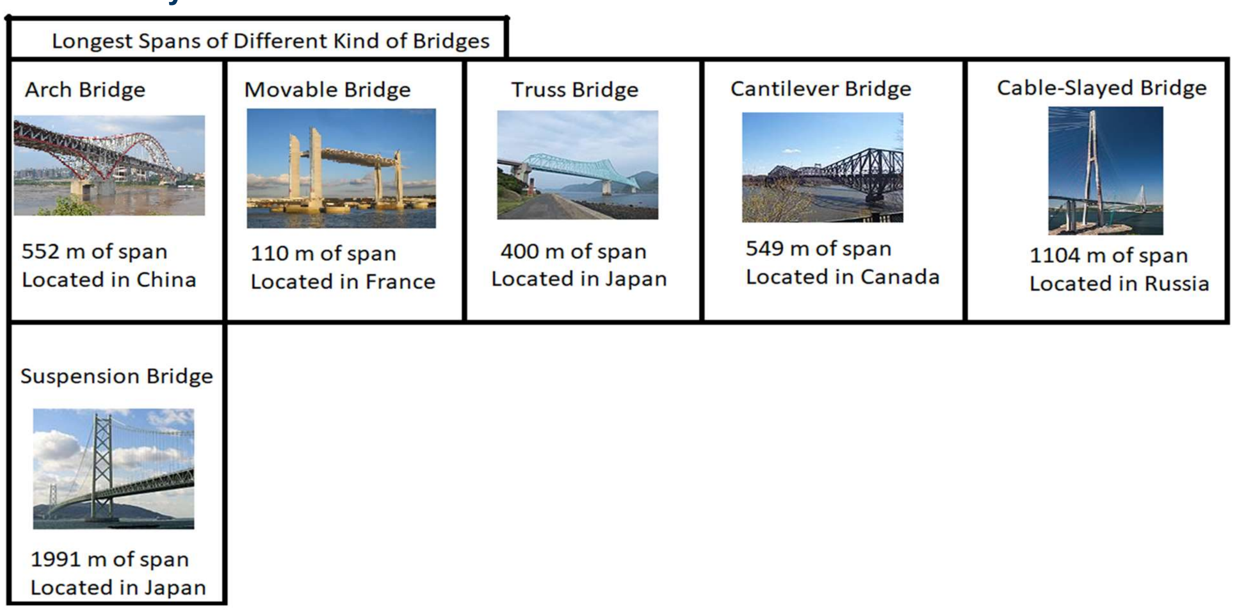


Figure 3.5.1– Longest Spans of Different Kind of Bridges

Type	Typical Span [m]	Critical Advantage	Critical Disadvantage
Arch Bridge	200-552	Aesthetics	Heavy falsework required
Movable Bridge	10-110	Facilitates road and water traffic	Maintenance
Truss Bridge	50-400	Cost effective, easy to install	Maintenance
Cantilever Bridge	10-549	No falsework required	Material Cost
Cable-Stayed Bridge	100-1104	Efficiency and aesthetics	Cost
Suspension Bridge	600-1991	Earthquake resistant,suitable for long spans	Design Complexity

Figure 3.5.1– Spans of Different Kind of Bridges, critical advantages and disadvantages

3.6 Conclusion

As a bridge engineer, it is important to study the different materials involved in bridge construction, different cross sections of bridges and different types of bridges. The construction of different types of bridges has its own advantages and disadvantages and provides a specific application that varies from location to location. The above-mentioned information along with the engineering toolbox helps determine what type of bridge should be constructed at a particular location.

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Chapter 4 – Design Criteria

Table of Contents

4.1 Introduction

In this chapter, three design codes and their critical points are discussed: Current Canadian Highway Bridge Design Code, CSA S6-14 rev. 17; Canadian Highway Bridge Design Code of 1966, CSA S6-66; and American Bridge Design Specifications, AASHTO LRFD 2014-17.

In Canada, Canadian Standard Associations Group (CSA) develops the standards and codes. In this section one of their latest (2017) revisions to the 2014 Canadian Highway Bridge Design Code is used. CSA also publishes and sells various handbooks with design examples.

In the US, American Association of State Highway and Transportation Officials (AASHTO) is responsible for the design of the bridges. In this chapter, their 8th edition which contains 2017 revisions is used. This code is available both in SI units and imperial units. Both of them can be referred depending on convenience.

CSA S6-66 solely included because it was the design code of De La Concorde Overpass and It is a great example to compare with the modern design codes.

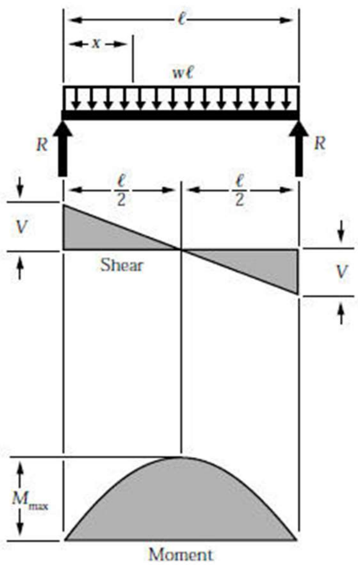
4.2 Bridge Behavior and Loading

In this section, the emphasis will be on different types of bridge behavior and loading. The bridge behaviors include shear and bending behavior, influence line, torsion and deformation. The bridge loading includes dead load, live load and dynamic loads.

4.2.1 Bridge Behavior

Considering the purpose of this report, the main focus of this section is on continuous or simply supported reinforced or prestressed concrete bridges. In this section, we are focusing on flexural behavior, shear and torsional behavior of these types of bridges.

4.2.1.1 Shear and Moment Behavior



$$R = V \quad \dots\dots\dots\dots\dots = \frac{w\ell}{2}$$

$$V_x \quad \dots\dots\dots\dots\dots = w\left(\frac{\ell}{2} - x\right)$$

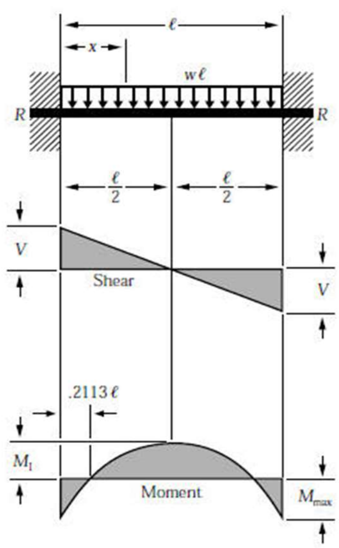
$$M_{max} \text{ (at center)} \quad \dots\dots\dots = \frac{w\ell^2}{8}$$

$$M_x \quad \dots\dots\dots\dots\dots = \frac{wx}{2}(\ell - x)$$

$$\Delta_{max} \text{ (at center)} \quad \dots\dots\dots = \frac{5w\ell^4}{384\,EI}$$

$$\Delta_x \quad \dots\dots\dots\dots\dots = \frac{wx}{24\,EI}(\ell^3 - 2\ell x^2 + x^3)$$

Figure 4.2.1.1.1 - Flexural Properties of Simply Supported Beam under Uniformly Distributed Load [1]



$$R = V \quad \dots\dots\dots\dots\dots = \frac{w\ell}{2}$$

$$V_x \quad \dots\dots\dots\dots\dots = w\left(\frac{\ell}{2} - x\right)$$

$$M_{max} \text{ (at ends)} \quad \dots\dots\dots = \frac{w\ell^2}{12}$$

$$M_1 \text{ (at center)} \quad \dots\dots\dots = \frac{w\ell^2}{24}$$

$$M_x \quad \dots\dots\dots\dots\dots = \frac{w}{12}(6\ell x - \ell^2 - 6x^2)$$

$$\Delta_{max} \text{ (at center)} \quad \dots\dots\dots = \frac{w\ell^4}{384EI}$$

$$\Delta_x \quad \dots\dots\dots\dots\dots = \frac{wx^2}{24EI}(\ell - x)^2$$

Figure 4.2.1.1.2 - Flexural Properties of Both Ends fixed beam under Uniformly Distributed Load [1]

In reality, none of the boundary conditions will be fully fixed or fully released. In most of the applications, error made because of this difference will be taken care of by the reduction factors or safety factors in codes. Popular finite element programs have an option available for this. If this becomes critical in design, it can be set up accordingly.

The images below show typical behavior encountered in bridges. By knowing these properties listed below, using a combination of two, it is possible to predict the flexural behavior of the bridge in consideration under linear-elastic conditions.

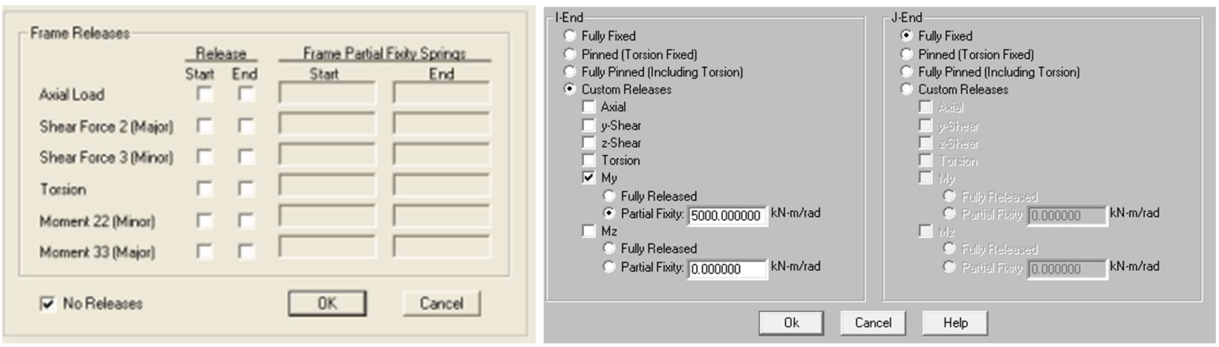


Figure 4.2.1.1.3 - Finite Element Programs

Another important concept to mention is the bending moments experienced by Continuous simply supported structures at interior supports. As an example, below is bending moments experienced by a simply supported continuous beam having 12 m, 18 m and 12 m spans. Under uniform loading of 100 kN/m, it can be seen that there are moments at the interior supports.

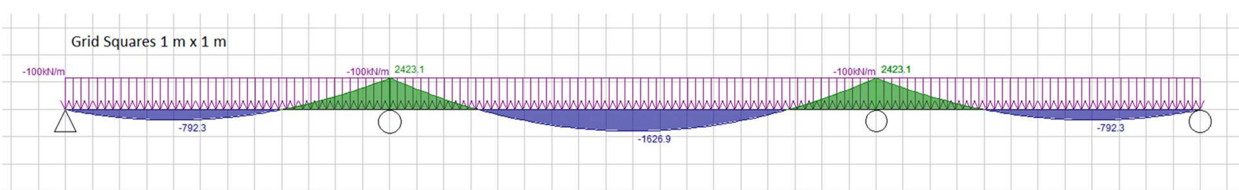


Figure 4.2.1.1.4 - Bending Moments in a Continuous Simply Structure

4.2.1.2 Influence Lines

A considerable portion of loading on the bridges are live loads. As the force applied is changing location due to a moving truck for example, the moments and shear experienced by the bridge changes. The moment and shear diagram created by a unit force at each location is called influence lines. Influence lines can be multiplied by the required factor to represent the load coming from a moving trucks wheel. The addition of the points where the absolute maximum occurs as the load moves forms a portion of the live load design loads.

There are various methods and theorems to calculate influence lines. However luckily, usually for concrete and prestressed concrete bridge design, checking every 10% of the span is enough. Thus, a brute force method can be easily applied. Below, the change in bending moments as the load moves from 10% to 50% can be seen.

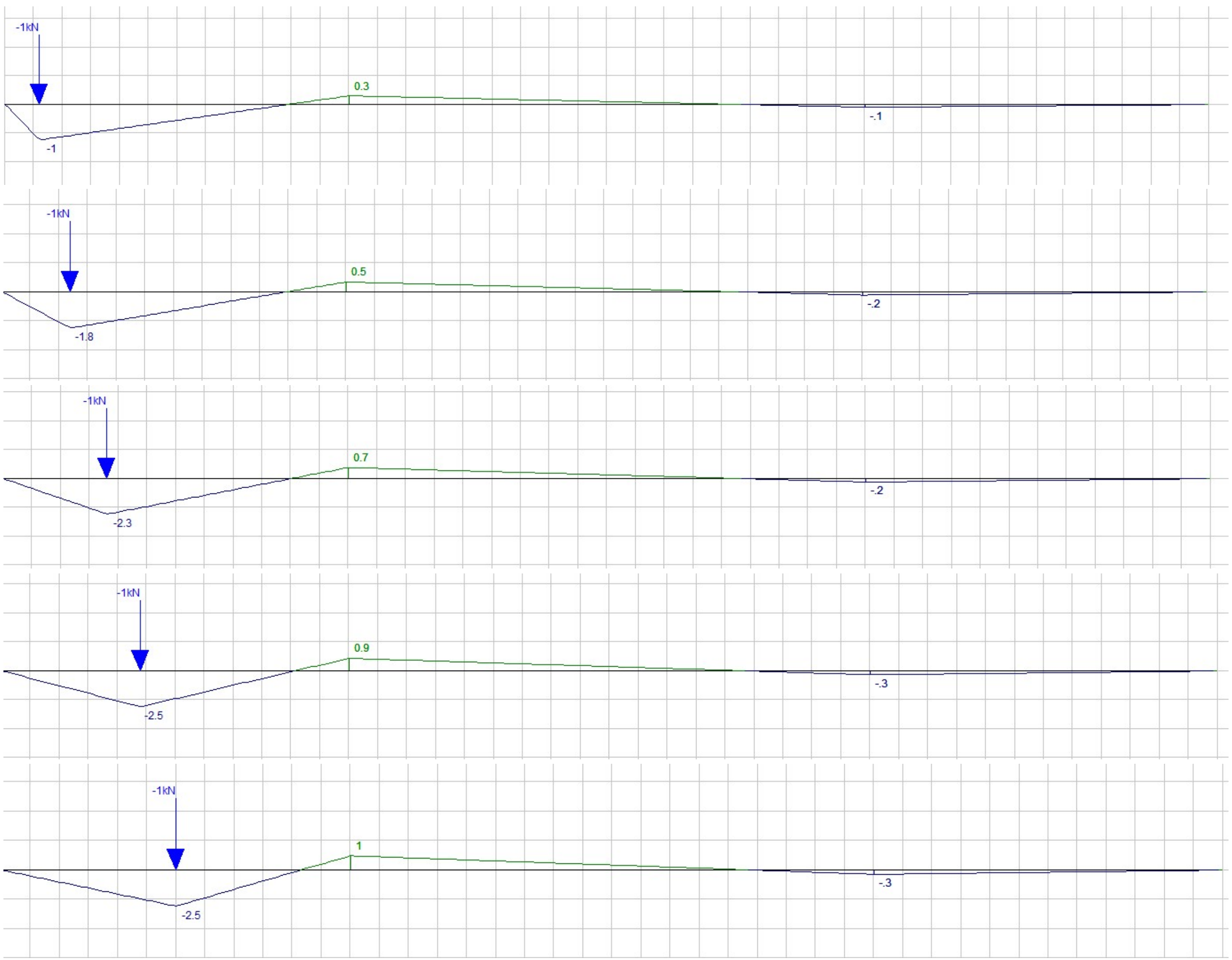


Figure 4.2.1.2.1- Pattern for Influence Lines of the 3-span beam mentioned above (1 m x 1 m grid squares)

The unit load can be applied as a distributed load to different spans. The moment diagram occurring from this unit distributed load in each location is called a live load envelope. It is very similar in concept to influence lines. Dead load can be added to the envelope or can be calculated separately. Depending on the county and code used, calculation methods vary.

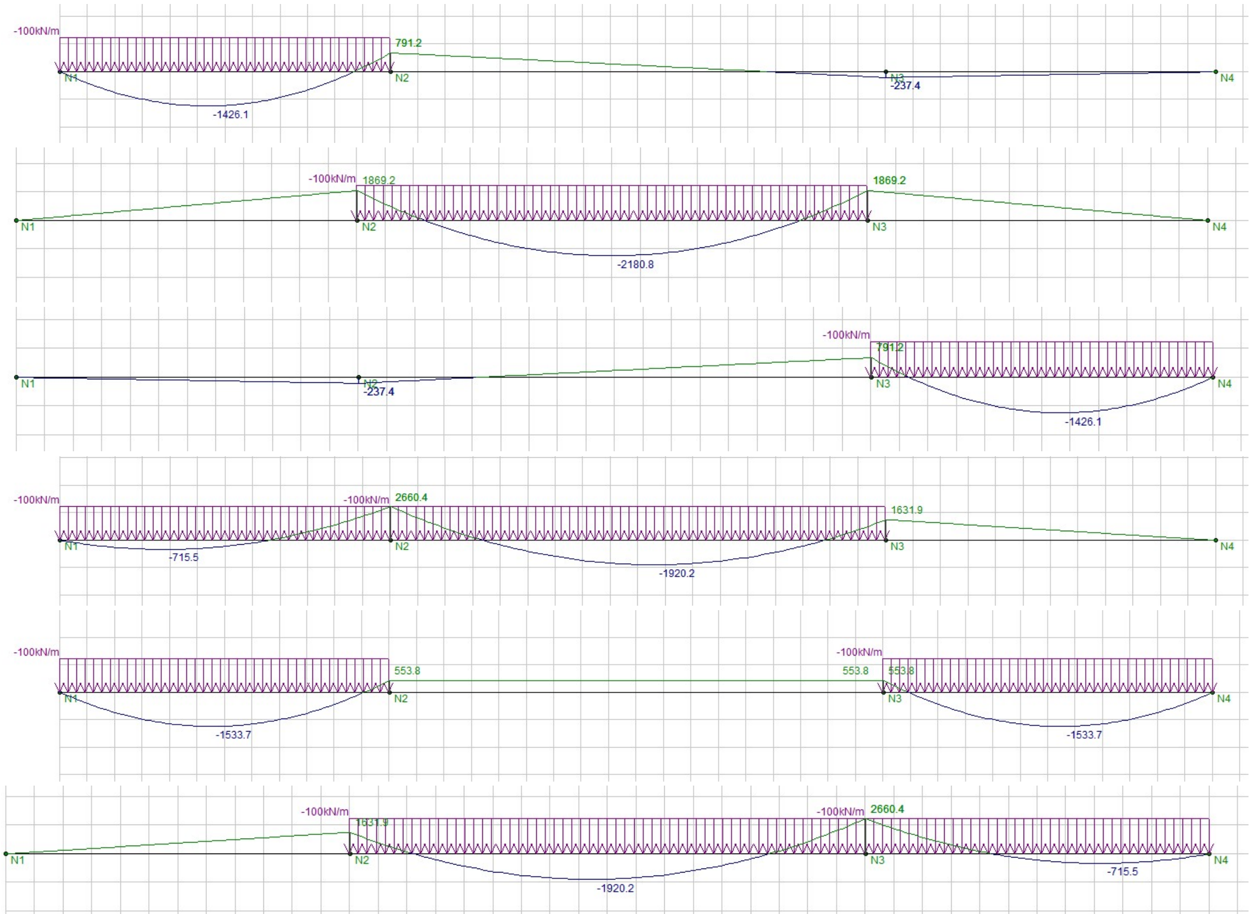


Figure 4.2.1.2.2- Live Load Envelopes for the 3-span beam mentioned above (1 m x 1 m grid squares, 100 kN/m as unit load)

4.2.1.3 Torsion

Torsion is the twisting of an object around its own axis due to torque applied. Torsion is usually caused by dynamic loading. Wind loads and earthquake loads can cause torsion on bridges. Torsion is usually a concern when designing pier columns for short span bridges in earthquake prone areas and for suspension and long span bridges. In 1940, Tacoma bridge collapsed due to torsional vibrations. In linear-elastic analysis, all structure members can be thought as springs with attached mass. If a spring is pushed to a point and released, it will start vibrating. A bridge is made of many springs. Many of these springs will be vibrating at a different frequency when pushed to a point and released. If an external force is applied at a frequency very close to the natural frequency of many critical members (springs) together, the amplitude of deflection will be significantly higher than it would be normally. These special vibrations are called modes and can be measured using ambient vibrations with the help of an accelerometer using frequency domain analysis. In 1940, the site of Tacoma Bridge, the wind speed was enough to cause Flutter effects.

Figure 4.2.1.3.1- Tacoma Narrows Bridge, 1940 [2]

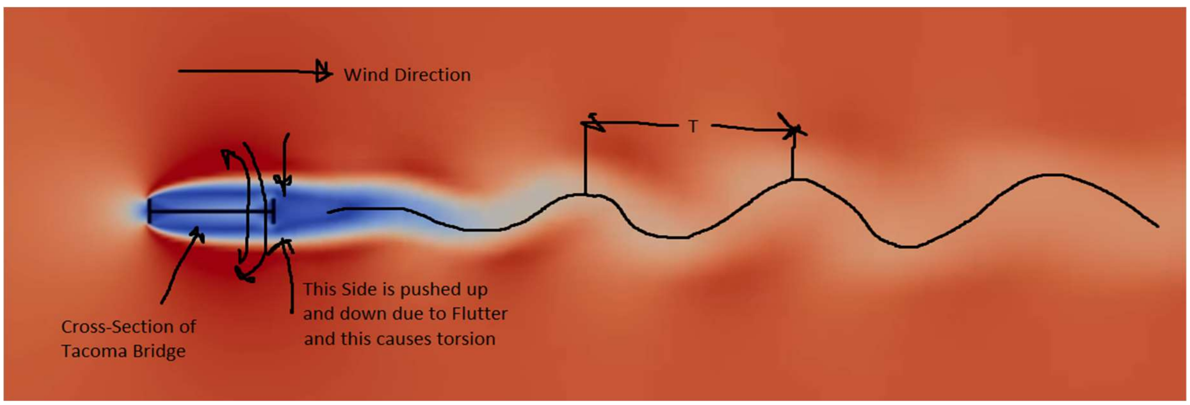


Figure 4.2.1.3.2- 2D Computational Fluid Dynamics simulation of Tacoma Bridge [3]

The frequency of torsional vibrations increased over time and approached one of the modes of the bridge. With the increasing amplitude of vibrations, cables gave up resulting in a collapse.

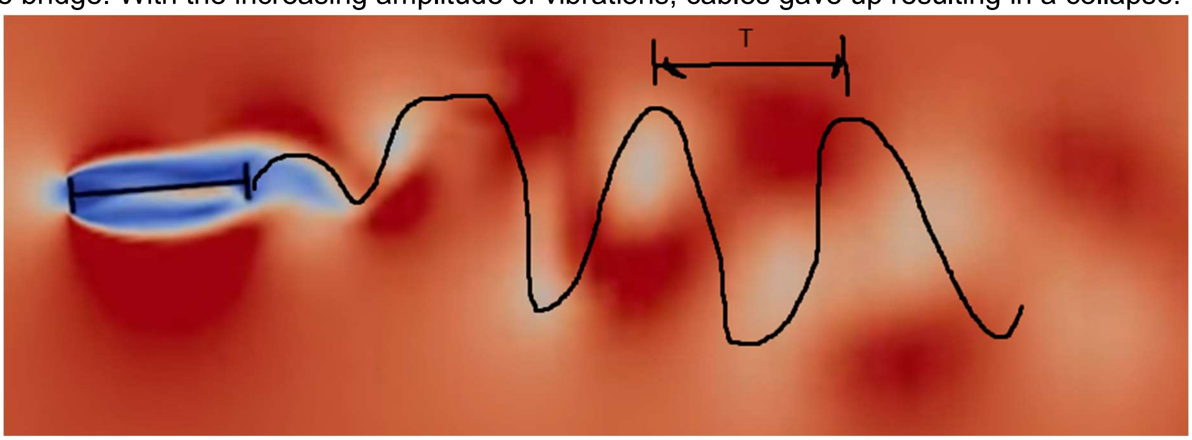


Figure 4.2.1.3.3- 2D Computational Fluid Dynamics simulation of Tacoma Bridge (close to failure) [3]

4.2.2 Bridge Loading

When building a bridge, engineers should consider three main types of loading: dead loads, live loads, and dynamic loads. This will help determine what material should be utilized to construct the bridge and what type of structure will best resist the loads throughout its service life. The bridge location and its purpose help determine the possible loads that are expected to act on the bridge.

4.2.2.1 Dead Load

Dead load is the self-weight of the structure - all the segments and materials used to construct the bridge come under permanent dead load. It's a load which doesn't move, that's how the name dead load was used. For Example, in the case of Florida International University Pedestrian Bridge collapse mentioned in chapter 2, the bridge failed to carry the dead load of the structure and collapsed.

4.2.2.2 Live Load

Live load is associated with the moving weight of the bridge that is the weight of the traffic applied on the bridge. This also includes loads created through maintenance as well as environmental conditions such as temperature, precipitation and snow for an accurate estimation of the live load. It is also called temporary loads that are applied to the bridge.

4.2.2.3 Dynamic Load

Dynamic loads are loads that are outside forces that don't occur on a daily basis. Although, they have a greater impact than live loads. This includes high wind speed, extreme weather conditions such as earthquakes and hurricanes and strong vibrations.

4.3 CSA-S6-14 rev. 17 - Canadian Highway Bridge Design Code

The following sections will introduce and summarize the design criteria, design requirements, design input, material technology and design methodology included in Canadian Highway and Bridge Design Code CSA S6-14 rev.17. All the bridge standards and code details and images in this section are taken from CSA-S6-14 rev. 17 [1].

4.3.1 Scope

The CSA-S6-14 standard applies to the design, evaluation, and structural rehabilitation design of fixed and movable highway bridges in Canada. There is no limit on span length, but the Code does not necessarily cover all aspects of design for every type of long-span bridge. poles and sign support structures. The Code does not apply to public utility structures or bridges used solely for railway or rail transit purposes.

4.3.2 Terminology

In this Code, “shall” is used to express a requirement, a provision that the user is obliged to satisfy in order to comply with the Code; “should” is used to express a recommendation or that which is advised but not required; and “may” is used to express an option or that which is permissible within the limits of the Code.

4.3.3 Design Philosophy

The primary concern shall be the safety of the public, including that of construction and maintenance workers. Design shall be based on the consideration of limit states in which, at the ultimate limit state, the factored resistance shall exceed the total factored load effect. Structural components shall be designed to comply with the ultimate limit state, serviceability limit state, and fatigue limit state requirements of this code. The design life of new structures shall be 75 years unless otherwise stated.

After safety, the total projected lifetime cost shall be the determining consideration in selecting the type of structure, configuration of spans and supports, and construction materials. This cost shall include allowances for inspection, maintenance, repair, and rehabilitation throughout the design life of the structure.

Bridges, culverts, and their associated works shall be designed to comply with all environmental requirements established for the site. Wherever possible, features of archaeological, historical, and cultural importance shall be preserved.

In the design and rehabilitation of structures, consideration shall be given to the appearance of the finished structure and its compatibility with its surroundings. Wherever possible, the appearance of a structure shall be such that it will be generally perceived as an enhancement to its surroundings.

4.3.4 Load Factors and Load Combinations

This section specifies the CSA-S6-14 rev. 17 loads, load factors, and load combinations that are used in calculating load requirements for the given design structure. It also includes requirements associated with vibration of highway and pedestrian bridges and requirements related to the construction loads and temporary structures. These apply to partially completed structures and structures necessary for construction purposes.

The different load combinations factors that should be considered are shown in Figures 4.3.4.1, 4.3.4.2 and 4.3.4.3.

	Permanent loads			Transitory loads					Exceptional loads			
Loads	***D***	***E***	***P***	***L****	***K***	***W***	***V***	***S***	***EQ***	***F***	***A***	***H***
Fatigue limit state												
FLS Combination 1	1.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	Table 3.2	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	Table 3.2	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	Table 3.2	1.00	0.45§	0.45	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.40§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.75§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

For the construction live load factor, see Clause 3.16.3.
†*For superstructure vibration only.*
‡*For ultimate limit states, the maximum or minimum values of α_D, α_E, and α_P specified in Table 3.3 shall be used.*
§*For wind loads determined from wind tunnel tests, the load factors shall be as specified in Clause 3.10.5.2.*
***For long spans, it is possible that a combination of ice load F and wind load W will require investigation.*

Legend:
A = ice accretion load
D = dead load
E = loads due to earth pressure and hydrostatic pressure, including surcharges but excluding dead load
EQ = earthquake load
F = loads due to stream pressure and ice forces or to debris torrents
H = collision load arising from highway vehicles or vessels
K = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains
L = live load (including the dynamic load allowance, when applicable), including barrier loads
P = secondary prestress effects
S = load due to differential settlement and/or movement of the foundation
V = wind load on traffic
W = wind load on structure

Figure 4.3.4.1- Load Factors and Load Combinations in CSA-S6-14 rev.17

	Live load factor				
		Special loads mixed with normal traffic		Special loads travelling alone on bridge under supervision	
Load	**Normal traffic**	**Short spans**	**Other spans**	**Short spans**	**Other spans**
ULS combination 1	1.70*	1.70	1.50	1.50	1.35
ULS combination 2	1.60	1.60	1.40	1.40	1.25
ULS combination 3	1.40	1.40	1.25	1.25	1.10

Also to be applied to the barrier loads.

Figure 4.3.4.2- Live load factors ultimate limit states in CSA-S6-14 rev.17

Dead load	Maximum α_D	Minimum α_D
Factory-produced components, excluding wood	1.10	0.95
Cast-in-place concrete, wood, and all non-structural components	1.20	0.90
Wearing surfaces, based on nominal or specified thickness	1.50	0.65
Earth fill, negative skin friction on piles	1.25	0.80
Water	1.10	0.90
Dead load in combination with earthquakes	**Maximum α_D**	**Minimum α_D**
All dead loads for ULS Combination 5 (see Table 3.1)	1.25	0.80
Earth pressure and hydrostatic pressure	**Maximum α_E**	**Minimum α_E**
Passive earth pressure, considered as a load*	1.25	0.50
At-rest earth pressure	1.25	0.80
Active earth pressure	1.25	0.80
Backfill pressure	1.25	0.80
Hydrostatic pressure	1.10	0.90
Prestress	**Maximum α_P**	**Minimum α_P**
Secondary prestress effects	1.05	0.95

**When passive earth pressure is considered as a resistance, it is factored in accordance with Section 6.*

Figure 4.3.4.3- Permanent loads — Maximum and minimum values of load factors for ULS in CSA-S6-14 rev. 17

4.3.4.1 Dead Loads

CSA-S6-14 rev. 17states that dead loads shall include the weight of all components of the structure and appendages fixed to the structure, including wearing surface, earth cover, and utilities. The unit material weights are set out as shown Figure 4.3.4.1.1 and should be used in case of absence of more precise information. The code also states that the weight of water shall be considered dead load.

Material	Unit weight, kN/m^3
Aluminum alloy	27.0
Bituminous wearing surface	23.5
Concrete	
Low-density concrete	18.1
Semi-low-density concrete	21.0
Plain concrete	23.5
Prestressed concrete	24.5
Reinforced concrete	24.0
Coarse-grained (granular) soil	22.0
Crushed rock	22.0
Fine-grained sandy soil	20.0
Glacial till	22.0
Rockfill	21.0
Slag	
Air-cooled slag	11.0
Water-cooled slag	15.0
Steel	77.0
Water	
Fresh water	9.8
Salt or polluted water	10.5
Wood	
Hardwood	9.5
Softwood	6.0

Figure 4.3.4.1.1- Unit material weights in CSA-S6-14

4.3.4.2 Live Loads

CSA-S6-14 rev. 17states that the number of design lanes for traffic shall be determined from Figure 4.3.4.2.1. The width of each design lane (W_e) should be equal to deck width (W_c) divided by the number of design lanes (n). $\boldsymbol{W_e = W_c/n}$.

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.

Figure 4.3.4.2.1- Number of design lanes in CSA-S6-14

CSA-S6-14 rev. 17states that the live load is composed of the following loads: traffic load, curb load, barrier load and pedestrian load. The traffic load is calculated using CL-W truck loading and CL-W lane load. The CL-W truck is an idealized five axle truck as shown in Figure 4.3.4.2.2. The W represents the gross load of the CL-W truck in kilonewtons. CSA-S6-14 rev. 17states three key rules for CL-W loading: First, a loading of not less than CL-625 shall be used for the design of a national highway network that is generally used for interprovincial transportation. Second, a loading exceeding CL-625 may be specified by a provincial or territorial authority for the design of certain bridges within the province or territory. Third, loadings lesser or greater than CL-625 shall be used only where justified by traffic conditions and shall require approval.

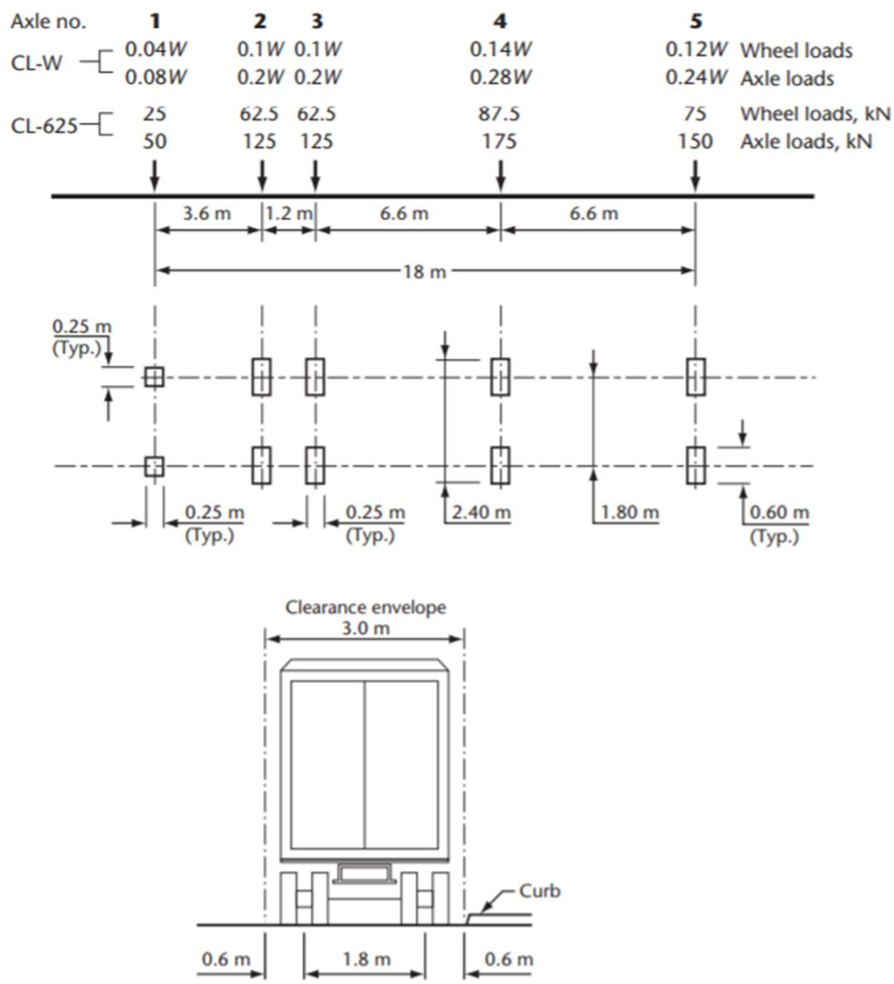


Figure 4.3.4.2.2- CL-W Truck in CSA-S6-14

CSA-S6-14 states that the CL-W Lane load consists of a CL-W Truck with each axle reduced to 80% of the value and superimposed within a uniformly distributed load of 9 kN/m and 3.0 m wide. Figure 4.3.4.2.3 shows the CL-W Lane Load.

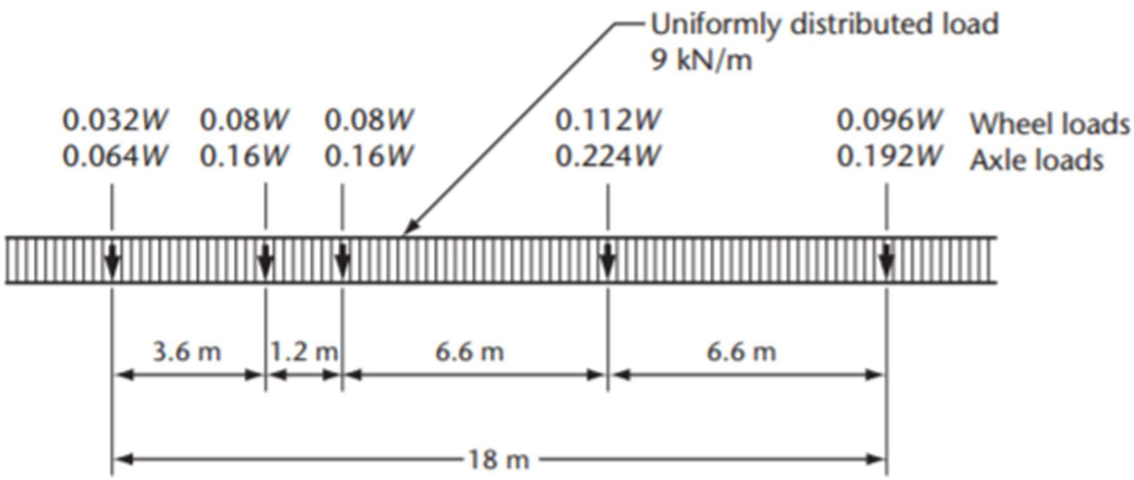


Figure 4.3.4.2.3- CL-W Lane Load in CSA-S6-14

4.3.4.3 Wind Loads

CSA-S6-14 rev. 17 states that the wind load for a superstructure is contributed by two components: horizontal drag load and vertical load.

The horizontal drag load is applied horizontally to the frontal area of the superstructure, it is calculated using equation shown below:

$F = qCeCgCh$

where, q, C_e, and C_g change depending on the situation and $C_h = 2.0$.

The vertical load is applied vertically to the plan area of the superstructure, it is calculated using equation shown below:

$F = qCeCgC$v

Where, q, C_e, and C_g change depending on the situation and $C_v = 1.0$.

4.3.4.4 Earthquake Effects

CSA-S6-14 rev. 17 states that seismic design shall be carried out using the performance-based design approach with criteria based on meeting specific structural, functional, and service performance criteria under specified seismic hazards. A force-based design approach shall be carried out for ductile substructure elements. Earthquake load effects for capacity-protected members shall be determined from elastic design forces or in accordance with capacity design principles for forces resulting from inelastic action of members with which they connect.

Bridges are assigned to one of three seismic performance categories 1 to 3 based on the site-specific spectral acceleration, for a return period of 2475 years as shown in Figure 4.16. CSA-S6-14 rev. 17 states that bridges in seismic category 1 need not be analyzed for seismic loads, regardless of their importance and geometry. In Seismic Performance Categories 2 and 3, all bridges except single span girder bridges shall be analyzed which also includes multi-span bridges.

		Seismic performance category	
For $T < 0.5$ s	For $T \geq 0.5$ s	Lifeline bridges	Major-route and other bridges
$S(0.2) < 0.20$	$S(1.0) < 0.10$	2	1
$0.2 \leq S(0.2) < 0.35$	$0.10 \leq S(1.0) < 0.30$	3	2
$S(0.2) \geq 0.35$	$S(1.0) \geq 0.30$	3	3

Figure 4.3.4.3.1- Seismic performance category based on 2475-year return period spectral values

4.3.5 Reinforced Concrete

This section states the CSA-S6-14 rev. 17design requirements for structural components made out of reinforced concrete with prestressed or non-prestressed steel. Concrete can be precast or cast in place and can have normal density, low density or semi low density.

4.3.5.1 Concrete Strength

CSA-S6-14 rev.17 states that the specified strength of concrete f'c, shall be a minimum of 30 MPa for non-prestressed members and a minimum of 35 MPa for prestressed members unless otherwise approved. However, concrete with strengths greater than 85 MPa shall be used only if approved.The modulus of elasticity of concrete, Ec is calculated using the equation shown below:

$$E_c = (3000 \times \sqrt{f_c'} + 6900) \times (\gamma_c / 2300)^{1.5}$$

where

E_c = modulus of elasticity of concrete in MPa

F'_c = specified compressive strength of concrete in MPa

γ_c = mass density of concrete in kg/m^3

4.3.5.2 Shrinkage of Concrete

CSA-S6-14 rev.17 states that the shrinkage strain of concrete can be calculated in two ways:

a) strain, ε_{cs}, due to shrinkage that develops in an interval of time, t-t_0, shall be calculated as shown below:

$$\varepsilon_{cs}(t-t_0) = \varepsilon_{cs0}\beta_s(t-t_0)$$

where

ε_{cs0} = notional shrinkage coefficient

$$= \beta_{RH}\left[160+50\left[9-\frac{f_c'+a}{10}\right]\right]\times 10^{-6}$$

where

$$\beta_{RH} = -1.55\left[1-\left[\frac{RH}{100}\right]^3\right]$$

a = difference between mean concrete strength and specified strength, f_c', at 28 days (in the absence of data from the concrete that is to be used, a may be taken as 10 MPa)

RH = annual mean relative humidity, %, as shown in Figure A3.1.3

$\beta_s(t-t_0)$, which describes the development of shrinkage with time, shall be calculated as follows:

$$\beta_s(t-t_0) = \sqrt{\frac{t-t_0}{350\left[\frac{2r_v}{100}\right]^2+(t-t_0)}}$$

b) Based on data obtained from physical tests on the same mix of concrete that is to be used in construction.

4.3.5.3 Creep of Concrete

CSA-S6-14 rev. 17 states that creep strains in normal-density concrete shall be calculated in 2 ways:

a) for structural components with serviceability limit state compressive stresses less than 0.4 x f'_c the total time-varying strain, $\varepsilon_{c\sigma}(t,t_0)$, due to a constant stress, $\sigma_c\,(t_0)$, applied at time t0 shall be calculated as follows:

$$\varepsilon_{c\sigma}(t,t_0) = \sigma_c(t_0)\left[\frac{1}{E_c(t_0)} + \frac{\phi(t,t_0)}{E_{c,28}}\right]$$

where

$E_c(t_0)$ = modulus of elasticity of concrete at time of loading

$\phi(t,t_0)$ = creep coefficient as specified in Clause 8.4.1.6.3

$E_{c,28}$ = modulus of elasticity of concrete at 28 days

The principle of superposition may be used to calculate strains due to a time-varying stress.

b) Based on data obtained from physical tests on the same mix of concrete that is to be used in construction.

4.3.5.4 Other properties of Concrete

a) Poisson's ratio for elastic strains shall be taken as 0.2 or 0.15, unless otherwise approved
b) Thermal coefficient of linear expansion of concrete shall be taken as 10^{-5} C.
c) Cracking strength for concrete f_{cr}:

$0.4\sqrt{f'_c}$ for normal-density concrete;

$0.34\sqrt{f'_c}$ for semi-low-density concrete; and

$0.30\sqrt{f'_c}$ for low-density concrete.

4.3.5.5 Steel Reinforcement

In CSA S16-14 rev. 17, the spacing of reinforcements is enforced as follows:

8.14.2 Spacing of reinforcement

8.14.2.1 Reinforcing bars

8.14.2.1.1

For cast-in-place concrete, the clear distance between parallel bars in a layer or a ring shall be not less than
(a) 1.5 times the nominal diameter of the bars;
(b) 1.5 times the maximum size of the coarse aggregate; and
(c) 40 mm.

8.14.2.1.2

For precast concrete, the clear distance between parallel bars in a layer or a ring shall be not less than
(a) the nominal diameter of the bars;
(b) 1.33 times the maximum size of the coarse aggregate; and
(c) 25 mm.

8.14.2.1.3

For parallel reinforcing bars placed in two or more layers, with a clear distance between layers of not more than 150 mm, the bars in the upper layers shall be placed directly above those in the lower layers (except in deck slabs). The clear distance between layers shall be not less than
(a) 25 mm; and
(b) the nominal diameter of the bars.

8.14.2.1.4

The clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8.14.2.1.5

In walls and slabs, primary flexural reinforcement shall be spaced not farther apart than
(a) 1.5 times the thickness of the component; and
(b) 450 mm.

The maximum spacing of hoops, spirals, ties and shrinkage, and temperature reinforcement shall satisfy Clauses 8.12.6, 8.14.3, and 8.14.4.

Figure 4.3.5.5.1- Spacing of Reinforcement

-See commentary appendix section 4.8 for an example.

4.3.5.6 Shear Reinforcement

Meaning of symbols used in this section:

V_c = Factored shear resistance provided by concrete
V_p = Factored shear resistance provided by vertical component of effective prestressing force
V_s = Factored shear resistance provided by transverse reinforcement steel
f_{cr} = Modulus of rupture of concrete

b_v = Effective web width within depth
f_y = Yield strength of steel used
s = Spacing
d_v = Effective shear depth
f'_c = Maximum compressive strength of concrete

Material resistance factors

Material	Material resistance factor
Concrete	$\phi_c = 0.75$
Reinforcement	
Reinforcing bars, wire, and wire fabric	$\phi_s = 0.90$
Prestressing strands	$\phi_p = 0.95$
High-strength bars	$\phi_p = 0.90$
Anchor rods and studs	In accordance with Section 10

Figure 4.3.5.6.1- Material Resistance Factors

8.9.1.2 Regions requiring transverse reinforcement

Except for solid slabs, walls, and footings, transverse reinforcement shall be provided in all regions where V_f is greater than $(0.20\phi_c f_{cr} b_v d_v + 0.5\phi_p V_p)$.

The minimum shear transverse reinforcement is specified as follows in CSA-S6-14:

8.9.1.3 Minimum amount of transverse reinforcement

When calculations show that transverse shear reinforcement is required, A_v shall not be less than $0.15 f_{cr}(b_v s/f_y)$.

According to CSA-S6-14, the factored shear resistance is determined by the following equation:

8.9.3.3 Factored shear resistance

The factored shear resistance, V_r, shall be calculated as $V_c + V_s + V_p$.

Use MPa - mm - N as units

According to CSA-S6-14 factored shear resistance should be calculated:
Maximum factored shear capacity:

$$V_{r,\max} = \underbrace{0.25\varphi_c f'_c b_v d_v}_{=\left(V_c + V_s\right)_{max}} + V_p$$

Determination of factored shear resistance provided by vertical transverse shear reinforcement is the following according to CSA S6-14:

8.9.3.5 Determination of V_s

V_s shall be determined as follows:

For components with transverse reinforcement perpendicular to the longitudinal axis, V_s shall be calculated as follows:

$$V_s = \frac{\phi_s f_y A_v d_v \cot\theta}{s}$$

-See commentary appendix section 4.8 for an explanation.

Determination of factored shear resistance provided by concrete is the following according to CSA S6-14:

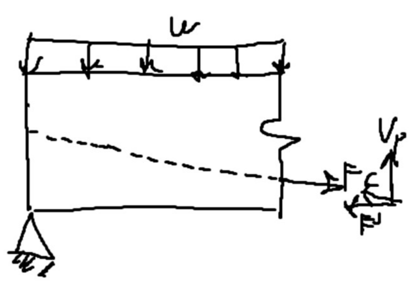
8.9.3.4 Determination of V_c

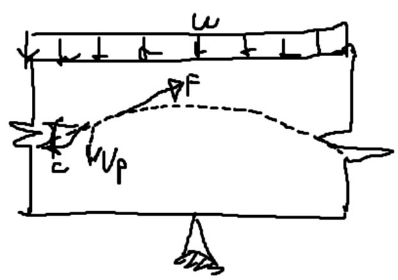
V_c shall be calculated as $2.5\beta\,\phi_c\,f_{cr}\,b_v\,d_v$. However, f_{cr} shall not be greater than 3.2 MPa.

-See commentary appendix section 4.8 for an explanation.

Determination of V_p:

Vp can be found using the effective prestressing force (after losses and anchorage slip) and the angle between horizontal. Refer to the drawings below:





Vp is positive if resisting shear, else negative.

4.3.6 Prestressed Concrete

CSA-S6-14 rev. 17 states that the tendons shall be stressed to provide a minimum effective prestress of 0.45 x f_{pu}, f_{pu} is the specified tensile strength of prestressing steel. The stress in the tendons shall not exceed the values specified in Figure 4.3.6.1.

	Tendon type		
		High-strength bar	
	Low-relaxation strand	Smooth	Deformed
At jacking			
Pretensioning	$0.78f_{pu}$	—	—
Post-tensioning	$0.80f_{pu}$	$0.76f_{pu}$	$0.75f_{pu}$
At transfer			
Pretensioning	$0.74f_{pu}$	—	—
Post-tensioning			
At anchorage and couplers	$0.70f_{pu}$	$0.70f_{pu}$	$0.66f_{pu}$
Elsewhere	$0.74f_{pu}$	$0.70f_{pu}$	$0.66f_{pu}$

Figure 4.3.6.1- Prestressing tendon stress limits in CSA-S6-14 rev. 17

Table A shows the concrete strength at transfer from tendons to concrete for pretensioned and post-tensioned components according to CSA-S6-14 rev. 17.

Table 4.3.6.1- Concrete Strength at Transfer

Tendons	Force in Tendons shall not be transferred to concrete until the compressive strength of concrete
Pretensioned Components	at least 25 MPa
Post-Tensioned Components	at least 20 MPa

According to CSA-S6-14 rev. 17, The following prestress loss shall be considered in the calculation process

(a) anchorage slip and friction.

(b) elastic shortening of concrete.

(c) relaxation of tendons.

(d) creep of concrete.

(e) shrinkage of concrete; and
(f) any other special circumstances

According to CSA-S6-14, The friction loss between tendons and the sheath, Delta f, at a distance x from the jacking end shall be calculated as:

$$f_{sj} = 1 - e^{-(Kx + \mu\alpha)}$$

-See commentary appendix section 4.8 for derivation.

The values of K and μ are specified in Table 4.3.6.2.

Table 4.3.6.2- Friction Factors in CSA-S6-14 rev. 17

	Strand		Smooth bar		Deformed bar	
Sheath type	***K***	***μ***	***K***	***μ***	***K***	***μ***
Internal ducts						
Rigid steel	0.002	0.18	—	—	—	—
Semi-rigid steel over 75 mm outside diameter	0.003	0.20	—	—	—	—
Semi-rigid steel up to 75 mm outside diameter	0.005	0.20	0.003	0.20	0.003	0.30
Plastic	0.001	0.14	—	—	—	—
External ducts						
Straight plastic	0.000	—	—	—	—	—
Rigid steel pipe deviators	0.002	0.25	—	—	—	—

4.3.6.1 Flexural and Axial Reinforcement

According to CSA-S6-14 rev. 17, minimum and maximum reinforcement for flexural and axial loading shall be as specified in Table B. M_r = Factored flexural resistance

Table 4.3.6.1.1- Flexural and Axial Reinforcement

Reinforcement	Limitation Criteria
Minimum Reinforcement	M_r at least 1.20 times the cracking moment
Maximum Reinforcement	M_r is developed with c/d not exceeding 0.5

4.3.7 Durability Design

This section will consider the durability of the structure to remain functional, with less maintenance required during its designed life. According to CSA-S6-14 rev. 17, the following deterioration mechanisms must be considered for design durability:

(a) Carbonation-induced corrosion without chloride.
(b) Chloride-induced corrosion due to seawater.
(c) Chloride-induced corrosion from sources other than seawater.
(d) Freeze-thaw deterioration.
(e) Alkali aggregate reaction.
(f) Chemical attack and
(g) abrasion.

Table 8.5 in the CSA-S6-14 rev. 17 is a good reference for durability design.

Table 4.3.7.1- Minimum Concrete cover and Tolerances

Table 8.5
Minimum concrete covers and tolerances
(See Clause 8.11.2.2.)

Environmental exposure	Component	Reinforcement/ steel ducts	Concrete covers and tolerances: Cast-in-place concrete, mm	Concrete covers and tolerances: Precast concrete, mm
De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray	(1) Top of bottom slab for rectangular voided deck	Reinforcing steel	40 ± 10	40 ± 10
		Pretensioning strands	—	55 ± 5
		Post-tensioning ducts	60* ± 10	60* ± 10
	(2) Top surface of buried structure with less than 600 mm fill† Top surface of bottom slab of buried structure	Reinforcing steel	70 ± 20	50 ± 10
		Pretensioning strands	—	65 ± 5
		Post-tensioning ducts	90* ± 15	70* ± 10
	(3) Top surface of structural component, except (1) and (2)‡	Reinforcing steel	70 ± 20	55 ± 10
		Pretensioning strands	—	70 ± 5
		Post-tensioning ducts		
		Longitudinal	130* ± 15	120* ± 10
		Transverse ($d_d \leq 60$ mm	90* ± 15	80* ± 10
		Transverse ($d_d > 60$ mm)	130* ± 15	120* ± 10
	(4) Soffit of precast deck form	Reinforcing steel	—	40 ± 10
		Pretensioning strands	—	38 ± 3
	(5) Soffit of slab less than 300 mm thick or soffit of top slab of voided deck	Reinforcing steel	50 ± 10	45 ± 10
		Pretensioning strands	—	60 ± 5
		Post-tensioning ducts	70* ± 10	65* ± 10
	(6) Soffit of slab 300 mm thick or thicker or soffit of structural component, except (4) and (5)	Reinforcing steel	60 ± 10	50 ± 10
		Pretensioning strands	—	65 ± 5
		Post-tensioning ducts	80* ± 10	70* ± 10
	(7) Vertical surface of arch, solid or voided deck, pier cap, T-beam, or interior diaphragm	Reinforcing steel	70 ± 10	60 ± 10
		Pretensioning strands	—	75 ± 5
		Post-tensioning ducts	90* ± 10	80* ± 10
	(8) Inside vertical surface of buried structure or inside surface of circular buried structure	Reinforcing steel	70 ± 20	50 ± 10
		Pretensioning strands	—	65 ± 5
		Post-tensioning ducts	90* ± 15	70* ± 10
	(9) Vertical surface of structural component, except (7) and (8)	Reinforcing steel	70 ± 20	55 ± 10
		Pretensioning strands	—	70 ± 5
		Post-tensioning ducts	90* ± 15	75* ± 10
	(10) Precast T-, I-, or box girder	Reinforcing steel	—	35 +10 or –5
		Pretensioning strands	—	50 ± 5
		Post-tensioning ducts	—	55* ± 10

4.3.7.1 Concrete Quality

CSA-S6-14 rev. 17 states that the maximum cement to water ratio should be specified as shown in Table 4.3.7.1.1.

Table 4.3.7.1.1 - Maximum water to cementing materials ratio

Deterioration mechanism	Environmental exposure	Maximum ratio*†‡
Chloride-induced corrosion	Marine	
	Airborne salts	0.45
	Tidal and splash spray	0.45
	Submerged	0.40
	Other than marine	
	Wet, rarely dry	0.40
	Dry, rarely wet	0.40
	Cyclic, wet/dry	0.40
Freeze-thaw attack§	Unsaturated	0.45
	Saturated	0.40
Carbonation-induced corrosion without chloride	Wet, rarely dry	0.50
	Dry, rarely wet	0.50
	Cyclic, wet/dry	0.45

**Unless otherwise Approved.*
†Water to cementing materials ratio by mass. Cementing materials include Portland cement, silica fume, fly ash, and slag.
‡The ratio shall be independently verified on the submitted concrete mix design and concrete materials. Quality control and quality assurance measures shall be taken to ensure uniformity of concrete production so that water/cement limits are maintained throughout production. Such measures shall include measurements of slump, air content, unit weight, and strength.
§Air content shall be in accordance with CSA A23.1. The minimum air content shall be 5.5% for concrete in saturated conditions unless otherwise Approved.

4.3.7.2 Concrete Cover and Tolerances

CSA-S6-14 rev. 17 states that the minimum concrete cover and tolerances shall not be less than the values specified in Table 4.3.7.1.1 in order to prevent the reinforcement from corrosion or chemical effects.

Table 4.3.7.1.1 A- Minimum concrete covers and tolerances in CSA-S6-14 rev. 17

			Concrete covers and tolerances	
Environmental exposure	**Component**	**Reinforcement/ steel ducts**	**Cast-in-place concrete, mm**	**Precast concrete, mm**
De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray	(1) Top of bottom slab for rectangular voided deck	Reinforcing steel	40 ± 10	40 ± 10
		Pretensioning strands	—	55 ± 5
		Post-tensioning ducts	60* ± 10	60* ± 10
	(2) Top surface of buried structure with less than 600 mm fill† Top surface of bottom slab of buried structure	Reinforcing steel	70 ± 20	50 ± 10
		Pretensioning strands	—	65 ± 5
		Post-tensioning ducts	90* ± 15	70* ± 10
	(3) Top surface of structural component, except (1) and (2)‡	Reinforcing steel	70 ± 20	55 ± 10
		Pretensioning strands	—	70 ± 5
		Post-tensioning ducts		
		Longitudinal	130* ± 15	120* ± 10
		Transverse ($d_d \le 60$ mm	90* ± 15	80* ± 10
		Transverse ($d_d > 60$ mm)	130* ± 15	120* ± 10
	(4) Soffit of precast deck form	Reinforcing steel	—	40 ± 10
		Pretensioning strands	—	38 ± 3
	(5) Soffit of slab less than 300 mm thick or soffit of top slab of voided deck	Reinforcing steel	50 ± 10	45 ± 10
		Pretensioning strands	—	60 ± 5
		Post-tensioning ducts	70* ± 10	65* ± 10
	(6) Soffit of slab 300 mm thick or thicker or soffit of structural component, except (4) and (5)	Reinforcing steel	60 ± 10	50 ± 10
		Pretensioning strands	—	65 ± 5
		Post-tensioning ducts	80* ± 10	70* ± 10
	(7) Vertical surface of arch, solid or voided deck, pier cap, T-beam, or interior diaphragm	Reinforcing steel	70 ± 10	60 ± 10
		Pretensioning strands	—	75 ± 5
		Post-tensioning ducts	90* ± 10	80* ± 10
	(8) Inside vertical surface of buried structure or inside surface of circular buried structure	Reinforcing steel	70 ± 20	50 ± 10
		Pretensioning strands	—	65 ± 5
		Post-tensioning ducts	90* ± 15	70* ± 10
	(9) Vertical surface of structural component, except (7) and (8)	Reinforcing steel	70 ± 20	55 ± 10
		Pretensioning strands	—	70 ± 5
		Post-tensioning ducts	90* ± 15	75* ± 10
	(10) Precast T-, I-, or box girder	Reinforcing steel	—	35 +10 or –5
		Pretensioning strands	—	50 ± 5
		Post-tensioning ducts	—	55* ± 10

(Continued)

Table 4.3.7.1.1 B- Minimum concrete covers and tolerances in CSA-S6-14 rev. 17

Environmental exposure	Component	Reinforcement/ steel ducts	Concrete covers and tolerances: Cast-in-place concrete, mm	Concrete covers and tolerances: Precast concrete, mm
No de-icing chemicals; no spray or surface runoff containing de-icing chemicals; no marine spray	(1) Top of bottom slab for rectangular voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	40 ± 10 — 60* ± 10	40 ± 10 55 ± 5 60* ± 10
	(2) Top surface of buried structure with less than 600 mm fill† or top surface of bottom slab of buried structure	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	40 ± 10 55 ± 5 60* ± 10
	(3) Top surface of structural component, except (1) and (2)‡	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	50 ± 10 70 ± 5 70 ± 10
	(4) Soffit of precast deck form	Reinforcing steel Pretensioning strands	— —	40 ± 10 38 ± 3
	(5) Soffit of slab less than 300 mm thick or soffit of top slab of voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	40 ± 10 — 60* ± 10	40 ± 10 55 ± 5 60* ± 10
	(6) Soffit of slab 300 mm thick or thicker or soffit of structural component, except (4) and (5)	Reinforcing steel Pretensioning strands Post-tensioning ducts	50 ± 10 — 70* ± 10	40 ± 10 55 ± 5 60* ± 10
	(7) Vertical surface of arch, solid or voided deck, pier cap, T-beam, or interior diaphragm	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 10 — 80* ± 10	50 ± 10 65 ± 5 70* ± 10
	(8) Inside vertical surface of buried structure or inside surface of circular buried structure	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	40 ± 10 55 ± 5 60* ± 10
	(9) Vertical surface of structural component, except (7) and (8)	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 20 — 80* ± 15	50 ± 10 70 ± 5 70* ± 10
	(10) Precast T-, I-, or box girder	Reinforcing steel Pretensioning strands Post-tensioning ducts	— — —	30 +10 or −5 45 ± 5 50* ± 10

(Continued)

Table 4.3.7.1.1 C- Minimum concrete covers and tolerances in CSA-S6-14 rev. 17

Environmental exposure	Component	Reinforcement/ steel ducts	Concrete covers and tolerances: Cast-in-place concrete, mm	Concrete covers and tolerances: Precast concrete, mm
Earth or fresh water	(1) Footing, pier, abutment, or retaining wall	Reinforcing steel	70 ± 20	55 ± 10
		Pretensioning strands	—	75 ± 5
		Post-tensioning ducts	90* ± 15	80* ± 10
	(2) Concrete pile	Reinforcing steel	—	40 ± 10
		Pretensioning strands	—	55 ± 5
		Post-tensioning ducts	—	60* ± 10
	(3) Caisson with liner	Reinforcing steel	60 ± 20	—
		Post-tensioning ducts	80* ± 15	—
	(4) Buried structure with more than 600 mm of fill†	Reinforcing steel	60 ± 20	40 ± 10
		Pretensioning strands	—	55 ± 5
		Post-tensioning ducts	80* ± 15	60* ± 10
Swamp, marsh, salt water, or aggressive backfill	(1) Footing, pier, abutment, or retaining wall	Reinforcing steel	80 ± 20	65 ± 10
		Pretensioning strands	—	85 ± 10
		Post-tensioning ducts	100* ± 15	90* ± 10
	(2) Concrete pile	Reinforcing steel	—	50 ± 10
		Pretensioning strands	—	65 ± 5
		Post-tensioning ducts	—	70* ± 10
	(3) Caisson with liner	Reinforcing steel	70 ± 20	—
		Pretensioning strands	—	—
		Post-tensioning ducts	90* ± 15	—
	(4) Buried structure with more than 600 mm of fill†	Reinforcing steel	70 ± 20	55 ± 10
		Pretensioning strands	—	70 ± 5
		Post-tensioning ducts	90* ± 15	80* ± 10
Cast against and permanently exposed to earth	(1) Footing	Reinforcing steel	100 ± 25	—
	(2) Caisson	Reinforcing steel	100 ± 25	—
		Post-tensioning ducts	120 ± 15	—
Various	Components other than those covered elsewhere in this Table	Reinforcing steel	70 ± 20§	55 ± 10§
		Pretensioning strands	—	70 ± 5§
		Post-tensioning ducts	90* ± 15§	80* ± 10§

**Or $0.5d_d$*, whichever is greater.*
†Buried structures with less than 600 mm of fill shall have a distribution slab.
‡For concrete decks without waterproofing and paving, increase the concrete cover by 10 mm to allow for wearing of the surface concrete.
§Or as Approved.

4.4 AASHTO

The American Association of State Highway and Transportation Officials LRFD-8th Edition code is meant as AASHTO in this section and AASHTO abbreviation will be used throughout the report. AASHTO is the American Bridge Design Specifications code and it is used as a reference in many other countries.

4.4.1 Scope

AASHTO aims to provide the minimum requirements for the designer to provide public safety. AASHTO uses Load and Resistance Factor Design (LFRD). LFRD is based on our current knowledge on statistical loading and structural performance. LFRD will later be explained in this section.

4.4.2 Definitions

Some of the most important definitions in AASHTO are the following:

-Any structure having an opening not less than 6100 mm that forms a part of a highway or that is located over or under a highway is called bridge.
-A design life is equal to (period of time on which the statistical derivation of transient loads is based) 75 yrs for these Specifications.
-A major change in the geometry of the bridge rendering it unfit for use is called collapse.

4.4.3 Design Philosophy

General AASHTO LRFD Design Equation

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \Longrightarrow$$ *AASHTO LRFD* Equation 1.3.2.1-1

where:

η_i = load modifier, relating to ductility, redundancy, and operational importance
γ_i = load factor; a statistically based multiplier applied to force effects
Q_i = force effect
ϕ = resistance factor; a statistically based multiplier applied to nominal resistance
R_n = nominal resistance
R_r = factored resistance

Figure 4.4.3.1 A - General AASHTO LRFD Design Equation

Load factors are factors greater than 1 that are used to account for the probability of the loads happening at the same time, inaccuracies in measurements and analysis and load variability.

Resistance factors are factors smaller than 1 accounting for variabilities in material properties, errors in workmanship and uncertainty in resistance of materials.

Load Modifier Factors η

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \Longrightarrow \textit{AASHTO LRFD}\text{ Equation 1.3.2.1-2}$$

However, for minimum values of γ_i: $\Longrightarrow$ *AASHTO LRFD* Equation 1.3.2.1-3

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$$

Figure 4.4.3.1 B - General AASHTO LRFD Design Equation

η_D = The ductility factor
η_R = The redundancy factor
η_I = The operational importance factor

η_D = 1.05 for nonductile components and connections, 1.00 for designs and details that comply with *AASHTO*, 0.95 for components and connections for which measures have been taken beyond those required in *AASHTO* for ductility, 1.00 for non-strength limit state design

η_R = 1.05 for strength limit state of nonredundant members, 1.00 for all limit states other than strength, 0.95 for exceptional levels of redundancy and torsionally-closed cross sections

η_I = 1.05 for critical or essential bridges to 0.95 for relatively less important bridges, 1.00 for all limit states other than strength

4.4.4 Load Factors and Load Combinations

AASHTO divides the load cases into 4 different categories each having subcategories:

Table 4.4.4.1 A- Strength Categories

1-Strength Categories	
Strength I	Basic load combination relating to the normal vehicular use of the bridge without wind.
Strength II	Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.
Strength III	Load combination relating to the bridge exposed to the design wind speed at the location of the bridge.
Strength IV	Load combination emphasizing dead load force effects in bridge superstructures.
Strength V	Load combination relating to normal vehicular use of the bridge with wind of 80* mph velocity.

*80 mph equals approximately 130 km/h

Table 4.4.4.1 B- Extreme Event Categories

2-Extreme Event Categories	
Extreme Event I	Load combination including earthquake. The load factor for live load γ_{EQ}, shall be determined on a project-specific basis.
Extreme Event II	Load combination relating to ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load.

Table 4.4.4.1 C- Service Categories

3-Service Categories	
Service I	Load combination relating to the normal operational use of the bridge with a 70*mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.
Service II	Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load. For structures with unique truck loading conditions, such as access roads to ports or industrial sites which might lead to a disproportionate number of permit loads, a site-specific increase in the load factor should be considered.
Service III	Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.
Service IV	Load combination relating only to tension in prestressed concrete columns with the objective of crack control.

*70 mph equals approximately 115 km/h

Table 4.4.4.1 D- Fatigue Categories

4-Fatigue Categories	
Fatigue I	Fatigue and fracture load combination related to infinite load-induced fatigue life.
Fatigue II	Fatigue and fracture load combination related to finite load-induced fatigue life.

There are two types of loading discussed in AASHTO used together with the categories given above. The first one is permanent loads. Permanent loads are any type of loads that don't vary over time or vary very slowly after the construction gets fully completed. The second one is transient loads. Transient loads are loads that vary over time quite significantly compared to permanent loads. They are not permanent as the name suggests. There are many different loads coming from different places placed under these two categories. The table below has a list of load combinations under different categories mentioned above for different loading types.

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Table 4.4.4.2- Load Combinations and Load Factors

Permanent Loads

CR	=	force effects due to creep
DD	=	downdrag force
DC	=	dead load of structural components and nonstructural attachments
DW	=	dead load of wearing surfaces and utilities
EH	=	horizontal earth pressure load
EL	=	miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES	=	earth surcharge load
EV	=	vertical pressure from dead load of earth fill
PS	=	secondary forces from post-tensioning for strength limit states; total prestress forces for service limit states
SH	=	force effects due to shrinkage

Transient Loads

BL	=	blast loading
BR	=	vehicular braking force
CE	=	vehicular centrifugal force
CT	=	vehicular collision force
CV	=	vessel collision force
EQ	=	earthquake load
FR	=	friction load
IC	=	ice load
IM	=	vehicular dynamic load allowance
LL	=	vehicular live load
LS	=	live load surcharge
PL	=	pedestrian live load
SE	=	force effect due to settlement
TG	=	force effect due to temperature gradient
TU	=	force effect due to uniform temperature
WA	=	water load and stream pressure
WL	=	wind on live load
WS	=	wind load on structure

LOAD COMBINATIONS AND LOAD FACTORS

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.00	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I— *LL*, *IM* & *CE* only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— *LL*, *IM* & *CE* only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

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4.4.4.1 Permanent Loads

In this section, for the purposes of this report, only dead loads, superimposed dead loads, force effects due to shrinkage, secondary prestressing forces due to post-tensioning, total reaction forces due to post-tensioning will be discussed.

4.4.4.1.1 Dead Loads

Dead Loads are the total weight of structural critical elements like slabs, beams and girders, columns etc. Dead load calculation is usually done using the following formula:

(Unit weight of material) x (Volume of material)

If the material is a composite material like reinforced/prestressed concrete, the unit weight of the material that has the significant volume is adjusted to account for the differences in unit weight between materials.

AASHTO gives a table showing the unit weight of different materials as a reference to be used if the material weight is not specified by the manufacturer and leaves the method to calculate the dead load to the engineer.

Table 4.4.4.1.1.1- Unit weight of different materials

Material		Unit Weight (kcf)	Unit Weight (kg/m^3)
Aluminum Alloys		0.175	2800
Bituminous Wearing Surfaces		0.140	2250
Cast Iron		0.450	7200
Cinder Filling		0.060	960
Compacted Sand, Silt, or Clay		0.120	1925
Concrete	Lightweight	0.110 to 0.135	1750 to 2160
	Normal Weight with $f'_c \leq 5.0$ ksi $f'_c \leq 35$ MPa	0.145	2325
	Normal Weight with $5.0 < f'_c \leq 15.0$ ksi $35 < f'_c \leq 100$ MPa	$0.140 + 0.001 f'_c$	$2240 + 2.32 f'_c$
Loose Sand, Silt, or Gravel		0.100	1600
Soft Clay		0.100	1600
Rolled Gravel, Macadam, or Ballast		0.140	2250
Steel		0.490	7850
Stone Masonry		0.170	2725
Wood	Hard	0.060	960
	Soft	0.050	800
Water	Fresh	0.0624	1000
	Salt	0.0640	1025
Item		Weight per Unit Length (klf)	kg/m
Transit Rails, Ties, and Fastening per Track		0.200	300

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4.4.4.1.2 Superimposed Dead Loads

Superimposed dead load (SDL) is any permanent load added to the structure after the main structural elements are completed. In a bridge, the weight of asphalt and waterproofing system, barrier walls, traffic lights can be shown as examples to SDLs

4.4.4.1.3 Forces due to Shrinkage of Concrete

Shrinkage is the decrease of concrete volume over time. There are three types of shrinkage:
-Autogenous Shrinkage that happens due to cement hydration
-Drying shrinkage related with relative humidity
-Chemical Shrinkage caused by the reaction of hydration products with CO_2

In reinforced concrete, some shrinkage forces can be accounted for by reinforcement to control cracks. In prestressed concrete, these forces are eliminated by changing the prestressing force or applying necessary tension in precast members.

Forces applied to abutments by shrinkage is usually not an issue for short span reinforced concrete bridges. For prestressed bridges, it is seen as a loss in stress and accounted for by the increased prestressing force.

