K & B

# PRESTRESSED CONCRETE BRIDGE DESIGN


KORAL SELCUK EREN - 1001677296
BIKRAMJEET SINGH TOOR - 1002055955

WINTER 2020

UNIVERSITY OF TORONTO | Civil + Mineral Engineering

UNIVERSITY OF
TORONTO

# 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

UNIVERSITY OF TORONTO

# 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.

K & B

UNIVERSITY OF
TORONTO

# Overall Table of Contents



K & B

# 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

## Table of Contents

UNIVERSITY OF TORONTO

## 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]

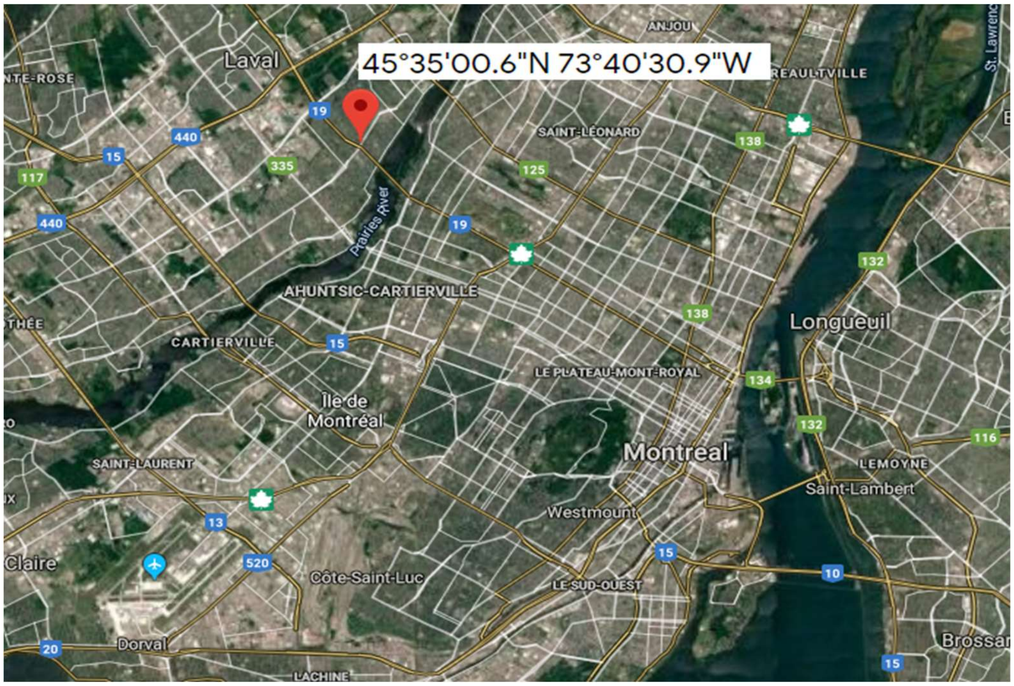



*Figure 2.2.1.1 - Location of the collapsed overpass*

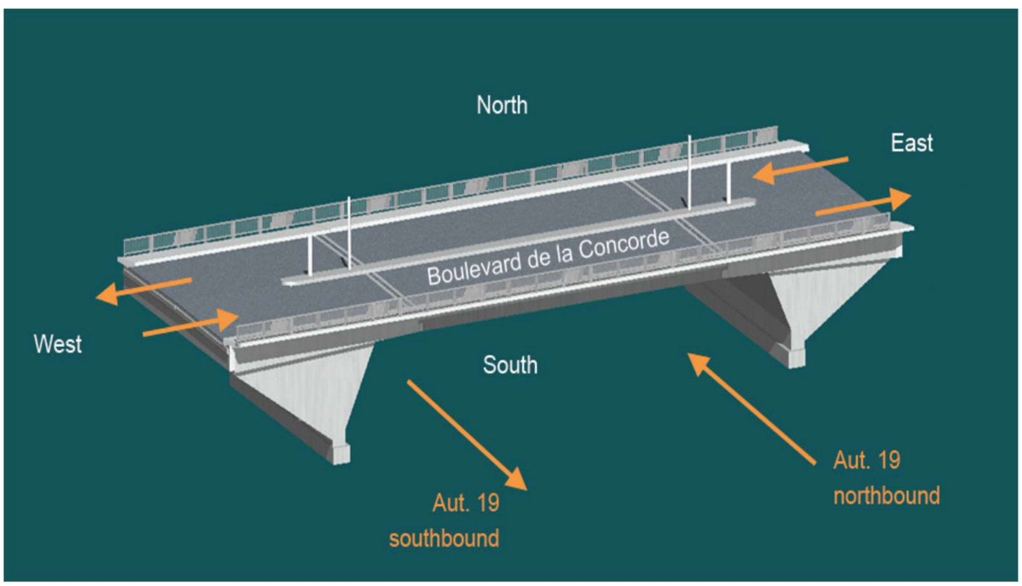



*Figure 2.2.1.2 - General Perspective of the collapsed overpass [1]*

*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]:

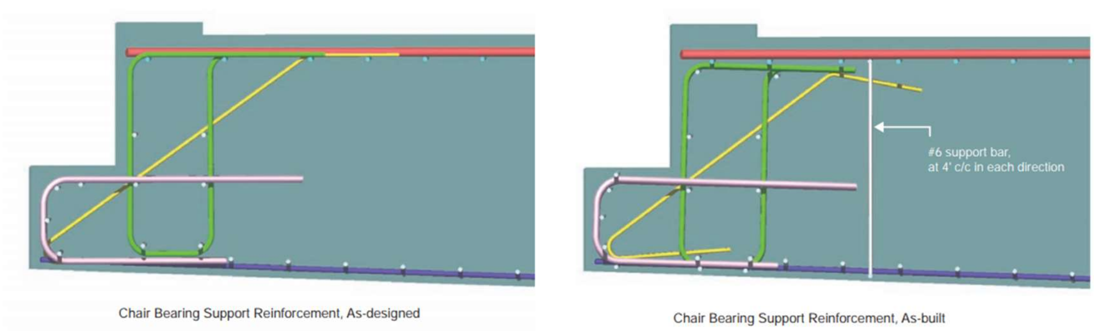



*Figure 2.2.2.1 - Chair Bearing Support Reinforcement*

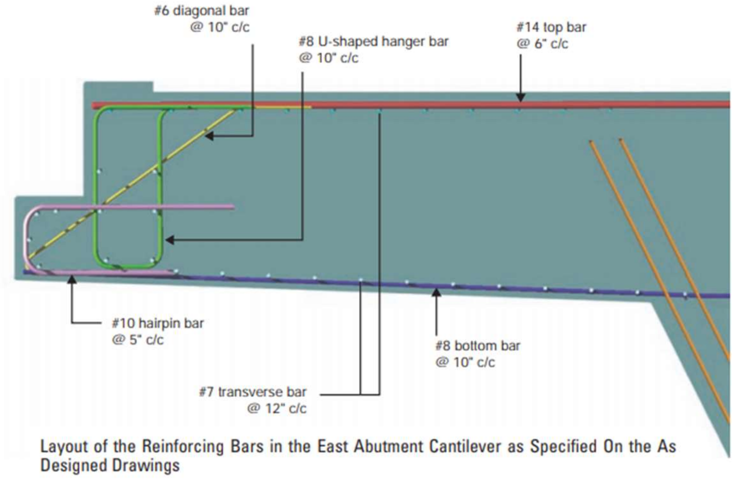



*Figure 2.2.2.2 - Layout of the Reinforcing Bars in the East Abutment Cantilever*

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.

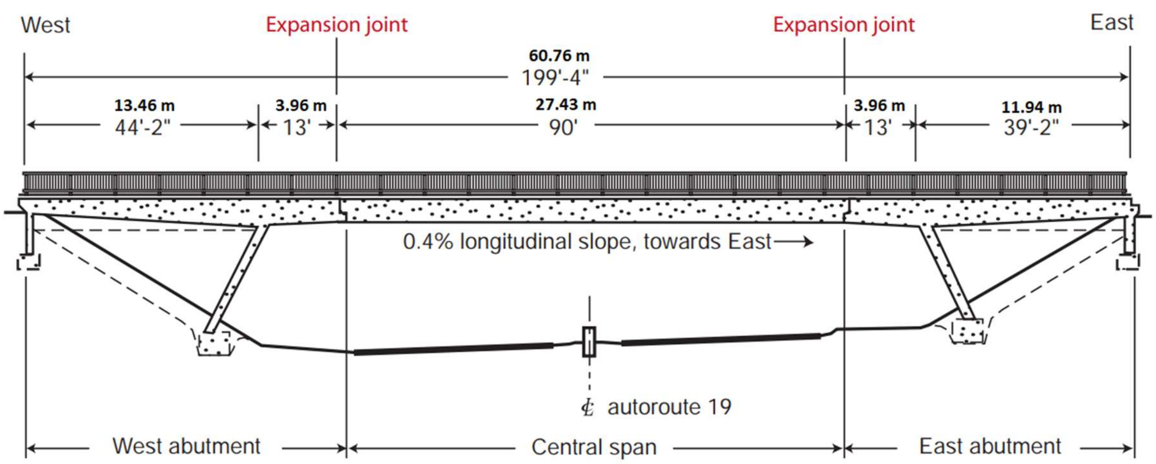



*Figure 2.2.2.3 - Elevation View of the de la Concorde Overpass from Autoroute (Highway) 19 [3]*

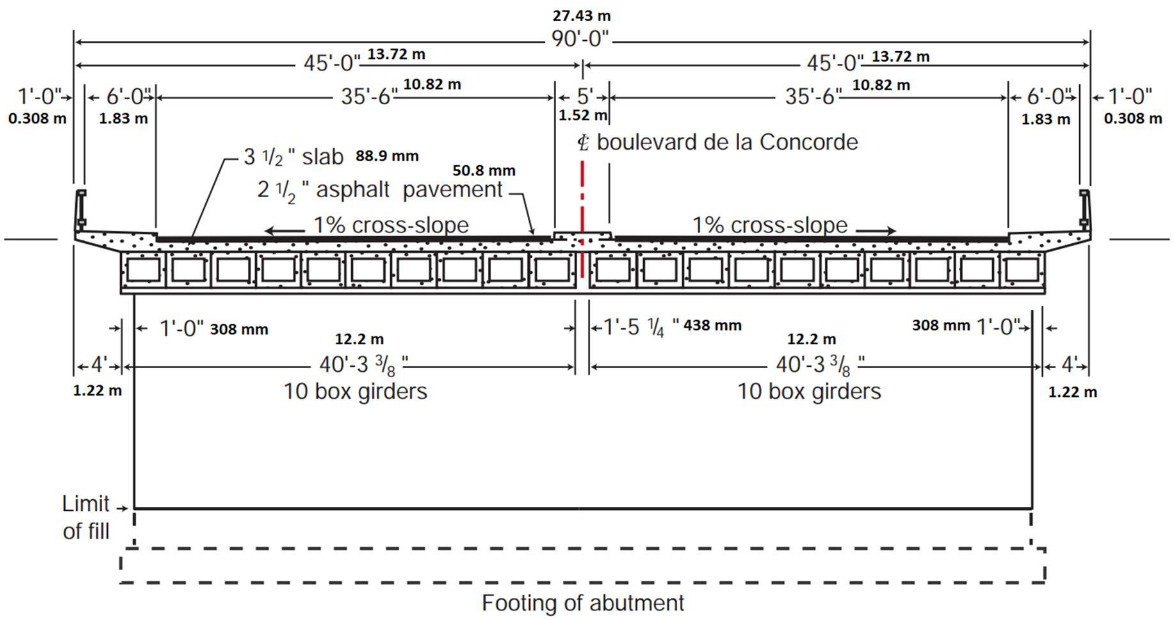



*Figure 2.2.2.4 - Cross-section of the de la Concorde Overpass from Autoroute (Highway) 19 [3]*

### 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]*

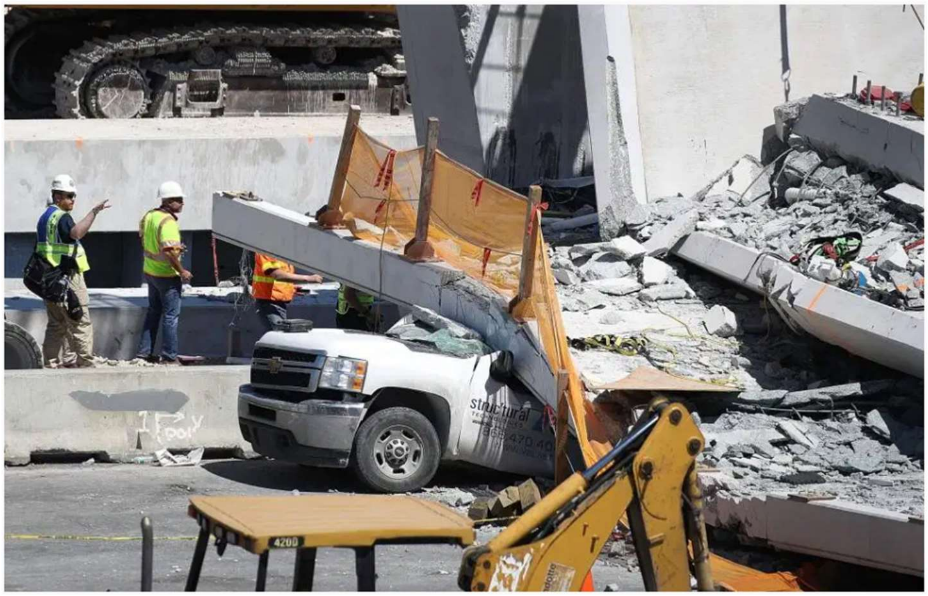



*Figure 2.3.1.2 - Ongoing Investigation after Bridge Collapse [4]*

### 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.

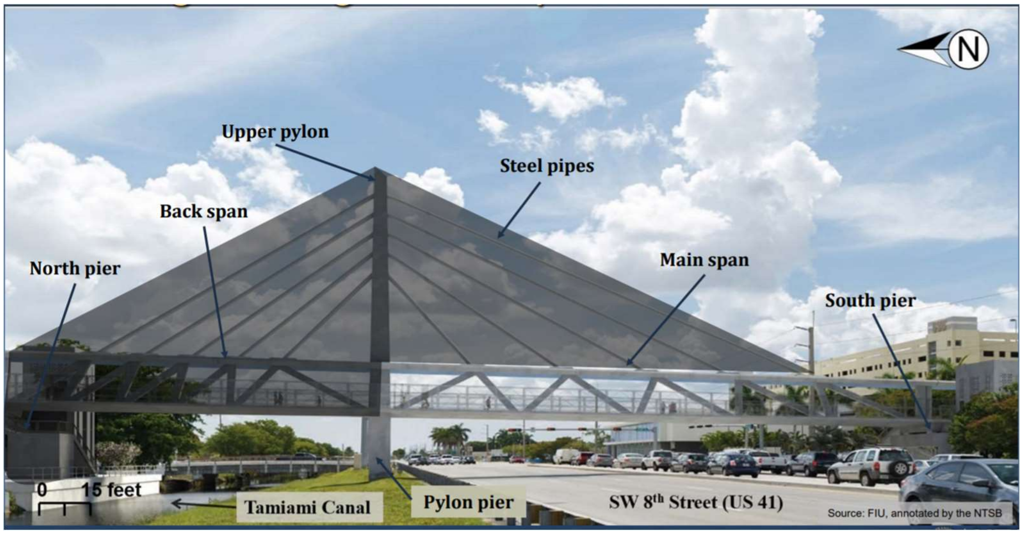



*Figure 2.3.2.1 - Bridge Design Concept [5]*

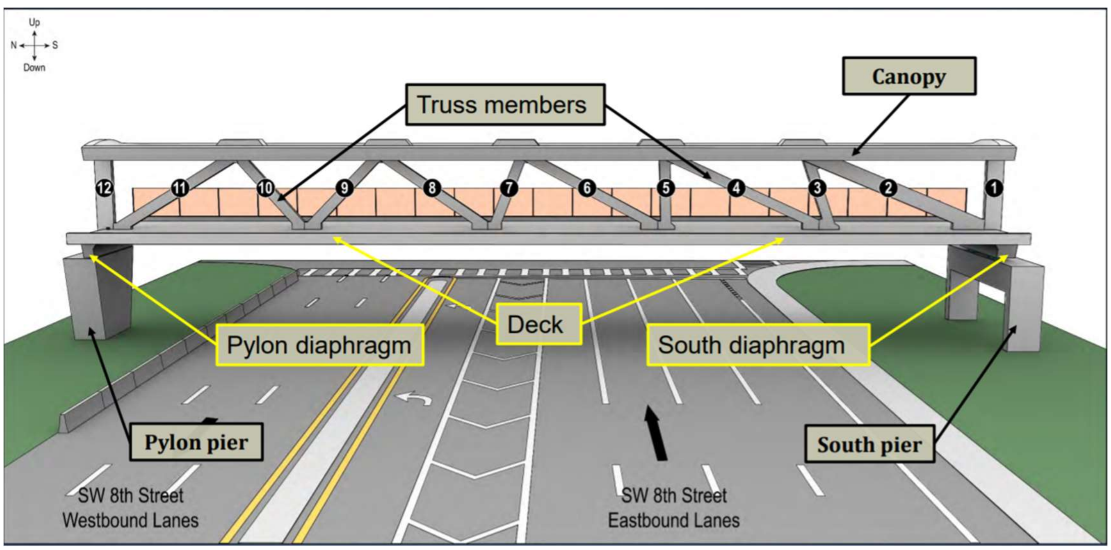



*Figure 2.3.2.2 - Pedestrian Bridge Main Span [5]*

### 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].

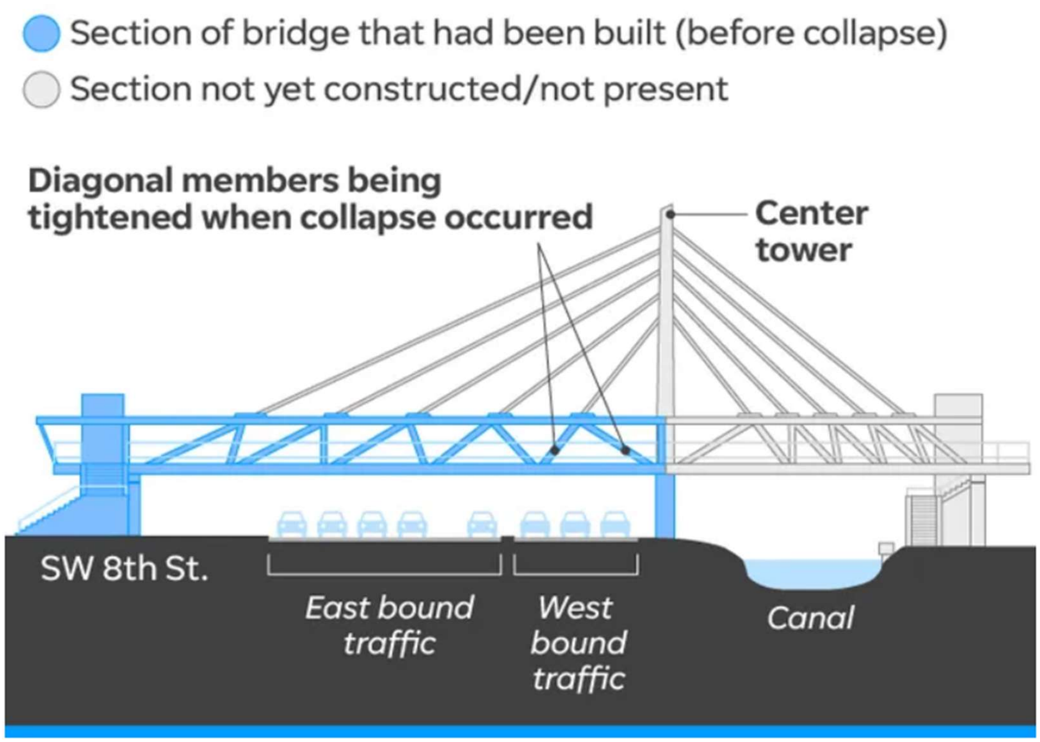



*Figure 2.3.3.1 - Profile of bridge before it collapsed [7]*

### 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].

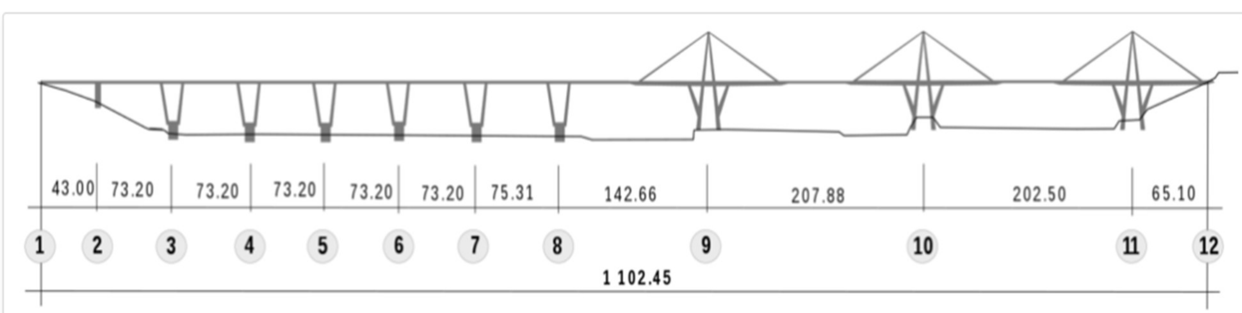



*Figure 2.4.2.1 - Perspective Drawing of Morandi Bridge [16]*

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].

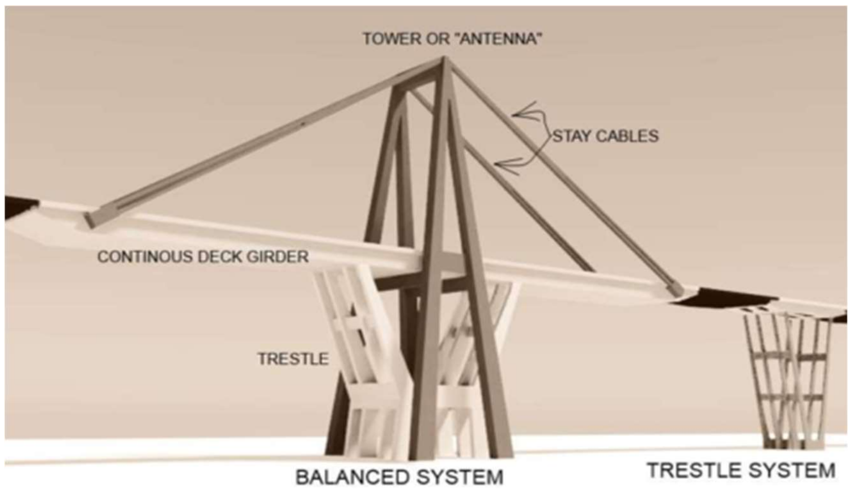



*Figure 2.4.2.2 - Trestle and Balanced System [16]*

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.

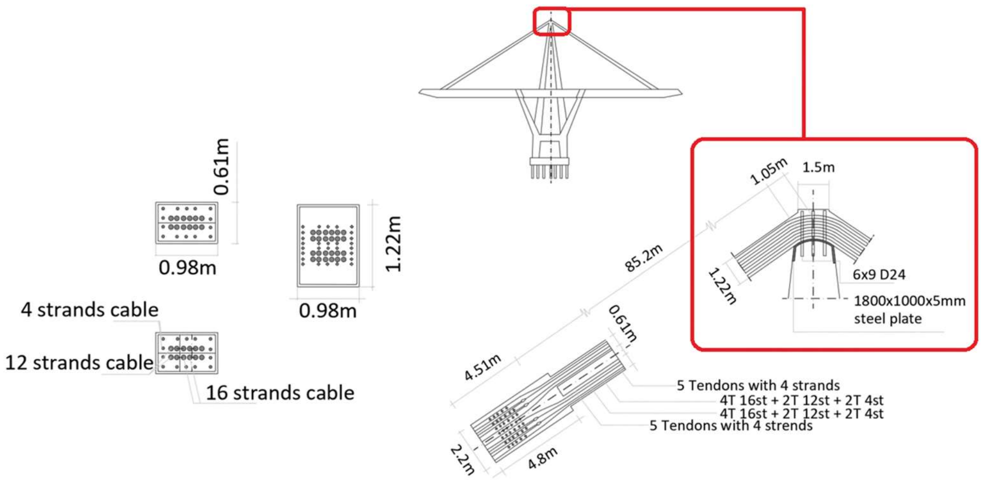



*Figure 2.4.3.1 - View of the stay cable system [17]*

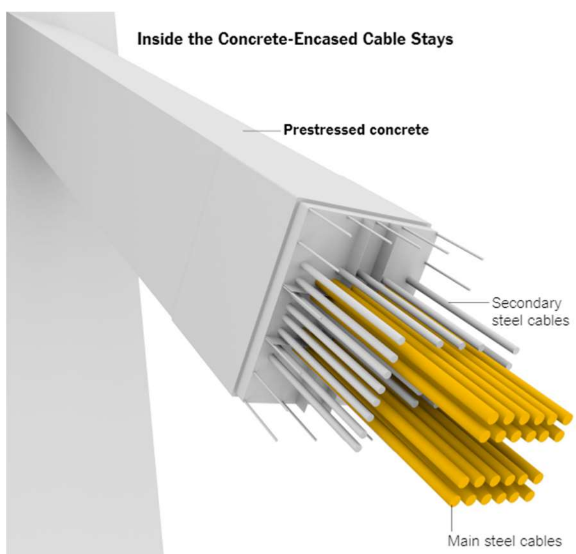



*Figure 2.4.3.2 - 3D View of the stay cable system [18]*

-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]*

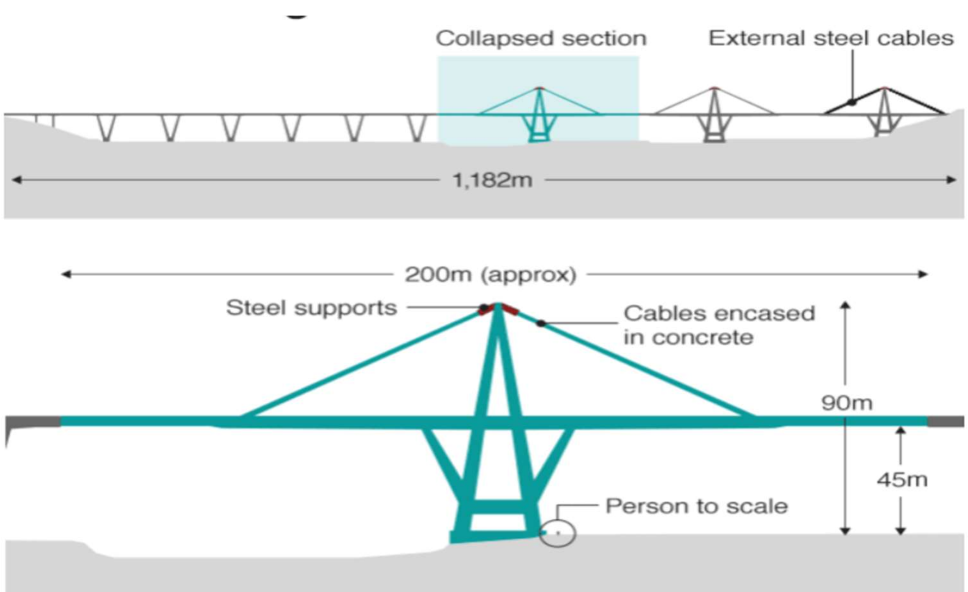



*Figure 2.4.3.4 - Section of the bridge that collapsed [13]*

### 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

[1]. Pierre Marc Johnson, Amand Couture, Roger Nicolet, “Commission of Inquiry into the Collapse of a Portion of the de la Concorde Overpass”, October 3, 2006 – October 15, 2007 [Online]. Available: http://cip.management.dal.ca/publications/report_eng%20concorde%20overpass%20montreal.pdf [Accessed: 04- Apr- 2020].