4.4.4.1.4 Secondary Prestressing Reactions/Forces

Having a cable profile that follows a path such that the effects of prestressing force and external force can be expressed as a single internal force at each cross-section throughout the length of the member (concordant cable: c-line = t-line) is not always possible in a continuous post-tensioned system because there are allowable stress limits for design. When there is a distance between the cable profile and concordant cable profile location, that distance acts like a moment arm.

(Prestressing Force) x (Moment arm: a) = M_2 (Secondary Moment)

This moment mentioned above creates an uplift in the piers and down lift at the abutments.

-See commentary appendix section 4.8 for an example.

4.4.4.1.5 Total Reactions/Forces due to Post-tensioning only

Total forces due to post-tensioning is equal to moment caused by eccentricity of center of gravity of cables + secondary moments.

-See commentary appendix section 4.8 for an example.

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4.4.4.2 Transient Loads

In this section, for the purposes of this report, only live loads, wind loads, water loads, and earthquake loads will be discussed.

4.4.4.2.1 Live Loads

Vehicle loading:

The main design for vehicles is done using a design truck called HL-93. This truck weighs 325 kN in total. Its distance between rear axles should be changed by the engineer to produce extreme load effects depending on the bridge. Usually in practice an average distance is used.

For fatigue calculations, the distance between axles is kept constant at 9 m.

This truck is moved along a design lane (defined in AASHTO) every 10% of the opening of each span. Moment and shear values for extremes are recorded.

Tire contact area should be following this rectangular area, the tire pressure should be distributed equally along the rectangle:

> The tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose width is 510 mm and whose length is 250 mm.

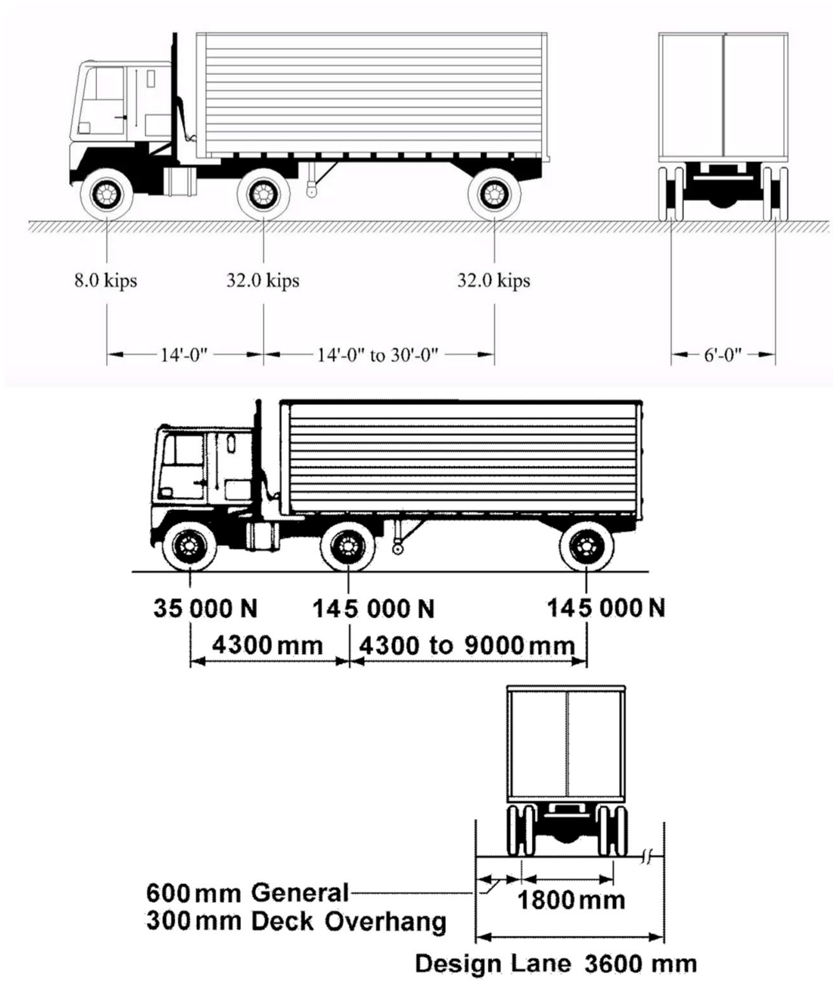


Figure 4.4.4.2.1.1- HL-93 Truck

Lane loading:
According to AASHTO, a distributed load of 3.1 N/mm^2 should be applied to the specified area (3000 mm x Lane Length between abutments mm).

(combinations)
The following load cases should be used after getting lane loading and truck loading:

- Strength & Service Limit States:
  1.33 Truck + Lane
- Fatigue Limit States:
  1.15 x Fatigue Truck
- Live Load Deflection:
  1.33 Truck
  0.25 (1.33 Truck) + Lane

Pedestrian Loads:

A pedestrian load of 3.6×10^{-3} MPa shall be applied to all sidewalks wider than 600 mm and considered simultaneously with the vehicular design live load.

Bridges for only pedestrian and/or bicycle traffic shall be designed for a live load of 4.1×10^{-3} MPa.

4.4.4.2.2 Wind Loads

According to AASHTO wind is a horizontal force applying to the bridge. Since wind can be multidirectional, the force effects should be taken vertically to the surfaces of the bridge (worst case).

Table 4.4.4.2.2.1- Wind Speed for Different Load Combinations

Wind Speed for Different Load Combinations

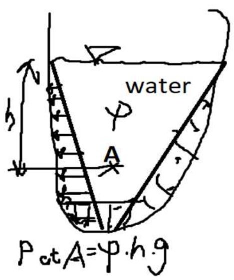
Load Combination	3-Second Gust Wind Speed (km/h)
Strength III	Wind speed to be taken from country specific data
Strength V	130
Service I	115
Service IV	0.75 of the speed used for the Strength III limit state

For super large structures and extreme cases where data is not available, a computational fluid dynamics simulation or wind tunnel tests can be done by the engineer.

-See commentary appendix section 4.8 for an example of wind load analysis.

4.4.4.2.3 Water Loads

Static Pressure of Water:



Static pressure of water shall be assumed to act perpendicular to the surface that is retaining the water. Pressure shall be calculated as the product of height of water above the point of consideration, the density of water, and g (the acceleration of gravity).

Design water levels for various limit states shall be as specified and/or approved by the Owner.

Buoyancy of Water:

If there are underwater components of the bridge, buoyancy is a force applied upwards to the centre of the fluid displaced (water or maybe salt water).

Longitudinal Stream Pressure

The pressure of flowing water acting in the longitudinal direction of substructures shall be taken as:

$$p = 5.14\times10^{-4} C_D V^2$$

where:

p = pressure of flowing water (MPa)

C_D = drag coefficient for piers as specified in Table

V = design velocity of water for the design flood in strength and service limit states and for the check flood in the extreme event limit state (m/sec.)

γ = density (unit mass) of water (kg/m^3)

V = velocity of water (m/sec.)

Drag coefficient for piers

Type	C_D
semicircular-nosed pier	0.7
square-ended pier	1.4
debris lodged against the pier	1.4
wedged-nosed pier with nose angle 90° or less	0.8

Table 4.4.4.2.3 A – Longitudinal Stream Pressure

Lateral Stream Pressure

The lateral, uniformly distributed pressure on a substructure due to water flowing at an angle, θ, to the longitudinal axis of the pier shall be taken as:

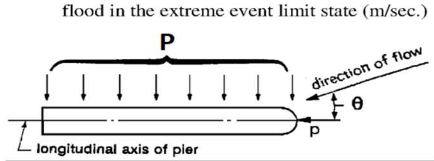
$$p = 5.14\times10^{-4} C_L V^2$$

where:

p = lateral pressure (MPa)

C_L = lateral drag coefficient specified in Table 1

V = design velocity of water for the design flood in strength and service limit states and for the check flood in the extreme event limit state (m/sec.)



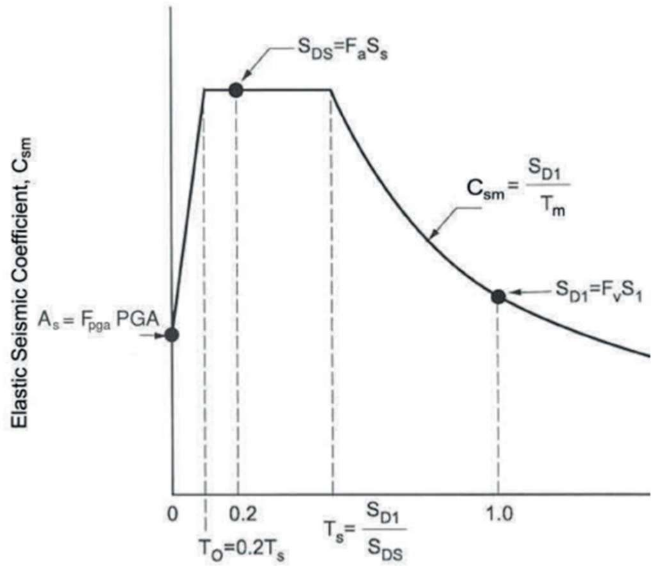
Lateral Drag Coefficient for Piers

Angle, θ, between direction of flow and longitudinal axis of the pier	C_L
0°	0.0
5°	0.5
10°	0.7
20°	0.9
≥30°	1.0

Table 4.4.4.2.3 B – Lateral Stream Pressure

4.4.4.2.4 Earthquake Loads

Earthquake loading is based on the area of interest. In the absence of site measurements, the following linear design spectrum can be used (Damping ratio of 5% assumed) analysis:



Seismic Zones

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

Figure 4.4.4.2.4.1 –Seismic Zones and Linear Elastic Design Spectrum

For more earthquake critical areas, time history analysis can be done for the entire structure's finite element model. Loads obtained from various big events of earthquakes can be assembled for design of piers and abutments as well as for deflection.

-See commentary appendix section 4.8 for an example of this usage.

Further detail in earthquake loading and direct integration methods is out of the scope of this report and therefore won't be discussed.

4.4.5 Reinforced Concrete

4.4.5.1 Concrete Strength and Important Properties

Modulus of Elasticity

In the absence of measured data, the modulus of elasticity, E_c, for normal weight concrete with design compressive strengths up to 15.0 ksi and lightweight concrete up to 10.0 ksi, with unit weights between 0.090 and 0.155 kcf, may be taken as:

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

For normal weight concrete with w_c = 0.145 kcf, E_c may be taken as:

$$E_c = 2{,}500 f_c'^{0.33}$$

Compressive Strength

Design concrete compressive strengths above 10.0 ksi for normal weight concrete shall be used only when allowed by specific Articles or when physical tests are made to establish the relationships between the concrete strength and other properties. Concrete with compressive strengths used in design below 2.4 ksi should not be used in structural applications.

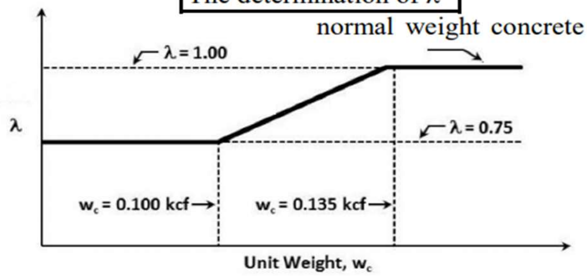
The design concrete compressive strength for prestressed concrete and decks shall not be less than 4.0 ksi.

Modulus of Rupture

Unless determined by physical tests, the modulus of rupture, f_r, for lightweight concrete with specified compressive strengths up to 10.0 ksi and normal weight concrete with specified compressive strengths up to 15.0 ksi may be taken as $\boxed{0.24\,\lambda \sqrt{f'_c}}$

where λ:

The determination of λ



Tensile Strength

For normal weight concrete with design concrete compressive strengths up to 10.0 ksi, the direct tensile strength may be estimated as $\boxed{f_t = 0.23\sqrt{f'_c}}$.

Coefficient of Thermal Expansion

For normal weight concrete: 6.0×10^{-6}/°F

For lightweight concrete: 5.0×10^{-6}/°F

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)

4.4.5.2 Shrinkage of Concrete

Creep and shrinkage provisions shall be applicable for design concrete compressive strengths up to 15.0 ksi. In the absence of more accurate data, the shrinkage coefficients may be assumed to be 0.0002 after 28 days and 0.0005 after one year of drying.

For concretes devoid of shrinkage-prone aggregates, the strain due to shrinkage, ε_{sh}, at time, t, may be taken as:

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} 0.48 \times 10^{-3}$$

in which:

$$k_{hs} = (2.00 - 0.014\,H)$$

where:

k_{hs} = humidity factor for shrinkage

k_{td} = time development factor

H = average annual ambient relative humidity (percent). In the absence of better information, H may be taken from country specific data

k_s = factor for the effect of the volume-to-surface ratio of the component

k_f = factor for the effect of concrete strength

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t}$$

f'_{ci} = design concrete compressive strength at time of prestressing for pretensioned members and at time of initial loading for nonprestressed members. If concrete age at time of initial loading is unknown at design time, f'_{ci} may be taken as $0.80\,f'_c$ (ksi).

Important:
Large concrete members may undergo substantially less shrinkage than that measured by laboratory testing of small specimens of the same concrete.

4.4.5.3 Creep of Concrete

Creep is influenced by the same factors as shrinkage, and also by the following:

- Magnitude and duration of the stress,
- Maturity of the concrete at the time of loading,
- Temperature of concrete.

4.4.5.5 Steel Reinforcement

Reinforcement steel below 75 ksi should be used whenever possible. Steel having higher strength than 75 ksi doesn't have a specified yield point and plateau, therefore more attention is required on calculations.
The usage of reinforcement steel between 75 ksi and 100 ksi is allowed only in special seismic applications in seismic zones with higher Peak Ground Accelerations.

Modulus of elasticity of steel can be taken as 29000 ksi for steels up to 100 ksi.

Minimum Spacing of Reinforcing Bars

Cast-in-Place Concrete

For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- 1.5 times the nominal diameter of the bars;
- 1.5 times the maximum size of the coarse aggregate; or
- 1.5 in.

Precast Concrete

For precast concrete manufactured under plant control conditions, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- the nominal diameter of the bars;
- 1.33 times the maximum size of the coarse aggregate; or
- 1.0 in.

Maximum Spacing of Reinforcing Bars

Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not be greater than the lesser of the following:

- 1.5 times the thickness of the member; or
- 18.0 in.

4.4.5.6 Shear Reinforcement

Regions Requiring Transverse Reinforcement

Except for slabs, footings, and culverts, transverse reinforcement shall be provided where:

$$V_u > 0.5\phi\left(V_c + V_p\right)$$

where:

V_u = factored shear force (kip)
V_c = nominal shear resistance of the concrete (kip)
V_p = component of prestressing force in the direction of the shear force
ϕ = resistance factor

Minimum Transverse Reinforcement

Where transverse reinforcement is required and nonprestressed reinforcement is used to satisfy that requirement, the area of steel shall satisfy:

$$A_v \geq 0.0316\ \lambda\sqrt{f_c'}\ \frac{b_v\, s}{f_y}$$

where:

A_v = area of transverse reinforcement within distance s (in.^{2})

b_v = width of web adjusted for the presence of ducts

s = spacing of transverse reinforcement (in.)

f_y = yield strength of transverse reinforcement (ksi) $\leq$ 100 ksi

λ = concrete density modification factor

where λ :

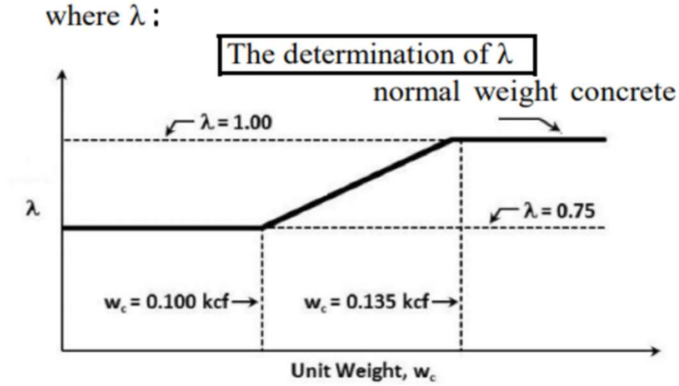


Figure 4.4.5.6.1 – Determination of Lambda

Illustration of the Terms b_v and d_v

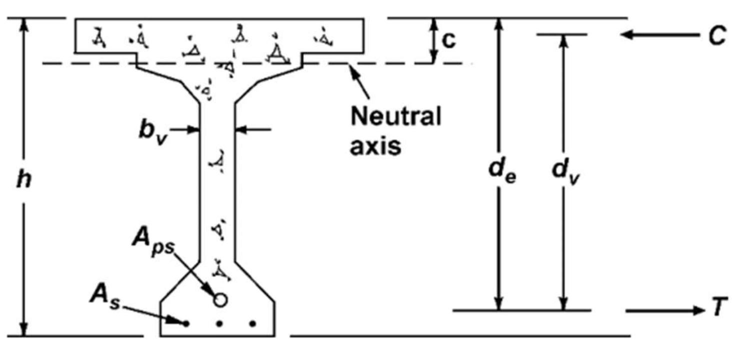


Figure 4.4.5.6.2 – Illustration of the Terms b_v and d_v

Illustration of Terms b_v, d_v, and d_e for Circular Sections

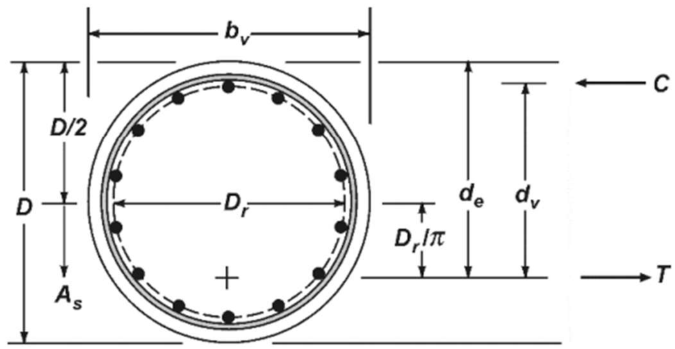


Figure 4.4.5.6.3 – Illustration of the Terms b_v, d_v and d_e for Circular Sections

One of the reaons why de la Concorde Overpass collapsed: Diagonal crack growth

A minimum amount of transverse reinforcement is required to restrain the growth of diagonal cracking and to increase the ductility of the section. A larger amount of transverse reinforcement is required to control cracking as the concrete strength is increased.

Maximum Spacing of Transverse Reinforcement

According to research done, prestressed girders having a transverse reinforcement bar spacing of 0.8 x d_v may result in cracks not getting intercepted by stirrups. The formulas below are good up to 100 ksi of concrete.

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.125 f'_c$, then:

$$s_{max} = 0.8 d_v \leq 24.0 \text{ in.}$$

- If $v_u \geq 0.125 f'_c$, then:

$$s_{max} = 0.4 d_v \leq 12.0 \text{ in.}$$

where:

$$v_u = \frac{\left|V_u - \phi V_p\right|}{\phi b_v d_v}$$

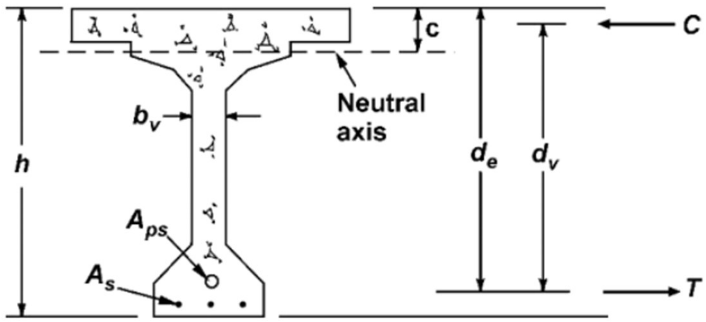


Figure 4.4.5.6.4 – Illustration of the Terms b_v and d_v

4.4.6 Prestressed Concrete

Table 4.4.6.1 – Stress Limits for Prestressed Concrete

Stress Limits for Prestressing Steel

Condition	Tendon Type		
	Plain High-Strength Bars	Low Relaxation Strand	Deformed High-Strength Bars
Pretensioning			
Immediately prior to transfer (f_{pbt})	$0.70f_{pu}$	$0.75f_{pu}$	—
At service limit state after all losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$
Post-Tensioning			
Prior to seating—short-term f_{pbt} may be allowed	$0.90f_{py}$	$0.90f_{py}$	$0.90f_{py}$
At anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.70f_{pu}$	$0.70f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.74f_{pu}$	$0.70f_{pu}$
At service limit state after losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$

Losses in prestressing:

Instantaneous Losses

Friction

Losses due to friction between the internal prestressing tendons and the duct wall may be taken as:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx + \mu\alpha)}\right)$$

f_{pj} = stress in the prestressing steel at jacking (ksi)
x = length of a prestressing tendon from the jacking end to any point under consideration (ft)
K = wobble friction coefficient (per ft of tendon)
μ = friction factor
α = sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad.)
e = base of natural logarithms

Table 4.4.6.2 – Friction Coefficients

Friction Coefficients

Type of Steel	Type of Duct	K	μ
Wire or strand	Rigid and semirigid galvanized metal sheathing	0.0002	0.15–0.25
	Polyethylene	0.0002	0.23
	Rigid steel pipe deviators for external tendons	0.0002	0.25
High-strength bars	Galvanized metal sheathing	0.0002	0.30

Elastic Shortening —Pretensioned Members

The loss due to elastic shortening in pretensioned members shall be taken as:

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$

where:

f_{cgp} = concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi).

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)

Elastic Shortening — Post-Tensioned Members

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be taken as:

$$\Delta f_{pES} = \frac{N-1}{2N} \frac{E_p}{E_{ci}} f_{cgp}$$

where:

N = number of identical prestressing tendons

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to the prestressing force after jacking and the self-weight of the member at the sections of maximum moment (ksi)

Approximate Estimate of Time-Dependent Losses

For standard precast, pretensioned members subject to normal loading and environmental conditions, where:

- members are made from normal weight concrete;
- the concrete is either steam- or moist-cured;
- prestressing is by bars or strands with low relaxation properties; and
- average exposure conditions and temperatures characterize the site,

the long-term prestress loss, Δf_{pLT}, due to creep of concrete, shrinkage of concrete, and relaxation of steel shall be estimated using the following formula:

$$\boxed{\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}}$$

in which: $\gamma_h = 1.7 - 0.01H$

$$\gamma_{st} = \frac{5}{(1+f'_{ci})}$$

where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

H = average annual ambient relative humidity (percent)

γ_h = correction factor for relative humidity of the ambient air

γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member

Δf_{pR} = an estimate of relaxation loss taken as 2.4 ksi for low relaxation strand and in accordance with manufacturers recommendation for other types of strand (ksi)

Compressive Stress Limits in Prestressed Concrete at Service Limit State after Losses

Location	Stress Limit
• Due to the sum of effective prestress and permanent loads	$0.45f'_c$ (ksi)
• Due to the sum of effective prestress, permanent loads, and transient loads as well as during shipping and handling	$0.60\ \phi_w f'_c$ (ksi)

Table 4.4.6.3 – Tensile Stress Limit at Service After Losses

Tensile Stress Limits in Prestressed Concrete at Service Limit State after Losses

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges These limits may be used for normal weight concrete with concrete compressive strengths for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.	Tension in the Precompressed Tensile Zone, Assuming Uncracked Sections	
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0.19\lambda\sqrt{f'_c} \leq 0.6$ (ksi)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0.0948\lambda\sqrt{f'_c} \leq 0.3$ (ksi)
	• For components with unbonded prestressing tendons	No tension
Segmentally Constructed Bridges These limits may be used for normal weight concrete with concrete compressive strengths for use in design up to 15.0 ksi and lightweight concrete up to 10.0 ksi.	Longitudinal Stresses through Joints in the Precompressed Tensile Zone	
	• Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5 fy; internal tendons or external tendons	$0.0948\lambda\sqrt{f'_c} \leq 0.3$ (ksi)
	• Joints without the minimum bonded auxiliary reinforcement through joints	No tension
	Transverse Stresses	
	• Tension in the transverse direction in precompressed tensile zone	$0.0948\lambda\sqrt{f'_c} \leq 0.3$ (ksi)
	Stresses in Other Areas	
	• For areas without bonded reinforcement	No tension
	• In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5 f_y$, not to exceed 30.0 ksi	$0.19\lambda\sqrt{f'_c}$ (ksi)

4.4.7 Durability Concerns in AASHTO

Table 4.4.7.1 – Durability Concerns in AASHTO

Factors in Concrete Durability

Type of Materials-Related Defect	Surface Distress Manifestations and Locations	Cause or Mechanisms	Time of Appearance	Prevention or Reduction
Due to Physical Mechanisms				
Mechanical wear of decks and wearing surfaces decks and wearing surfaces	Abrasion and polishing polishing, rutting	Tire contact, improper curing, water floating to surface	Varies	Proper curing, sealants
Freezing and thawing deterioration of hardened cement paste	Scaling or map cracking, generally initiating near joints or cracks; possible internal disruption of concrete matrix.	Deterioration of saturated cement paste due to repeated cycles of freezing and thawing.	1–5 years	Addition of air-entraining agent to establish protective air-void system.
Deicer scaling and deterioration	Scaling or crazing of the slab surface.	Deicing chemicals can amplify deterioration due to freezing and thawing and may interact chemically with cement hydration products.	1–5 years	Limiting W/C ratio to no more than 0.45, and providing a minimum 30-day drying period after curing before allowing the use of deicers.
Deterioration of aggregate due to freezing and thawing	Cracking parallel to joints and cracks and later spalling; maybe accompanied by surface staining.	Freezing and thawing of susceptible coarse aggregates results in fracturing or excessive dilation of aggregate.	10–15 years	Use of nonsusceptible aggregates or reduction in maximum coarse aggregate size.
Early age cracking	Map cracking	Shrinkage of concrete	<28 days	Shrinkage limits, fibers continuous wet cure
Due to Chemical Mechanisms				
Alkali-silica reaction (ASR)	Map cracking (rarely more than 2.0 in. deep) over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Reaction between alkalis in cement and reactive silica in aggregate, resulting in an expansive gel and the degradation of the ggregate particle.	5–15 years	Use of nonsusceptible aggregates, addition of pozzolans, limiting of alkalis in concrete, addition of lithium salts.
Alkali-carbonate reaction	Map cracking over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Expansive reaction between alkalis in cement and carbonates in certain Aggregates containing clay fractions.	5–15 years	Avoiding susceptible aggregates, or blending susceptible aggregate with nonreactive aggregate.
External sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Expansive formation of ettringite or gypsum that occurs when external sources of sulfate (e.g., groundwater, deicing chemicals) react with aluminates in cement or fly ash.	1–5 years	Minimizing tricalcium aluminate content in cement or using blended cements, class F fly ash, or GGBFS.
Internal sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Formation of ettringite from internal sources of sulfate that results in either expansive disruption on the paste phase or fills available air voids.	1–5 years	Minimizing tricalcium aluminate content in cement, using low sulfate cement, eliminating source of slowly soluble sulfate, and use cements conforming to ASTM C150, *C595,* or CllS7, and avoiding high curing temperatures.
Corrosion of embedded steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel.	Chloride ions penetrate concrete and corrode embedded steel.	3–10 years	Reducing the permeability of the concrete, providing adequate concrete cover, and coating steel.

4.5 CSA-S6-1966 Design of Highway Bridges

The following sections will introduce and summarize the design criteria, design requirements, design input, material technology and design methodology included in CSA-S6-1966 Design of Highway Bridges. All the bridge standards and codes details and images in this section are taken from CSA-S6-1966 [2].

4.5.1 Scope

The scope of CSA-S6-1966 Design of Highway Bridges is for bridges which have a span length up to 400 ft. For bridges with a span length greater than 400 ft, additional structure specification is required.

4.5.2 Load Factors and Load Combinations

CSA-S6-1966 states that the following loads should be considered when designing structures:

- Dead Load
- Live Load
- Impact or Dynamic effect of the live load
- Wind Loads
- Longitudinal forces, centrifugal forces, thermal forces, earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, earthquake stresses and structural stability stresses

Table 4.5.2.1 – Load Combinations

Load Combinations

		Percentage of unit stress
Group I	= D+L+I+E+B+SF	100
Group II	= D+E+B+SF+W	125
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

4.5.4.1 Live Loads

In CSA-S6-1966, there are two main design trucks defined for live load

a) Standard H truck
   The total weight of the design truck is 40kips (177.9 kN) with 20% of the force gets dissipated to front tires and the remaining gets distributed to rear tires as shown in Figure 4.5.4.1 on the right.

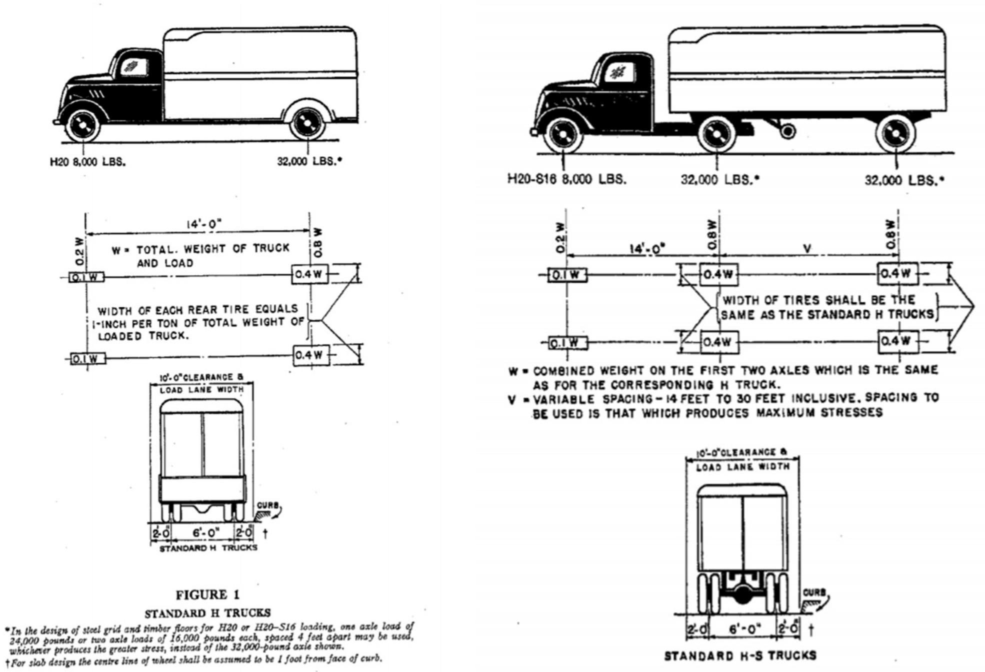


Figure 4.5.4.1 – CSA S6-66 Design Trucks (Left - Standard H Truck, Right - Standard H-S truck)

b) Standard H-S truck
   The total weight of the design truck is 325 kN, similar to AASHTO LRFD 2014-17.

   <u>Lane Loading:</u>

   Unfactored, undistributed lane loads are calculated by superimposing a point load and a uniformly distributed load of 9.34 kN/m acting on bridge.

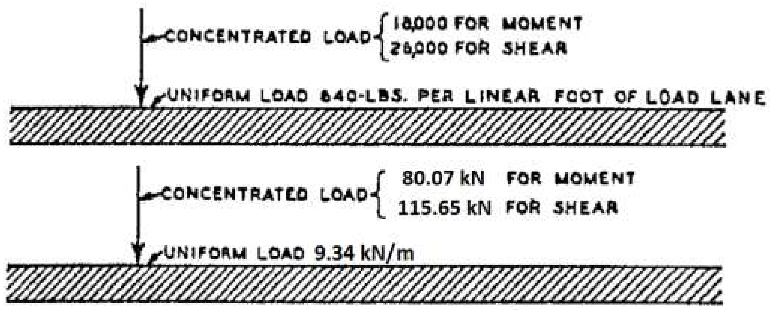


Figure 4.5.4.2 – CSA S6-66 Lane Loading

4.5.4.2 Wind Loads

3.6 kN/m for trusses and arches
2.4 kN/m for girders and beams

Must be vertically applied to all surfaces mentioned above.

4.5.4.3 Earthquake Loads

Earthquake loads are covered as a factor of dead load in this code. No extensive earthquake coverage exists.

4.5.5 Concrete Strength

The minimum allowed concrete compressive strength is 20 MPa. At that time, concrete technology was not as advanced as nowadays, so this was a reasonable value. Today, this is not acceptable.

4.5.6 Allowable Stresses

Allowable Stresses

Standard Notations. Standard notations are as follows:

f_c = permissible extreme fibre stress in compression;
f'_c = unit ultimate compressive strength of concrete as determined by cylinder tests at the age of 28 days.

Concrete. Allowable stresses for concrete are as follows:

(a) **Flexure:**
Extreme fibre in compression .... $f_c = 0.40f'_c$
Extreme fibre in tension in plain concrete footings and walls .... $f_c = 1.6\sqrt{f'_c}$
Extreme fibre in tension, reinforced concrete .... none;

(b) **Shear:**
Beams without web reinforcement .... $1.1\sqrt{f'_c}$
Beams with web reinforcement, proportioned to resist entire shear force .... $5\sqrt{f'_c}$
Slabs and footings (peripheral shear) .... $2\sqrt{f'_c}$
Horizontal shear in shear keys between slab and stem of T-beams and boxes .... $0.15\ f'_c$

4.5.7 Calculation of Shear Reinforcement

Calculation of Shear Reinforcement. Diagonal tension and shear in reinforced concrete beams shall be calculated by the following formulas:

Notations

A_v = total area of web reinforcement in tension within a distance "s" (measured in a direction parallel to that of the main reinforcement) or the total area of all bars bent-up in any one plane

b = width of the beam

d = distance from compression face to centroid of tension reinforcement

f_v = tensile unit stress in web reinforcement

s = spacing of web reinforcement bars in a direction parallel to the longitudinal reinforcement

v = shearing unit stress

V = external shear on any section

α = angle between inclined web bars and axis of beam

Formulas

Shearing unit stress, as a measure of diagonal tension:

Formula (1)

$$v = \frac{V}{bd}$$

Area of steel required in stirrups placed perpendicular to the longitudinal reinforcement:

Formula (2)

$$A_v = \frac{Vs}{f_v d}$$

Area of steel required when the web reinforcement consists of a single bent bar or a single group of bent bars:

Formula (3)

$$A_v = \frac{V}{f_v \sin\alpha}$$

in which V shall not exceed $1.5\, bd\sqrt{f'_c}$

Area of steel when there is a series of parallel bent bars:

Formula (4)

$$A_v = \frac{Vs}{f_v d\,(\sin\alpha + \cos\alpha)}$$

4.5.8 Prestressing Losses

9.3.4 Prestressing Losses

9.3.4.1 General. Allowance shall be made for decrease in prestress in steel from the following, where applicable:

(a) Shrinkage of the concrete;

(b) Elastic deformation of the concrete during prestressing or at transfer;

(c) Creep in concrete;

(d) Relaxation of stress due to creep of steel;

(e) Anchoring; and

(f) Friction.

9.3.4.2 Calculation of Losses. Loss in steel stress not including friction and anchoring loss shall be either:

(a) Calculated on the basis of the factors in Clause 9.3.4.1; or

(b) Assumed as follows:
  (i) For post-tensioning — 25,000 psi; or
  (ii) For pretensioning — 35,000 psi.

9.3.4.3 Anchoring. Where applicable, the designer shall make an allowance for the anchoring loss, the magnitude of which shall be checked on the site by the supervising authority.

9.3.4.4 Friction on Post-tensioned Steel

9.3.4.4.1 Friction losses shall be based on the relationship:

$$T_o = T_x \cdot e^{(KL + u\alpha)}$$

For values of $(KL + u\alpha)$ below 0.1, the following formula may be used:

$$T_o = T_x\ (1 + KL + u\alpha)$$

9.3.4.4.2 The values of K and u in Tables 12 and 13 respectively are typical and may be used as a guide. Values of K and u used in design shall be indicated on the plans for guidance in selection of materials and methods that will satisfy the assumed values.

TABLE 12

FRICTION WOBBLE COEFFICIENT (K)

Type of Duct	$K \times 10^{-4}$		
	Wire	Strand	Bar
Rigid Duct	5	5	3
Flexible Duct	25	20	5

4.6 Conclusion

Engineers must follow the latest codes and standards to build a durable design bridge. At times, there are special needs and requirements that that client ask for, it is then we need to follow the client's requirements keeping in mind the codes at the time.

4.7 References

[1]. Canadian Standard Association, Canadian Highway Bridge Design Code, S6-14, Mississauga, ON, Canada.

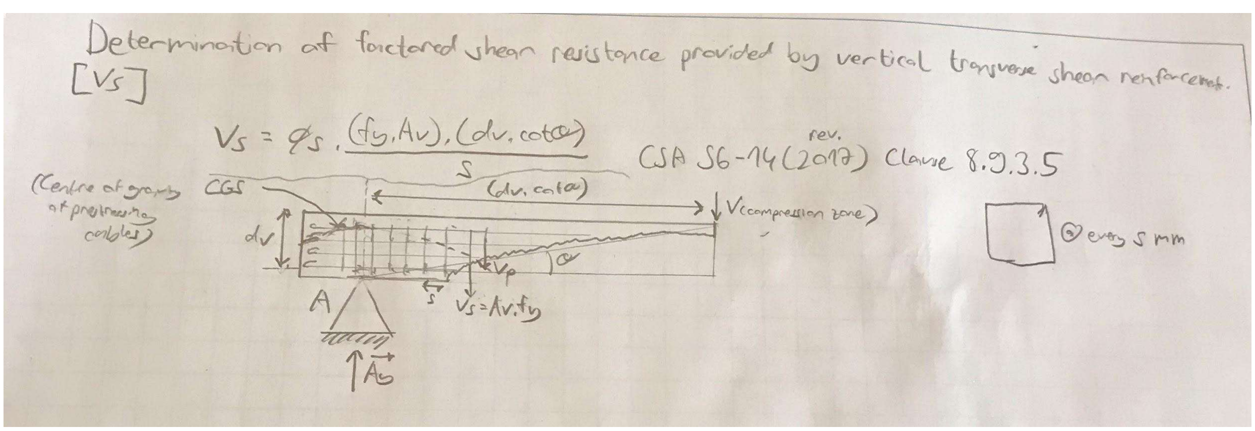
[2] American Association of State Highway and Transportation Officials. Bridge Design Specifications - 8th Edition - September 2017

[3] CANADIAN STANDARDS ASSOCIATION (1966). Design of Highway Bridges. CSA STANDARD S6-1966.

4.8 Commentary Appendix on Design Codes

<u>Example on reinforcement spacing according to section 4.2.5.5:</u>

<u>Determination of factored shear resistance provided by vertical transverse shear reinforcement is the following according to CSA S6-14 explained:</u>



Determination of factored shear resistance provided by concrete according to CSA S6-14 explained:

$$V_c = \phi_c \cdot 2.5 \cdot \beta \cdot f_{cr} \cdot b_v \cdot d_v \quad \text{max } f_{cr} = 3.2 \text{ MPa}$$

fcr can be replaced with its formula in CSA $\quad \therefore \text{max } f'_c = 64 \text{ MPa}$

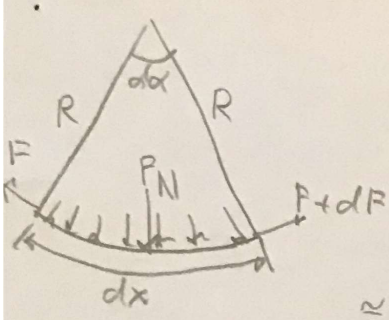
$$f_{cr} = 0.4\sqrt{f'_c} \Rightarrow$$

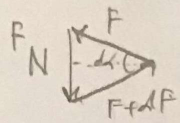
$$V_c = \varphi_c \beta \sqrt{f'_c}\, b_v d_v \quad \text{Max } f'_c \leq 60 \text{ MPa}$$

Do not use this equation with very high strength concrete. Lambda factors come into equation

*A simplified method (for reinforced concrete only) and an exact method for calculating Beta factor is proposed in code. The details can be found at clauses: 8.9.3.6 and 8.9.3.7

Derivation of circular assumption friction formula for parabolic cable profiles:





$$F = F + dF + (\mu F \cdot d\alpha + k F \cdot dx)$$

$$dF = -(\mu F d\alpha + k F \cdot dx)$$

$$N = 2F \cdot \sin\frac{d\alpha}{2}$$

$$\simeq 2F\frac{d\alpha}{2} = F d\alpha$$

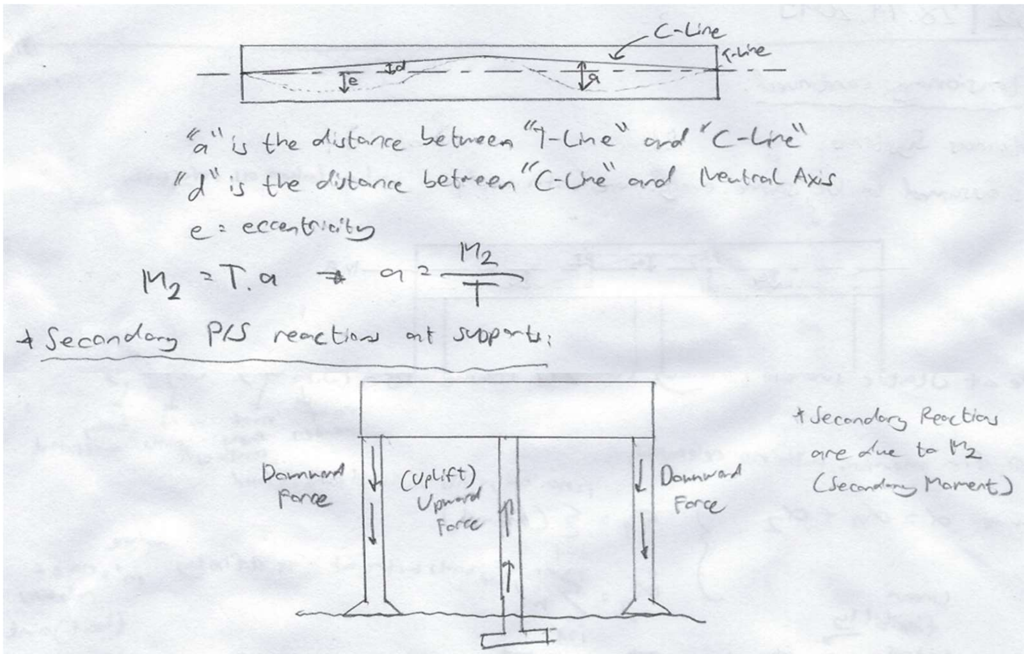
small angle assumption

$$\int_{F_i}^{F_x} \frac{dF}{F} = k\int_{\alpha}^{x} dx + \mu\int_{0}^{\alpha} d\alpha$$

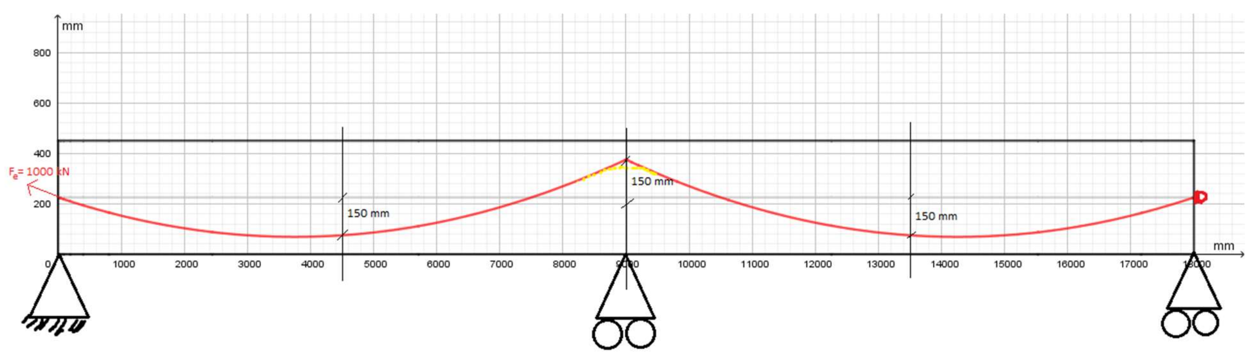
$$\ln\frac{F(x)}{F_i} = -(\mu\alpha + k.x)$$

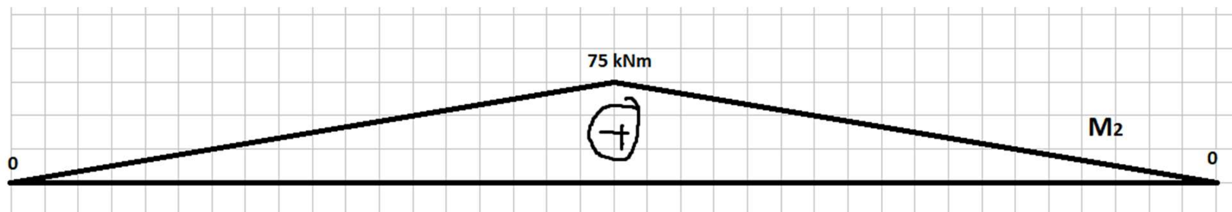
$$F(x) = F_i \cdot e^{-(\mu\alpha + kx)}$$

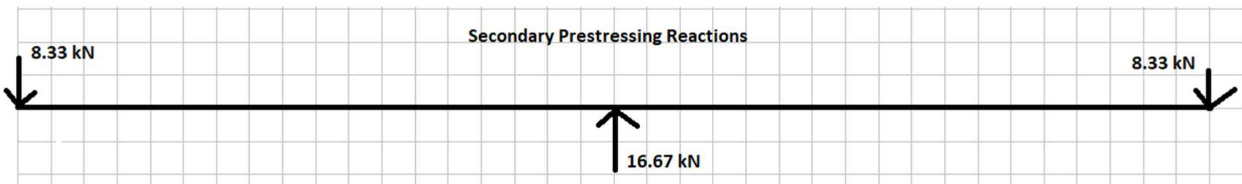
Secondary Prestressing Forces/Reactions Explained:



Ex: The following post-tensioned beam has an assumed constant effective force of 1000 kN throughout its cable profile. The cable profile is assumed to form two parabolas as shown below in red. (The yellow dotted line is what the cable does is in reality)



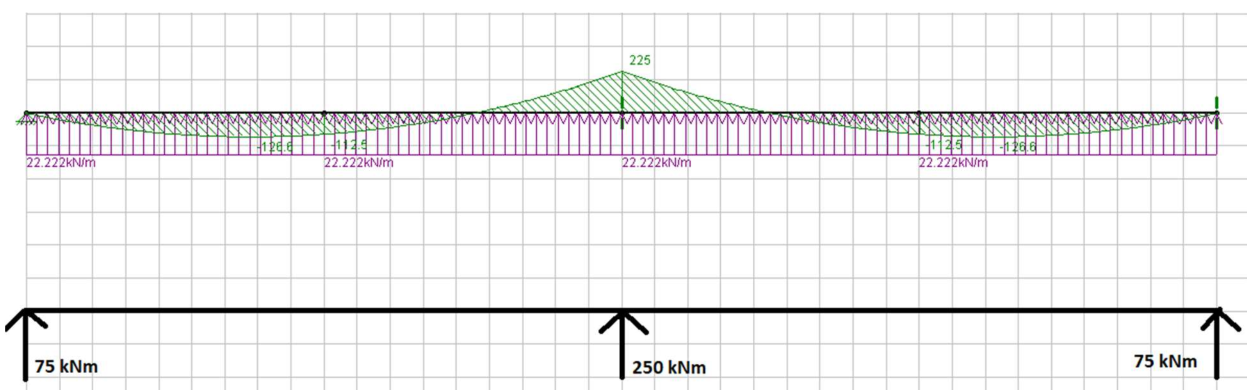




<u>Total Reactions/Forces due to Post-tensioning only example:</u>

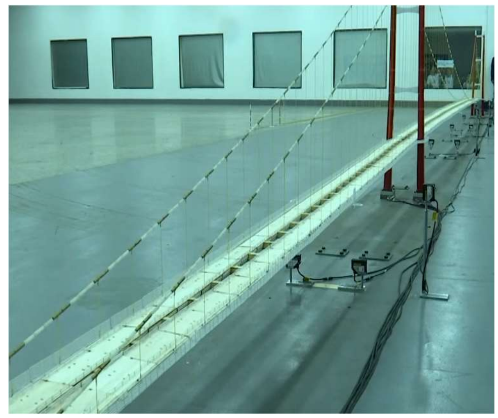
Total forces due to post-tensioning is equal to moment caused by eccentricity of centre of gravity of cables + secondary moments.

Same example above produces the following reactions and forces due to post-tensioning:



-Wind Tunnel Analysis Example:

1915 Canakkale Bridge Wind Tunnel Tests: (2023 m of span expected to open fully in 2023). Candidate for world's longest suspension bridge. AASHTO was one of the codes used in the design along with 3 other codes.



-Time History Analysis on a bridge using software:

The Hilber-Hugses-Taylor method is used in softwares using SAP2000 solver for direct integration. It is a more accurate version of Newmark's method.

Chapter 5 – Required Design Input

Table of Contents

5.1 Introduction

At the beginning of any project it is important for the engineering design team to communicate with clients, potential contractors and local government, to gather sufficient information regarding the client requirements, site conditions as well as the local government regulations. Constructability should also be taken into consideration of every phase of the project to facilitate the information collection.

5.2 Client Requirement

First and foremost, the design team is responsible to communicate with the client and confirm what exactly is the client looking for and fulfill the needs of the client as requested by bringing it to existence using their professional expertise. The design scope, budget restrictions, schedule, service life are the few important things that need to be discussed and finalized during the meetings between the design team and client. The design team needs to make sure that its client requirements are practical and feasible. In the majority of construction projects, the client itself is part of the design team throughout the project life.

5.2.1 Scope of Design Work

The scope of the design must be clearly stated by the client which includes the function, objectives and constraints of the project. This gives the design team a clear understanding of what they are going to design. It is basically an agreement between the design team and the client regarding the work to be performed. The scope of work includes the following:

<u>Function</u> is the purpose of the design project
- if the bridge is for pedestrian use only or vehicle use only or both or rail use
- if its partial bridge construction or replacing the existing bridge

<u>Objectives</u> are the requirements set out by the client.
- Specified span length of the bridge

<u>Constraints</u> are limits or restriction set out by the client that cannot be compromised.
- The river below the bridge should not be polluted
- The bridge should be capable to resist natural disasters

5.2.2 Budget

The budget of the project is determined by the owner of the project. It should be set out as soon as possible by the owner and reported to the design team. It helps the design team know what type of bridge type and materials are affordable and should be used to design according to the client's needs. The budget for the project can be determined by assessing the project costs throughout the project, by comparing it to similar types of projects and preliminary design analysis [1]. The budget of the project includes the following construction cost, land acquisition, permit approval fees, planning costs, financing costs, machinery, insurance, consultation fees.

At times, the owner can give a wrong estimation of the project budget, this is where the estimators of the construction team step in and have discussions with the owner to reconfirm the budget.

5.2.3 Schedule

The schedule will help determine how the project will be constructed or completed in the required time period. The schedule also identifies major milestones dates and breaks down the whole project into small phases. It is mostly shown as a graphical representation; bar chart schedule or critical path method schedule are the two common methods used in construction projects [2]. The budget and schedule are strongly correlated to each other, therefore, a delay in the schedule will affect the budget of the project. Therefore, the budget provided by the owner must be compatible with the desired schedule.

5.2.4 Service Life

The expected service life of the bridge will be provided by the client. This in turn will influence the final design inputs. For example, the choice of materials for bridges that have a service life of 100 years must be of high strength such as steel or concrete and also regular maintenance and investigation needs to be carried out. The service life also impacts the safety levels of the bridge design. For instance, when designing the drainage system for a 100-year service life 100-year rainfall needs to be considered to prevent road flooding and surface detention.

5.2.5 Additional Requirement

The design team needs to consider any additional specific requirements made by the client such as aesthetics of the bridge, material used for the construction of the bridge and also structural type of bridge. For example, the client might request to build an aesthetically appealing bridge design located in the core part of the city.

5.3 Site Condition

It is important for the bridge designer to assess the site conditions on which the bridge is going to be built on as well as nearby conditions. Each bridge project will have different site conditions, and sometimes it is not evident at the time of bidding or initial site investigation. The identification of different site conditions such as soil with inadequate bearing capacity to support the bridge, unanticipated ground water, quicksand, rock formations is crucial to understand as they can cause major delays in the project and also create unplanned costs [3].

5.3.1 Site Location

The location of the bridge is as crucial as the characteristics of the bridge itself. A poor bridge location is prone to failure, susceptible to cracks resulting in regular maintenance and a host to many other problems. In choosing an appropriate bridge site location, several factors need to be considered such as selection of bridge type, topography, soil conditions, seismic conditions, weather conditions and availability of material.

5.3.1.1 Selection of Bridge Type

The selection of bridge type is dependent on-site characteristics, owner preferences, aesthetics, geography and cost [4]. If a bridge is being constructed over a water source, hydraulic analysis of the site and profile grade will influence the bridge type. To determine the size of the bridge, hydraulic analysis and risk assessment needs to be conducted. The profile grade will determine the height of the bridge which will help figure out if the water traffic can pass underneath the bridge. The unique geological features will affect the selection process of the bridge type. For instance, if the bridge is located within a gorge, the bridge will be limited to have a span length and height less than 500 ft and 200 ft respectively [5]. The loading of the bridge will affect the selection process of the bridge type. For instance, if the bridge is being constructed for rail usage, the high vertical loads need to be considered and longitudinal loads need to be transferred effectively to the abutments and piers of the bridge. In this case, a truss bridge would be more suitable than a cable stayed or suspension bridge type. The aesthetics need to be considered if the bridge is being built between hills or mountains, the client wouldn't want to obstruct the natural scenery of the place, so a slender bridge would be a good fit for this location type. A cost comparison is done for different bridge types to balance both the client budget and the site needs.

5.3.1.2 Topography

Topographic surveys are usually conducted for new construction or addition of new development to the area [6]. The topographic survey must be performed before bridge erection, it helps in mapping and identifying the terrain features of the site. They provide an accurate measurement of any nearby reservoir, dams, highways, underground utilities and retaining walls. They also provide contour line details which helps the designers know the depression and rises in ground which helps in the design of the foundation and geometry of the bridge structure. They help the designers and architects determine where construction is not possible or if the land requires significant grading. Therefore, it is important to perform topographic surveys in order to avoid costly or time-consuming surprises [6].

5.3.1.3 Soil Conditions

The soil conditions are examined by geotechnical engineers which help determine the type and thickness of soil layers, location and depth of ground water level and any environmental concerns of the area that pose a threat to the bridge construction [7].The soil conditions are very critical to any construction project, it helps determine the bearing pressure and settlement of soil plus whether deep or shallow foundation should be used. Having sound knowledge in soil type and its geological setting, helps the designers construct a structure that can stay strong without the need of regular maintenance and upgrades throughout its life period considering technical, economic and environmental factors.

5.3.1.4 Seismic Conditions

The seismic conditions must be considered when designing a bridge especially if the bridge is located in an earthquake prone area or in an area which is under frequent seismic loads. The bridge type, configuration and layout all contribute significantly to the seismic performance of the bridge. For example, bridges which have wide piers or top cap, to prevent span loss and allow for movement that minimize the seismic loading on the substructure, simple spans should be used [8]. The bridge configuration for seismic loading must focus on its simplicity, symmetry, regularity, integrity, deformation capability, and reparability [8]. The performance of the bridge can be improved which is located in an area which is prone to constant seismic loading by including the following three guidelines: serviceability, ultimate and survivability limit states [8]. The purpose of serviceability limit state is for the structure to survive a moderate earthquake with minor damages and doesn't affect the vehicular traffic over the bridge [8]. The purpose of ultimate limit state is to maintain the structural integrity even during a higher magnitude earthquake, minimize damage and prevent structure loss [8]. The purpose of survivability limit state is to prevent bridge collapse after a severe earthquake which is quite rare [8].

5.3.1.5 Weather Conditions

The weather conditions need to be accounted for when constructing a bridge as it can cause damage, reduce performance, and limit the accessibility of bridges. The bridges need to continue serving the transportation needs even during extreme weather conditions unlike buildings. The bridge structure needs to be enhanced to handle wind, precipitation, cold and hot conditions. The Tacoma Narrows Bridge that collapsed in 1940, upon investigation, it was reported that the structural design was not designed for wind loads and twisted and collapsed when it faced 42mph wind speed [9].

5.3.2 Site Impacts

This section provides us information on how the bridge itself impacts its surroundings. A bridge which satisfies all the other bridge requirements but fails to satisfy the impacts which it has on the environment which in turn will affect human and aquatic life will be rejected by the local authorities. This section will talk about the vibration, noise pollution, air pollution and water pollution caused by construction activities.

5.3.2.1 Vibration and Noise Pollution

Vibration and noise pollution during construction cannot be extinct but can be reduced to a desirable level to minimize the inconvenience and impacts on local communities and environment. The use of large and heavy machinery contributes a major chunk to vibration and noise pollution. The vibration and noise from construction should be limited to standard construction hours where possible. If any work that needs to be done outside the standard construction hours, special permission should be taken in advance from the local municipality. The use of squawkers for reversing vehicles, acoustic sheds and noise barriers should be made mandatory where possible. The vibration and noise pollution also affect the construction workers on site, proper training and guidance must be provided to workers to wear PPE on site and keep a safe working distance for noise causing activity. Notice in advance and Proper communication with nearby residents and business should be carried out before carrying out major noise causing activity. The ground vibration can also affect the surrounding areas causing major changes in soil layers which can lead to serious consequences and hefty fines.

5.3.2.2. Air and Water Pollution

The construction industry is one of the leading contributors to air and water pollution. About 4% of particulate emission comes from the construction industry [10]. It is also the industry that is causing the most water pollution than any other industry [10]. During the construction process, the dust created from the construction activity can travel long distances affecting the air quality and causing serious health issues. Land clearing, demolition of existing structures and operation of diesel engines are few activities that are contributing to air pollution. The dust particles from construction are classified as PM10 (particulate matter) which has a diameter less than 10 microns and cannot be seen by a naked eye [10]. The use of water sprays to dampen the site and cover piles of materials and truck loads, screen the site to prevent spreading of dust are few of the techniques to control air pollution [10]. The deposition of sediments in the water beneath the bridge during its construction affects the aquatic habitat and clogs waterways. Construction materials like heavy metal, debris, oil spillage, toxic substances that either fall straight into the water bodies or absorbed by the soil which is later carried into streams, lakes and rivers are the major contributors of water pollution. To minimize the impact, the construction team needs to make sure that the work is being carried out according to the codes and regulations provided and have a zero-tolerance policy.

5.4 Government Regulation

Government regulation is important and must be followed for construction design. It provides a criterion of how bridges need to be designed and constructed. The following section will provide the essential regulation required for bridge construction.

5.4.1 Required Design Codes

The design codes are standards or guidelines set by the government to evaluate whether the bridge design meets all the structural and safety requirements such as strength, durability, serviceability, etc. The design codes will differ from region to region, for instance if the bridge is being constructed in Canada, the design should satisfy Canadian Highway Bridge Design Code CSA S6-14, 2017 revision. Alternatively, if the bridge is designed and constructed in America, the design should satisfy AASHTO LRFD 2014-17 Bridge Code. Also, other national codes that will have an impact on the design process should be considered such as CSA A23.1-14 for concrete materials, CSA A23.3-14 for concrete structures and CSA S16-14 for steel structures.

5.4.2 Applicable Additional Requirements and Regulations

Alongside the design codes, there are additional requirements and regulations that must be followed before the construction process can begin.

5.4.2.1 Construction Permits

Before any type of construction can begin, it is mandatory to obtain the construction permit from the local authorities. It is mandatory even if the bridge is under renovation, being demolished or changing the use of the bridge. The construction permits ensure that the work is done in accordance with the local standards in regard to land use or zoning. The design should adhere to any local, provincial or national regulations in all aspects in order to start the construction process. Any issue with the permit, can cause delay to the project timeline and also increase the cost.

5.4.2.2 Traffic Permits

Traffic Permits are required when construction activities use transportation infrastructure such as local streets, laneways, sidewalks, bicycle lanes, etc. They are issued by the local authority. The traffic permits include the traffic management plan and traffic control plan. The traffic management plan helps local traffic navigate in and around the construction zone and also protects workers from live local traffic. The traffic control plan is submitted to the local authority outlining the type of temporary traffic control devices they will be using and its placement on the map is provided. The designer must account for local traffic as it will impact the material and machinery delivery process and other potential issues. At times, use of police constable or traffic coordinator is required to guide the pedestrian and vehicular traffic through the construction zone which adds to the construction cost, must also be considered.

5.4.2.3 Safety Regulations

During the construction process, the region's health and safety regulations must be followed to do work in a safe manner. For instance, if the work is taking place in Ontario, it should follow the guidelines in the Occupational Health and Safety Act.

5.4.2.4 Noise Regulations

The amount of noise created during construction work needs to be monitored and regulated to keep it at a desirable level, to avoid safety risk to the workers, or the nearby residents. The use of heavy machinery is one of the major causes of noise pollution at construction work zones. There are noise regulations that the construction companies need to adhere to. It varies from region to region. In the city of Ottawa, all the heavy civil work must take place in the allotted time period between 7:00am and 11:00 pm [11]. A special permit must be obtained in case of emergency or special cases.

5.5 Constructability

Constructability is a project management technique used to evaluate and assess construction processes from start to finish [12]. It is an iterative process because of the complex nature of the designs involved in bridge construction with an infinite number of potential solutions. In order to come up with a good design, it is required to go back and forth between the designers, owners and the builders. Before beginning any construction on site, the owner, designer, builder and all the other parties involved in the project need to reach an agreement on the constructability of the project and also have a backup plan ready if things don't go as planned. The word constructability defines the ease and efficiency with which structures can be constructed [12].

5.5.1 Construction Knowledge

Construction knowledge is one of the important factors for bridge constructability in order to construct it efficiently, economically as well as in a safer and faster manner. The effective use of construction knowledge into the conceptual planning, design, construction, and field operations of a project helps fulfill the overall project target in the best possible time with precision and also in a cost-effective level [12].

5.5.2 Construction Techniques

The construction techniques that will be used to construct the bridge must be communicated clearly between the owner, designer and contractors. This is important since bridge construction is a complex process requiring knowledge and expertise. To reduce the engineering constraints, costs and environmental impacts, it is important to decide which construction technique should be used to erect the bridge. For example, is it efficient whether the concrete used for the structure is precast or cast-in-place. When enhancing or repairing bridges which are close to local vehicular traffic and pedestrians, it is impossible to shut down the entire bridge as it connects two cities. The parties involved in the construction process need to come up with a construction technique that suits the situation. At times, the construction technique discussed on paper is not viable on site with a lot of constraints, therefore alternate construction techniques must be proposed to facilitate the ease of site construction. Use of proper construction techniques can greatly improve the constructability of the project which in turn reduces the cost and time.

5.5.3 Design Consideration

Designers must pay close attention to the construction process. Similar to the construction knowledge, designers should always consider the construction process of any design they performed because its quality and details are closely related to the constructability. Good designs are simple, standardized and modularized, so that the construction process can be repetitive, thus minimizing potential construction errors in the field. Comprehensive considerations should be made in the design office since any on-site adjustments are expensive and time-consuming.

5.5.4 Project Management and Communication

Proper communication and project management go hand by hand. Project management is crucial to keep the project flowing in the correct direction with minimal constraints. In the case of owners, they need to make sure that they have sufficient funds available to complete the project. In the case of designers, project management helps all different personnel to work as a team towards the same objective in an efficient manner. In the case for contractors, good project management helps boost the constructability of the project. Communication and cooperation are vital both in the preliminary planning phase and in the construction phase. In the design process, communication is reflected on how detailed and clear the design drawings are. In the construction phase, communication between all trades, general contractor and subcontractors and between the office and site team is important to get the work done on time. It also helps in resolving daily on-site issues, potential challenges and reporting progress to the region and nearby residents and businesses. Therefore, proper communication and project management will help deliver a high-quality project.

5.6 Conclusion

This section highlights how the designer should communicate clearly with the client regarding scope of design work, budget, schedule and the service life of the bridge to be constructed. This section also highlights how the site location determines what type of bridge should be constructed by assessing the topography, soil conditions, seismic conditions and weather conditions of the site. Next, it highlights the site impacts of bridge construction such as vibration, noise pollution, air pollution and water pollution. Later on, it highlights how the bridge construction needs to abide by required design codes, safety and noise regulation. All the construction permits, and traffic permits must be approved by the local authorities and be up to date. Lastly, it highlights the factors that are vital for an efficient constructability process.

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Chapter 6 – Conceptual Design

Table of Contents

6.1 Introduction
This section discusses three types of girders commonly used in concrete bridges and proposes a prestressed girder design for a conceptual replacement bridge for the collapsed section of the de la Concorde Overpass. Discussion begins by giving a brief overview about the collapsed de la Concorde Overpass. Afterwards, certain design objectives are set to ensure a safer bridge this time. Based on the evaluation against these design objectives, one of the girders will be proposed using a design matrix.

Since the design dimensions are chosen to be different in Part B, design might differ
from what is proposed in this chapter

6.2 Background Information of de la Concorde Overpass
The overpass on the Boulevard de la Concorde was a bridge located near Montreal crossing over Highway (19) Papineau. It was built in 1970 with a life expectancy of 70 years but because of the unfortunate series of events the bride collapsed after 36 years in 2006. The 1970 overpass was designed to not block the visibility of the highway underneath, make future excavation easier and pass the opening with near constant depth. It was an elegant, innovative structure at its time, but it had flaws.

Figure 6.2.1, 6.2.2, 6.2.3 show dimensions of the overpass.

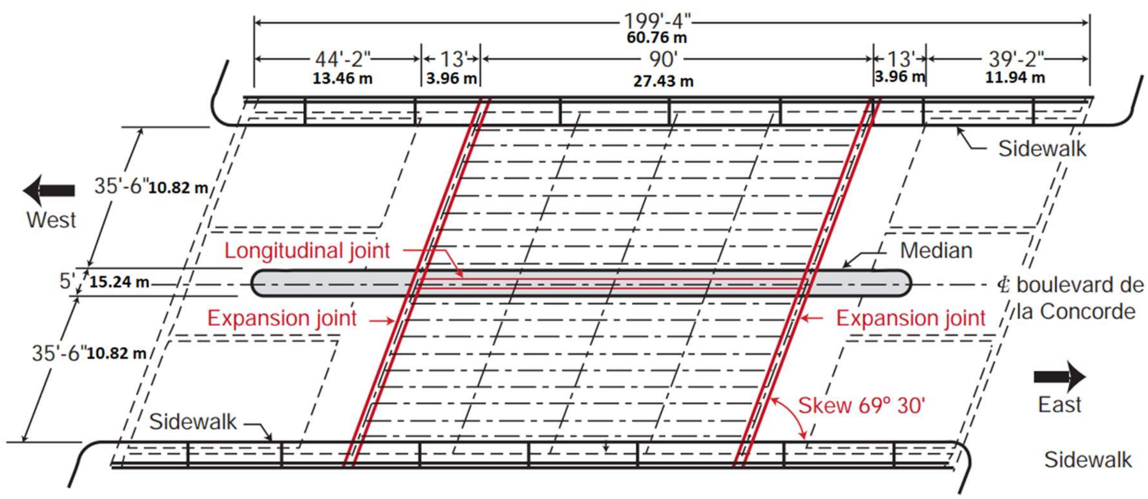

Figure 6.2.1 - Plan View of the de la Concorde Overpass from Autoroute (Highway) 19
Reference: Exhibit COM-62, p. 7 (Court Materials)

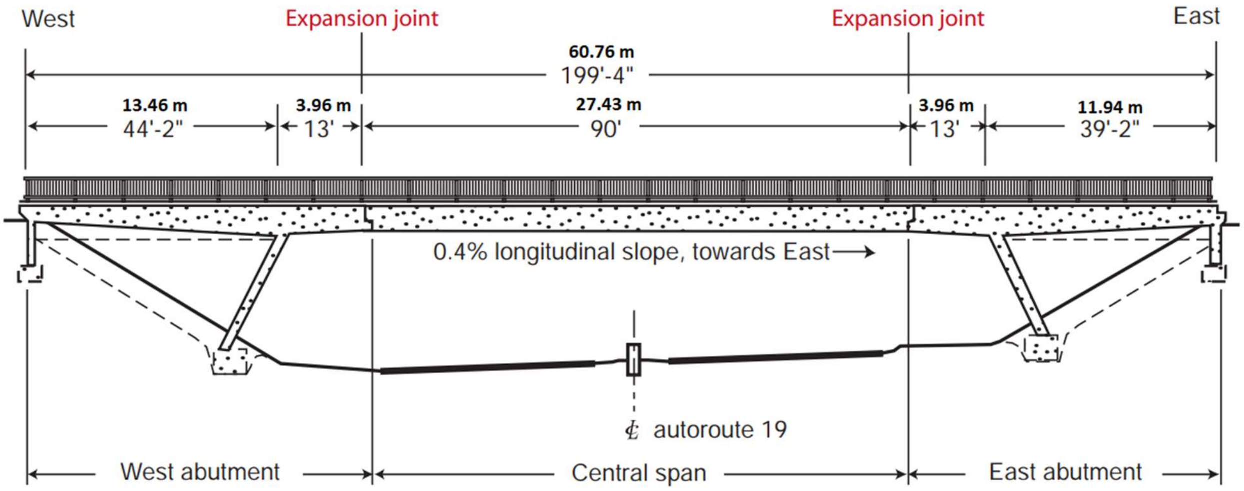

Figure 6.2.2 - Elevation View of the de la Concorde Overpass from Autoroute (Highway) 19
Reference: Exhibit COM-62, p. 7 (Court Materials)

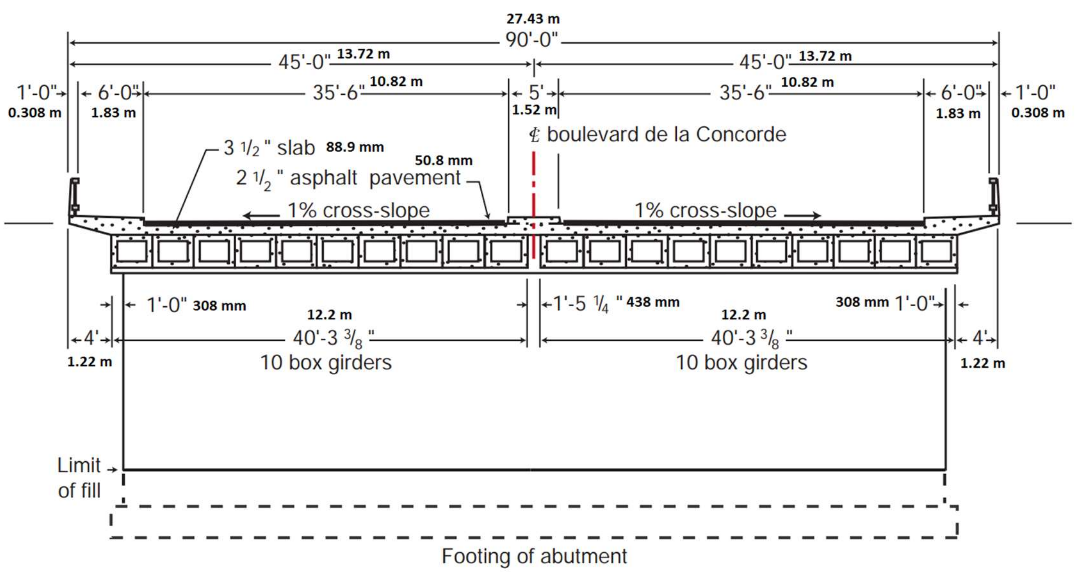

Figure 6.2.3 - Cross-section of the de la Concorde Overpass from Autoroute (Highway) 19
Reference: Exhibit COM-62, p. 7 (Court Materials) - We converted the dimensions to SI units.

The cantilever section of the overpass designed and what was actually built had slight differences. From the images below the differences can be seen visually. In 2006, this portion of the overpass collapsed due to shear failure.

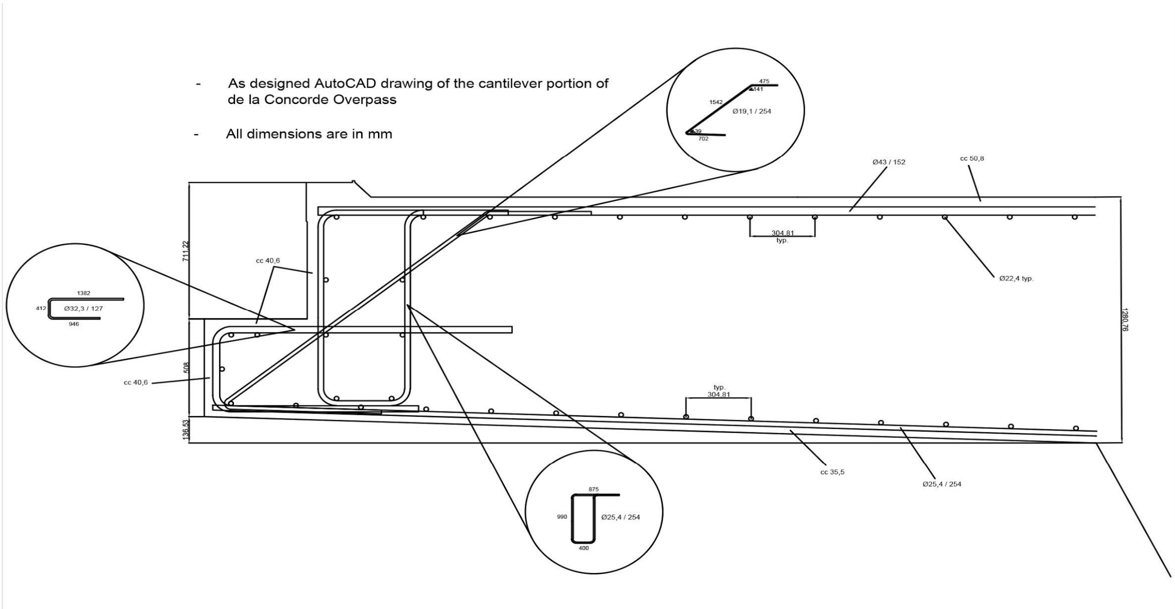

Figure 6.2.4 - Our AutoCAD drawing of the cracked cantilever portion of de la Concorde Overpass (as designed)

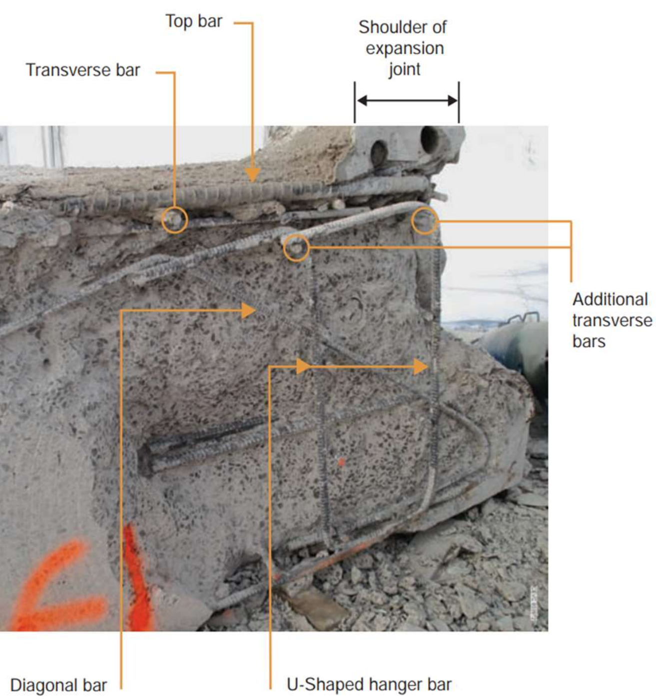

Figure 6.2.5 - Bars at bearing support area as built [5]

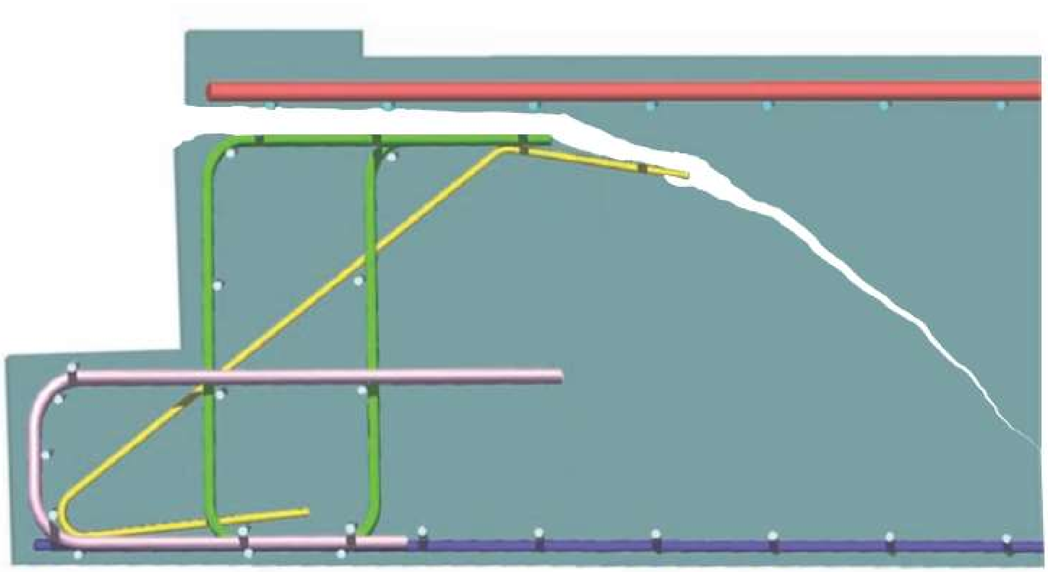

Figure 6.2.6 - The crack that occurred at the cantilever portion of de la Concorde Bridge (taken from [5] and edited to show the crack)

6.3 Conceptual Design Objectives

6.3.1 Bridge Performance

6.3.1.1 Structural Performance

In a replacement for a bridge that collapsed, this is a very important factor. The design of the new bridge should follow CSA S6-14 Revision 17. The new design should deeply investigate the lessons learned from the previous collapse and also look at other collapses. The mistakes made shouldn't repeat at no cost. Using the advances in technology, a finite element analysis is suggested to minimize risks and better understand load distribution inside microelements of the structure.

6.3.1.2 Durability

The replacement bridge should have a life expectancy of 100 years. It should consider all the provisions for durability stated in CSA S6-14 Revision 17.

6.3.2 Uniformity and Aesthetics

The replacement bridge should consider the collapsed overpasses appearance while making it better. The outside look of the bridge shouldn't disturb the public.

6.3.3 Constructability

The replacement bridge should be constructible with a practical solution within the required time frame meeting the budget requirements. Construction of the new bridge should consider the location for ease of construction. It also should consider future maintenance and should be constructed in a maintenance friendly way.

6.3.3.1 Costs

Designers should still consider costs even though a replacement is planned. Things done for cutting costs that disobeys the code in any way should never be done or even considered. In a developed country like Canada, manpower costs significant money so the construction method and timeframe should be chosen accordingly. Material and equipment cost should also be considered.

6.3.3.2 Ease of Construction

The design shouldn't be something very hard to construct. Using similar bar sizes, not adding things that could be skipped since it is hard to install, or reach should be avoided.

6.3.3.3 Ease of Maintenance

Critical areas of the new bridge should be open to easy investigation. Bearings should be accessible. Regular investigation schedules should be recorded and applied.

6.3.4 Other Considerations

The impacts of construction on the surrounding community and environment should also be considered. The bridge will likely have a positive impact on the community once finished and negative impacts on the environment during construction. These negative effects should be minimized.

6.3.4.1 Community Impact

A new construction has some negative impacts on the surrounding community. The traffic that occurred after the collapse might get worse so this should be considered. Also, local sound level regulations and hours of allowed construction should be obeyed.

6.3.4.2 Environment Impact

Environmental impacts should be minimized using sustainable equipment and using an appropriate construction method if budget permits.

6.4 Proposed Conceptual Design

In this section, three proposed conceptual designs are presented for the replacement of the collapsed section of the De la Concorde overpass. The preliminary design specifications, advantages and disadvantages are determined for each proposed design.

6.4.1 Precast Post-tensioned Box Girder

A precast post-tensioned box girder is a suitable design to replace the collapsed eastbound segment of the bridge because it is similar to the westbound segment of the bridge in terms of visual characteristics and uniform geometry. The box girder is ideal for spans from 15m to 36m [1]. The collapsed segment of the bridge is 19.8m in length, thus satisfying the ideal span range criteria. The option of precast sections is recommended overcast in situ sections in order to minimize the on-site operation time and variability in quality from site conditions. High strength concrete material with a compressive strength of over 35 MPa will be utilized for the precast post-tensioned box girder. A combination of steel rebars and prestressing tendons will be placed in the top and bottom flange. This helps with resisting both positive and negative moments as well as provides better crack control under both dead and live load. The use of additional steel bars in the top flange will help with the transverse reinforcement requirement for shear failure. In terms of geometry, 5 box girders will be used, each with a span length and width of 19.8 m and 2 m respectively. The installation of box girders will be done in sections using a special type of crane as shown in Figure 6.4.1.1. The bridge deck and the remaining surface works will be constructed once all the box girders are installed. Proper traffic control plans will be set up for the lane closures underneath the bridge for vehicular safety.

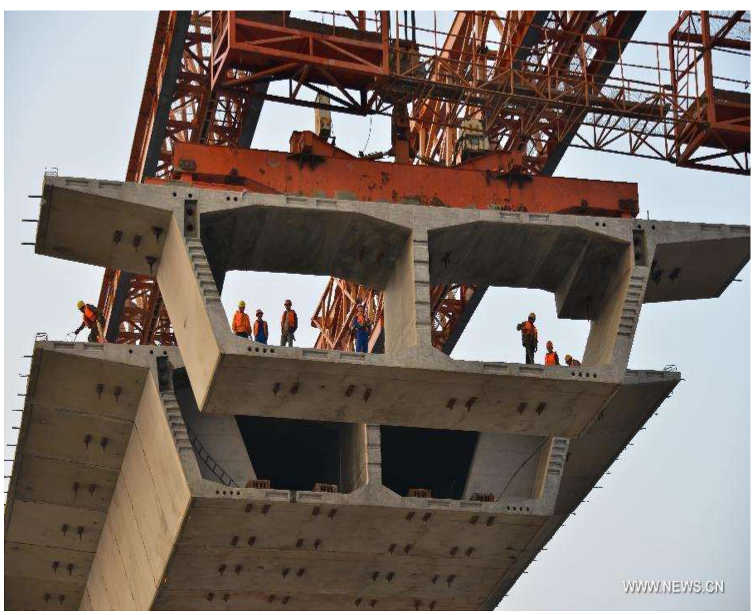

Figure 6.4.1.1 - Construction of Box Girder in section using special type of cranes [2].

6.4.1.1 Advantages

-Structural Performance: The closed structure of the box girder has strong torsional rigidity and both the flanges can resist stress and positive and negative bending moments [1]. It also has a better load distribution when exposed to eccentric loading [1]. The long span of box girders reduces the need for support points [2].

-Aesthetics: It is aesthetically appealing as it is similar to the existing bridge in terms of shape and appearance and doesn't block the view when seen from the ground level.

-Cost: Although the prefabrication of box girders is expensive as well as requires the use of special cranes to install, the high quality and less on-site construction time are much greater in terms of cost savings. Also, it has high structural efficiency which implies the use of less material and therefore reduces cost.

-Uniformity: The existing westbound portion is also constructed using box girders, thus using box girders for the collapsed eastbound section adds to the uniformity of the bridge.

6.4.1.2 Disadvantages

-Ease of Construction: The transportation of large prefabricated girders requires proper coordination in order to make sure they arrive on time without any discrepancies in order to avoid any logistical efficiencies. Also, this extensive transportation of prefabricated girders has high transportation costs.

-Ease of Maintenance: Since the box girders are closed structures, all the inspection and repair work will be done from the exterior as the interior of the box girder will be difficult to access.

6.4.2 Extradosed Bridge

This conceptual design will start from scratch and replace the entire bridge including the abutments and piers in order to avoid any design or construction errors. The extradosed bridge is a structure that looks like a cable stayed bridge and the deck is composed of a prestressed box girder section. The construction of the extradosed bridge will have zero influence on the traffic flow underneath the bridge. This unique style of bridge will create an important landmark in the memory of the previous collapsed bridge. The design of the extradosed bridge will include one tower at the center of the bridge with the cables acting as tendons to prestress the box girder section. The deck of the bridge is shallow when compared to the cable stayed bridge. The optimal span length of the extradosed bridge is between 100 m to 250 m [3]. The De la Concorde Overpass has a total span length of 60m, thus satisfying the span length criteria by a big margin. The ideal height to span ratio of the tower must be around 1:10 [3]. The lower tower height helps create a flatter cable angle roughly 17 degrees to the horizontal and higher axial compressive force [3]. The cables carry only 20% of the live load, rest is carried by the prefabricated box girder [3]. Figure 6.4.2.1 shows the comparison between extradosed and cable stayed bridges. The precast box girders will be assembled using the balanced cantilever segment approach, with cables attached along with the completed box girder deck as shown in Figure 6.4.2.1.

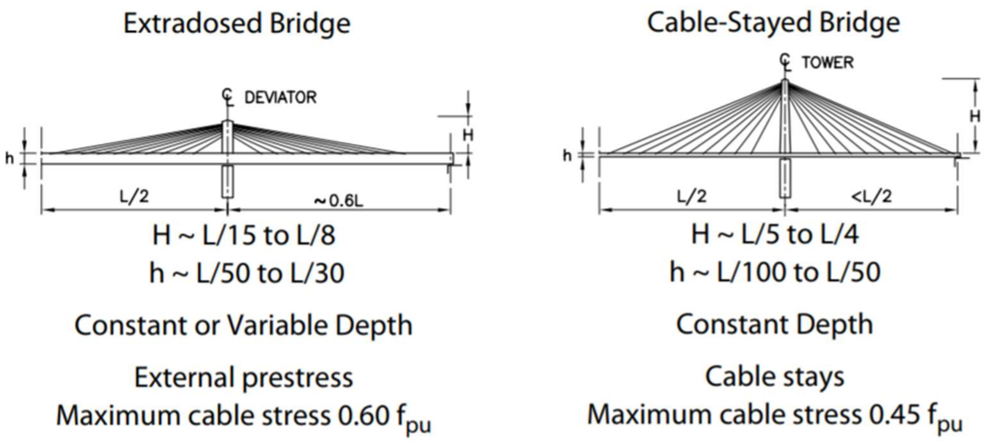

Figure 6.4.2.1 - Comparison between Extradosed and Cable Stayed Bridge [3]

Figure 6.4.2.2 - Balanced Cantilever Segment Approach

6.4.2.1 Advantages

<u>-Ease of Construction:</u> The cantilever segment approach is an economical type of construction method that offers repetitive types of work. The construction work can be done in a top down fashion, thus minimizing the use of any temporary support [4].

-Aesthetics: The shape and appearance of the extradosed bridge is aesthetically appealing because of the fan shaped cables attached between from the tower to the cable. Its unique design is dedicated towards the remembrance of the lost lives during the bridge collapse.

-Structural Performance: There will be no need to check for any defaults in the existing section of the bridge as the whole bridge is being built from scratch with new technology and latest design codes. Also, the steel cables attached between the deck and the tower are less sensitive to vibration. The high stiffness of the deck will reduce the deformations under live load.

6.4.2.2 Disadvantages

-Cost: The cost would be higher for the extradosed bridge because of high material consumption used in the construction of the tower and cables. The balanced cantilever method has significant unbalanced loads on the piers and foundations, thus requiring strong expensive foundations [4]. Furthermore, it is not economical to build bridges of shorter spans less than 100 m [3].

-Ease of Maintenance: The steel cables need to be inspected and maintained on a regular basis, as they are susceptible to corrosion. Also, it is difficult to access and conduct inspections at the anchorage area on the tower.

-Community Impact: The construction of the entire bridge will take a longer time due to its complex design. This for sure will increase the construction cost but also at the same time will close the whole roadway rerouting the people to other routes causing heavier delays.

6.4.3 New England Bulb Tee (NEBT) Girder

This conceptual design will start from scratch and replace the entire bridge including the abutments in order to avoid any design or construction errors. NEBT Girders look like AASHTO I girders but they have some differences. NEBT Girder suggested for the replacement bridge is NEBT 1400 having a girder depth of 1.4 meters. When bending capacity is examined, this performs better than PCI BT 54 Girder and AASHTO TYPE 4 girder [7]. Deck can be 225 mm with 2% slope each side to prevent water accumulating. Asphalt and waterproofing systems are suggested at a thickness of 65 to 90 mm (typ.). CL 625 Truck load and lane loading combinations should be examined for every 10% of the simply supported span. There will be one pier in the middle. Approximately 30m of two simply supported spans are expected. A total of 24 30 m NEBT 1400 Girders are expected. An earthquake design is suggested for the piers and abutments with a peak horizontal ground acceleration (PHA) 0.08 and a zonal acceleration ratio of 0.10 g.

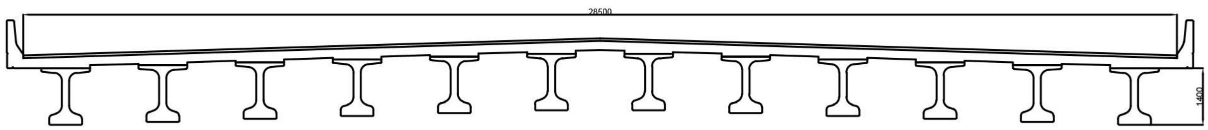

Figure 6.4.3.1 - AutoCAD Drawing of cross-section of proposed replacement bridge @10 m.

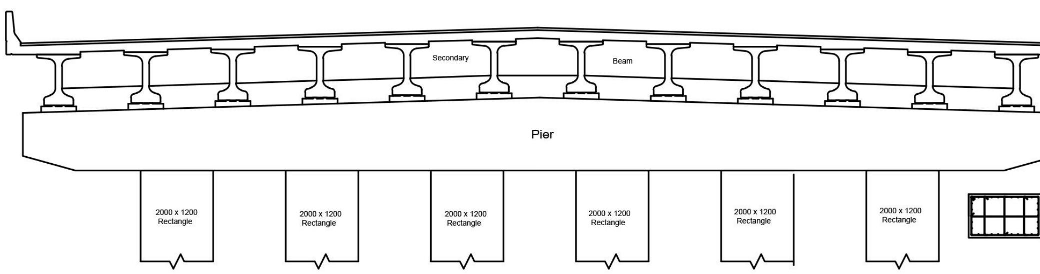

Figure 6.4.3.2 - AutoCAD Drawing of midspan cross-section of proposed replacement bridge

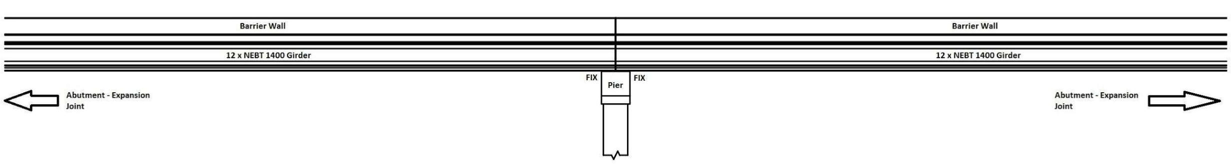

Figure 6.4.3.3 - AutoCAD Drawing of elevation view of proposed replacement bridge

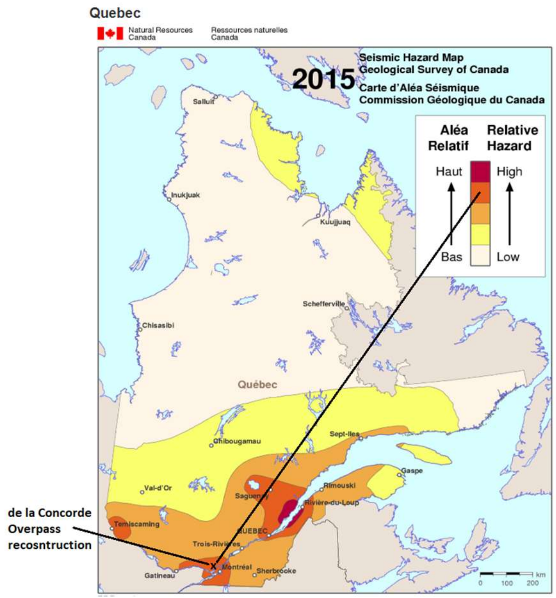

Figure 6.4.3.4 - 2015 Seismic Hazard Map of Quebec [6]

<u>Some possible design properties:</u>
Precast girders: f'_c 50 MPa concrete*
*A minimum strength of 35 MPa is suggested at transfer, 30 required
Slab: f'_c 25 MPa concrete
Remainder reinforced concrete: f'_c 40 MPa concrete
Clear cover deck top: 75 +- 10 mm
Clear cover Bottom of deck 40 +- 10 mm
Clear cover Remainder 60 +- 10 mm
Reinforcement: Standard f_y= 400 MPa Canadian Reinforcement
Prestressing strands in girders: Stress-Relieved 7 wire: f_u = 1860 MPa
Force per strand after losses should be at least 100 kN at any cross section

6.4.3.1 Advantages

<u>-Cost:</u> Precast girders are cheaper than post tensioning equipment and bridges like extradosed bridges. This design is one of the cheapest possible solutions for a span like this.

<u>-Ease of Construction:</u> The precast NEBT girder approach is very easy and fast to install. Since there is a major highway under where the construction is planned, the precast girders can be brought by large trucks and installed fast. The construction of the pier may require lane closures, but a pier is usually necessary for this kind of span and depth requirements for most other types.

-Ease of Maintenance: Can be maintained by a temporary pier. Lots of space under to work. The concrete used in girders is high quality making maintenance less problematic.

-Structural Performance: Structural performance of NEBT 1400 girder system proposed is pretty good under dead, live and lane loading. Since the bridge proposed is designed for earthquakes as well, the piers and abutments are able to handle static loads with ease.

6.4.2.2 Disadvantages

-Aesthetics: The shape and appearance of the NEBT girder bridge is aesthetically similar to other highway bridges in the area. Most people won't notice a difference between other bridges.

-Community Impact: This construction will cause delays while the pier gets installed. Highway needs to be partially or fully closed for installation of the girders which will make some people upset.

6.5 Evaluation

The chart below is an evaluation matrix which gives scores to the proposed bridges above based on the design objectives mentioned. Structural performance is obviously the most important factor for a replacement bridge for a collapsed bridge. We gave a weight of 20 % to structural performance. Durability, ease of construction and ease of maintenance have a weight of 15 % as we think they are the second most important consideration. Then comes the uniformity with 10 % and it is followed by aesthetics, community impact and environment impact at 5%.

Points are given out of 10 and then taken as a weighted average.

Factor	Weight (%)	Box Girder	Extradosed Bridge	NEBT Girder
Structural Performance	20	8	8	9
Durability	15	7	7	9
Uniformity	10	8	6	8
Aesthetics	5	6	9	5
Costs	10	7	5	10
Ease of Construction	15	7	6	8
Ease of Maintenance	15	6	7	8
Community Impact	5	7	6	4
Environmental Impact	5	5	5	8
Sum	**100**	**7**	**6.7**	**8.2**

6.6 Conclusion

This chapter analyzed three conceptual designs for the replacement of the De La Concorde Overpass in order to determine which design option is the best based on the following factors: structural performance, durability, uniformity, aesthetics, cost, ease of construction, ease of maintenance, community impact and environmental impact. Each of the factors was given a certain weightage, with structural performance having the highest weightage in order to avoid another similar collapse. The first design is the precast post tensioned box girder, the second design was the replacement of the whole bridge by an extradosed bridge and the third design is a I Girder bridge. Based on the factors provided in section 6.4, the Bulb Tee girder was ultimately recommended.

6.7 References

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Chapter 7 – Structural Analysis

Table of Contents

7.1 Introduction

Structural analysis is an essential step in the bridge design process once the conceptual design is finalized. It helps provide engineers and designers with the required information such as loading and deflection as well as provides a check if the design satisfies the bridge standards and codes. This chapter provides a general overview of structural analysis followed by various types and steps of analysis. Lastly, it also mentions the common software used for structural analysis.

7.2 Structural Analysis

In this section, the required inputs, equilibrium and compatibility equations, and structural indeterminacy will be discussed

7.2.1 Required Inputs

7.2.1.1 Materials

Materials are very important as that is what everything is made of. Material properties are very important. Materials have different properties. Most common materials used in construction are steel and concrete.

The following properties must be known by the engineer to be able to do a structural analysis and design:

If only linear behavior is considered (Stiffness doesn't change with displacement):

Steel:
-Modulus of Elasticity (Usually 200000 MPa, sometimes 195000 MPa)
-Density
-Shear Modulus (Only if shear deformations or torsional deformations are considered, otherwise not to be specified or G must be a very large value with poisson's ratio being 0)
-Poisson's Ratio (Only if shear deformations or torsional deformations are considered)

Concrete:
-Initial Modulus of Elasticity
-Modulus of Elasticity 28-Days
-Shear Modulus
-Poisson's Ratio
-Density
-Thermal Coefficient

If non-linear behavior is also considered, these additional factors should be considered:

>Stress-Strain Curve or some points of it so curve fitting can be done to update stiffness matrix
Some important points are:
-Yield Strength
-Ultimate Strength
-Ultimate strength in concrete and f'_c

7.2.1.2 Geometry and Cross-Section Properties

The conceptual design determines the overall geometry of the structure. The type of bridge that is being recommended and the site characteristics are the main components to determine the geometry of the bridge. The span length, width and depth of the main deck, width of flanges, spacing of the girders are integral structural components and its geometry must be determined carefully to prevent structural failures or collapses. The geometry of the individual components will vary depending on the structural component used such as box girder, bulb girder, etc. Therefore, the designer should come up with a geometry that is structurally stable and cost efficient.

Each section consisting of the geometry explained above must be then considered in analysis. Sections can be:

- Drawn in a built-in section designer where users can draw the section (SAP2000, CSI Bridge Section Designer).
- Drawn in a CAD software like AutoCAD and imported as a DXF file (Useful for 3D Model).
- Chosen from the default sections available in the software package.
- Drawn in an external software related with the main analysis software (RISA Section ... etc.)

Reinforcement detailing is also included during this process.

Sections can also be determined by directly specifying properties. In this case the following properties should be specified:

- Cross-Sectional Area (A)
- Moment of Inertia about z_axis (I_{zz})
- Moment of Inertia about y_axis (I_{yy})
- Moment of Inertia about x_axis (I_{xx})

Moment of Inertia having the same axis with the member has a special name. It's called Torsional Constant and Shown with (J). This can be one of the three of the above depending on the member.

7.2.1.3 Boundary Conditions and Initial Conditions

In structural analysis, for equilibrium to be satisfied, a set of constraints must be applied to the state of the body in question. Boundary conditions are the restrictions that describe the state of a node on the body at a certain location, independent of time. Initial conditions are the restrictions that describe the state of a node on the body at a certain time we pick as start (t = 0), independent of location.

In (2D) plane, the following boundary conditions or a combination of them are usually used in structural analysis:

- Translation in x_axis
- Translation in y_axis
- Rotation about z_axis

Roller Support on the x_axis means at that point; the body is restricted in translation in y_axis. It cannot make any displacements in y_direction. It can freely rotate about_z axis and move in the x axis.

A pin support in 2D restricts translation in x_axis and translation in y_axis, however, allows rotation aboux z_axis.

For some specific applications, the following boundary conditions can also be used:

Partial translation in x_axis with resistance stiffness Kx
Partial translation in y_axis with resistance stiffness Ky
Partial rotation about z_axis with resistance stiffness Kz

In (2D) plane, the following initial conditions or a combination of them are usually used in structural analysis:

Note: Usually initial conditions are assumed to be 0

- Acceleration in x_axis
- Acceleration in y_axis
- Rotational acceleration about z_axis

- Initial velocity in x_axis
- Initial velocity in y_axis
- Rotational velocity about z_axis

In (3D) space, the following boundary conditions or a combination of them are usually used in structural analysis:

- Translation in x_axis
- Translation in y_axis
- Translation in z_axis
- Rotation about x_axis
- Rotation about y_axis
- Rotation about z_axis

7.2.1.4 Dynamic Loading Properties

Most common dynamic loads consist of earthquakes and wind loads. Earthquake analysis can be done in various ways.
For response spectrum analysis, the following inputs are required:

- The response spectrum for that specific location (interactive seismic maps) or a code standardized spectrum
- Mass source (Can be structure mass or external loading converted to mass)
- The contribution of maximum number of modes to be considered (For performance purposes. Can be chosen as much as DOFs exist in the structure for small stuff)
- Modal Combination Method (Since adding everything will be very conservative -SRSS - CQC)
- Damping Ratio for each mode or a method to determine damping (Modal, Rayleigh…)

For time history analysis, the following additional inputs are required:

- Earthquake acceleration vs time data
- Time Integration Method (Average acc, Newmark)
- Output Time step numbers and time steps size (Dt)

For wind analysis:
1st approach is to assign them directly as static loads (Good for simple structures)
2nd approach is to do a wind tunnel analysis:
Input required for wind tunnel analysis are the following:

- Mesh Density (No of small elements within space)
- Wind Speed and Direction

- Output Time step numbers and time steps size (Dt)
- Air density
- Temperature

7.2.1.5 Loading and Load Combinations

In order to perform structural analysis, having everything inputted, loads can be applied. Loads can be distributed loads, area loads, point loads, moving loads and time dependent loads. After determining these, the load combinations can be calculated. Usually design is based on the critical combinations. Since some members may experience different stresses, usually one combination from static loads, one with the wind and one with earthquake loading is chosen for the design. Depending on the region of construction, one combination may hugely dominate for every member and chosen for design as the only combination. This is crucial as it will help ensure the load bearing structural members satisfy the required strength and serviceability limits.

7.2.2 Equilibrium Equations

Equations of equilibrium are an important factor in flexibility analysis.

>In Newtonian Mechanics:

The following equations of equilibrium are Valid:

Sum of Forces in x_direction = total mass x resultant acceleration in x_direction
Sum of Forces in y_direction = total mass x resultant acceleration in y_direction
Sum of Forces in z_direction = total mass x resultant acceleration in z_direction
Sum of Moments about x_axis = total mass x resultant rotational acceleration about x_axis
Sum of Moments about y_axis = total mass x resultant rotational acceleration about y_axis
Sum of Moments about z_axis = total mass x resultant rotational acceleration about z_axis

If the mass experiences no acceleration, these equations simplify to:

Sum of Forces in x_direction = 0
Sum of Forces in y_direction = 0
Sum of Forces in z_direction = 0
Sum of Moments about x_axis = 0
Sum of Moments about y_axis = 0
Sum of Moments about z_axis = 0

>In Lagrangian Mechanics:

ACCELERATION = 0

$$\frac{\partial L}{\partial x} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{x}}\right) \qquad \frac{\partial L}{\partial x} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{x}}\right) = 0$$

$$\frac{\partial L}{\partial y} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{y}}\right) \qquad \frac{\partial L}{\partial y} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{y}}\right) = 0$$

$$\frac{\partial L}{\partial z} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{z}}\right) \qquad \frac{\partial L}{\partial z} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{z}}\right) = 0$$

$$\frac{\partial L}{\partial \theta_x} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{\theta}_x}\right) \qquad \frac{\partial L}{\partial \theta_x} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{\theta}_x}\right) = 0$$

$$\frac{\partial L}{\partial \theta_y} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{\theta}_y}\right) \qquad \frac{\partial L}{\partial \theta_y} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{\theta}_y}\right) = 0$$

$$\frac{\partial L}{\partial \theta_z} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{\theta}_z}\right) \qquad \frac{\partial L}{\partial \theta_z} = \frac{d}{dt}\left(\frac{\partial L}{\partial \dot{\theta}_z}\right) = 0$$

Figure 7.2.2.1- Equlibrium Equations in Lagrangian Mechanics

7.2.3 Structural Indeterminacy

7.2.3.1 Statical Indeterminacy

In a flexibility-based analysis approach, a structure is said to be statically indeterminate if the number of equilibrium equations required to solve for the support reactions is not enough. In other words, a structure is indeterminate when another solution other than the trivial solution exists for the system of equilibrium equations.

Degree of Statical Indeterminacy in a 2D Frame can be determined from the following equation:

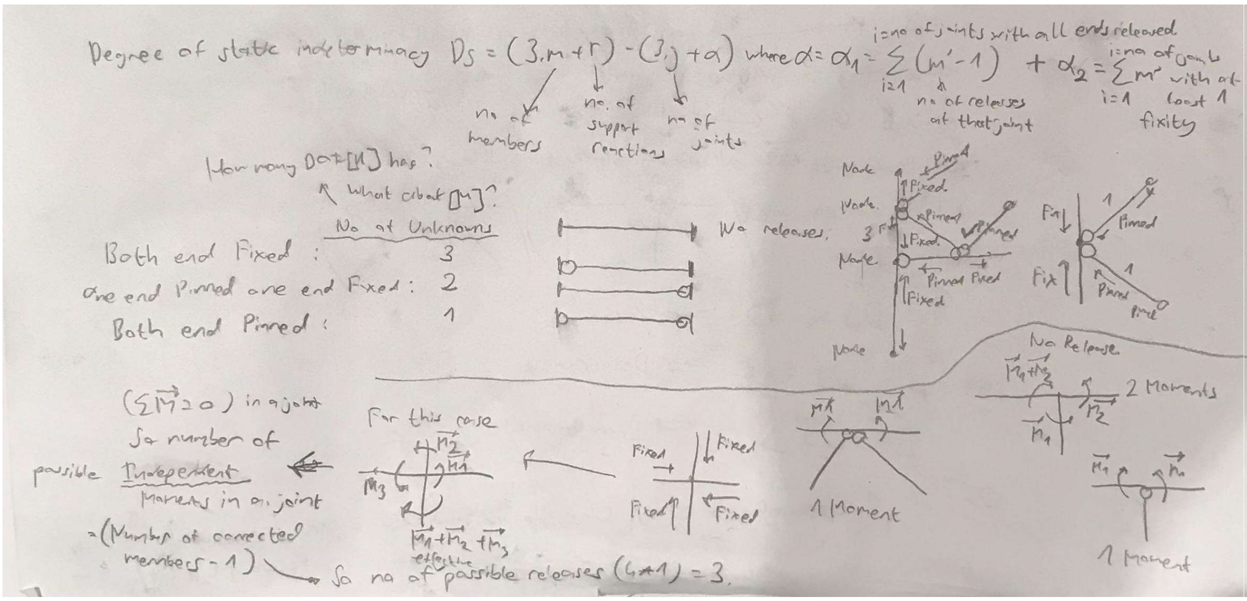


Figure 7.2.3.3.1- Degree of Statical Indeterminacy

7.2.3.2 Kinematic Indeterminacy

Kinematic indeterminacy is the number of independent movements a structure can make at support nodes (non-zero values to be determined in the displacement vector). Which also implies the structure is kinematically indeterminate when there exists at least 1 unknown displacement associated with a degree of freedom (DOF). The number of such unknown DOFs give the number of kinematic indeterminacy.

7.3 Analysis Methods

7.3.1 Hand Calculations

Hand calculations are calculating the stuff by hand as the name suggests. Most of the hand calculations are done using excel or some sort of calculator so what is done actually by hand is the procedure, approximations and evaluation.

Hand calculations are essential for evaluating analysis results obtained from structural analysis software. The engineer should know what the values they enter actually mean and what the software is doing in the background. Nowadays, there is softwares that does everything for the engineer and even produces the application drawings (STA4CAD). These softwares are usually dangerous in the hands of an inexperienced engineer.

7.3.2 Linear vs Nonlinear Analysis

Linear analysis is useful when deformations the member experiences are close to reversible. A member acts linearly if the displacements caused by the loads are a linear combination of the loads (**F** = [K] x **u)**. Designing a structure linearly is usually very costly so engineers tend to go to the non-linear range quite commonly.

Non-linear analysis is done usually when the member behaves non-linearly. Nonlinearity in general means that the forces become a function of displacements. What this means is that as the displacement changes, [K] changes. So, the iterative process is required starting from linear case and updating the stiffness matrix as the loads change and then do another analysis and so on. Non-linearity happens because of several reasons:

>Geometric Nonlinearity

Geometric Nonlinearity happens when the deformations of a member exceeds a certain limit that the deformations start to affect the members response. The stiffness (how much force required to deflect a member by 1 unit at that node) of the structure will change due to new forces generated. Maxwell-Betti Law and small deformation approximations are also not valid in a geometrically nonlinear member.
Geometric nonlinearity usually comes into consideration in the buckling calculations of steel columns, huge deflections of columns during an earthquake (P - Delta Effects).

>Material Non-linearity

Material nonlinearity happens when the stiffness changes due to material properties changing. Materials start behaving nonlinear when they reach their yield point. Some materials have a specific yield point but for some of them, it is very hard to call an exact point a yield point. At a non-linear range before ultimate, materials become ductile: taking load but deflecting even more. After the ultimate, the load required to cause a deformation becomes negative so some negative stiffness values following an instability will be experienced. In design, exceeding ultimate is almost never done and is not acceptable.

>Boundary Condition Non-linearity

Boundary Condition nonlinearity happens when boundary conditions change after some deflection causing immediate changes in the response of the structure if the load is further increased. Ex: The tip of a cantilever beam that touches the concrete wall below due to excessive deflection.

7.3.3 Finite Element Analysis

Continuous members that form the structures have infinite no of internal DOFs. The goal of finite element analysis is to divide the members into smaller members and to approximate the behavior of the main member by analyzing the smaller members and relating them with an approximate shape function. The shape function must satisfy at least the boundary conditions to get accurate results. It is also preferred to have a non-zero second derivative for dynamic analysis.

7.4 Analysis Sequences

This section will discuss the three stages of software modelling processes to perform structural analysis for the bridge design: pre-processing, processing and post-processing.

7.4.1 Pre-processing

The first stage is the pre-processing stage which involves inputting the site conditions of the bridge in the software. The site conditions include details of material property, geometry of design, loads on the bridge and boundary conditions. This is a very crucial stage as mistakes can be easily made at this stage, making the process time consuming. For example, incorrect calculations or human error while entering values for each node and element can lead to unexpected results. To ensure your work's accuracy, it is good practice to have your work double checked by another designer. This will help save time and increase the calculation accuracy.

7.4.2 Processing

The second stage is the processing stage which allows computers to solve any unknown forces, displacements or parameters assigned during the pre-processing stage. This could be done using different types of structural analysis softwares which will be discussed in section 7.4. Each software will have a different approach, but the fundamental goal is the same which is to solve for unknown parameters in the given structure. The different analysis methods mentioned in section 7.2 can be used here to solve for unknowns to get a result which is reasonable from the information provided.

7.4.3 Post-process

The third and final stage is the post-process stage which involves using engineering judgements to check the reliability and accuracy of the model from the processing stage. The results can be validated and sanctioned by comparing results from different softwares used.

7.5 Structural Analysis

7.5.1 RISA 3D

RISA 3D is one of the well-known 3D design and analysis software in the industry. It is a simple, straight forward software which helps you create geometry graphically in two ways. First, by using advanced modeling tools and secondly by using spreadsheets to process data directly. In terms of design, the software has codes for different material types such as steel, concrete, aluminum, masonry and timber as well as codes from different areas in the world such as American, Canadian, European and Chinese codes. Therefore, providing a use friendly platform for engineers to design structures for different areas and materials and ensure adequate designs. Also, RISA 3D can be easily integrated with other programs such as RISA Floor, RISA Foundations and RISA Connection. This helps provide a comprehensive design solution. Furthermore, RISA 3D is compatible with CAD, thereby reducing the learning curve of the software. Overall, RISA 3D is a powerful tool to use because of its user friendly and versatility.

As shown in Figure 7.5.1.1, design codes can be selected according to the purpose of the design. If a design of a bridge is required, use CSA-S6-14 instead of CSA A23.3-14 (blue highlighted section in Figure 7.5.1.1). Figure 7.5.1.2 shows a custom selection of units or standard metric or standard imperial units. Figure 7.5.1.3 shows material properties that are custom defined by the user. For example, rigid tab, modulus of elasticity E is set at a value of 1E+15 because the largest value the software accepts is this. Poisson ratio is set to zero to remove shear deformations from the material (only bending considered in those members). Figure 7.5.1.4 shows different sections that can be entered by the user. Figure 7.5.1.5 shows a variety of sections that a member can be drawn with. Figure 7.5.1.6 shows a moment diagram of 6 m, 3 m and 5 m spanning rigid steel beams under a distributed load of 100 kN/m. Figure 7.5.1.7 shows a sample 3D drawing of a bridge pier.

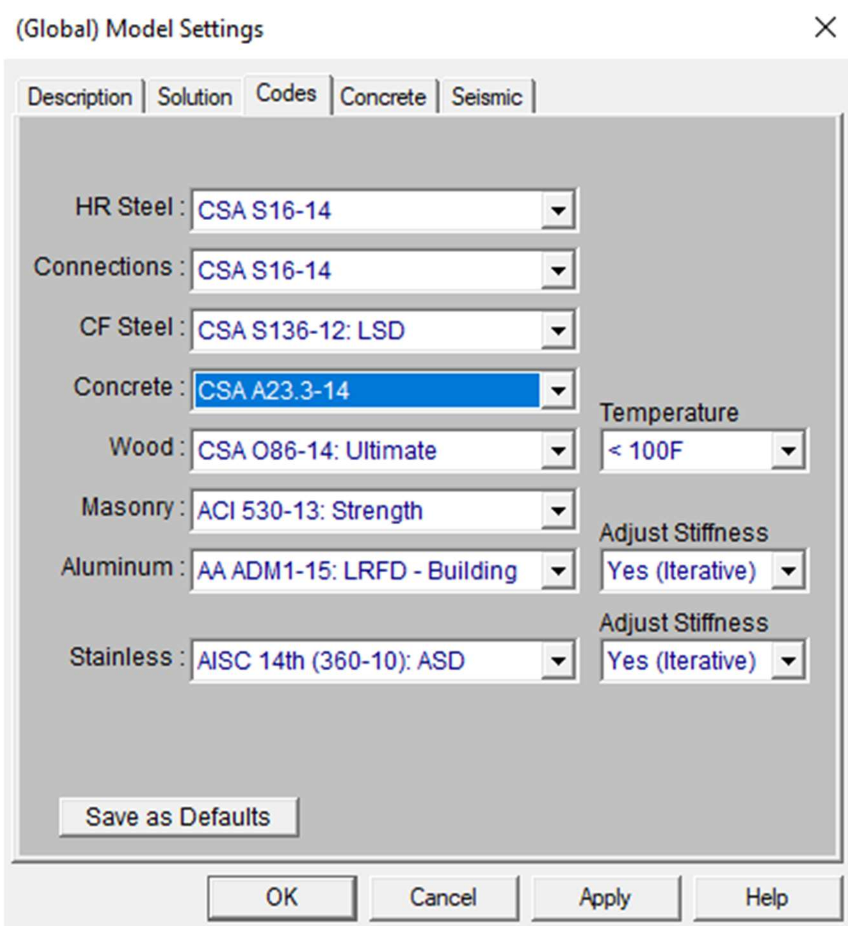


Figure 7.5.1.1- Screenshot of the Design Code Selection Menu [1]

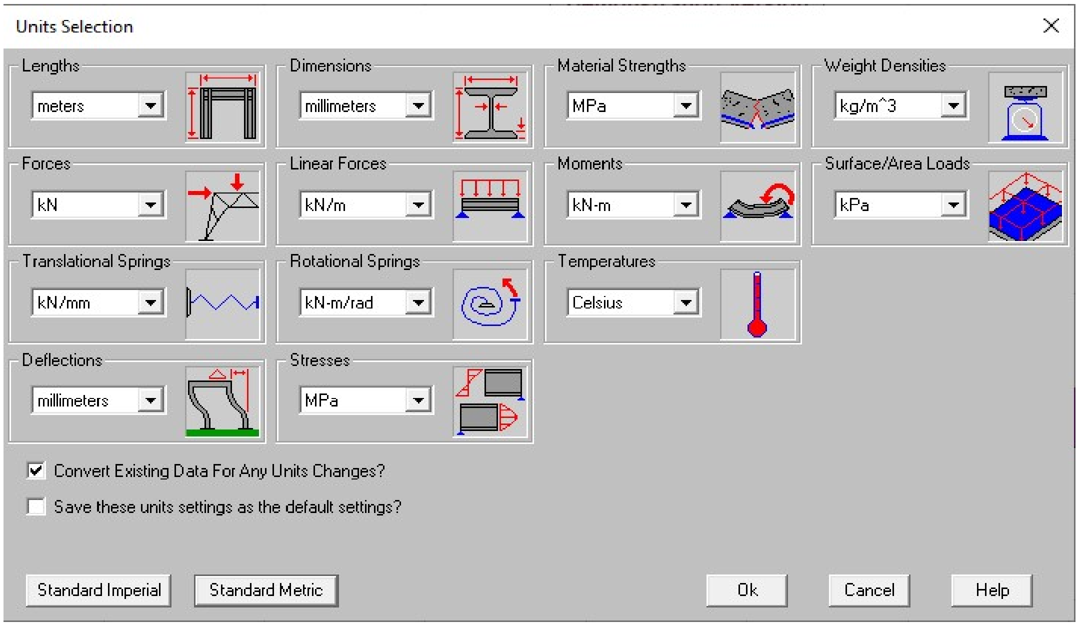
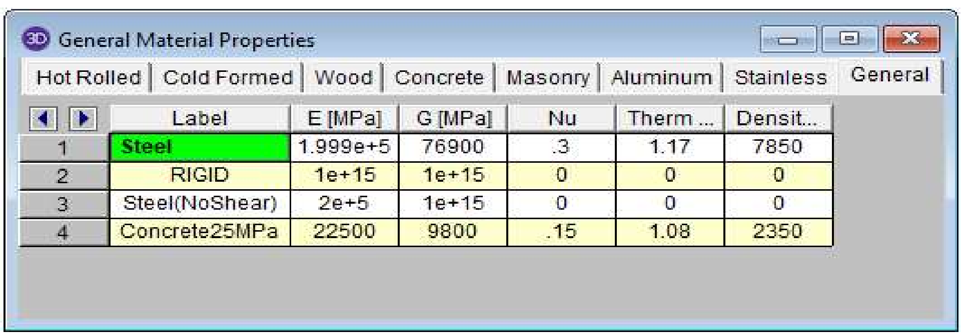
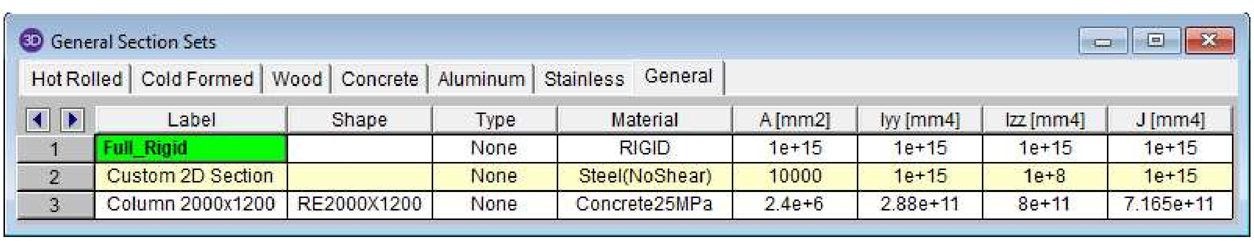


Figure 7.5.1.2- Unit Selection [1]



	Label	E [MPa]	G [MPa]	Nu	Therm ...	Densit...
1	Steel	1.999e+5	76900	.3	1.17	7850
2	RIGID	1e+15	1e+15	0	0	0
3	Steel(NoShear)	2e+5	1e+15	0	0	0
4	Concrete25MPa	22500	9800	.15	1.08	2350

Figure 7.5.1.3- Material Properties [1]



	Label	Shape	Type	Material	A [mm2]	Iyy [mm4]	Izz [mm4]	J [mm4]
1	Full_Rigid		None	RIGID	1e+15	1e+15	1e+15	1e+15
2	Custom 2D Section		None	Steel(NoShear)	10000	1e+15	1e+8	1e+15
3	Column 2000x1200	RE2000X1200	None	Concrete25MPa	2.4e+6	2.88e+11	8e+11	7.165e+11

Figure 7.5.1.4- Section Properties [1]

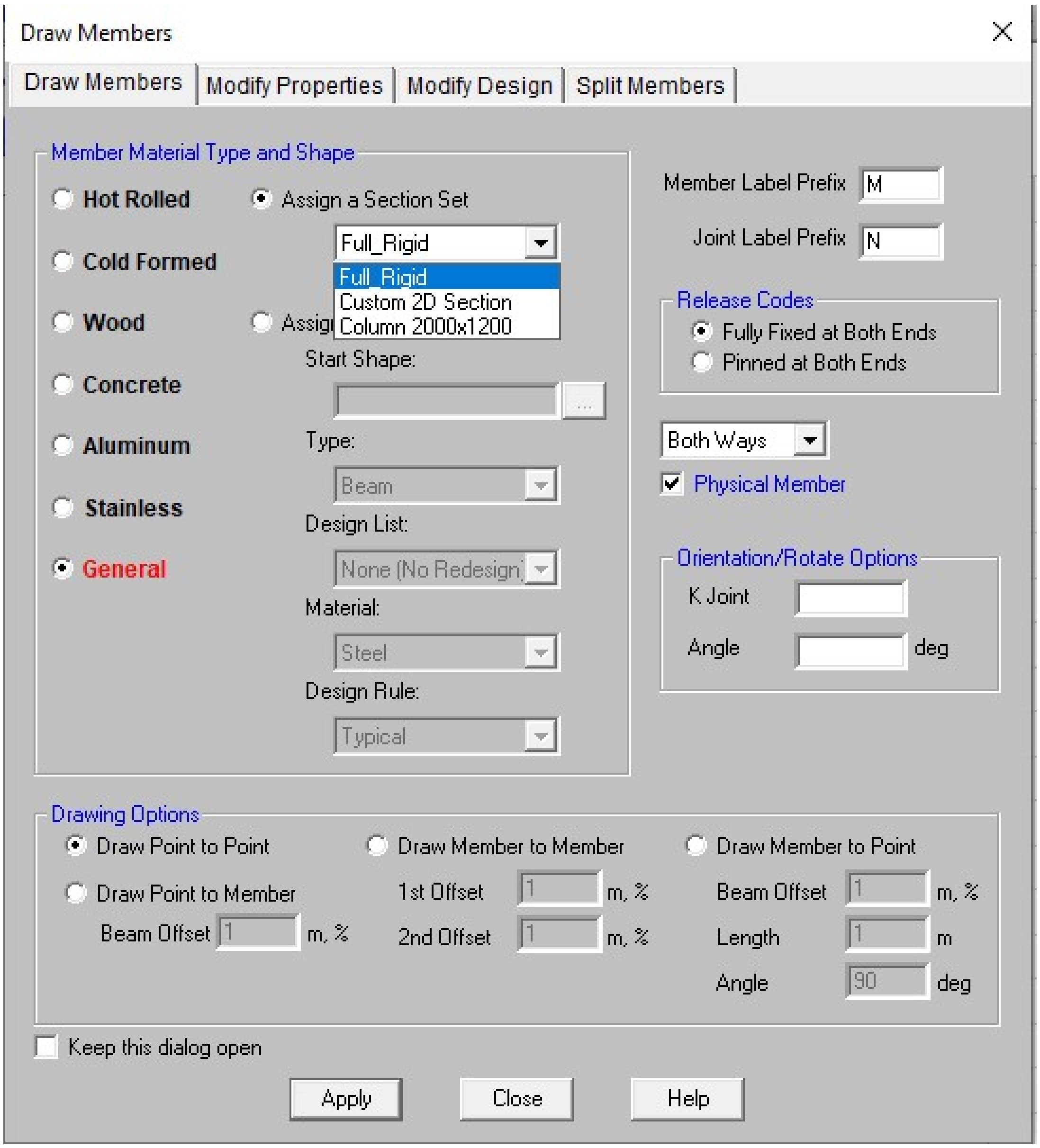


Figure 7.5.1.5- Member Drawings Menu [1]

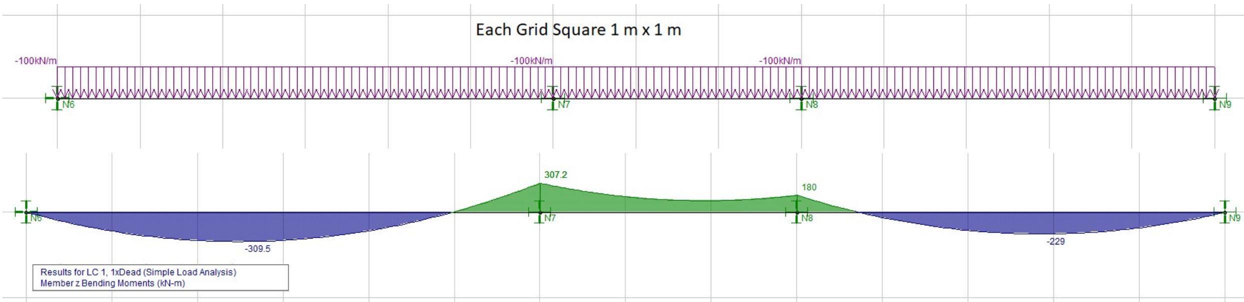


Figure 7.5.1.6- Moment Diagram of a Continuous Three Span Beam under Distributed Load [1]

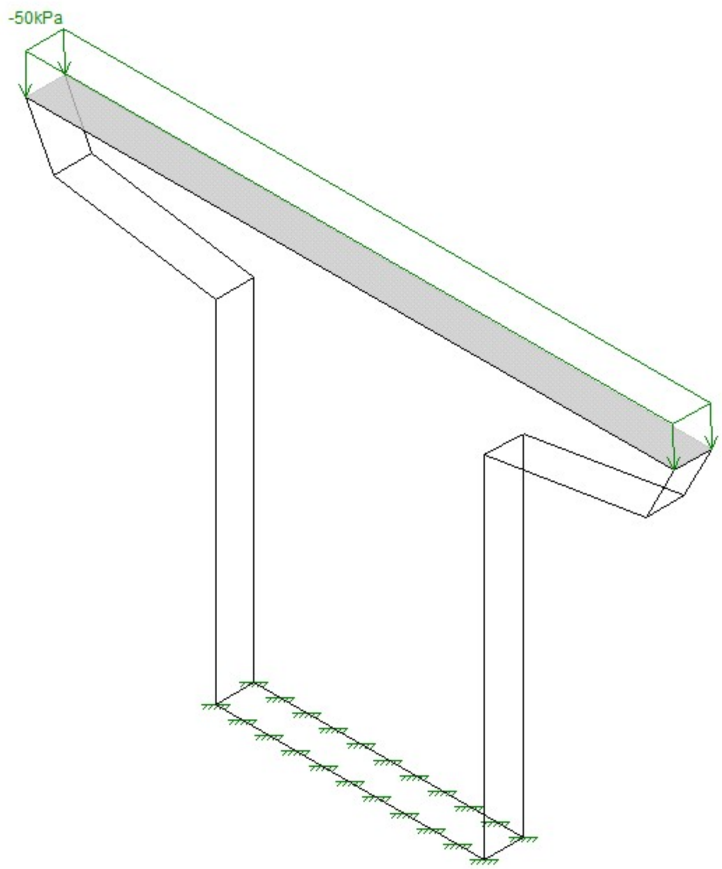


Figure 7.5.1.7- Sample Bridge Pier Drawing [1]

7.5.2 S-Frame

S-Frame is a Canadian structural design and analysis software. The software is widely used around the world. It was one of the softwares used in the foundation design of the world tallest structure Burj Khalifa [2]. S-Frame can be easily integrated with S-Concrete, S-Steel, S-Foundation, S-Pad, etc. It has a graphical user interface similar to other popular structural software (SAP2000). S-Frame allows any angle interactive rotation, however softwares like RISA 3D V17 doesn't have interactive rotation of 3D models. S-Frame can do code check for various codes and also nonlinear static and dynamic analysis.

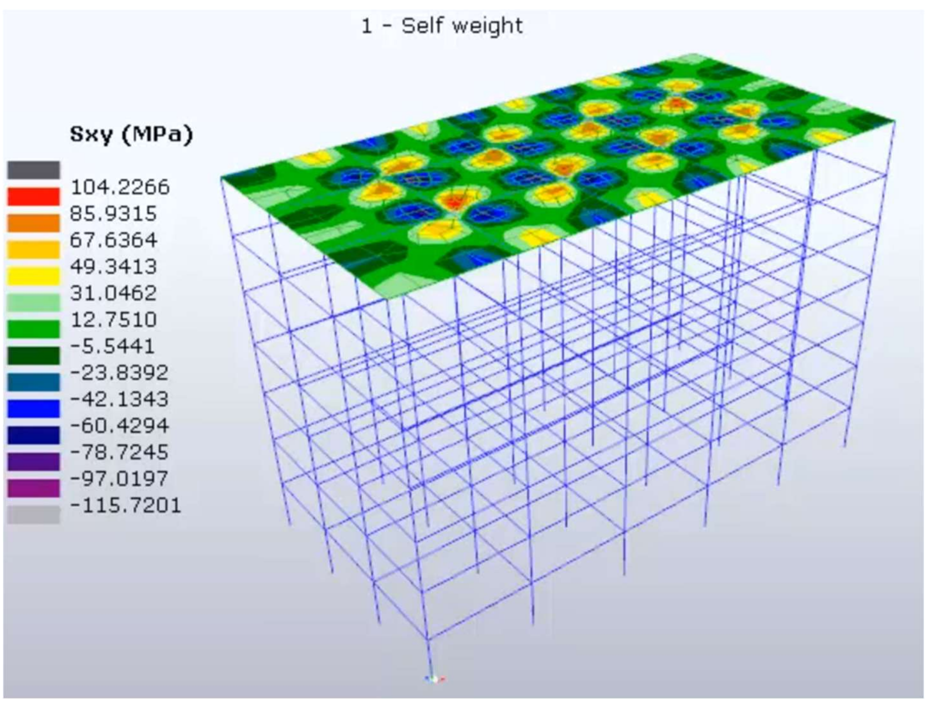


Figure 7.5.2.1- Self-Weight stress contours of a seventh story of a sample building in S-Frame [3]

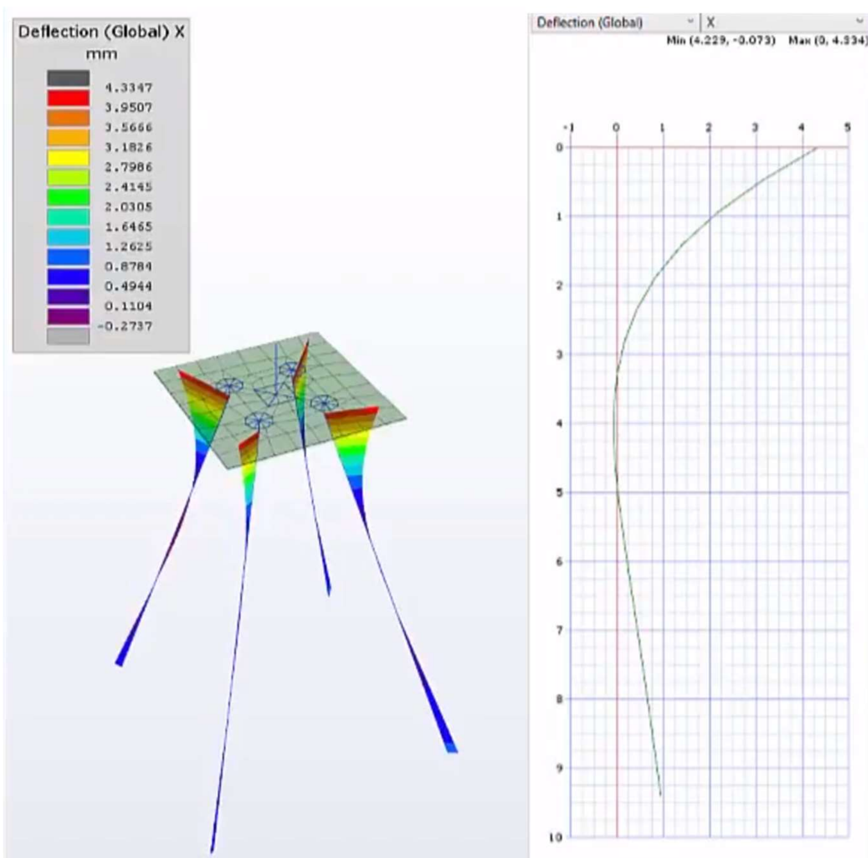


Figure 7.5.2.2- x-Deflections in a typical pile foundation in S-Foundation

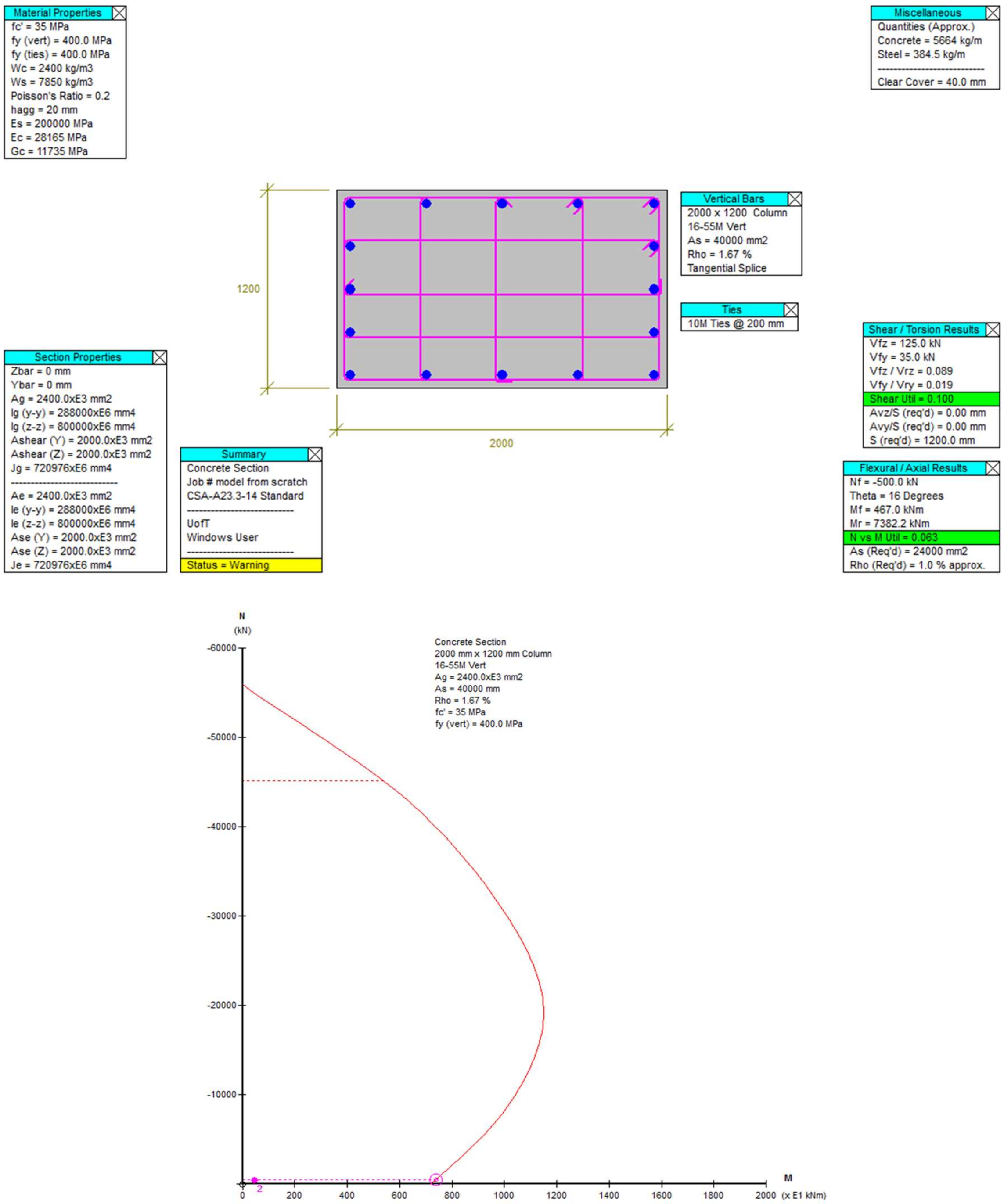


Figure 7.5.2.3- Interaction Diagram of a 2000x1200mm Column in S-Frame

7.5.3 CSI Bridge

CSI Bridge software is used to model, analyze and design bridge structures using computerized engineering tools. It is the most versatile and productive software program available in the market as it can easily design different bridge elements such as spans, bearings, abutments, piers and hinges. Bridge geometries, boundary conditions and different loading conditions can be easily determined by the engineers using this software. It also provides a single interface platform to model, analyze, design, schedule, load rating and reporting as shown in Figure 7.13 [4]. Figure 7.14 shows influence surfaces plotted against the load point along a traffic lane [4].

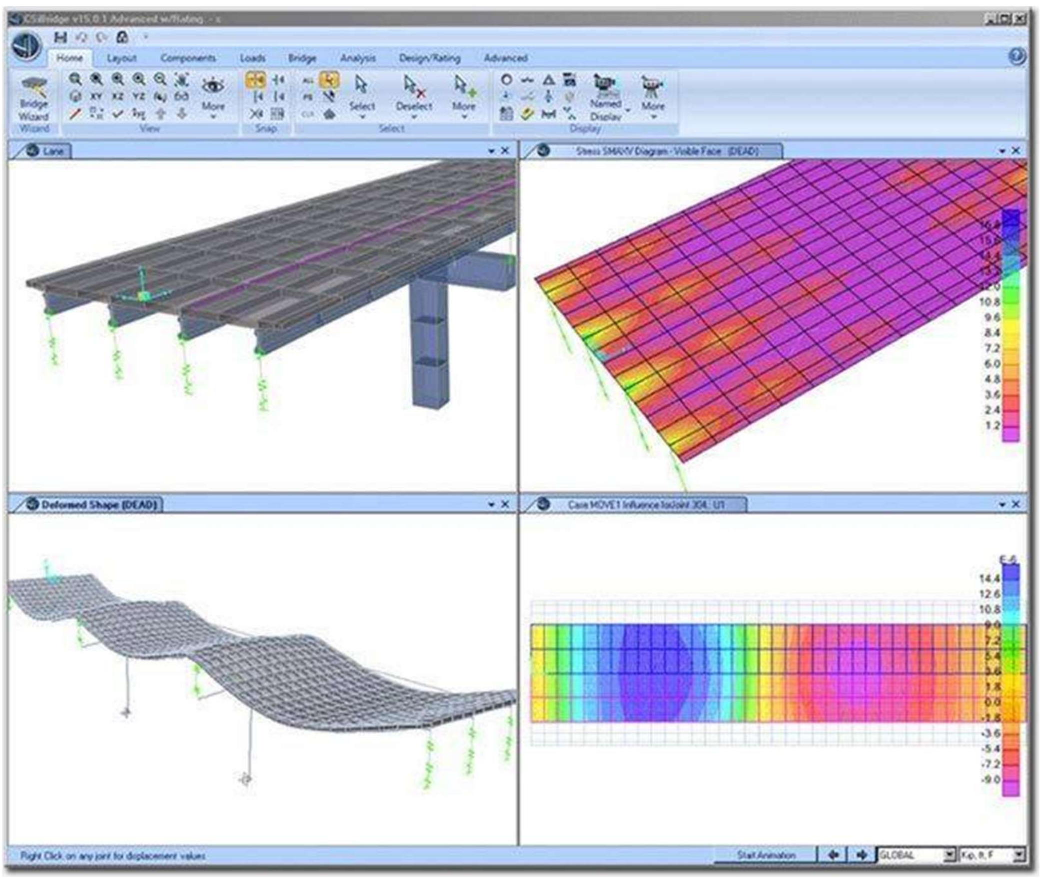


Figure 7.5.3.1- User Interface Platform in CSI Bridge [4]

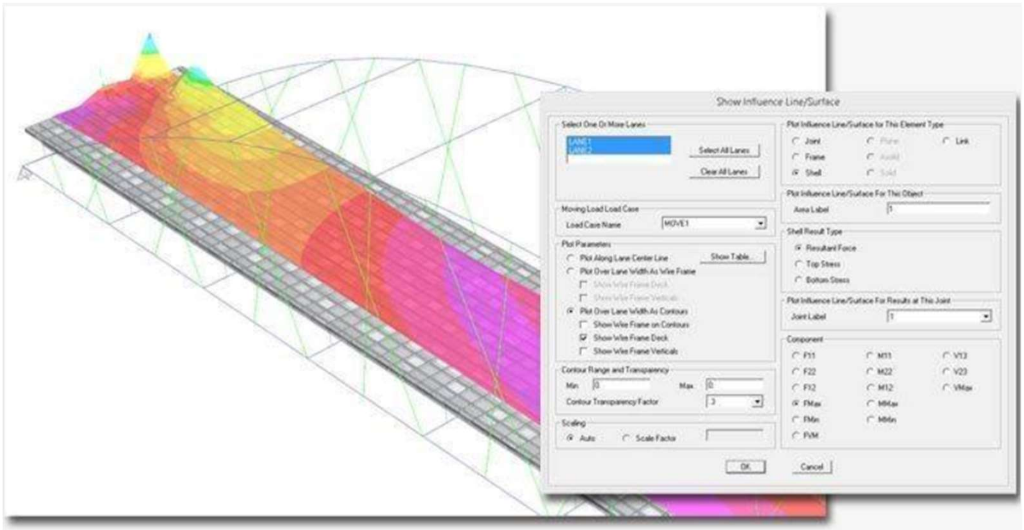


Figure 7.5.3.2- Influence Surface along a Traffic Lane

7.6 Conclusion

This chapter discussed the main principles and different approaches of structural analysis. The required inputs, equilibrium and compatibility equations and structural indeterminacy were reviewed in this chapter. The different analysis methods and analysis sequences were then summarized. Lastly, the different structural analysis softwares were investigated. Overall, the structural analysis helps to design a safe and serviceable structure.

7.7 References

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[2]. “Tall building foundations: design methods and applications”, 13 June-2016. [Online]. Available: https://link.springer.com/article/10.1007/s41062-016-0010-2

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[4]. Asif@csiamerica.com, “Superstructure,” *Features | Bridge Analysis, Design and Rating | CSiBridge*, 19-Apr-2014. [Online]. Available: https://www.csiamerica.com/products/csibridge/features. [Accessed: 06-Apr-2020].