[2]. Wikipedia, “De la Concorde overpass collapse” [Online]. Available: https://en.wikipedia.org/wiki/De_la_Concorde_overpass_collapse [Accessed: 04- Apr- 2020].

[3]. De La Concorde Overpass Court Materials, Exhibit COM-2

[4]. "Where is the Florida pedestrian bridge collapse?", *ajc*, 2018. [Online]. Available: https://www.ajc.com/news/where-the-florida-pedestrian-bridge-collapse/cCwfjyJZiy2ITNlJufffVP/. [Accessed: 04- Apr- 2020].

[5]. M. Rodriguez, "Pedestrian Bridge In The Works For FIU Students To Cross 8th Street", *Miami.cbslocal.com*, 2017. [Online]. Available: https://miami.cbslocal.com/2017/08/22/pedestrian-bridge-in-the-works-for-fiu-students-to-cross-busy-8th-street/. [Accessed: 04- Apr- 2020].

[6]. "FIU Pedestrian Bridge Construction Using Innovative Approach", *Miami.cbslocal.com*, 2018. [Online]. Available: https://miami.cbslocal.com/2018/03/10/fiu-pedestrian-bridge-construction/. [Accessed: 04- Apr- 2020].

[7]. S. Dwilson, "FIGG Bridge Engineers: 5 Fast Facts You Need to Know | Heavy.com", *Heavy.com*, 2018. [Online]. Available: https://heavy.com/news/2018/03/figg-bridge-collapse-florida-fiu-history/. [Accessed: 04- Apr- 2020].

[8]. R. Accetta, "Florida International University (FIU) Pedestrian Bridge Collapse, Miami, Florida", *Ntsb.gov*, 2018. [Online]. Available: https://www.ntsb.gov/news/events/Documents/2019-HWY18MH009-BMG-01-Investigator-In-Charge.pdf. [Accessed: 04- Apr- 2020].

[9]. "Preliminary Report- Highway: Collapse of Pedestrian Bridge Under Construction Miami, Florida (HWY18MH009)", *Ntsb.gov*, 2018. [Online]. Available: https://www.ntsb.gov/investigations/AccidentReports/Pages/HWY18MH009-prelim-.aspx. [Accessed: 04- Apr- 2020].

[10]. A. Ware, "Collapsed bridge meant to last 100 years", 2018. [Online]. Available: https://www.wpbf.com/article/collapsed-bridge-meant-to-last-100-years/19455044. [Accessed: 04- Apr- 2020].

[11]. https://www.11alive.com/article/news/nation-now/miami-bridge-that-collapsed-was-a-truss-design-despite-the-cosmetic-tower-support-cables/465-73d5b632-b049-4e81-a388-415b3582a599

[12]. https://www.youtube.com/watch?v=hBjntrebxj8

[13]. M. Ayub, "Investigation of March 15, 2018 Pedestrian Bridge Collapse at Florida International University, Miami, FL", *U.S Department of Labor*, 2019. [Online]. Available: https://www.osha.gov/doc/engineering/pdf/2019_r_03.pdf. [Accessed: 04- Apr- 2020].

[14]. R. Sumwalt, “Board Meeting: Pedestrian Bridge Collapse Over SW 8th Street, Miami, Florida, March 15, 2018 - Closing Statement.” [Online]. Available: https://www.ntsb.gov/news/speeches/RSumwalt/Pages/sumwalt-20191022c.aspx. [Accessed: 04-Apr-2020].

[15]. “What caused the Genoa bridge collapse – and the end of an Italian national myth?,” 2019. [Online]. Available: https://www.theguardian.com/cities/2019/feb/26/what-caused-the-genoa-morandi-bridge-collapse-and-the-end-of-an-italian-national-myth. [Accessed: 04-Apr-2020].

[16]. E. Pollock, “Italy’s Morandi Bridge Collapse—What Do We Know?,” *engineering.com*, 22-Aug-2018. [Online]. Available: https://www.engineering.com/BIM/ArticleID/17517/Italys-Morandi-Bridge-CollapseWhat-Do-We-Know.aspx. [Accessed: 04-Apr-2020].

[17]. V. Hache, “Genoa's Morandi bridge disaster 'a tragedy waiting to happen',” 15-Aug-2018. [Online]. Available: https://www.france24.com/en/20180815-italy-genoa-morandi-bridge-disaster-structural-problems. [Accessed: 04-Apr-2020].

[18]. V. Oliphant, “Genoa bridge collapse: How tall was Ponte Morandi motorway bridge? When was it built?,” 14-Aug-2018. [Online]. Available: https://www.express.co.uk/news/world/1003224/Genoa-bridge-collapse-news-Ponte-Morandi-bridge-when-built-how-tall-Italy-news. [Accessed: 04-Apr-2020].

[19]. F. Marzullo, “Polcevera viaduct: a premature end of a milestone of the Italian engineering heritage,” *Monodaq*, 01-Dec-2018. [Online]. Available: https://www.monodaq.com/applications/polcevera-viaduct/. [Accessed: 04-Apr-2020].

[20]. “Post-collapse analysis of Morandi’s Polcevera viaduct in Genoa Italy,” 2020. [Online]. Available: https://link.springer.com/article/10.1007/s13349-019-00370-7/figures/4?shared-article-renderer. [Accessed: 04-Apr-2020].

[21]. J. Glanz and G. Pianigiani, “Genoa Bridge Collapse: The Road to Tragedy,” 06-Sep-2018. [Online]. Available: https://www.nytimes.com/interactive/2018/09/06/world/europe/genoa-italy-bridge.html. [Accessed: 04-Apr-2020].

[22]. “Lessons Learned From the Italian Morandi Bridge Collapse,” *bridge masters*, 30-Aug-2018. [Online]. Available: https://bridgemastersinc.com/italian-morandi-bridge-collapse/. [Accessed: 04-Apr-2020].

# 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.

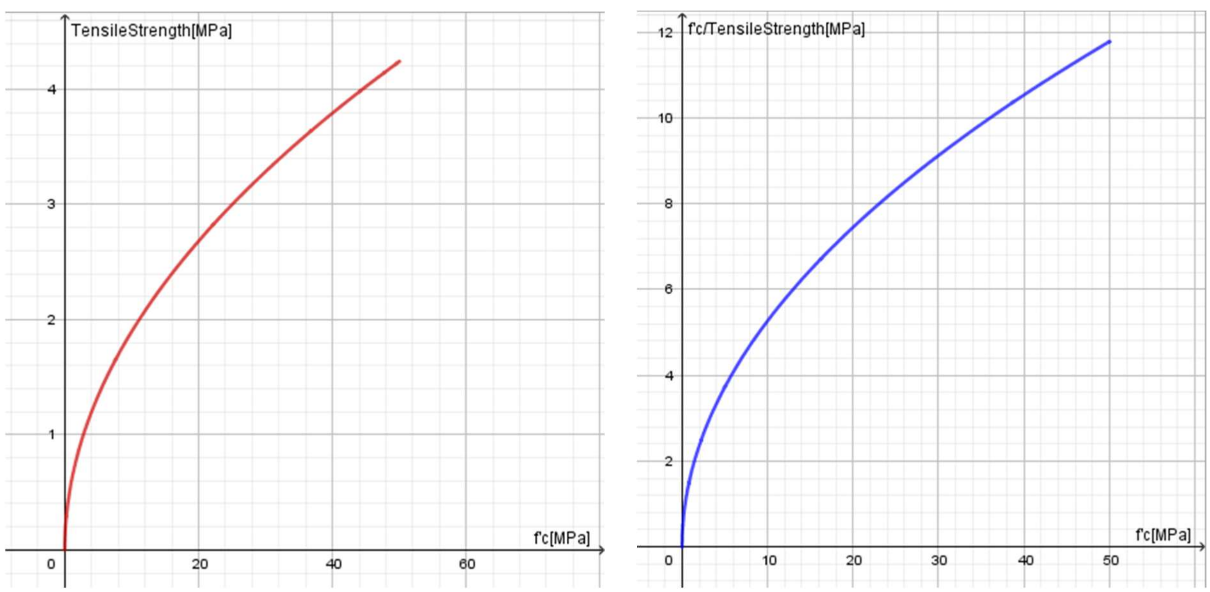



*Figure 3.2.4.3* – (Left) Tensile strength of concrete versus its maximum cylindrical compressive strength and (Right) Concrete maximum cylindrical compressive strength / tensile strength of concrete

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:

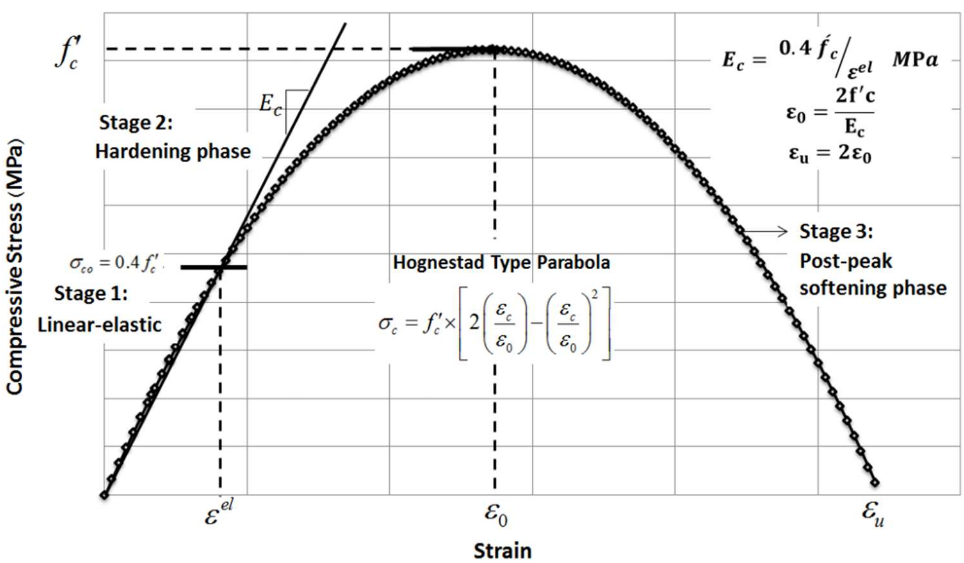
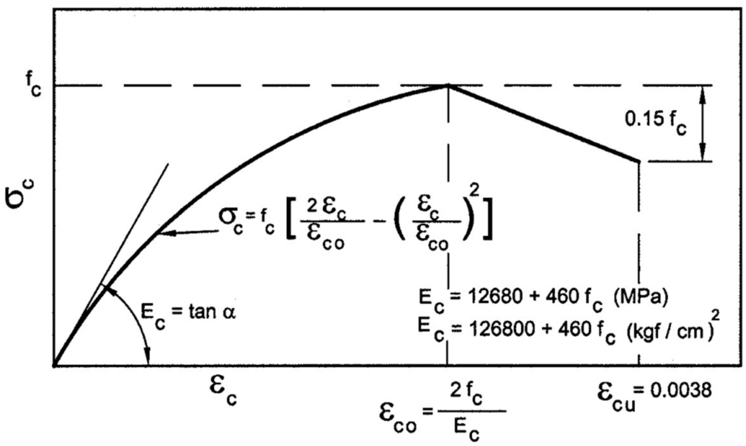



*Figure 3.2.4.4 – Concrete Uniaxial Stress-Strain Diagram (Theoretical) [10]*

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).



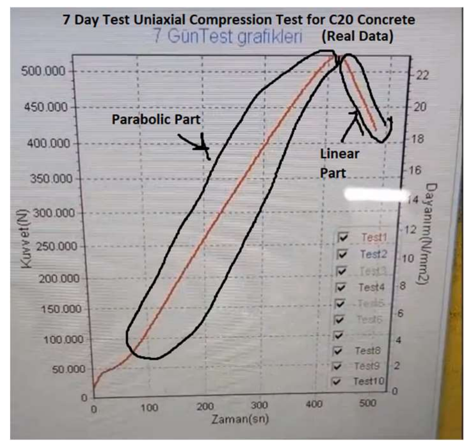




*Figure 3.2.4.5 – (Left): An example of Modified Hognestad Parabola From Turkish BC 2018 and (Right) Real test data from a test done by an assistant in Turkey*

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 $10 \times 10^{-6}$ / °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* $6 \times 10^{-6}$ / °C *to* $13 \times 10^{-6}$ / °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:

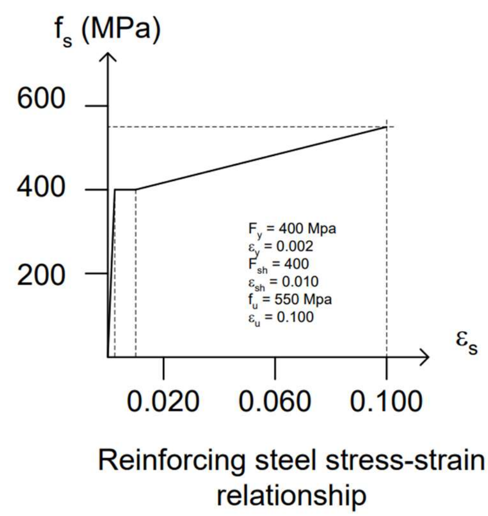



*Figure 3.2.5.1 – Reinforcing Steel-Strain Relationship*

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.

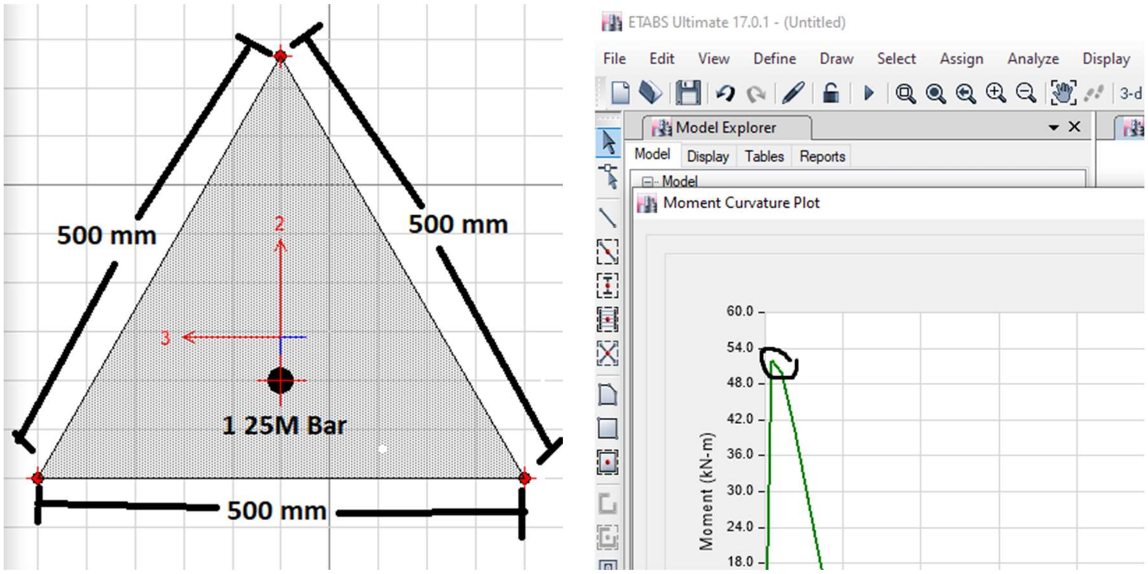



*Figure 3.2.5.3 – CSI ETABS software*

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

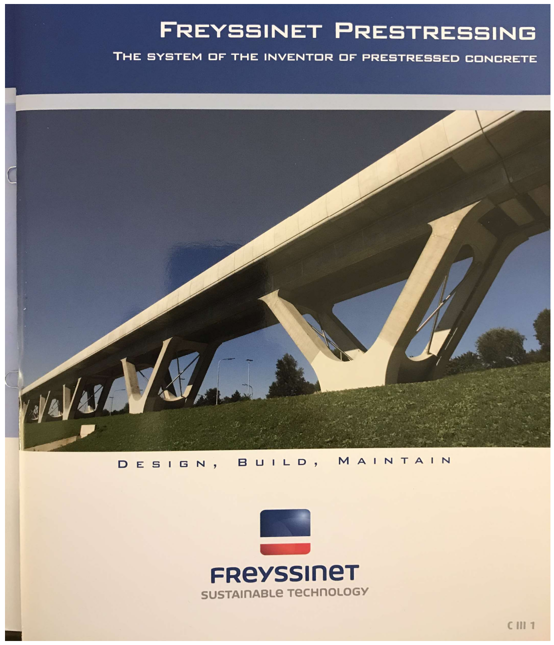
<u>Historical Background of Prestressed Concrete:</u>

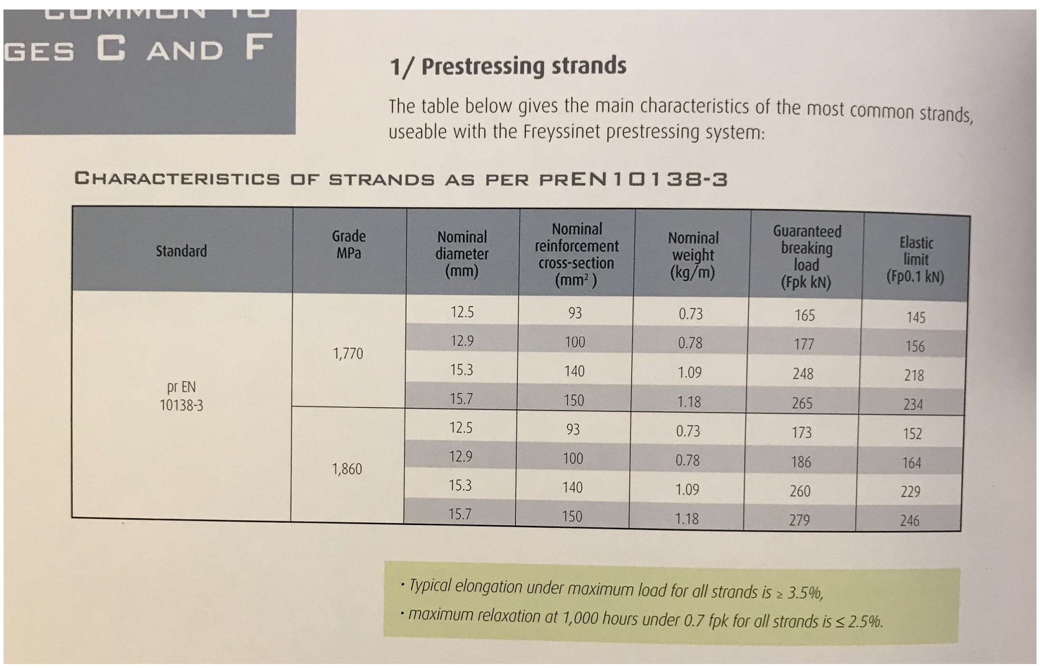
Prestressing attempts started back in 1872 when an engineer from California, US got a patent for a simple prestressing system to construct beams or arches part by part. [11]. In 1888, an engineer from Germany named Doehring got a patent for prestressing slabs using some sort of metal wires [11]. Both 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)*

<u>Prestressing Steel:</u>

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 ($mm^2$) | 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]*

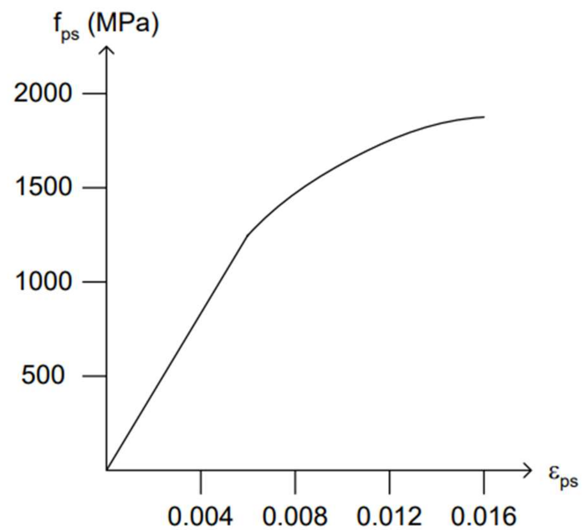



*Figure 3.2.6.3 – Stress-Strain Curve of a sample prestressing steel*

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.

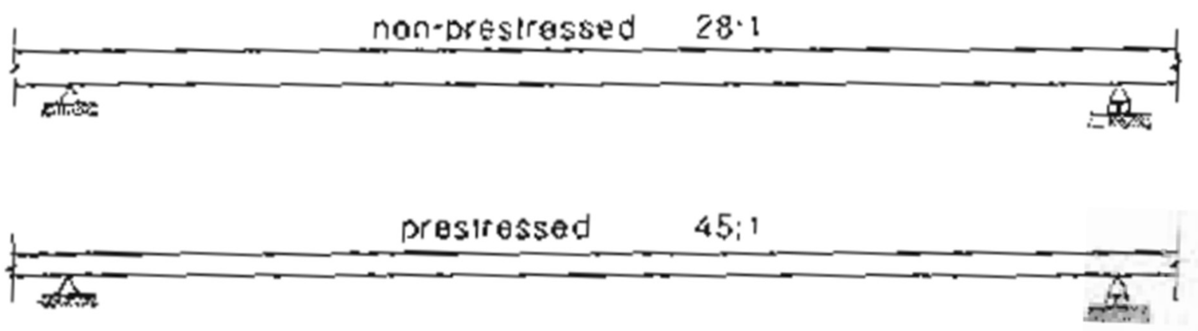



*Figure 3.2.6.4 – Typical One-Way Span to depth ratios of beams [15]*

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.

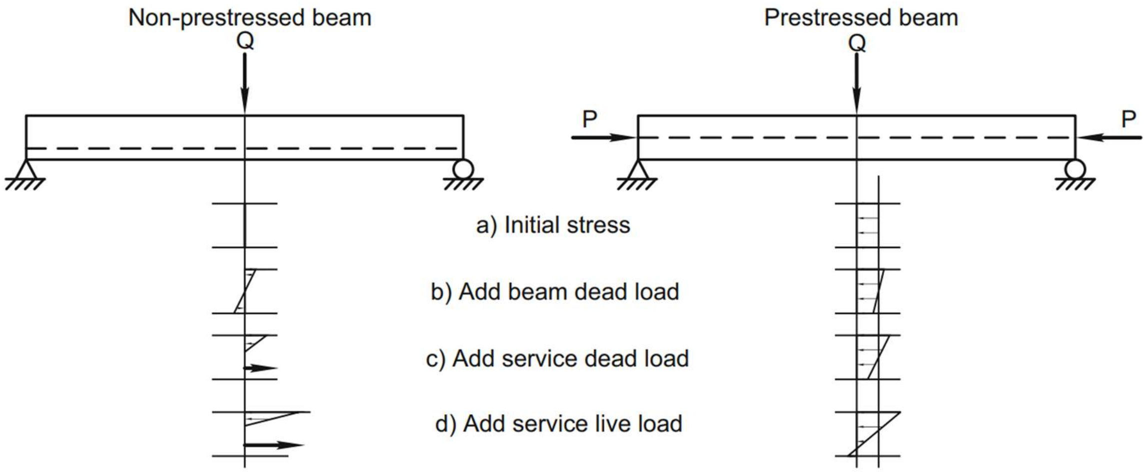



*Figure 3.2.6.5 – Stress profiles encountered at beams during different stages (Reinforced Concrete to the left, Prestressed Concrete to the right) [16]*

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***

| | |
|---|---|
| Ag | 1.0825e+05 |
| As | 500 |
| As_T | 2.6623e+03 |
| Axial_Strain | 2.0448e-04 |
| Axial_Stress | 6.4663 |
| Cur | 2.4571e-06 |
| d | 333.0127 |
| e | 43.2733 |
| Ec | 3.1623e+04 |
| Es | 200000 |
| Fps | 700000 |
| fr | 3.7947 |
| Ig | 1.1276e+09 |
| It | 1.1327e+09 |
| M | 1.1105e+08 |
| $\boxed{\text{M\_kNm} \quad 111.0513}$ | |
| n | 6.3246 |
| NA | 289.7394 |
| Prestressing_Strain | 0.0070 |
| Strain_at_bot | 3.2619e-04 |
| Strain_at_bot_cr | 1.2000e-04 |

| | |
|---|---|
| As | 500 |
| b | 135.7560 |
| c | 117.5682 |
| C | 200000 |
| C_kN | 200 |
| csyms | 1x1 sym |
| Cur | 2.5517e-05 |
| d | 333.0127 |
| Epsilon_0 | 0.0020 |
| Epsilon_bot | 0.0080 |
| Epsilon_c | 1x1 sym |
| Epsilon_c_syms | 1x1 sym |
| Epsilon_ctop | 0.0030 |
| Epsilon_s | 0.0055 |
| Es | 200000 |
| fc_prime | 40 |
| fs | 1.0995e+03 |
| integration_limit | 39.1894 |
| integration_limit_s... | 1x1 sym |
| M | 5.3463e+07 |
| M_kNm | 53.4635 |
| T | 200000 |
| T_kN | 200 |
| y | 1x1 sym |
| y_bar | 65.6953 |

| | |
|---|---|
| Epsilon_ctop | 0.0030 |
| Es | 200000 |
| fc_prime | 40 |
| fps | 1.5112e+03 |
| Fps | 700000 |
| fps_syms | 1x1 sym |
| fr | 3.7947 |
| Ig | 1.1276e+09 |
| integration_limit | 76.1722 |
| integration_limit_s... | 1x1 sym |
| It | 1.1327e+09 |
| M | 1.5514e+08 |
| M_kNm | 155.1385 |
| n | 6.3246 |
| NA | 289.7394 |
| Prestressing_Strain | 0.0070 |
| Strain_at_bot | 3.2619e-04 |
| Strain_at_bottom_... | 1.2171e-04 |
| Strain_at_steel | 0.0086 |
| Strain_at_steel_syms | 1x1 sym |
| Strain_at_top | 4.1642e-05 |
| Strain_at_top_Due... | 2.4613e-04 |
| Stress_at_bot | 10.3150 |

*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].

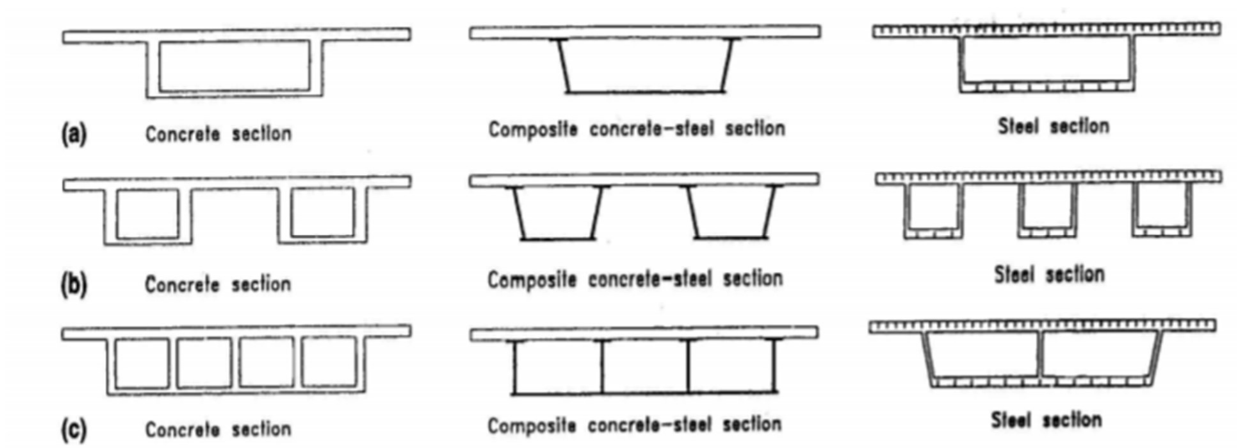



*Figure 3.3.1.1 – Different types of cross sections for a box girder bridge [19]*

### 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.

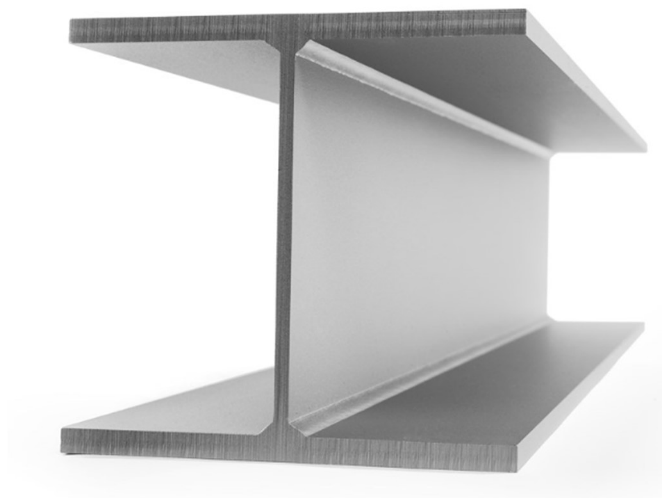


*Figure 3.3.2.1 – Stainless Steel I-Beam [22]*

### 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]*

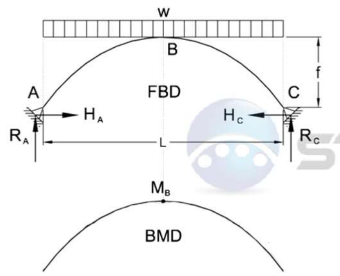
## 3.4 Geometry Types and Application

### 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**

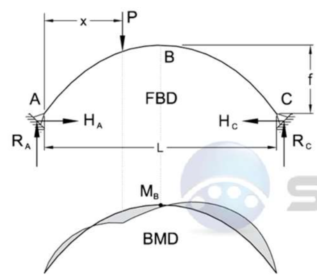
$$R_A = R_C = \frac{wL}{2}$$

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

**Bending Moments**

$$M_B = 0$$

*Figure 3.4.1.1.1 – Parabolic two hinged arch with distributed load*




**Support Reactions**

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

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

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

**Bending Moments**

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

*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.