Chapter 8 – Detailed Design

Table of Contents

8.1 Introduction

This chapter will emphasize on the detailed design methodology for designing girders together with reinforced concrete bridge deck for CSA S6-14 rev.17, AASHTO 2014-17 and CSA S6-66 design codes. Firstly, the different bridge design elements as well as material properties for each design code is set out. Secondly, the geometry, tendon profile, flexural and shear design is outlined for the design of the girder for each design code. Lastly, the design of reinforced concrete bridge deck is included with the crack control check and deflection check. The rest of the checks follow the same approach as the girder.

8.2 Bridge Design Elements

A Bridge can be subdivided into three main elements: The superstructure, the substructure and joints and bearings as shown in Figure 8.2.1.

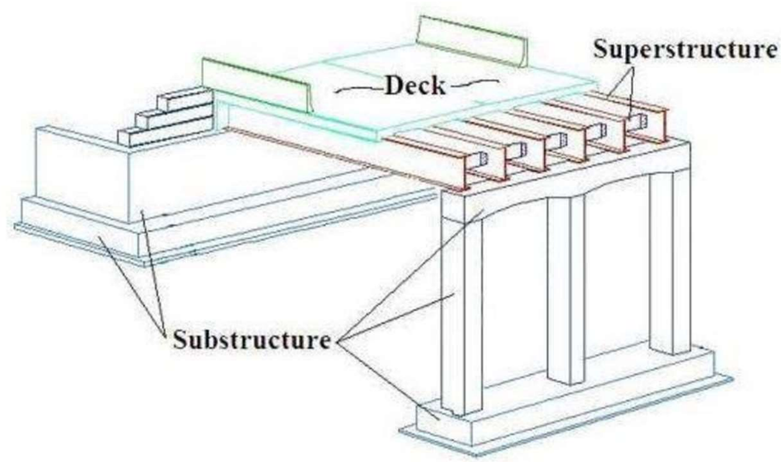


Figure 8.2.1 - Three Main Structural Elements of a Bridge [1]

8.2.1 Superstructure

The superstructure element of the bridge is composed of deck slab, girder, truss, etc. The superstructure element of the bridge that carries the bridge load and passes it over to the structure below which is called the substructure through joints and bearings.

8.2.2 Substructure

The substructure element of the bridge is composed of piers, abutment, wing walls, foundations, etc. The substructure element of the bridge provides support to the superstructure and transfers the load from the superstructure to the foundation of the bridge.

8.2.3 Joints and Bearings

The bearings are the intermediators between the superstructure and the substructure. They help in transferring the load from the superstructure to the substructure of the bridge. They allow only controlled movement longitudinally, transversely and rotationally and thereby reducing stresses within the structure. Figure 8.2.3.1 shows some common types of bearings. Pin bearing which allows for rotational movement and roller bearing allows both rotational and translational movement [2].

Figure 8.2.3.1 - Pin Bearing (Left) and Roller Bearing (Right) [2].

8.3 Materials

8.3.1 Concrete

8.3.1.1 Concrete - CSA-S6-14

The clause 8.4.1.2 states that the specified strength of concrete f'_c must be at least 30 MPa for non-prestressed members and 35 MPa for prestressed members [3]. If required, concrete with a strength greater than 85 MPa can be used once approved [3]. Based on clause 8.4.1.7, the modulus of elasticity of concrete E_c is:

$$(3000 \times \sqrt{f'c} + 6900)(\gamma_c/2300)^{1.5}$$

where

γ_c = mass density of concrete, kg/m^3
f′c = specified compressive strength of concrete, MPa

The rest of the concrete properties such as the creep, thermal expansion and shrinkage have been previously discussed in chapter 4 of this report.

8.3.1.2 Concrete – AASHTO

The clause 5.4.2.4 states that concrete with a unit weight between 0.090 and 0.155 kcf and specified compressive strength of 15 ksi (103 MPa), the modulus of elasticity of concrete, E_c shall be taken as [4]:

$$E_c = 33{,}000\, K_1\, w_c^{1.5} \sqrt{f'_C}$$

Where

K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test
W_c = unit weight of concrete (kcf)
f'_C = specified compressive strength of concrete (ksi)

The rest of the concrete properties such as the rupture strength and tensile strength have been previously discussed in chapter 4 of this report

8.3.1.3 Concrete - CSA S6-66

According to CSA S6-66, the ultimate strength of concrete must be between 3.0 ksi (20.7 MPa) to 5.0 ksi (34.5 MPa) [5].

8.3.2 Reinforcing Steel

8.3.2.1 Reinforcing Steel - CSA S6-14

According to clause 8.4.2.1 the specified yield strength of the reinforced steel bars must be between 300 MPa and 500 MPa and a modulus of elasticity must be at least 200,000 MPa [3]. Figure 8.3.2.1.1 shows the available bar sizes in Canada.

Deformed Bar Designation Numbers*, Nominal Dimensions†, Unit Masses, and Deformation Requirements

Bar Designation Number	Nominal Dimensions: Cross-Sectional Area mm²	Diameter mm	Perimeter mm	Mass (Weight) Per Unit Length kg/m	Deformation Requirements mm: Maximum Average Spacing	Minimum Average Height	Maximum Gap Chord of 12.5 Per Cent of Nominal Perimeter
10	100	11.3	35.5	0.785	7.9	0.45	4.4
15	200	16.0	50.1	1.570	11.2	0.72	6.3
20	300	19.5	61.3	2.355	13.6	0.98	7.7
25	500	25.2	79.2	3.925	17.6	1.26	9.9
30	700	29.9	93.9	5.495	20.9	1.48	11.7
35	1000	35.7	112.2	7.850	25.0	1.79	14.0
45	1500	43.7	137.3	11.775	30.6	2.20	17.2
55	2500	56.4	177.2	19.625	39.4	2.55	22.2

* *Bar numbers are based on the number of millimetres included in the nominal diameter of the bars.*
† *The nominal dimensions of a deformed bar are equivalent to those of a plain round bar having the same mass per metre as the deformed bar.*

Figure 8.3.2.1.1 - Standard Reinforcement Bars Available in Canada and their properties [6]

The spacing of reinforcing bars must be according to clause 8.14.2.1 as previously stated in chapter 4 of this report.

8.3.2.2 Reinforcing Steel – AASHTO

According to clause 5.5.3.2, the specified minimum yield strength, must be between 60.0 ksi (413 MPa) and 100 ksi (689 MPa) [4]. The minimum and maximum spacing of bars are shown in Figure 8.3.2.2.1 and Figure 8.3.2.2.2 respectively.

Minimum Spacing of Reinforcing Bars

Cast-in-Place Concrete

For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- 1.5 times the nominal diameter of the bars;
- 1.5 times the maximum size of the coarse aggregate; or
- 1.5 in.

Precast Concrete

For precast concrete manufactured under plant control conditions, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- the nominal diameter of the bars;
- 1.33 times the maximum size of the coarse aggregate; or
- 1.0 in.

Figure 8.3.2.2.1 - Minimum Spacing of Reinforcing Bars [4]

Maximum Spacing of Reinforcing Bars

Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not be greater than the lesser of the following:

- 1.5 times the thickness of the member; or
- 18.0 in.

Figure 8.3.2.2.2 - Maximum Spacing of Reinforcing Bars [4]

8.3.2.3 Reinforcing Steel - CSA S6-66

The minimum spacing requirement for parallel steel bars must be the lesser value of 1.5 x maximum size of coarse aggregate or 25 mm [5]. The spacing must be 25 mm if they are two or more parallel layers of steel bars [5].

8.3.3 Prestressed Tendons

8.3.3.1 Prestressing Tendons - CSA-S6-14

The clause 8.4.3.1 states that the tendons can either have high-tensile-strength, low-relaxation strand or high-strength bars and must also meet the requirements of CSA G279 [3]. Also, for pretensioned construction, tendons shall be of size designation 9, 13, or 15 strands [3]. Coated strands shall not be used unless approved.

According to clause 8.4.3.3, the modulus of elasticity of tendons E_p must be determined using Figure 8.3.3.1.1 if the stress-strain curves are not available [3].

(a) seven-wire high-strength strand:
   (i) Size 9, 13, or 15: 200 000 MPa; and
   (ii) Size 16: 195 000 MPa; and
(b) high-strength bar: 205 000 MPa.

Figure 8.3.3.1.1 - Modulus of Elasticity of Tendons According to Clause 8.4.3.3 [3]

According to clause 8.7.1, the stress limitations for tendons must have a minimum effective prestress of 0.45 x f_{pu}, where f_{pu} is the specified tensile strength of prestressing steel [3]. Table 8.3.3.1.2 shows the prestressing tendon stress limits.

Table 8.3.3.1.1- Prestressing tendon stress limits [3]

	Tendon type		
		High-strength bar	
	Low-relaxation strand	Smooth	Deformed
At jacking			
Pretensioning	$0.78f_{pu}$	—	—
Post-tensioning	$0.80f_{pu}$	$0.76f_{pu}$	$0.75f_{pu}$
At transfer			
Pretensioning	$0.74f_{pu}$	—	—
Post-tensioning			
At anchorage and couplers	$0.70f_{pu}$	$0.70f_{pu}$	$0.66f_{pu}$
Elsewhere	$0.74f_{pu}$	$0.70f_{pu}$	$0.66f_{pu}$

8.3.3.2 Prestressing Tendons – AASHTO

According to clause 5.4.4.2, the modulus of elasticity for prestressing steel, E_p must be 28500 ksi for strand and 30000 ksi for bar based on nominal cross-sectional area [4]. Table 8.3.3.2.1 shows the tensile and yield strength of strand and bar.

Table 8.3.3.2.1 - Prestressing tendon stress limits [3]
Properties of Prestressing Strand and Bar from AASHTO [4]

Material	Grade or Type	Diameter (in.)	Tensile Strength, f_{pu} (ksi)	Yield Strength, f_{py} (ksi)
Strand	250 ksi 270 ksi	1/4 to 0.6 3/8 to 0.6	250 270	85% of f_{pu}, except 90% of f_{pu} for low-relaxation strand
Bar	Type 1, Plain Type 2, Deformed	3/4 to 1-3/8 5/8 to 1-3/8	150 150	85% of f_{pu} 80% of f_{pu}

According to clause 5.9.3, the tendon stress either due to prestress or at the service limit state must not exceed the values shown in Table 8.3.3.2.2 [4].

Table 8.3.3.2.2 - Stress Limits for Prestressing Tendons from AASHTO [4]

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars	Tendon Type: Low Relaxation Strand	Tendon Type: Deformed High-Strength Bars
Pretensioning			
Immediately prior to transfer (f_{pbt})	$0.70f_{pu}$	$0.75f_{pu}$	—
At service limit state after all losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$
Post-Tensioning			
Prior to seating—short-term f_{pbt} may be allowed	$0.90f_{py}$	$0.90f_{py}$	$0.90f_{py}$
At anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.70f_{pu}$	$0.70f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70f_{pu}$	$0.74f_{pu}$	$0.70f_{pu}$
At service limit state after losses (f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$

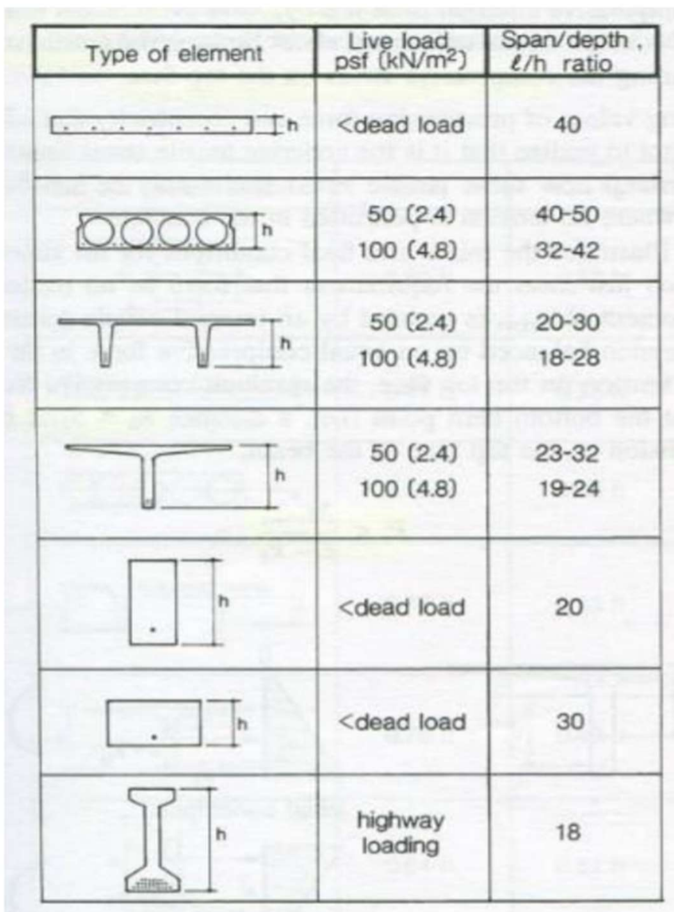
8.4 Design of Girder

Prior to designing of the girder, the structural engineer has to specify the geometry which includes the shape and dimension of the cross-section, material properties, tendon profile, and satisfy the prestressing requirement. Also, the structural engineer needs to be considerate towards construction feasibility throughout the design process. The mandatory checks must be carried out to check if the design is following its adequate strength, deflections and stress limits.

8.4.1 Geometry

The approximate span to depth ratio of the cross section can be determined using Table 8.4.1.1.

Table 8.4.1.1 - Typical Span to Depth Ratio for Simply Supported Prestressed Members [7]



Type of element	Live load psf (kN/m²)	Span/depth, ℓ/h ratio
h	<dead load	40
h	50 (2.4) 100 (4.8)	40-50 32-42
h	50 (2.4) 100 (4.8)	20-30 18-28
h	50 (2.4) 100 (4.8)	23-32 19-24
h	<dead load	20
h	<dead load	30
h	highway loading	18

8.4.2 Tendon Profile

The general rule in determining tendon profile is to drape down the tendon as low as the concrete cover allowance at midspan, until the tendon can resist the maximum positive moment. The tendons close to the supports have a rising parabolic shape in order to decrease the eccentricity induced by the prestressing. The basic principle to determine the tendon profile is to minimize the no. of tendons along the length of the member in order to optimize the member performance. The optimized tendon profile can be found easily by finite element modeling or even by using EXCEL.

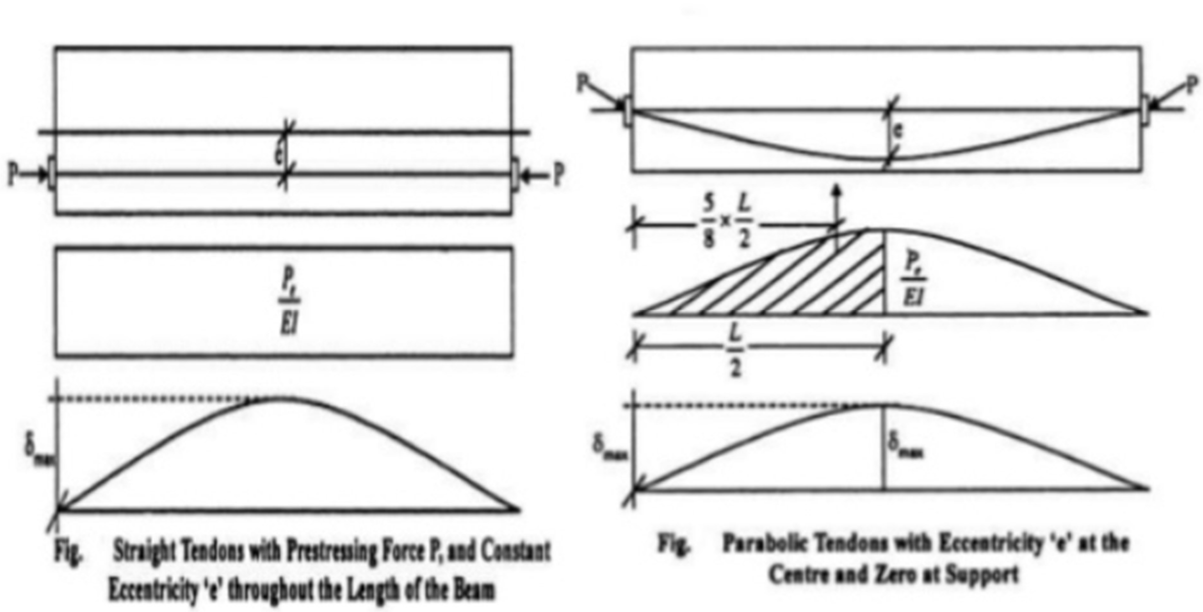


Figure 8.4.2.1 - Different tendon profiles and their impacts on deflection

8.4.3 Design for Flexure

The flexural design procedure shown below is the code procedure. All the codes allow for strain-compatibility analysis, a more accurate way of determining flexural capacity at any point (not just ultimate).

8.4.3.1 Design for Flexure - CSA-S6-14

The factored flexural resistance shall be calculated according to Clause 8.8.3 which is shown in Figure 8.4.3.1.1.

8.8.3 Assumptions for the ultimate limit states

In addition to the conditions of equilibrium and compatibility of strains, the calculations for the ultimate limit states shall be based on the material resistance factors specified in Clause 8.4.6 and the following shall apply to such calculations:

(a) Strain in the concrete shall be assumed to vary linearly over the depth of the section, except for deep beams, which shall satisfy the requirements of Clause 8.10.

(b) Strain changes in bonded reinforcement shall be assumed to be equal to strain changes in the surrounding concrete.

(c) The maximum usable strain at the extreme concrete compression fibre shall be assumed to be 0.0035 unless the concrete is confined and a higher value of strain can be justified. In the latter case, a strain compatibility analysis shall be used.

(d) Except for the strut-and-tie model of Clause 8.10, the stress in the reinforcement shall be taken as the value of the stress determined using strain compatibility based on a stress-strain curve representative of the steel reinforcement to be used, multiplied by ϕ_s or ϕ_p.

(e) The tensile strength of the concrete shall be neglected in the calculation of the factored flexural resistance.

(f) The relationship between concrete strain and the concrete compressive stress may be assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with the results of comprehensive tests. In this regard, an equivalent rectangular concrete stress distribution may be used, i.e., a concrete stress of $\alpha_1 \phi_c f_c'$ is uniformly distributed over an equivalent compression zone, bounded by the edges of the cross-section and a straight line parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain, where c is the shortest length between the fibre of maximum compressive strain and the neutral axis, $\alpha_1 = 0.85 - 0.0015 f_c' \geq 0.67$ and $\beta_1 = 0.97 - 0.0025 f_c' \geq 0.67$.

Figure 8.4.3.1.1 - Assumptions for the ultimate limit states to calculate factored flexural resistance [3]

According to Clause 8.8.4.2, the tensile strength of prestressing steel, f_{ps} with bonded components is determined using strain compatibility and stress strain curves of steel, except if c/d_p is less than or equal to 0.5, the following equation must be used:

$f_{ps} = f_{pu} (1 - k_p\, c/d_p)$

Where

f_{ps} = tensile strength of prestressing steel, MPa
f_{pu} = ultimate tensile strength of prestressing steel, MPa
k_p = 0.3 for for low-relaxation strands, 0.4 for smooth high-strength bars, and 0.5 for deformed high-strength bars
c = determined assuming a stress of fps in the tendons
d_p = distance from extreme compression fibre to centroid of prestressed reinforcement, mm

For unbonded components, $f_{ps} = f_{se}$. The minimum reinforcement shall be such that the factored flexural resistance, Mr , of the component is at least 1.20 times the cracking moment. A component is cracked when the moment at a section is such that a tensile stress of f_{cr}, as specified in Clause 8.4.1.8, is induced in the concrete. The maximum reinforcement provided shall be such that the factored flexural resistance, Mr, is developed with c/d not exceeding 0.5.

Other considerations in Clause 8.8.4.6 shown in Figure 8.4.3.1.2 must also be satisfied.

The stresses in the concrete shall not exceed the following:

(a) At transfer and during construction:
   (i) compression: $0.60f'_{ci}$;
   (ii) tension in components without reinforcing bars in the tension zone: $0.50f_{cri}$. Where the calculated tensile stress exceeds $0.50f_{cri}$, reinforcing bars in which the tensile stress is assumed to be 240 MPa shall be provided to resist the total tensile force in the concrete, calculated on the basis of an uncracked section; and
   (iii) tension at joints in segmental components:
      (1) without reinforcing bars passing through the joint in the tension zone: zero; and
      (2) with reinforcing bars passing through the joint in the tension zone: $0.50f_{cri}$.
      Where the calculated tensile stress is between zero and $0.50f_{cri}$, reinforcing bars in which the tensile stress is assumed to be 240 MPa shall be provided to resist the total tensile force in the concrete calculated on the basis of an uncracked section.

(b) At the serviceability limit states, if the tension in the concrete exceeds f_{cr}, Clause 8.12 shall apply. Tension shall not be permitted across the joints of segmental components unless bonded reinforcing bars pass through the joints in the tensile zone.

(c) In prestressed slabs with circular voids, the average compressive stress due to effective longitudinal prestress alone shall not exceed 6.5 MPa. In post-tensioned slabs with circular voids, the following shall apply:
   (i) an effective transverse prestress shall be provided to give a compressive stress of 4.5 MPa in the concrete above the longitudinal voids; and
   (ii) the thicknesses of the concrete above and below the voids shall not be less than 175 mm and 125 mm, respectively.

Figure 8.4.3.1.2 - Prestressed concrete stress limitations from CSA S6-14 rev.17 [3]

8.4.3.2 Design for Flexure – AASHTO

The factored flexural resistance M_r must be determined according to Clause 5.7.3.2 shown in Figure 8.4.3.2.1.

$$M_r = \phi M_n \qquad (5.7.3.2.1\text{-}1)$$

where:

M_n = nominal resistance (N-mm)

ϕ = resistance factor as specified in Article 5.5.4.2

5.7.3.2.2 Flanged Sections

For flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used and the tendons are bonded and where the compression flange depth is less than c, as determined in accordance with Eq. 5.7.3.1.1-3, the nominal flexural resistance may be taken as:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y \left(d_s - \frac{a}{2}\right) - A'_s f'_y \left(d'_s - \frac{a}{2}\right) + 0.85 f'_c (b - b_w)\beta_1 h_f \left(\frac{a}{2} - \frac{h_f}{2}\right) \qquad (5.7.3.2.2\text{-}1)$$

where:

A_{ps} = area of prestressing steel (mm^2)

f_{ps} = average stress in prestressing steel at nominal bending resistance specified in Eq. 5.7.3.1.1-1 (MPa)

d_p = distance from extreme compression fiber to the centroid of prestressing tendons (mm)

A_s = area of nonprestressed tension reinforcement (mm^2)

f_y = specified yield strength of reinforcing bars (MPa)

d_s = distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement (mm)

A'_s = area of compression reinforcement (mm^2)

f'_y = specified yield strength of compression reinforcement (MPa)

d'_s = distance from extreme compression fiber to the centroid of compression reinforcement (mm)

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (MPa)

b = width of the compression face of the member (mm)

b_w = web width or diameter of a circular section (mm)

β_1 = stress block factor specified in Article 5.7.2.2

h_f = compression flange depth of an I or T member (mm)

a = $c\beta_1$; depth of the equivalent stress block (mm)

Figure 8.4.3.2.1 - Factored Flexural Resistance from AASHTO [4]

The maximum and minimum reinforcement for a flexural component must be according to Clause 5.7.3 shown in Figure 8.4.3.2.2 and Figure 8.4.3.2.3 respectively.

$$\frac{c}{d_e} \leq 0.42 \qquad (5.7.3.3.1\text{-}1)$$

in which:

$$d_e = \frac{A_{ps}\, f_{ps}\, d_p + A_s\, f_y\, d_s}{A_{ps}\, f_{ps} + A_s\, f_y} \qquad (5.7.3.3.1\text{-}2)$$

where:

c = the distance from the extreme compression fiber to the neutral axis (mm)

d_e = the corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (mm)

Figure 8.4.3.2.2 - Maximum Reinforcement for Flexural Component from AASHTO [4]

- 1.2 times the cracking moment, M_{cr}, determined on the basis of elastic stress distribution and the modulus of rupture, f_r, of the concrete as specified in Article 5.4.2.6, where M_{cr} may be taken as:

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r \qquad (5.7.3.3.2\text{-}1)$$

where:

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (MPa)

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (N-mm)

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (mm^3)

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (mm^3)

Figure 8.4.3.2.3 - Minimum Reinforcement for Flexural Component from AASHTO [4]

8.4.3.3 Design for Flexure - CSA S6-66

The allowable flexural stresses in concrete must be determined according to Clause 9.3.2 shown in Figure 8.4.3.3.1.

9.3.2.1 Working Load Stresses

(a) Compression.. $0.40 f'_c$

(b) Tension (in precompressed tensile zone) pretensioned members.. $3\sqrt{f'_c}$, but not to exceed 250 psi.

post-tensioned members — nil

The above stresses are appropriate for members in which the creep and shrinkage of the concrete have taken place.

Figure 8.4.3.3.1 - Working Load Stresses

Flexure at ultimate load according to Clause 9.3.6.2 is categorized as either normally reinforced or over reinforced as shown in Figure 8.4.3.3.1.

9.3.6.2 Normally Reinforced Members

9.3.6.2.1 Normally reinforced members are defined as those in which either of the following is less than 0.3:

(a) $\frac{p \cdot f_{su}}{f'_c} - \frac{p' \cdot f'_y}{f'_c}$ for rectangular sections; or

(b) $\frac{A_{sr} \cdot f_{su}}{b' \cdot d \cdot f'_c} - \frac{p' f'_y}{f'_c}$ for flanged sections

where $A_{sr} = A_s - A_{sf}$ = steel area required to develop the ultimate compression strength of the web of a flanged section.

$A_{sf} = 0.85 f'_c (b-b') \frac{t}{f_{su}}$ = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

9.3.6.2.2 In the absence of a more accurate value substantiated by tests to the satisfaction of the authority having jurisdiction, and provided that f_{se} is not less than $0.50 f'_s$, f_{su} shall be computed from the following:

$$f_{su} = f'_s \left(1 - 0.5\, p \frac{f'_s}{f'_c}\right)$$

in pretensioned or bonded post-tensioned construction.

9.3.6.2.3 The ultimate flexural strength, M_u, for normally reinforced members shall be computed from the following:

(a) $M_u = A_s \cdot d \cdot f_{su} \left(1 - 0.6p \frac{f_{su}}{f'_c}\right)$ for rectangular sections; or

(b) $M_u = A_{sr} \cdot d \cdot f_{su} \left(1 - 0.6 \frac{A_{sr} \cdot f_{su}}{b' \cdot d \cdot f'_c}\right) + 0.85 f'_c (b-b')(d-0.5t)t$ for flanged sections

9.3.6.3 Over-Reinforced Members

9.3.6.3.1 Over-reinforced members* are defined as those in which either of the following is greater than 0.3:

(a) $\frac{p \cdot f_{su}}{f'_c} - \frac{p' \cdot f'_y}{f'_c}$ for rectangular sections; or

(b) $\frac{A_{sr} \cdot f_{su}}{b' \cdot d \cdot f'_c} - \frac{p' f'_y}{f'_c}$ for flanged sections.

**Clause 9.3.1.1 states that ultimate strength should be governed preferably by a ductile type of failure.*

9.3.6.3.2 The ultimate flexural strength shall be computed from the following:

(a) $M_u = 0.25 f'_c \cdot b \cdot d^2$ for rectangular sections; or

(b) $M_u = 0.25 f'_c b' \cdot d^2 + 0.85 f'_c (b-b')(d-0.5t)t$ for flanged sections.

Figure 8.4.3.3.1 - Normally and Over Reinforced Members from CSA S6-66 [5]

8.4.4 Design for Shear

8.4.4.1 Design for Shear - CSA S6-14

The criteria for shear resistance is that V_f is less than V_r.

The factored shear resistance, V_r is calculated according to Clause 8.9.3 as shown in Figure 8.4.4.1.1.

8.9.3.3 Factored shear resistance

The factored shear resistance, V_r, shall be calculated as $V_c + V_s + V_p$. However, $V_c + V_s$ shall not exceed $0.25\phi_c f_c' b_v d_v$.

8.9.3.4 Determination of V_c

V_c shall be calculated as $2.5\beta\,\phi_c\, f_{cr}\, b_v\, d_v$. However, f_{cr} shall not be greater than 3.2 MPa.

8.9.3.5 Determination of V_s

V_s shall be determined as follows:

(a) For components with transverse reinforcement perpendicular to the longitudinal axis, V_s shall be calculated as follows:

$$V_s = \frac{\phi_s f_y A_v d_v \cot\theta}{s}$$

(b) For components with transverse reinforcement inclined at an angle to the longitudinal axis and in the direction that will intersect diagonal cracks caused by the shear, V_s shall be calculated as follows:

$$V_s = \frac{\phi_s f_y A_v d_v \left(\cot\theta + \cot\alpha\right)\sin\alpha}{s}$$

Figure 8.4.4.1.1 - Factored Shear Resistance from CSA S6-14 [3]

The value of β and θ must be calculated according to Clause 8.9.3.7.

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

The angle of inclination, θ, shall be calculated as:

$$(29 + 7000\varepsilon_x)(0.88 + s_{ze}/2500)$$

$\boldsymbol{\varepsilon}_x$ shall be calculated according to clause 8.9.3.8 as shown below:

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

Evaluation of this equation shall be based on the following:

(a) V_f and M_f are positive quantities and M_f shall not be less than $(V_f - V_p)d_v$.

(b) N_f shall be taken as positive for tension and negative for compression. For rigid frames and rectangular culverts, the value of N_f used to determine ε_x may be taken as twice the compressive axial thrust calculated by elastic analysis.

(c) A_s and A_{ps} are the areas of reinforcing bars and prestressing tendons in the half-depth of the section containing the flexural tension zone.

(d) f_{po} may be taken as $0.7f_{pu}$ for bonded tendons outside the transfer length and f_{pe} for unbonded tendons.

(e) In calculating A_s, the area of bars that terminate less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.

(f) If the value of ε_x is negative, it shall be taken as zero or recalculated with the denominator replaced by $2(E_sA_s + E_pA_{ps} + E_cA_{ct})$. However, ε_x shall not be less than -0.20×10^{-3}.

(g) For sections closer than d_v to the face of the support, the value of ε_x calculated at d_v from the face of the support may be used in evaluating θ and β.

(h) If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in ε_x shall be taken into account. In lieu of more accurate calculations, the value calculated from the equation shall be doubled.

(i) θ and β may be determined from Clause 8.9.3.7 using a value of ε_x that is greater than that calculated from the equation in this Clause. However, ε_x shall not be greater than 3.0×10^{-3}.

Figure 8.4.4.1.2 - Determination of ε_x [3]

8.4.4.2 Design for Shear – AASHTO

The nominal shear resistance V_n, must be determined according to Clause 5.8.3.3 as shown in Figure 8.4.4.2.1.

The nominal shear resistance, V_n, shall be determined as the lesser of:

$$V_n = V_c + V_s + V_p \quad (5.8.3.3\text{-}1)$$

$$V_n = 0.25 f'_c b_v d_v + V_p \quad (5.8.3.3\text{-}2)$$

in which:

$$V_c = 0.083\beta\sqrt{f'_c}\, b_v\, d_v \quad (5.8.3.3\text{-}3)$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \quad (5.8.3.3\text{-}4)$$

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (mm)

d_v = effective shear depth as determined in Article 5.8.2.9 (mm)

s = spacing of stirrups (mm)

β = factor indicating ability of diagonally cracked concrete to transmit tension as specified in Article 5.8.3.4

θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (°)

α = angle of inclination of transverse reinforcement to longitudinal axis (°)

A_v = area of shear reinforcement within a distance s (mm²)

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear (N)

The shear resistance of a concrete member may be separated into a component, V_c, that relies on tensile stresses in the concrete, a component, V_s, that relies on tensile stresses in the transverse reinforcement, and a component, V_p, that is the vertical component of the prestressing force.

The expressions for V_c and V_s apply to both prestressed and nonprestressed sections, with the terms β and θ depending on the applied loading and the properties of the section.

The upper limit of V_n, given by Eq. 2, is intended to ensure that the concrete in the web of the beam will not crush prior to yield of the transverse reinforcement.

where $\alpha = 90°$, Eq. 4 reduces to:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \quad (C5.8.3.3\text{-}1)$$

The angle θ is, therefore, also taken as the angle between a strut and the longitudinal axis of a member.

Figure 8.4.4.2.1 - Nominal Shear Resistance from AASHTO [4]

8.4.4.3 Design for Shear - CSA S6-66

The shear resistance is calculated using Clause 8.7.4.

8.5 Design of Reinforced Concrete Bridge Deck

This section will provide a general procedure for flexure design for all three design codes. Next, crack control, design for shear, deflections will be summarized for all three design codes: CSA S6-14 rev.17, AASHTO LRFD 2014-17 and CSA S6-66.

8.5.1 General Procedure

The general procedure for flexure design is summarized below:

1. Compute factored loads both for deck and girder, including the self-weight.
2. Calculate the bending moment at various locations along the deck surface based on the loading combination that governs.
3. Approximate the slab thickness, h for the first iteration.
4. Compute effective depth, d using the equation below:

$$d = h - c - d_{bar}/2$$

5. Compute coefficient K_r using the equation below:

$$K_r = M_r\ x10^6\ kNm\ /\ b\ x\ d^2$$

6. Compute the ratio using table 2.1 from CSA A23.3 given calculated K_r and concrete strength.
7. Compute the area required using the equation below:

$$A_{sreqd.} = \rho_{reqd.}\ x\ b\ x\ d$$

8. Check the requirement than $A_{smin} \leq A_{sreqd.}$

$$A_{smin} = 0.002\ x\ b\ x\ h$$

9. Compute bar spacing using the equation below:

$$s = (A_b / A_S)\ x\ 1000$$

10. Check spacing requirements:

$$s \le min(1.5 \times h, 450)$$

11. Choose a reinforcement bar based on $A_{sreqd.}$

12. Compute M_r and ensure the criteria $M_r \ge M_f$

$$M_r = T_s \times (d - a/2)$$
$$T_s = \Phi \times A_s \times f_y$$
$$a = T_s / (\alpha \times f'_c \times b)$$

8.5.2 CSA S6-14

8.5.2.1 Design for Shear

For the shear design, both the critical section and shear resistance must be examined. In order to make sure the structure doesn't fail, V_r must be greater than or equal to V_f.

$$V_r = V_C + V_S$$

$$V_S = (\phi_S \times f_s \times A_V \times d_v \times \cot\theta) / (s)$$

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

β for slab thickness no more than 350mm shall be taken as 21°.

8.5.2.2 Crack Control

The shrinkage and temperature reinforcement must be determined according to Clause 8.12.6 which states, the minimum area of shrinkage and temperature reinforcement in each face and in each direction shall be 500 mm^2/m and the spacing of the bars shall not exceed 300 mm. The distribution of reinforcement must be determined according to Clause 8.12.4.

The calculation of crack width must be determined according to Clause 8.12.3.3 as shown below in Figure 8.5.2.2.1.

Crack width, *w*, shall be taken as $k_b \beta_c s_{rm} \varepsilon_{sm}$.

k_b shall be taken as 1.2 for components with epoxy-coated reinforcing steel and 1.0 for all other components.

When cracking is caused by load, β_c shall be taken as 1.7.

When cracking is caused by superimposed deformations, β_c shall be taken as 1.7 for cross-sections with a minimum dimension exceeding of 800 mm and 1.3 for cross-sections with a minimum dimension of

300 mm or less. Linear interpolation may be used to calculate β_c for cross-sections with a minimum dimension between these limits.

s_{rm} shall be calculated as follows (in millimetres):

$$s_{rm} = 50 + 0.25k_c \frac{d_b}{\rho_c}$$

k_c shall be taken as 0.5 for bending and 1.0 for pure tension.

ρ_c is the ratio A_s/A_{ct}, where A_{ct} is the effective tension area of the concrete cross-section and A_s is the area of reinforcement contained within A_{ct}. The depth of A_{ct} shall be taken as the lesser of

(a) 2.5 times the distance from the extreme tensile fibre of the cross-section to the centroid of tensile reinforcement; and

(b) one-third the distance from the neutral axis of the cross-section to the extreme tensile fibre.

ε_{sm} shall be calculated as follows:

$$\varepsilon_{sm} = \frac{f_s}{E_s}\left[1 - \left[\frac{f_w}{f_s}\right]^2\right]$$

where f_s is stress in reinforcement at the serviceability limit state and f_w is stress in reinforcement under the conditions causing initial cracking. Both f_s and f_w shall be calculated on the basis of a cracked section.

Figure 8.5.2.2.1 - Calculation of crack width from CSA S6-14 [3]

The maximum crack width must be determined according to table 8.6 as shown below in Figure 8.5.2.2.2.

Type of structural component	Type of exposure	Maximum crack width, mm
Non-prestressed	De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray; swamp; marsh; salt water; aggressive backfill	0.25
	Other environmental exposures	0.35
Prestressed	De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray; swamp; marsh; salt water; aggressive backfill	0.15
	Other environmental exposure	0.20

Figure 8.5.2.2.2 - Maximum Crack Width from CSA S6-14 [3]

8.5.2.3 Design for Deflection

The deflection is calculated using effective moment of inertia as shown below:

$$I_e = I_{cr} + (I_g - I_{cr})\ [M_{cr}/\ M_a] \leq I_g$$

For spans that are continuous, the effect moment of inertia is the average of critical positive and negative moment sections.

Total instantaneous and long term deflection is calculated using the equation shown below:

$$[\ 1 + (s/1+50p')]$$

where

s = factor for duration, 1 for three months, 1.2 for six months, 1.4 for twelve months and 2 for five years or more

ρ' = the value at midspan for simple and continuous spans at the support for the cantilevers

8.5.2 AASHTO

8.5.2.1 Design for Flexure

Section 9 in AASHTO is fully dedicated to the design of bridge decks. The depth of concrete deck should not be less than 175 mm as stated in Clause 9.7.1.1. The minimum cover must be determined according to Article 5.12.3 which is shown below in Figure 8.5.2.1.1.

SITUATION	COVER (mm)
Direct exposure to salt water	100
Cast against earth	75
Coastal	75
Exposure to deicing salts	60
Deck surfaces subject to tire stud or chain wear	60
Exterior other than above	50
Interior other than above	
• Up to No. 36 bar	40
• No. 43 and No. 57 bars	50
Bottom of cast-in-place slabs	
• Up to No. 36 bar	25
• No. 43 and No. 57 bars	50
Precast soffit form panels	20
Precast reinforced piles	
• Noncorrosive environments	50
• Corrosive environments	75
Precast prestressed piles	50
Cast-in-place piles	
• Noncorrosive environments	50
• Corrosive environments	
- General	75
- Protected	75
• Shells	50
• Auger-cast, tremie concrete, or slurry construction	75

Figure 8.5.2.1.1 - Cover for unprotected Main Reinforcing Steel [4]

8.5.2.2 Design for Shear

Regions Requiring Transverse Reinforcement

Except for slabs, footings, and culverts, transverse reinforcement shall be provided where:

$$V_u > 0.5\phi\left(V_c + V_p\right)$$

where:

V_u = factored shear force (kip)
V_c = nominal shear resistance of the concrete (kip)
V_p = component of prestressing force in the direction of the shear force
ϕ = resistance factor

Figure 8.5.2.2.1 - Regions requiring transverse reinforcement

Minimum Transverse Reinforcement

Where transverse reinforcement is required and nonprestressed reinforcement is used to satisfy that requirement, the area of steel shall satisfy:

$$A_v \geq 0.0316\ \lambda\sqrt{f'_c}\,\frac{b_v s}{f_y}$$

where:

A_v = area of transverse reinforcement within distance s (in.^{2})

b_v = width of web adjusted for the presence of ducts

s = spacing of transverse reinforcement (in.)
f_y = yield strength of transverse reinforcement (ksi) $\leq$ 100 ksi
λ = concrete density modification factor

Figure 8.5.2.2.2 - Minimum Transverse Requirement

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.125 f'_c$, then:

$$s_{max} = 0.8 d_v \leq 24.0 \text{ in.}$$

- If $v_u \geq 0.125 f'_c$, then:

$$s_{max} = 0.4 d_v \leq 12.0 \text{ in.}$$

where:

$$v_u = \frac{|V_u - \phi V_p|}{\phi b_v d_v}$$

Figure 8.5.2.2.3 - Maximum Transverse Requirement

8.5.2.3 Crack Control

The minimum crack control requirement must be determined using the strut and tie method. The horizontal A_h and vertical reinforcement A_v requirement are shown below:

$$A_h / (b_w \times s_h) \geq 0.003$$

$$A_v / (b_w \times s_v) \geq 0.003$$

where

b_w = web width
s_h s_v = vertical and horizontal spacing of crack control requirement

8.5.2.4 Deflection

The gross moment of inertia I_e is used to calculate the deflection as shown below:

$$I_e = [M_{cr}/ M_a]^3 I_g + [1 - (M_{cr}/ M_a)^3] \leq I_g$$

$$M_{cr} = f_r (I_g / y_t)$$

8.5.3 CSA - S6-66

8.5.3.1 Design for Shear

Shear force can be determined using the equation below:

$$v = V/(b \times d)$$

Where V is the shearing unit stress

The area required for stirrups is determined using the equation below:

$$A_v = V_s / (f_v \times d)$$

The required steel area of web reinforcement using the equation below:

$$\mathbf{A_v = V/(f_v \times \sin\alpha)}$$
$$\mathbf{Where\ V \leq 1.5\ BD\ \sqrt{f'c}}$$

The required steel area for a series of parallel bent bars must be determined using the equation below

$$A_v = V_s / (f_v \times d\ (\sin\alpha + \cos\alpha)$$

8.5.3.2 Design for Deflection

The elastic theory should be used in order to determine the initial deflection. The value of modulus of elasticity for concrete and steel are defined below:

$$E\ (concrete) = 6x10^4\ \sqrt{f'c}\ (psi)$$
$$E\ (steel) = 29{,}000{,}000\ (psi)$$

The long-term deflection is determined by using the short-term deflection based on the ratio between area of tension reinforcement and compression reinforcement.

8.6 Conclusion

This chapter summarizes the detailed design for the girder and the deck which includes design for flexure, design for shear, crack control and deflections. Also, in the beginning of the chapter, the concepts of superstructure and substructure is discussed. Finally, the material properties of concrete, reinforcing steel and prestressing tendons is also discussed.

8.7 References

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Chapter 9 – Durability Design

Table of Contents

9.1 Introduction

This chapter highlights the importance of durability in bridge design as it has an impact on the long-life rate of the structure, which is one of the most important characteristics of a bridge. Firstly, the material durability is discussed which includes the material composition of concrete, different concrete mixtures and the curing process. Secondly, the external effects the surroundings have on the bridge, both physical effects and chemical effects are mentioned. Thirdly, durability design components in different design codes are explained. Lastly, analysis of a case study to understand the significance of durability in bridge design is examined.

9.2 Material Durability

This section will discuss the material durability of the two main materials used in bridge construction: Concrete, and Reinforcing/Prestressing steel.

9.2.1 Concrete

The durability of concrete relies on the composition of concrete and different aggregates mixed together, cement to water ratio and different admixtures used.

9.2.1.1 Composition

Concrete is composed of three basic constituents: Water, aggregate and cement. Each constituent has its own function. The water reacts chemically with cement and also adds workability to the concrete. In general, water to cement ratio (w/c) is inversely proportional to the strength of concrete. The aggregates can be rock, sand or gravel depending on the requirement. The cement hardens overtime and binds the aggregate together. The amount and ratio of these constituents depends on the ultimate goal of designer and budget. Different types of admixtures are added at times to achieve certain goals.

9.2.1.1.1 Aggregate

Aggregate is the major portion of a concrete mix, roughly about 60%-80% of a typical concrete mix [1]. Therefore, the following factors must be considered when selecting the type of aggregate: Durability, efficiency, consistent concrete strength and workability.

There are classifications of aggregates based on grain size:

Fine Aggregate - 4.75mm or smaller [2]
Example: Sand, Silt and Clay
Improves Workability

Coarse Aggregate - above 4.75mm limit [2]
Example: Natural Stone, Gravel
Improves Strength

9.2.1.1.2 Cement

Portland Cement is the most common type of cement used in concrete structures. It normally comes in the form of powder and acts as a binding agent when combined with water and aggregates. There are five type of cements as shown below in Figure 9.2.1.1.2.1:

Type 1 - Normal portland cement. Type 1 is a general use cement.

Type 2 - Is used for structures in water or soil containing moderate amounts of sulfate, or when heat build-up is a concern.

Type 3 - High early strength. Used when high strength are desired at very early periods.

Type 4 - Low heat portland cement. Used where the amount and rate of heat generation must be kept to a minimum.

Type 5 - Sulfate resistant portland cement. Used where the water or soil is high in alkali.

Figure 9.2.1.1.2.1 - Types of Cement [3]

9.2.1.1.3 Water

Water is one of the key ingredients in a concrete mix, which when combined with cement forms a paste that binds the aggregate together. Water hardens the concrete through the process of hydration. The water used must be pure in order to prevent side reactions. The water helps react chemically with cement and also provides workability with the concrete. The water to cement ratio (w/c) is inversely proportional to strength of concrete.

9.2.1.1.4 Admixtures

Admixtures are added to concrete to achieve certain special properties. They could either be natural or manufactured chemicals. The most common types of admixtures are air entraining agents, water reducers, water reducing retarders and accelerators. Admixtures help increase durability, workability and strength of concrete.

9.2.1.2 Concrete Mixtures

The biggest and unpredictable factor in a concrete mixture is water. The amount of water required depends on air temperature, humidity, sunlight and the concrete mix [4]. The more water there is in a concrete mix, the weaker the cement tensile and adhesive strength gets.

9.2.1.3 Curing

Concrete curing is the process of maintaining adequate moisture in concrete within a proper temperature range in order to aid cement hydration at early ages. Hydration is a chemical reaction between cement and water that results in the formation of various chemicals contributing to setting and hardening [5]. The process of hydration is affected by initial concrete temperature, air temperature, dimensions of concrete and mix design [5]. Concrete curing is an integral part of both quality control and quality assurance of the concrete structure. Curing helps prevent concrete from drying, shrinking, cracking, and ultimately affecting the performance of the structure, particularly at the cover zone [5]. The concrete curing process should be done as soon as it is placed, and its monitoring should be done over the next seven days [5]. If the water evaporates before the concrete has reached its maximum strength, that implies that there is insufficient water in concrete to complete the hydration process and attain its compressive strength. Some of the techniques for curing concrete are shown in Table 9.2.1.3.

Table 9.2.1.3 - Concrete Curing Techniques [5]

	Technique
Maintain Moisture	Ponding and Immersion Spraying and Fogging Saturated Wet Coverings Left pin-place forms
Reducing Loss of Water	Covering concrete with impervious paper or plastic sheets Applying membrane-forming curing compounds
Accelerating Concrete Strength Gain	Live steam Heating Coils Electrical heated forms or pads Concrete Blankets

9.2.2 Reinforcing and Prestressing Steel

The reinforcing and prestressing steel must not be exposed to the environment. Corrosion of steel causes 90% of damage to concrete structures [6]. It is caused when materials such as CO_2 and salts penetrate and reach steel.

9.3 External Effects

In this section, both physical and chemical effects will be discussed that impact the durability of a concrete structure.

9.3.1 Physical Effect

Freeze and thaw, and abrasion are the two physical effects that are explored in this section. Both of them significantly affect the durability of concrete structure.

9.3.1.1 Freeze and Thaw

Freeze thaw cycles occur in concrete structures when the structure is exposed to temperature below freezing. The effect is devastating when the concrete is wet and in contact with deicing chemicals. The water enters the concrete structure when it gets wet. The water freezes within the structure as the temperature drops causing expansion. Due to lack of volume within the structure, the concrete cracks and distresses causing serious internal damage to the structure. The bond between the cement paste and aggregate weakens, rupturing the structure. Continuous freeze and thaw cycles deteriorates the concrete structure more and more throughout the winter. The solution to prevent freeze and thaw cycles is to use low water to cement ratio. Also, air entrainment helps resist concrete being exposed to the external environment. The result of freeze and thaw cycles are random cracking, surface scaling and joint deterioration due to D-cracking [7]. Figure 9.3.1.1.1 shows cracked concrete staircase due freeze and thaw cycles.

Figure 9.3.1.1.1 - Cracked Concrete Staircase due to Freeze and Thaw Cycles [7]

9.3.1.2 Abrasion

Surface abrasion is the mechanical deterioration of concrete structure when it comes in contact with external materials. This contact is through rubbing or friction with external materials. Surface abrasion can have significant impact on the concrete structure such as cracking, corrosion of reinforcing steel, and completely wear away concrete from the structural component [8]. In order to prevent surface abrasion, concrete which is smooth with higher strength, low water to cement ratio must be used and also adequate curing techniques must be applied. Figure 9.3.1.2.1 shows the inverse relationship between water to cement ratio and abrasion.

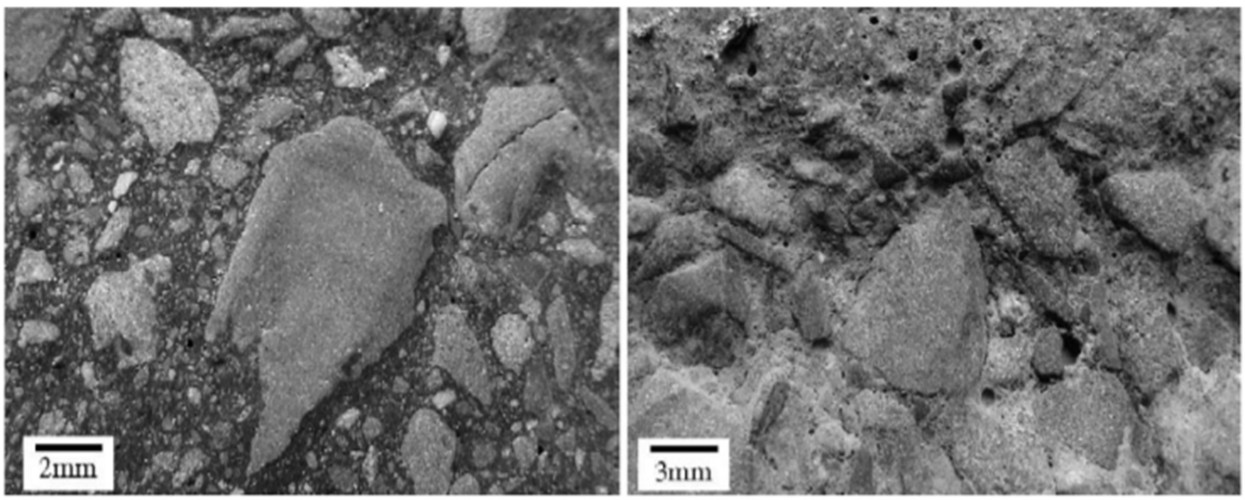

Figure 9.3.1.2.1 - Abraded surface of concretes with varying w/cm ratios - 0.28 (left) and 0.36 (right) [8]

9.3.2 Chemical Effects

This section discusses two types of chemical effects on concrete structure: Acid exposure and corrosion of reinforcing steel.

9.3.2.1 Acids

In this section, the impact of sulfate attacks and alkali silica reactions on concrete structure will be discussed.

Sulfate attack: It can affect the concrete structure either internally or externally [9]. It occurs when sulfate particles react with the concrete paste resulting in change of texture, composition and integrity of the concrete structure [9]. These sulfate attacks are highly expansive and progressive. Magnesium sulfate is the most vulnerable compared to other sulfate salts such as sodium, potassium or calcium sulfates [9].

Figure 9.3.2.1.1 - Sulfate attack on a Concrete Bridge [11]

Alkali Silica Reactions: commonly known as concrete cancer. It occurs when silica in aggregates react with alkali hydroxide in concrete forming a gel like substance which causes swelling that adsorbs water from the surrounding paste [10]. The expansive nature of gel causes closed joints and attendant spalled concrete [10].

Figure 9.3.2.1.2 - Alkali Silica Reaction on Concrete Jersey Barrier [10]

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9.3.2.2 Corrosion of Reinforcing Steel

The corrosion of both reinforcing and prestressed steel takes place when iron present in the steel reacts with oxygen to produce rust. It is an electrochemical process. The rust takes about 6 times the volume of the original material [12]. This change in volume creates tensile stresses in concrete causing cracking, delamination, and spalling of concrete structures [13].

CSA A23.1 specifies parameters for different classes of exposure. Different concrete compositions for different exposures which are shown below [14]:

- The highest minimum compressive strength
- The lowest maximum water to cement ratio
- The highest range in air content
- The most stringent cement type requirement of exposure conditions considered

Table 1
Definitions of C, F, N, A,S and R classes of exposure
(See Clauses 3, 4.1.1.1.1, 4.1.1.1.3, 4.1.1.5, 4.1.1.8.1, 4.1.2.3, 4.4.4.1.1.1, 4.4.4.1.1.2, 6.1.4, 6.6.7.5.1, 8.12.1, 9.1, L.3, and R.1, Tables 2, 3, and 17, and Annex L.)

C-XL	Structurally reinforced concrete exposed to chlorides or other severe environments with or without freezing and thawing conditions, with higher durability performance expectations than the C-1 classes.
C-1	Structurally reinforced concrete exposed to chlorides with or without freezing and thawing conditions. Examples: bridge decks, parking decks and ramps, portions of structures exposed to seawater located within the tidal and splash zones, concrete exposed to seawater spray, and salt water pools. For seawater or seawater-spray exposures the requirements for S-3 exposure also have to be met.
C-2	Non-structurally reinforced (i.e., plain) concrete exposed to chlorides and freezing and thawing. Examples: garage floors, porches, steps, pavements, sidewalks, curbs, and gutters.
C-3	Continuously submerged concrete exposed to chlorides, but not to freezing and thawing. Examples: underwater portions of structures exposed to seawater. For seawater or seawater-spray exposures the requirements for S-3 exposure also have to be met.
C-4	Non-structurally reinforced concrete exposed to chlorides, but not to freezing and thawing. Examples: underground parking slabs on grade.
F-1	Concrete exposed to freezing and thawing in a saturated condition, but not to chlorides. Examples: pool decks, patios, tennis courts, freshwater pools, and freshwater control structures.
F-2	Concrete in an unsaturated condition exposed to freezing and thawing, but not to chlorides. Examples: exterior walls and columns.
N	Concrete that when in service is neither exposed to chlorides nor to freezing and thawing nor to sulphates, either in a wet or dry environment. Examples: footings and interior slabs, walls, and columns.
N-CF	Interior concrete floors with a steel-trowel finish that are not exposed to chlorides, nor to sulphates either in a wet or dry environment. Examples: interior floors, surface covered applications (carpet, vinyl tile) and surface exposed applications (with or without floor hardener), ice-hockey rinks, freezer warehouse floors.
A-XL	Structurally reinforced concrete exposed to severe manure and/or silage gases, with or without freeze-thaw exposure. Concrete exposed to the vapour above municipal sewage or industrial effluent, where hydrogen sulphide gas might be generated, with higher durability performance expectations than A-1 class.
A-1	Structurally reinforced concrete exposed to severe manure and/or silage gases, with or without freeze-thaw exposure. Concrete exposed to the vapour above municipal sewage or industrial effluent, where hydrogen sulphide gas might be generated. Examples: reinforced beams, slabs, and columns over manure pits and silos, canals, and pig slats; and access holes, enclosed chambers, and pipes that are partially filled with effluents.
A-2	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure. Examples: reinforced walls in exterior manure tanks, silos and feed bunkers, and exterior slabs.
A-3	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure in a continuously submerged condition. Concrete continuously submerged in municipal or industrial effluents. Examples: interior gutter walls, beams, slabs, and columns; sewage pipes that are continuously full (e.g., forcemains); and submerged portions of sewage treatment structures.
A-4	Non-structurally reinforced concrete exposed to moderate manure and/or silage gases and liquids, without freeze-thaw exposure. Examples: interior slabs on grade.

(Continued)

Figure 9.3.2.2.1 - Classes of Exposure [14]

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Table 1 (Concluded)

S-1	Concrete subjected to very severe sulphate exposures (Tables 2 and 3).
S-2	Concrete subjected to severe sulphate exposure (Tables 2 and 3).
S-3	Concrete subjected to moderate sulphate exposure and to seawater or seawater spray (Tables 2 and 3).
R-1	Residential concrete for footings for walls, columns, fireplaces and chimneys.
R-2	Residential concrete for foundation walls, grade beams, piers, etc.
R-3	Residential concrete for interior slabs on ground not exposed to freezing and thawing or deicing salts.

Notes:

(1) *"C" classes pertain to chloride exposure.*
(2) *"F" classes pertain to freezing and thawing exposure without chlorides.*
(3) *"N" class is exposed to neither chlorides nor freezing and thawing.*
(4) *All classes of concrete exposed to sulphates shall comply with the minimum requirements of S class noted in Tables 2 and 3. In particular, Classes A-1 to A-4 and A-XL in municipal sewage elements could be subjected to sulphate exposure.*
(5) *No hydraulic cement concrete will be entirely resistant in severe acid exposures. The resistance of hydraulic cement concrete in such exposures is largely dependent on its resistance to penetration of fluids.*
(6) *Decision of exposure class should be based upon the service conditions of the structure or structural element, and not upon the conditions during construction.*

Table 2
Requirements for C, F, N, A, and S classes of exposure

(See Clauses 4.1.1.1.1, 4.1.1.1.3, 4.1.1.3, 4.1.1.4, 4.1.1.5, 4.1.1.6.2, 4.1.1.8.1, 4.1.1.10.1, 4.1.2.1, 4.3.1, 4.3.5.2.2, 4.3.7.2, 4.3.7.3, 7.4.1.1, 8.7.5.1, 8.12.1, 9.4, 9.5, L.1, L.3, and R.3 and Table 1.)

Class of exposure*	Maximum water-to-cementing materials ratio†	Minimum specified compressive strength (MPa) and age (d) at test†,***	Air content category as per Table 4	Curing type (see Table 19)			Chloride on penetrability requirements and age at test‡
				Normal concrete	HVSCM-1	HVSCM-2	
C-XL or A-XL	0.40	50 within 56 d	1 or 2§	3	3	3	< 1000 coulombs within 91 d
C-1 or A-1	0.40	35 within 56 d	1 or 2§	2	3	2	< 1500 coulombs within 91 d
C-2 or A-2	0.45§§	32 at 28 d	1	2	2	2	—
C-3 or A-3	0.50	30 at 28 d	2	1	2	2	—
C-4** or A-4	0.55	25 at 28 d	2	1	2	2	—
F-1	0.50	30 at 28 d	1	2	3	2	—
F-2 or R-1 or R-2	0.55	25 at 28 d	2††	1	2	2	—
N	As per the mix design for the strength required	For structural design	None	1	2	2	—
N-CF or R-3	0.55	25 at 28 d	None	1	2	2	—
S-1	0.40	35 within 56 d	1 or 2§	2	3	2	—
S-2	0.45†††	32 within 56 d	1 or 2§	2	3	2	—
S-3	0.50†††	30 within 56 d	1 or 2§	1	2	2	—

**See Table 1 for a description of classes of exposure.*
†The minimum specified compressive strength may be adjusted to reflect proven relationships between strength and the water-to-cementing materials ratio provided that freezing and thawing and de-icer scaling resistance have been demonstrated to be satisfactory. The water-to-cementing materials ratio shall not be exceeded for a given class of exposure.

(Continued)

Figure 9.3.2.2.2 - Classes of Exposure [14]

Table 3
Additional requirements for concrete subjected to sulphate attack*
(See Clauses 4.1.1.1.1, 4.1.1.6.2, 4.1.1.6.3, and L.3 and Tables 1, 7, 24, and 25.)

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§††	Performance requirements§,§§ Maximum expansion when tested using CSA A3004-C8 Procedure A at 23 °C, % At 6 months	At 12 months††	Maximum expansion when tested using CSA A3004-C8 Procedure B at 5 °C, % ††† At 18 months‡‡
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS** ,HSb, HSLb*** or HSe	0.05	0.10	0.10
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS**, HSb, HSLb*** or HSe	0.05	0.10	0.10
S-3	Moderate (including seawater exposure*)	0.10–0.20	150–1500	0.20–0.60	MS, MSb, MSe, MSLb***, LH, LHb, HS**, HSb, HSLb*** or HSe	0.10		0.10

**For sea water exposure, also see Clause 4.1.1.5.*
†In accordance with CSA A23.2-3B.
‡In accordance with CSA A23.2-2B.
§Where combinations of supplementary cementing materials and portland or blended hydraulic cements are to be used in the concrete mix design instead of the cementing materials listed, and provided they meet the performance requirements demonstrating equivalent performance against sulphate exposure, they shall be designated as MS equivalent (MSe) or HS equivalent (HSe) in the relevant sulphate exposures (see Clauses 4.1.1.6.2, 4.2.1.1, and 4.2.1.3, and 4.2.1.4).
***Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates, including seawater. See Clause 4.1.1.6.3.*
††The requirement for testing at 5 °C does not apply to MS, HS, MSb, HSb, and MSe and HSe combinations made without portland limestone cement.
‡‡ If the increase in expansion between 12 and 18 months exceeds 0.03%, the sulphate expansion at 24 months shall not exceed 0.10% in order for the cement to be deemed to have passed the sulphate resistance requirement.
§§For demonstrating equivalent performance, use the testing frequency in Table 1 of CSA A3004-A1 and see the applicable notes to Table A3 in A3001 with regard to re-establishing compliance if the composition of the cementing materials used to establish compliance changes.

Figure 9.3.2.2.3 - Requirements for concrete subjected to sulphate attack [14]

9.4 Durability Design - CSA S6-14

The deterioration mechanisms to be considered for concrete components shall include, but not be limited to, the following [17]:

(a) carbonation-induced corrosion without chloride,
(b) chloride-induced corrosion due to seawater,
(c) chloride-induced corrosion from sources other than seawater,
(d) freeze-thaw deterioration,
(e) alkali aggregate reaction,
(f) chemical attack; and
(g) abrasion

The maximum water to cementing materials ratio is shown below in Figure 9.4.1.

Maximum water to cementing materials ratio
(See Clause 8.11.2.1.1.)

Deterioration mechanism	Environmental exposure	Maximum ratio*†‡
Chloride-induced corrosion	Marine	
	Airborne salts	0.45
	Tidal and splash spray	0.45
	Submerged	0.40
	Other than marine	
	Wet, rarely dry	0.40
	Dry, rarely wet	0.40
	Cyclic, wet/dry	0.40
Freeze-thaw attack§	Unsaturated	0.45
	Saturated	0.40
Carbonation-induced corrosion without chloride	Wet, rarely dry	0.50
	Dry, rarely wet	0.50
	Cyclic, wet/dry	0.45

**Unless otherwise Approved.*
†Water to cementing materials ratio by mass. Cementing materials include Portland cement, silica fume, fly ash, and slag.
‡The ratio shall be independently verified on the submitted concrete mix design and concrete materials. Quality control and quality assurance measures shall be taken to ensure uniformity of concrete production so that water/cement limits are maintained throughout production. Such measures shall include measurements of slump, air content, unit weight, and strength.
§Air content shall be in accordance with CSA A23.1. The minimum air content shall be 5.5% for concrete in saturated conditions unless otherwise Approved.

Figure 9.4.1 - Maximum Water to Cementing Materials Ratio [17]

The concrete composition shall be such that the concrete satisfies all specified performance criteria [17];

(a) contains durable materials,
(b) can be placed, compacted, and cured to form a dense cover to the reinforcement,
(c) is free of harmful internal reactions, e.g., alkali-aggregate reactions,
(d) withstands the action of freezing and thawing, including the effects of de-icing salts (where applicable),
(e) withstands external exposures, e.g., weathering, gases, liquids, and soil; and
(f) withstands mechanical attacks, e.g., abrasion.

The methods used for mixing, placing, and compacting the fresh concrete shall be shown on the plans to ensure that [17]

(a) the constituents are distributed uniformly in the mixture,
(b) the concrete is well consolidated; and
(c) the reinforcement, pre tensioning strands, and post-tensioning ducts are not damaged by vibrating operations.

Corrosion protection for reinforcement, ducts, and metallic components [17]:

Unless otherwise approved, steel reinforcement, anchorages, and mechanical connections specified for use within 75 mm of a surface exposed to moisture containing de-icing chemicals shall have an approved protective coating, be protected by other approved methods of corrosion protection or prevention, or be of non-corrosive materials. Exposed inserts, fasteners, and plates shall be protected from corrosion by approved methods. Sheaths for internal post-tensioning ducts specified for use within 100 mm of a surface subject to moisture containing de-icing chemicals shall be made of non-corroding material or with an approved coating. The ends of pre tensioning strands shall be protected by approved methods when they are not encased in concrete.

Sulphate-resistant cements [17]:
Sulphate-resistant cement shall be specified for concrete in deep foundation units, footings, buried structures made of reinforced concrete, or other substructure components exposed to soils or water to an extent sufficient to cause a strong sulphate attack on concrete. Protection against sulphate attack shall be in accordance with CSA A23.1.

Alkali-reactive aggregates [17]:
Aggregates for concrete shall be tested for susceptibility to alkali aggregate reaction. The evaluation and use of aggregates susceptible to alkali aggregate reaction shall be in accordance with CSA A23.1 and CSA A23.2-27A.

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9.5 Durability Design - CSA A23.2

The CSA A23.1-14 states various durability requirements that are summarized below in this section.

Table 1
Definitions of C, F, N, A,S and R classes of exposure
(See Clauses 3, 4.1.1.1.1, 4.1.1.1.3, 4.1.1.5, 4.1.1.8.1, 4.1.2.3, 4.4.4.1.1.1, 4.4.4.1.1.2, 6.1.4, 6.6.7.5.1, 8.12.1, 9.1, L.3, and R.1, Tables 2, 3, and 17, and Annex L.)

C-XL	Structurally reinforced concrete exposed to chlorides or other severe environments with or without freezing and thawing conditions, with higher durability performance expectations than the C-1 classes.
C-1	Structurally reinforced concrete exposed to chlorides with or without freezing and thawing conditions. Examples: bridge decks, parking decks and ramps, portions of structures exposed to seawater located within the tidal and splash zones, concrete exposed to seawater spray, and salt water pools. For seawater or seawater-spray exposures the requirements for S-3 exposure also have to be met.
C-2	Non-structurally reinforced (i.e., plain) concrete exposed to chlorides and freezing and thawing. Examples: garage floors, porches, steps, pavements, sidewalks, curbs, and gutters.
C-3	Continuously submerged concrete exposed to chlorides, but not to freezing and thawing. Examples: underwater portions of structures exposed to seawater. For seawater or seawater-spray exposures the requirements for S-3 exposure also have to be met.
C-4	Non-structurally reinforced concrete exposed to chlorides, but not to freezing and thawing. Examples: underground parking slabs on grade.
F-1	Concrete exposed to freezing and thawing in a saturated condition, but not to chlorides. Examples: pool decks, patios, tennis courts, freshwater pools, and freshwater control structures.
F-2	Concrete in an unsaturated condition exposed to freezing and thawing, but not to chlorides. Examples: exterior walls and columns.
N	Concrete that when in service is neither exposed to chlorides nor to freezing and thawing nor to sulphates, either in a wet or dry environment. Examples: footings and interior slabs, walls, and columns.
N-CF	Interior concrete floors with a steel-trowel finish that are not exposed to chlorides, nor to sulphates either in a wet or dry environment. Examples: interior floors, surface covered applications (carpet, vinyl tile) and surface exposed applications (with or without floor hardener), ice-hockey rinks, freezer warehouse floors.
A-XL	Structurally reinforced concrete exposed to severe manure and/or silage gases, with or without freeze-thaw exposure. Concrete exposed to the vapour above municipal sewage or industrial effluent, where hydrogen sulphide gas might be generated, with higher durability performance expectations than A-1 class.
A-1	Structurally reinforced concrete exposed to severe manure and/or silage gases, with or without freeze-thaw exposure. Concrete exposed to the vapour above municipal sewage or industrial effluent, where hydrogen sulphide gas might be generated. Examples: reinforced beams, slabs, and columns over manure pits and silos, canals, and pig slats; and access holes, enclosed chambers, and pipes that are partially filled with effluents.
A-2	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure. Examples: reinforced walls in exterior manure tanks, silos and feed bunkers, and exterior slabs.
A-3	Structurally reinforced concrete exposed to moderate to severe manure and/or silage gases and liquids, with or without freeze-thaw exposure in a continuously submerged condition. Concrete continuously submerged in municipal or industrial effluents. Examples: interior gutter walls, beams, slabs, and columns; sewage pipes that are continuously full (e.g., forcemains); and submerged portions of sewage treatment structures.
A-4	Non-structurally reinforced concrete exposed to moderate manure and/or silage gases and liquids, without freeze-thaw exposure. Examples: interior slabs on grade.

(Continued)

Figure 9.5.1 - Different Classes of Exposure [14]

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Table 1 (Concluded)

S-1	Concrete subjected to very severe sulphate exposures (Tables 2 and 3).
S-2	Concrete subjected to severe sulphate exposure (Tables 2 and 3).
S-3	Concrete subjected to moderate sulphate exposure and to seawater or seawater spray (Tables 2 and 3).
R-1	Residential concrete for footings for walls, columns, fireplaces and chimneys.
R-2	Residential concrete for foundation walls, grade beams, piers, etc.
R-3	Residential concrete for interior slabs on ground not exposed to freezing and thawing or deicing salts.

Notes:

(1) *"C" classes pertain to chloride exposure.*

(2) *"F" classes pertain to freezing and thawing exposure without chlorides.*

(3) *"N" class is exposed to neither chlorides nor freezing and thawing.*

(4) *All classes of concrete exposed to sulphates shall comply with the minimum requirements of S class noted in Tables 2 and 3. In particular, Classes A-1 to A-4 and A-XL in municipal sewage elements could be subjected to sulphate exposure.*

(5) *No hydraulic cement concrete will be entirely resistant in severe acid exposures. The resistance of hydraulic cement concrete in such exposures is largely dependent on its resistance to penetration of fluids.*

(6) *Decision of exposure class should be based upon the service conditions of the structure or structural element, and not upon the conditions during construction.*

Table 2
Requirements for C, F, N, A, and S classes of exposure

(See Clauses 4.1.1.1.1, 4.1.1.1.3, 4.1.1.3, 4.1.1.4, 4.1.1.5, 4.1.1.6.2, 4.1.1.8.1, 4.1.1.10.1, 4.1.2.1, 4.3.1, 4.3.5.2.2, 4.3.7.2, 4.3.7.3, 7.4.1.1, 8.7.5.1, 8.12.1, 9.4, 9.5, L.1, L.3, and R.3 and Table 1.)

Class of exposure*	Maximum water-to-cementing materials ratio†	Minimum specified compressive strength (MPa) and age (d) at test†,***	Air content category as per Table 4	Curing type (see Table 19)			Chloride on penetrability requirements and age at test‡
				Normal concrete	HVSCM-1	HVSCM-2	
C-XL or A-XL	0.40	50 within 56 d	1 or 2§	3	3	3	< 1000 coulombs within 91 d
C-1 or A-1	0.40	35 within 56 d	1 or 2§	2	3	2	< 1500 coulombs within 91 d
C-2 or A-2	0.45§§	32 at 28 d	1	2	2	2	—
C-3 or A-3	0.50	30 at 28 d	2	1	2	2	—
C-4** or A-4	0.55	25 at 28 d	2	1	2	2	—
F-1	0.50	30 at 28 d	1	2	3	2	—
F-2 or R-1 or R-2	0.55	25 at 28 d	2††	1	2	2	—
N	As per the mix design for the strength required	For structural design	None	1	2	2	—
N-CF or R-3	0.55	25 at 28 d	None	1	2	2	—
S-1	0.40	35 within 56 d	1 or 2§	2	3	2	—
S-2	0.45†††	32 within 56 d	1 or 2§	2	3	2	—
S-3	0.50†††	30 within 56 d	1 or 2§	1	2	2	—

**See Table 1 for a description of classes of exposure.*

†The minimum specified compressive strength may be adjusted to reflect proven relationships between strength and the water-to-cementing materials ratio provided that freezing and thawing and de-icer scaling resistance have been demonstrated to be satisfactory. The water-to-cementing materials ratio shall not be exceeded for a given class of exposure.

(Continued)

Figure 9.5.2 - Additional Requirements for Classes of Exposure [14]

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Table 3
Additional requirements for concrete subjected to sulphate attack*
(See Clauses 4.1.1.1.1, 4.1.1.6.2, 4.1.1.6.3, and L.3 and Tables 1, 7, 24, and 25.)

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§††	Performance requirements§,§§ Maximum expansion when tested using CSA A3004-C8 Procedure A at 23 °C, % At 6 months	At 12 months††	Maximum expansion when tested using CSA A3004-C8 Procedure B at 5 °C, % ††† At 18 months‡‡
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS** ,HSb, HSLb*** or HSe	0.05	0.10	0.10
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS**, HSb, HSLb*** or HSe	0.05	0.10	0.10
S-3	Moderate (including seawater exposure*)	0.10–0.20	150–1500	0.20–0.60	MS, MSb, MSe, MSLb***, LH, LHb, HS**, HSb, HSLb*** or HSe	0.10		0.10

*For sea water exposure, also see Clause 4.1.1.5.
†In accordance with CSA A23.2-3B.
‡In accordance with CSA A23.2-2B.
§Where combinations of supplementary cementing materials and portland or blended hydraulic cements are to be used in the concrete mix design instead of the cementing materials listed, and provided they meet the performance requirements demonstrating equivalent performance against sulphate exposure, they shall be designated as MS equivalent (MSe) or HS equivalent (HSe) in the relevant sulphate exposures (see Clauses 4.1.1.6.2, 4.2.1.1, and 4.2.1.3, and 4.2.1.4).
**Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates, including seawater. See Clause 4.1.1.6.3.
††The requirement for testing at 5 °C does not apply to MS, HS, MSb, HSb, and MSe and HSe combinations made without portland limestone cement.
‡‡ If the increase in expansion between 12 and 18 months exceeds 0.03%, the sulphate expansion at 24 months shall not exceed 0.10% in order for the cement to be deemed to have passed the sulphate resistance requirement.
§§For demonstrating equivalent performance, use the testing frequency in Table 1 of CSA A3004-A1 and see the applicable notes to Table A3 in A3001 with regard to re-establishing compliance if the composition of the cementing materials used to establish compliance changes.

Figure 9.5.3 - Additional Requirement for Concrete Subjected to Sulphate Attack [14]

9.6 Durability Design – AASHTO

Section 5.12 in AASHTO LRFD 2014-17, lists all the durability design details for concrete structures.
The portion of the concrete section which has durability design issues should be identified and protective action must be taken against it. Some of these actions are listed below [18]:

- Air-entrainment of the concrete
- Epoxy-coating or galvanizing the reinforcement
- Adding special concrete additives
- Applying special curing procedures

The protective measures for durability shall satisfy the requirements specified in Article 2.5.2.1. Design considerations for durability include concrete quality, protective coatings, minimum cover, distribution and size of reinforcement, details, and crack widths. The principal aim of these specifications, with regard to durability is the prevention of corrosion of the reinforcing steel.

The critical factors contributing to the durability of concrete structures are [18]:

- Adequate cover over reinforcement
- Non-reactive aggregate-cement combinations
- Thorough consolidation of concrete
- Adequate cement content
- Low W/C ratio
- Thorough curing, preferably with water

The use of air-entrainment is generally recommended when 20 or more cycles of freezing and thawing per year are expected at the location and exposure.

Situation	Cover (in.)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above	
• Up to No. 11 bar	1.5
• No. 14 and No. 18 bars	2.0
Bottom of cast-in-place slabs	
• Up to No. 11 bar	1.0
• No. 14 and No. 18 bars	2.0
Precast soffit form panels	0.8
Precast reinforced piles	
• Noncorrosive environments	2.0
• Corrosive environments	3.0
Precast prestressed piles	2.0
Cast-in-place piles	
• Noncorrosive environments	2.0
• Corrosive environments	
○ General	3.0
○ Protected	3.0
• Shells	2.0
• Auger-cast, tremie concrete, or slurry construction	3.0
Precast concrete box culverts	
• Top slabs used as a driving surface	2.5
• Top slabs with less than 2 ft of fill not used as a driving surface	2.0
• All other members	1.0

Figure 9.6.1 - Cover for Unprotected Main Reinforcing Steel (in.) [18]

9.7 Case Study of Confederation Bridge

The Confederation Bridge is the world's longest bridge over ice-covered water [15]. The bridge was designed to last a span of 100 years which is two times longer than the average lifespan for bridges [15]. It is a box girder bridge connecting Prince Edward Island to New Brunswick. The total length of the bridge is 12.9 km and was opened in 1997 [15]. It was an $840 million dollar project [16]. Figure 9.7.1 below shows Confederation Bridge over a frozen Northumberland Strait.

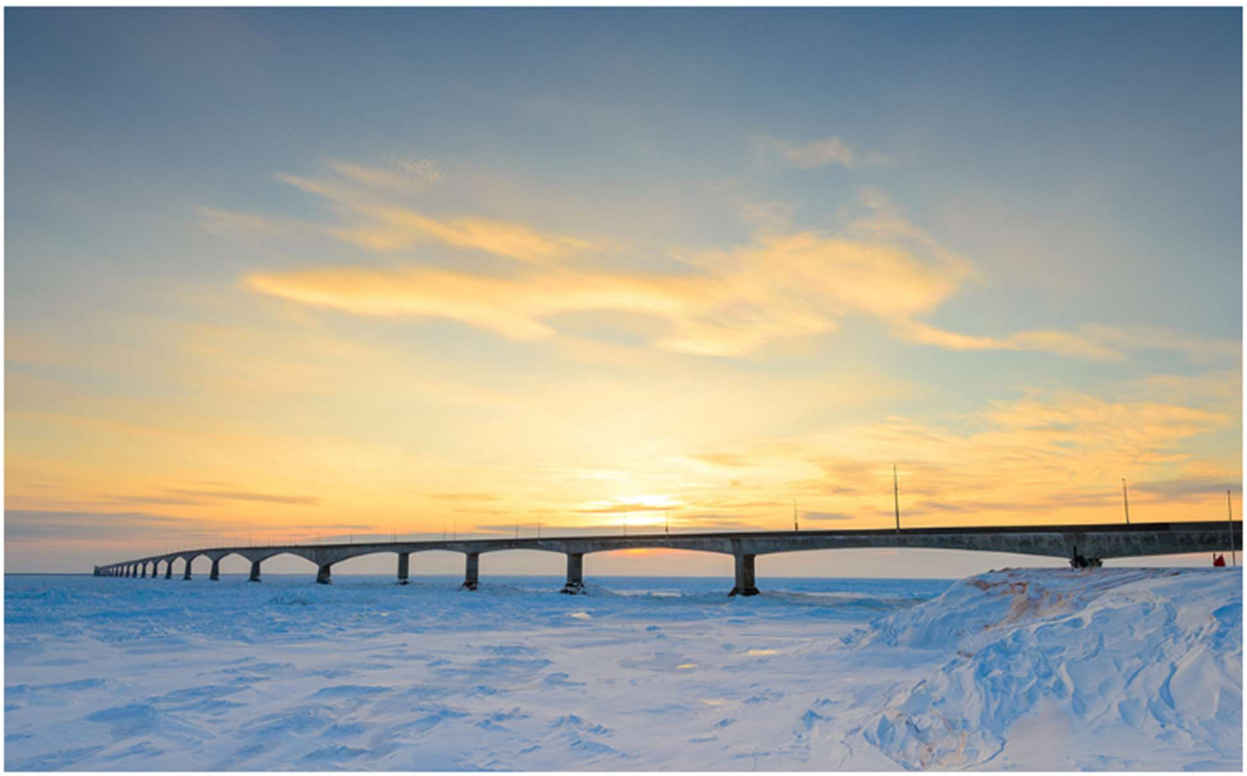

Figure 9.7.1 - Confederation Bridge over a frozen Northumberland Strait [16]

The construction of the bridge was a giant concrete puzzle. Workers fabricated and connected 175 major structural pieces including pier bases, and main girders. Special ice-shields were designed and installed to protect the support piers from the pack ice that flows through the Strait every winter. Each of the pieces, some weighing more than 7,500 tons, were transported from the fabrication yard by a 102 m high floating crane. Newly developed GPS systems allowed engineers to place the components on the ocean floor with an accuracy of 2 cm. Figure 10 shows the elevation view of the Confederation Bridge.

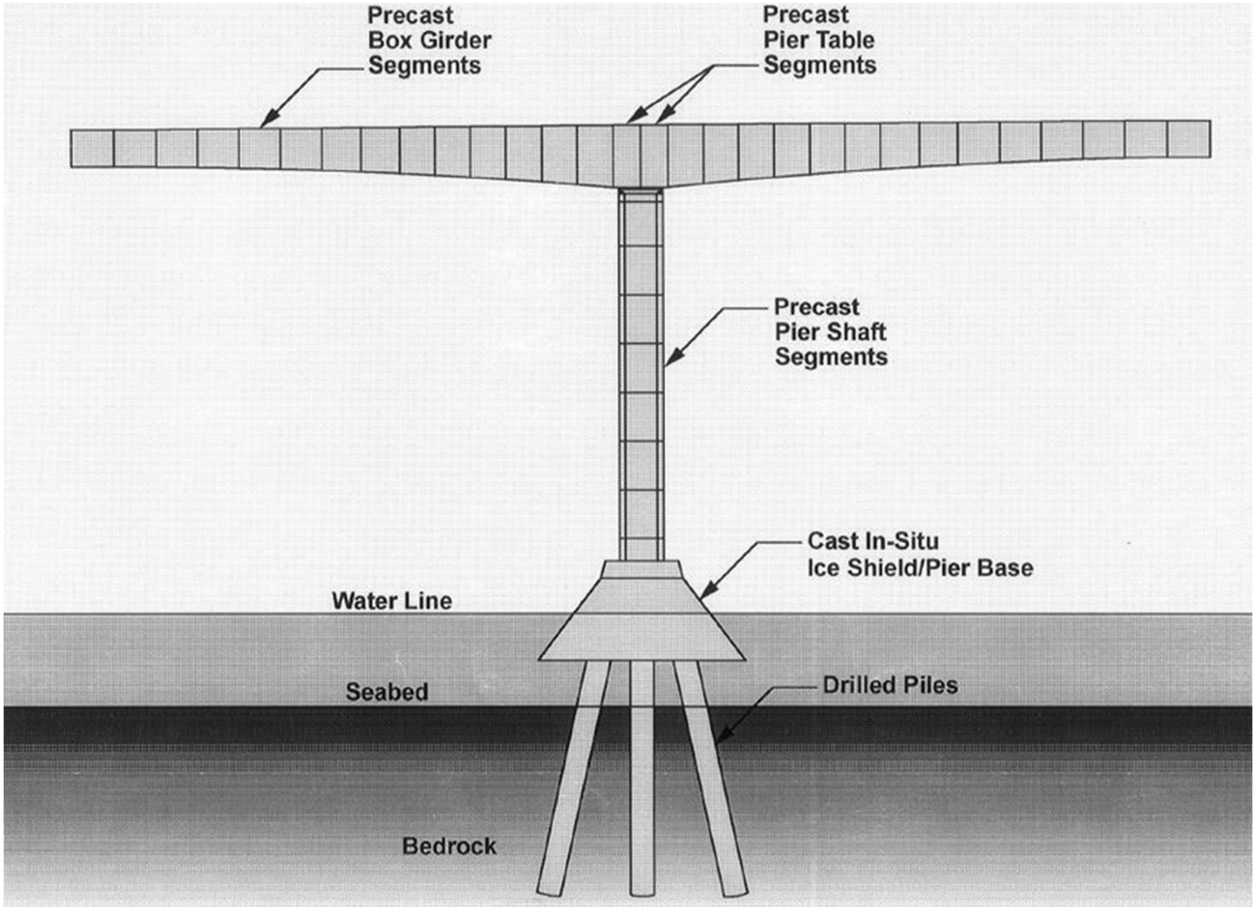

Figure 9.7.2 - Typical Elevation View of Confederation Bridge [19]

9.8 Conclusion

This chapter summarizes the durability design regulations that a structural engineer should follow to design a structure that can stay strong throughout its life span without a lot of maintenance. Firstly, we examined the durability design characteristic of concrete followed by physical and chemical effects on concrete structures. Three different codes were discussed that engineers should stick to when designing a bridge with a life span of 100 years. In the end, a case study was presented that showed how important is the durability factor when constructing a bridge as it not only adds to economic problems but also can cause devastating safety issues.

9.9 References

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UNIVERSITY OF TORONTO

Chapter 10 - Construction Issues

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K & B

10.1 Introduction

Construction site and real-world construction is different in many ways than the office design. The design engineer must consider these differences and construction issues that may arise from an impractical design. The consideration of constructability during the design process will prevent or reduce delays, reduce the risk of unwanted revisions and loss of money, and also make the construction site safer.

This chapter will discuss the possible construction issues that may be prevented if the necessary precaution is taken beforehand. Some of the issues discussed in the sections below are safety, site preparation, planning and schedule, quality assurance and control and formwork related issues. The construction process must be reviewed several times at the initial stages of construction in order to avoid any problems throughout the project in terms of many aspects like safety and quality. This will help reduce delays, injuries and repetitive errors in the project lifespan. The design engineer also needs to consider the constructability of the structure while designing to ensure a safe, practical design.

10.2 Safety Requirements

Construction safety is the most crucial factor in any given construction site and should be given top priority amongst all other factors with zero tolerance policy. Construction sites are hazardous land-based jobs with common activities such as excavation, use of heavy machinery, noise and dust pollution, working at heights and using power tools. The workplace injuries on construction sites in Canada are increasing at an alarming rate. In 2015, about 26000 workers were injured, and 186 workers were fatally injured while at work on a construction site [1]. Some of the possible construction safety hazards are falls (from heights), trench collapse, scaffold collapse, electric shock and arc flash/arc blast, failure to use proper personal protective equipment, and repetitive motion injuries [2]. The existence of these hazards made governments create strict regulations to prevent workplace accidents and protect workers with enough insurance coverage [2].

10.2.1 Occupational Health and Safety Act

The Occupational Health and Safety Act, R.S.O. 1990, c.O.1 (OHSA) provides detailed information on the rules and regulations that all employees must follow in order to perform work in a safe manner. In total, the document has 10 parts [3]. Part one and two provides the application and administration of the OHSA. Part three outlines the duties of employees and other related persons. Part four includes the details of all the different toxic substances at a construction site. Part five states the right to refuse or stop work where health or safety is in danger for all employees. Part six and seven states the reprisals employer prohibited and notices respectively. Part eight, nine and ten states the enforcement, offences and penalties and regulations respectively.

10.2.2 Workplace Hazardous Materials Information System

The Workplace Hazardous Materials Information System (WHMIS) is a system used in Canadian workplaces to provide employees with all the required information on the hazardous materials before it can be used, handled or stored [4]. The law was implemented back in 1988 with the communication of federal, provincial and territorial legislation and regulations [4]. The main components of WHMIS are hazard identification and product classification, labelling, safety data sheets, and worker education and training [4]. WHMIS has also aligned with the worldwide hazard communication system known as GHS – the Globally Harmonized System of Classification and Labelling of Chemicals [4].

10.2.3 Personal Protective Equipment

Personal protective equipment (PPE) protects workers from serious workplace injuries and illnesses resulting from physical, electrical, mechanical, chemical, or other workplace hazards [5]. Figure below shows the different PPE that a construction worker must wear when on a construction site doing a specific task.

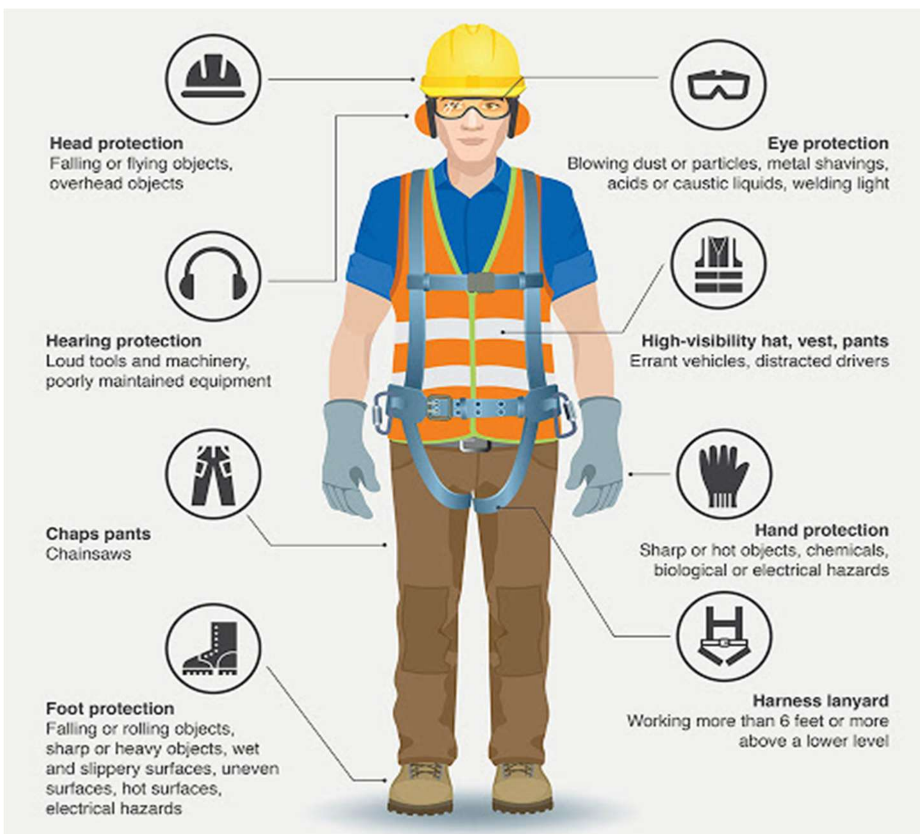

Figure 10.2.3.1 - Example of different personal protective equipment [5]

10.2.4 Construction Safety Signs

Construction safety signs are important visuals on a construction which helps workers identify warnings and other safety information. Some common safety signs are biohazard and hazardous materials, electrical safety, first aid and lockout tagout. The construction safety signs must be compliant with OHSA. Safety signs are bold and bright providing high visibility to critical messages and come in many different sizes and materials. This facilitates the applicability of these signs to different facilities and sites. Figure below shows the examples of common construction safety signs**.**

Figure 10.2.4.1 - Example of Common Construction Safety Signs [6]

10.3 Site Preparation

Site preparation is all the work that needs to be done on the land before construction starts. Contractor is responsible for arranging this preparation. Site preparation is examined under two categories: Land preparation and storage control. Some of the stuff land preparation includes are the demolition of an existing structure, blasting, test drilling, landfill, levelling, earth-moving, excavating, and land drainage. Site preparation also includes storage control and planning.

10.3.1 Land Preparation

Land preparation is the first phase which involves determining where the structure will be located and its elevation. The different factors must be determined that are important for good site access. This will create an efficient and easy way to allow materials and equipment to be delivered. The perimeter access must be planned out at this phase as it helps focus on people and equipment once they're on site; how they will maneuver within the given workspace. Safety is the next step that is very important. It includes keeping a full-time safety manager which helps you identify safety hazards on site. Builder's risk insurance policy is required as it will protect the contractor from site theft, damage or vandalism. Finally, all the temporary facilities must be planned in terms of location, size such as electricity, telephone line, portable washrooms, etc.

10.3.2 Storage Control

All the materials and equipment on site must be stored properly and safely (sometimes at the end of day's work). The common type of materials used in construction such as cement, aggregates, steel work, etc. must be stored carefully in order to prevent the materials from deteriorating or damaging due external conditions such as rain, snow, etc. The storage facility could either be on or off site depending on the site perimeter. The location of the storage facility must be properly planned, so that it doesn't block any site exit or entry points or movement of machinery. Also, the amount of material ordered must be planned out, so that the contractor doesn't order more than what the storage facility can hold. At times, contractors order more material ahead of schedule which can create storage shortage. Therefore, proper storage planning must be implemented.

10.4 Planning and Schedule

The bridge construction process requires proper planning and scheduling in order to finish the construction on time without any interruptions.

10.4.1 Project Procurement Schedule

There are several procurement methods available for bridge construction. The financial situation of the owner, the project nature and size are the most important factors that most decisions for a procurement schedule are based on. Nowadays, one of the most common procurement methods is the Private-Public Partnership. This model is used on large projects where the owner gives the right to operate to the contractor and therefore contractor keeps the income the bridge makes for x amount of years. This way the owner doesn't spend any money and build a bridge. Another common one is Design-Build which is used for smaller size projects. Compared to a traditional Design-Bid-Build procurement, the cost is estimated before the design is fully made in this method. In Design-Bid-Build procurement, the detailed drawings are ready before the cost is established and contractors should bid in order to get the project. As the private sector involvement rises, the risks the owner takes increases [7].

10.4.2 Site Schedule

Site schedule helps develop chronological sequence of the activities which would be done throughout the whole construction process. Some activities might have to come one after the other while other activities may be done simultaneously, and all the directions for these will be placed on the site schedule. Not only that, the schedule should also contain information regarding the amount of time needed for each activity. In order to ensure the site schedule is on time, delivery of materials and structural components (girders, precast) on time and in the correct sequence is very crucial. Figure below shows a template of a construction schedule.

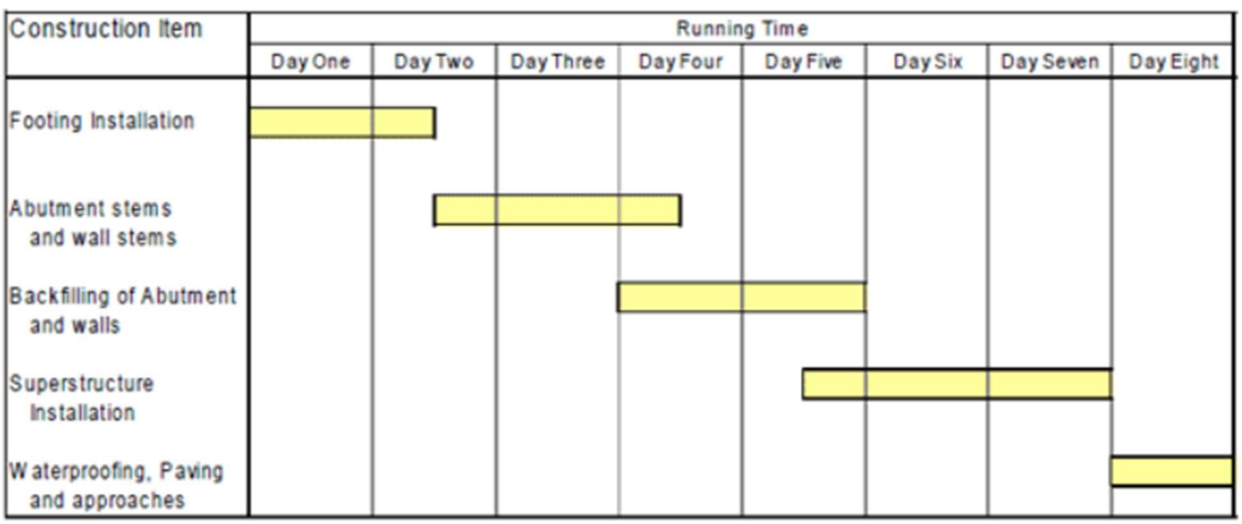

Figure 10.4.2.1 - Construction Timeline Template [8]

10.3.3 Budget

Budgets for construction projects is very important as it helps determine what can be done and what should be done as early as possible. It must be based on previous cost of similar designs and also be realistic at the same time. The budget includes construction cost, as well as the cost of certain items required post - completion and during the project's operational use [9]. The project budget can be established by:

- Assessment of projected income and expenses through the life of the project.
- Comparison with similar projects
- Assessment of the funds available
- Pre-design analysis of requirements
- Pre-design analysis of requirements

It is also very important to account for any unexpected issues and set aside an extra budget.

10.5 Quality Assurance and Control

Quality control helps ensure that the bridge and its maintenance comply with all the requirements and established standards. A productive and efficient project is one which maintains a balance between the three key indicators cost, time and quality [10]. Commonly, quality controls checks are done on the finished portion of a product before the project is delivered back to the owner or client [10]. Also, the quality team needs to examine daily the tasks that are performed as per the required quality standards. Quality management is divided into two components: quality assurance and quality control. Quality assurance is the process oriented; planned and systematic activities executed in a quality system so that quality requirements for a product or service are met according to the established standards [11]. Quality control is product oriented; determines whether the final product has been constructed or implemented correctly [11].

10.5.1 Quality of Concrete

Concrete is the main material used to construct bridges; therefore, the quality of concrete must strictly follow the standards in order to ensure the bridge doesn't collapse and have enough strength to maintain its structural performance. There are three steps for quality of concrete. They are quality control before concrete pour, quality control after concrete pour and quality control after construction.

10.5.1.1 Quality Control before Concrete Pour

This stage consists of two steps as outlined below [12]:

- Checking the specification requirements regarding excavation, forms, reinforcement and embedded fixtures etc.
- Control test on concrete ingredients (i.e. on cement, aggregate & water)

10.5.1.2 Quality Control during Concrete Pour

During concrete pour, careful supervision is required during the manufacturing process for different types of concrete operations such as batching, mixing, transporting, laying, compacting and curing [12]. All the required precautions must be followed during concreting operations. Figure below is one of them, concrete slump test.

Figure 10.5.1.2.1 - Concrete Slump Test [13]

10.5.1.3 Quality Control after Construction

Compression tests must be carried out once the concrete is laid and compacted. The tests done in laboratory conditions usually result in higher strength values. This difference should be accounted if the tests are done in a such condition. The hardened concrete must also be checked for trueness in dimensions, shape and sizes as per design specifications [13]. All the reinforcement must have adequate concrete cover, if not, they should immediately be rejected and replaced. In general, the concrete strength is determined from cube or cylindrical samples after 28 days of placement. It must be ensured that every sample at least meets the specified concrete strength. If they don't meet such requirements, immediate communication with the designer is required and if something needs to be done, it must be done as soon as possible.

Some of the possible things that can be done might be additional load tests on different samples, measurements of deflection under load and chemical analysis [13].

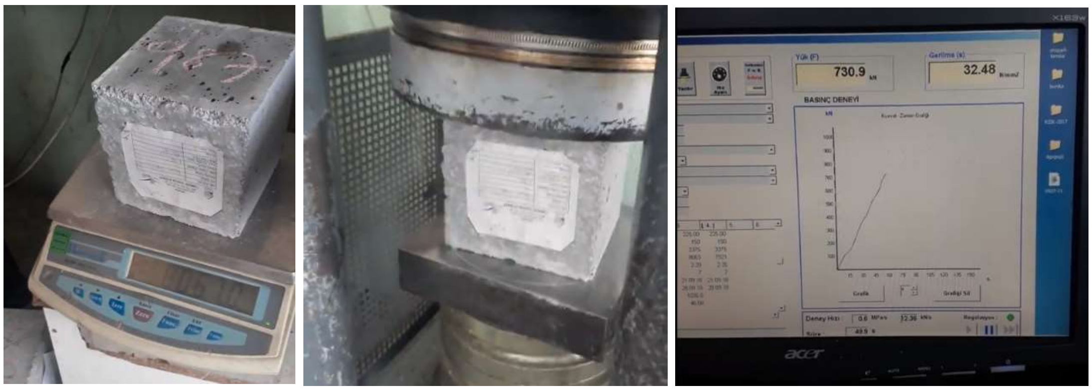

Figure 10.5.1.3.1 – Compression test on a cubic C30/37 concrete sample

10.5.2 Quality Control of Steel Reinforcing

Tensile tests, bending tests and chemical analysis tests are the three common quality checks that are carried out to ensure the quality of steel coming to site [14]. Bending tests help to determine the flexibility and soundness of materials. Tension tests help to determine how the material reacts when tension is applied. These two tests also reveal the weaknesses in the atomic structure of possible faulty steel due to improper heat treating which might make it very brittle. They also provide information about the ultimate tensile strength, yield strength and the elasticity of the sample [14]. Chemical tests help to determine the chemical contents of a sample and learn the composition, structures, and material properties from the atomic scale up to molecule scale [14]. Visual inspections are also necessary. For example, the alignment of ribs can be verified by visually looking at the bar.

10.6 Formwork

Formworks are temporary or permanent molds into which concrete or similar materials are poured. Type of formwork to be used depends on the formwork material and the type of structural element [15]. Constructing the formwork is a time-consuming process and involves expenditure up to 20 to 25% of the cost of a structure. Sometimes it can be even more [15]. The most common material used for formwork is timber. It is water impermeable and its cost outweighs all, but it has also some disadvantages such as susceptibility to warp, swell and shrinkage [15]. Below are few of the requirements that a formwork should meet:

- It should be strong enough to handle the load of fresh concrete.
- It should be rigid enough and braced both horizontally and vertically.

- Its joints must be tight to avoid the leakage of fresh concrete.
- It shouldn’t be heavy
- It should rest on firm base

10.7 Conclusion

To have a feasible, safe and as requested design, the engineer should design a constructible structure. Also, all the different construction stages must be well planned. The construction workers of the bridge should comply with all the safety requirements and standards. The site must be well prepared before any construction starts. Work should be done on time without any delays if possible. The importance of proper planning and scheduling should be realized. Lastly, the quality control of the structure must be fully done before it opens to service.

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Chapter 11 – Plant Life Management

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11.1 Introduction
In this final chapter, plant life management of the bridge will be analyzed, a strategy whose goal is to identify all the factors that are essential to ensure the structure is stable, durable and safe throughout its life span. It is a good program for long term operations of structures. They incorporate ageing and economic planning to maintain a high level of safety and optimize structural performance by dealing successfully with extended life ageing issues, maintenance prioritization, periodic safety reviews, and education and training [1]. It is also useful in helping owners make an informed decision on continuing to operate the structure longer than their originally assumed design life. The three main areas of PLiM that will be discussed in this chapter are Aging Management Program (AMP), durability design and strategic material selection.

11.2 Plant Life Management (PLiM)
The proper use of a plant life management program is to use it throughout the whole process that is: Design, construction, erection, commissioning, handover, operation, maintenance/ refurbishment/life extension, and decommissioning. It helps identify areas that are critical to structures safety and operation.

Some of the benefits of PliM are [1]:
- Ageing degradation mechanisms help determine unexpected or unplanned functional failure
- It provides opportunity to create value by considering alternative operation and maintenance practices
- Long term ageing plans
- Financial optimization

This section will highlight design methodology, material performance, and lifecycle management.

11.3 Design Methodology
Figure 11.3.1 below shows the two components of design methodology: Strength & serviceability and Durability. Both these components are affected by the selection of the material used in the design. This chapter will focus on how these two components are related to each other and their dependence on time.

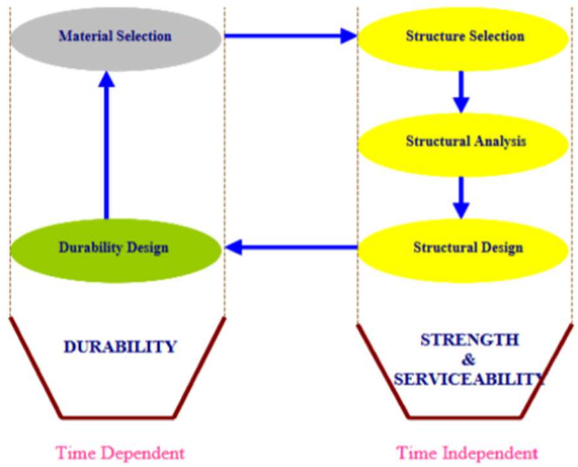

Figure 11.3.1 - Components of Design Methodology [2]

11.3.1 Strength and Serviceability

The conceptual design is done before the detailed design to help get a rough idea of the requirements that are critical to the design. Strength & serviceability design and durability design must be done simultaneously by coordinating between the two. The material selection and geometry selection are according to different bridge codes, CSA S6-14 rev.17, AASHTO LRFD 2014-17 and CSA S6-66. Then, structural analysis is carried out by checking the design behavior under different load combinations. After those are established, the construction process can begin. If some modifications are required in the design process, then the conceptual design must be revisited to change or fix the issue.

11.3.2 Durability Design

Durability design is one of the most crucial factors when constructing a bridge. It affects the bridge in the long run, when exposed to different climatic conditions. A good durability design helps ensure that a structure can withstand the governing load combinations and meet the objectives of the design throughout its lifespan. Chapter 9 in this report discusses the durability design in detail.

11.3.3 Construction

The construction phase is where execution takes place according to the design drawings and also it is the phase where a lot of issues occur. The construction of the structure usually doesn’t exactly match with design drawings. Therefore, the designers should plan ahead and allow a certain margin of errors for practicality. Also, transporting and storing material can affect the material’s performance. Therefore, precaution must be taken when transporting and storing materials as it will affect the durability and strength of the material which in turn will result in design failure.

11.4 Material Selection, Storage and Performance

The material selection process depends on factors such as location of site, soil and ground conditions and varying temperatures the materials will be exposed to. The selection of materials must also satisfy economic factors, mechanical and non-mechanical properties, production and transportation considerations. The materials chosen must maintain its strength and durability throughout its lifespan. The storage of materials can either be on site or off site depending on the availability of space and ease of transportation. The performance of materials must be monitored regularly through the Ageing Management Program (AMP).

11.4.1 Storage of Materials

The storage of materials can either be on site or off site depending on the availability of space and ease of transportation. Storage of materials is a critical component of any construction work and must be done in a planned and orderly manner so that it does not block any site exit and entry point, endanger the safety of workers, and also does not be in the way of heavy machinery as they can damage the material. Any hazardous and corrosive material must be stored in accordance with the regional guidelines. The stockpiles that can be easily damaged by external environment conditions must be stored in storage containers. Figure 11.4.1.1 below shows rusted rebar and incorrect storage of rebars making it difficult to access.

Figure 11.4.1.1 - Rusted Rebars due to Incorrect Storage [3]

11.4.2 Quality Control of Materials

The quality control of materials helps ensure that the responsible party complies with requirements and established standards in terms of materials and how work is done. It helps minimize the chance of defects in the finished design by inspecting each material and structure, for example, a precast girder. This helps ensure strength and durability of design in order to prevent any design failure or collapses because of material discrepancy. Quality control in general helps ensure that the work is carried out as planned by doing daily quality checks. There are different quality tests for different construction materials such as concrete, steel. For example, fatigue tests are used to determine the behavior of steel materials when subjected to repeated or fluctuating loads. There are different tests to check the quality of concrete itself such as: slump test, compressive strength test, water permeability, rapid chloride ion penetration, water absorption, initial surface absorption.

11.5 Ageing Management Program (AMP)

Ageing Management Program is another important phase which must be done after construction. It helps detect any malfunctions and errors in the structure and its components throughout its service life. It also helps ensure the integrity and functional capability of the structure throughout its lifespan. Since each project has different conditions and requirements, every project will have a unique AMP. Figure 11.5.1 below shows the general flow chart of AMP.

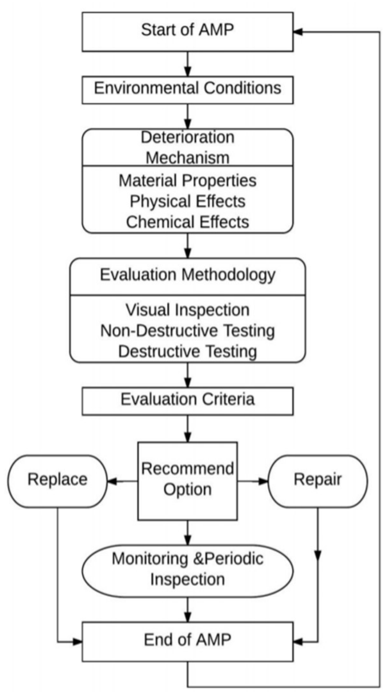

Figure 11.5.1 - Ageing Management Program Flow Chart [4]

11.5.1 Environmental Conditions

The environmental conditions change over time and no one has control over it. Therefore, it is difficult to interpret the environmental effects on the structure and its components at the beginning of the project. In order to keep track of things, regular monitoring must be done and noted down as daily reports are crucial for AMP. The daily report can include change in temperature, radiation, humidity levels, etc.

11.5.2 Deterioration Mechanisms

The deterioration of both concrete and reinforced concrete can occur at any time during its lifespan. Figure 11.5.2.1 below shows the different deterioration mechanisms, and also shows how they do not necessarily start at the same time [4]. It is important to understand the different deterioration mechanisms when developing AMP. Chapter 9 explains in detail the different deterioration mechanisms.

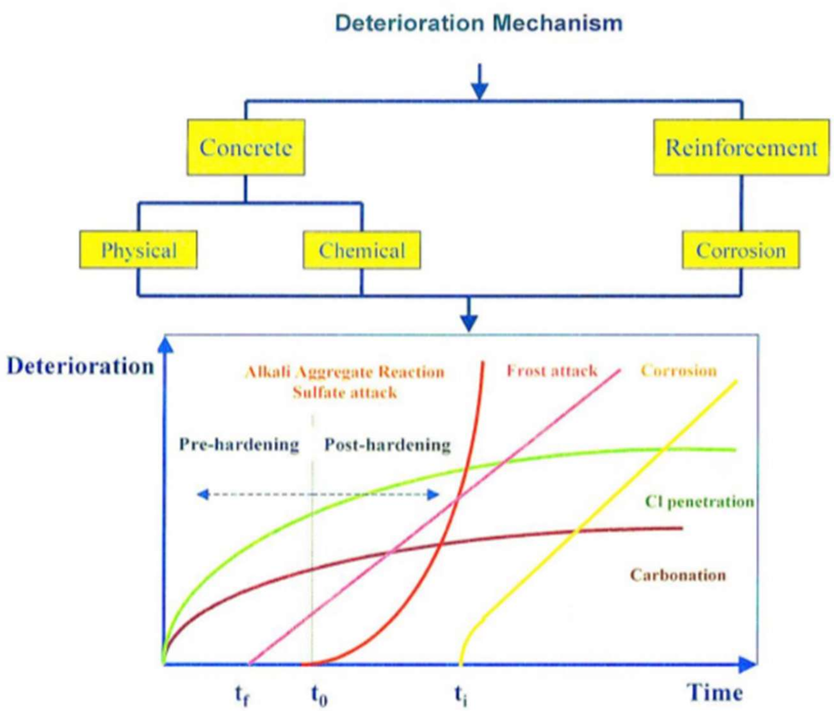

Figure 11.5.2.1 - Different Deterioration Mechanisms [4]

11.5.3 Evaluation Methodology and Criteria

Evaluation Methodology is important when designing an AMP. The three components of evaluation methodology and criteria are visual inspection, non-destructive tests, and destructive tests.

11.5.3.1 Visual Inspection

Visual inspection is usually done by an experienced professional. It is an easy and fast method. It helps detect external deficiencies and internal deficiencies. External deficiencies include concrete cracks, corrosion damage, leakages, etc. The degree of accuracy of this method is not highly reliable but gives a rough estimate of the situation. The inspection can be performed by using video cameras.

11.5.3.2 Non-Destructive Test (NDT)

Non-destructive test is an analysis technique used to evaluate the strength and integrity of the material. It is a more accurate method in comparison with visual inspection. Some of the common non-destructive test are [5]:

- Acoustic Emission Testing
- Electromagnetic Emission Testing
- Ground Penetration Radar
- Laser Testing Methods
- Leak Testing
- Schmidt Rebound Hammer Testing

11.5.3.3 Destructive Test

Destructive tests are used when the structure in question is determined to have an issue. Some of the goals of these kinds of tests are determining concrete strength, locating rebar sizes and spacing and so on. This testing method reveals the unpredictable behavior of a structure that is not possible to understand with other testing methods. Some of the common destructive test are:

-Core testing
-Probe Testing
-Partial Break-off test

Figure 11.5.3.3.1 – Core Testing Preparation and the Core Extracted

11.5.4 Recommend Option

Based on the three tests shown in the previous subsection: Visual inspection, destructive and non-destructive testing, the professional engineer must use his/her judgement and determine whether the material is safe or not.

11.5.5 Decommissioning

Once the structure strength does not satisfy the minimum requirement and the cost of repairing the structure is greater than replacing the structure, the structure will be decommissioned. This is the end of the service life of the structure. Moreover, the decommissioning phase must be planned in detail before starting the process, as sometimes some components of the structure can be separated and recycled and be used for other projects. Also, traffic plans and environmental plans must be considered in the planning face of the decommissioning process in order to avoid any delays.

11.6 Conclusion

Plant Life Management (PLiM) is a powerful tool to keep track of the structure performance and functionality throughout its service life. The preconstruction, construction and post construction phases are all important to ensure the structure keeps its integrity throughout its expected lifespan. The designers must design a structure which is feasible in terms of constructability.

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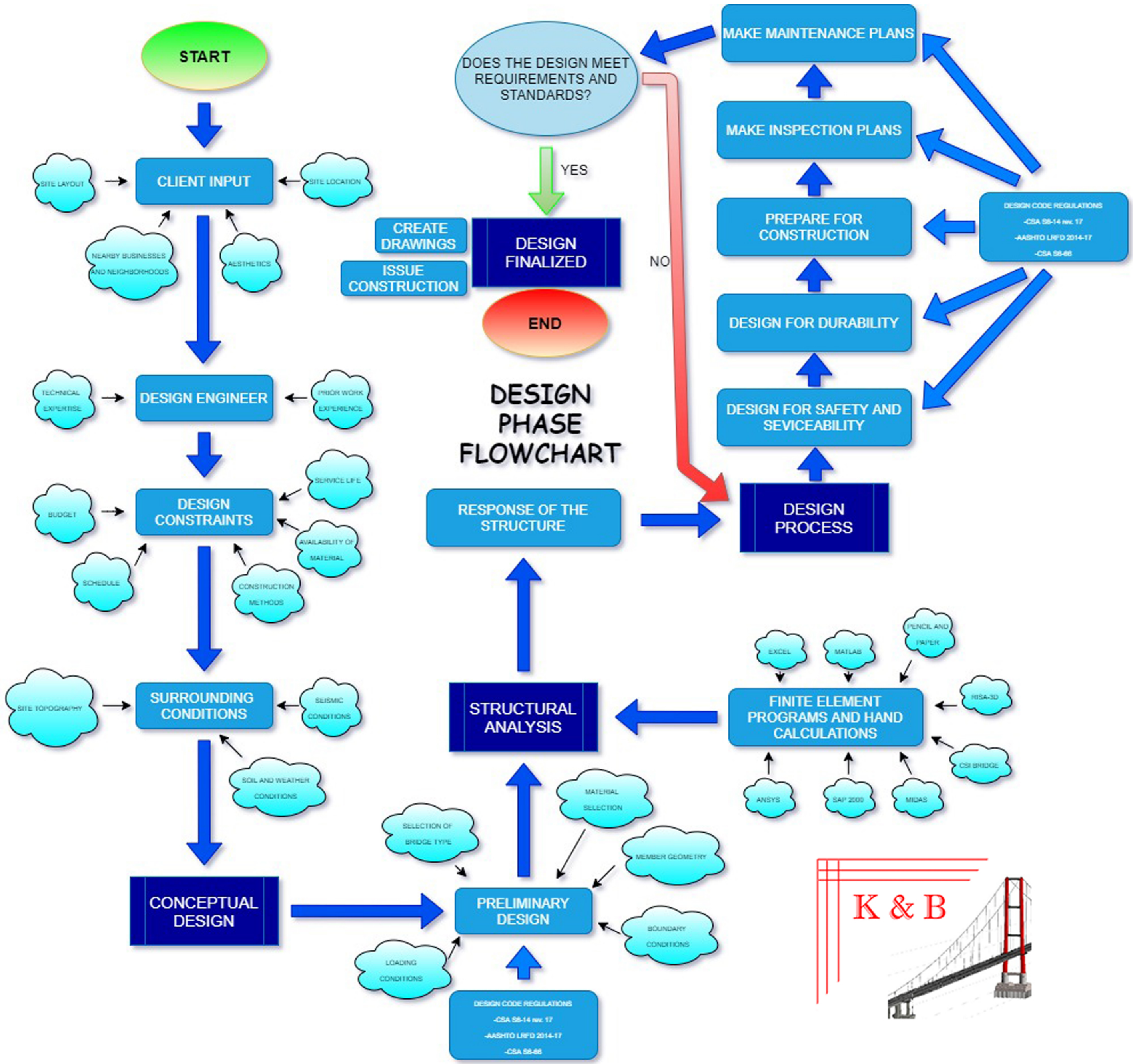
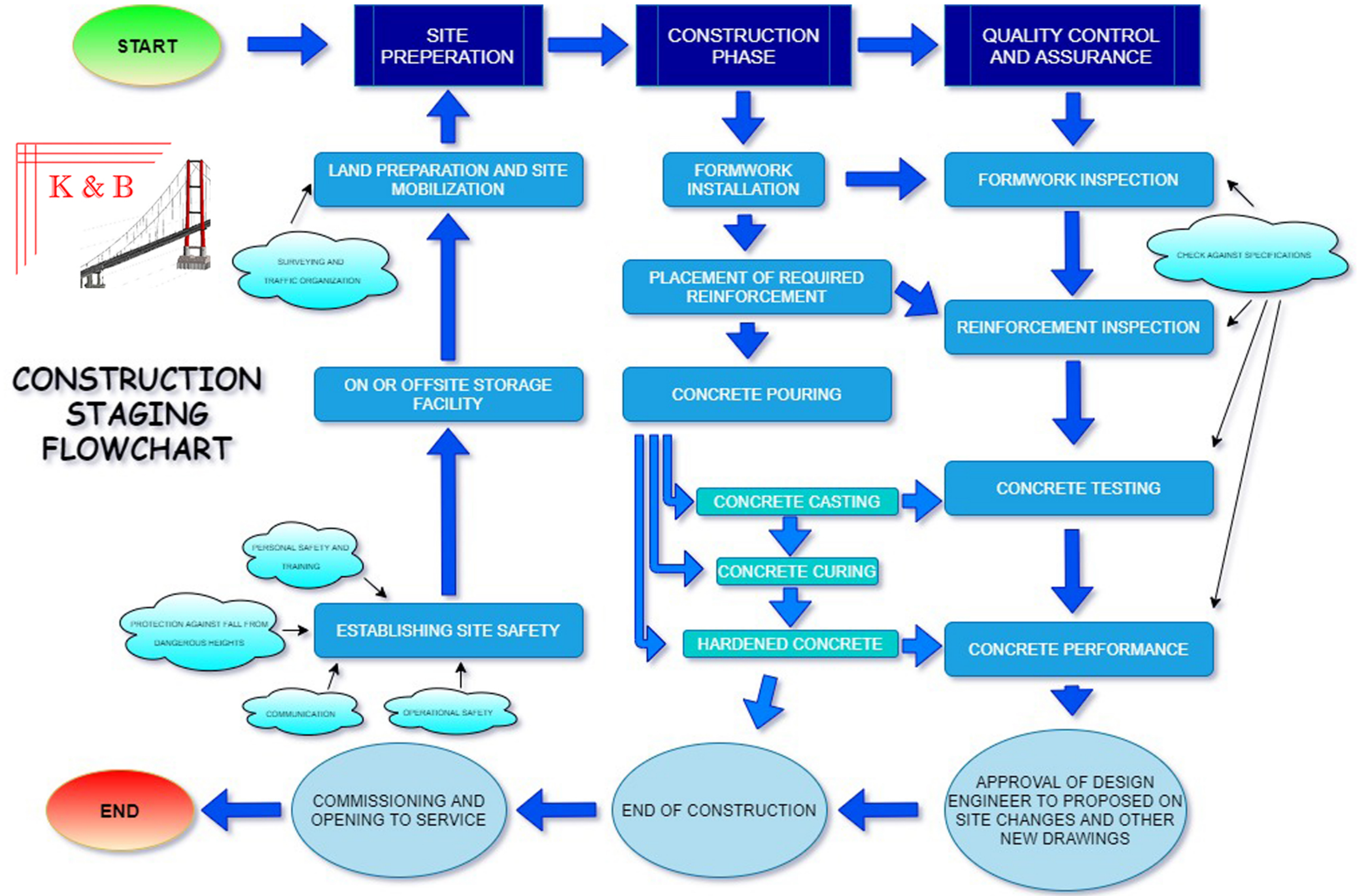
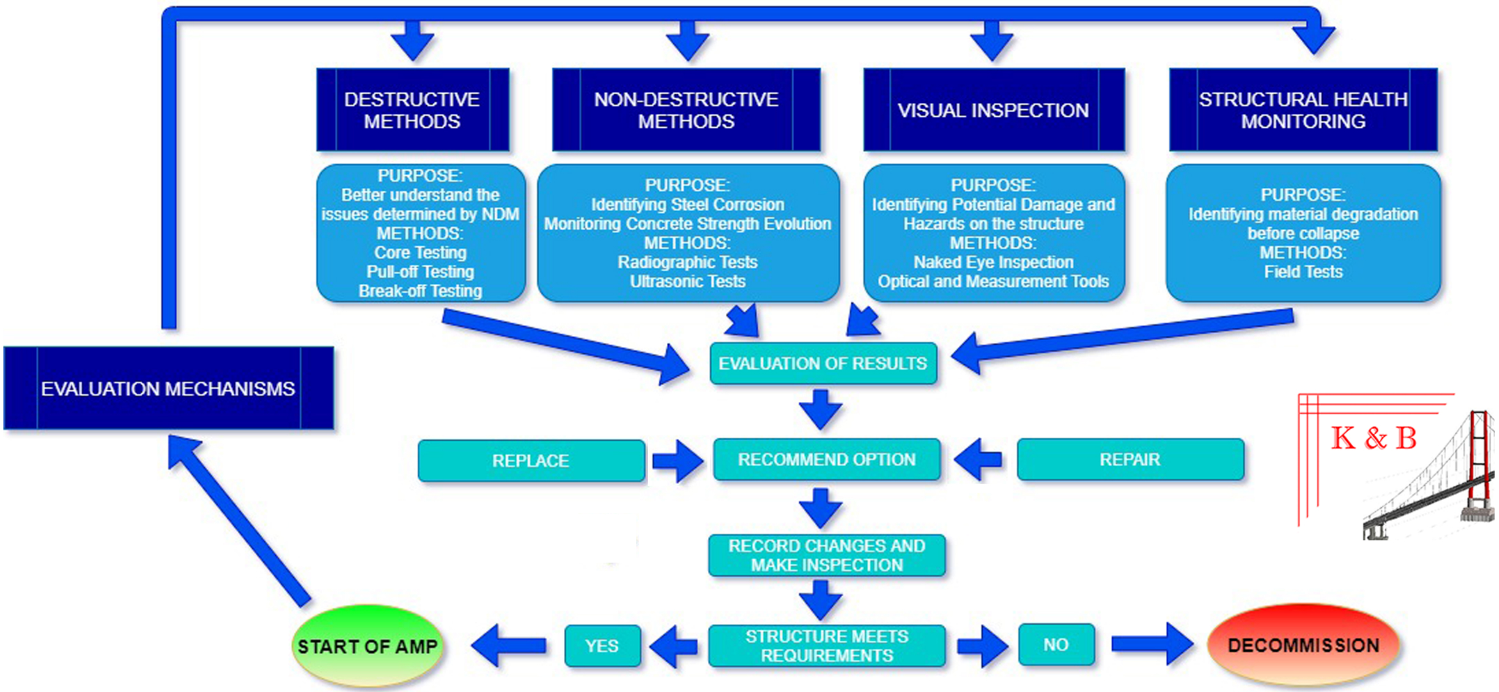


DESIGN PHASE FLOWCHART
START
CLIENT INPUT
SITE LAYOUT
SITE LOCATION
NEARBY BUSINESSES AND NEIGHBORHOODS
AESTHETICS
DESIGN ENGINEER
TECHNICAL EXPERTISE
PRIOR WORK EXPERIENCE
DESIGN CONSTRAINTS
BUDGET
SERVICE LIFE
AVAILABILITY OF MATERIAL
SCHEDULE
CONSTRUCTION METHODS
SURROUNDING CONDITIONS
SITE TOPOGRAPHY
SEISMIC CONDITIONS
SOIL AND WEATHER CONDITIONS
CONCEPTUAL DESIGN
PRELIMINARY DESIGN
SELECTION OF BRIDGE TYPE
MATERIAL SELECTION
MEMBER GEOMETRY
BOUNDARY CONDITIONS
LOADING CONDITIONS
DESIGN CODE REGULATIONS
-CSA S6-14 rev. 17
-AASHTO LRFD 2014-17
-CSA S6-66
STRUCTURAL ANALYSIS
FINITE ELEMENT PROGRAMS AND HAND CALCULATIONS
EXCEL
MATLAB
PENCIL AND PAPER
RISA-3D
CSI BRIDGE
ANSYS
SAP 2000
MIDAS
RESPONSE OF THE STRUCTURE
DESIGN PROCESS
DESIGN FOR SAFETY AND SEVICEABILITY
DESIGN FOR DURABILITY
PREPARE FOR CONSTRUCTION
MAKE INSPECTION PLANS
MAKE MAINTENANCE PLANS
DESIGN CODE REGULATIONS
-CSA S6-14 rev. 17
-AASHTO LRFD 2014-17
-CSA S6-66
DOES THE DESIGN MEET REQUIREMENTS AND STANDARDS?
NO
YES
DESIGN FINALIZED
CREATE DRAWINGS
ISSUE CONSTRUCTION
END
K & B


START
SITE PREPERATION
CONSTRUCTION PHASE
QUALITY CONTROL AND ASSURANCE
K & B
LAND PREPARATION AND SITE MOBILIZATION
SURVEYING AND TRAFFIC ORGANIZATION
FORMWORK INSTALLATION
FORMWORK INSPECTION
CHECK AGAINST SPECIFICATIONS
PLACEMENT OF REQUIRED REINFORCEMENT
REINFORCEMENT INSPECTION
CONSTRUCTION STAGING FLOWCHART
ON OR OFFSITE STORAGE FACILITY
CONCRETE POURING
CONCRETE CASTING
CONCRETE TESTING
CONCRETE CURING
HARDENED CONCRETE
CONCRETE PERFORMANCE
PERSONAL SAFETY AND TRAINING
PROTECTION AGAINST FALL FROM DANGEROUS HEIGHTS
ESTABLISHING SITE SAFETY
COMMUNICATION
OPERATIONAL SAFETY
END
COMMISSIONING AND OPENING TO SERVICE
END OF CONSTRUCTION
APPROVAL OF DESIGN ENGINEER TO PROPOSED ON SITE CHANGES AND OTHER NEW DRAWINGS

AGING MANAGEMENT PROGRAM FLOWCHART



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.

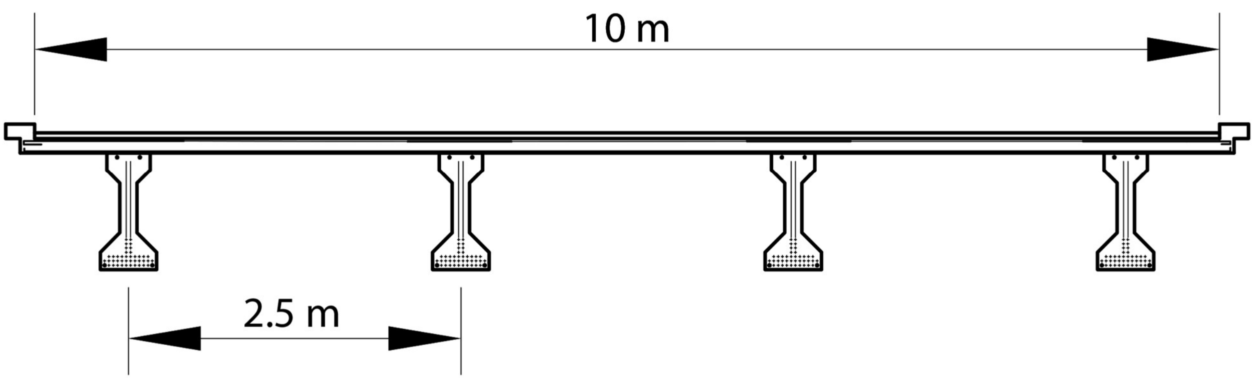


Figure 2.1.1 - Sample cross-section of the bridge to be designed

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*

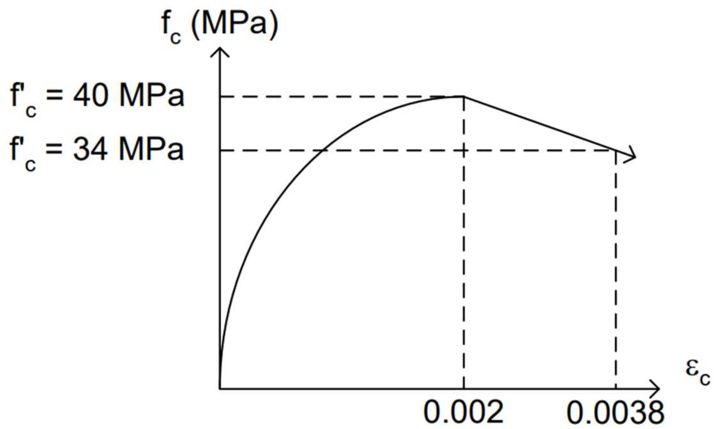


Figure 2.2.1 Girder concrete stress-strain relationship

*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

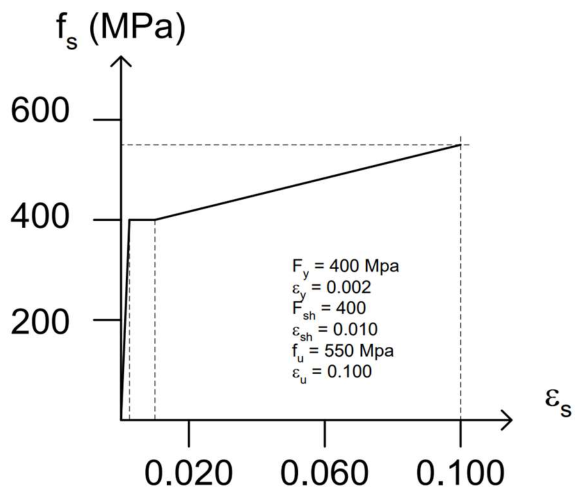


Figure 2.2.2 Reinforcing steel stress-strain relationship

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

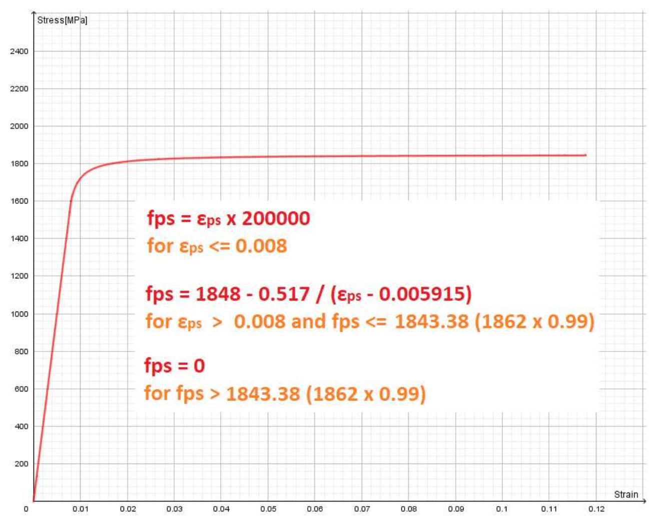


Figure 2.2.3 Prestressing steel stress-strain relationship

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.

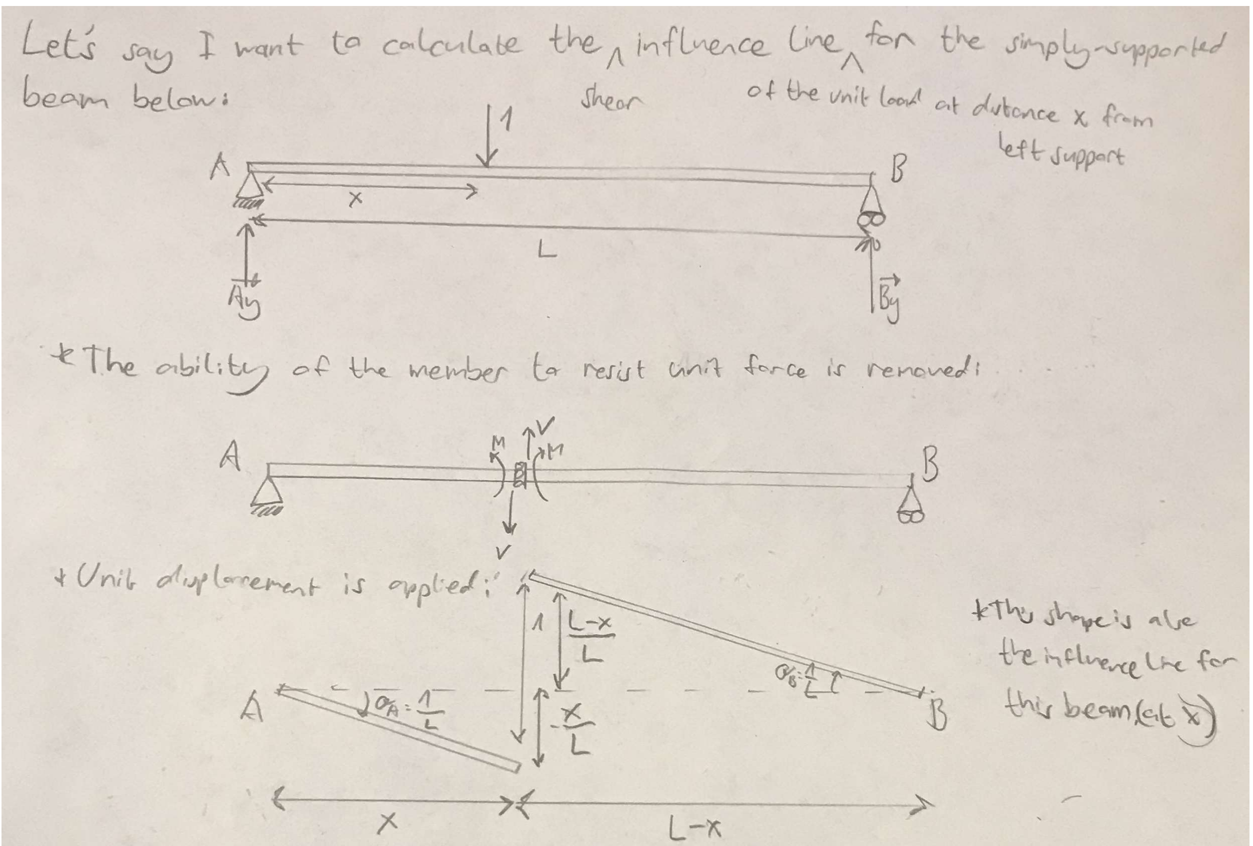


Figure 3.2.3.1: Qualitative Influence Lines

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.

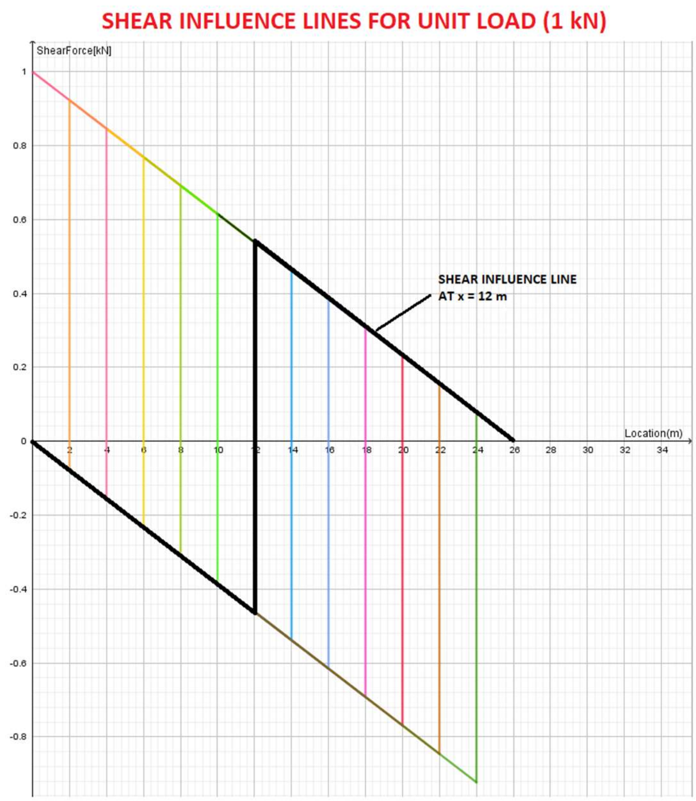


Figure 3.3.1

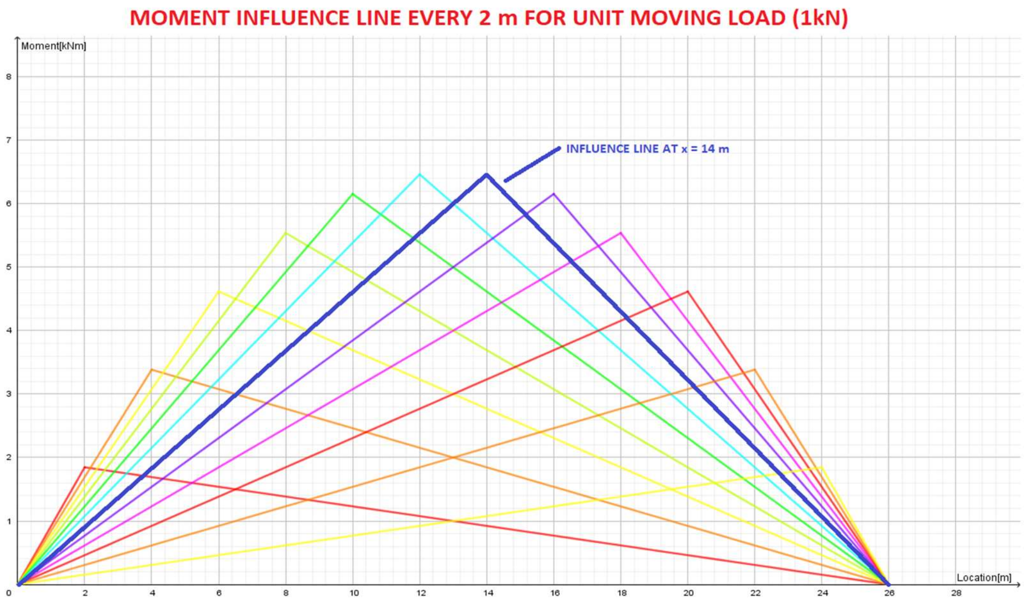


Figure 3.3.2

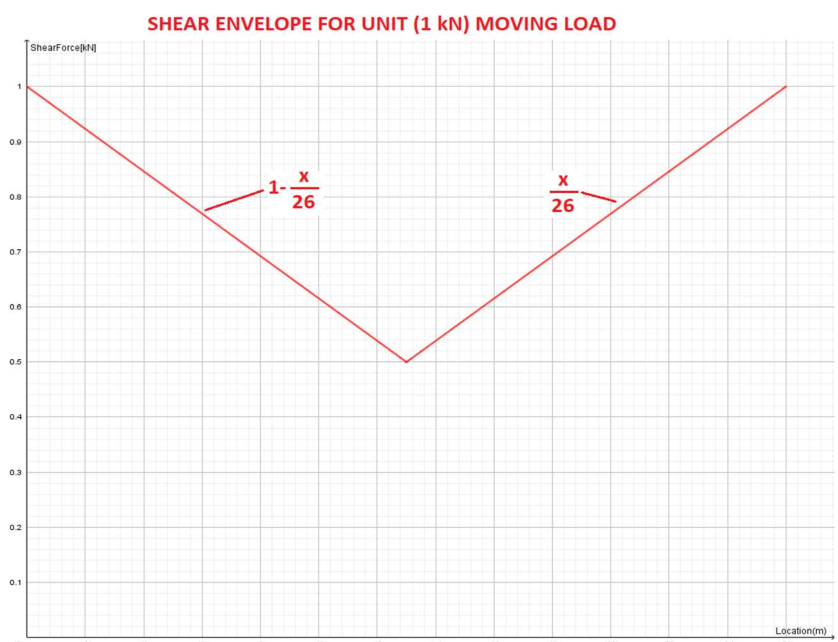


Figure 3.3.3

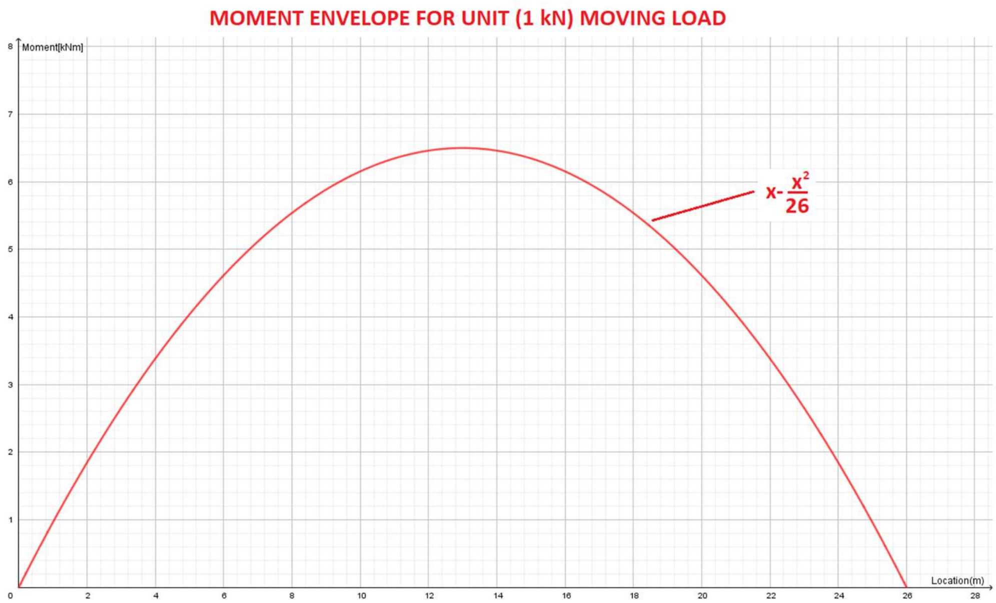


Figure 3.3.4

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:

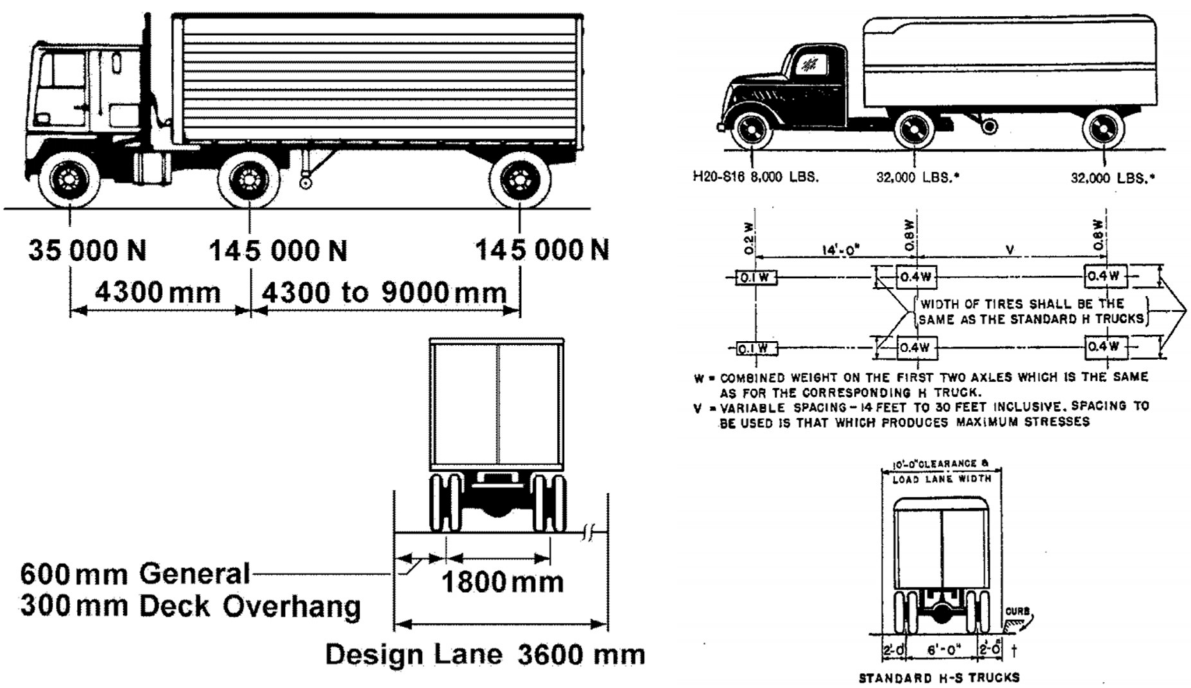


Figure 3.4.1 Design Truck for CSA S6-66 and AASHTO LRFD 2014-17 [4][6]

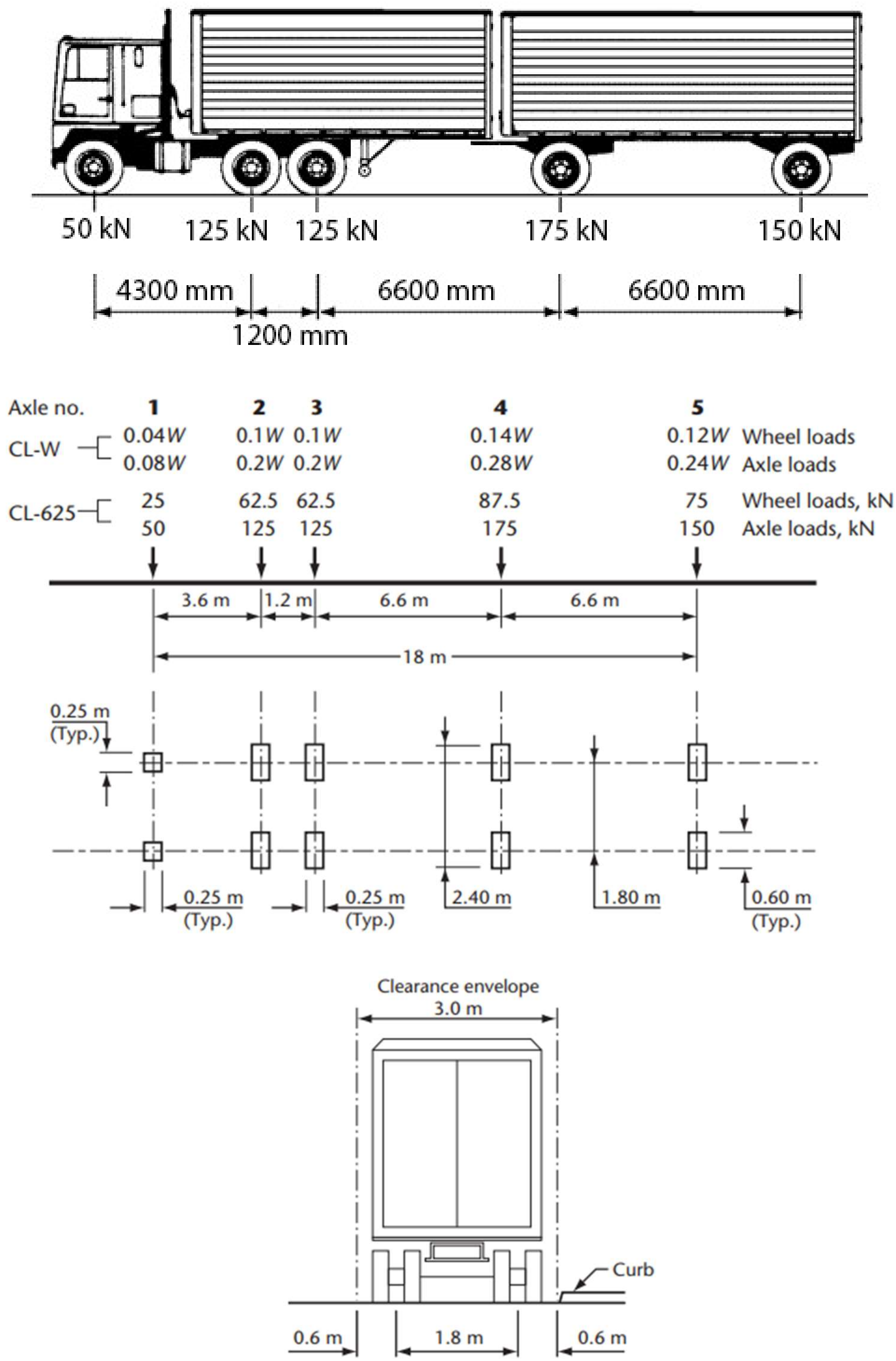


Figure 3.4.2 Design Truck For CSA S6-14 [5]

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.