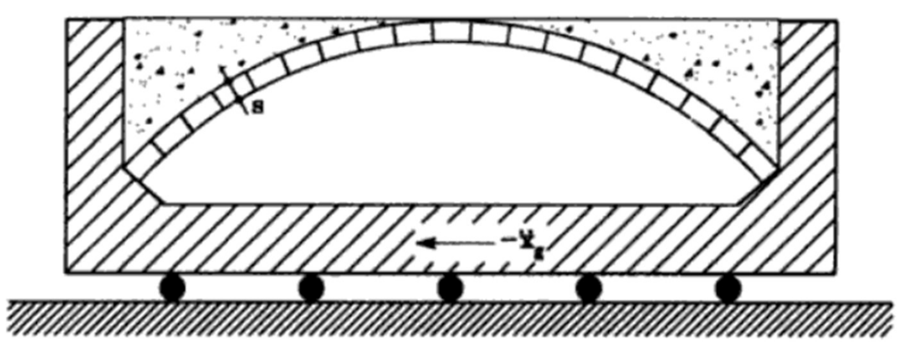



*Figure 3.4.1.1.3 – Structural Model of a stone arch under horizontal ground motion loading [27] (Arch Bridges, Anna Sipoli, University of Architecture, Venice).*

## 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).

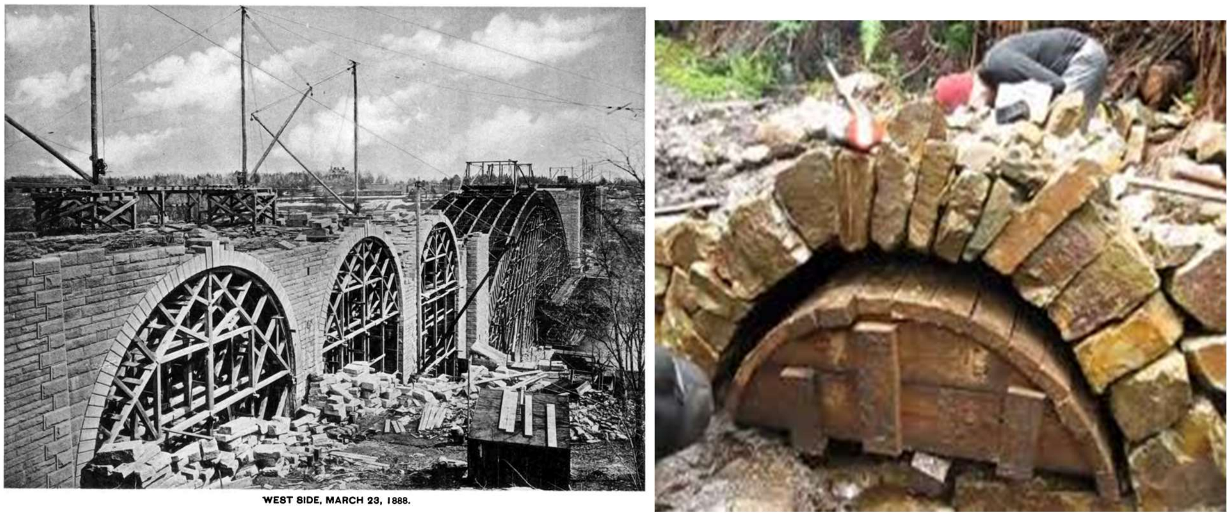



*Figure 3.4.1.2.1 – (Left) Washington Bridge west side 1888 and (Right) A small arch model bridge made by a person by putting stones together by hand [28]*

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

Travelling formwork method for concrete:

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.

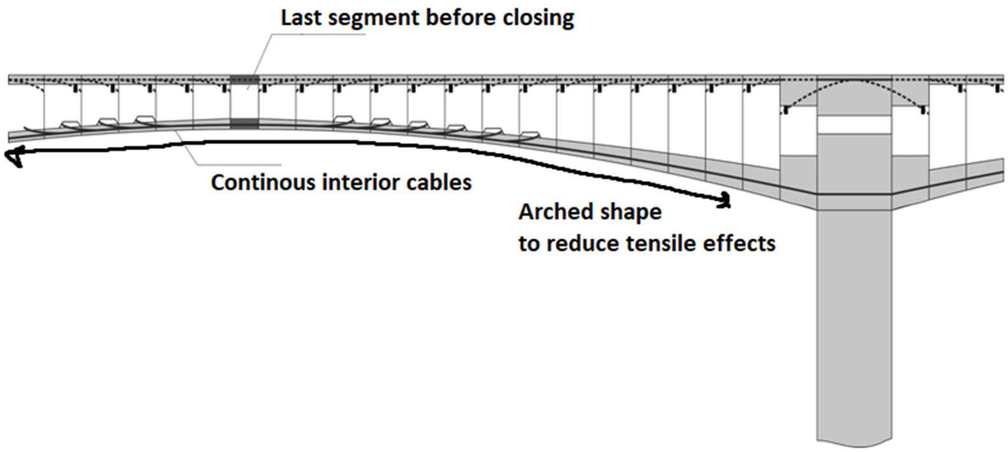



*Figure 3.4.1.2.4 – Possible cable layout for segmental construction drawing of Prof Erhan Karaesmen Middle East Technical University, Turkey [31]*

*Figure 3.4.1.2.5 – Stolmadunsed Bridge, Norway [32]*

UNIVERSITY OF TORONTO

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]*

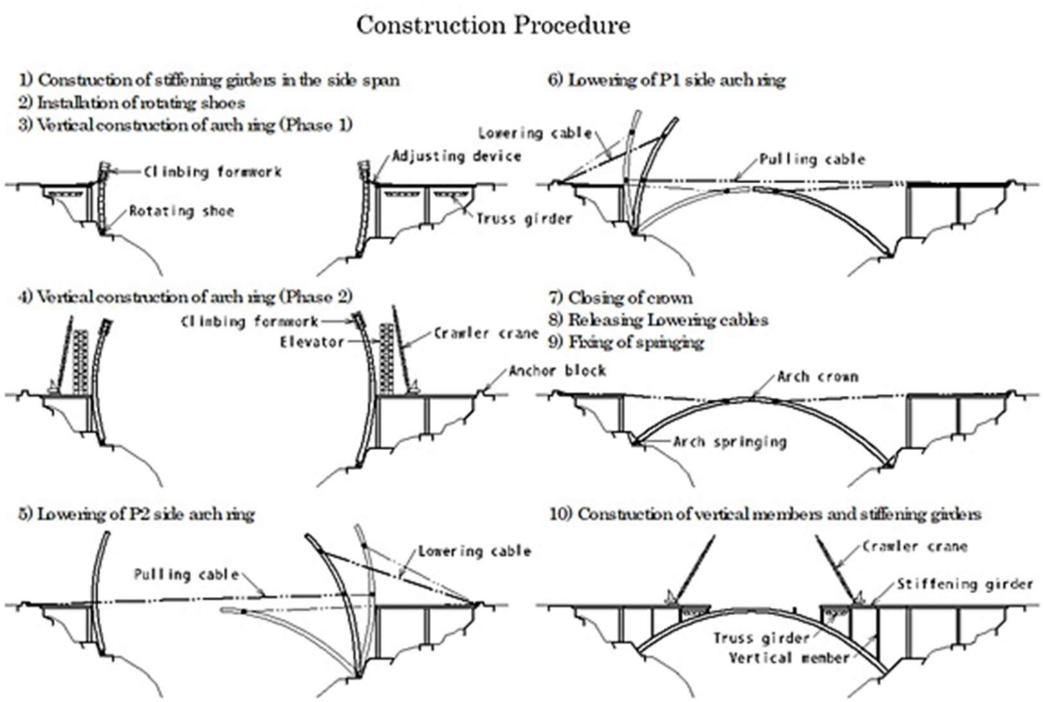



*Figure 3.4.1.2.7 – Construction Procedure [34]*

K & B

### 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]*

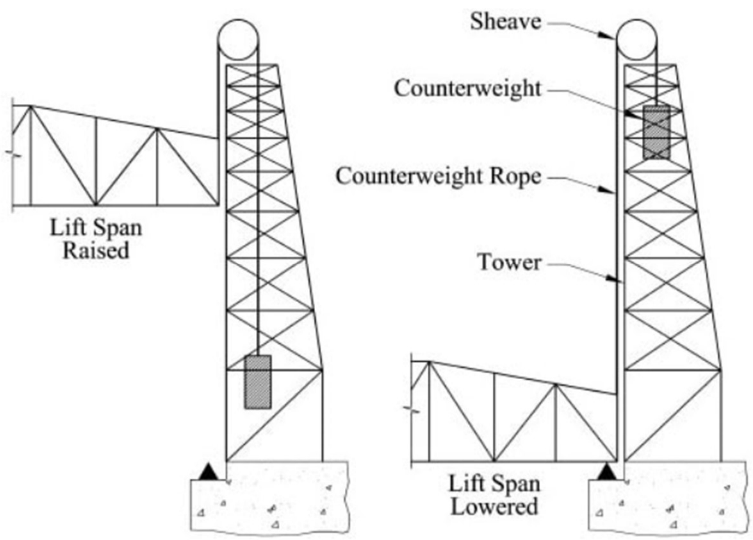



*Figure 3.4.2.1.3 – Details of Vertical Lifting Bridge [37]*

-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.

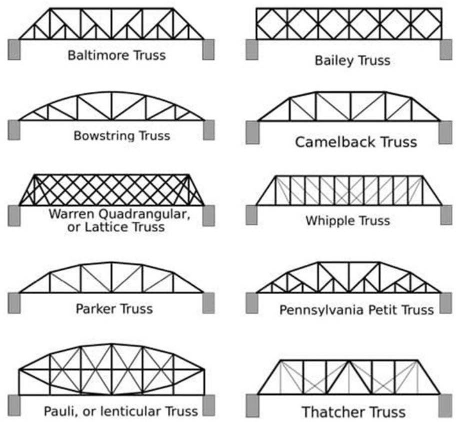



*Figure 3.4.3.1.1 – Some of the different types of trusses [40]*

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]*

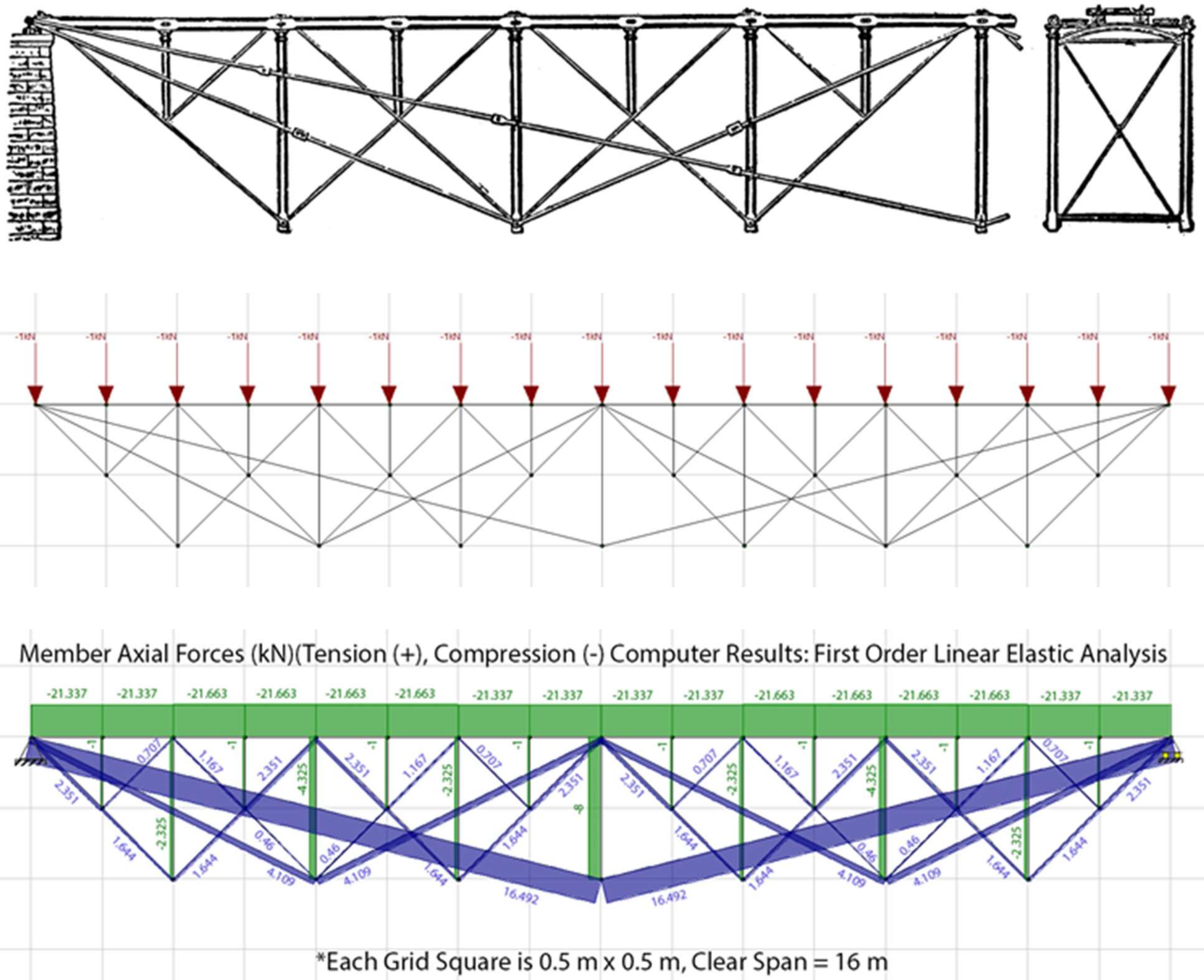



*Figure 3.4.3.1.3 – Force distribution in one of the old Truss Bridge Design, Fink Truss*

-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)

<u>Advantages of Truss Bridges:</u>

-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.

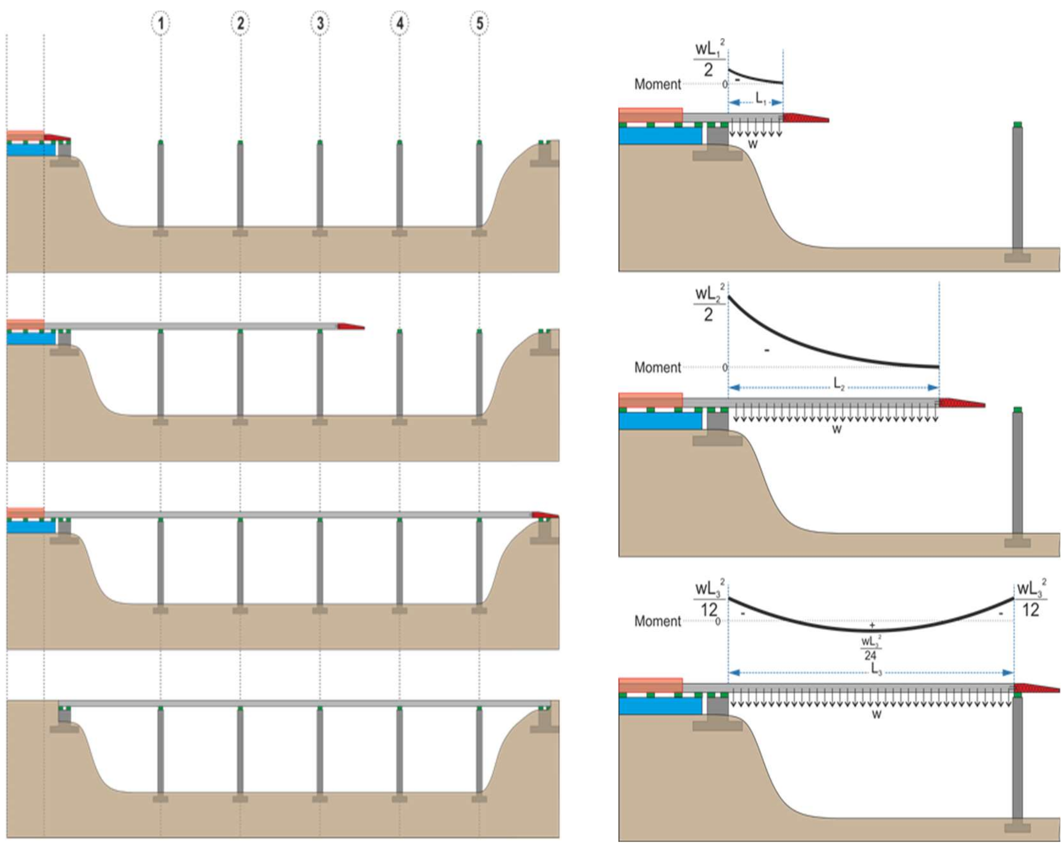



*Figure 3.4.3.2.1 – Incremental Launching Method [45]*


*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.

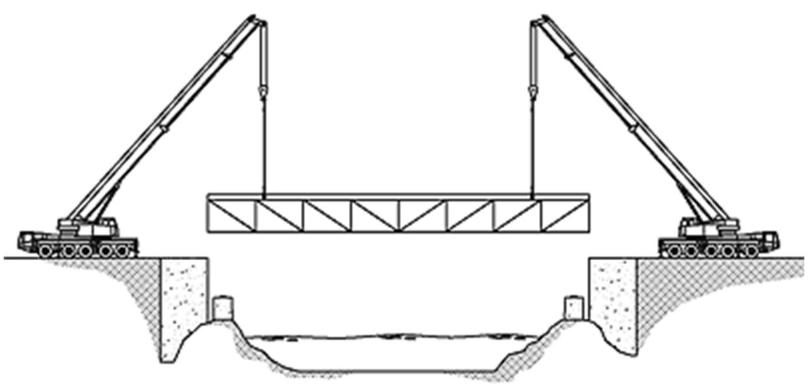


*Figure 3.4.3.2.4 – Direct Placement [47]*

### 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.

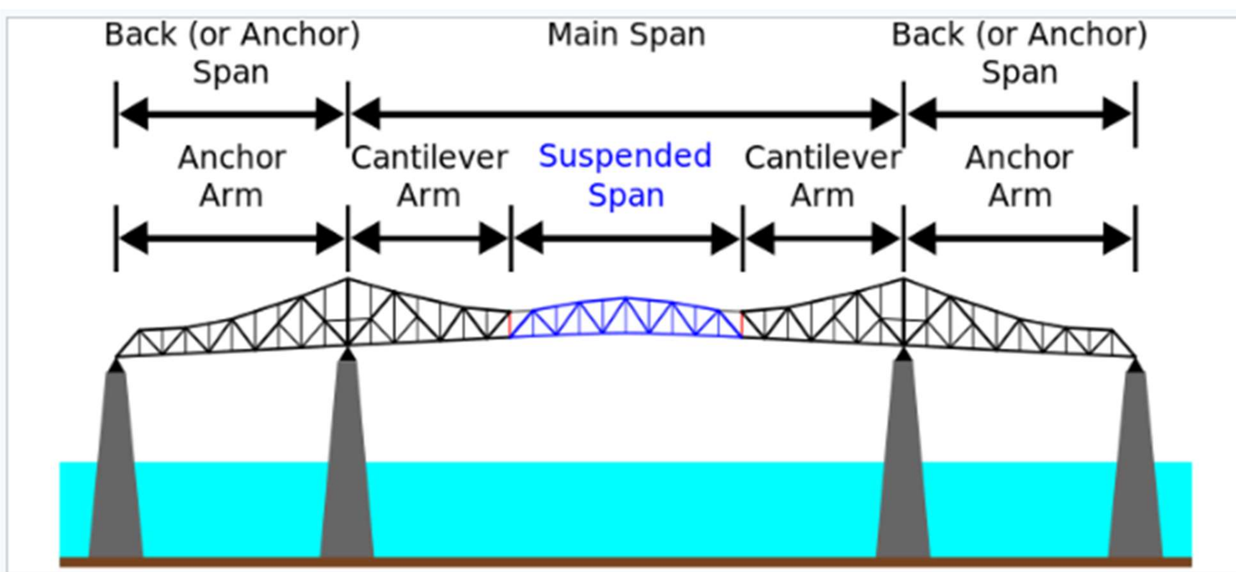



*Figure 3.4.4.1.1 – Simple Cantilever Bridge [49]*

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

<u>Advantages:</u>

-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].

<u>Disadvantages:</u>

-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].

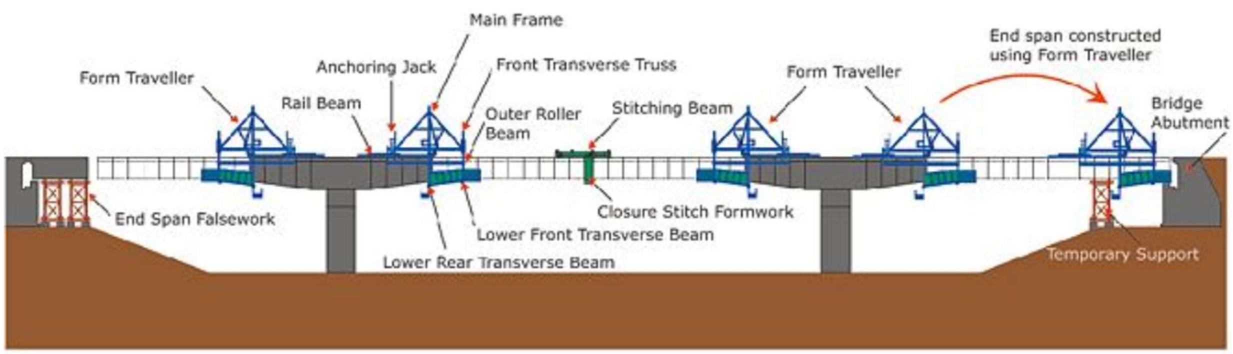



*Figure 3.4.4.2.1– Construction Technique for Cantilever Bridge Construction [53]*

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]*

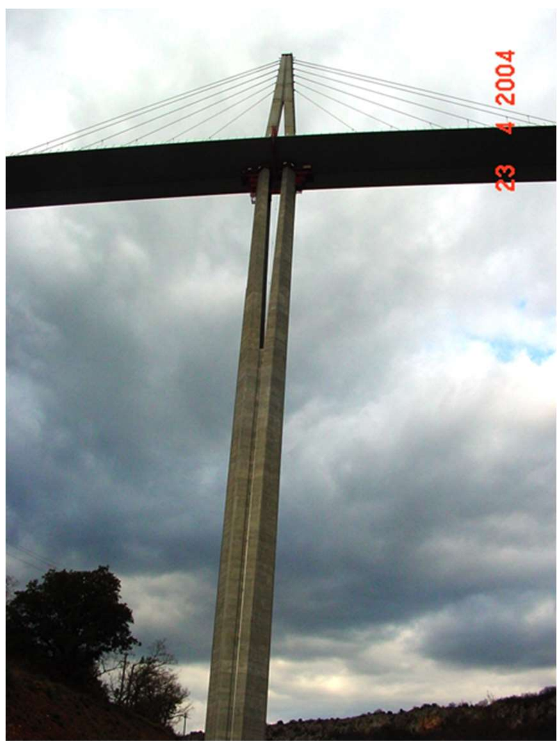



*Figure 3.4.5.1.1– Milau Viaduct - Tallest Bridge of the world approx. 340 m height [57]*

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.

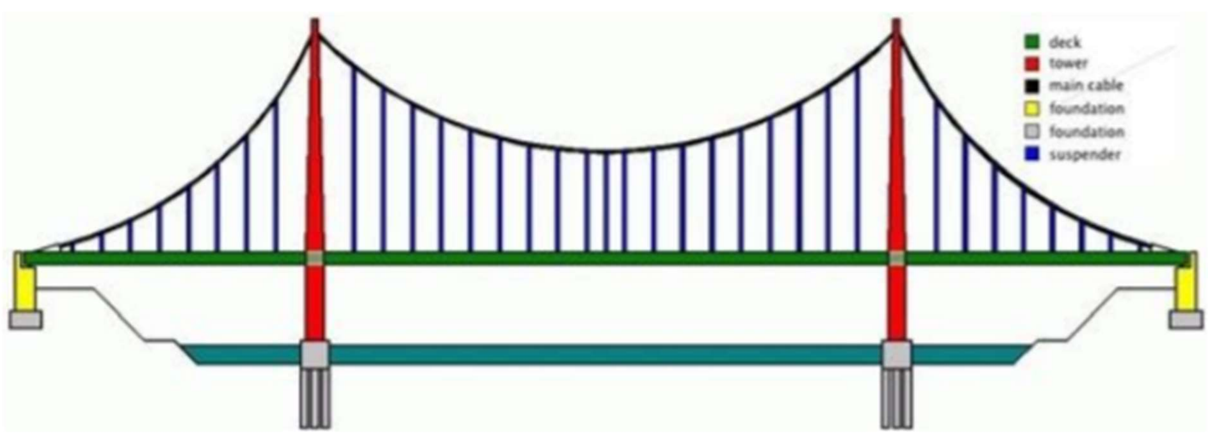



*Figure 3.4.6.1.1– Labelled Suspension Bridge [59]*

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.

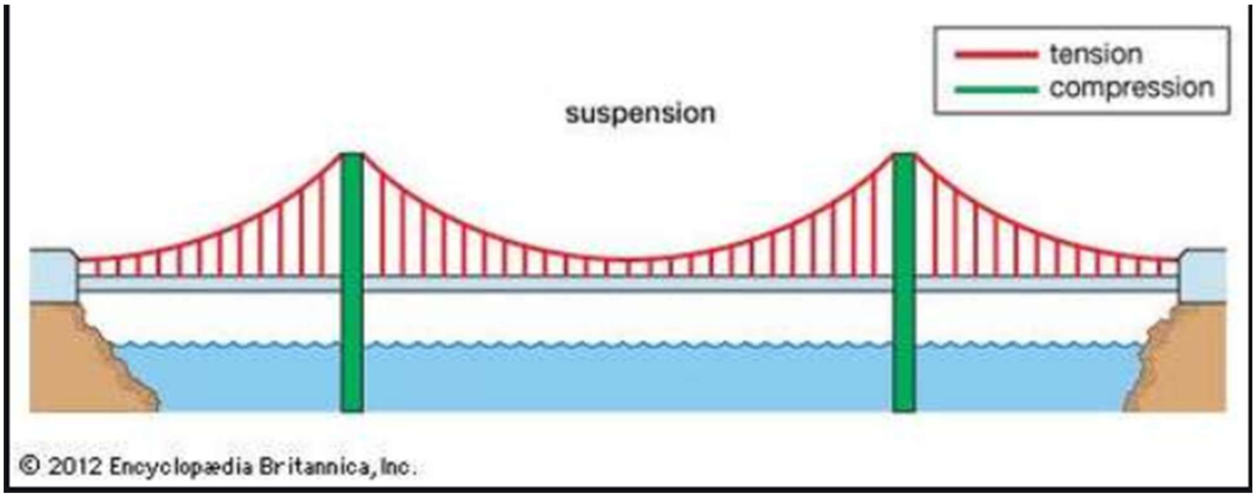



*Figure 3.4.6.1.2– Compression and Tension Force in Suspension Bridge [60]*

*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].