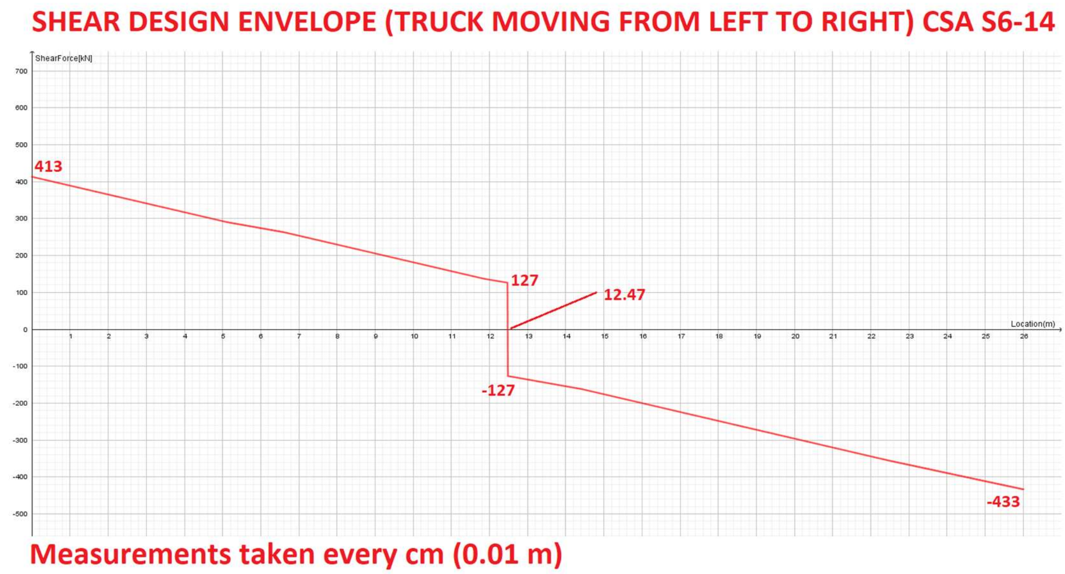


Figure 3.4.1.1

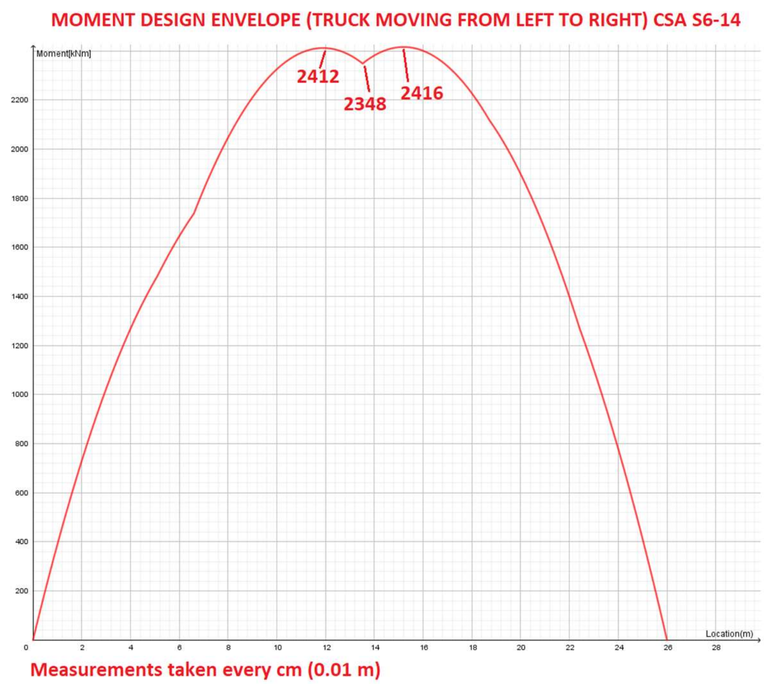


Figure 3.4.1.2

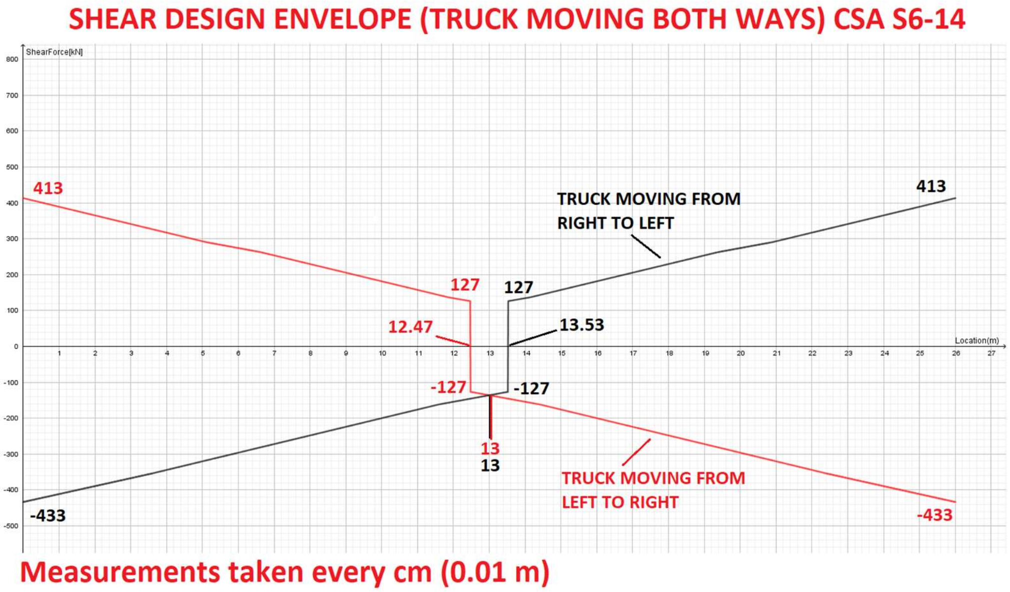


Figure 3.4.1.3

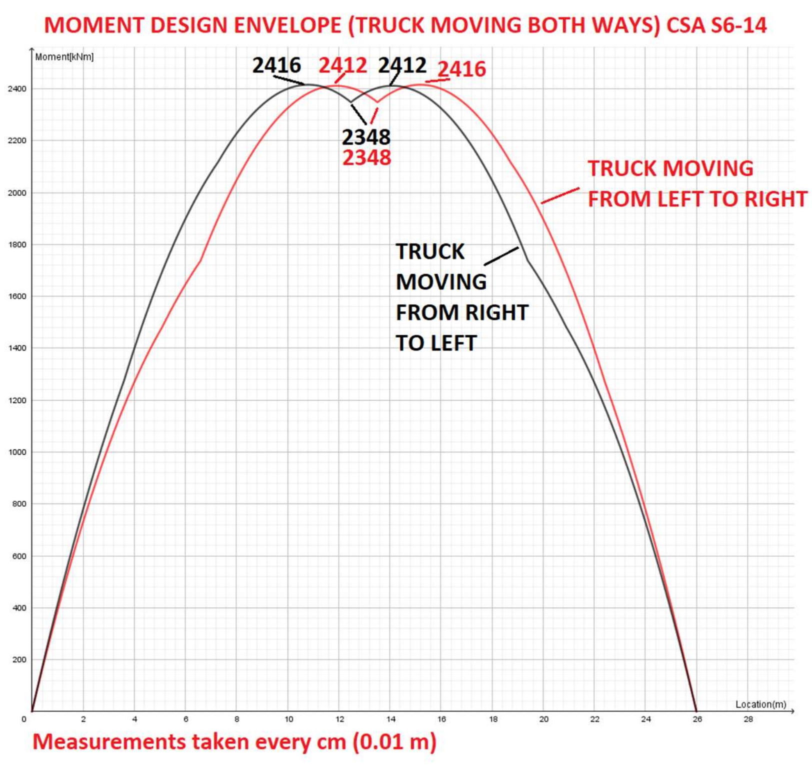
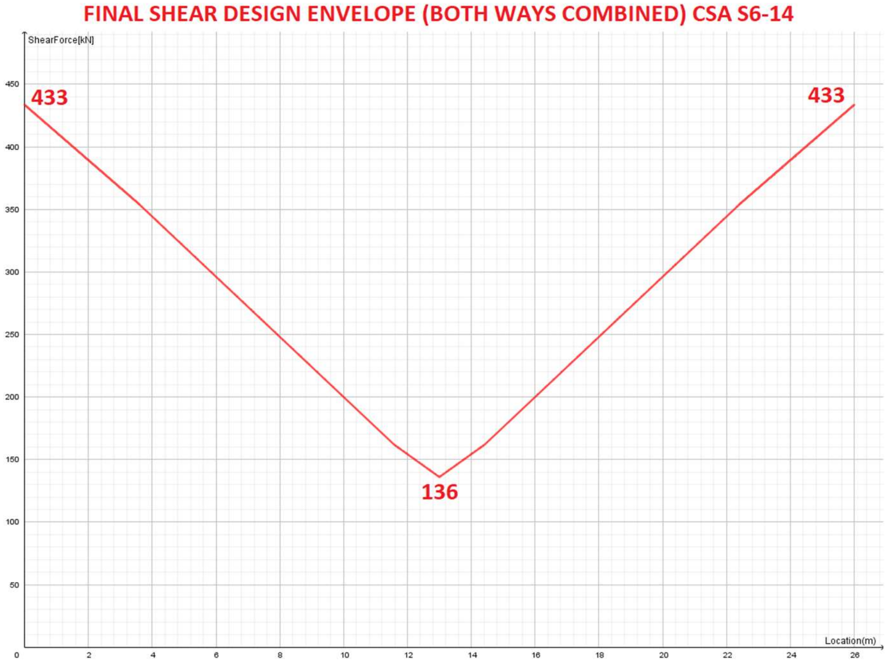


Figure 3.4.1.4

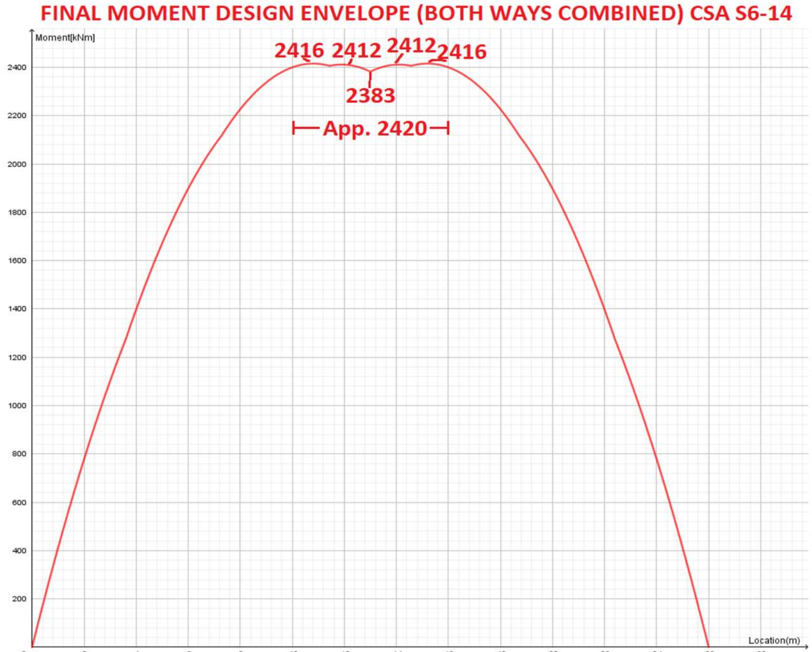
FINAL SHEAR DESIGN ENVELOPE (BOTH WAYS COMBINED) CSA S6-14



Measurements taken every cm (0.01 m)

Figure 3.4.1.5

FINAL MOMENT DESIGN ENVELOPE (BOTH WAYS COMBINED) CSA S6-14



Measurements taken every cm (0.01 m)

Figure 3.4.1.6

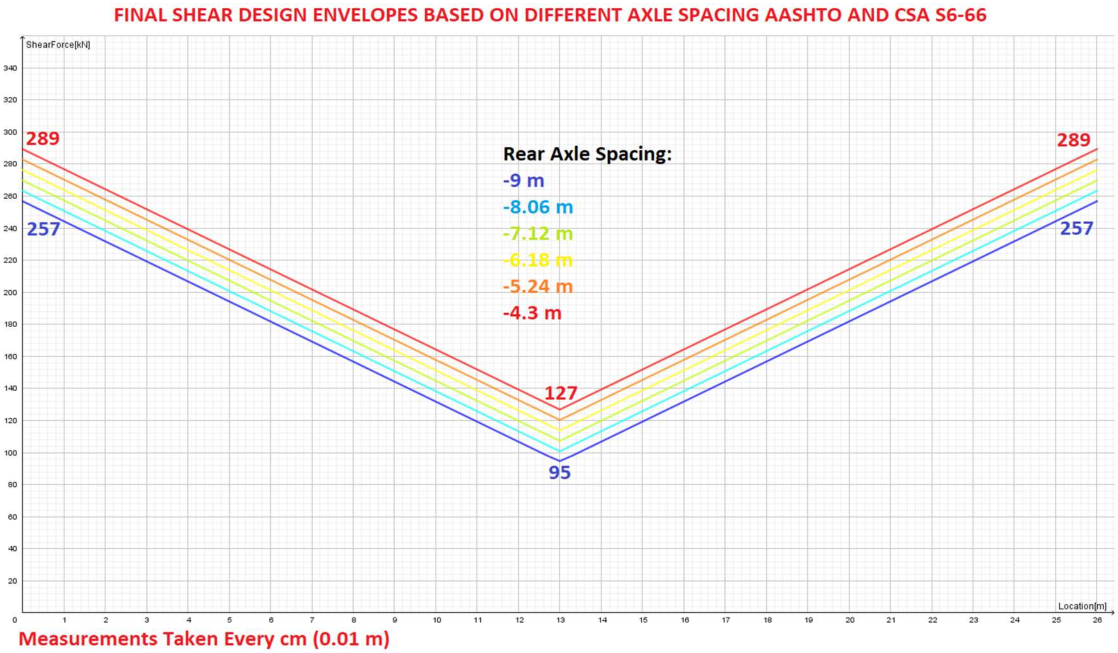


Figure 3.4.1.7

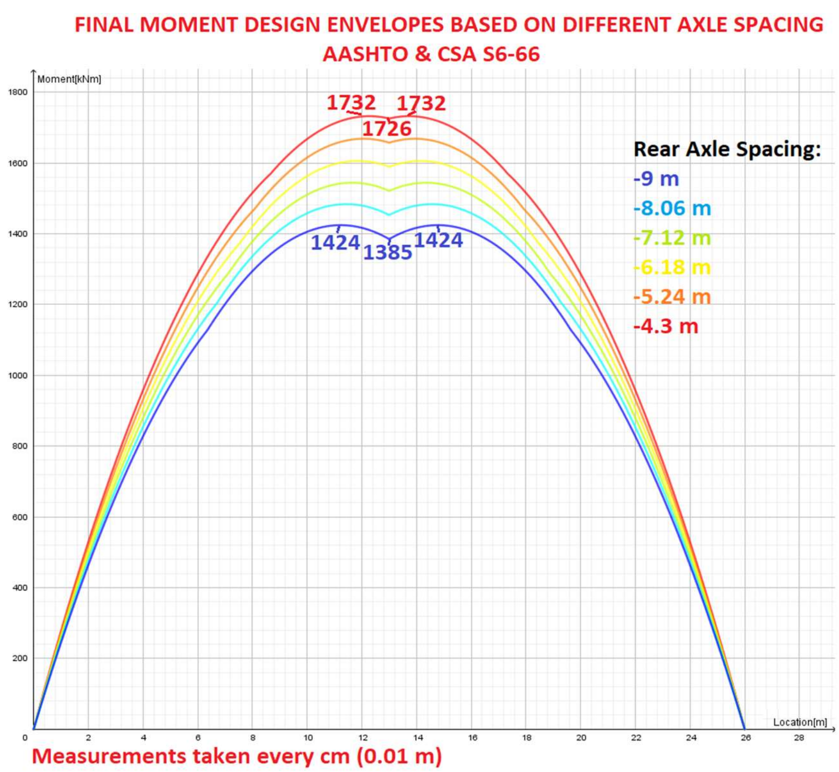


Figure 3.4.1.8

Figure 3.4.1.9

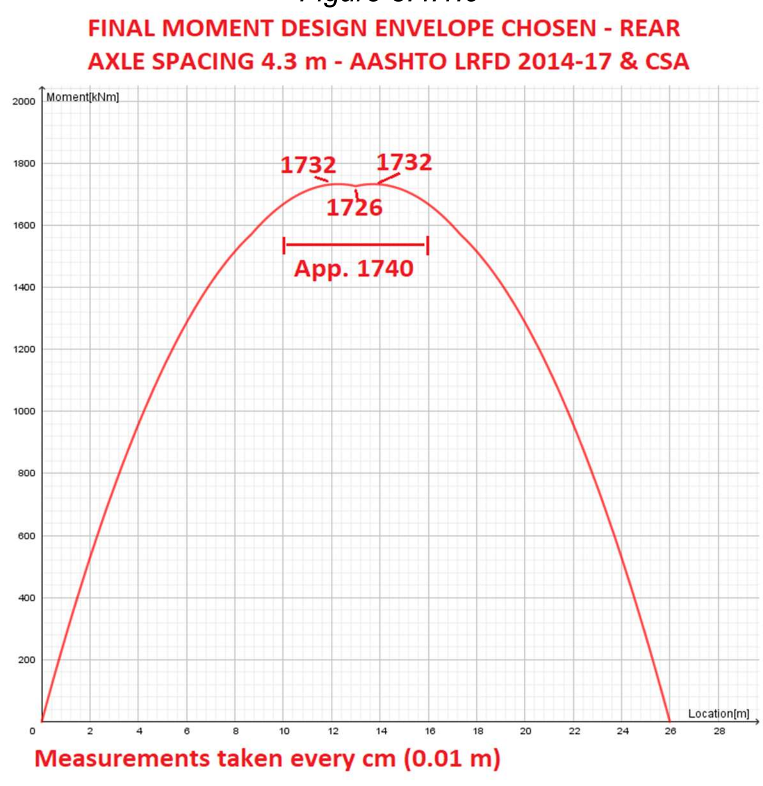


Figure 3.4.1.10

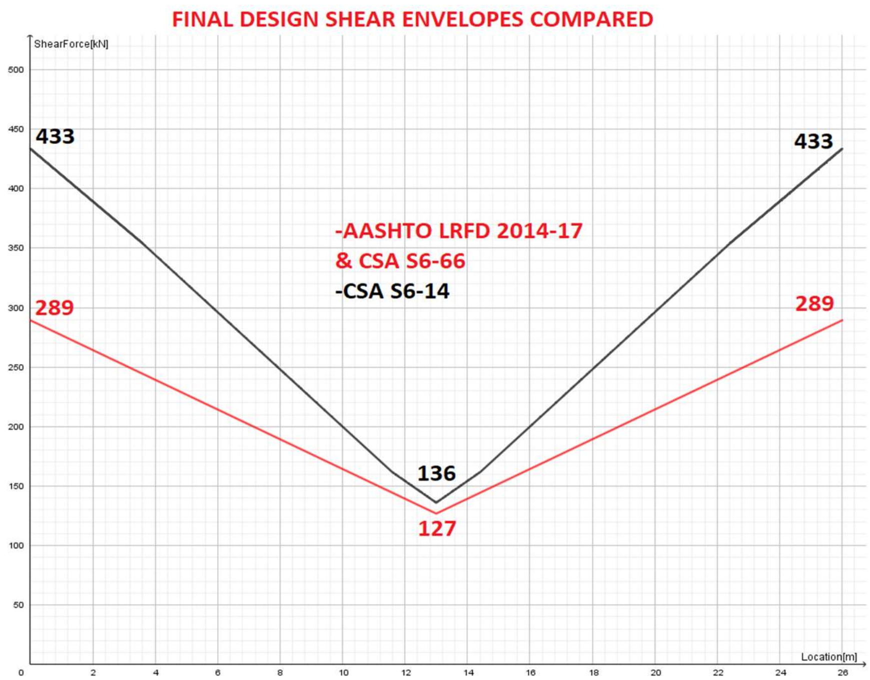


Figure 3.4.1.11

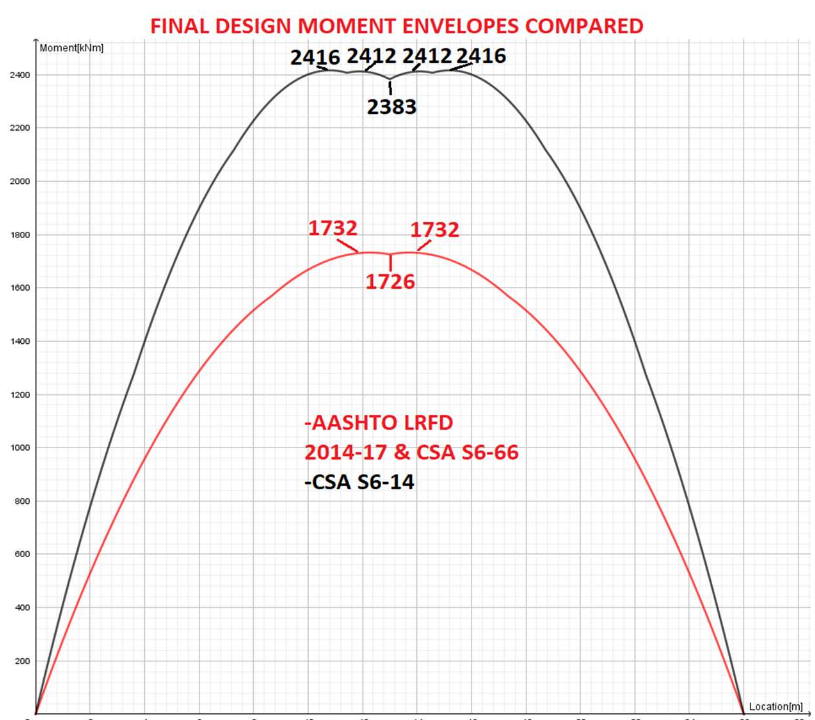
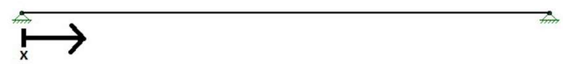


Figure 3.4.1.12

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

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.

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[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

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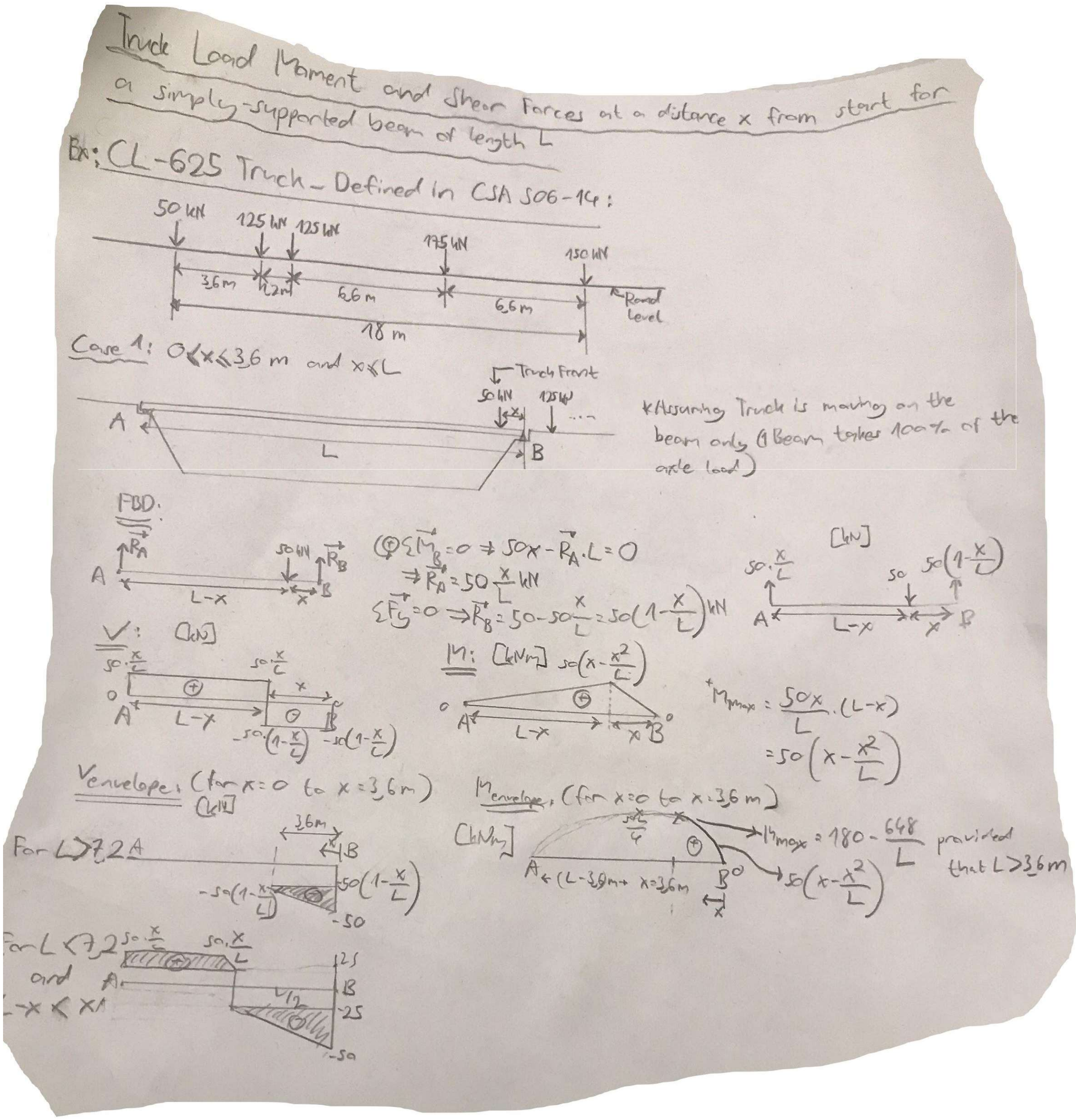
[6] CSA S6-66 Design of Highway Bridges: Canadian Standards Association, 1966

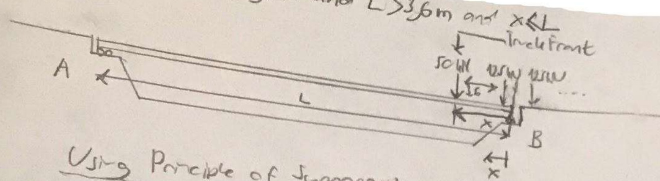
UNIVERSITY OF TORONTO

3.8 Appendix

3.8.1 Sample hand calculations for showing procedure



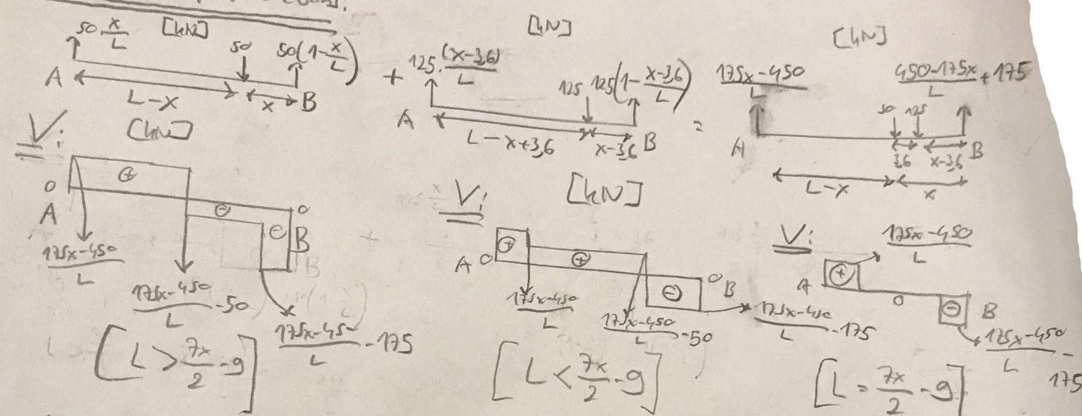
Case 2: $3.6 < x \leq 4.8$ m and $L > 3.6$ m and $x \leq L$



* Same Assumptions with Case 1

Using Principle of Superposition of Linear Systems:

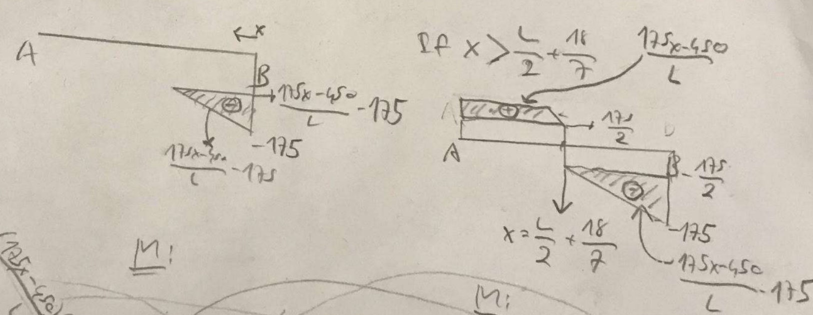
FBD and support reactions:



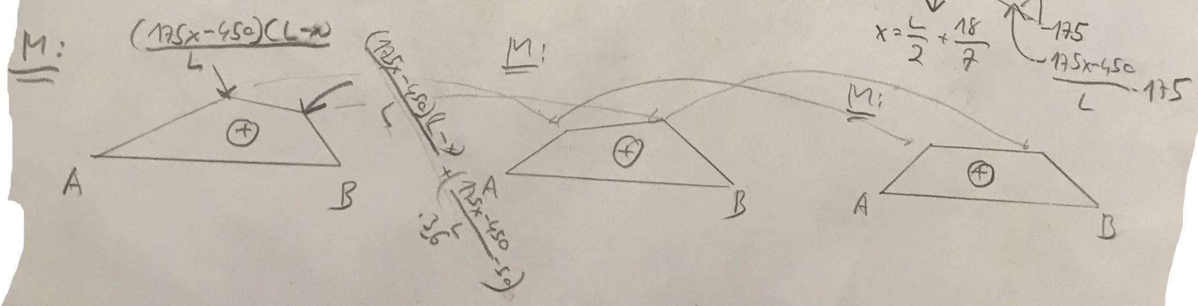
V envelope:

If $x \leq \frac{L}{2} + \frac{18}{7}$

If $x > \frac{L}{2} + \frac{18}{7}$



M:



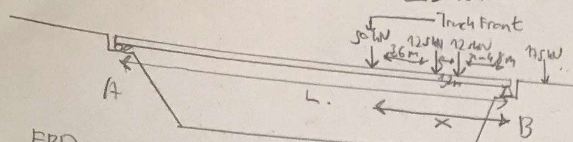
* From now on, I will proceed with $L > 18$ m since:

a) I showed the calculation procedure

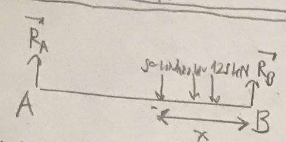
b) Our design length is 26 m.

c) Calculations get complicated.

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



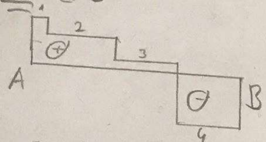
FBD and support reaction using superposition:



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

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

V: [kN]



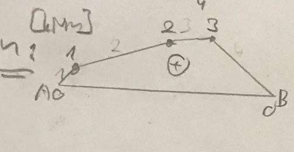
$$1 = \frac{300x - 1050}{L}$$

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

$$3 = \quad '' \quad - 175$$

$$4 = \quad '' \quad - 300$$

M: [kNm]



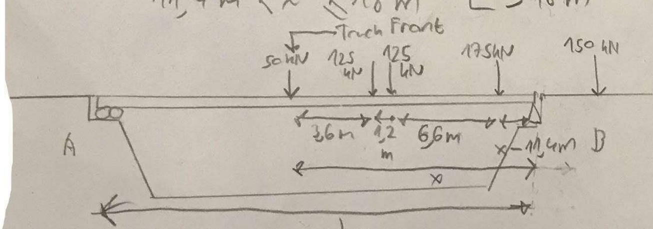
$$1 = \frac{300x - 1050}{L}(L - x)$$

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

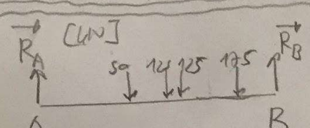
$$3 = \frac{300x - 1050}{L}(L-x) + \left(\frac{300x - 1050}{L} - 50\right)(3{,}6) + \left(\frac{300x - 1050}{L} - 175\right)(1{,}2)$$

Case 4:

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



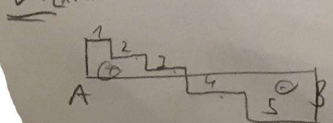
FBD and support reactions using superposition:



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

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

V: [kN]



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

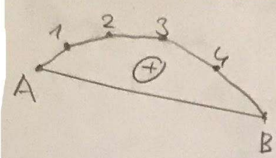
$$2 = \quad '' \quad - 50$$

$$3 = \quad '' \quad - 175$$

$$4 = \quad '' \quad - 300$$

$$5 = \quad '' \quad - 475$$

M : [kNm]



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

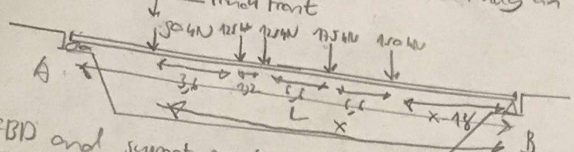
$2 = a + \frac{475x-3045}{L}\cdot(3{,}6) = b$

$3 = b + \frac{475x-3045}{L}\cdot(1{,}2) = c$

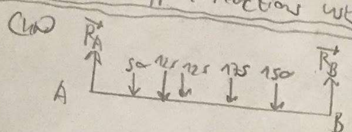
$4 = c + \frac{475x-3045}{L}\cdot(6{,}6)$

Case 5:

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



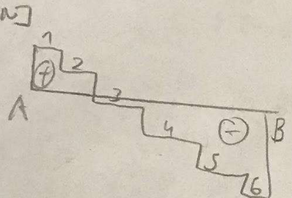
FBD and support reactions using superposition:



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

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

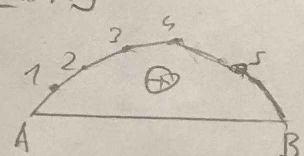
V : [kN]



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

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

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
```

```
        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))
```

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```
                  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);
%}
```

Chapter 4 - Design Loads

Table of Contents

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:

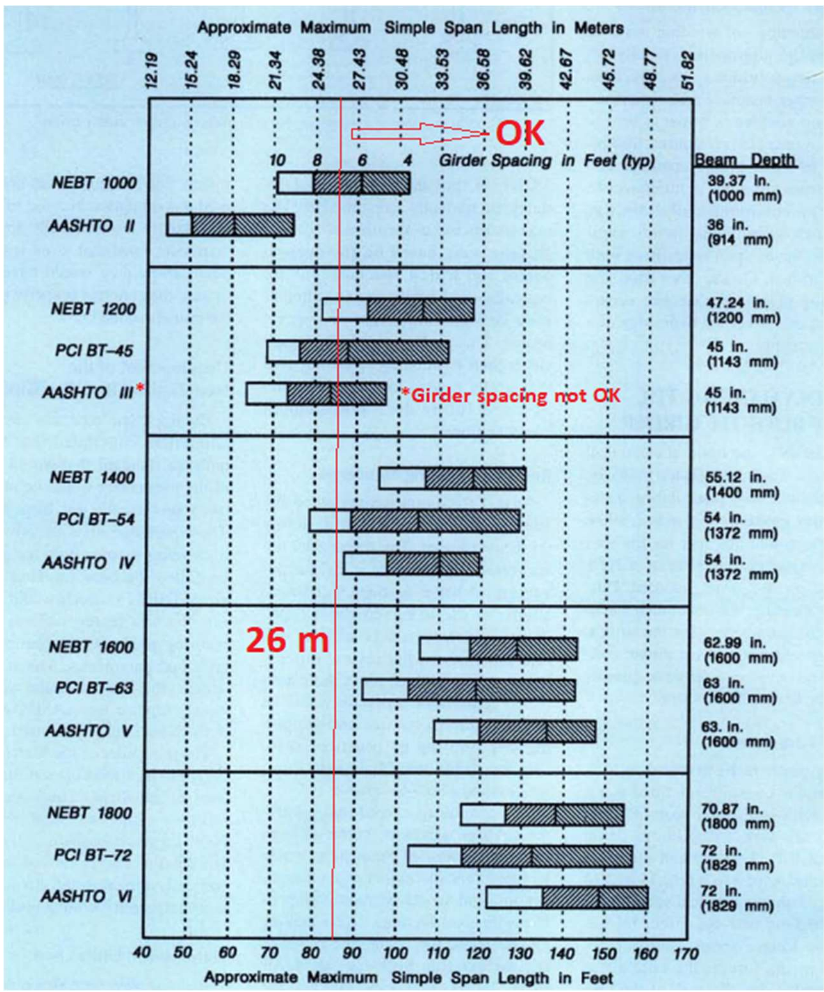
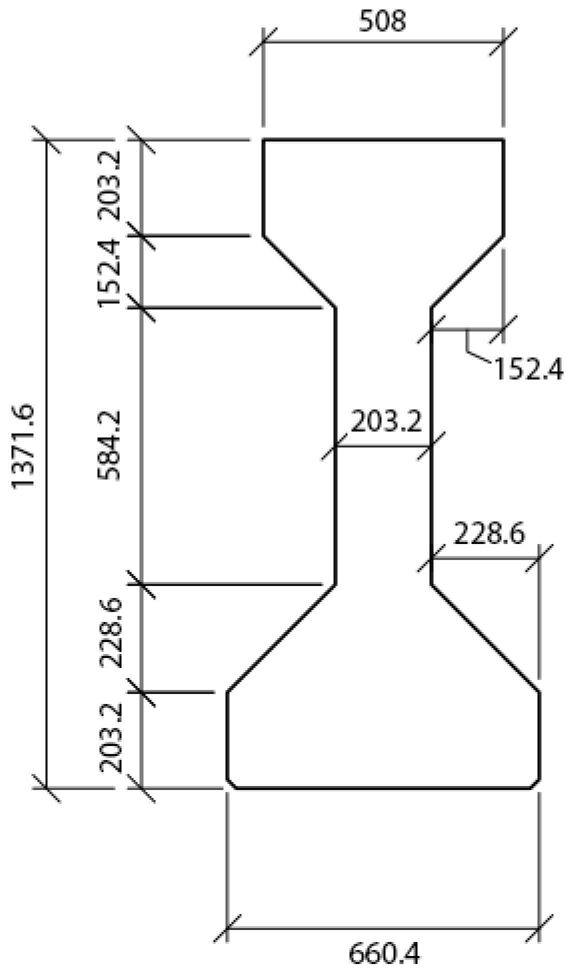


Figure 4.2.1 - Approximate Maximum Simple Span Length of Different Girders [2]

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:



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.

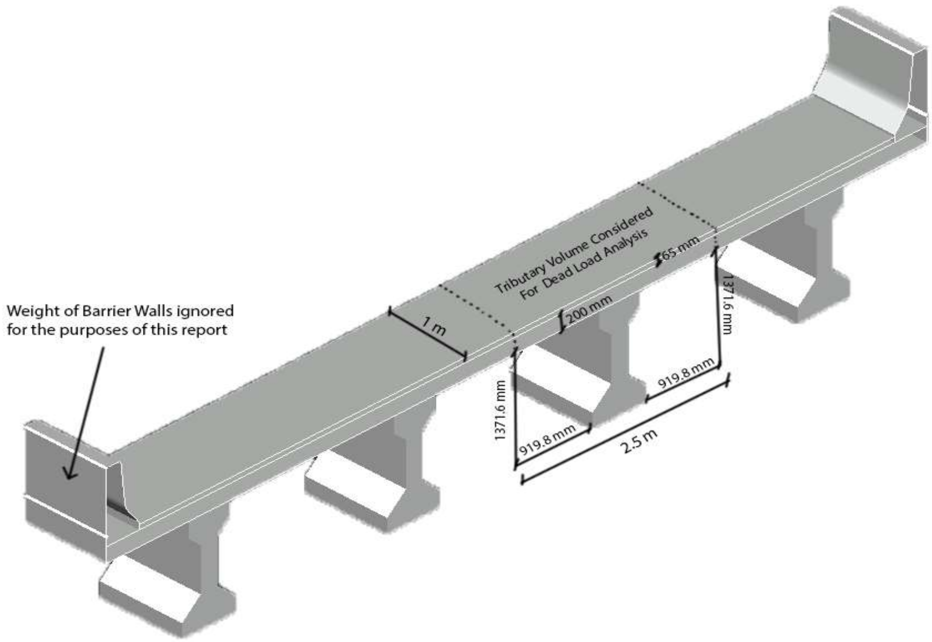


Figure 4.3.1.1 - Dead Load Analysis Model

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.

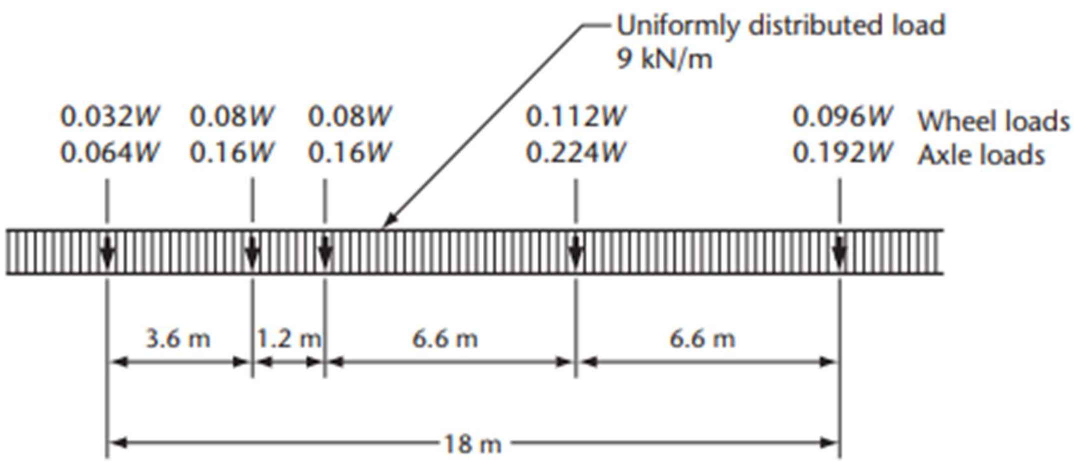


Figure 4.3.2.1 - CL-W Lane Load loading details [3]

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])

Lane Modification factor $\mu = \dfrac{W_e - 3.3}{0.6} = \dfrac{17}{6}$. 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	$4.60 - \dfrac{5.30}{\sqrt{L_e + 5}} \geq 2.80$	$0.10 - \dfrac{0.25}{L_e}$	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**

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

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	RL	Not Applicable	0.9
Truck Load Distribution Width for Shear	DT_V	Not Applicable	3.4
Truck Load Distribution Width for Moment	DT_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	FT_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	FT_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)

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	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

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

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

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

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

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

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

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)

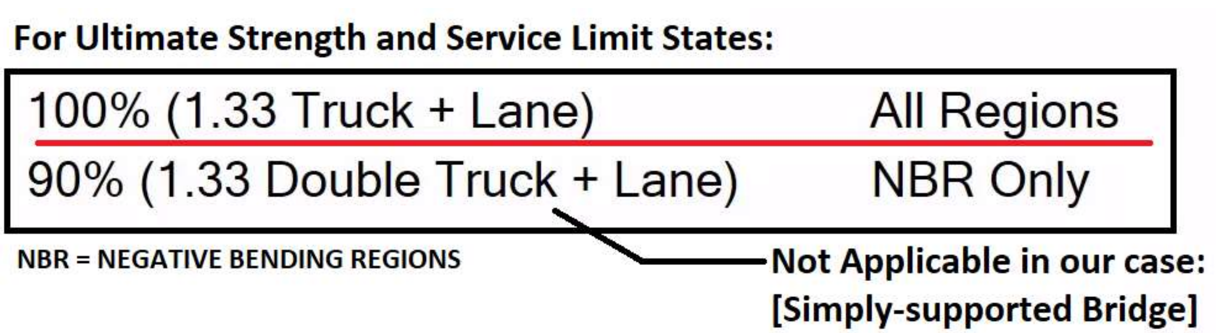
Table 4.4.2.1 (Right)- Undistributed Absolute Shear and Moment values for Lane Load

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.

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

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:

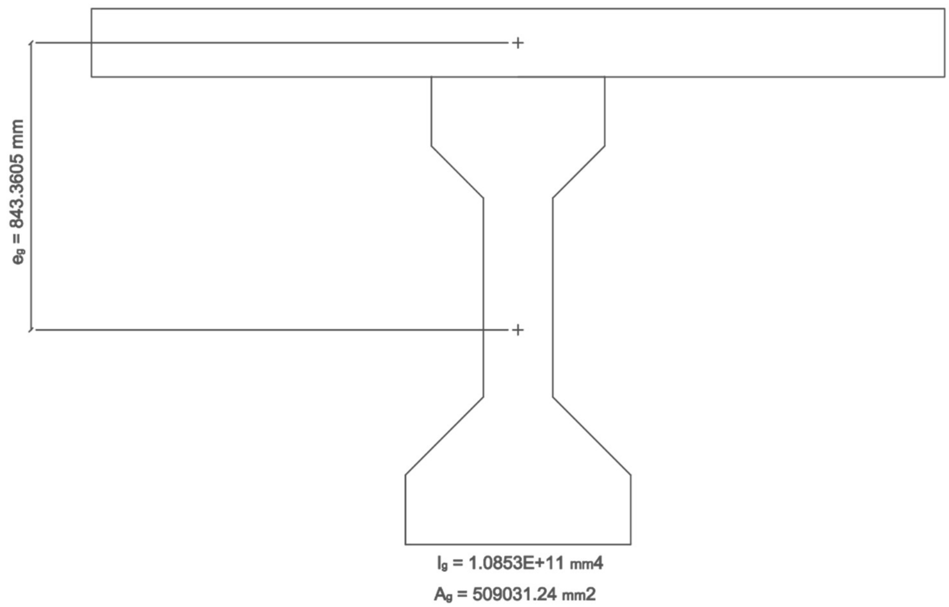
K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the owner
w_c = unit weight of concrete (kcf)
f'_c = compressive strength of concrete for use in design (ksi)

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

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]:

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]:

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].

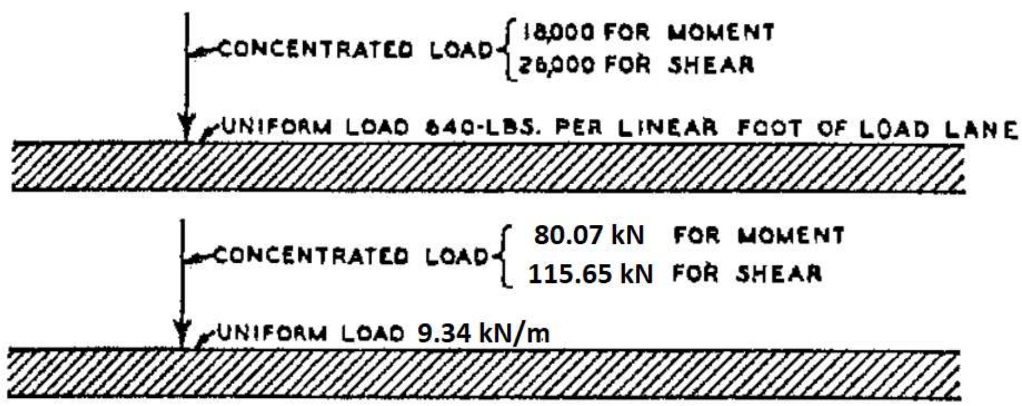


Table 4.5.2.1 (Left)- Undistributed Absolute Maximum Moment and Shear values for Truck Load (obtained from chapter 3)

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 pre-stressed 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		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

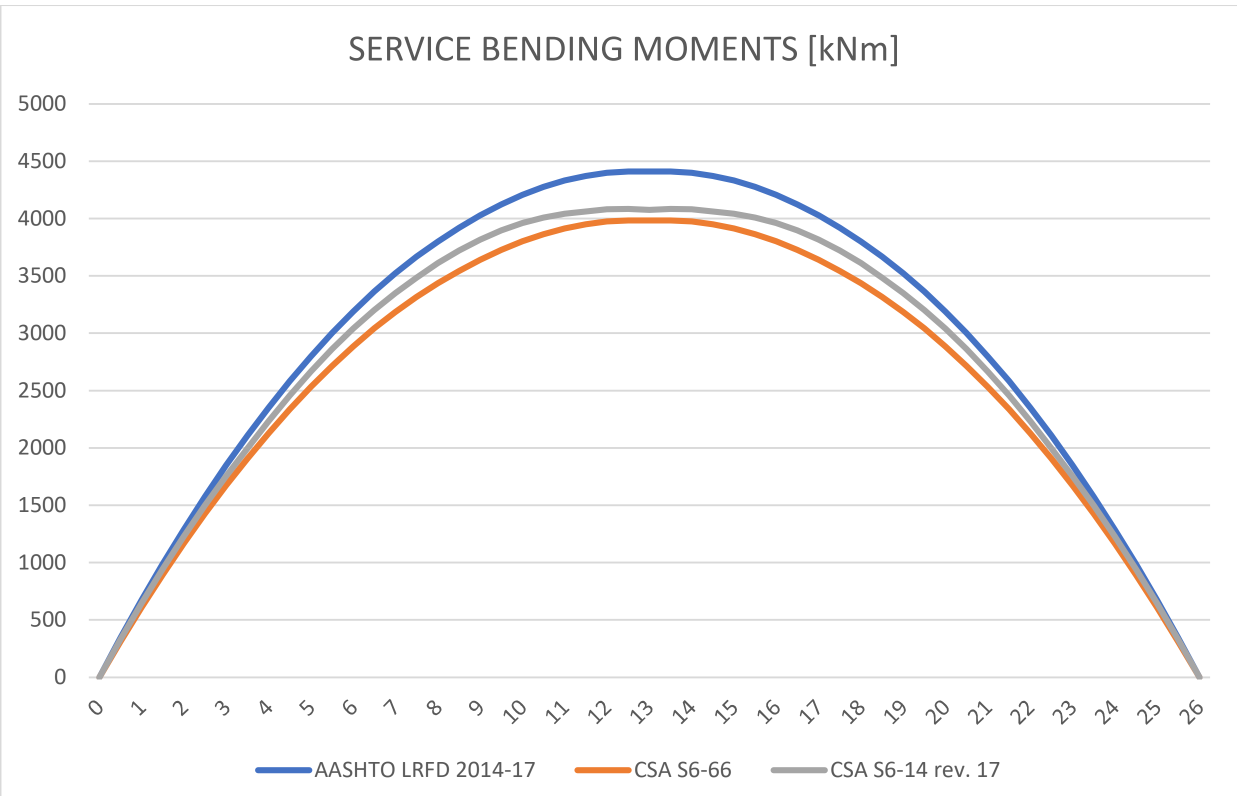


Figure 4.6.1 – Service Bending Moments – Graphical Results

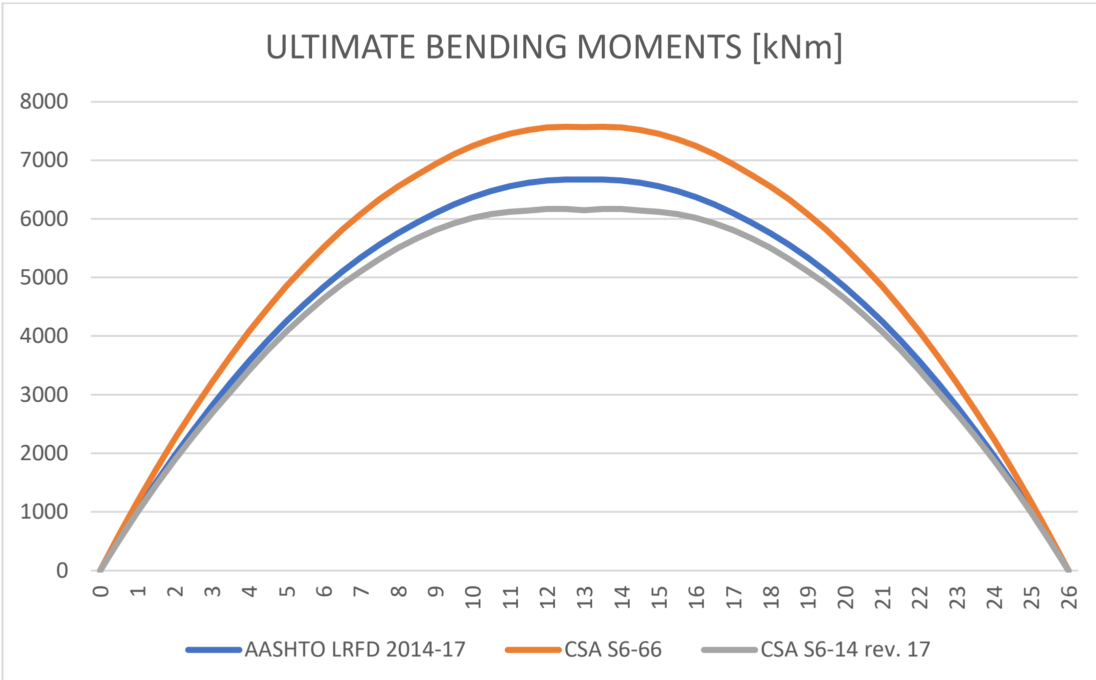


Figure 4.6.1 – Ultimate Bending Moments – Graphical Results

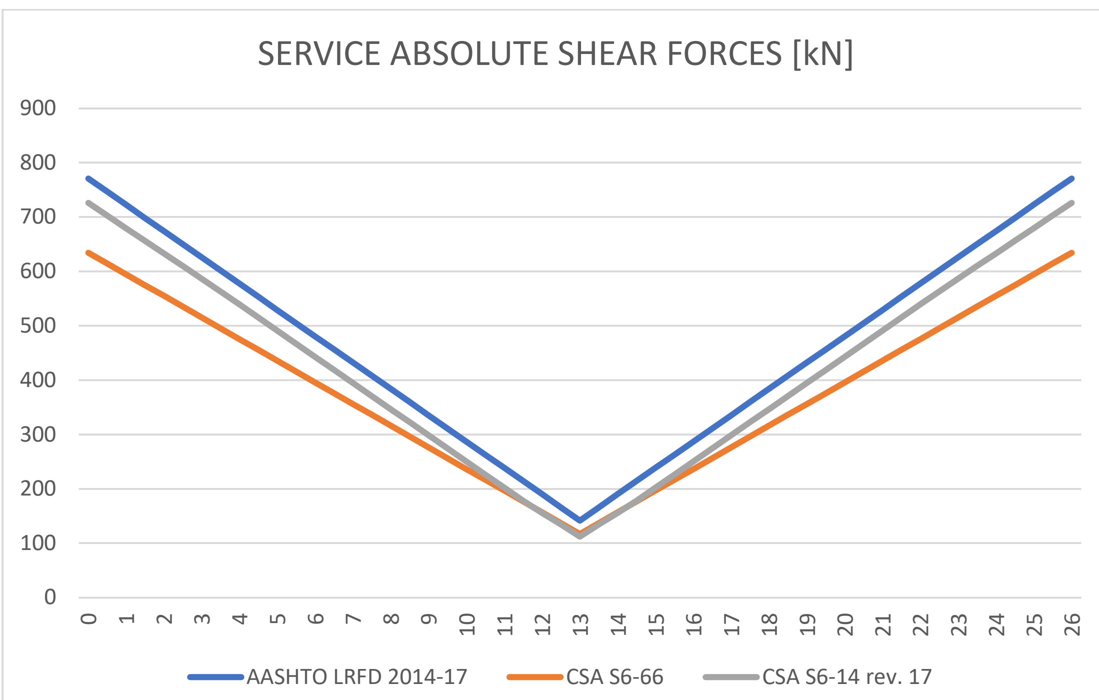


Figure 4.6.3 – Service Absolute Shear Forces – Graphical Results

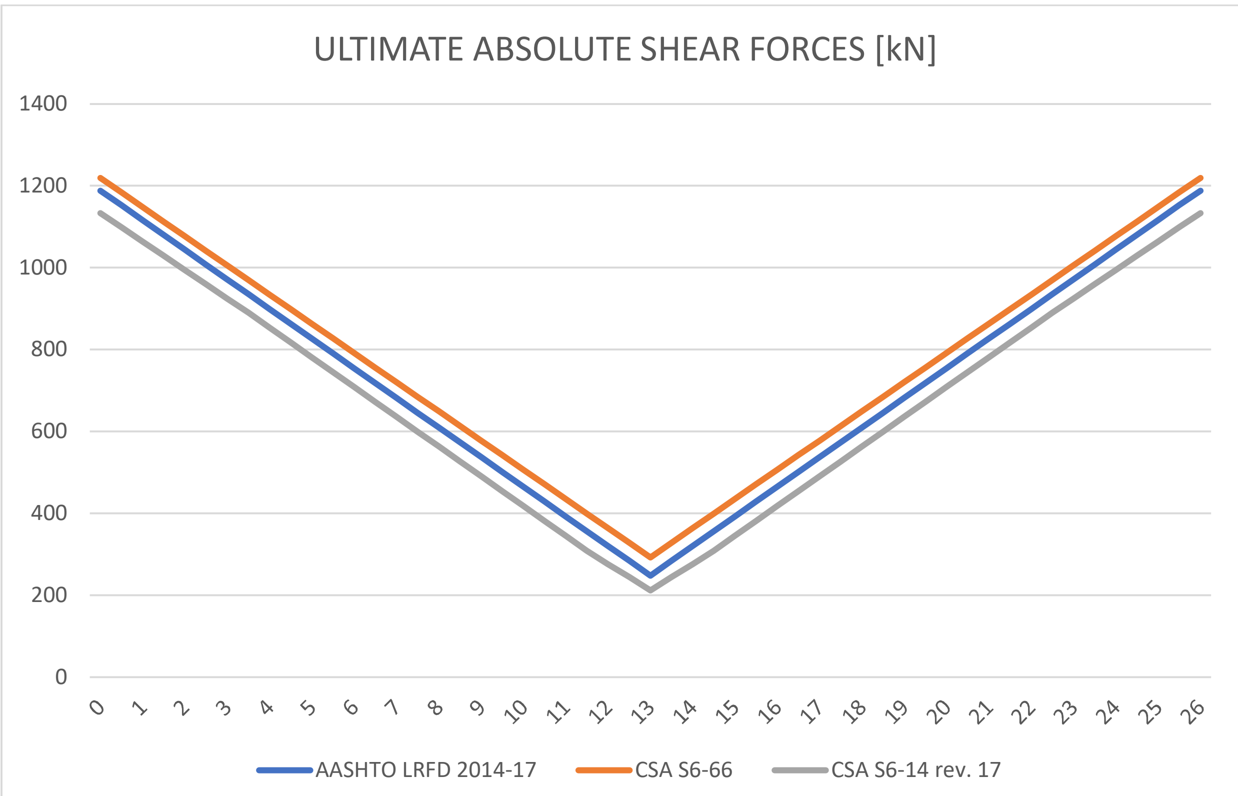


Figure 4.6.4 – Ultimate Absolute Shear Forces – Graphical Results

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.

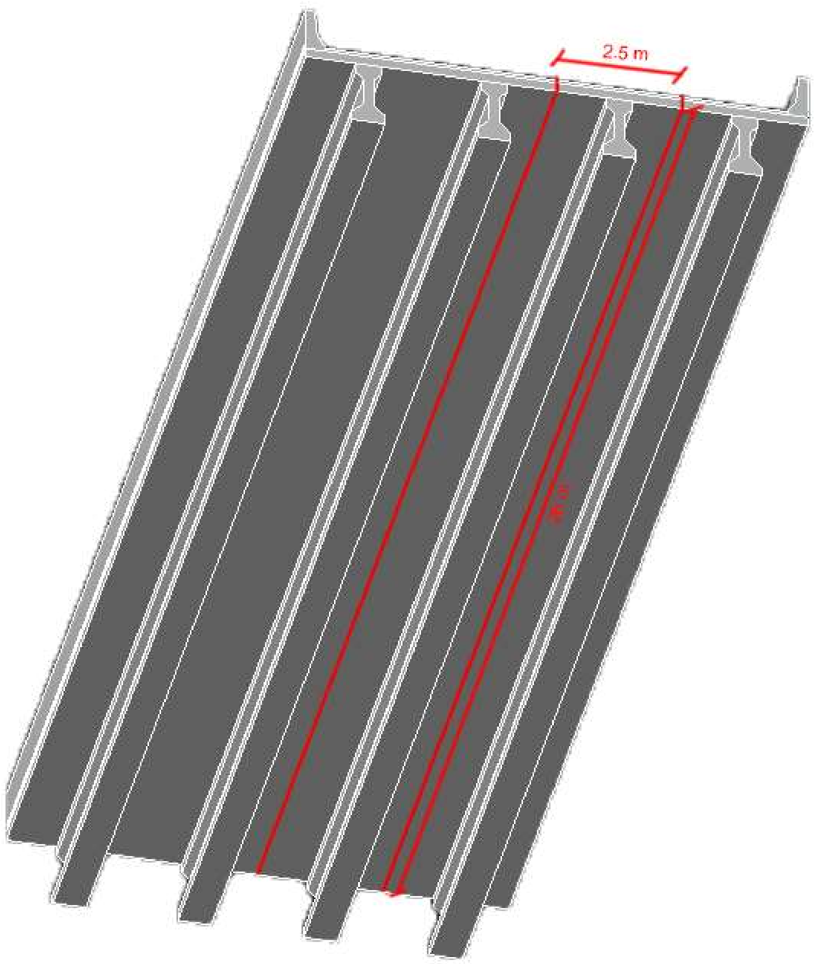


Figure 5.1.1 – 3D view of interior girder designed (girder between red longitudinal lines)

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)

$Yield\ Stress,\ f_y = 400\ MPa$

$strain\ at\ yield,\ \varepsilon_s = 0.002$

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

$Ultimate\ Stress,\ f_u = 550\ MPa$

$strain\ at\ ultimate,\ \varepsilon_u = 0.1$

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

where:

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

γ_c = *Concrete Density* [kg/m^3]

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

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

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

where:

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

γ_c = *Concrete Density* [kg/m^3]

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

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

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

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

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

where:

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

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

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

Section Properties of the composite section and the girder

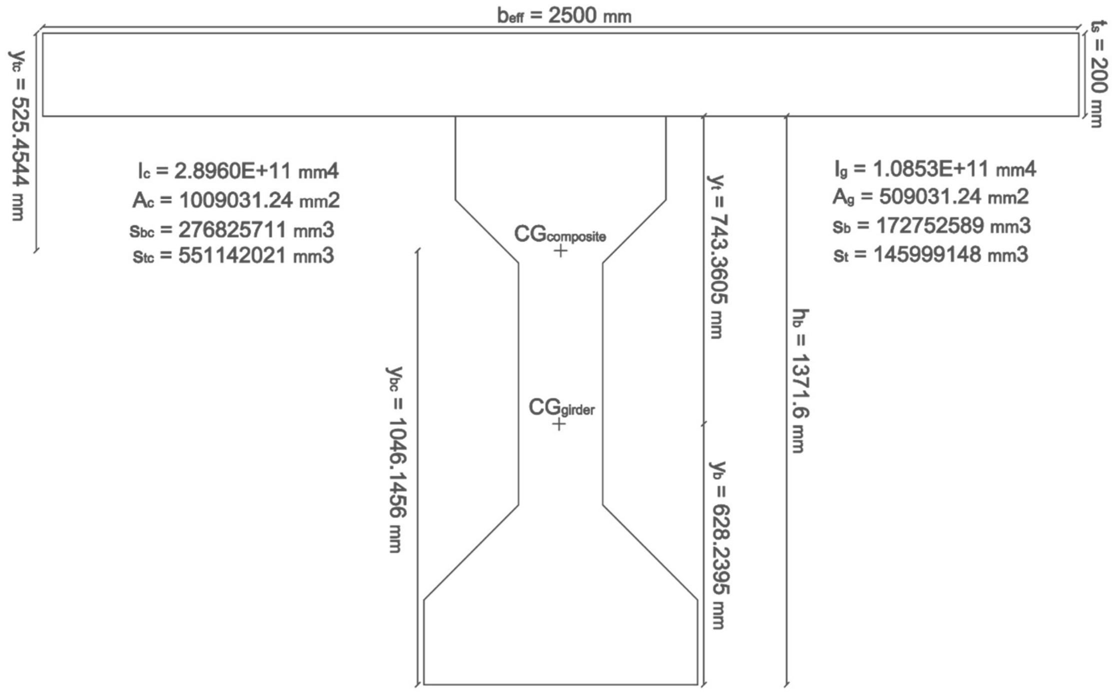


Figure 5.2.1 – Composite Girder Parameters

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:

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

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

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:

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

Required Number of Strands:

Required precompressive stress in the bottom fiber after losses:

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

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

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

Strand eccentricity at midspan:

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

Bottom fiber stress due to prestress after losses:

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

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

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

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

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

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

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

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

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

Effective strand eccentricity at midspan after strand arrangement

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

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

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

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

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

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

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

16.5635 MPa < 16.7739 MPa therefore NOT OK

Therefore use **32 strands**

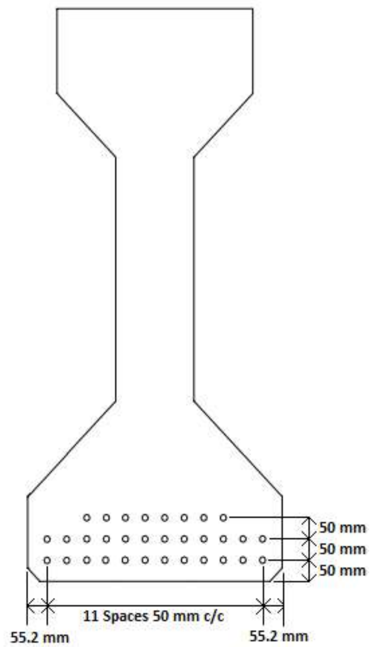


Figure 5.3.1.1 – Initial Strand Pattern

5.3.2 Prestressing Losses

Total prestress loss:

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

where

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

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

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

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

Elastic Shortening:

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

where:

$f_{\text{cir}} =$ *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*

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

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.