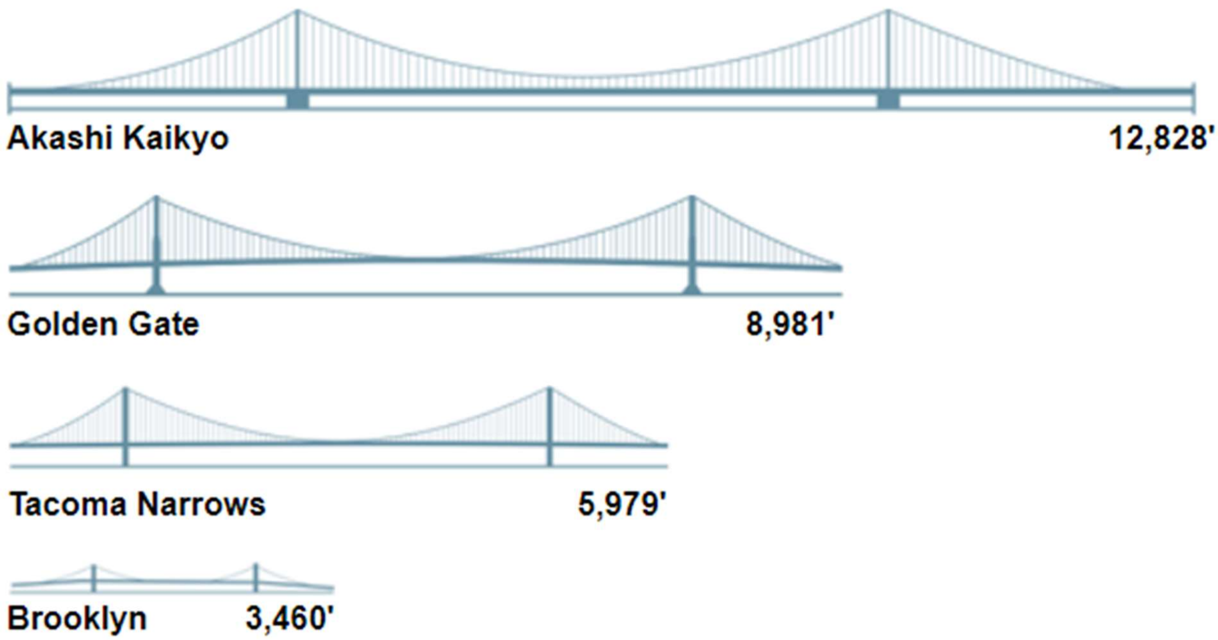



*Figure 3.4.6.1.4– Akashi Kaikyo Compared to other Iconic Suspension Bridges [61]*

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

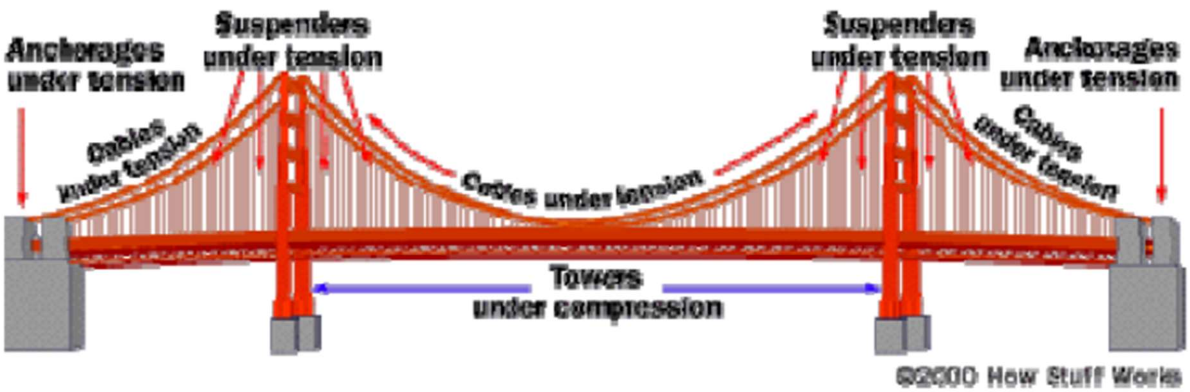



*Figure 3.4.6.2.1– Typical Construction Sequence of Suspension Bridge [65]*

### 3.5 Summary

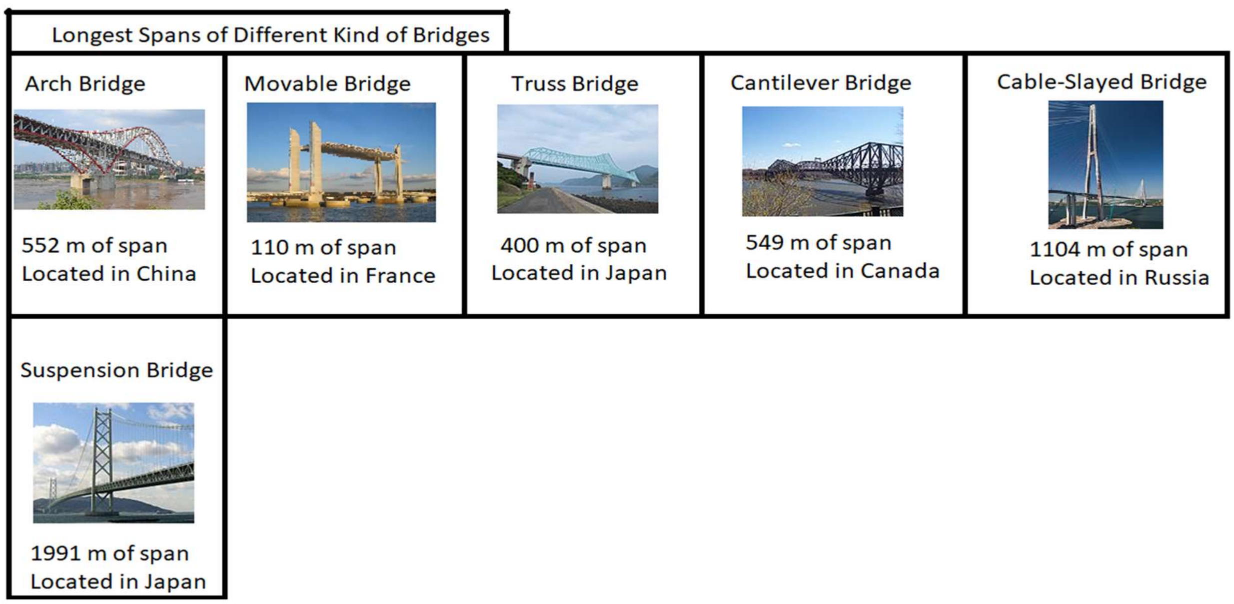



*Figure 3.5.1– Longest Spans of Different Kind of Bridges*

| 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.

### 3.7 References

[1]. “Stone arch bridges,” 2013. [Online]. Available: https://www.enabel.be/sites/default/files/manual_stone_arch_bridges.pdf. [Accessed: 05-Apr-2020].

[2]. “Masonry bridges from around the world,” *Structurae*, 28-Sep-2018. [Online]. Available: https://structurae.net/en/structures/bridges/masonry-bridges. [Accessed: 05-Apr-2020].

[3]. C. Lombardo, “Crystal Lombardo,” *Samsung Galaxy Blog*, 28-Jan-2016. [Online]. Available: https://thenextgalaxy.com/the-advantages-and-disadvantages-of-arch-bridges/. [Accessed: 05-Apr-2020].

[4]. “TIMBER AS A BRIDGE MATERIAL.” [Online]. Available: https://www.fpl.fs.fed.us/documnts/misc/em7700_8_chapter01.pdf. [Accessed: 05-Apr-2020].

[5]. “Mass timber bridges,” *Nordic Structures | nordic.ca | Engineered Wood | Projects | Mass timber bridges*. [Online]. Available: https://www.nordic.ca/en/projects/mass-timber-bridges. [Accessed: 05-Apr-2020].

[6]. *What is steel - different types of steel*. [Online]. Available: http://resources.schoolscience.co.uk/Corus/14-16/steel/msch3pg1.html. [Accessed: 05-Apr-2020].

[7]. “Strong, Flexible & Beautiful: The Benefits of Steel Bridge Construction,” *Bridge Masters*, 09-Jun-2017. [Online]. Available: https://bridgemastersinc.com/strong-flexible-beautiful-the-benefits-of-steel-bridge-construction/. [Accessed: 05-Apr-2020].

[8]. J. Afsar, “Disadvantages of steel structures,” *Engineering Intro*, 15-Mar-2012. [Online]. Available: https://www.engineeringintro.com/introduction-to-steel/steel-structure/disadvantages-of-steel-structures/. [Accessed: 05-Apr-2020].

[9]. *Galvanized Steel Cable Suspension Bridge*. [Online]. Available: http://www.steel-trussbridge.com/quality-2184042-galvanized-steel-cable-suspension-bridge. [Accessed: 05-Apr-2020].

[10]. “Finite Element Analysis of Reinforced Concrete Beam-Column Connections with Governing Joint Shear Failure Mode,” 15-Jan-2017. [Online]. Available: http://www.scielo.br/pdf/lajss/v14n7/1679-7825-lajss-14-07-01200.pdf. [Accessed: 05-Apr-2020].

[11] Edward Nawy Prestressed Concrete Fifth Edition Upgrade: ACI, AASHTO, IBC 2009 Codes Version (5th Edition)

[12]. “Freyssinet UK: Civil & Structural Engineering Specialists,” *FreyssinetUK*, 09-Apr-2020. [Online]. Available: https://freyssinet.co.uk/. [Accessed: 09-Apr-2020].

[13]. “Find quality Manufacturers, Suppliers, Exporters, Importers, Buyers, Wholesalers, Products and Trade Leads from our award-winning International Trade Site. Import & Export on alibaba.com,” *Alibaba*. [Online]. Available: https://www.alibaba.com/?from_http=1. [Accessed: 09-Apr-2020].

[14] “Innovative Bridge Design Handbook,” *Google Books*. [Online]. Available: https://books.google.ca/books?id=9NIVBQAAQBAJ&pg=PA594&lpg=PA594&dq=Freyssinet+Design+Handbook&source=bl&ots=TTEmK9udar&sig=ACfU3U2BvzEjk87tfpHYxzEtM4cLkW_zmg&hl=en&sa=X&ved=2ahUKEwjhvq3hwdzoAhWSHhQKHZqPClYQ6AEwAHoECAkQKQ#v=onepage&q=Freyssinet Design Handbook&f=false. [Accessed: 09-Apr-2020].

[15]. Prestressed Concrete Structures - Michael P. Collins

[16]. Charles W. Dolan, H. R. (Trey) Hamilton - Prestressed Concrete_ Building, Design, and Construction - 2019

[17]. “Box Girder Bridges,” *Box Girder Bridges - an overview | ScienceDirect Topics*. [Online]. Available: https://www.sciencedirect.com/topics/engineering/box-girder-bridges. [Accessed: 09-Apr-2020].

[18]. P. byS. O. A. O. 7 and P. byS. O. A, “Advantages of Steel Box Girder in Bridges: Disadvantages: CivilDigital |,” *CivilDigital*, 03-Oct-2016. [Online]. Available: https://civildigital.com/advantages-steel-box-girders-bridges-disadvantages/. [Accessed: 09-Apr-2020].

[19]. A. Romeijn, “Design Box Girder.” [Online]. Available: http://bmeeokepito-cd.bmeeok.hu/CD/Össze_vissza_tárgyak/Acélhíd/Steel Bridges/steel/design_box_girder.pdf. [Accessed: 09-Apr-2020].

[20]. “I Shaped Girders in Engineering Physics,” *Tech Glads*, 01-May-2019. [Online]. Available: https://www.techglads.com/cse/sem1/i-shaped-girders/. [Accessed: 09-Apr-2020].

[21]. “What's an I-beam?” *St. Louis Pipe & Supply*. [Online]. Available: http://www.stlpipesupply.com/blog/whats-an-i-beam/. [Accessed: 09-Apr-2020].

[22].”Stainless Steel Profiles” Available: https://www.stalatube.com/News-and-events/2015/i-beam-customized-stainless-steel-profiles/. [Accessed: 09-Apr-2020]

[23]. S. Sharma, “T beams and terms used in T beams in Reinforced cement concrete,” *Free Civilengineering subject Tutorial*, 22-Feb-2020. [Online]. Available: https://civilengineering.blog/2017/11/18/t-beams-and-terms-used-in-t-beams/. [Accessed: 09-Apr-2020]

[24]. “What does t-beam mean,” *What does t-beam mean - Definition of t-beam - Word finder*. [Online]. Available: https://findwords.info/term/t-beam. [Accessed: 09-Apr-2020].

[25]. “TT - Flat beam / reinforced concrete / double t / for flooring by Olmet: ArchiExpo,” *The Online Architecture and Design Exhibition*. [Online]. Available: https://www.archiexpo.com/prod/olmet/product-105207-1020939.html. [Accessed: 09-Apr-2020].

[26]. “Slab Bride” [Online]. Available: https://www.researchgate.net/figure/Dubois-Creek-FRP-Slab-Bridge_fig1_326539851[Accessed: 09-Apr-2020]

[27]. Arch Bridges, Anna Sipoli, University of Architecture, Venice

[28]. “Washington Bridge,” *Washington Bridge (Washington Heights Bridge) - HistoricBridges.org*. [Online]. Available: https://historicbridges.org/bridges/browser/?bridgebrowser=newyork/washington/. [Accessed: 09-Apr-2020].

[29]. “Galena Bridge,” *CH Bull Jacking Solutions*, 25-Sep-2018. [Online]. Available: https://chbulljackingsolutions.com/galena-bridge/. [Accessed: 09-Apr-2020].

[30]. “Construction of Arch Bridge” [Online]. Available FTP: https://www.researchgate.net/figure/Construction-of-arch-bridge-with-traveling-formwork-method-5-353-Frame-bridges-Rigid_fig25_328175297 [Accessed: 09-Apr-2020]

[31]. Drawing of Prof Erhan Karaesmen Middle East Technical University, Turkey

[32]. Prof Erhan Karaesmen Middle East Technical University, Turkey

[33]. Construction of an Arch Bridge by Lowering Method - Yuki Kaminaga

[34]. “Lowering Method for Concrete Arch Bridges,” *Lowering Method for Concrete Arch Bridges | Oriental Shiraishi Corporation*. [Online]. Available: http://www.orsc.co.jp/english/tec/con02_3.html. [Accessed: 09-Apr-2020].

[35]. “Movable Bridges Types, Design and History,” *Movable Bridge - Facts and Types of Movable Bridge*. [Online]. Available: http://www.historyofbridges.com/facts-about-bridges/movable-bridge/. [Accessed: 09-Apr-2020].

[36]. P. byN. M. 9 and P. byNamita, “Movable bridges and types: Advantages: Special types pf movable bridges,” *CivilDigital*, 09-May-2018. [Online]. Available: https://civildigital.com/movable-bridges-special-types-of-movable-bridges-photos/. [Accessed: 09-Apr-2020].

[37]. S. Posted in Bridges, “Types of Movable Bridges and their Construction Details,” *The Constructor*, 09-Sep-2017. [Online]. Available: https://theconstructor.org/structures/types-movable-bridges-details/17941/. [Accessed: 09-Apr-2020].

[38]. “Draw Bridge,” *How Products Are Made*. [Online]. Available: http://www.madehow.com/Volume-6/Draw-Bridge.html. [Accessed: 09-Apr-2020].

[39]. Sluszka, Peter “Design and construction of a bascule bridge” [Online]. Available: https://www.e-periodica.ch/cntmng?pid=bse-re-003:1991:64::119 [Accessed 09-Apr-2020]

[39]. “Spaghetti Bridges - Activity,” *TeachEngineering.org*, 28-Jan-2020. [Online]. Available: https://www.teachengineering.org/activities/view/wpi_spag_act_joy. [Accessed: 09-Apr-2020].

[40]. “Truss Bridge - Types, History, Facts and Design,” *Truss Bridge - Facts and Types of Truss Bridges*. [Online]. Available: http://www.historyofbridges.com/facts-about-bridges/truss-bridge/. [Accessed: 09-Apr-2020].

[41]. “List of longest continuous truss bridge spans,” *Wikipedia*, 10-Mar-2020. [Online]. Available: https://en.wikipedia.org/wiki/List_of_longest_continuous_truss_bridge_spans. [Accessed: 09-Apr-2020].

[42]. Structurae, “Ikitsuki Bridge (Hirado, 1991),” *Structurae*, 21-Oct-2016. [Online]. Available: https://structurae.net/en/structures/ikitsuki-bridge. [Accessed: 09-Apr-2020].

[43]. C. Lombardo, “Crystal Lombardo,” *Samsung Galaxy Blog*, 28-Jan-2016. [Online]. Available: https://thenextgalaxy.com/advantages-and-disadvantages-of-truss-bridges/. [Accessed: 09-Apr-2020].

[44] Middle East Technical University - Prof Erhan Karaesmen lecture notes

[45] From construction of North Marmara Highway, Istanbul, Turkey

[46]. High Ltd, “Bridge Construction Techniques - Over Truss Bridge,” *Bridge Construction Methods & Techniques | Over Truss – REIDsteel*. [Online]. Available: http://www.steel-bridges.com/tech-over-truss-bridge.html. [Accessed: 09-Apr-2020].

[47]. High Ltd, “Bridge Construction Techniques - Through Truss Bridge,” *Through Truss Bridge | Bridge Construction Methods & Techniques - REIDsteel*. [Online]. Available: http://www.steel-bridges.com/tech-through-bridge.html. [Accessed: 09-Apr-2020].

[48]. “Cantilever Bridge Facts, Design and History,” *Cantilever Bridge - Types, Facts and Longest Cantilever Bridges*. [Online]. Available: http://www.historyofbridges.com/facts-about-bridges/cantilever-bridge/. [Accessed: 09-Apr-2020].

[49]. “Is Howrah Bridge a cantilever bridge? If so, how?” *Quora*. [Online]. Available: https://www.quora.com/Is-Howrah-Bridge-a-cantilever-bridge-If-so-how. [Accessed: 05-Apr-2020].

UNIVERSITY OF TORONTO

[50]. *Quebec Bridge - Quebec City, and Lévis, Quebec - Wikipedia Entries on Waymarking.com*. [Online]. Available: https://www.waymarking.com/waymarks/WMXEQB_Quebec_Bridge_Quebec_City_and_Lvis_Quebec. [Accessed: 05-Apr-2020].

[51]. “22 Important Advantages and Disadvantages of a Cantilever Bridge,” *Science Struck*, 06-Feb-2015. [Online]. Available: https://sciencestruck.com/advantages-disadvantages-of-cantilever-bridge. [Accessed: 05-Apr-2020].

[52]. “Bridges of Dublin,” *Cantilever | Types | Bridge Building | Bridges of Dublin*. [Online]. Available: http://www.bridgesofdublin.ie/bridge-building/types/cantilever. [Accessed: 05-Apr-2020].

[53]. S. Posted in Bridges, “Balanced Cantilever Method of Bridge Construction,” *The Constructor*, 09-Aug-2018. [Online]. Available: https://theconstructor.org/structures/balanced-cantilever-method-bridge-construction/6137/. [Accessed: 05-Apr-2020].

[54]. “Cable-stayed bridge.” [Online]. Available: https://en.wikipedia.org/wiki/Cable-stayed_bridge. [Accessed: 05-Apr-2020].

[55] “Cable-stayed Bridge - History, Facts and Types,” *Cable-stayed Bridge - Types, Advantages and Disadvantages*. [Online]. Available: http://www.historyofbridges.com/facts-about-bridges/cable-stayed-bridges/. [Accessed: 05-Apr-2020].

[56] “Brooklyn Bridge.” [Online]. Available: https://en.wikipedia.org/wiki/Brooklyn_Bridge. [Accessed: 05-Apr-2020].

[57]. Middle East Technical University - Prof Erhan Karaesmen lecture notes

[58]. “Suspension Bridge - From Earliest to Modern Designs,” *Suspension Bridge - History, Facts and Construction*. [Online]. Available: http://www.historyofbridges.com/facts-about-bridges/suspension-bridges/. [Accessed: 05-Apr-2020].

[59]. “Labeled Suspension Bridge Diagram, HD Png Download, Transparent Png Image - PNGitem,” *PNGitem.com*. [Online]. Available: https://www.pngitem.com/middle/hxRTow_labeled-suspension-bridge-diagram-hd-png-download/. [Accessed: 05-Apr-2020].

K & B

[60]. R. Lamb and M. Morrissey, “How Bridges Work,” *HowStuffWorks Science*, 27-Jan-2020. [Online]. Available: https://science.howstuffworks.com/engineering/civil/bridge6.htm. [Accessed: 05-Apr-2020].

[61]. *Tacoma Narrows Bridge: Suspension Bridge Basics*. [Online]. Available: https://www.wsdot.wa.gov/TNBhistory/Machine/machine1.htm. [Accessed: 05-Apr-2020].

[62]. “This date in science: First modern suspension bridge completed,” *EarthSky*. [Online]. Available: https://earthsky.org/human-world/this-date-in-science-first-modern-suspension-bridge-completed. [Accessed: 05-Apr-2020].

[63]. “This date in science: First modern suspension bridge completed,” *EarthSky*. [Online]. Available: https://earthsky.org/human-world/this-date-in-science-first-modern-suspension-bridge-completed. [Accessed: 05-Apr-2020].

[64]. C. Lombardo, “Crystal Lombardo,” *Samsung Galaxy Blog*, 28-Jan-2016. [Online]. Available: https://thenextgalaxy.com/8-disadvantages-and-advantages-of-suspension-bridges/. [Accessed: 05-Apr-2020].

[65]. B. Evran, “The Construction of a Suspension Bridge,” *Academia.edu*. [Online]. Available: https://www.academia.edu/4969578/The_Construction_of_a_Suspension_Bridge. [Accessed: 05-Apr-2020].

# 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 8$^{th}$ 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.

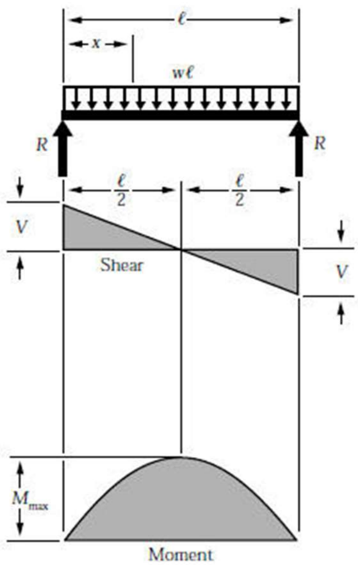
## 4.2 Bridge Behavior and Loading

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




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

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

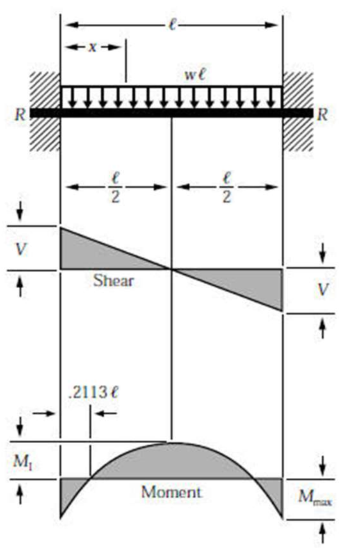
$$M_{max} \text{ (at center)} \quad \ldots\ldots \quad = \frac{w\ell^2}{8}$$

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

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

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

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




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

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

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

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

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

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

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

*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.

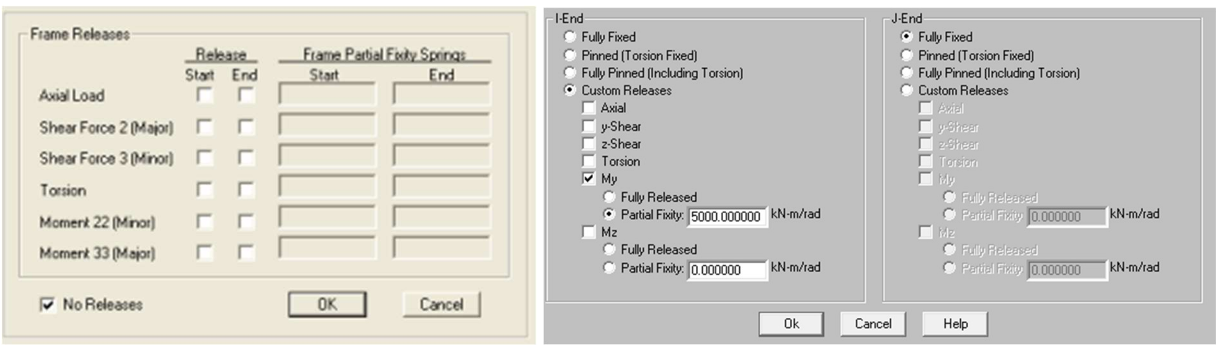



*Figure 4.2.1.1.3 - Finite Element Programs*

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.

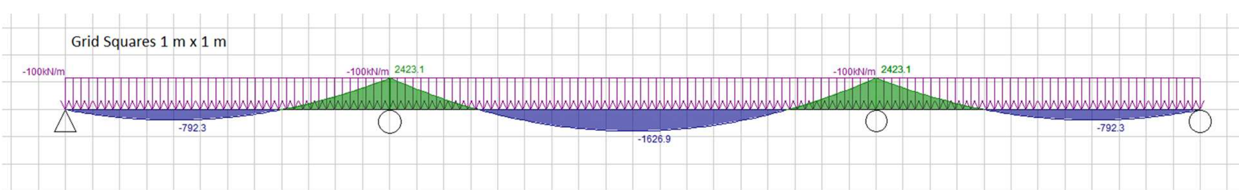



*Figure 4.2.1.1.4 - Bending Moments in a Continuous Simply Structure*

### 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.

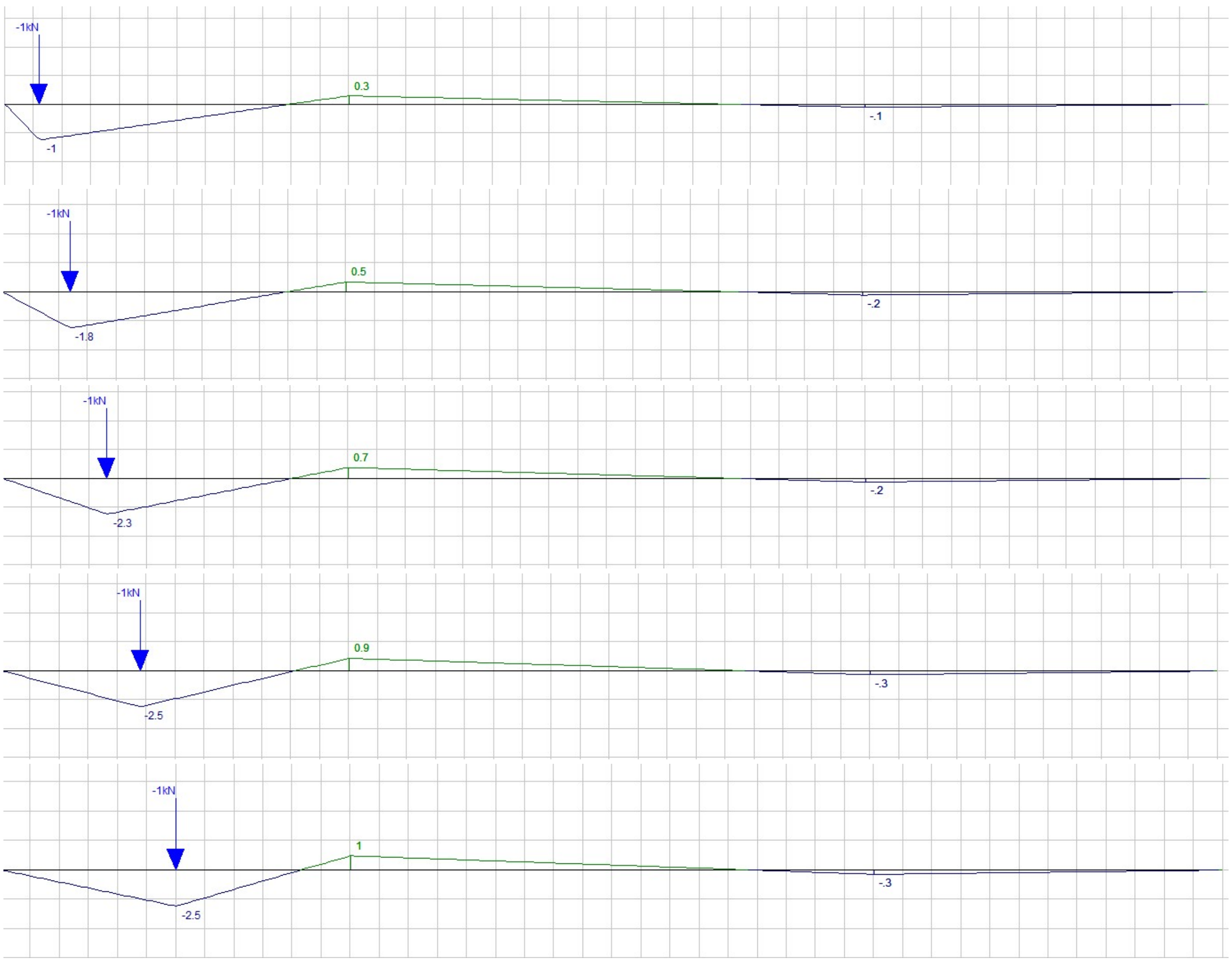



*Figure 4.2.1.2.1- Pattern for Influence Lines of the 3-span beam mentioned above (1 m x 1 m grid squares)*

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.

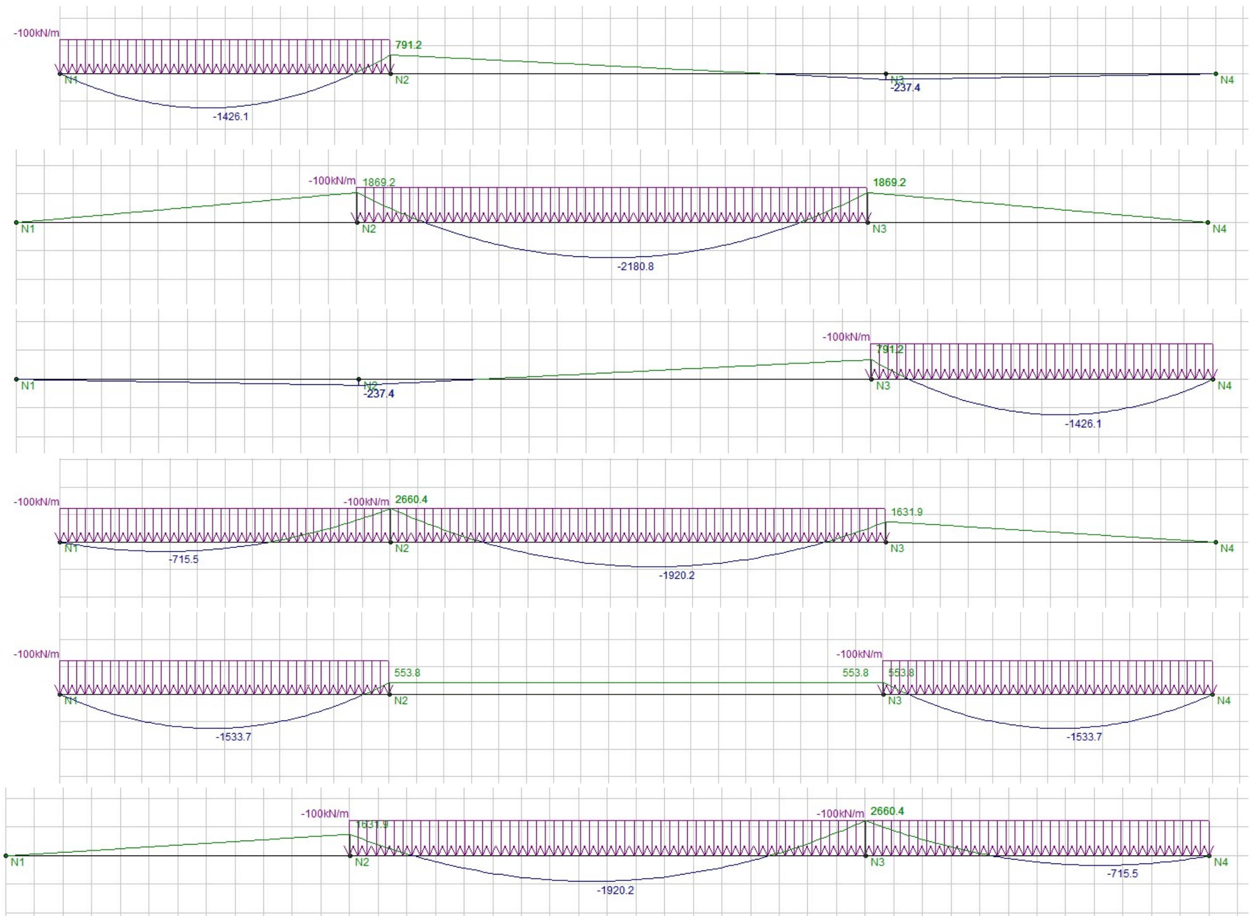



*Figure 4.2.1.2.2- Live Load Envelopes for the 3-span beam mentioned above (1 m x 1 m grid squares, 100 kN/m as unit load)*

### 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]*

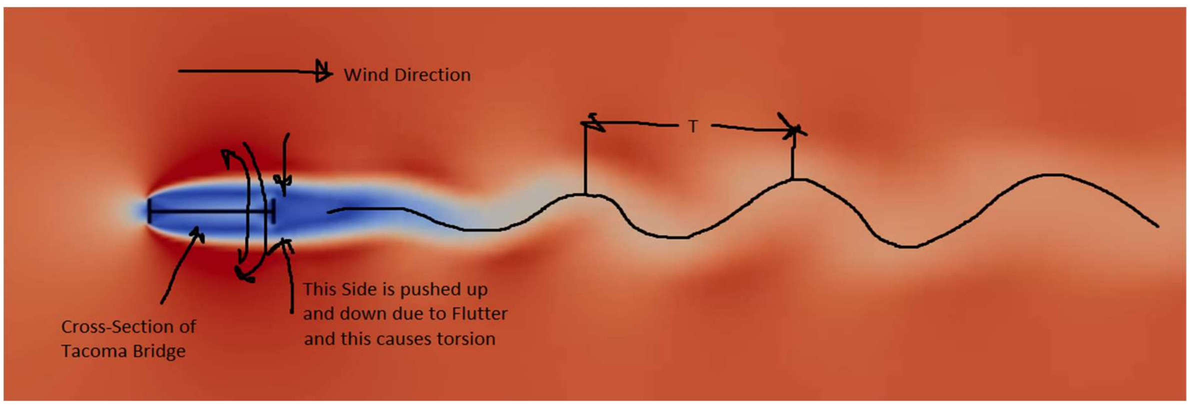



*Figure 4.2.1.3.2- 2D Computational Fluid Dynamics simulation of Tacoma Bridge [3]*

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.

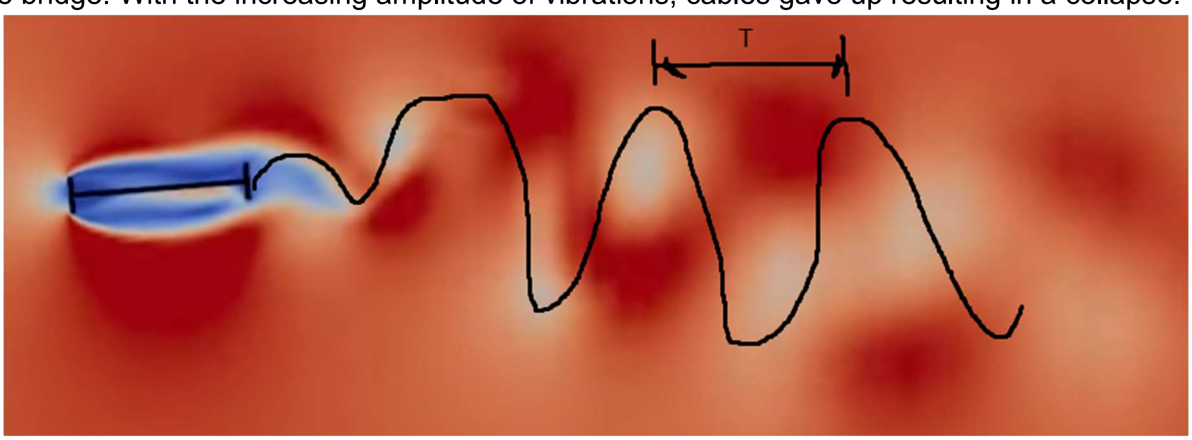



*Figure 4.2.1.3.3- 2D Computational Fluid Dynamics simulation of Tacoma Bridge (close to failure) [3]*

### 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.

| Loads | Permanent loads D | E | P | Transitory loads $L^*$ | K | W | V | S | Exceptional loads 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 | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | Table 3.2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| ULS Combination 2 | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | Table 3.2 | 1.15 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| ULS Combination 3 | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | Table 3.2 | 1.00 | 0.45§ | 0.45 | 0 | 0 | 0 | 0 | 0 |
| ULS Combination 4 | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | 0 | 1.25 | 1.40§ | 0 | 0 | 0 | 0 | 0 | 0 |
| ULS Combination 5 | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | 0 | 0 | 0 | 0 | 0 | 1.00 | 0 | 0 | 0 |
| ULS Combination 6** | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | 0 | 0 | 0 | 0 | 0 | 0 | 1.30 | 0 | 0 |
| ULS Combination 7 | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | 0 | 0 | 0.75§ | 0 | 0 | 0 | 0 | 1.30 | 0 |
| ULS Combination 8 | $\alpha_D$ | $\alpha_E$ | $\alpha_P$ | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1.00 |
| ULS Combination 9 | 1.35 | $\alpha_E$ | $\alpha_P$ | 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 $\alpha_D$, $\alpha_E$, and $\alpha_P$ 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*

| Load | Live load factor: Normal traffic | Special loads mixed with normal traffic: Short spans | Special loads mixed with normal traffic: Other spans | Special loads travelling alone on bridge under supervision: Short spans | Special loads travelling alone on bridge under supervision: 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 $\alpha_D$ | Minimum $\alpha_D$ |
|---|---|---|
| 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 $\alpha_D$** | **Minimum $\alpha_D$** |
| All dead loads for ULS Combination 5 (see Table 3.1) | 1.25 | 0.80 |
| **Earth pressure and hydrostatic pressure** | **Maximum $\alpha_E$** | **Minimum $\alpha_E$** |
| 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 $\alpha_P$** | **Minimum $\alpha_P$** |
| 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.

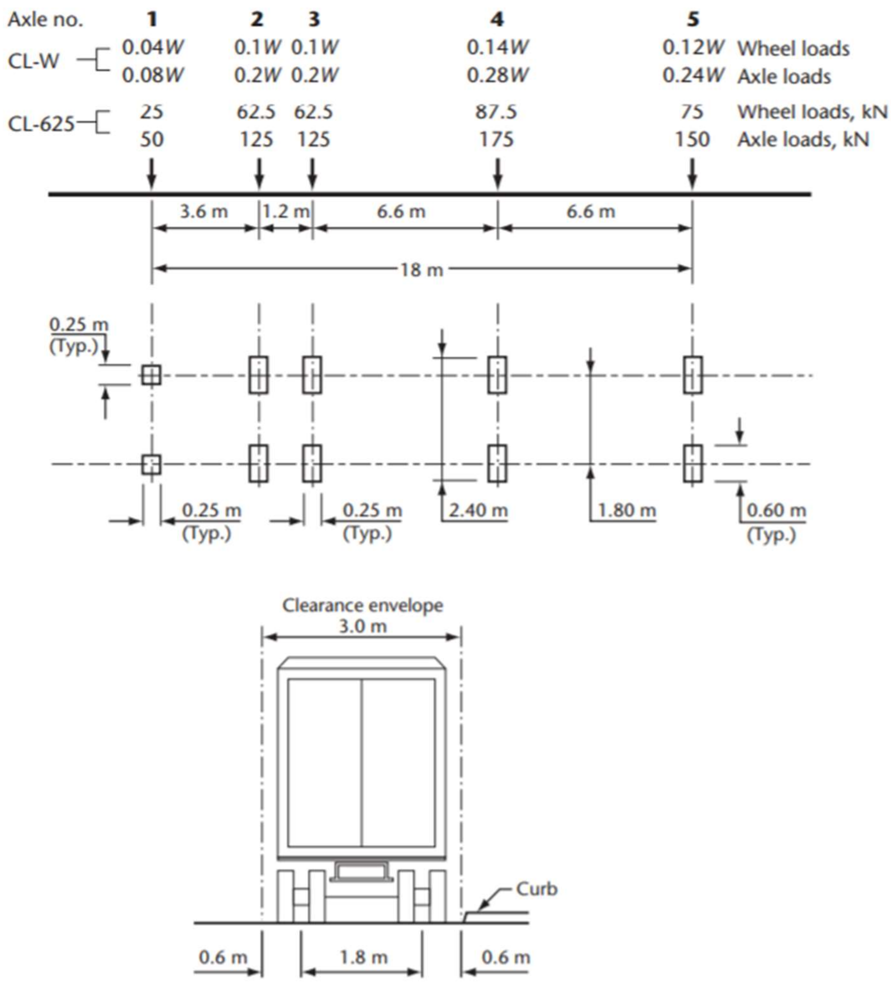



*Figure 4.3.4.2.2- CL-W Truck in CSA-S6-14*

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.

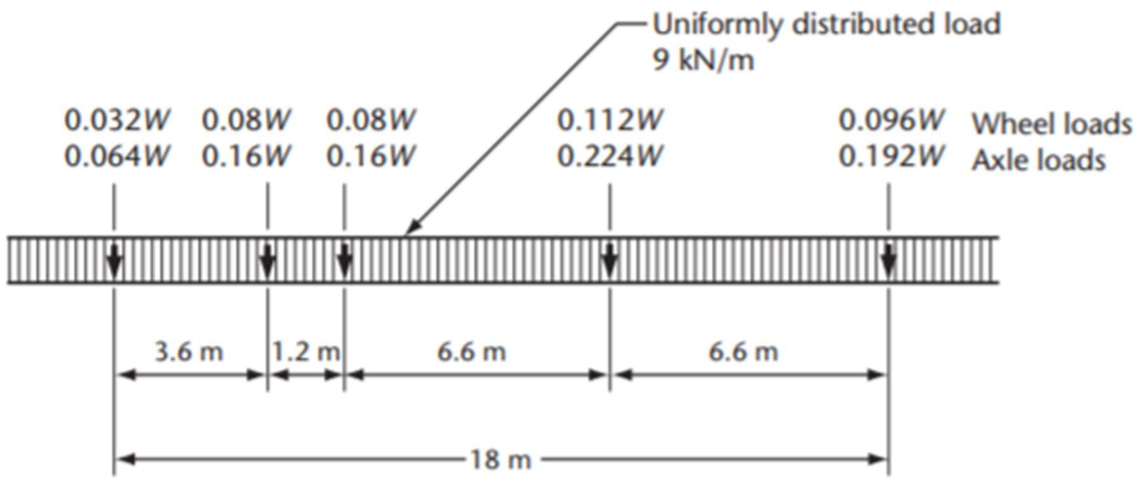



*Figure 4.3.4.2.3- CL-W Lane Load in CSA-S6-14*

### 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:

$F = qCeCgCv$

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

UNIVERSITY OF TORONTO

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

$\gamma_c$ = mass density of concrete in kg/m$^3$

K & B

#### 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, $\varepsilon_{cs}$, 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

$\varepsilon_{cs0}$ = notional shrinkage coefficient

$$= \beta_{RH}\left[160+50\left[9-\frac{f_c'+a}{10}\right]\right]\times10^{-6}$$

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:

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

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:

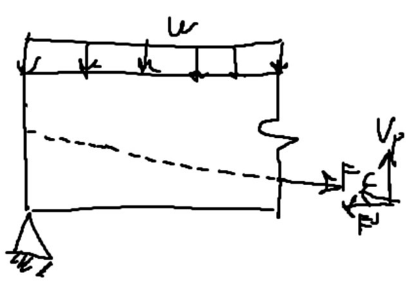
### 8.9.3.4 Determination of $V_c$

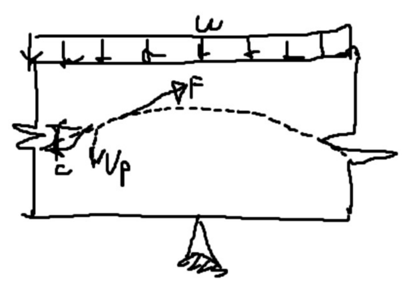
$V_c$ shall be calculated as $2.5\beta\,\phi_c\,f_{cr}\,b_v\,d_v$. However, $f_{cr}$ shall not be greater than 3.2 MPa.

-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*

| Sheath type | Strand $K$ | Strand $\mu$ | Smooth bar $K$ | Smooth bar $\mu$ | Deformed bar $K$ | Deformed bar $\mu$ |
|---|---|---|---|---|---|---|
| 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 ($d_d \leq 60$ 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 |

*(Continued)*

*Table 4.3.7.1.1 B- 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** |
| 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<br>Pretensioning strands<br>Post-tensioning ducts | 40 ± 10<br>—<br>60* ± 10 | 40 ± 10<br>55 ± 5<br>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<br>Pretensioning strands<br>Post-tensioning ducts | 60 ± 20<br>—<br>80* ± 15 | 40 ± 10<br>55 ± 5<br>60* ± 10 |
| | (3) Top surface of structural component, except (1) and (2)‡ | Reinforcing steel<br>Pretensioning strands<br>Post-tensioning ducts | 60 ± 20<br>—<br>80* ± 15 | 50 ± 10<br>70 ± 5<br>70 ± 10 |
| | (4) Soffit of precast deck form | Reinforcing steel<br>Pretensioning strands | —<br>— | 40 ± 10<br>38 ± 3 |
| | (5) Soffit of slab less than 300 mm thick or soffit of top slab of voided deck | Reinforcing steel<br>Pretensioning strands<br>Post-tensioning ducts | 40 ± 10<br>—<br>60* ± 10 | 40 ± 10<br>55 ± 5<br>60* ± 10 |
| | (6) Soffit of slab 300 mm thick or thicker or soffit of structural component, except (4) and (5) | Reinforcing steel<br>Pretensioning strands<br>Post-tensioning ducts | 50 ± 10<br>—<br>70* ± 10 | 40 ± 10<br>55 ± 5<br>60* ± 10 |
| | (7) Vertical surface of arch, solid or voided deck, pier cap, T-beam, or interior diaphragm | Reinforcing steel<br>Pretensioning strands<br>Post-tensioning ducts | 60 ± 10<br>—<br>80* ± 10 | 50 ± 10<br>65 ± 5<br>70* ± 10 |
| | (8) Inside vertical surface of buried structure or inside surface of circular buried structure | Reinforcing steel<br>Pretensioning strands<br>Post-tensioning ducts | 60 ± 20<br>—<br>80* ± 15 | 40 ± 10<br>55 ± 5<br>60* ± 10 |
| | (9) Vertical surface of structural component, except (7) and (8) | Reinforcing steel<br>Pretensioning strands<br>Post-tensioning ducts | 60 ± 20<br>—<br>80* ± 15 | 50 ± 10<br>70 ± 5<br>70* ± 10 |
| | (10) Precast T-, I-, or box girder | Reinforcing steel<br>Pretensioning strands<br>Post-tensioning ducts | —<br>—<br>— | 30 +10 or −5<br>45 ± 5<br>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 0.5$d_d$, 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.*

K & B

## 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:

$\eta_i$ = load modifier, relating to ductility, redundancy, and operational importance
$\gamma_i$ = load factor; a statistically based multiplier applied to force effects
$Q_i$ = force effect
$\phi$ = 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 $\eta$**

$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$ ⟹ *AASHTO LRFD* Equation 1.3.2.1-2

However, for minimum values of $\gamma_i$: ⟹ *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*

$\eta_D$ = The ductility factor
$\eta_R$ = The redundancy factor
$\eta_I$ = The operational importance factor

$\eta_D$ = 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

$\eta_R$ = 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

$\eta_I$ = 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 $\gamma_{EQ}$, 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.

UNIVERSITY OF TORONTO

*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) | $\gamma_p$ | 1.75 | 1.00 | — | — | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Strength II | $\gamma_p$ | 1.35 | 1.00 | — | — | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Strength III | $\gamma_p$ | — | 1.00 | 1.00 | — | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Strength IV | $\gamma_p$ | — | 1.00 | — | — | 1.00 | 0.50/1.20 | — | — | — | — | — | — | — |
| Strength V | $\gamma_p$ | 1.35 | 1.00 | 1.00 | 1.00 | 1.00 | 0.50/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Extreme Event I | 1.00 | $\gamma_{EQ}$ | 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 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| Service II | 1.00 | 1.30 | 1.00 | — | — | 1.00 | 1.00/1.20 | — | — | — | — | — | — | — |
| Service III | 1.00 | $\gamma_{LL}$ | 1.00 | — | — | 1.00 | 1.00/1.20 | $\gamma_{TG}$ | $\gamma_{SE}$ | — | — | — | — | — |
| 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 | — | — | — | — | — | — | — | — | — | — | — | — |

K & B

### 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 |

#### 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.

## 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.

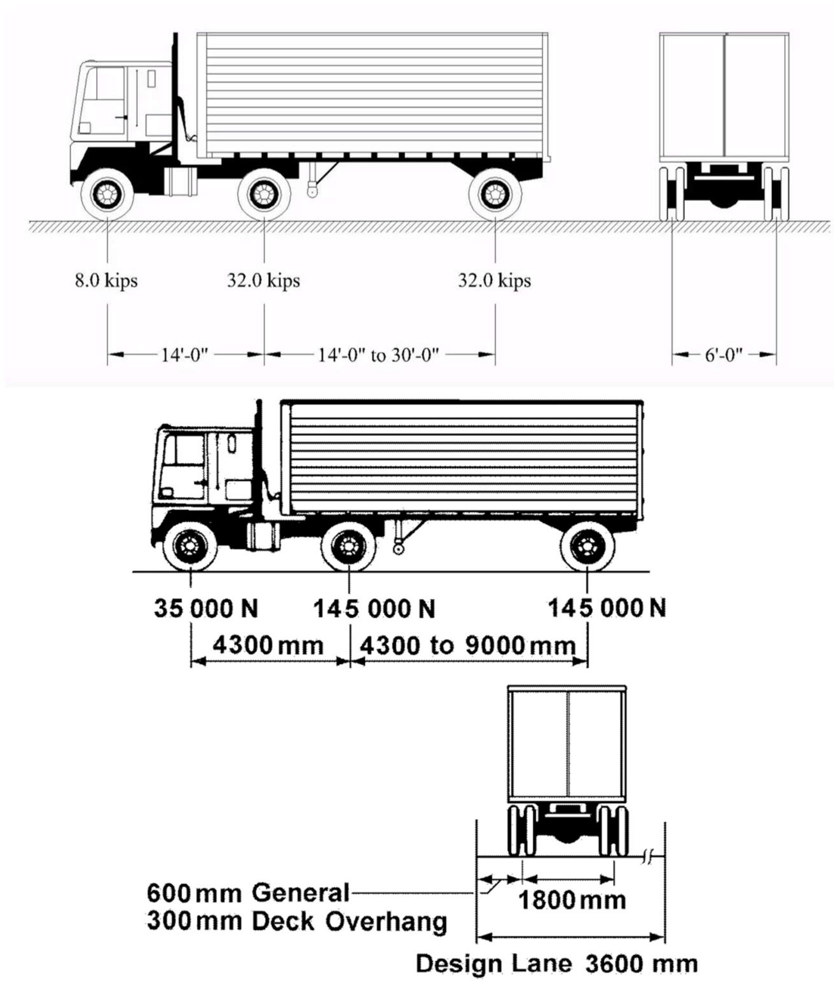



*Figure 4.4.4.2.1.1- HL-93 Truck*

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

<u>Pedestrian Loads:</u>

A pedestrian load of $3.6\times10^{-3}$ 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 $4.1\times10^{-3}$ 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**

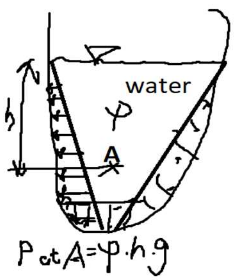
| Load Combination | 3-Second Gust Wind Speed (km/h) |
|---|---|
| Strength III | Wind speed to be taken from country specific data |
| Strength V | 130 |
| Service I | 115 |
| Service IV | 0.75 of the speed used for the Strength III limit state |

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.)

$\gamma$ = 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:

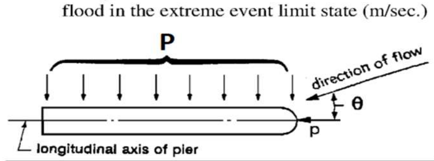
$$p = 5.14\times10^{-4} C_L V^2$$

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.)



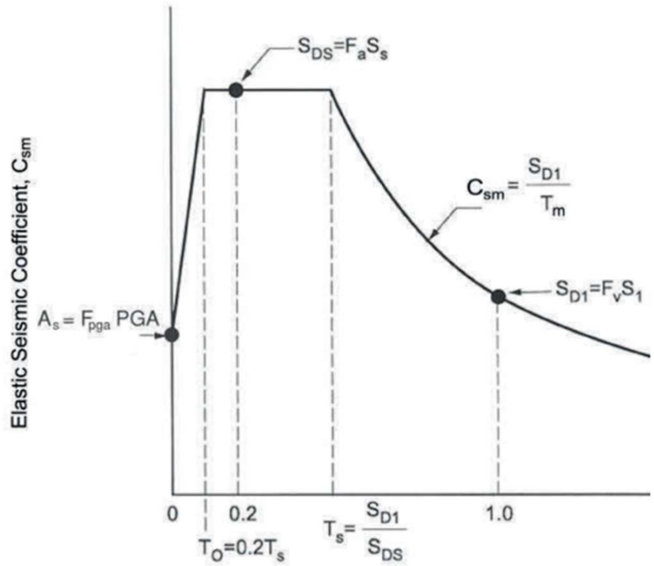

Lateral Drag Coefficient for Piers

| 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.

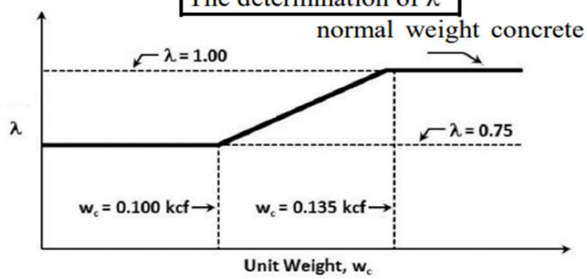
The design concrete compressive strength for prestressed concrete and decks shall not be less than 4.0 ksi.