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

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

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

Using gross area here also gives acceptable results.

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

$I_{gt} =$ *Transformed moment of Inertia*

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

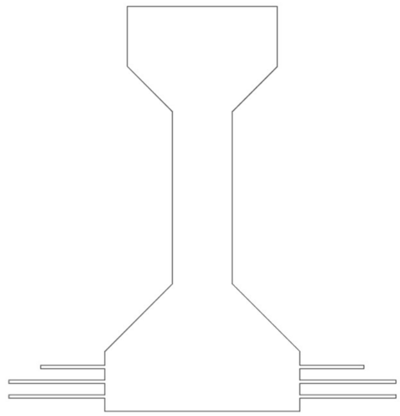


Figure 5.3.2.1 – Transformed Girder

$e_c = 534.4895\ mm$

$M_G = 1053.82\ kNm$

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

$fcir = 12.6533\ MPa$

$Eci = 27932\ MPa$

$Ep = 200000\ MPa$

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

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

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.*

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

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

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

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

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

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

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

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*

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

$7.7488\ \%$ ***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:

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

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

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

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

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

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

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

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

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.

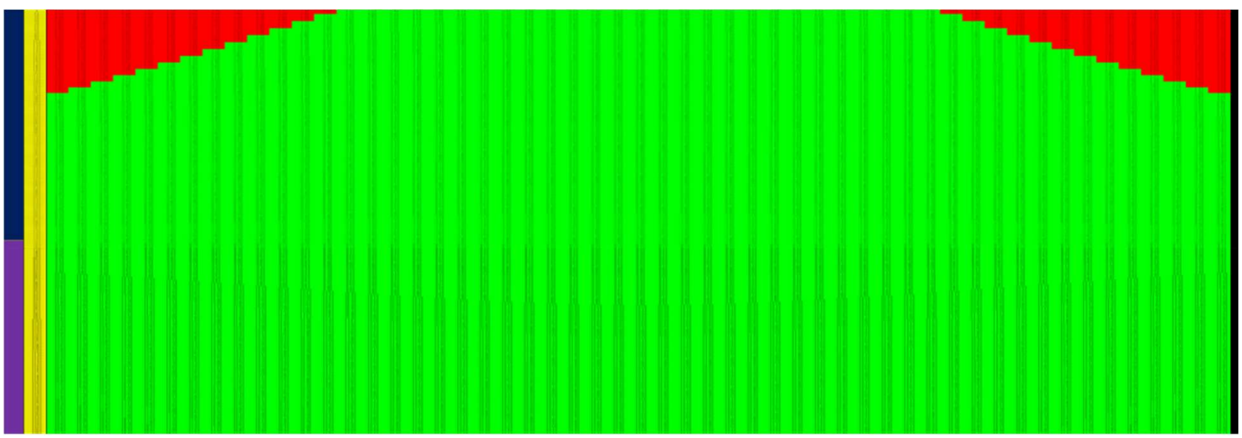


Figure 5.3.3.1.1 – Straight Strands – Stresses experienced

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):

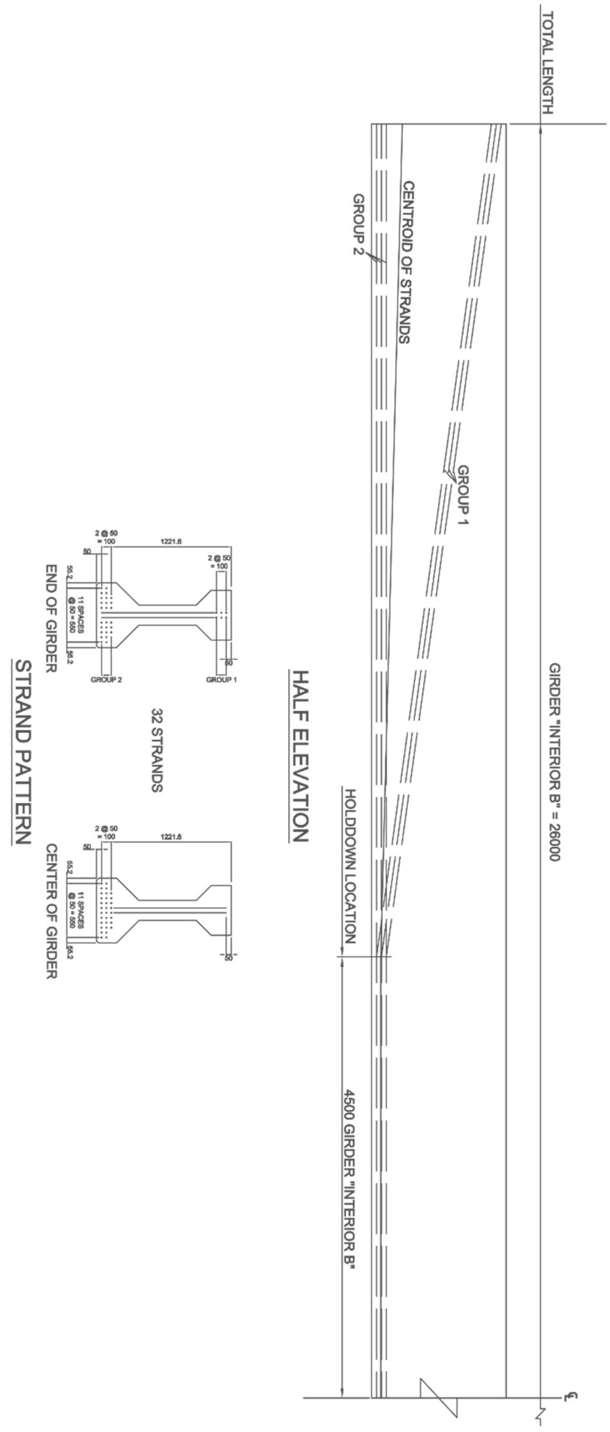


Figure 5.3.3.1.2 – Strand Pattern

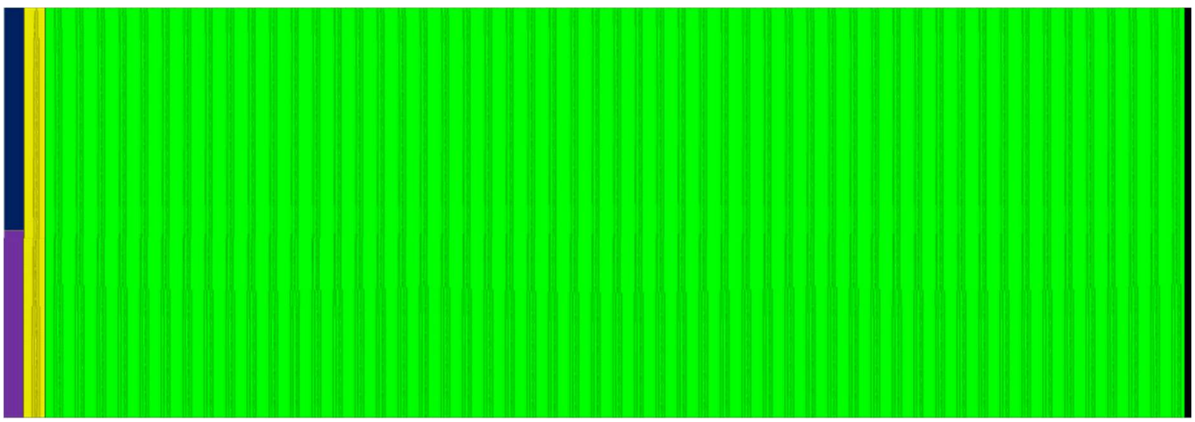


Figure 5.3.3.1.3 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced

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:

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

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

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

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

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

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

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

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

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

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

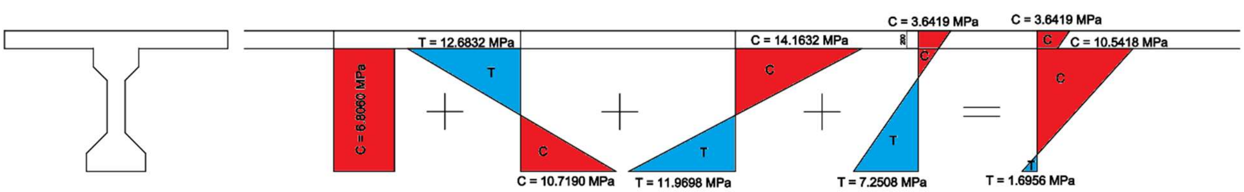


Figure 5.3.3.2.1 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced visualized at midspan

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.

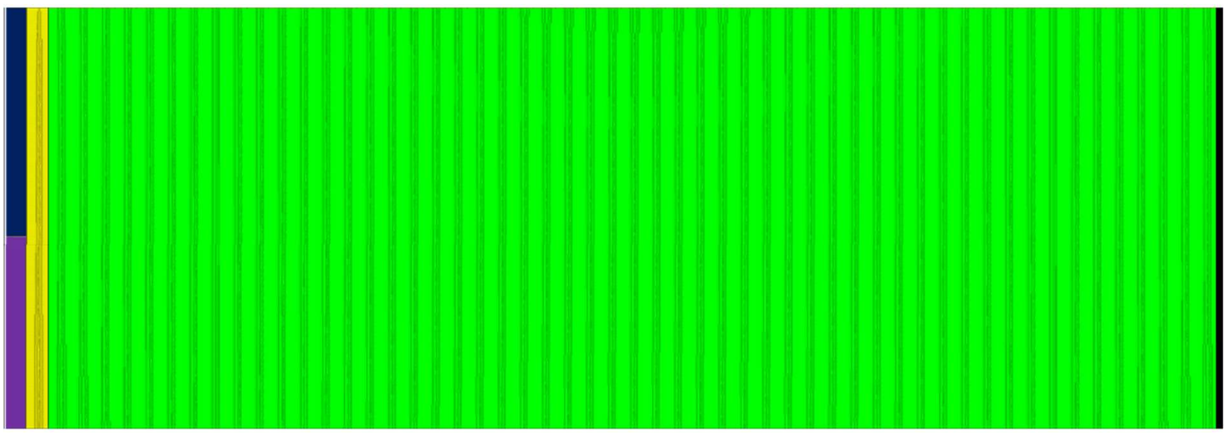


Figure 5.3.3.2.2 – Harped Strands with groups as in figure 5.3.3.1.2 – Stresses experienced at service conditions

Maximum stresses recorded at service are:

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

$13.11\,MPa \text{ for compression } < 0.45 \times 40\ (18\,MPa)\ OK$

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

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.

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:

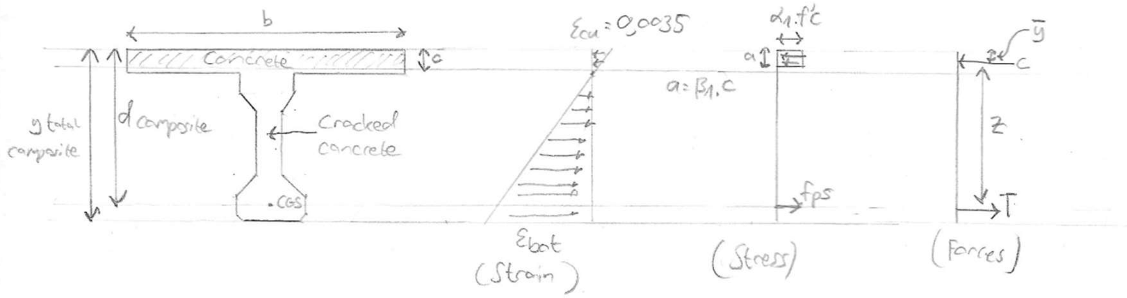
Stress block parameters:

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

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

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

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

$a = 0,04 \cdot f_{ps} = 0,04 \cdot 1830 = 73,2$ mm $\qquad a < t_s$ (200 mm) ∴ Rectangular assumption can be used.

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

$$\boxed{M_r = 8330 \text{ kNm} > 6147,31 \text{ kNm} \quad \text{ok}}$$

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 $\quad a < t_s$ (200 mm) ∴ Rectangular Assumption can be used.

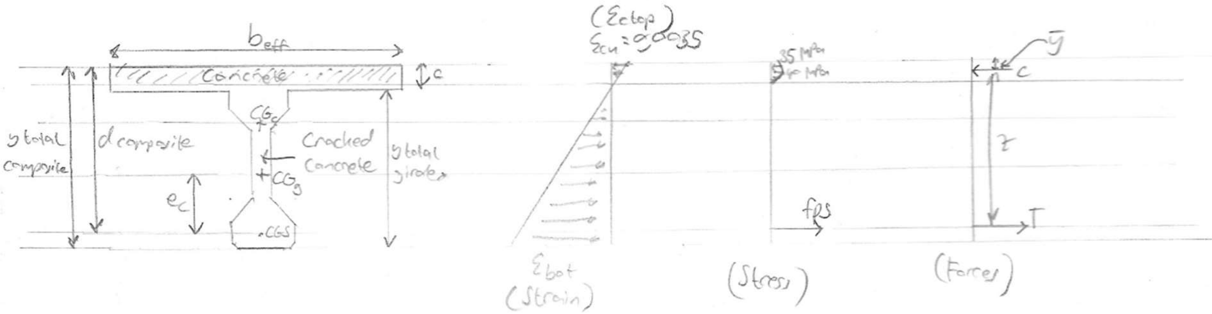
$$M_r = A_{ps} \cdot f_{ps} \cdot \left(d - \frac{a}{2}\right) \cdot \phi_p = 98{,}7 \cdot 32 \cdot 1822 \cdot \left(1477{,}8 - \frac{92{,}3}{2}\right) \cdot 0{,}95 \cdot 10^{-6}$$

$$\boxed{M_r = 7827 \text{ kNm} > 6147{,}31 \text{ kNm OK}}$$

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

$E_c = \left(3000 \cdot \sqrt{40} + 6900\right)\left(\frac{2500}{2300}\right)^{1{,}5} = 29321$ 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

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

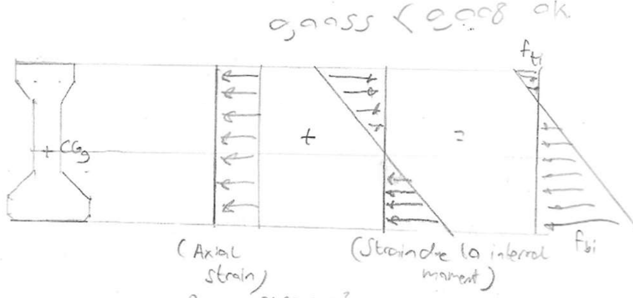
$\varepsilon_o = 0{,}002$

$f'_c = 40$ 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



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

$$\varepsilon_{ti_p} = -\frac{P_{pe}}{A_{gt}.E_{ps}} + \frac{P_{pe}.e_c.y_t}{I_{gt}.E_{ps}} = -3,2777.10^{-5} + \frac{3464,5.10^3.534,4895.743,3605}{1,1391.10^{11}.200000} = 2,7643.10^{-5}$$

$$\varepsilon_{bi_p} = -\frac{P_{pe}}{A_{gt}.E_{ps}} - \frac{P_{pe}.e_c.y_b}{I_{gt}.E_{ps}} = -3,2777.10^{-5} - \frac{3464,5.10^3.534,4895.628,2395}{1,1391.10^{11}.200000} = -8,3841.10^{-5}$$

$$\varepsilon_{ci_p} = \varepsilon_{ti_p} + (\varepsilon_{bi_p} - \varepsilon_{ti_p}).d.\frac{1}{y_{total}} = 2,7643.10^{-5} + (-8,3841.10^{-5} - 2,7643.10^{-5}).1277,8.\frac{1}{1371,6} = -7,6221.10^{-5}$$

Assuming unshored construction

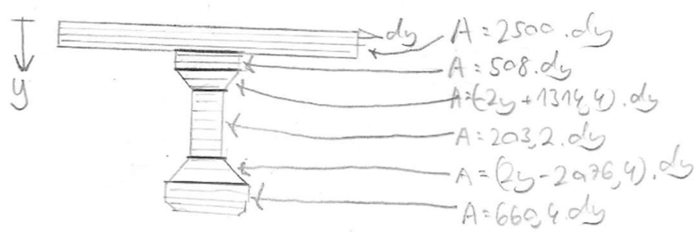
$$\varepsilon_{ti_{DL}} = -\frac{(M_G + M_S).y_t}{I_{gt}.E_{ps}} = -\frac{(1053,82+1014).10^6.743,3605}{1,1391.10^{11}.200000} = -6,7472.10^{-5}$$

$$\varepsilon_{bi_{DL}} = \frac{(M_G + M_S).y_b}{I_{gt}.E_{ps}} = \frac{(1053,82+1014).10^6.628,2395}{1,1391.10^{11}.200000} = 5,7023.10^{-5}$$

$$\varepsilon_{ci_{DL}} = \frac{\overbrace{d-y_t}^{e_c}}{y_b}.\varepsilon_{bi_{DL}} = \frac{534,4895}{628,2395}.5,7023.10^{-5} = 4,8113.10^{-5}$$

$$\varepsilon_{ci} = |\varepsilon_{ci_p}| + \varepsilon_{c_{DL}} = 1,2473.10^{-4}$$

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



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

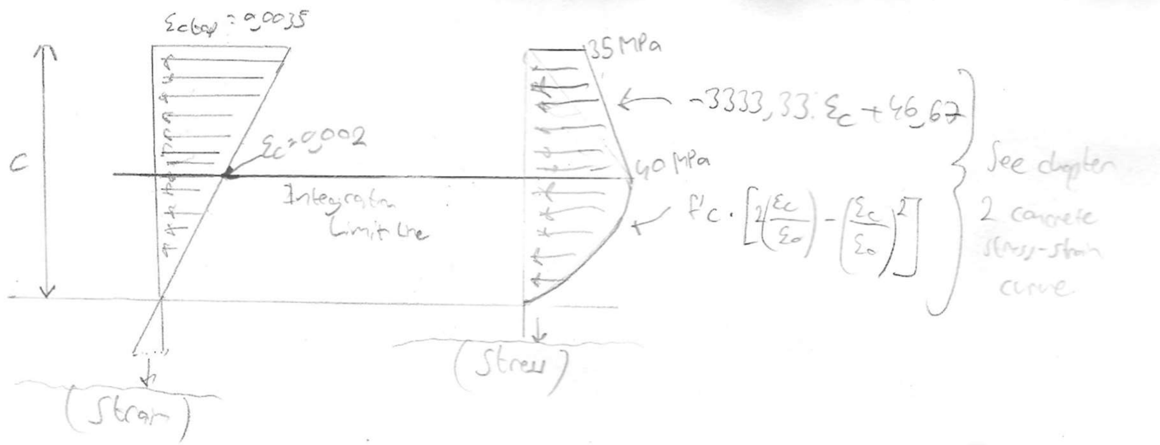
$$\varepsilon_s = 0,0055 + 1,2473.10^{-4} + 0,0035.\frac{1477,8 - c}{c}$$

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

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

$$f_{ps} = \alpha \quad \text{for } f_{ps} > 0,99.f_{pu}$$

*See chapter 2 Prestressing Steel Stress-Strain curve



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

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

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

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

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

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

Let $c = 74,26$ mm

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

$$\varepsilon_s > 0,008$$

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

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

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

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

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

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

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

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

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

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

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

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.

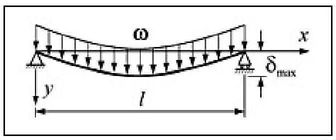
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED BRIDGE DEFLECTION AND MAXIMUM DEFLECTION	
	$y=\frac{\omega x}{24EI}\left(l^3-2lx^2+x^3\right)$	$\delta_{max}=\frac{5\omega l^4}{384EI}$

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

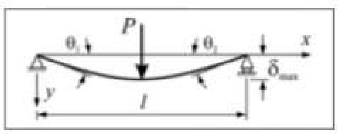
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED DEFLECTION AND MAXIMUM DEFLECTION	
	$y=\frac{Px}{12EI}\left(\frac{3l^2}{4}-x^2\right)$ for $0<x<\frac{l}{2}$	$\delta_{max}=\frac{Pl^3}{48EI}$

Figure 5.3.6.2 – 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

$e_e = 628.2395 - 313.425 = 314.8145\ mm$

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

$\boldsymbol{\Delta_c = 58\ mm\ upwards}$

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

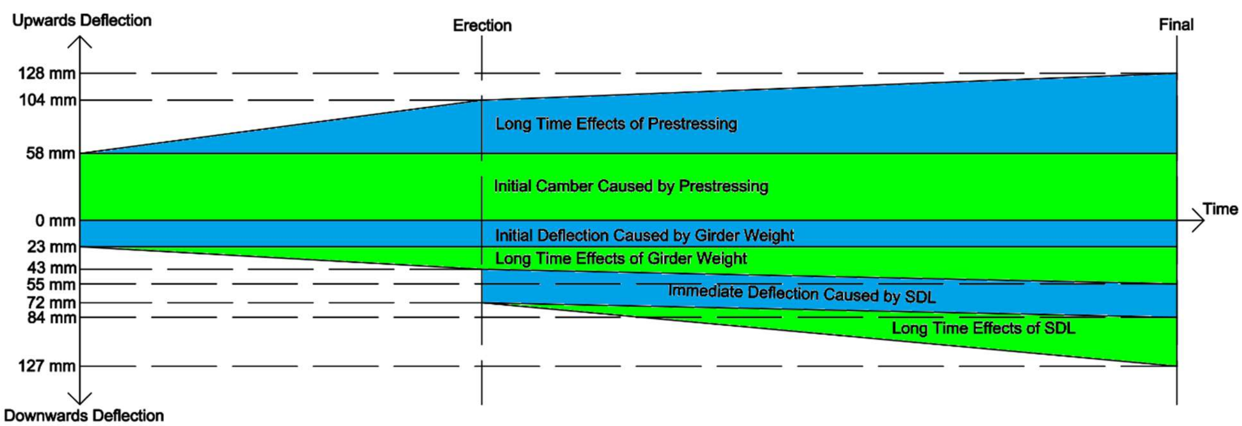


Figure 5.3.6.3 – Visual Representation of the Deflections Experienced

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>

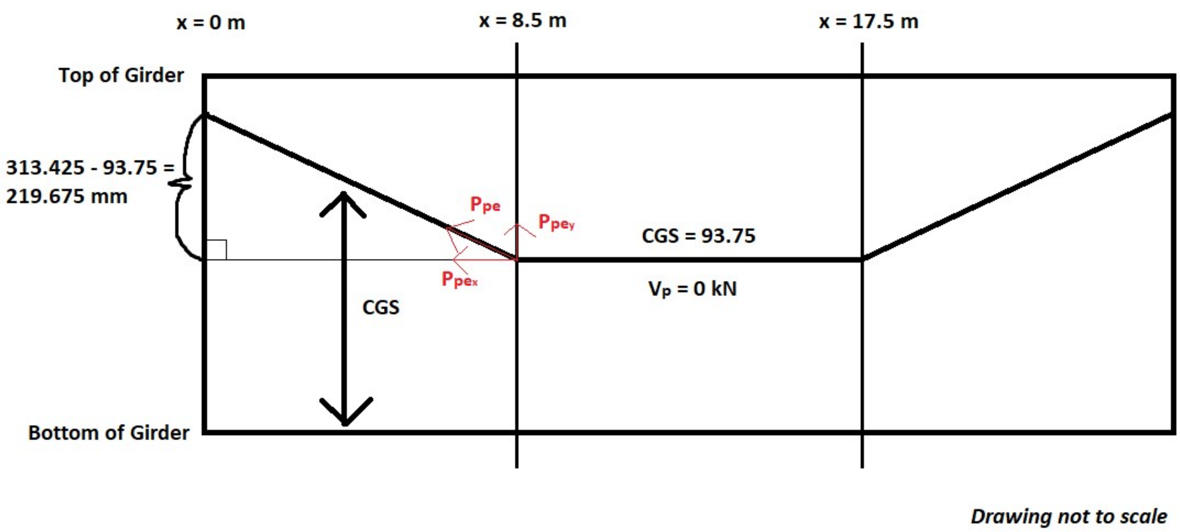


Figure 5.3.7.1 – Calculation of V_p

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):

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:

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

where:

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

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

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

At midspan:

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

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

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

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

At midspan:

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

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

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:

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

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

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

At midspan:

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

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:

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

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

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 0.25×1371.6.

$A_{vrequired} = 1292\ mm^2$ *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:

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

<u>Determination of V_c required and checking it against V_c available:</u>

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

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

At midspan:

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

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

<u>Forces in strands compared with force at ultimate design for flexure:</u>

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

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

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

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	Constructibility	457.83	547.38	160.94	OK	4845.88	5811.93
6.5	205.95	376.49	626.37	600	115.66	525	Constructibility	449.19	538.74	133.26	OK	4958.55	5811.93
7	195.19	350.93	663.01	600	115.66	525	Constructibility	443.18	532.73	102.94	OK	5049.30	5811.93
7.5	187.37	322.43	714.62	600	115.66	525	Constructibility	438.88	528.43	70.91	OK	5125.36	5811.93
8	181.62	291.85	784.27	600	115.66	525	Constructibility	435.98	525.52	37.49	OK	5188.65	5811.93
8.5	171.03	355.66	630.85	600	115.66	525	Constructibility	427.36	427.36	99.32	OK	5350.91	5811.93
9	164.56	325.80	676.83	600	115.66	525	Constructibility	420.02	420.02	70.34	OK	5422.32	5811.93
9.5	160.03	294.01	740.39	600	115.66	525	Constructibility	414.63	414.63	39.40	OK	5473.92	5811.93
10	157.15	260.55	828.34	600	115.66	525	Constructibility	411.10	411.10	6.61	OK	5505.13	5811.93
10.5	155.77	225.61	952.60	600	115.66	525	Constructibility	409.36	409.36	0	OK	5515.37	5811.93
11	155.80	189.25	1135.73	600	115.66	525	Constructibility	409.40	409.40	0	OK	5504.05	5811.93
11.5	156.96	151.76	1421.30	600	115.66	525	Constructibility	410.86	410.86	0	OK	5475.04	5811.93
12	157.59	118.50	1823.67	600	115.66	525	Constructibility	411.64	411.64	0	OK	5455.35	5811.93
12.5	159.63	84.65	2568.66	600	115.66	525	Constructibility	414.14	414.14	0	OK	5414.05	5811.93
13	163.25	49.21	4464.75	600	115.66	525	Constructibility	418.48	418.48	0	OK	5349.45	5811.93
13.5	159.63	84.65	2568.66	600	115.66	525	Constructibility	414.14	414.14	0	OK	5414.05	5811.93
14	157.59	118.50	1823.67	600	115.66	525	Constructibility	411.64	411.64	0	OK	5455.35	5811.93
14.5	156.96	151.76	1421.30	600	115.66	525	Constructibility	410.86	410.86	0	OK	5475.04	5811.93
15	155.80	189.25	1135.73	600	115.66	525	Constructibility	409.40	409.40	0	OK	5504.05	5811.93
15.5	155.77	225.61	952.60	600	115.66	525	Constructibility	409.36	409.36	0	OK	5515.37	5811.93
16	157.15	260.55	828.34	600	115.66	525	Constructibility	411.10	411.10	6.61	OK	5505.13	5811.93
16.5	160.03	294.01	740.39	600	115.66	525	Constructibility	414.63	414.63	39.40	OK	5473.92	5811.93
17	164.56	325.80	676.83	600	115.66	525	Constructibility	420.02	420.02	70.34	OK	5422.32	5811.93
17.5	171.03	355.66	630.85	600	115.66	525	Constructibility	427.36	427.36	99.32	OK	5350.91	5811.93
18	181.62	291.85	784.27	600	115.66	525	Constructibility	435.98	525.52	37.49	OK	5188.65	5811.93
18.5	187.37	322.43	714.62	600	115.66	525	Constructibility	438.88	528.43	70.91	OK	5125.36	5811.93
19	195.19	350.93	663.01	600	115.66	525	Constructibility	443.18	532.73	102.94	OK	5049.30	5811.93
19.5	205.95	376.49	626.37	600	115.66	525	Constructibility	449.19	538.74	133.26	OK	4958.55	5811.93
20	221.79	396.99	605.46	600	115.66	525	Constructibility	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

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 $= 0.2 \times 1395 =$ **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

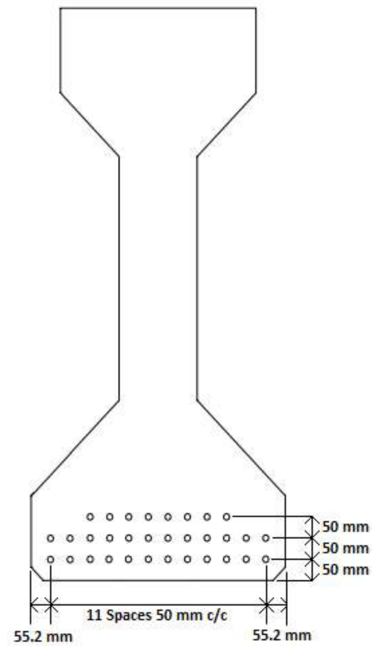


Figure 5.4.1.1 – Initial Strand Pattern

5.4.2 Prestressing Losses

Total prestress loss:

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

where

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*:**

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

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*

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

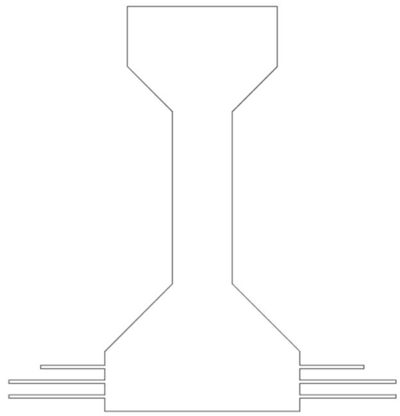


Figure 5.4.2.1 – Transformed Girder

$e_c = 534.4895\ mm$

$M_G = 985.16\ kNm$

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

$E_{ci} = 31800\ MPa$

$E_p = 200000\ MPa$

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

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

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.

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

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

$\Delta f_{pR1} = \Delta f_{pR2} = 17\ MPa$ **for low – relaxation steel**

$\gamma_h = 1.7 - 0.01 \times H$ *where H 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:

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

5.4.3 Concrete stress limits at top and bottom

5.4.3.1 Stress limits at transfer and Strand Pattern

$$\textbf{\textit{At Midspan:}}$$

$$\textbf{\textit{At transfer, the tensile stress in the top fiber connot exceed:}}$$

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

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

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

$$\textbf{\textit{At transfer, the compressive stress in the bottom fiber cannot exceed:}}$$

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

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

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

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.

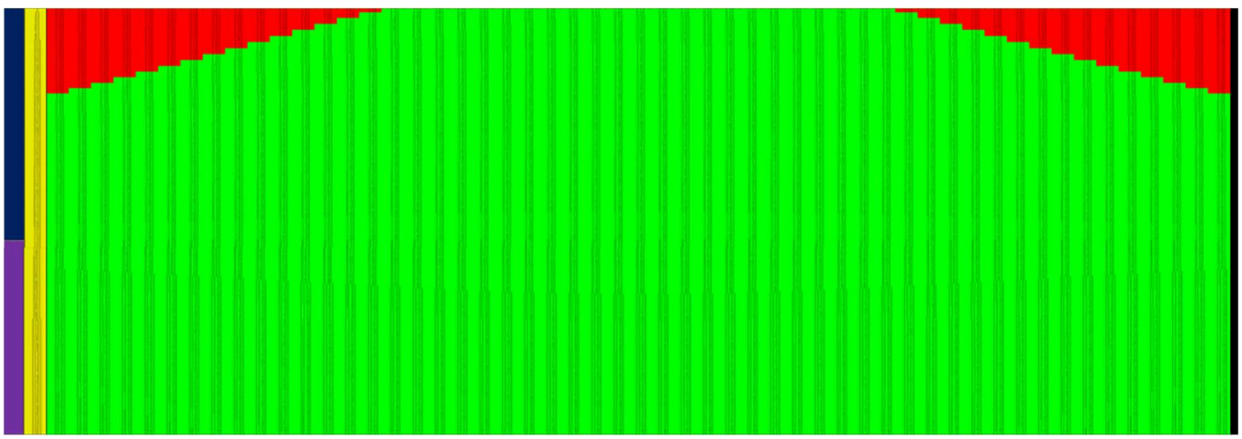


Figure 5.4.3.1.1 – Straight Strands – Stresses experienced

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):

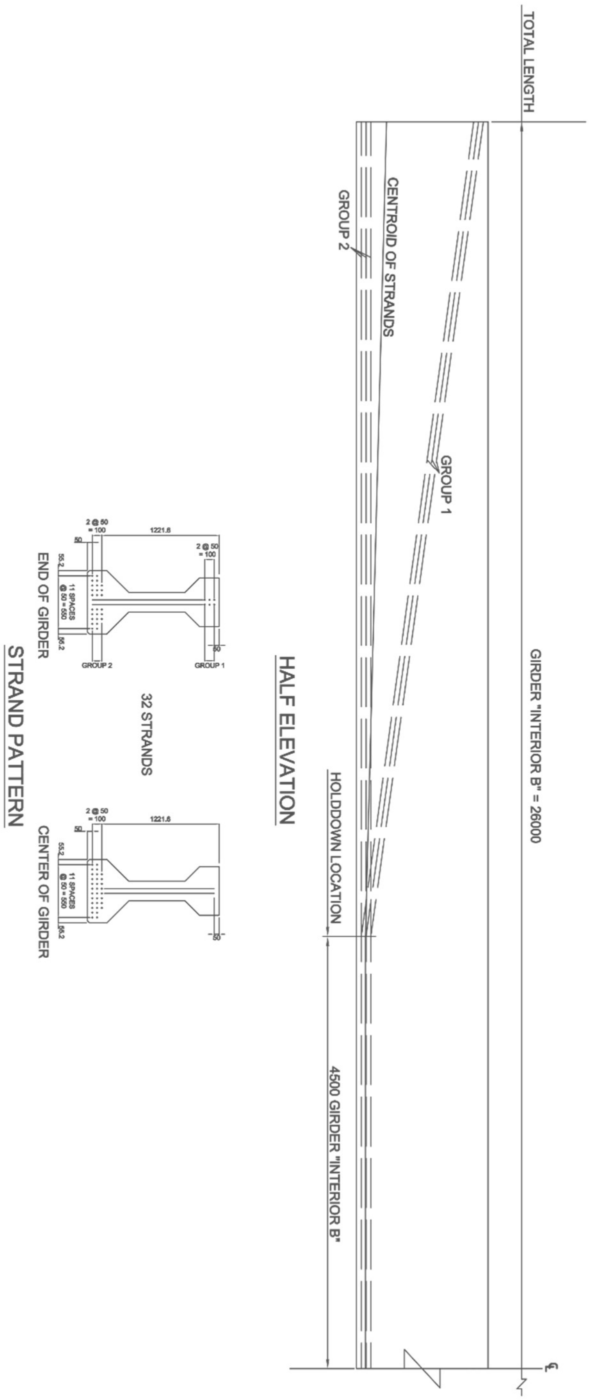


Figure 5.4.3.1.2 – Strand Pattern

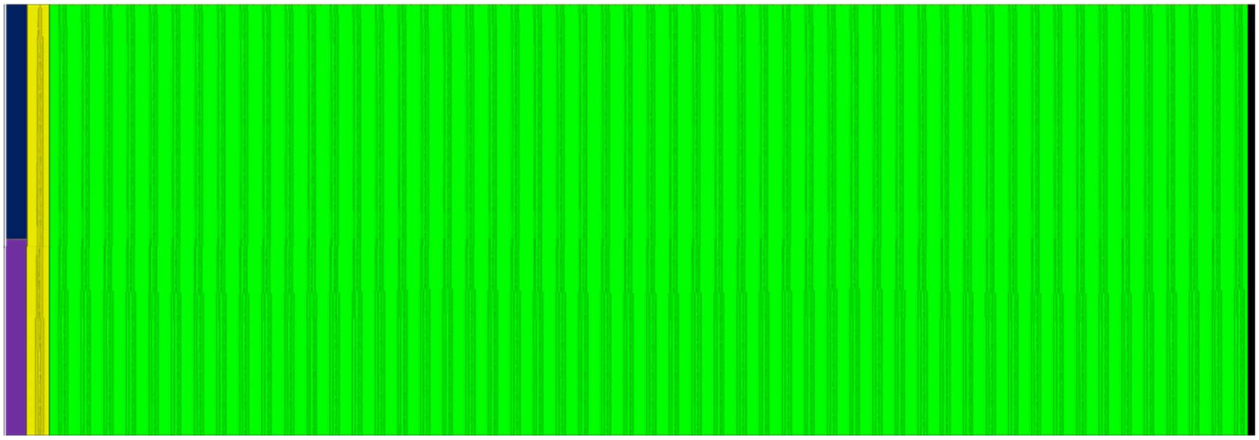


Figure 5.4.3.1.3 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced

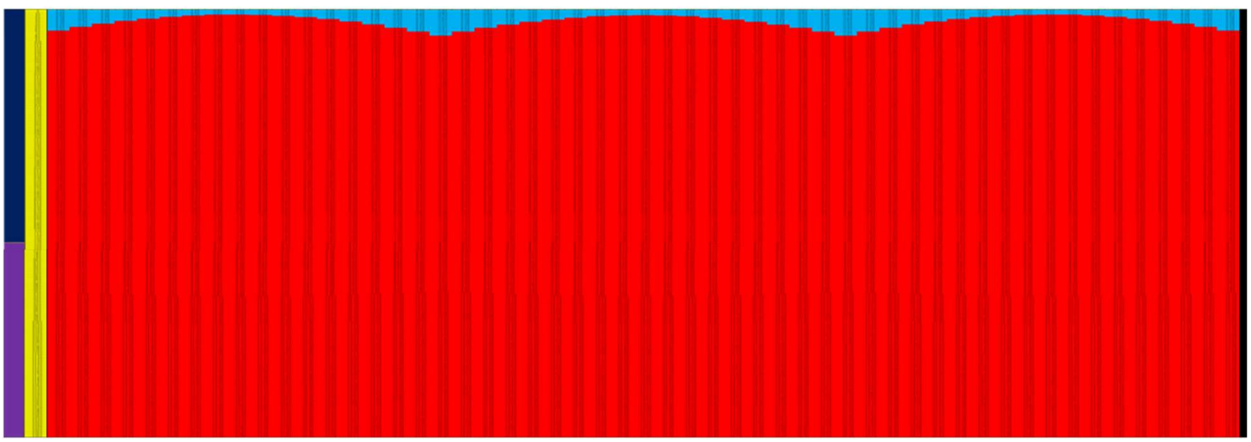


Figure 5.4.3.1.4 – Harped Strands with groups as in figure 5.4.3.1.2 – Elements in tension Blue, Elements in compression Red

Maximum stresses recorded at transfer are:

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

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

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:

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

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

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

$$f_{ts} = \frac{3545.2 \times 10^3}{509031} - \frac{3545.2 \times 10^3 \times 534.4895}{1.46 \times 10^8} + \frac{(985.16 + 962.86) \times 10^6}{1.46 \times 10^8}$$

$$+ \frac{(301.98 + 2160.66) \times 10^6}{\dfrac{2.896 \times 10^{11}}{(525.4544 - 200)}} = \mathbf{10.0962}\ MPa\ OK$$

At service, the tensile stress in the bottom fiber cannot exceed:

$$f_{bs} = 0.5 \times \sqrt{40} = 3.162\ MPa$$

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

$$f_{bs} = -\frac{3545.2 \times 10^3}{509031} - \frac{3545.2 \times 10^3 \times 534.4895}{1.7275 \times 10^8} + \frac{(985.16 + 962.86) \times 10^6}{1.7275 \times 10^8}$$

$$+ \frac{(301.98 + 2160.66) \times 10^6}{2.7683 \times 10^8} = 2.239\ MPa\ OK$$

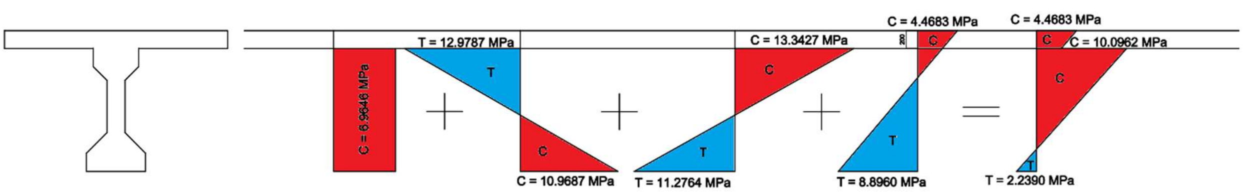


Figure 5.4.3.2.1 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced visualized at midspan

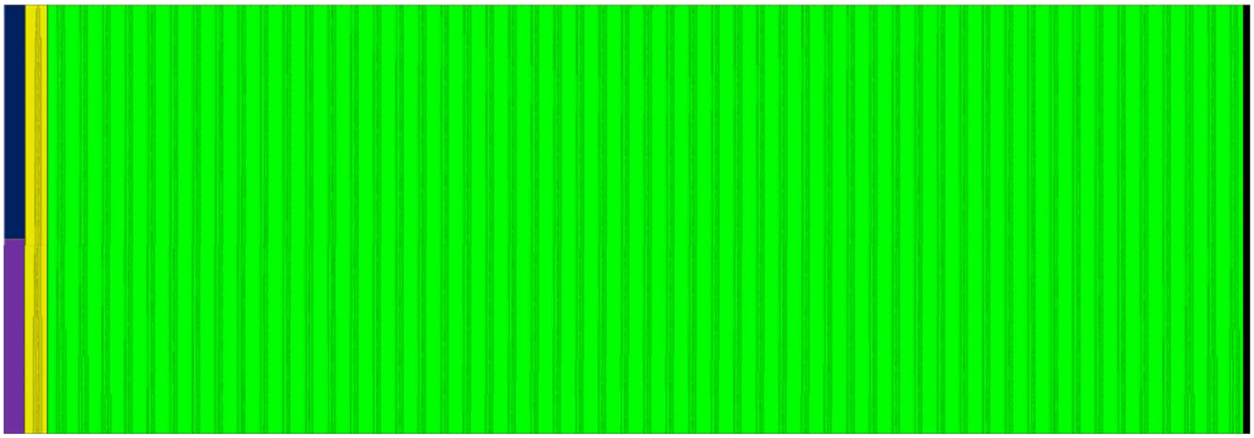


Figure 5.4.3.2.2 – Harped Strands with groups as in figure 5.4.3.1.2 – Stresses experienced at service conditions

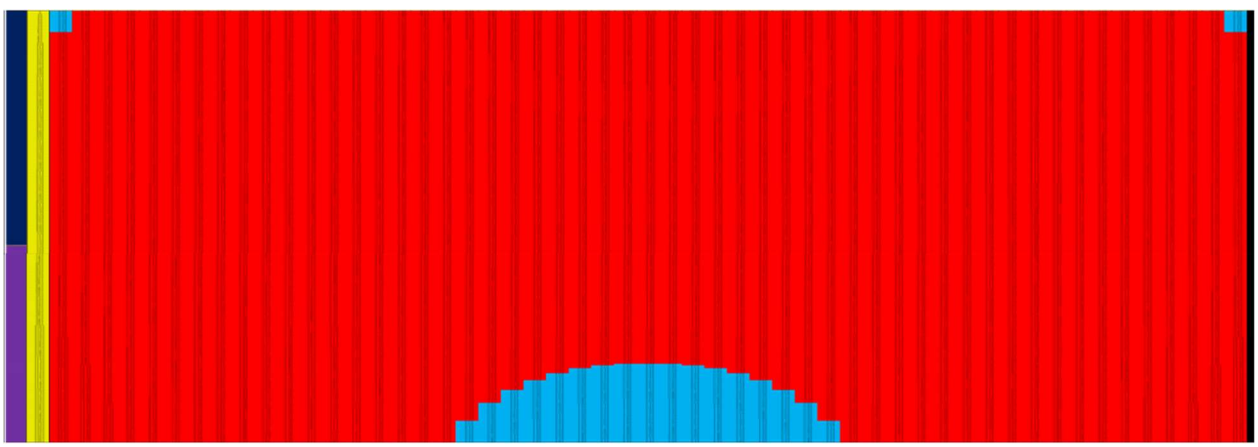


Figure 5.4.3.2.3 – Harped Strands with groups as in figure 5.4.3.1.2 – Elements in tension Blue, Elements in compression Red – Service Conditions

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

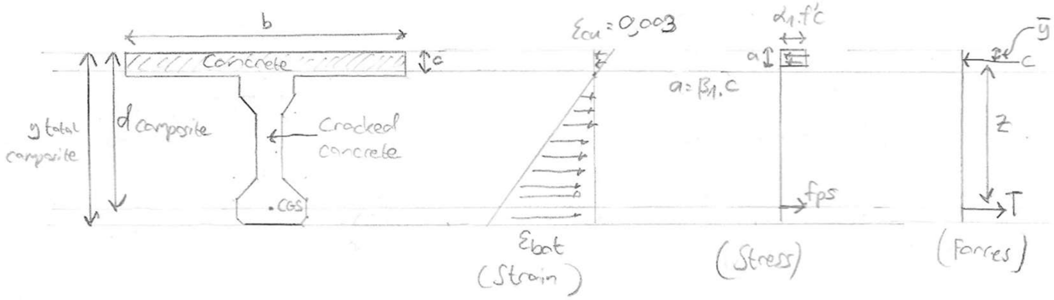
Strength Limit state:

Stress block parameters:

$\phi_p = 1$ $\quad f_{pu} = 1860$ MPa

$\alpha_1 = 0{,}85$ $\quad \phi_c = 1$ $\quad f'_c = 40$ MPa

$\beta_1 = 0{,}85$



a) Without reduction factors ϕ_p, ϕ_c - they are 1 in AASHTO anyways -

$$a = \frac{A_{ps} . f_{ps}}{\alpha_1 . f'c . b} = \frac{98{,}7 . 32 . f_{ps}}{0{,}85 . 40 . 2500} = 0{,}0372 . f_{ps}$$

$$\frac{c_u}{d_p} = \frac{A_{ps} . f_{pu}}{\alpha_1 . \beta_1 . f'c . b . d} = \frac{98{,}7 . 32 . 1860}{0{,}85 . 0{,}85 . 40 . 2500 . 1477{,}8} = 0{,}055$$

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

both OK

$$f_{ps} = f_{pu}\left(1 - k_p . \frac{c_u}{d_p}\right) = 1860 . (1 - 0{,}28 . 0{,}055) = 1831 \text{ MPa}$$

$$a = 0{,}0372 . f_{ps} = 0{,}0372 \ 1831 = 68{,}05 \text{ mm}$$

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

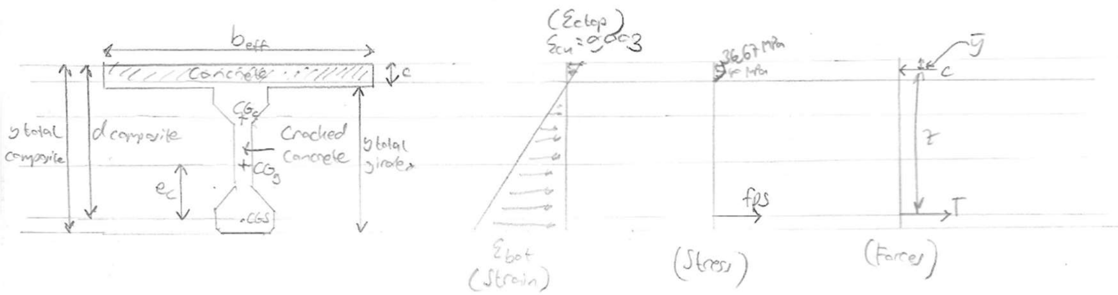
$$M_n = \underbrace{A_{ps} . f_{ps}}_{T} . \underbrace{\left(d - \frac{a}{2}\right)}_{z} = 98{,}7 . 32 . \left(1477{,}8 - \frac{68{,}05}{2}\right) . 1831 . 10^{-6}$$

$$\boxed{M_n = 8351 \text{ kNm} > 6669{,}16 \text{ kNm ok}}$$

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

$\varepsilon_{ctop} = \varepsilon_{cu} = 0,003$

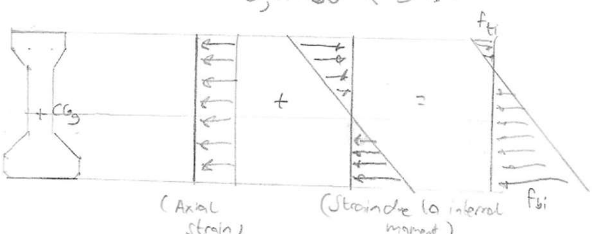
$\varepsilon_o = 0,002$

$f'_c = 40$ 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



$$-\frac{P_{pe}}{A_{gt} \cdot E_{ps}} = \frac{-3545,2 \cdot 10^3}{528487,6385 \cdot 200000} = -3,3541 \cdot 10^{-5}$$

$$\varepsilon_{ti_p} = -\frac{P_{pe}}{A_{gt} \cdot E_{ps}} + \frac{P_{pe} \cdot e_c \cdot y_t}{I_{gt} \cdot E_{ps}} = -3{,}3541 \cdot 10^{-5} + \frac{3545{,}2 \cdot 10^3 \cdot 534{,}4895 \cdot 743{,}3605}{1{,}1391 \cdot 10^{11} \cdot 200000} = 2{,}8287 \cdot 10^{-5}$$

$$\varepsilon_{bi_p} = -\frac{P_{pe}}{A_{gt} \cdot E_{ps}} - \frac{P_{pe} \cdot e_c \cdot y_b}{I_{gt} \cdot E_{ps}} = -3{,}3541 \cdot 10^{-5} - \frac{3545{,}2 \cdot 10^3 \cdot 534{,}4895 \cdot 628{,}2395}{1{,}1391 \cdot 10^{11} \cdot 200000} = -8{,}5794 \cdot 10^{-5}$$

$$\varepsilon_{ci_p} = \varepsilon_{ti_p} + (\varepsilon_{bi_p} - \varepsilon_{ti_p}) \cdot d \cdot \frac{1}{y_{total}} = 2{,}8287 \cdot 10^{-5} + (-8{,}5794 \cdot 10^{-5} - 2{,}8287 \cdot 10^{-5}) \cdot 1277{,}8 \cdot \frac{1}{1371{,}6} = -7{,}7997 \cdot 10^{-5}$$

Assuming unshored construction

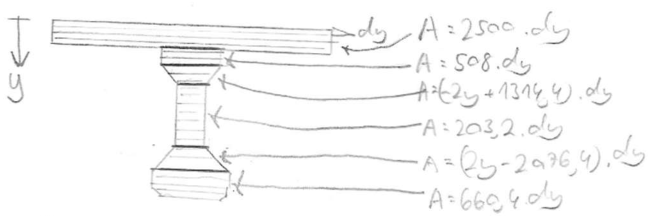
$$\varepsilon_{ti_{DL}} = -\frac{(M_G + M_S) \cdot y_t}{I_{gt} \cdot E_{ps}} = -\frac{(985{,}16 + 982{,}86) \cdot 10^6 \cdot 743{,}3605}{1{,}1391 \cdot 10^{11} \cdot 200000} = -6{,}3562 \cdot 10^{-5}$$

$$\varepsilon_{bi_{DL}} = \frac{(M_G + M_S) \cdot y_b}{I_{gt} \cdot E_{ps}} = \frac{(985{,}16 + 982{,}86) \cdot 10^6 \cdot 628{,}2395}{1{,}1391 \cdot 10^{11} \cdot 200000} = 5{,}3179 \cdot 10^{-5}$$

$$\varepsilon_{ci_{DL}} = \frac{\overset{e_c}{d - y_t}}{y_b} \cdot \varepsilon_{bi_{DL}} = \frac{534{,}4895}{628{,}2395} \cdot 5{,}3179 \cdot 10^{-5} = 4{,}5703 \cdot 10^{-5}$$

$$\varepsilon_{ci} = |\varepsilon_{cip}| + \varepsilon_{c_{DL}} = 1{,}2370 \cdot 10^{-4}$$

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

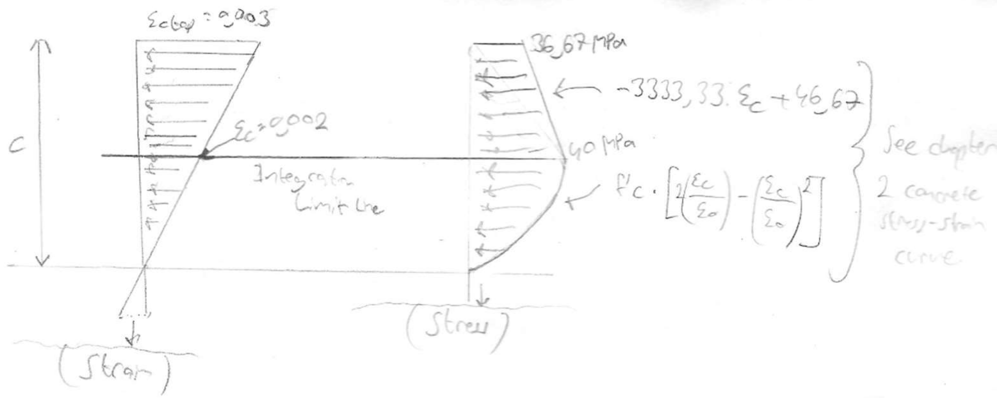


$$\varepsilon_s = \varepsilon_{si} + \varepsilon_{ci} + \varepsilon_{ctop} \cdot \frac{d_{composite} - c}{c}$$

$$\varepsilon_s = 0{,}0056 + 1{,}2370 \cdot 10^{-4} + 0{,}003 \cdot \frac{1477{,}8 - c}{c}$$

$$\left.\begin{array}{ll} f_{ps} = E_{ps} \cdot \varepsilon_s = 200000 \cdot \varepsilon_s & \text{for } \varepsilon_s \leq 0{,}008 \\ f_{ps} = 1848 - 0{,}517 \cdot \dfrac{1}{\varepsilon_s - 0{,}005915} & \text{for } \varepsilon_s > 0{,}008 \\ f_{ps} = \alpha \quad \text{for} \quad f_{ps} > 0{,}99 \cdot f_{pu} & \end{array}\right\}$$

* 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

$$\varepsilon_s = 0{,}0056 + 1{,}2370 \cdot 10^{-4} + 0{,}003 \cdot \frac{1477{,}8 - 76{,}02}{76{,}02} = 0{,}0611$$

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

$$\therefore f_{ps} = 1848 - 0{,}517 \cdot \frac{1}{0{,}0611 - 0{,}005915} = 1838{,}61 \text{ MPa}$$

$$1838{,}61 \text{ MPa} < 0{,}99 \cdot f_{pu} \text{ OK}$$
$$(1841{,}4)$$

$$\text{Integration Limit} = 0{,}333\,c = 25{,}34 \text{ mm}$$

$$T \cong C = f_{ps} \cdot A_{ps} = 1838{,}6 \cdot 3158{,}4 = 5{,}807 \cdot 10^6 \text{ N}$$

$$\bar{y} = \frac{\int_{0}^{0{,}333c}(-3333{,}33 \cdot \varepsilon_c + 46{,}67) \cdot A \cdot y + \int_{0{,}333c}^{c} 40 \cdot \left[2 \cdot \left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right] \cdot A \cdot y}{T}$$

$$\varepsilon_c = 0{,}003 - 0{,}003 \cdot \frac{y}{76{,}02}$$

$$\bar{y} = \frac{\left(\int_{0}^{25{,}34}\left(-3333{,}33 \cdot \left(0{,}003 - 0{,}003 \frac{y}{76{,}02}\right) + 46{,}67\right) \cdot 2500 \cdot y \cdot dy + \int_{25{,}34}^{76{,}02} 40 \cdot \left[2 \cdot \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}02}}{0{,}002}\right) - \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}02}}{0{,}002}\right)^2\right] \cdot 2500 \cdot y \cdot dy\right)}{(5{,}807 \cdot 10^6)}$$

$$\bar{y} = \frac{1{,}8104 \cdot 10^8}{5{,}807 \cdot 10^6} = 31{,}1783 \text{ mm}$$

$$z = d\ \text{composite} - \bar{y} = 1477{,}9 - 31{,}1783 = 1446{,}7 \text{ mm}$$

$$M_r = z \cdot T \cdot 10^{-6} = 1446{,}7 \cdot 5{,}807 = 8401 \text{ kNm} \pm 0{,}5 \text{ kNm}$$

$$\boxed{8401 \text{ kNm} > 6669{,}16 \text{ kNm} \quad \text{OK}}$$

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.

$$1.33 \times 6673.18 = \mathbf{8875.34\ kNm} > \mathbf{8401\ kNm}$$

Therefore reserve capacity must be checked.

At service and at midspan:

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

$$f_b = -\frac{3545.2 \times 10^3}{509031} - \frac{3545.2 \times 10^3 \times 534.4895 \times 628.2395}{1.0853 \times 10^{11}} + \frac{(985.16 + 962.86) \times 10^6 \times 628.2395}{1.0853 \times 10^{11}}$$

$$+ \frac{(301.98 + 2165.4) \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = \mathbf{2.2562\ MPa\ T}$$

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 $3.7947 - 2.2562 = \mathbf{1.5386\ MPa\ T}$

$$\frac{M_{add} \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = 1.5386, \text{ solving for } M_{add},\ \mathbf{M_{add} = 425.9120\ kNm}$$

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.

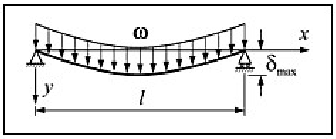
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED BRIDGE DEFLECTION AND MAXIMUM DEFLECTION	
	$y = \frac{\omega x}{24EI}\left(l^3 - 2lx^2 + x^3\right)$	$\delta_{max} = \frac{5\omega l^4}{384EI}$

Figure 5.4.6.1 – Deflection equations for UDL on a simply supported beam

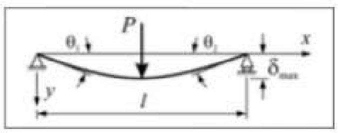
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED DEFLECTION AND MAXIMUM DEFLECTION	
	$y = \frac{Px}{12EI}\left(\frac{3l^2}{4} - x^2\right)$ for $0 < x < \frac{l}{2}$	$\delta_{max} = \frac{Pl^3}{48EI}$

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

$e_c = 628.2395 - 313.425 = 314.8145\ mm$

$$\Delta_c = \left(\frac{534.4895}{8} - 0.327^2 \times \frac{(534.4895 - 314.815)}{6} \right) \times 4089.8 \times 10^3 \times \frac{26000^2}{(33994 \times 1.0853 \times 10^{11})}$$

$\Delta_c = 47\ mm\ upwards$

Total deflection at erection $= 1.85 \times \Delta_{DL} + 1.8 \times \Delta_c = 1.85 \times 19 + 1.8 \times -47 =$ ***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 $= 2.4 \times 19 + 2.2 \times -47 + 2.3 \times 18 + 3 \times 6 =$ ***2 mm downwards***

All deflections are within limit of $\frac{1}{800}$ *so this design is safe*

FLEXURAL DESIGN NOW COMPLETE

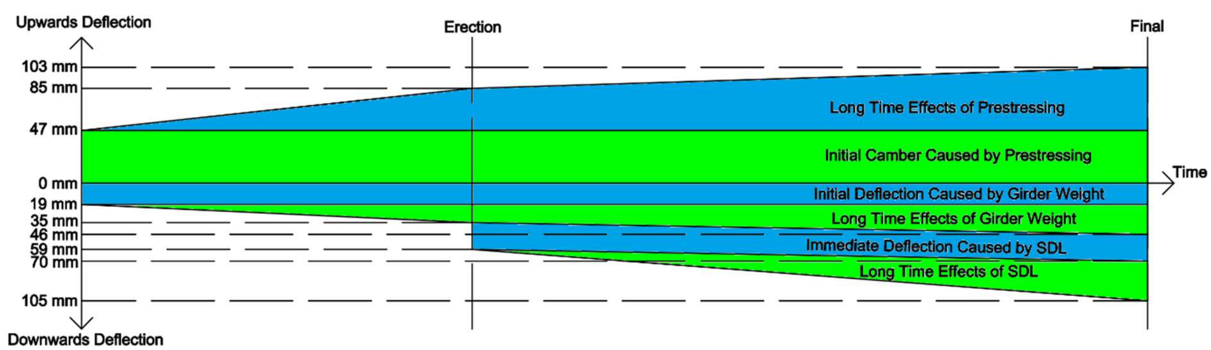


Figure 5.4.6.3 – Visual Representation of the Deflections Experienced

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>

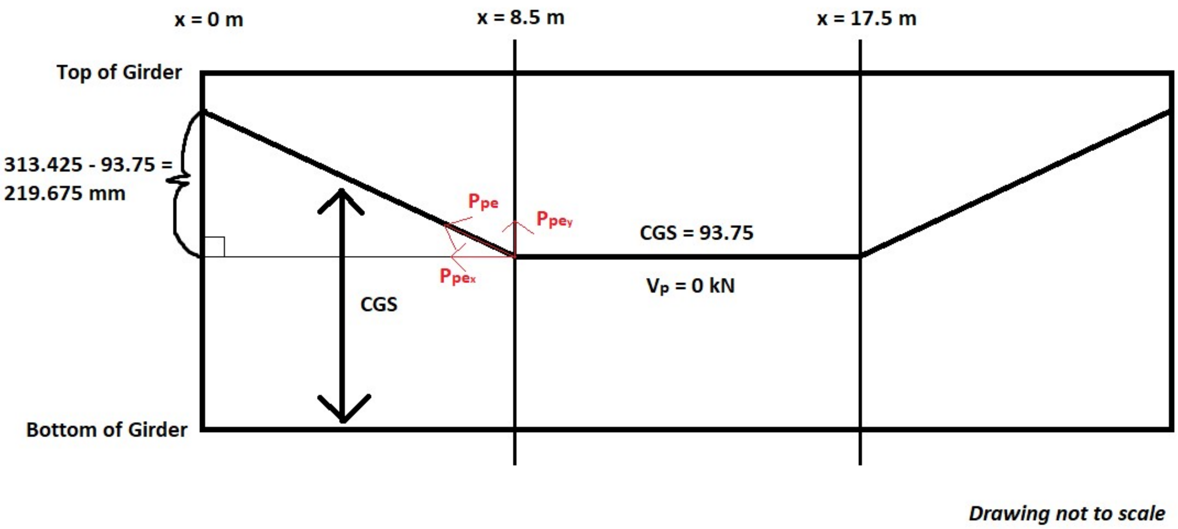


Figure 5.3.7.1 – Calculation of V_p

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):

$\boldsymbol{V_p}$ ***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

$$V_p = P_{pe} \times \sin\left(arc\tan\left(\frac{219.675}{8500}\right)\right) = \mathbf{91.6\,kN}\ (P_{pe}\ was\ 3545.2\,kN)$$

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.

<u>Determination of equivalent cracking parameter S_{ze}:</u>

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

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

<u>Determination of the longitudinal strain at the centroidal axis of the critical section ε_x:</u>

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + V_u - V_p + 0.5 \times N_u - A_{ps} \times f_{po}}{2 \times (E_s \times A_s + E_p \times A_{ps})}$$

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.*

<u>Determination of the angle of inclination of the compressive stresses and value of beta:</u>

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

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

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:

$If\ v_u \geq 0.125 \times f'_c$

$then,\ s_{max} = Lesser\ of\ 0.4 \times d_v\ or\ 300\ mm$

$If\ v_u \geq 0.5 \times \sqrt{f'_c}$

$then,\ s_{max} = Lesser\ of\ 0.4 \times d_v\ or\ 450\ mm$

$If\ v_u < 0.125 \times f'_c$

$then,\ s_{max} = Lesser\ of\ 0.8 \times d_v\ or\ 600\ mm$

At midspan:

$v_u (1.18\ MPa) < 0.125 \times f'c\ (5\ MPa)$

$therefore\ s_{max} = Lesser\ of\ 920\ and\ 600\ mm$

$therefore\ s_{max} = 600\ mm$

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.***

$A_{s,provided} = \mathbf{1600\ mm^2 > 1170\ mm^2\ OK.}$

(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:

$$V_{s,design} = \frac{0.9 \times 400 \times 400 \times 1150.07 \times cot(39.72)}{570} \times 10^{-3} = \mathbf{349.72\ kN}$$

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)

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

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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

K & B

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 − 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* $A_s = 300\ mm^2\ (577\ mm^2/m)$

5.5 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:

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

$$15.7459 = \frac{P_{pe} \times 10^3}{5.0903 \times 10^5} + \frac{P_{pe} \times 528.2395 \times 10^3}{1.7275 \times 10^8}$$

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***

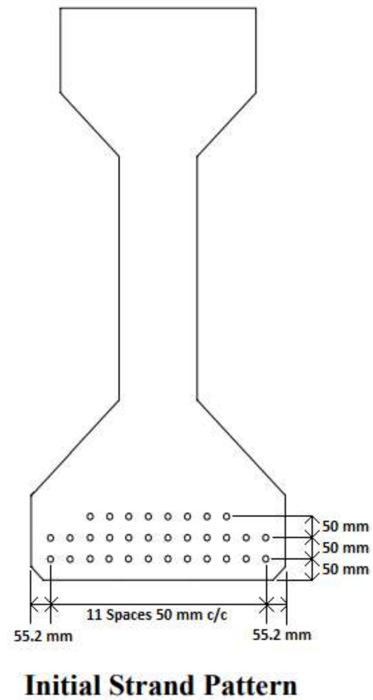


Figure 5.5.1.1 – Initial Strand Pattern

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)

$$\% \, Loss = \frac{240}{f_{pi}} \times 100 = \frac{240}{1395} \times 100 = 17.2043 \, \%$$

$$\% \, Initial \; Loss = \frac{105}{f_{pi}} \times 100 = \frac{105}{1395} \times 100 = 7.5269 \, \%$$

$$Final \; effective \; prestress, \, f_{pe} = f_{pi} - \Delta f_{pT} = 1395 - 240 = 1155 \, MPa$$

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

$$Total \; prestressing \; force \; after \; all \; losses, \, P_{pe} = 32 \times 98.7 \times 1155 \times 10^{-3} = 3647.95 \, kN$$

$$Initial \; prestressing \; force \; after \; initial \; losses, \, P_i = 32 \times 98.7 \times (1395 - 105) \times 10^{-3} = 4074.34 \, kN$$

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.