**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: $6.0 \times 10^{-6}/°F$

For lightweight concrete: $5.0 \times 10^{-6}/°F$

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, $\varepsilon_{sh}$, 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
$\phi$ = 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

$\lambda$ = concrete density modification factor

where $\lambda$ :

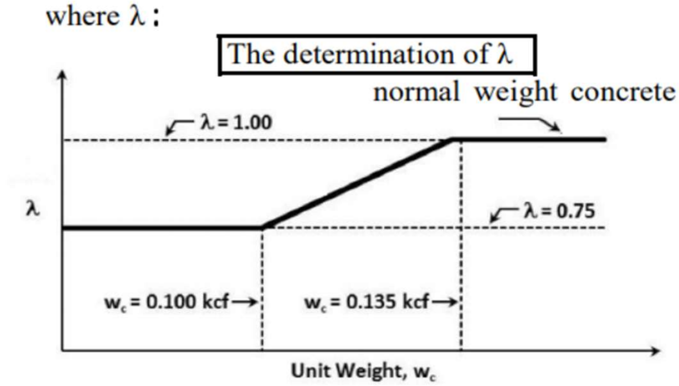



*Figure 4.4.5.6.1 – Determination of Lambda*

**Illustration of the Terms $b_v$ and $d_v$**

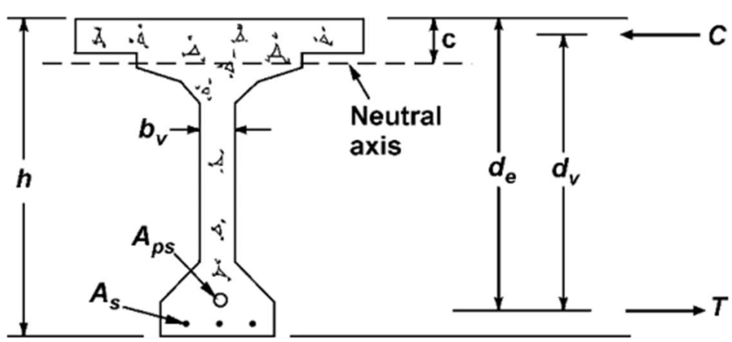



*Figure 4.4.5.6.2 – Illustration of the Terms $b_v$ and $d_v$*

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

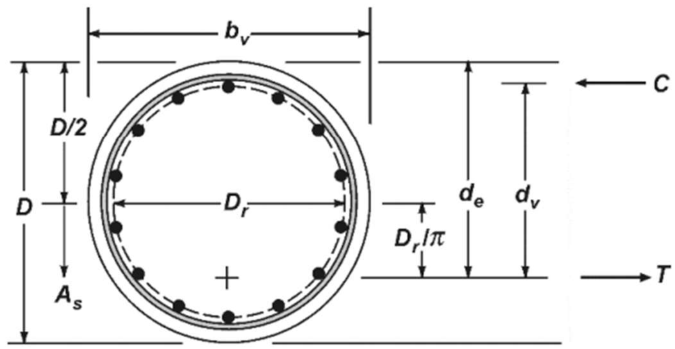



*Figure 4.4.5.6.3 – Illustration of the Terms $b_v$, $d_v$ and $d_e$ for Circular Sections*

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:

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

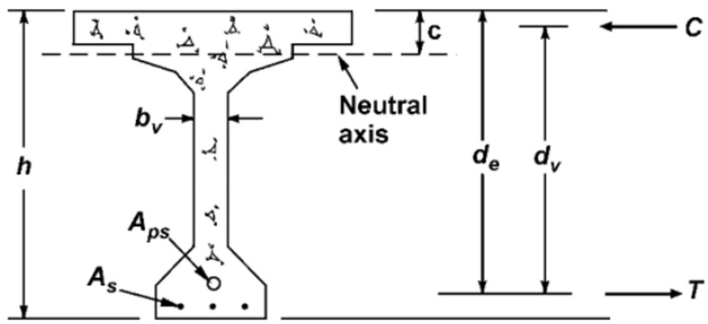



*Figure 4.4.5.6.4 – Illustration of the Terms $b_v$ and $d_v$*

## 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)
$\mu$ = friction factor
$\alpha$ = 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, $\Delta f_{pLT}$, 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)

$\gamma_h$ = correction factor for relative humidity of the ambient air

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

$\Delta f_{pR}$ = 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<br><br>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<br><br>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 | 133 1/3 |
| 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.

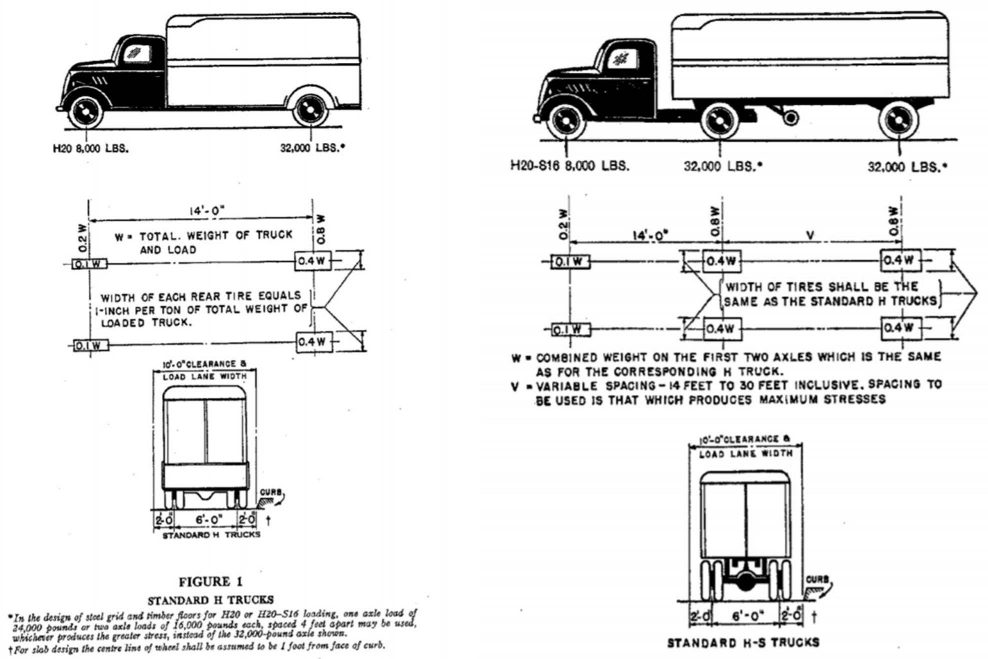



*Figure 4.5.4.1 – CSA S6-66 Design Trucks (Left - Standard H Truck, Right - Standard H-S truck)*

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.

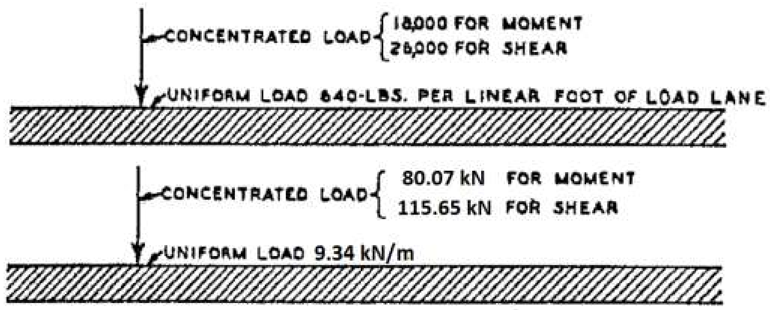



*Figure 4.5.4.2 – CSA S6-66 Lane Loading*

### 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
- $\alpha$ = 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.

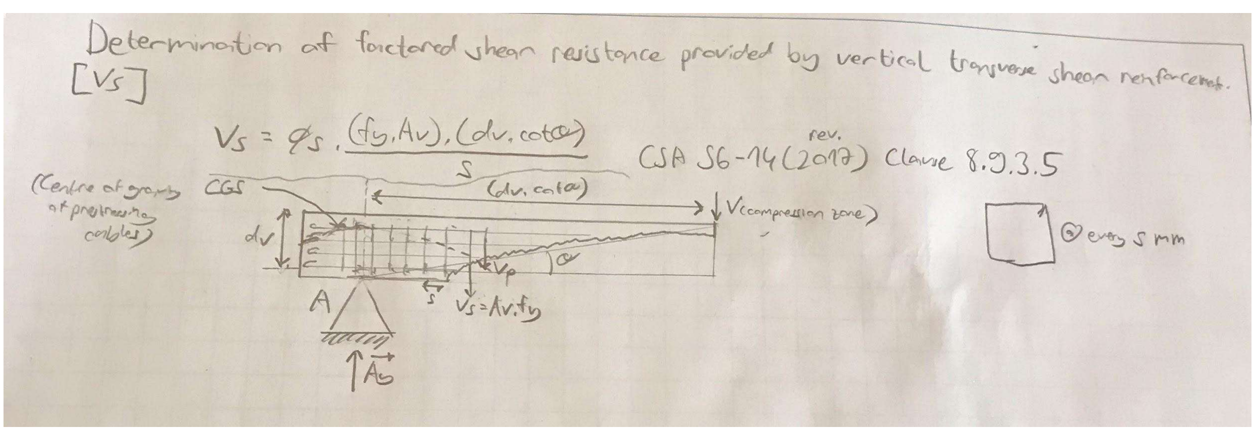
[2] American Association of State Highway and Transportation Officials. Bridge Design Specifications - 8th Edition - September 2017

[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:

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

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

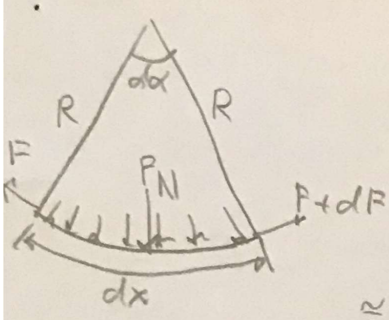
$$f_{cr} = 0.4\sqrt{f'_c} \Rightarrow$$

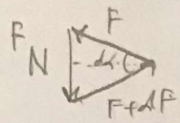
$$V_c = \varphi_c \beta \sqrt{f'_c}\, b_v d_v \quad \text{Max } f'_c \leq 60\ \text{MPa}$$

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:




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

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

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

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

small angle assumption

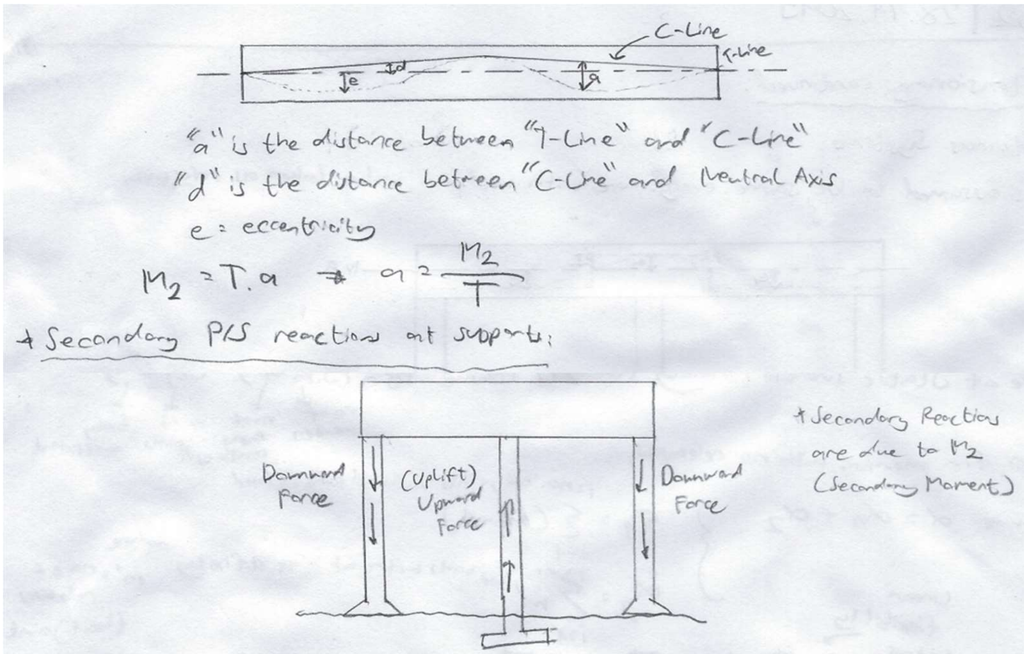
$$\int_{F_i}^{F_x} \frac{dF}{F} = k\int_{\alpha}^{x} dx + \mu\int_{0}^{\alpha} d\alpha$$

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

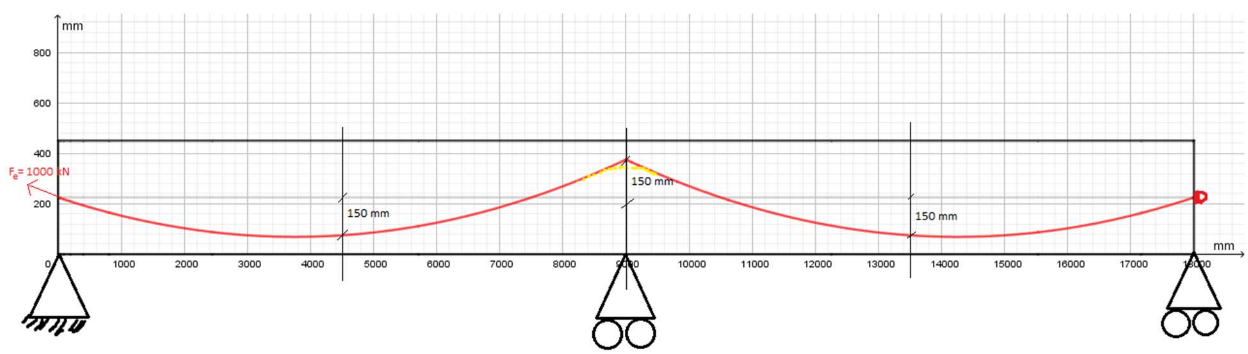
$$F(x) = F_i \cdot e^{-(\mu\alpha + kx)}$$

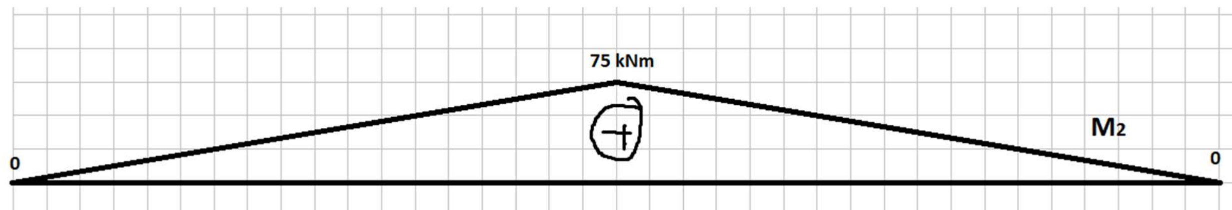
K & B

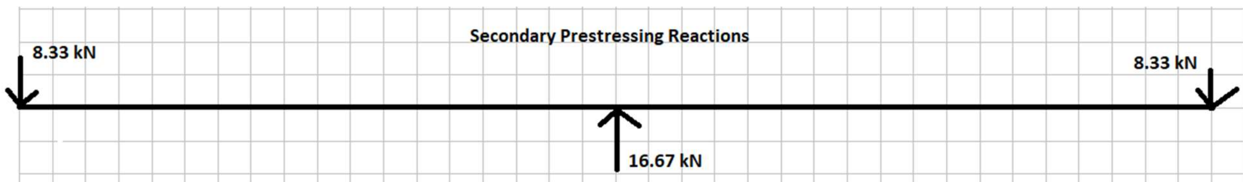
## Secondary Prestressing Forces/Reactions Explained:




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)



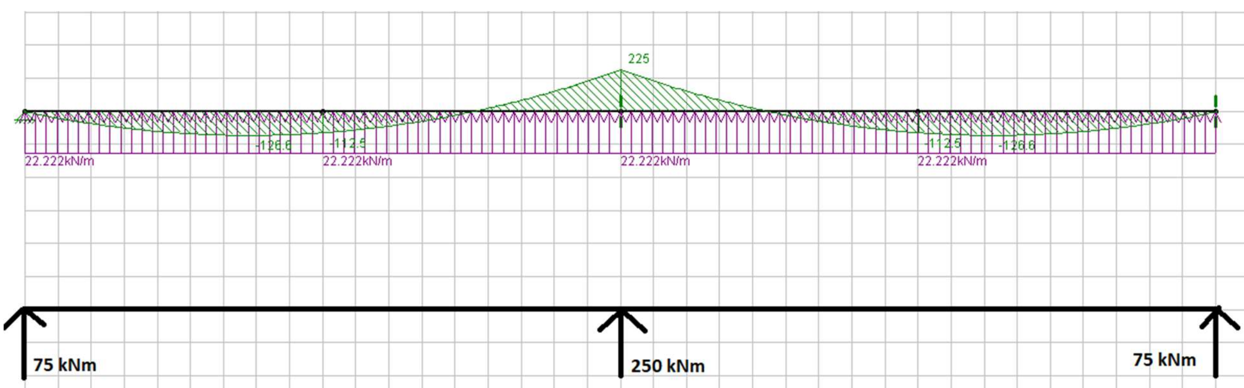


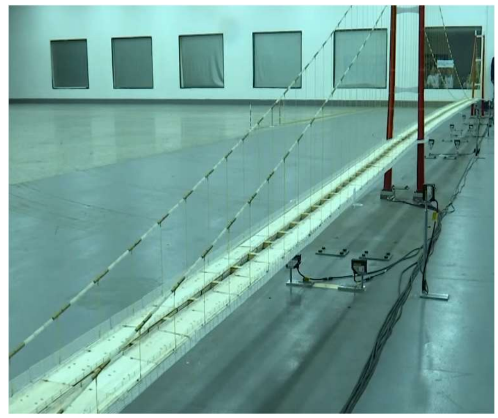

Total Reactions/Forces due to Post-tensioning only example:

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:



<u>-Wind Tunnel Analysis Example:</u>
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.

<u>-Time History Analysis on a bridge using software:</u>


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.

UNIVERSITY OF TORONTO

# Chapter 5 – Required Design Input

## Table of Contents



K & B

## 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.

## 5.7 References
[1]. “Budget for building design and construction projects,” *Budget for building design and construction projects - Designing Buildings Wiki*. [Online]. Available:

https://www.designingbuildings.co.uk/wiki/Budget_for_building_design_and_construction_projects. [Accessed: 05-Apr-2020].

[2]. *Construction Project Scheduling: A Basic Guide*. [Online]. Available: https://www.traunerconsulting.com/construction-project-scheduling-a-basic-guide/. [Accessed: 05-Apr-2020].

[3]. T. Pruiett, “Differing Site Conditions,” *https://www.smithcurrie.com/*, 04-Dec-2018. [Online]. Available: https://www.smithcurrie.com/publications/common-sense-contract-law/back-basics-differing-site-conditions/. [Accessed: 05-Apr-2020].

[4]. “Bridge Type Selection,” *Department of Transportation*. [Online]. Available: http://www.dot.state.mn.us/stateaid/bridge/docs/bridge-type-selection.pdf. [Accessed: 05-Apr-2020].

[5]. “Selection of Bridge Type.” [Online]. Available: https://www.dot.ny.gov/divisions/operating/opdm/passenger-rail/passenger-rail-repository/Appendix C - Selection of Bridge Type.pdf. [Accessed: 05-Apr-2020].

[6]. “WHEN IS A TOPOGRAPHIC SURVEY NEEDED,” *challengergeomatics*, 03-Jan-2018. [Online]. Available: https://www.challengergeomatics.com/when-is-a-topographic-survey-needed/. [Accessed: 05-Apr-2020].

[7]. “A Good Foundation - Lesson,” *TeachEngineering.org*, 12-Jun-2019. [Online]. Available: https://www.teachengineering.org/lessons/view/cub_brid_lesson03. [Accessed: 05-Apr-2020].

[8]. Prucz1 and A. Pourbohloul, “BRIDGE CONFIGURATIONS AND DETAILS THAT IMPROVE SEISMIC PERFORMANCE.” [Online]. Available: https://www.arema.org/files/library/1999_Conference_Proceedings/00005.pdf. [Accessed: 05-Apr-2020].

[9]. *WIND MONITORING IMPORTANCE IN BRIDGE CONSTRUCTION AND OPERATION*. [Online]. Available: https://www.windcrane.com/blog/construction/wind-monitoring-importance-bridge-construction-and-operation. [Accessed: 05-Apr-2020].

[10]. *Pollution From Construction*. [Online]. Available: http://www.sustainablebuild.co.uk/pollutionfromconstruction.html. [Accessed: 05-Apr-2020]

[11]. Protective Services Dept., “Noise (By-law No.2017-255),” *City of Ottawa*, 17-Mar-2020. [Online]. Available: https://ottawa.ca/en/living-ottawa/laws-licences-and-permits/laws/law-z/noise-law-no2017-255. [Accessed: 05-Apr-2020]

[12]. “Definitions for constructability,” *What does constructability mean?* [Online]. Available: https://www.definitions.net/definition/constructability. [Accessed: 05-Apr-2020].

# 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.

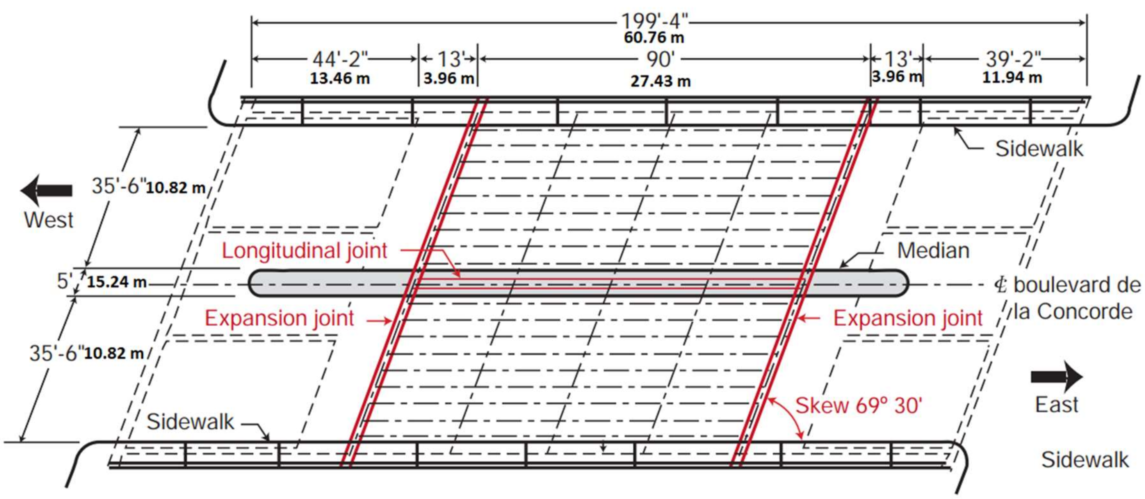



*Figure 6.2.1 - Plan View of the de la Concorde Overpass from Autoroute (Highway) 19*
*Reference: Exhibit COM-62, p. 7 (Court Materials)*

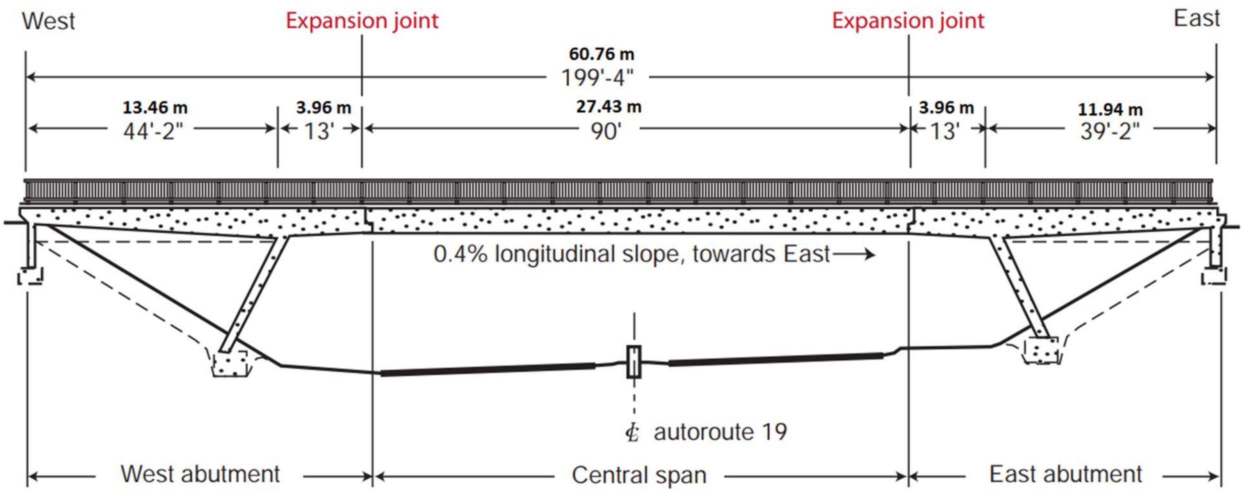



*Figure 6.2.2 - Elevation View of the de la Concorde Overpass from Autoroute (Highway) 19*
*Reference: Exhibit COM-62, p. 7 (Court Materials)*

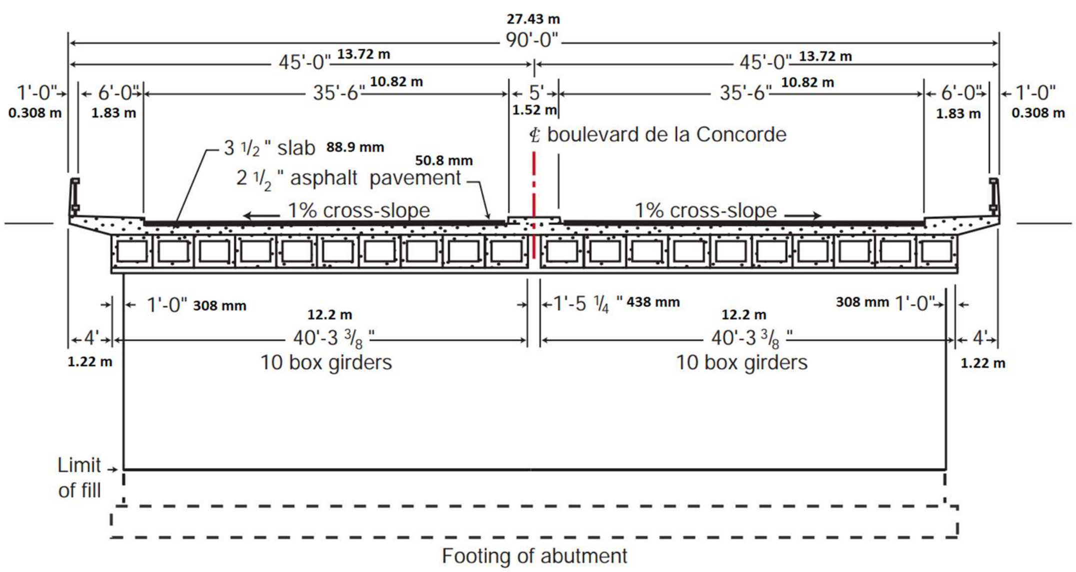



*Figure 6.2.3 - Cross-section of the de la Concorde Overpass from Autoroute (Highway) 19*
*Reference: Exhibit COM-62, p. 7 (Court Materials) - We converted the dimensions to SI units.*

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.

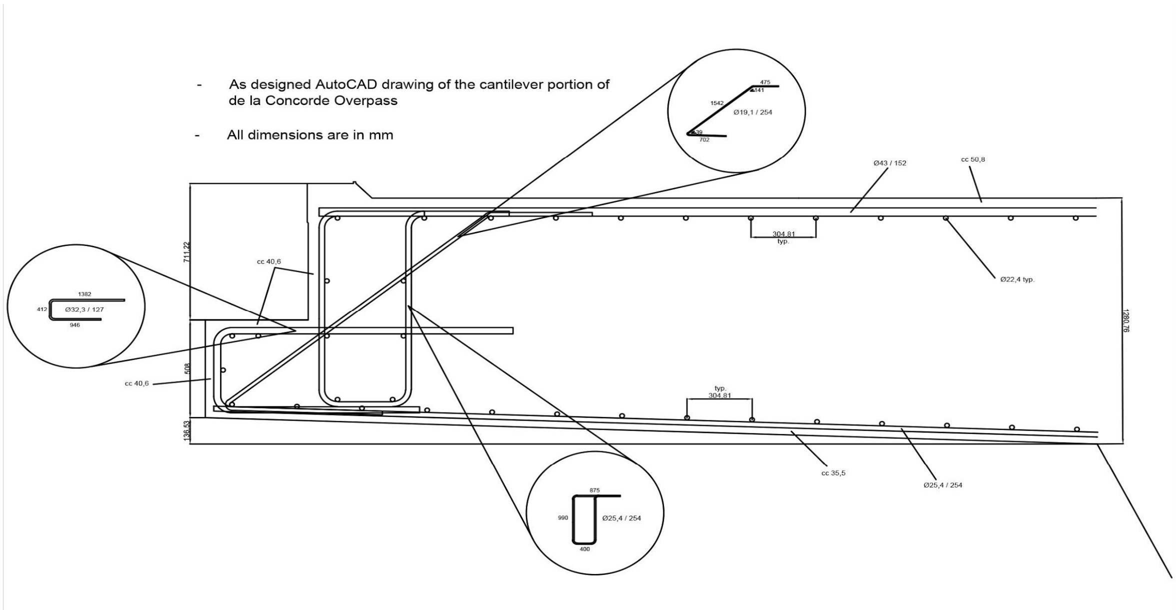



*Figure 6.2.4 - Our AutoCAD drawing of the cracked cantilever portion of de la Concorde Overpass (as designed)*

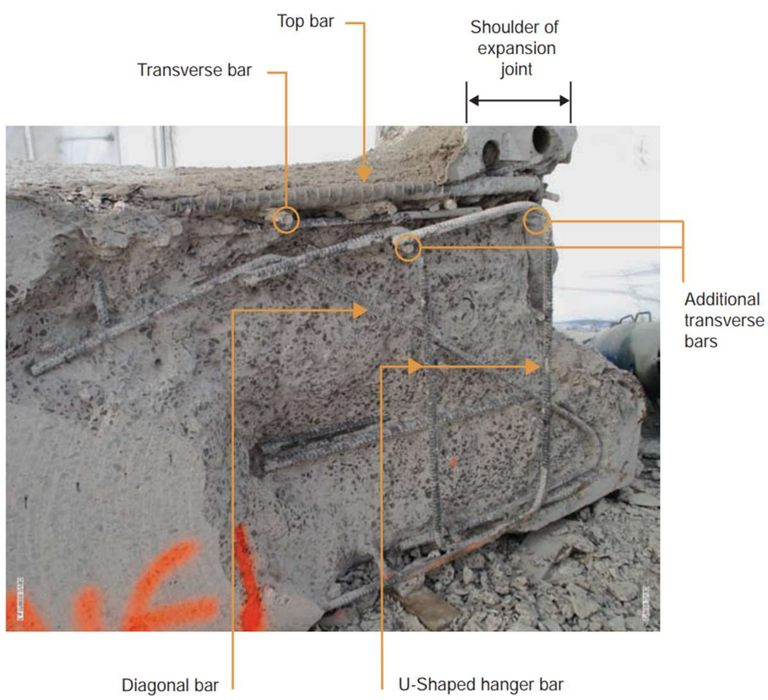



*Figure 6.2.5 - Bars at bearing support area as built [5]*

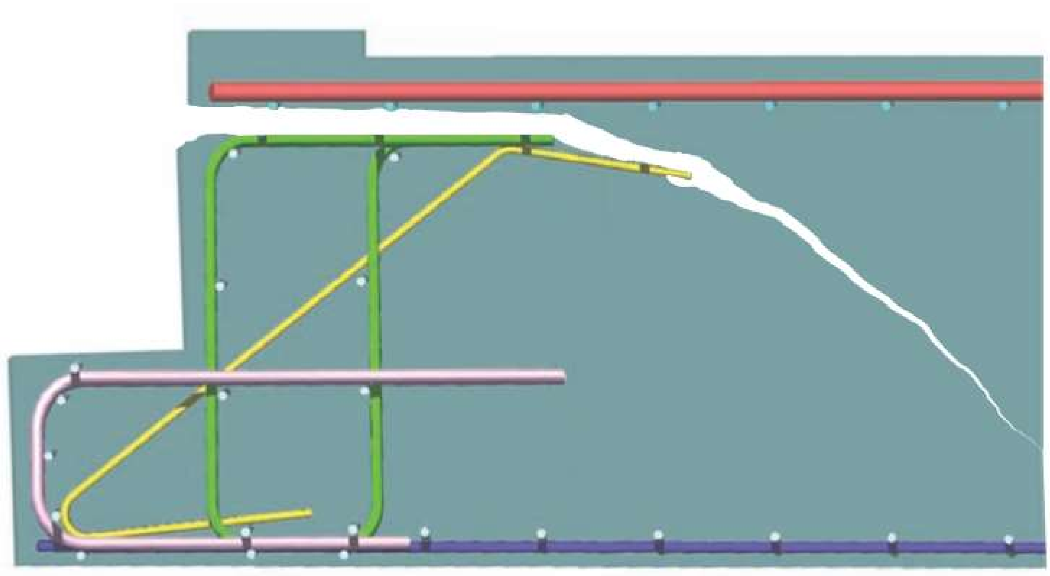


*Figure 6.2.6 - The crack that occurred at the cantilever portion of de la Concorde Bridge (taken from [5] and edited to show the crack)*

## 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.

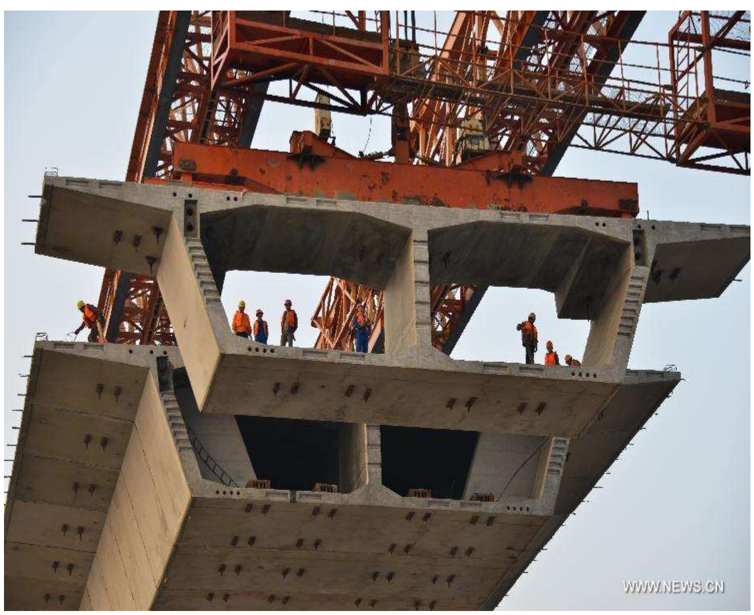



*Figure 6.4.1.1 - Construction of Box Girder in section using special type of cranes [2].*

#### 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.

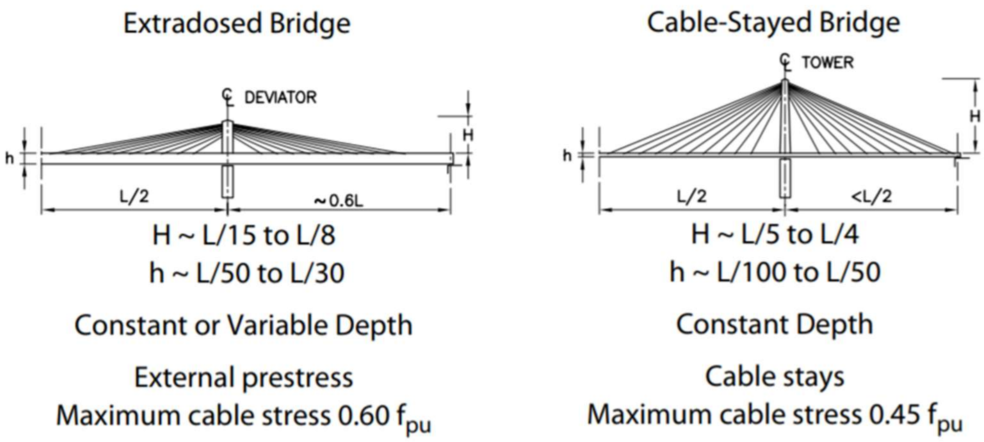



*Figure 6.4.2.1 - Comparison between Extradosed and Cable Stayed Bridge [3]*

*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].

<u>-Aesthetics:</u> 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.

<u>-Structural Performance:</u> 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

<u>-Cost:</u> 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].

<u>-Ease of Maintenance:</u> 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.

<u>-Community Impact:</u> 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.

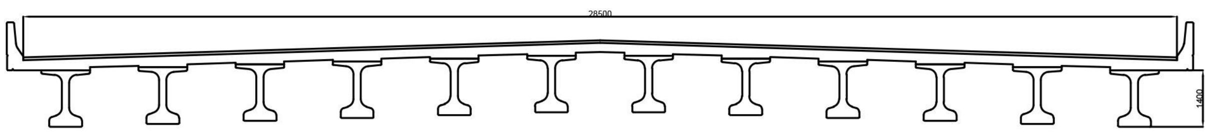



*Figure 6.4.3.1 - AutoCAD Drawing of cross-section of proposed replacement bridge @10 m.*

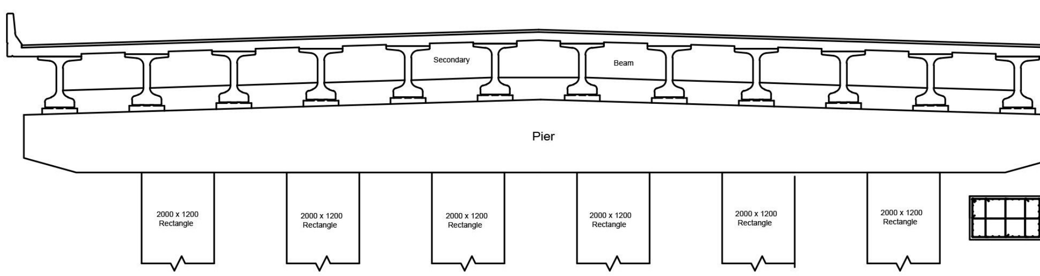



*Figure 6.4.3.2 - AutoCAD Drawing of midspan cross-section of proposed replacement bridge*

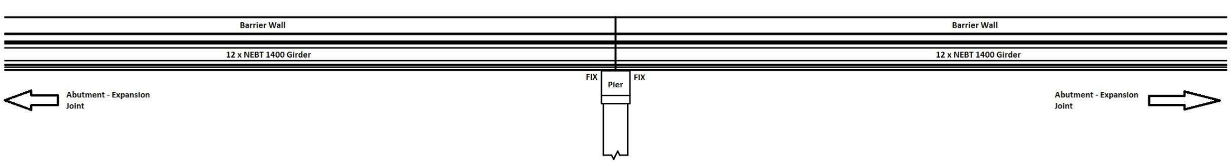



*Figure 6.4.3.3 - AutoCAD Drawing of elevation view of proposed replacement bridge*

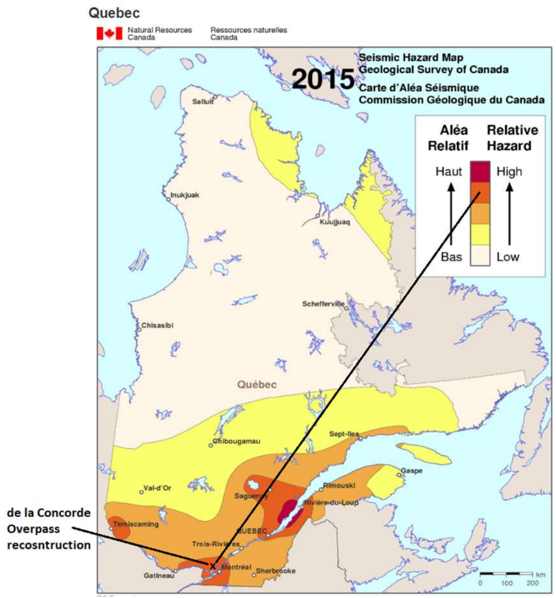



*Figure 6.4.3.4 - 2015 Seismic Hazard Map of Quebec [6]*

<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

[1]. “Box Girder Bridges,” *Box Girder Bridges - an overview | ScienceDirect Topics*. [Online]. Available: https://www.sciencedirect.com/topics/engineering/box-girder-bridges. [Accessed: 06-Apr-2020].

[2]. S. Posted in Bridges, “Construction of Box Girder Bridges - Specifications, Uses, and Benefits,” *The Constructor*, 07-Mar-2019. [Online]. Available: https://theconstructor.org/structures/construction-box-girder-bridge/2166/. [Accessed: 06-Apr-2020].

[3]. K. Mermigas, “Behaviour and Design of EXTRADOSED BRIDGES,” 2008. [Online]. Available: https://tspace.library.utoronto.ca/bitstream/1807/17203/1/Mermigas_Konstantinos_K_200811_MASc_thesis.pdf. [Accessed: 05-Apr-2020].

[4]. “Extradosed/Cable-Stayed.” [Online]. Available: https://ddot.dc.gov/sites/default/files/dc/sites/ddot/publication/attachments/LongBridge_Chapter6B.pdf. [Accessed: 05-Apr-2020].

[5]. “Commission of Inquiry into the collapse of a portion of de la Concorde Overpass,” 15-Oct-2007. [Online]. Available: http://cip.management.dal.ca/publications/report_eng concorde overpass montreal.pdf. [Accessed: 05-Apr-2020]

[6]. Natural Resources Canada, “Simplified seismic hazard map for Canada, the provinces and territories,” *Government of Canada, Natural Resources Canada, Earthquakes Canada*. [Online]. Available: https://earthquakescanada.nrcan.gc.ca/hazard-alea/simphaz-en.php. [Accessed: 06-Apr-2020].

[7]. A. Bardow and R. Seraderian, “Design, Fabrication and Construction of the New England Bulb Tee Girder.” [Online]. Available: https://www.pci.org/PCI_Docs/PCI_Northeast/Technical_Resources/Bridge/NEBT_Journal_Article_JR410.pdf. [Accessed: 05-Apr-2020].

# 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):

<u>Steel:</u>
-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)

<u>Concrete:</u>
-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:

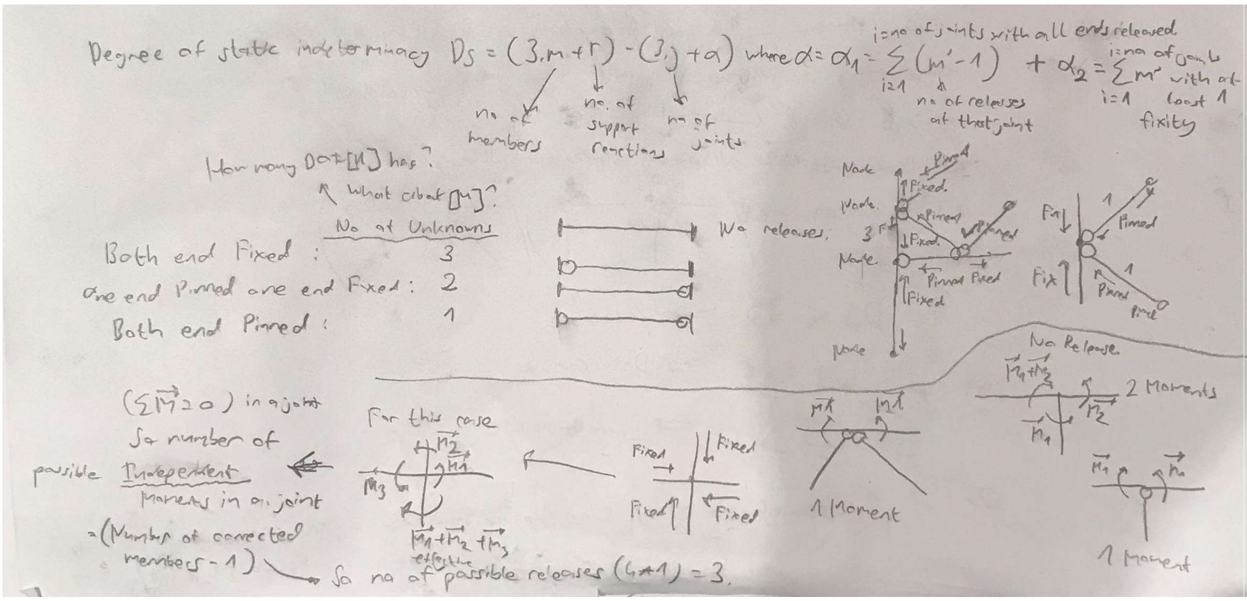



*Figure 7.2.3.3.1*- Degree of Statical Indeterminacy

### 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.

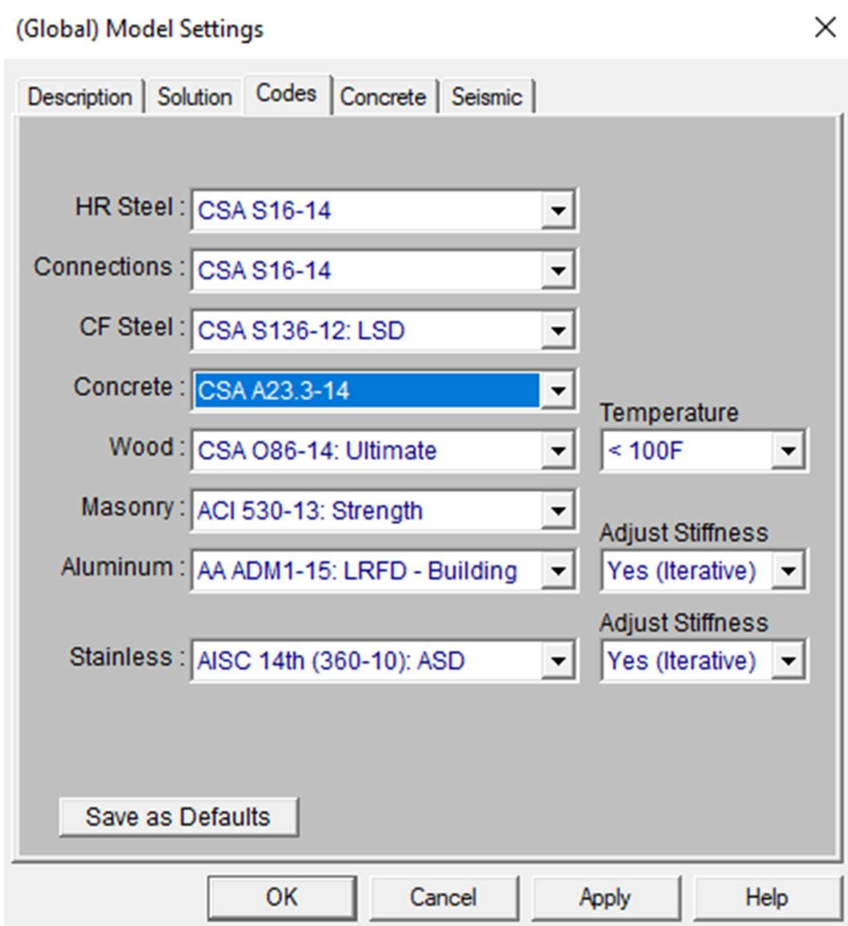



*Figure 7.5.1.1- Screenshot of the Design Code Selection Menu [1]*

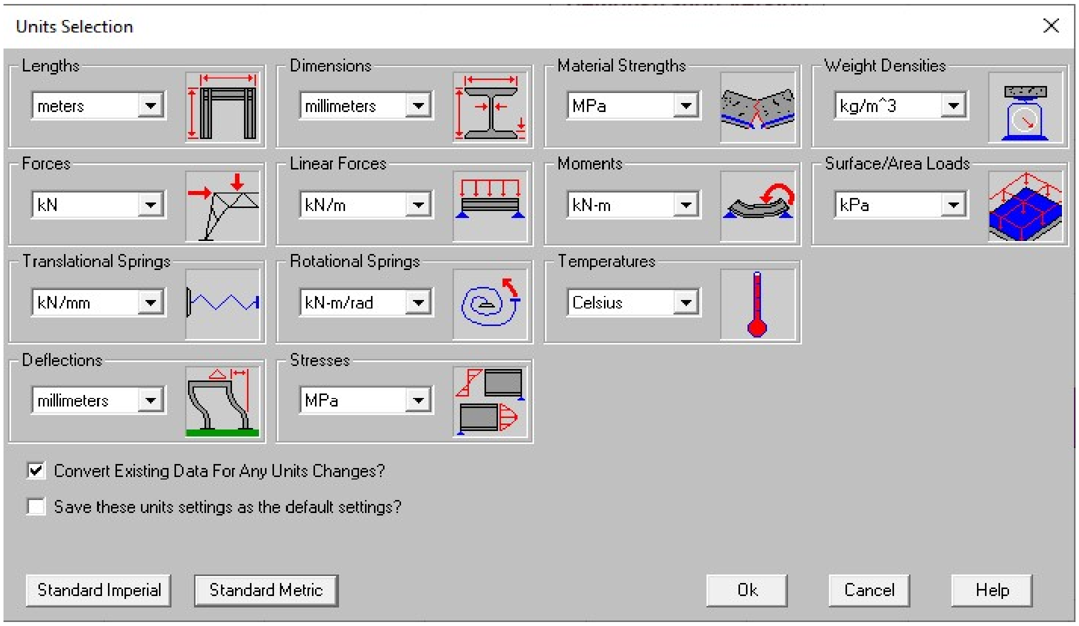
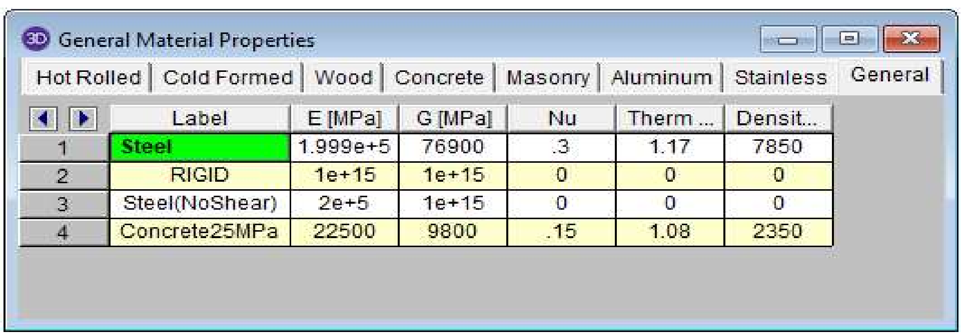
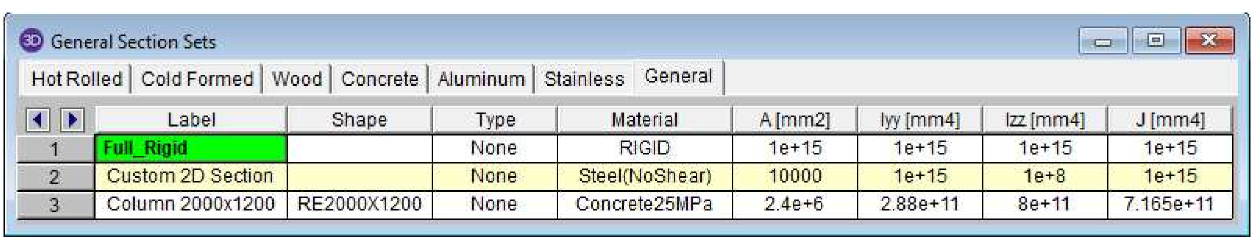



*Figure 7.5.1.2- Unit Selection [1]*




| | 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]*

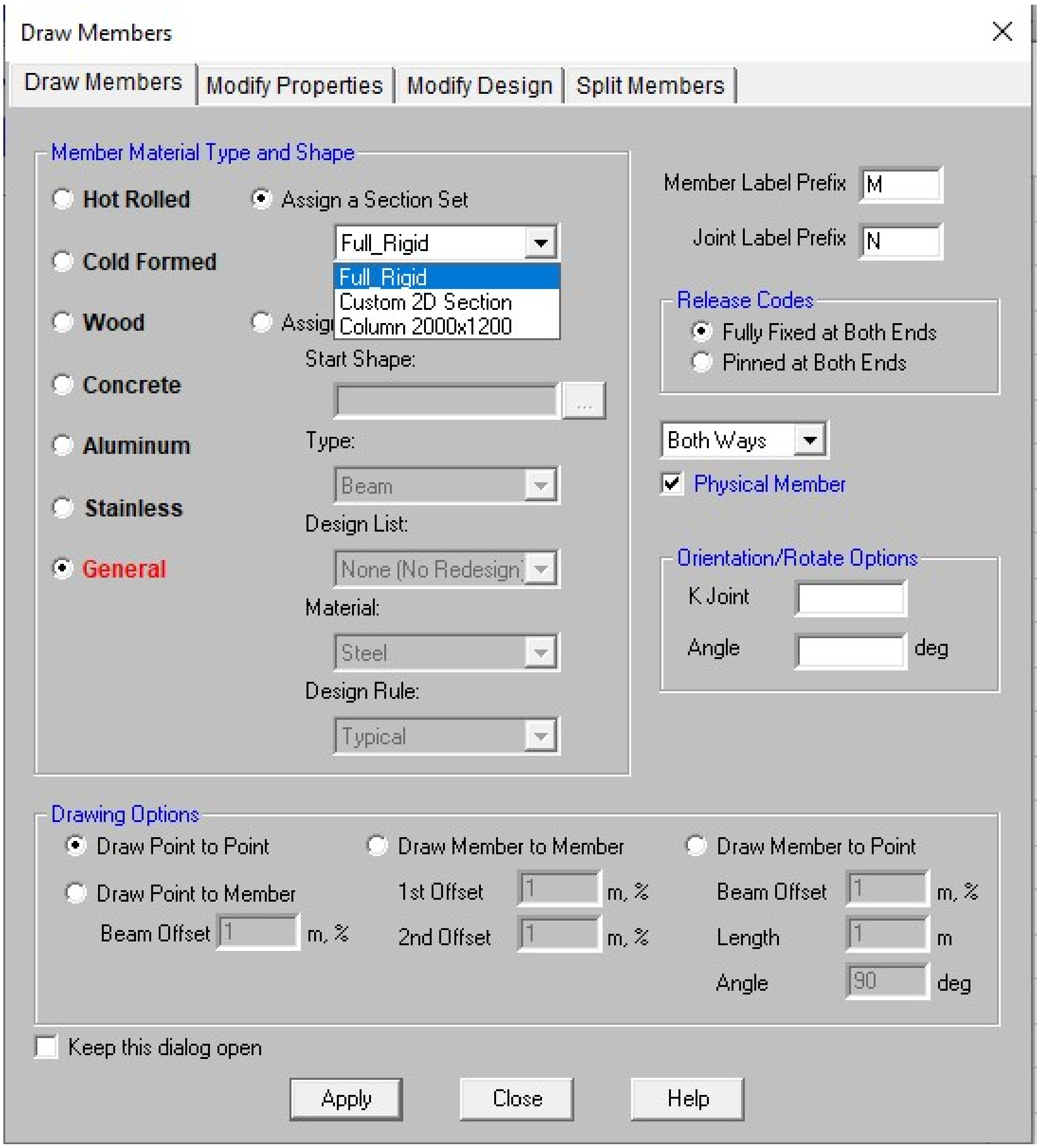



*Figure 7.5.1.5- Member Drawings Menu [1]*

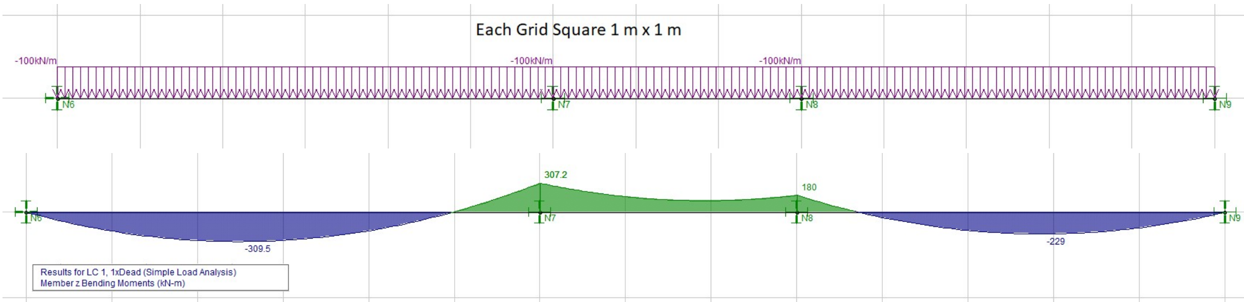



*Figure 7.5.1.6- Moment Diagram of a Continuous Three Span Beam under Distributed Load [1]*