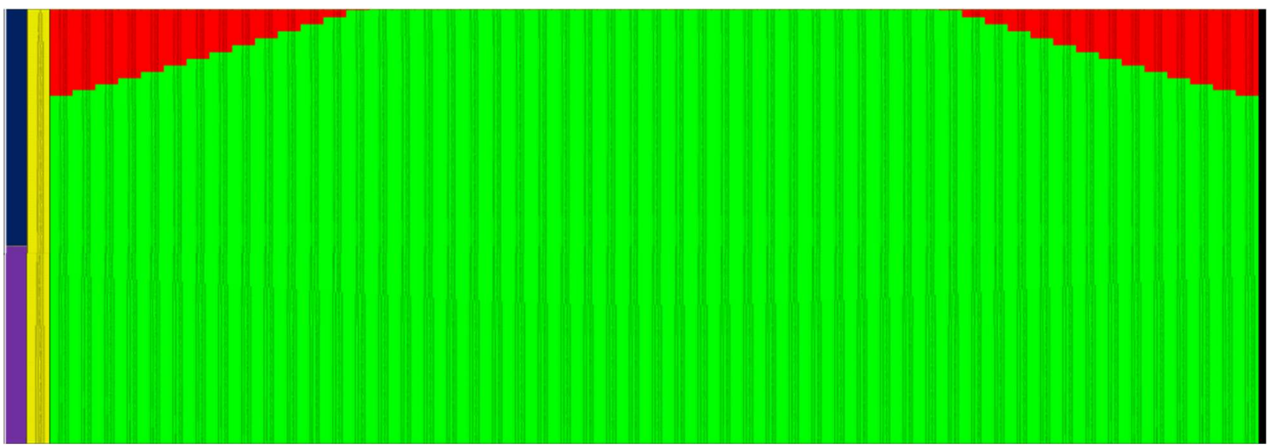


Figure 5.5.3.1.1 – Straight Strands – Stresses experienced

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):

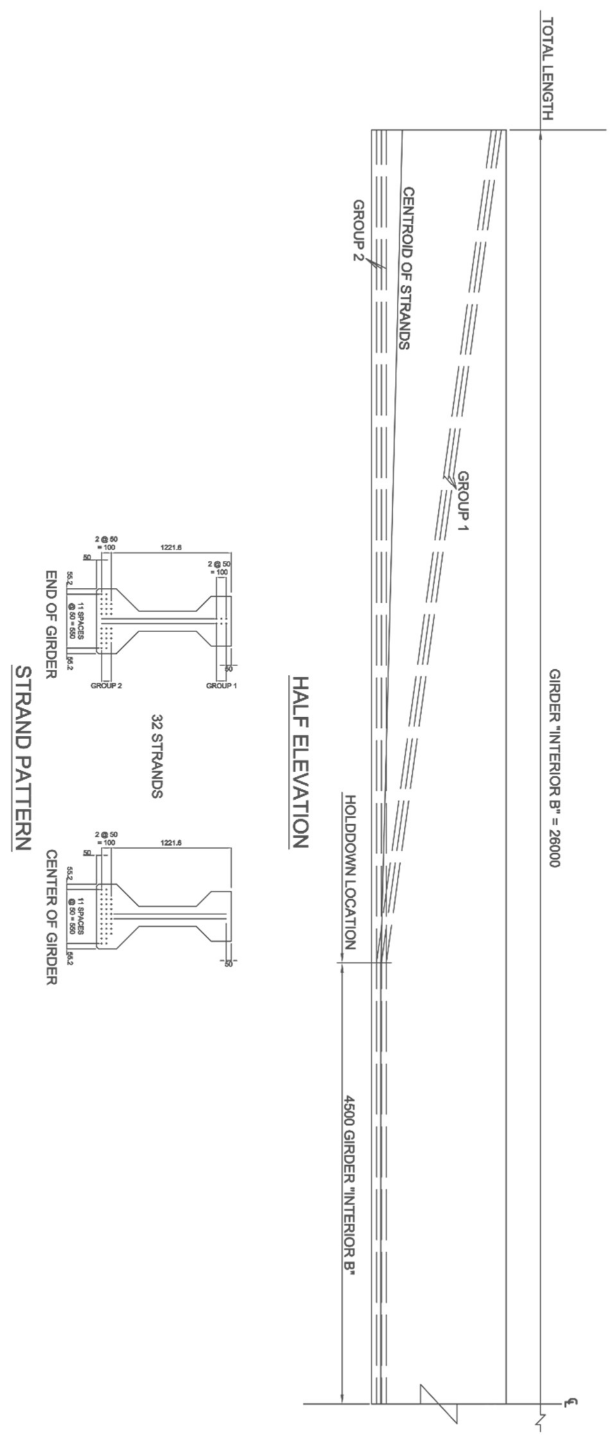


Figure 5.5.3.1.2 – Strand Pattern

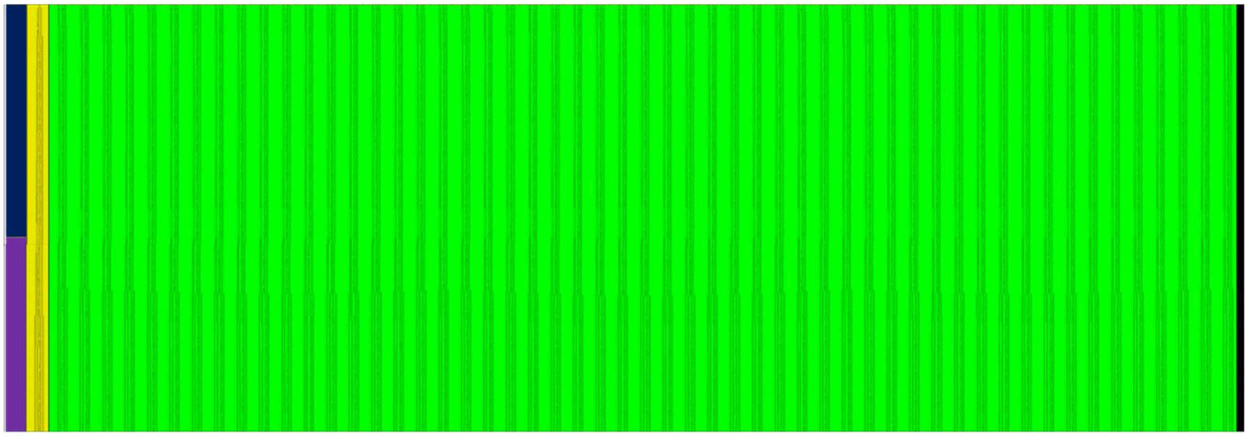


Figure 5.5.3.1.3 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced

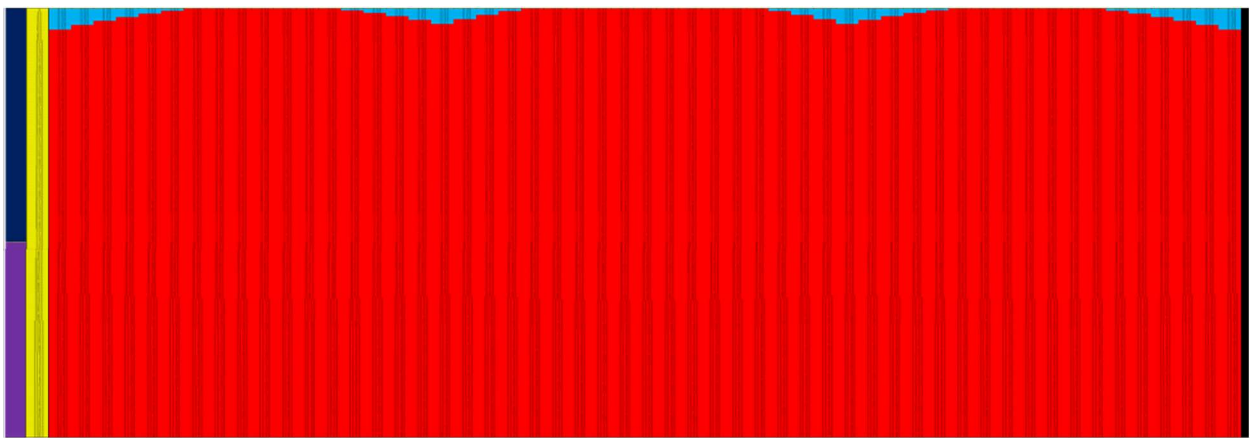


Figure 5.5.3.1.4 – Harped Strands with groups as in figure 5.5.3.1.2 – Elements in tension Blue, Elements in compression Red

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:

$$f_{bs} = 0.5 \times \sqrt{40} = 3.162\,MPa$$

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

$$f_{bs} = -\frac{3648 \times 10^3}{509031} - \frac{3648 \times 10^3 \times 534.4895}{1.7275 \times 10^8} + \frac{(1053.82 + 1014) \times 10^6}{1.7275 \times 10^8}$$

$$+ \frac{(322.68 + 1598.02) \times 10^6}{2.7683 \times 10^8} = 0.4352\,MPa\ OK$$

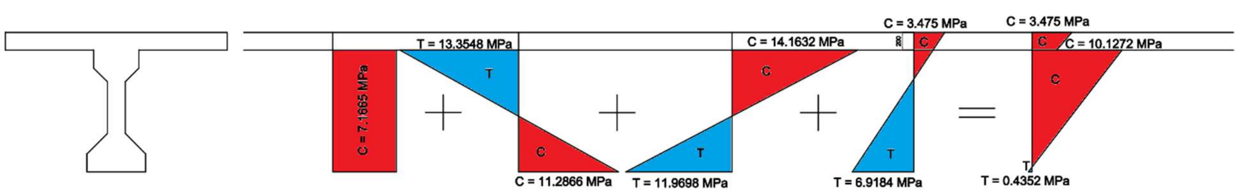


Figure 5.5.3.2.1 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced visualized at midspan

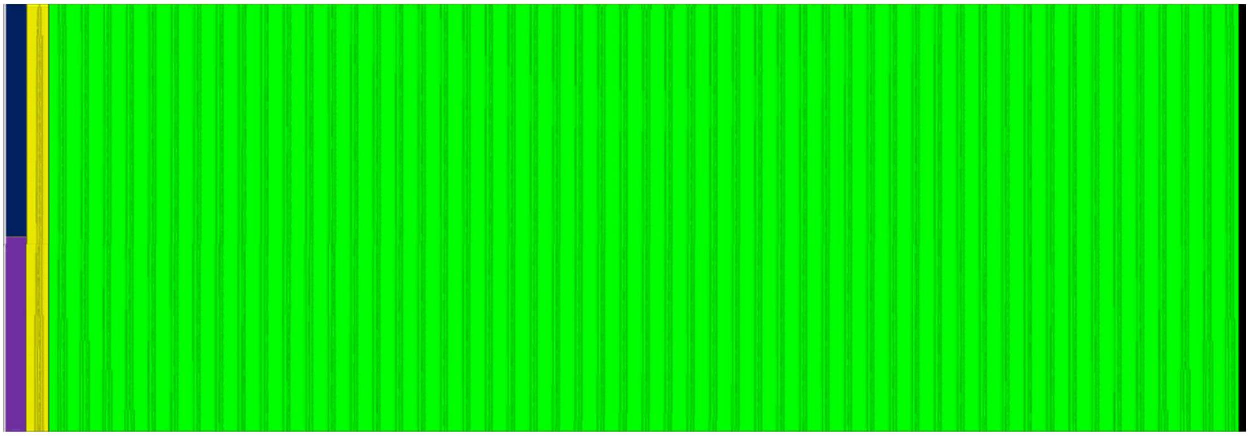


Figure 5.5.3.2.2 – Harped Strands with groups as in figure 5.5.3.1.2 – Stresses experienced at service conditions

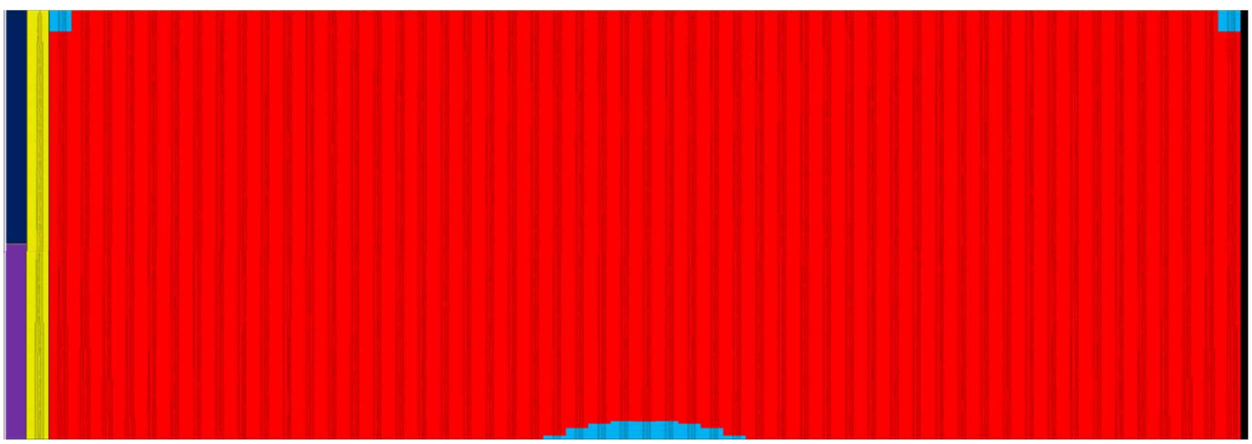


Figure 5.5.3.2.3 – Harped Strands with groups as in figure 5.5.3.1.2 – Elements in tension Blue, Elements in compression Red – Service Conditions

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:

$\alpha_1 = 0{,}85$

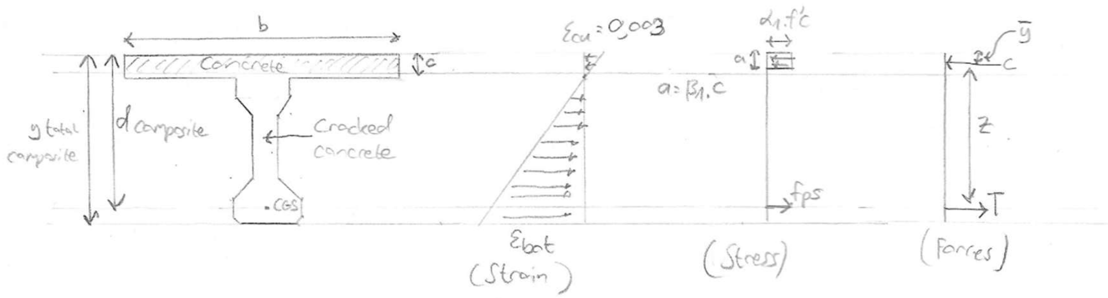
$\beta_1 = 0{,}85$

$\phi_p = 1$

$\phi_c = 1$

$f_{pu} = 1860$ MPa

$f'_c = 40$ MPa



a) Without reduction factors ϕ_p, ϕ_c – they are assumed 1

$$a = \frac{A_{ps} . f_{ps}}{\alpha_1 . f'c . b} = \frac{98{,}7 . 32 . f_{ps}}{0{,}85 . 40 . 2500} = 0{,}0372 . f_{ps}$$

$$\frac{c_u}{d_p} = \frac{A_{ps} . f_{pu}}{\alpha_1 . \beta_1 . f'c . b . d} = \frac{98{,}7 . 32 . 1860}{0{,}85 . 0{,}85 . 40 . 2500 . 1477{,}8} = 0{,}055$$

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

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

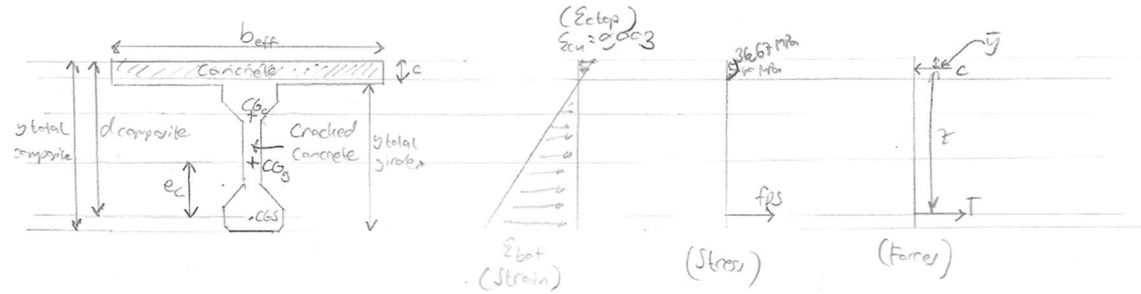
$$M_n = \underbrace{A_{ps} . f_{ps}}_{T} . \underbrace{\left(d - \frac{a}{2}\right)}_{z} = 98{,}7 . 32 . \left(1477{,}8 - \frac{68{,}05}{2}\right) . 1831 . 10^{-6}$$

$$\boxed{M_n = 8351 \text{ kNm} > 7567{,}03 \text{ kNm ok}}$$

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 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

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

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

P_{pe} = 3648 kN

$\varepsilon_{ctop} = \varepsilon_{cu}$ = 0,003

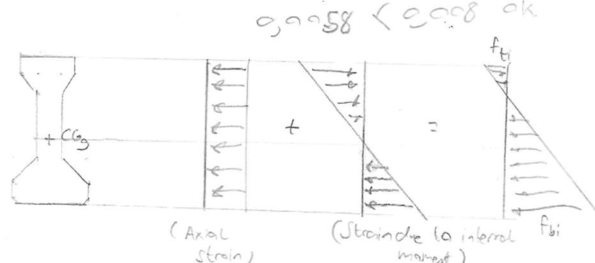
ε_o = 0,002

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

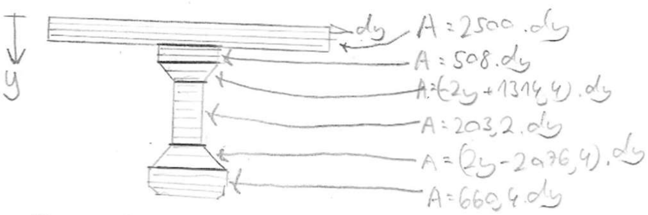
$$\varepsilon_{ti_{DL}} = -\frac{(M_G + M_S) \cdot y_t}{I_{gt} \cdot E_{ps}} = -\frac{(1053{,}82 + 1014) \cdot 10^6 \cdot 743{,}3605}{1{,}1391 \cdot 10^{11} \cdot 200000} = -6{,}7472 \cdot 10^{-5}$$

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

$$\varepsilon_{ci_{DL}} = \frac{\overset{e_c}{d - y_t}}{y_b} \cdot \varepsilon_{bi_{DL}} = \frac{534{,}4895}{628{,}2395} \cdot 5{,}7023 \cdot 10^{-5} = 4{,}8513 \cdot 10^{-5}$$

$$\varepsilon_{ci} = |\varepsilon_{cip}| + \varepsilon_{ci_{DL}} = 1{,}2877 \cdot 10^{-4}$$

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

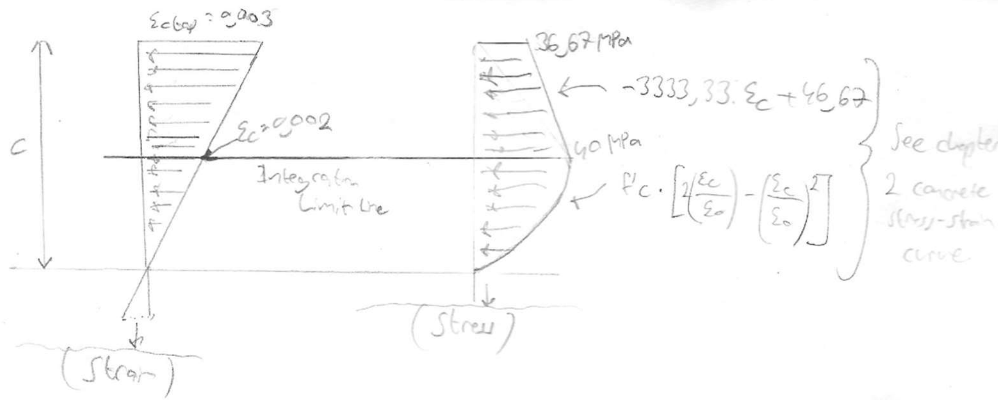


$$\varepsilon_s = \varepsilon_{si} + \varepsilon_{ci} + \varepsilon_{ctop} \cdot \frac{d_{composite} - c}{c}$$

$$\varepsilon_s = 0{,}0058 + 1{,}2877 \cdot 10^{-4} + 0{,}003 \cdot \frac{1477{,}8 - c}{c}$$

$$\left.\begin{aligned} f_{ps} &= E_{ps} \cdot \varepsilon_s = 200000 \cdot \varepsilon_s \quad \text{for } \varepsilon_s \leq 0{,}008 \\ f_{ps} &= 1848 - 0{,}517 \cdot \frac{1}{\varepsilon_s - 0{,}005915} \quad \text{for } \varepsilon_s > 0{,}008 \\ f_{ps} &= \alpha \quad \text{for } f_{ps} > 0{,}99 \cdot f_{pu} \end{aligned}\right\}$$

* See chapter 2 Prestressing Steel Stress-Strain curve



$$\text{Integration Limit} = -(0{,}002 - \varepsilon_{c\,top})\cdot\frac{c}{\varepsilon_{c\,top}} = -(0{,}002 - 0{,}003)\cdot\frac{c}{0{,}003}$$

$$= 0{,}333\,c \text{ from top}$$

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

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

$$C = \int_0^{0{,}333c}(-3333{,}33\cdot\varepsilon_c + 46{,}67)\cdot A + \int_{0{,}333c}^{c} 40\cdot\left[2\cdot\left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2\right]\cdot A$$

*Solving $T = C$ numerically using MATLAB, $c = 76{,}03$ mm
(code available in the appendix section)

Let $c = 76{,}03$ mm

$$\varepsilon_s = 0{,}0058 + 1{,}2877\cdot 10^{-4} + 0{,}003\cdot\frac{1477{,}8 - 76{,}03}{76{,}03} = 0{,}0612$$

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

$$\therefore f_{ps} = 1848 - 0{,}517\cdot\frac{1}{0{,}0612 - 0{,}005915} = 1838{,}7 \text{ MPa}$$

$$1838{,}7 \text{ MPa} < 0{,}99\cdot f_{pu} \quad \text{OK}$$
$$(1841{,}4)$$

$$\text{Integration Limit} = 0{,}333\,c = 25{,}34 \text{ mm}$$

$$T \cong C = f_{ps}\cdot A_{ps} = 1838{,}7\cdot 3158{,}4 = 5{,}807\cdot 10^6\,\text{N}$$

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$$\bar{y} = \frac{\int_{0}^{0{,}333c} (-3333{,}33 \cdot \varepsilon_c + 46{,}67) \cdot A \cdot y + \int_{0{,}333c}^{c} 40 \cdot \left[2 \cdot \left(\frac{\varepsilon_c}{0{,}002}\right) - \left(\frac{\varepsilon_c}{0{,}002}\right)^2 \right] \cdot A \cdot y}{T}$$

$$\varepsilon_c = 0{,}003 - 0{,}003 \cdot \frac{y}{76{,}03}$$

$$\bar{y} = \frac{\left(\int_{0}^{25{,}34} \left(-3333{,}33 \cdot \left(0{,}003 - 0{,}003 \frac{y}{76{,}03}\right) + 46{,}67\right) \cdot 2500 \cdot y \cdot dy + \int_{25{,}34}^{76{,}02} 40 \cdot \left[2 \cdot \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}03}}{0{,}002} \right) - \left(\frac{0{,}003 - 0{,}003 \frac{y}{76{,}03}}{0{,}002} \right)^2 \right] \cdot 2500 \cdot y \cdot dy \right)}{(5{,}807 \cdot 10^6)}$$

$$\bar{y} = \frac{1{,}8109 \cdot 10^8}{5{,}807 \cdot 10^6} = 31{,}1824 \text{ mm}$$

$$z = d \text{ composite} - \bar{y} = 1477{,}9 - 31{,}1824 = 1446{,}7 \text{ mm}$$

$$M_r = z \cdot T \cdot 10^{-6} = 1446{,}7 \cdot 5{,}807 = 8401 \text{ kNm} \pm 0{,}5 \text{ kNm.}$$

$$\boxed{8401 \text{ kNm} > 7567{,}03 \text{ kNm} \quad \text{OK}}$$

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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.

$$1.33 \times 7575.5 = 10100\ kNm > 8401\ kNm$$

Therefore reserve capacity must be checked.

At service and at midspan:

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

$$f_b = -\frac{3648 \times 10^3}{509031} - \frac{3648 \times 10^3 \times 534.4895 \times 628.2395}{1.0853 \times 10^{11}} + \frac{(1053.82 + 1014) \times 10^6 \times 628.2395}{1.0853 \times 10^{11}}$$

$$+ \frac{(322.68 + 1598.02) \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = 0.4551\ MPa\ T$$

At cracking, the bottom stress $= 0.6 \times \sqrt{f'_c} = 0.6 \times \sqrt{40} = 3.7947\ MPa\ T$

The additional moment must create a bottom stress of $3.7947 - 0.4551 = 3.3397\ MPa\ T$

$$\frac{M_{add} \times 10^6 \times 1046.1}{2.8960 \times 10^{11}} = 3.3397,\ \text{solving for } M_{add},\ M_{add} = 924.5014\ kNm$$

Therefore, $M_{cr} = 924.5014 + 1053.82 + 1014 + 322.68 + 1598.02 = 4913\ kNm$

$$1.2 \times 4913\ kNm = 5895.6\ kNm < 8401\ kNm\ OK$$

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.

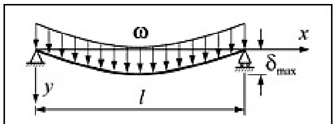
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED BRIDGE DEFLECTION AND MAXIMUM DEFLECTION	
	$y=\frac{\omega x}{24EI}\left(l^3-2lx^2+x^3\right)$	$\delta_{max}=\frac{5\omega l^4}{384EI}$

Figure 5.5.6.1 – Deflection equations for UDL on a simply supported beam

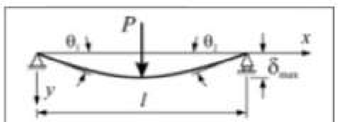
SIMPLY SUPPORTED BEAM	DEFLECTION AT ANY SECTION IN TERMS OF x	MAXIMUM AND CENTER DEFLECTION
	SIMPLY SUPPORTED DEFLECTION AND MAXIMUM DEFLECTION	
	$y=\frac{Px}{12EI}\left(\frac{3l^2}{4}-x^2\right)$ for $0<x<\frac{l}{2}$	$\delta_{max}=\frac{Pl^3}{48EI}$

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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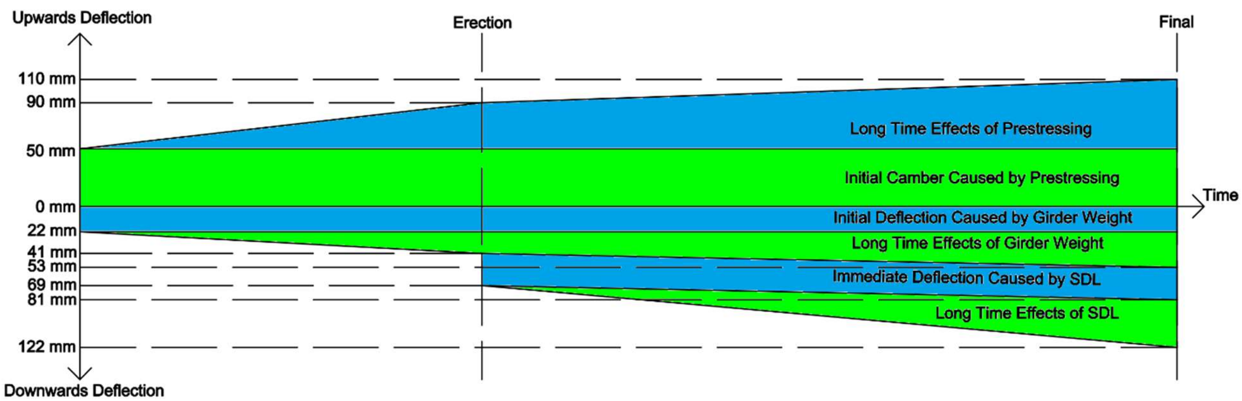

Figure 5.5.6.3 – Visual Representation of the Deflections Experienced

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:

$$V_c = v_c \times b_v \times h \text{ where } b_v = \text{Smallest width of the section (203.2 mm for AASHTO Type 4 Girders)}.$$

$$V_c = 2.1 \times 203.2 \times 1371.6 \times 10^{-3} = \mathbf{585.46\ kN}$$

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, Atotal = 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, Atotal = 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, Atotal = 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;
```

```
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:
```

```
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);
```

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Chapter 6 – Reinforced Concrete Deck Design

Table of Contents

K & B

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:

$$M_D^{-} = \frac{w \times L^2}{11} \qquad M_D^{+} = \frac{w \times L^2}{16}$$

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:

$$M_{CL-625} = \left(0.8 \times \left(S_e + 0.6\right) \times P\right) \times \frac{(1 + DLA)}{10}$$

$where:$

$M_{CL-625} = Negative\ and\ positive\ transverse\ bending\ moment,\ kNm/m$

$S_e = Equivalent\ span\ legnth\ of\ concrete\ deck\ in\ meters$

$P = Maximum\ wheel\ load\ of\ CL-625\ Truck = 175/2 = 87.5\ kN$

$DLA = Dynamic\ Load\ Allowance = 0.4$

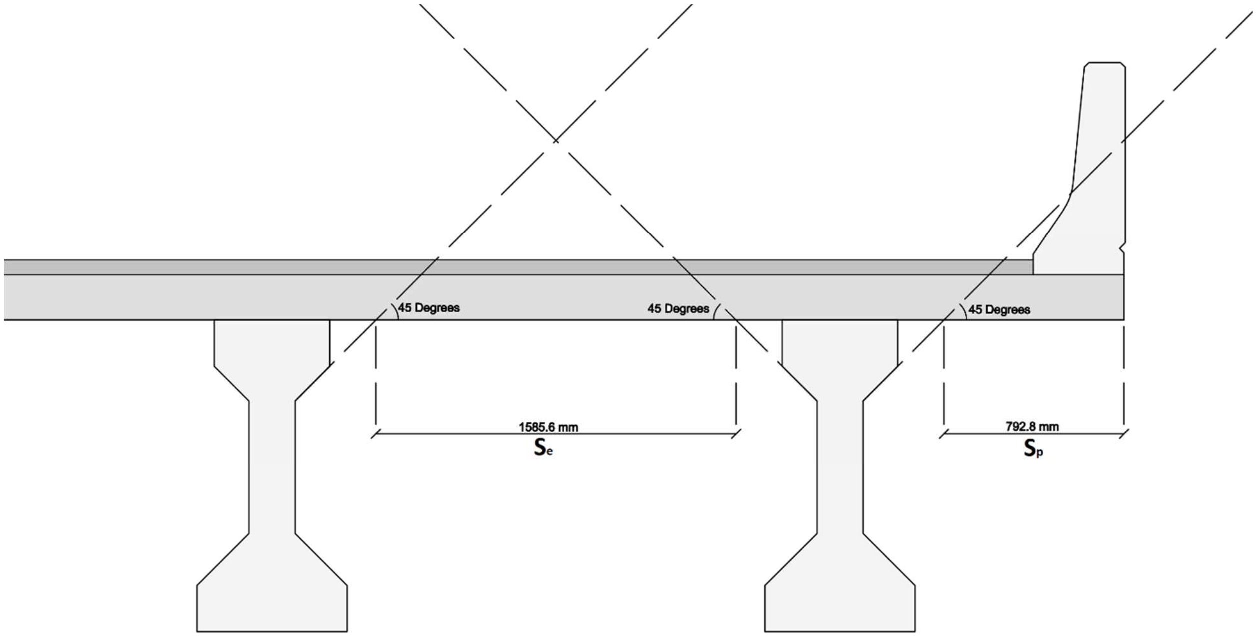

Figure 6.2.3.1 – Graphical Representation of S_e and S_p for the bridge

$$S_e = 1585.6\,mm$$

$$S_p = 792.8\,mm$$

$$M_{CL-625} = (0.8 \times (1.5856 + 0.6) \times 87.5) \times \frac{(1+0.4)}{10} = 21.42\,kNm/m$$

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.

$$\frac{120}{S_e^{\,0.5}} = \frac{120}{1.5856^{0.5}} = 95.3\,\% > 2/3\,(66.7\,\%)$$

Therefore, longitudinal moment can be calculated as follows:

$$M_{CL-625} = 21.42 \times \frac{2}{3} = 14.28\,kNm/m$$

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:

$$b = 1000\ mm\ as\ unit\ length$$

$$K_r = \frac{M_r}{\left(b \times d^2\right)} = \frac{39.56 \times 10^6}{\left(1000 \times 152.02^2\right)}$$

$$= 1.71$$

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Determination of the reinforcement ratio, ρ (%):

$-f'_c = 35\ MPa$

$-K_r = 1.71$

From table 2.1 CSA – A23.3 – 14:

$\boldsymbol{\rho = 0.53\ \%}$

$Therefore,\ \mathbf{A_{s,required}} = \rho \times b \times d = 0.0053 \times 1000 \times 152.02 = \mathbf{805.71\ mm^2}$

$\mathbf{Assuming}\ 5 \times 15M\ Bars\ which\ gives\ \mathbf{A_s = 1000\ mm^2}$

Minimum reinforcement check:

$A_{s,min} = 0.002 \times b \times t_s = 0.002 \times 1000 \times 200 = \mathbf{400\ mm^2}$

$A_s > A_{s,min}\ \mathbf{so\ 5 \times 15\ M\ OK}$

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

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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

K & B

Calculation of the crack control parameter:

$$z = f_s \times \sqrt[3]{(d_c \times A)} \quad [\text{N/mm}]$$

$$z < 25000\ N/mm \text{ for exterior exposure}$$

$$d_c = 40 + \sqrt{\frac{200}{\pi}} = 47.98\ mm$$

$$g = t_s - d = 47.98\ mm$$

$$A = \frac{2 \times g \times b}{n} = \frac{2 \times 47.98 \times 1000}{\frac{1000}{200}} = 19192\ mm^2$$

$$f_s = 0.6 \times f_y = 0.6 \times 400 = 240\ MPa$$

$$z = f_s \times \sqrt[3]{d_c \times A} = 240 \times \sqrt[3]{47.98 \times 19192} = 23348\ N/mm < 25000\ N/mm\ OK$$

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 $A_s = 1000\ mm^2$

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 $5 \times 15\ M$ 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

$S = The\ lesser\ of:$

$S = The\ larger\ of:$

$-\ 160\ mm$

$-\ Lesser\ of\ 23.94,\ 30,\ 40\ = 23.94\ mm$

$\mathbf{-\ 160\ mm\ chosen}$

$S = The\ lesser\ of:$

$-\ 160\ mm$

$-\ Lesser\ of\ 300,\ 450 = 300\ mm$

$\mathbf{-\ 160\ mm\ chosen}$

$\mathbf{-\ 160\ mm\ chosen}$

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, $\alpha = 0.85 + 0.0015 \times f'_c = 0.85 + 0.0015 \times 35 = \mathbf{0.9025}$

$$a = \frac{A_s \times \Phi_s \times f_y}{\left(\alpha \times \Phi_c \times f'_c \times b\right)} = \frac{1200 \times 0.9 \times 400}{0.9025 \times 0.75 \times 35 \times 1000} = \mathbf{18.24\ mm}$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 1200 \times 0.9 \times 400 \times \left(132.02 - \frac{18.24}{2}\right) \times 10^{-6} = \mathbf{53.09\ kNm/m}$$

$M_r\ (53.09\ kNm/m) > M_f\ (40.99\ kNm/m)\ OK$

Checking of design against maximum and minimum reinforcement:

$$Maximum\ reinforcement\ requirement = \frac{c}{d} < 0.5$$

$$Minimum\ reinforcement\ requirement = Adequate\ reinforcement\ must\ be\ placed\ so\ that\ M_r > 1.2 \times M_{cr}$$

$$c = \frac{a}{\alpha} = \frac{18.24}{0.9} = 20.21\ mm$$

$$\frac{c}{d} = \frac{20.21}{132.02} = \mathbf{0.153 < 0.5\ OK - Max.\ req.\ satisified}$$

$$f_{cr} = 0.4 \times \sqrt{f'_c} = 0.4 \times \sqrt{35} = 2.37\ MPa$$

$$S = \frac{b \times t_s^2}{6} = \frac{1000 \times 200^2}{6} = 6666667$$

$$M_{cr} = S \times f_{cr} = 6666667 \times 2.37 \times 10^{-6} = 15.78\ kNm/m$$

$$\mathbf{M_r (53.09\ kNm/m) > 1.2 \times M_{cr} (18.93\ kNm/m)\ OK - Min.\ req.\ satisfied}$$

Calculation of the crack control parameter:

$$z = f_s \times \sqrt[3]{d_c \times A}\ [N/mm]$$

$$z < 25000\ N/mm\ for\ exterior\ exposure$$

$$d_c = 50^* + \sqrt{\frac{200}{\pi}} = 57.98\ mm$$

* "*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 × *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*

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*

$$d_c = g = 40 + 2 \times \sqrt{\frac{200}{\pi}} + \sqrt{\frac{100}{\pi}} = 61.6\ mm$$

$$A = \frac{2 \times g \times b}{n} = \frac{2 \times 61.6 \times 1000}{\frac{1000}{140}} = 17248\ mm^2$$

$$f_s = 0.6 \times 400 = 240\ MPa$$

$$z = f_s \times \sqrt[3]{d_c \times A} = 240 \times \sqrt[3]{61.6 \times 17248} = \mathbf{24490\ N/mm < 25000\ N/mm\ OK}$$

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

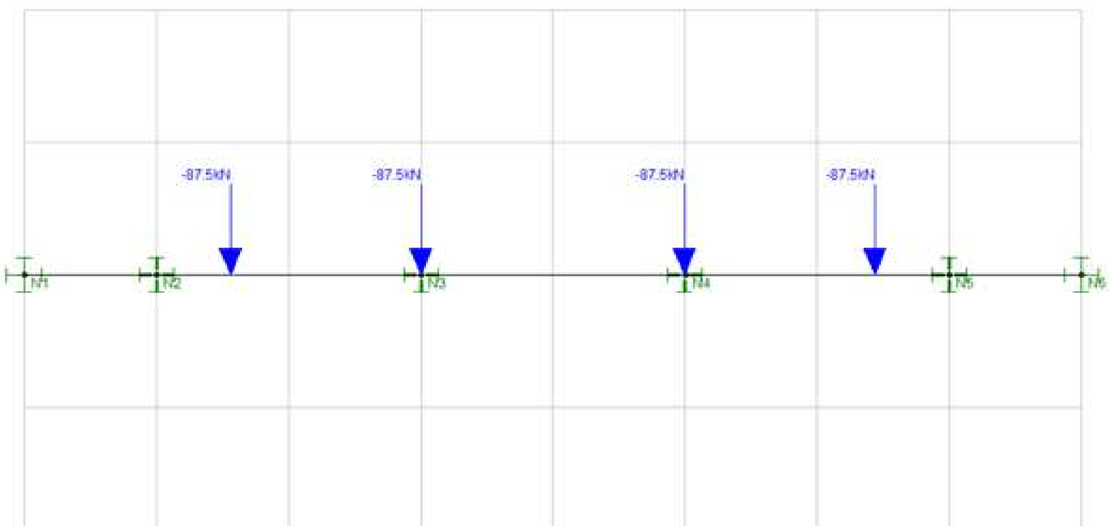
l_n (unsupported length between girders) = 1.992 m

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:

$\beta = 0.18$

$f'_c = 35\ MPa$

$d_v = \text{Greater of } 0.9 \times d \text{ or } 0.72 \times t_s = 144\ mm$

$b_{eff} = \text{Effective width (Calculated below)}$

$$b_e = \frac{1 - \left(1 - \dfrac{L}{15 \times b}\right)^3}{b} \quad \text{where } L/b < 15,\ \ b_e = b \text{ for } L/b \geq 15$$

where:

$b = \text{Half of the unsupported distance between girders} = 0.996\ m$

$L = 26\ m$

$L/b = 26.1$

$b_e = 0.996\ m$

$b_{eff} = 0.996 \times 2 + 0.508 = 2.5\ m$

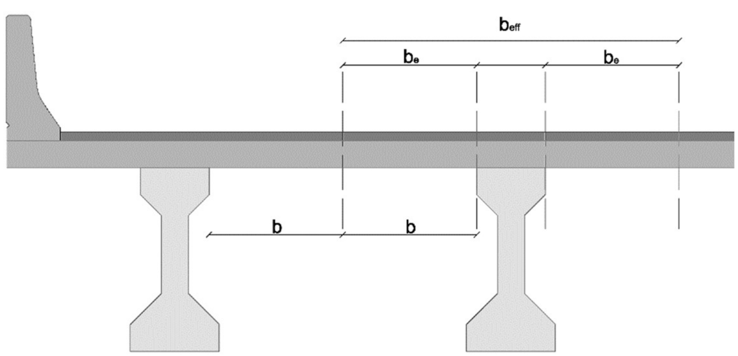


Figure 6.2.7.2 – Graphical Representation of b_{eff} for the bridge

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

$$\Delta = \frac{P \times L^3}{48 \times E_c \times I_e} \text{ where } I_e = I_{cr} + (I_g - I_{cr}) \times \left(\frac{M_{cr}}{M_a}\right)^3 \le I_g$$

$$I_g = \frac{l_n \times t_s^3}{12} = \frac{1992 \times 200^3}{12} = 1.328 \times 10^9\ mm^2$$

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 $A_s = l_n = 1992\ mm^2$

$$B = \frac{l_n}{n \times A_s} = \frac{1992}{7.38 \times 1992} = 0.135$$

$$c = \frac{\sqrt{2 \times d \times B + 1} - 1}{B} = \frac{\sqrt{2 \times 152.02 \times 0.315 + 1} - 1}{0.135} = 40.56\ mm$$

$$I_{cr} = \frac{l_n \times c^3}{3} + n \times A_s \times (d - c)^2 = \frac{1992 \times 40.56^3}{3} + 7.38 \times 1992 \times (152.02 - 40.56)^2 = 226.96 \times 10^6\ mm^4$$

$$M_{cr} = S \times f_{cr} = \frac{1992 \times 200^2}{6} \times 0.4 \times \sqrt{35} \times 10^{-6} = 31.43\ kNm$$

$$I_e = 226.96 \times 10^6 + \left(1.328 \times 10^9 - 226.96 \times 10^6\right) \times \left(\frac{31.43}{40.99}\right)^3 = 723.27 \times 10^6\ mm^4$$

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.*

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:

$$w = d \times h_w \times \gamma_w = 1 \times 0.065 \times 23.5 = 1.5275 \frac{kN}{m}$$

$$M_D^- = \frac{1.5275 \times (2.5)^2}{11} = 0.868 \frac{kNm}{m}$$

$$M_D^+ = \frac{1.5275 \times (2.5)^2}{16} = 0.5967 \frac{kNm}{m}$$

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:

$$a = \frac{A_s \times \Phi_s \times f_y}{\left(0.85 \times \Phi_c \times f'_c \times b\right)} = \frac{1000 \times 0.9 \times 400}{0.85 \times 0.75 \times 35 \times 1000} = 16.13\ mm$$

$$M_r = A_s \times \Phi_s \times f_y \times \left(d - \frac{a}{2}\right) = 1000 \times 0.9 \times 400 \times \left(167.02 - \frac{16.13}{2}\right) \times 10^{-6} = 57.22\ kNm/m$$

$M_r(57.22\ kNm/m) > M_f(56.86\ kNm/m)\ OK$

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

$$Bottom\ distribution\ reinforcement\ in\ \% = \frac{125}{\sqrt{S}} < 66.7\%\ \ S\ in\ meters$$

$$Bottom\ distribution\ reinforcement\ in\ \% = 66.7\%$$

$$A_{s,bottom} = \frac{2}{3} \times Area\ per\ meter\ of\ Bottom\ transverse\ reinforcement = 667\ mm^2 / m$$

$$Number\ of\ Bars = \frac{667}{200} = 3.33$$

$$Spacing = \frac{1000}{3.33} = 300\ mm$$

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***.

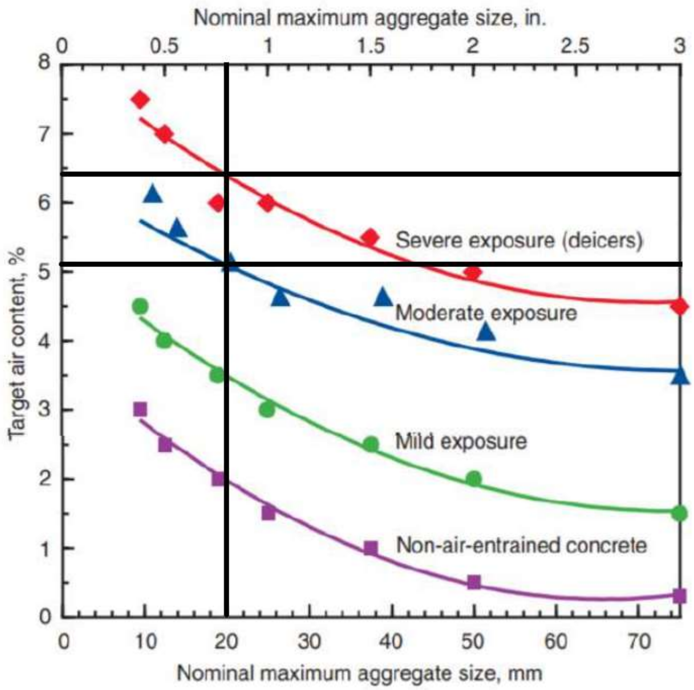


Figure 7.3.3.1 – Relationship between exposure type, aggregate size and air content

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

Concrete construction	Slump, mm (in.) Maximum*	Slump, mm (in.) 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.

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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/m^3
Cement for girder = 160 / 0.27 = Approximately 600 kg/m^3

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/m^3 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.

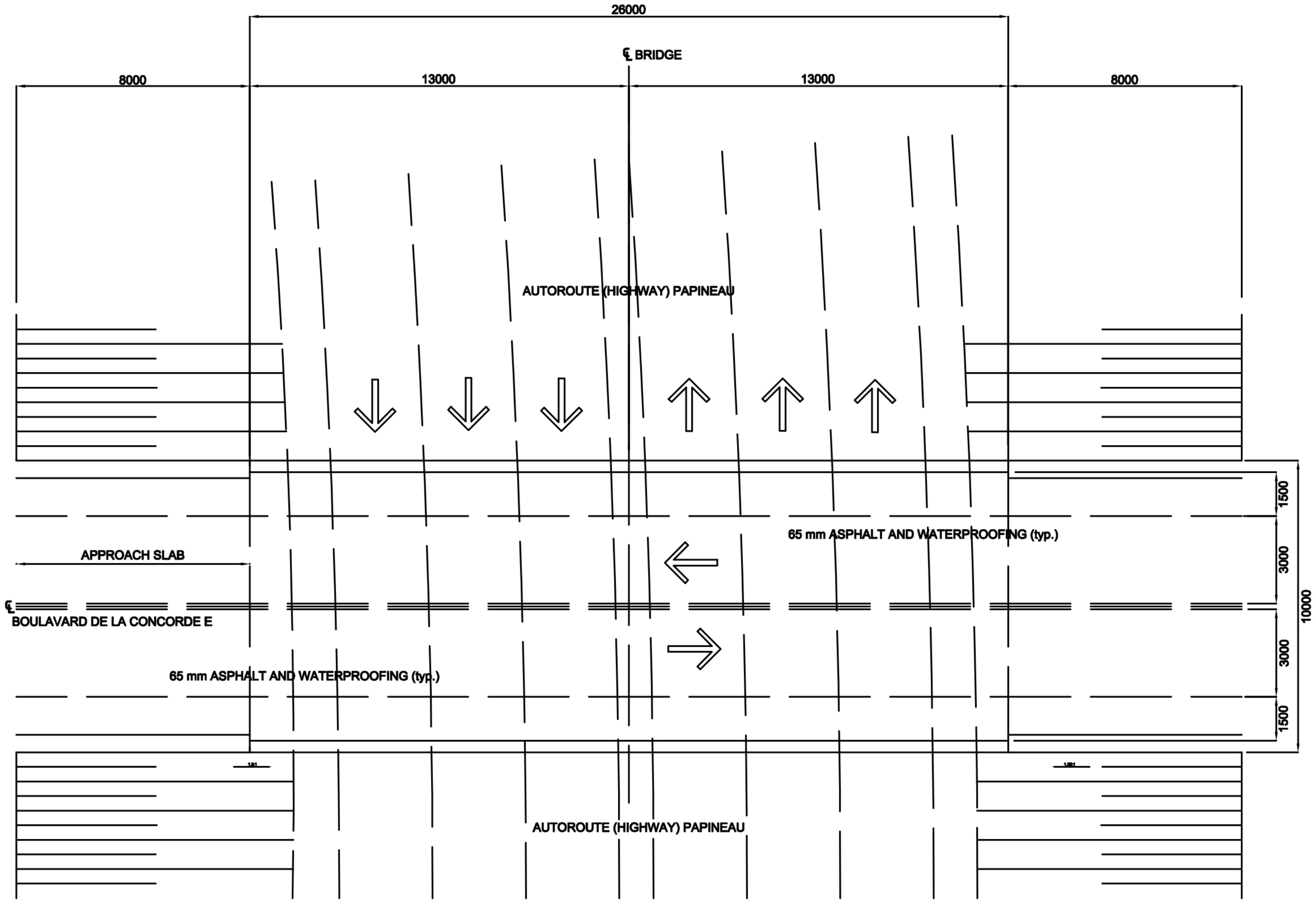
-Daravair, Product Data Sheet. Available: https://gcpat.hk/sites/hk.gcpat.com/files/2017-08/Daravair.pdf

-The Constructor, “Effects of Air Entrainment on Concrete Strength,” Available: https://theconstructor.org/concrete/effect-air-entrainment-concrete-strength/8427/

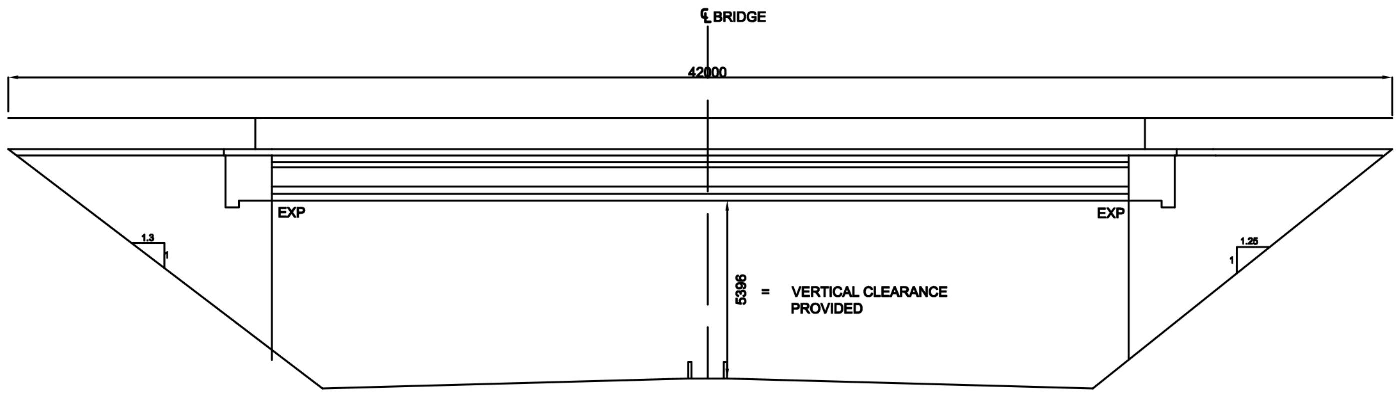
-S.H.Kosmatka and M.L.Wilson (2011). Design and Control of Concrete Mixtures: The guide to applications, methods, and materials - Portland Cement Association

-American Society for Testing and Materials (2014). Tolerances in Slump or Slump Flow - ASTM MNL49-2ND-EB

APPENDIX
DESIGN DRAWINGS



PLAN



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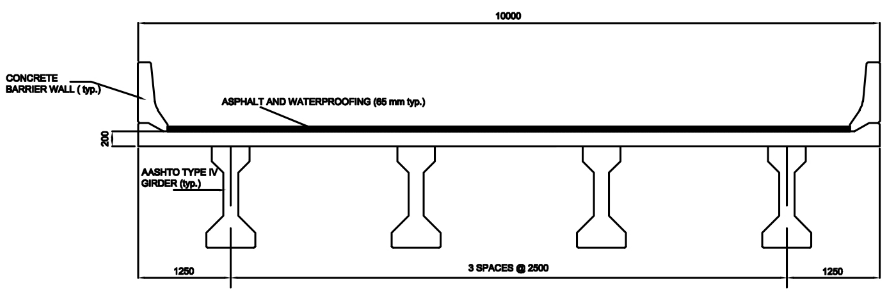

GENERAL NOTES

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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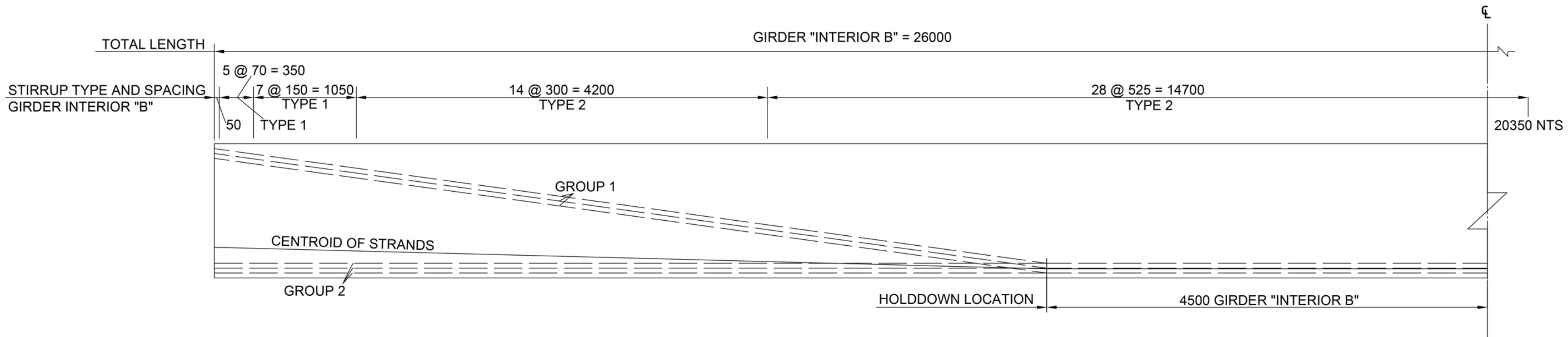
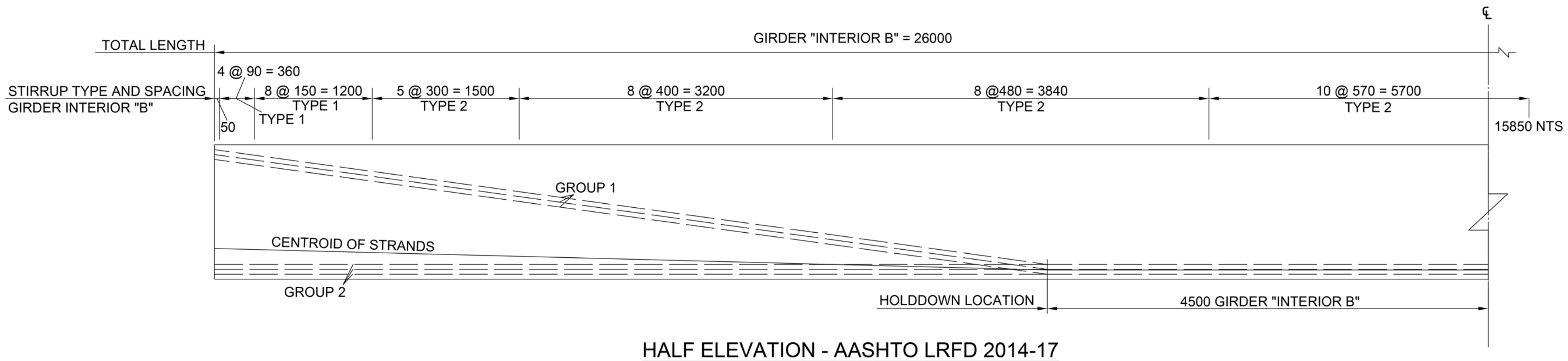
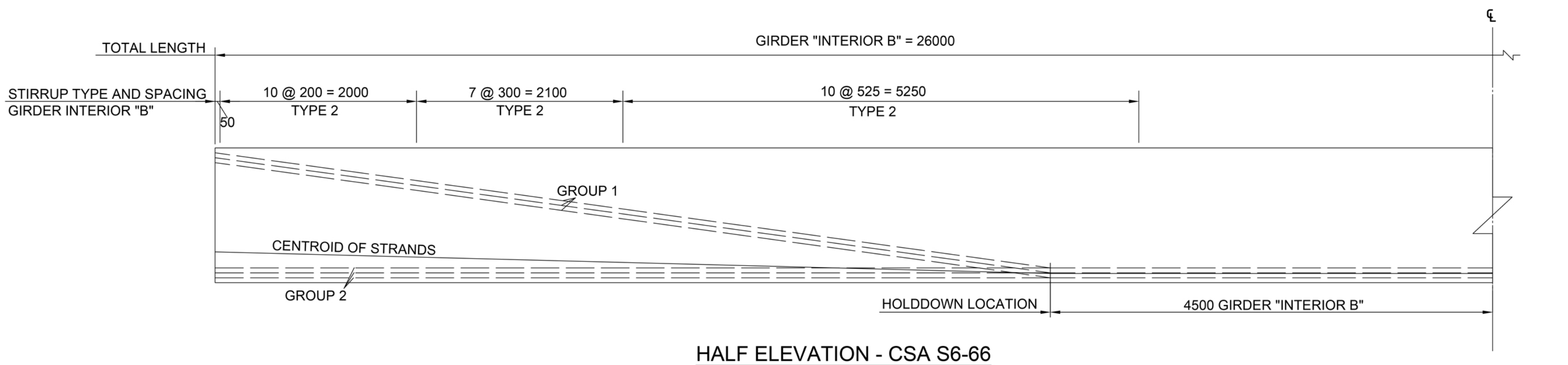


HALF ELEVATION - CSA S6-14



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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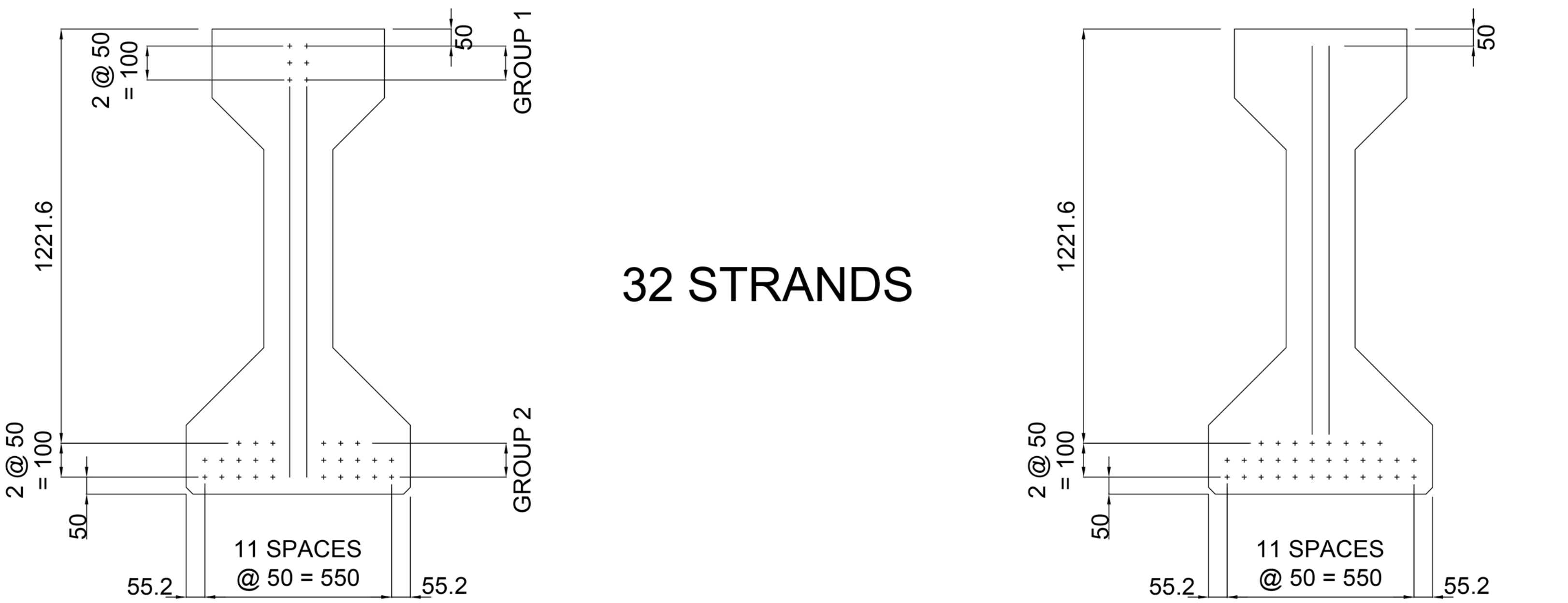
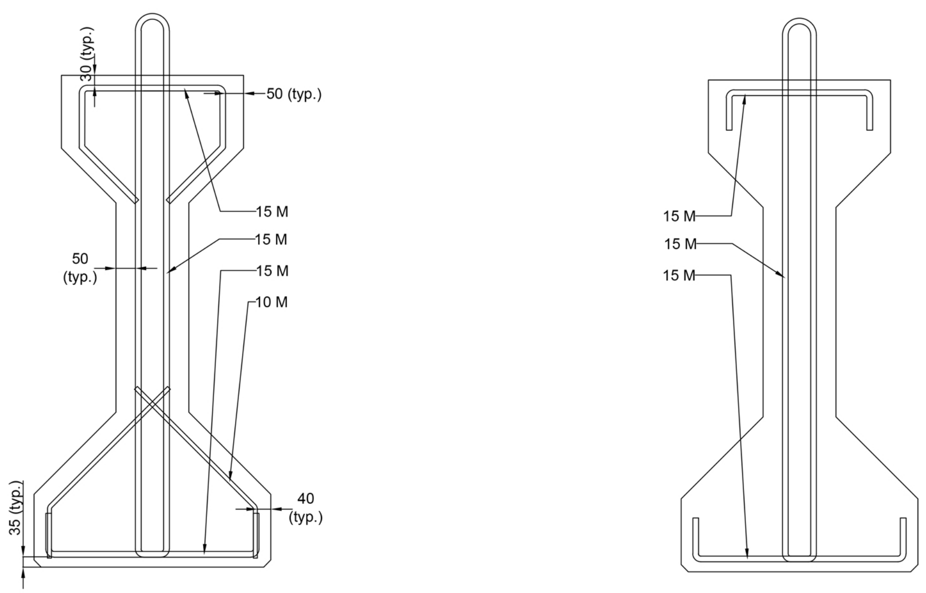


END OF GIRDER

CENTER OF GIRDER

STRAND PATTERN



TYPE 1

TYPE 2

STIRRUP DETAILS

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: STRAND PATTERN AND STIRRUP DETAILS
Drawn by: Koral Eren
Date: April 2020
Drawing Number: 3

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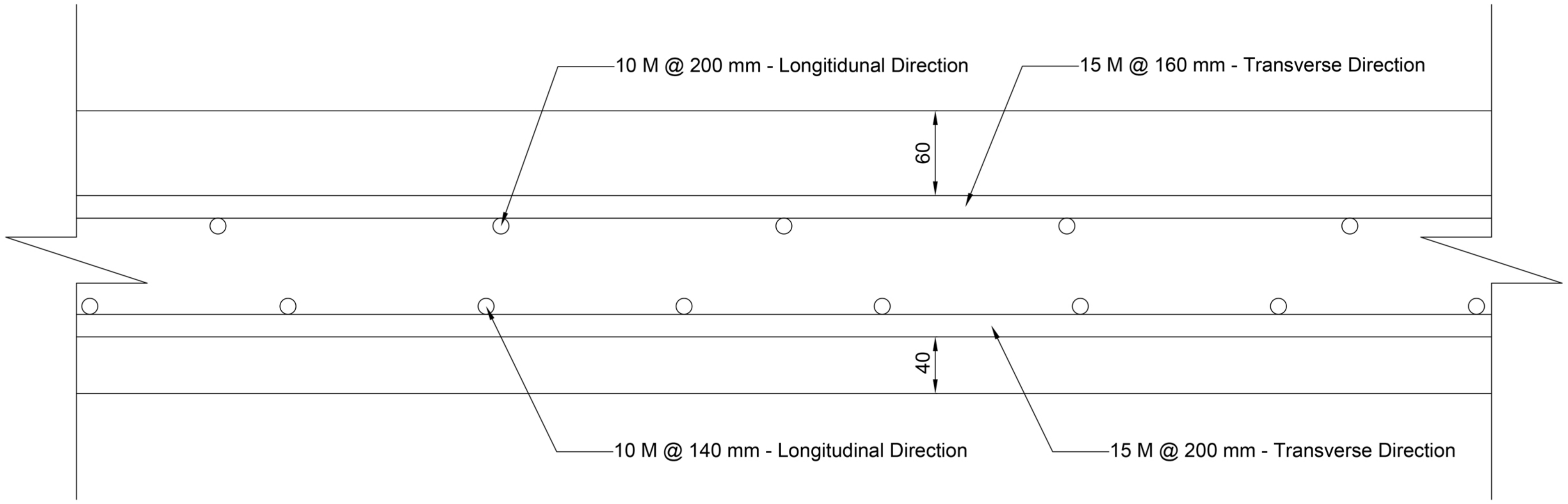
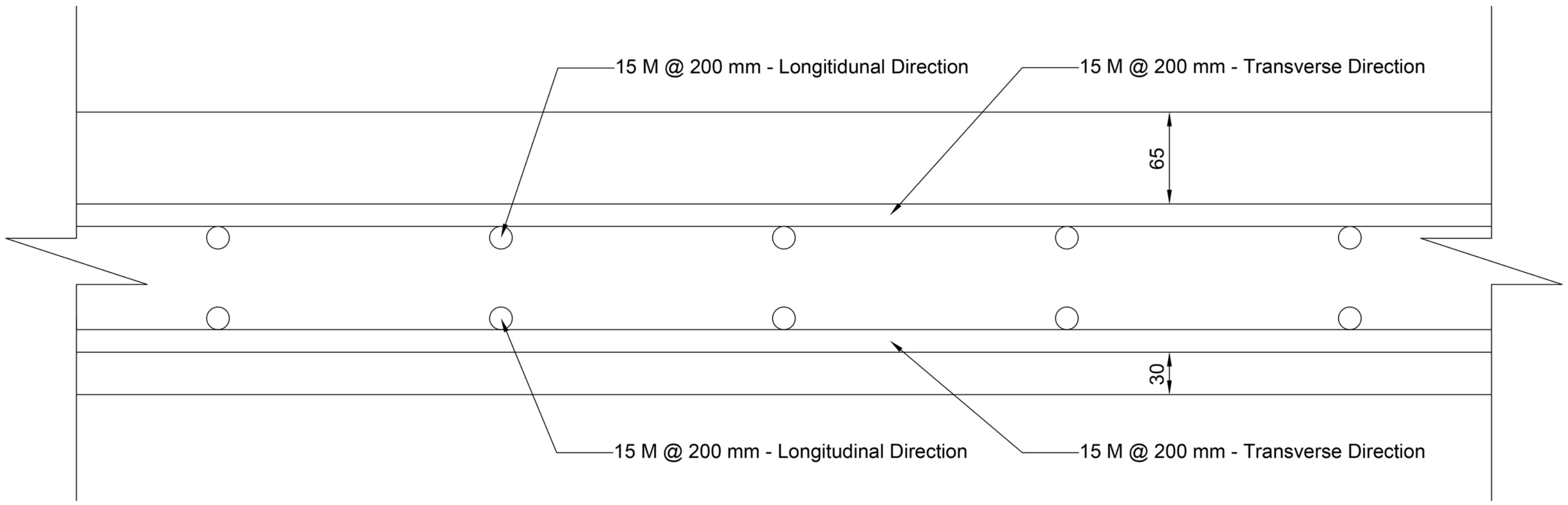
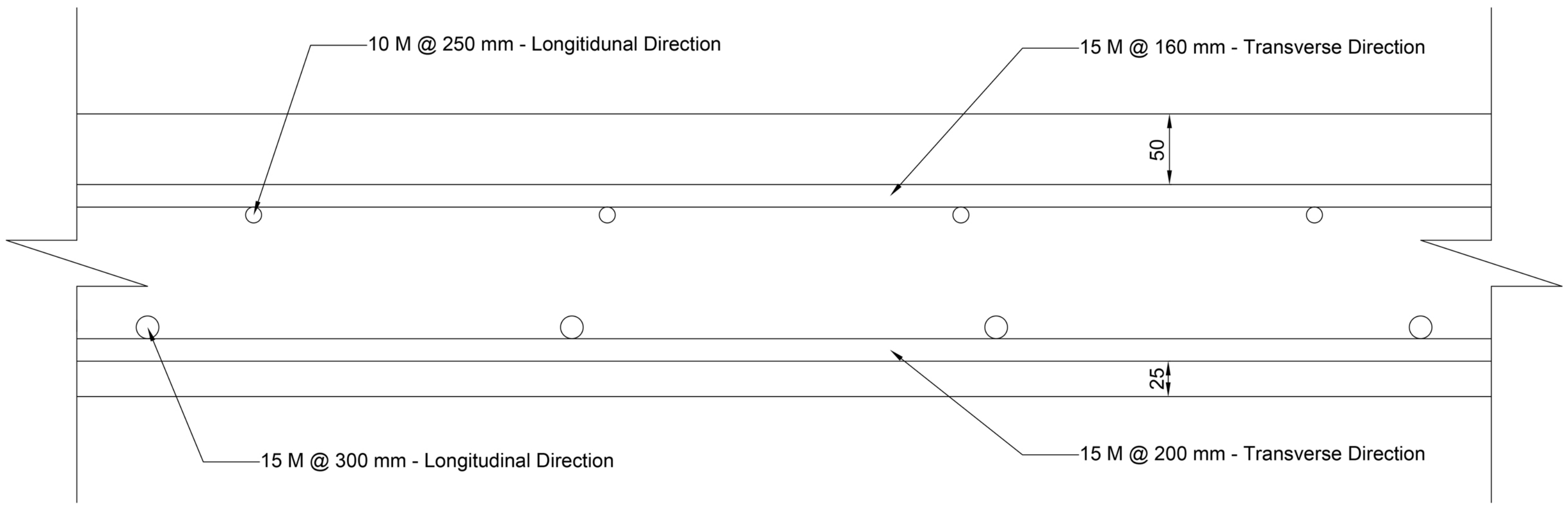


BRIDGE DECK CROSS-SECTION - CSA S6-14 rev.17



BRIDGE DECK CROSS-SECTION - AASHTO LRFD 2014-17



BRIDGE DECK CROSS-SECTION - CSA S6-66

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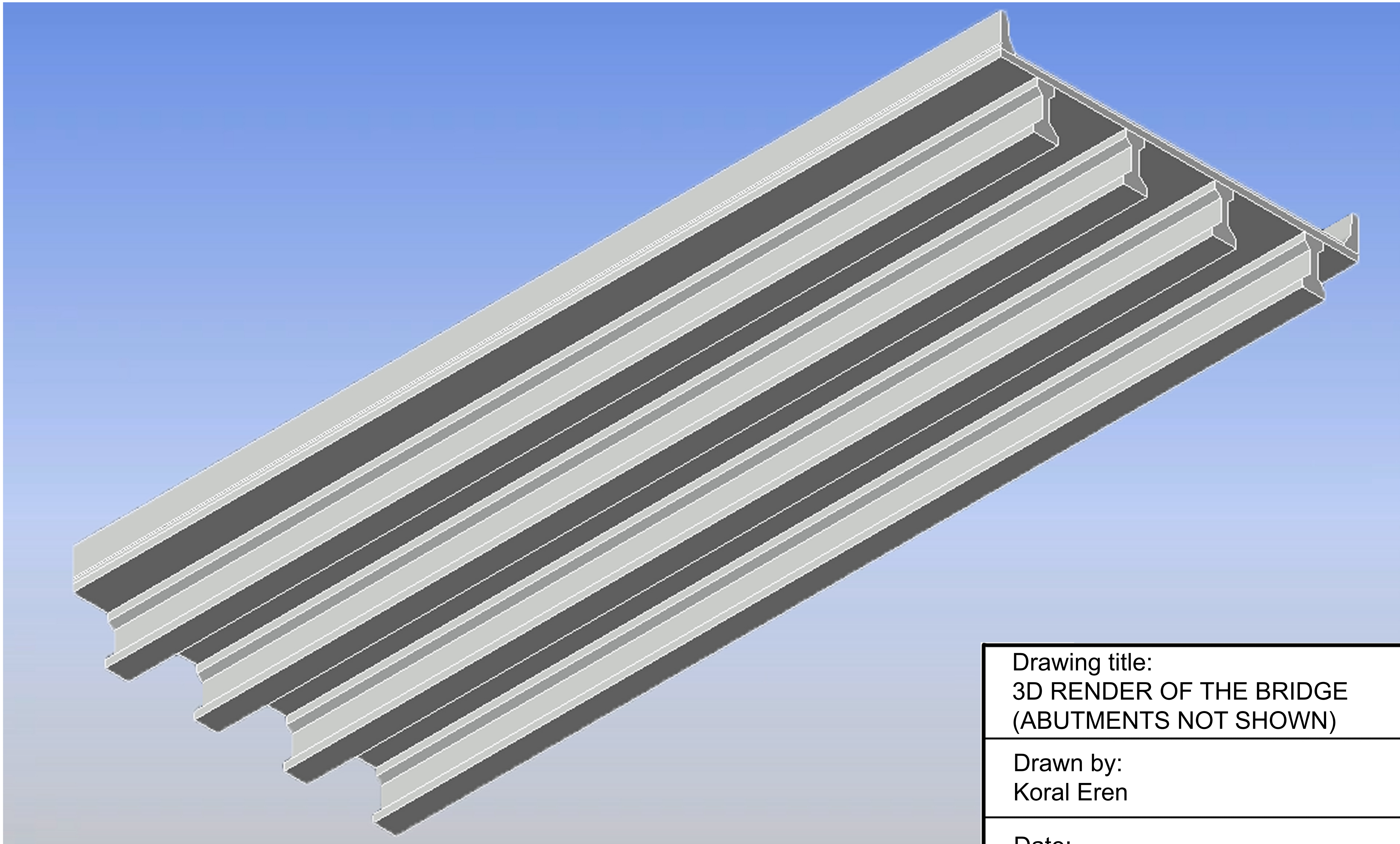
Date: April 2020
Drawing Number: 4



Drawing title:
3D RENDER OF THE BRIDGE
(ABUTMENTS NOT SHOWN)

Drawn by:
Koral Eren

Date:
April 2020

Drawing Number:
5

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