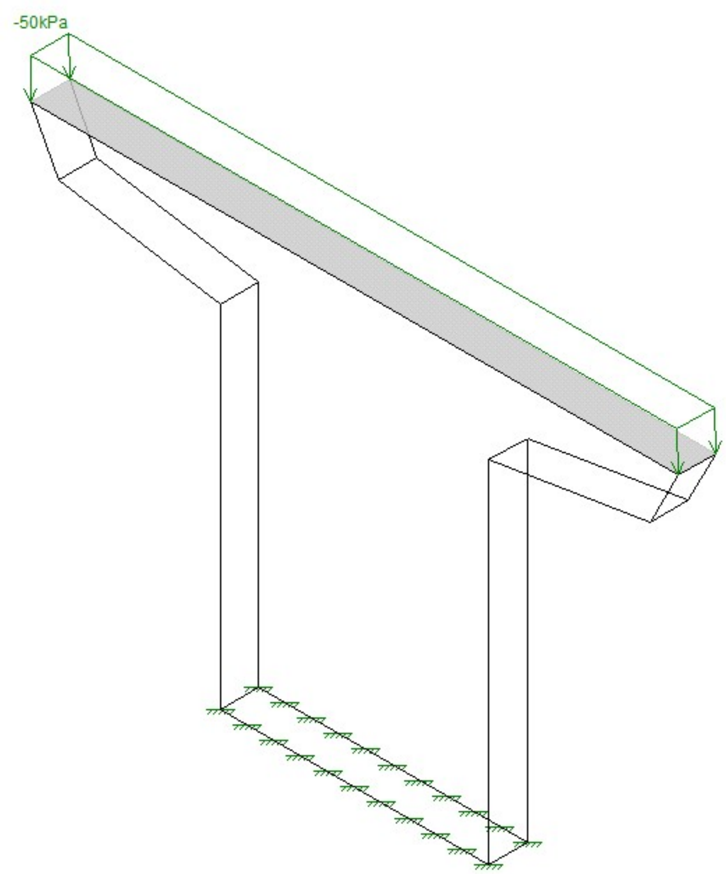



*Figure 7.5.1.7- Sample Bridge Pier Drawing [1]*

### 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.

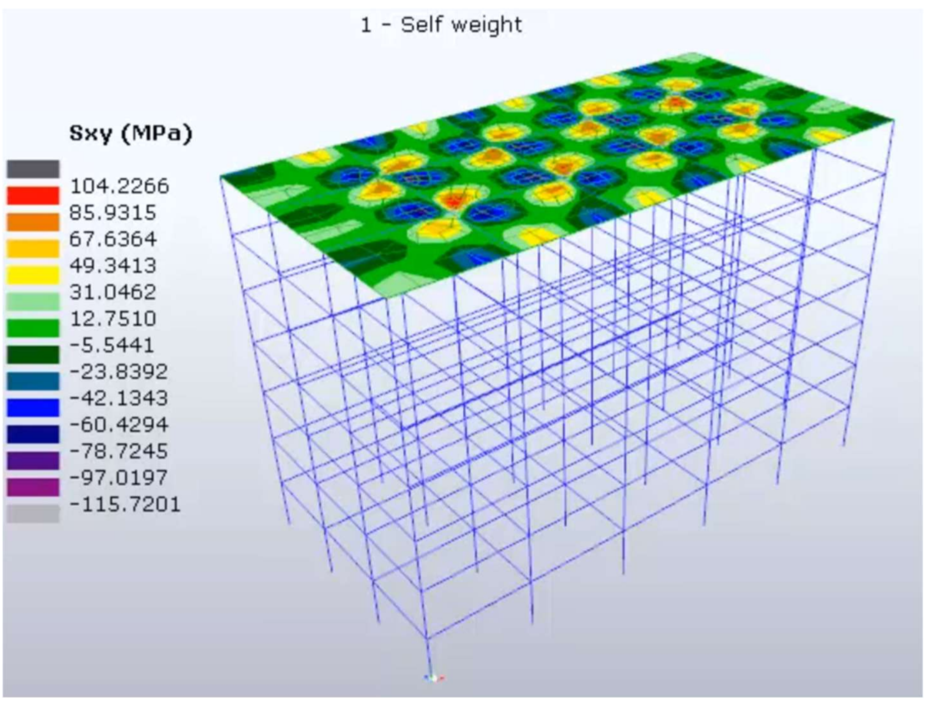



*Figure 7.5.2.1- Self-Weight stress contours of a seventh story of a sample building in S-Frame [3]*

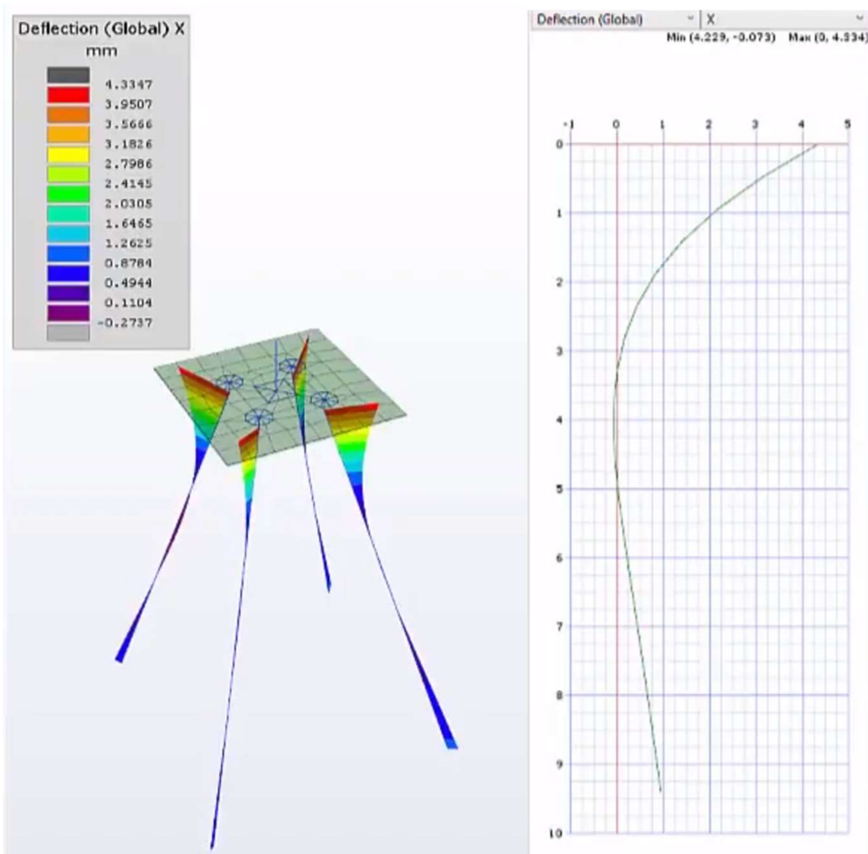



*Figure 7.5.2.2- x-Deflections in a typical pile foundation in S-Foundation*

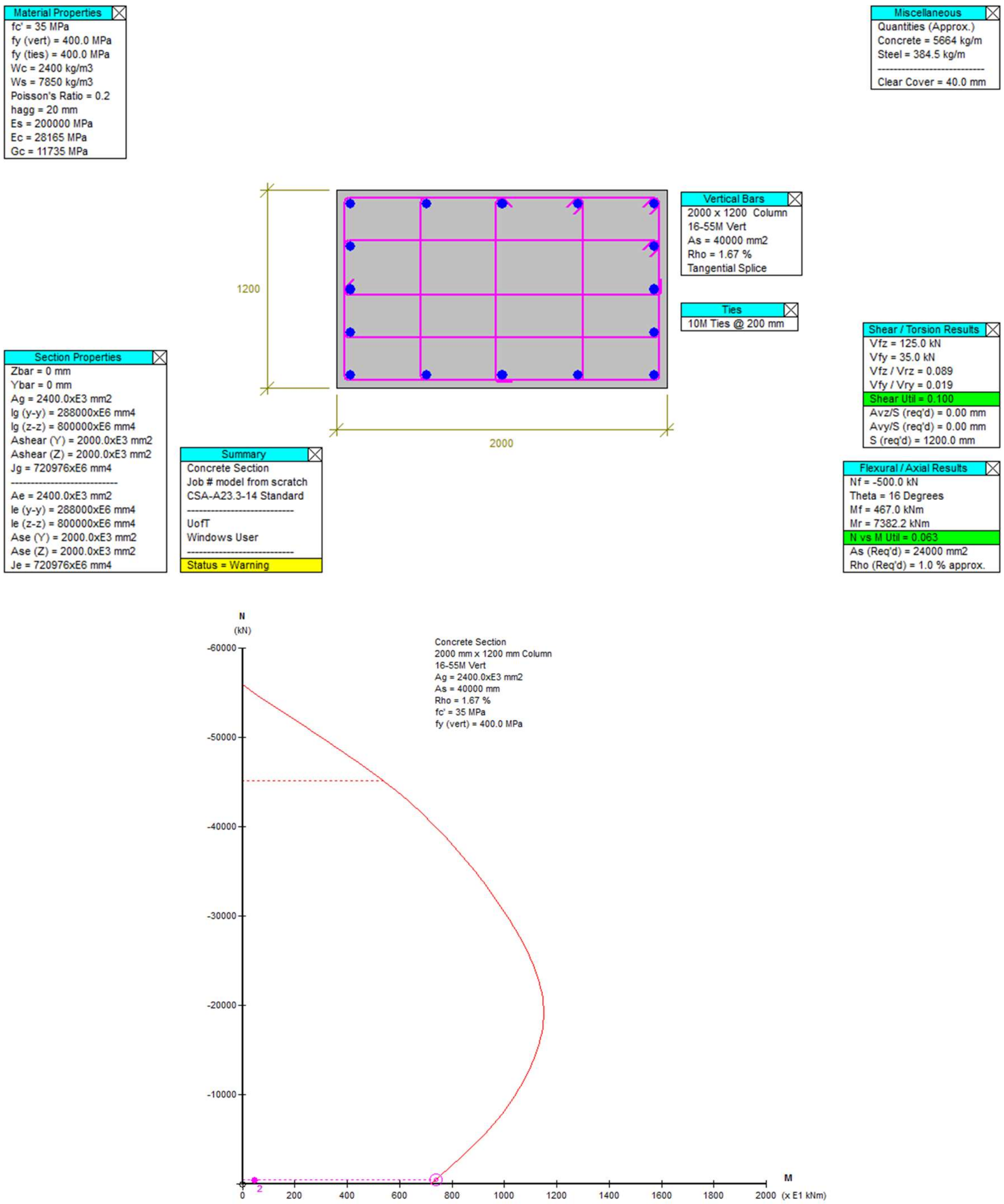



*Figure 7.5.2.3- Interaction Diagram of a 2000x1200mm Column in S-Frame*

### 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].

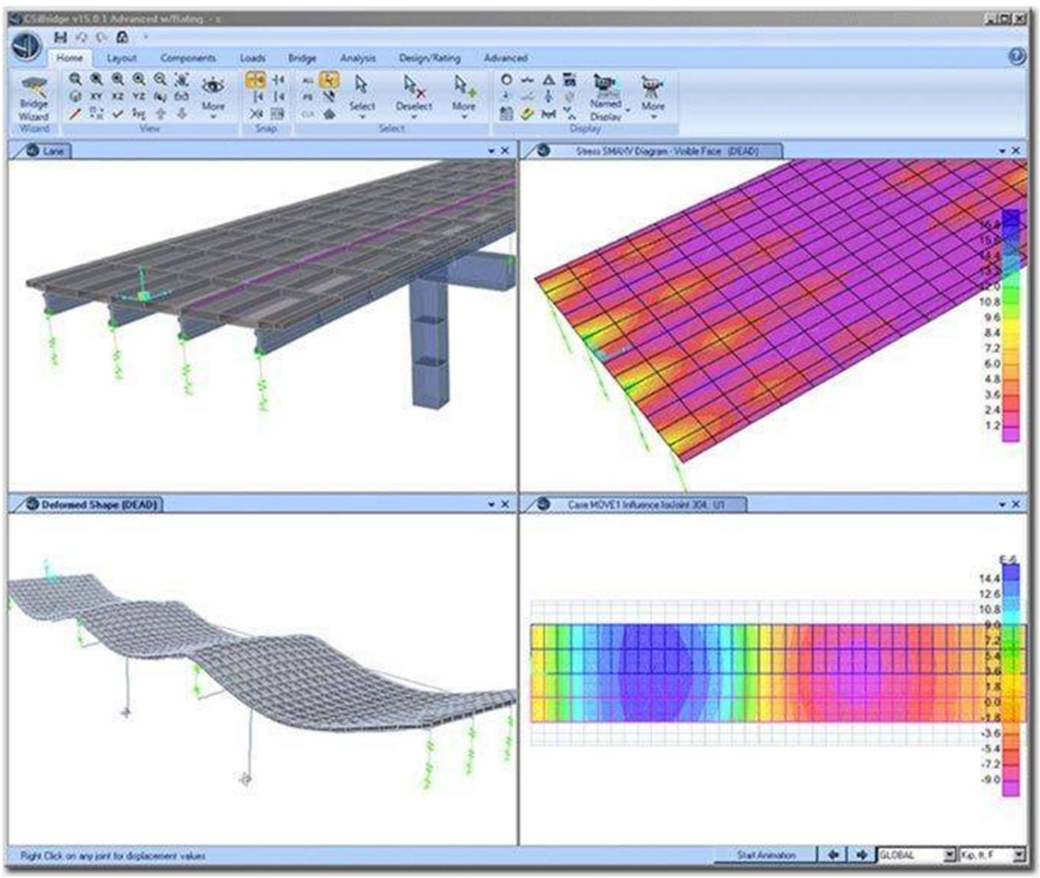


*Figure 7.5.3.1- User Interface Platform in CSI Bridge [4]*

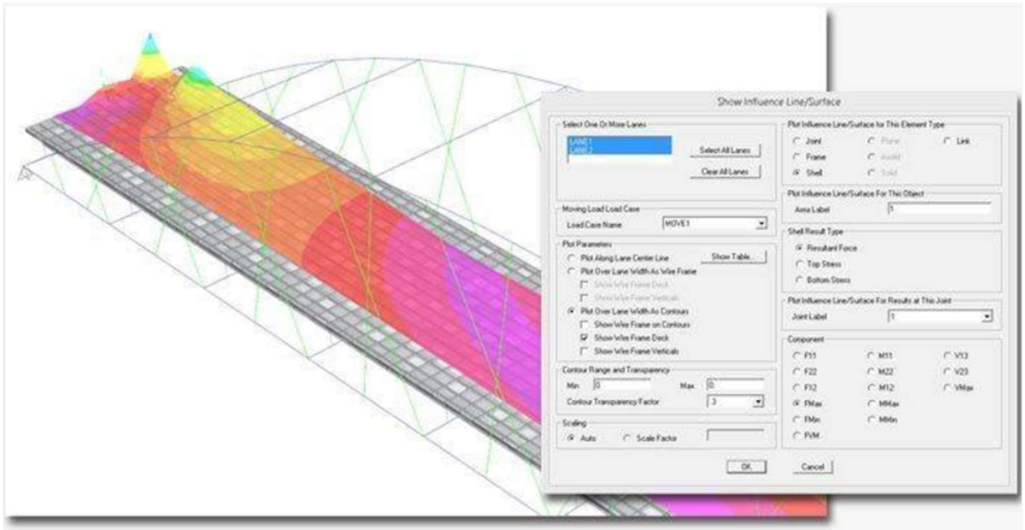



*Figure 7.5.3.2- Influence Surface along a Traffic Lane*

## 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

[1]. RISA 3D v.17.0.4

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

[3]. S-Frame Advanced 2017

[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.

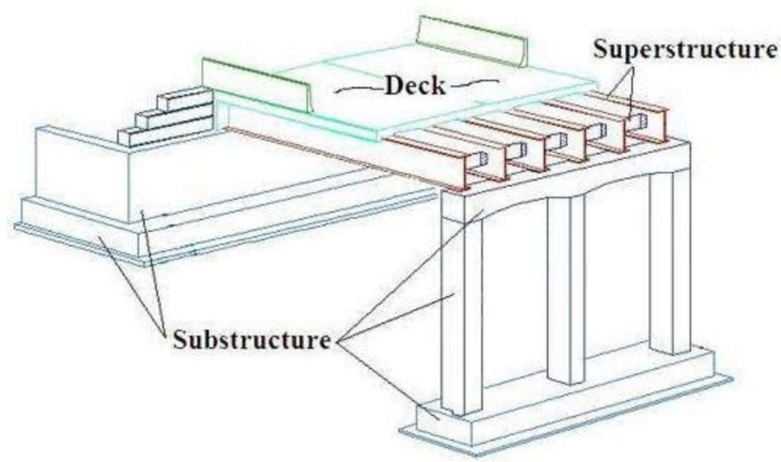



*Figure 8.2.1 - Three Main Structural Elements of a Bridge [1]*

### 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

$\gamma_c$ = 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**

| | Nominal Dimensions | | | | Deformation Requirements mm | | |
|---|---|---|---|---|---|---|---|
| Bar Designation Number | Cross-Sectional Area mm² | Diameter mm | Perimeter mm | Mass (Weight) Per Unit Length kg/m | 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<br>270 ksi | 1/4 to 0.6<br>3/8 to 0.6 | 250<br>270 | 85% of $f_{pu}$, except 90% of $f_{pu}$ for low-relaxation strand |
| Bar | Type 1, Plain<br>Type 2, Deformed | 3/4 to 1-3/8<br>5/8 to 1-3/8 | 150<br>150 | 85% of $f_{pu}$<br>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 | 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 ($f_{pe}$) | $0.80f_{py}$ | $0.80f_{py}$ | $0.80f_{py}$ |

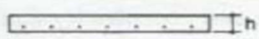
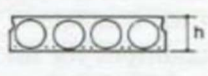
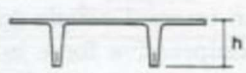
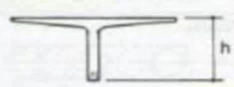
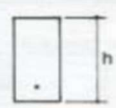
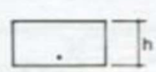
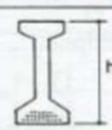
## 8.4 Design of Girder

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 |
|---|---|---|
|  | <dead load | 40 |
|  | 50 (2.4)<br>100 (4.8) | 40-50<br>32-42 |
|  | 50 (2.4)<br>100 (4.8) | 20-30<br>18-28 |
|  | 50 (2.4)<br>100 (4.8) | 23-32<br>19-24 |
|  | <dead load | 20 |
|  | <dead load | 30 |
|  | 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.

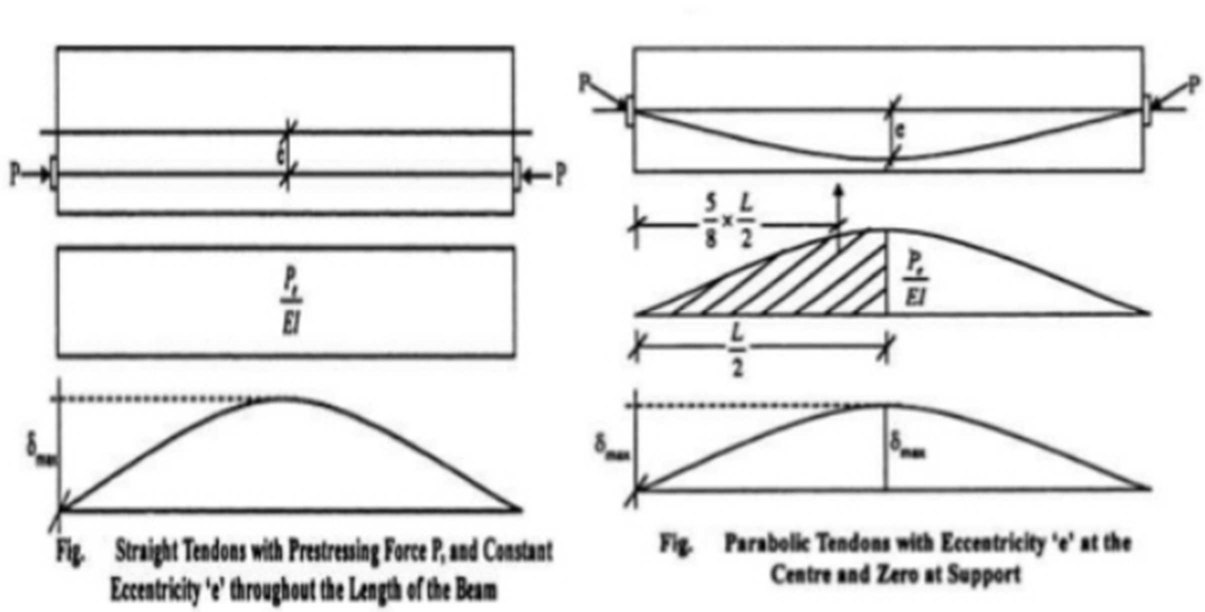



*Figure 8.4.2.1 - Different tendon profiles and their impacts on deflection*

### 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 $\phi_s$ or $\phi_p$.

(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, $\alpha_1 = 0.85 - 0.0015f_c' \geq 0.67$ and $\beta_1 = 0.97 - 0.0025f_c' \geq 0.67$.

*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)

$\phi$ = 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)

$\beta_1$ = 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:

$$\varepsilon_x = \frac{M_f/d_v + V_f - V_p + 0.5N_f - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}$$

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 $\varepsilon_x$ 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 $\varepsilon_x$ 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, $\varepsilon_x$ shall not be less than $-0.20 \times 10^{-3}$.

(g) For sections closer than $d_v$ to the face of the support, the value of $\varepsilon_x$ calculated at $d_v$ from the face of the support may be used in evaluating $\theta$ and $\beta$.

(h) If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in $\varepsilon_x$ shall be taken into account. In lieu of more accurate calculations, the value calculated from the equation shall be doubled.

(i) $\theta$ and $\beta$ may be determined from Clause 8.9.3.7 using a value of $\varepsilon_x$ that is greater than that calculated from the equation in this Clause. However, $\varepsilon_x$ shall not be greater than $3.0 \times 10^{-3}$.

*Figure 8.4.4.1.2 - Determination of $\varepsilon_x$ [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:

$$V_n = V_c + V_s + V_p \qquad (5.8.3.3\text{-}1)$$

$$V_n = 0.25 f'_c b_v d_v + V_p \qquad (5.8.3.3\text{-}2)$$

in which:

$$V_c = 0.083\beta\sqrt{f'_c}\, b_v\, d_v \qquad (5.8.3.3\text{-}3)$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad (5.8.3.3\text{-}4)$$

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)

$\beta$ = factor indicating ability of diagonally cracked concrete to transmit tension as specified in Article 5.8.3.4

$\theta$ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (°)

$\alpha$ = angle of inclination of transverse reinforcement to longitudinal axis (°)

$A_v$ = area of shear reinforcement within a distance $s$ (mm$^2$)

$V_p$ = 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 $\beta$ and $\theta$ 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:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \qquad (C5.8.3.3\text{-}1)$$

The angle $\theta$ 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:

$$s \leq min(1.5 \times h, 450)$$

**11.** Choose a reinforcement bar based on $A_{sreqd.}$

**12.** Compute $M_r$ and ensure the criteria $M_r \geq M_f$

$$M_r = T_s \times (d - a/2)$$
$$T_s = \Phi \times A_s \times f_y$$
$$a = T_s / (\alpha \times f'_c \times b)$$

### 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, $\beta_c$ shall be taken as 1.7.

When cracking is caused by superimposed deformations, $\beta_c$ 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 $\beta_c$ for cross-sections with a minimum dimension between these limits.

$s_{rm}$ shall be calculated as follows (in millimetres):

$$s_{rm} = 50 + 0.25 k_c \frac{d_b}{\rho_c}$$

$k_c$ shall be taken as 0.5 for bending and 1.0 for pure tension.

$\rho_c$ 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.

$\varepsilon_{sm}$ 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

$\rho'$ = 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
$\phi$ = 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
$\lambda$ = 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:

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

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

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

where:

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

*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:

$$\boldsymbol{A_h / (b_w \times s_h) \ge 0.003}$$

$$\boldsymbol{A_v / (b_w \times s_v) \ge 0.003}$$

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:

$$\boldsymbol{I_e = [M_{cr}/ M_a]^3 \, I_g + [1 - (M_{cr}/ M_a)^3] \le I_g}$$

$$\boldsymbol{M_{cr} = f_r \,(I_g / y_t)}$$

## 8.5.3 CSA - S6-66

### 8.5.3.1 Design for Shear

Shear force can be determined using the equation below:

$$\boldsymbol{v = V/(b \times d)}$$

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:

$$A_v = V/(f_v \times \sin\alpha)$$
$$\text{Where } V \leq 1.5 \text{ BD } \sqrt{f'c}$$

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:

$$E \text{ (concrete)} = 6\times10^4 \sqrt{f'c} \text{ (psi)}$$
$$E \text{ (steel)} = 29{,}000{,}000 \text{ (psi)}$$

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

[1]. Ashwin Kesiraju“Structural Elements of a Typical Highway Bridge”, 2017. [Online]. Available”
https://www.researchgate.net/figure/Structural-Elements-of-a-Typical-Highway-Bridge-MDOT-2017_fig1_325477691 [Accessed 06-Apr-2020]

[2]. S. Posted in Bridges, “Bridge Bearings -Types of Bearings for Bridge Structures and Details,” The Constructor, 12-Sep-2017. [Online]. Available:
https://theconstructor.org/structurehttps://theconstructor.org/structures/bridge-bearings-types-details/18062/s/bridge-bearings-types-details/18062/. [Accessed: 06-Apr-2020].

[3]. CSA-S6-14 Canadian Highway Bridge Design Code

[4]. AASHTO 2017

[5]. CSA S6-66 Design of Highway Bridges

[6]. CSA G30.18-09 (R2017)

[7]. Michael P. Collins and Denis Mitchell, “Design for Flexure,” in Prestressed Concrete Structures,. Toronto and Montreal, Canada: Response, 1997.

[8]. M. Rahman, “Shear, bond bearing,camber & deflection in prestressed concrete,” LinkedIn SlideShare, 12-Apr-2014. [Online]. Available: https://www.slideshare.net/Mahfuzcv/shear-bond-bearingcamber-deflection-in-prestressed-concrete. [Accessed: 06-Apr-2020].

# 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<br>Spraying and Fogging<br>Saturated Wet Coverings<br>Left pin-place forms |
| Reducing Loss of Water | Covering concrete with impervious paper or plastic sheets<br>Applying membrane-forming curing compounds |
| Accelerating Concrete Strength Gain | Live steam<br>Heating Coils<br>Electrical heated forms or pads<br>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.

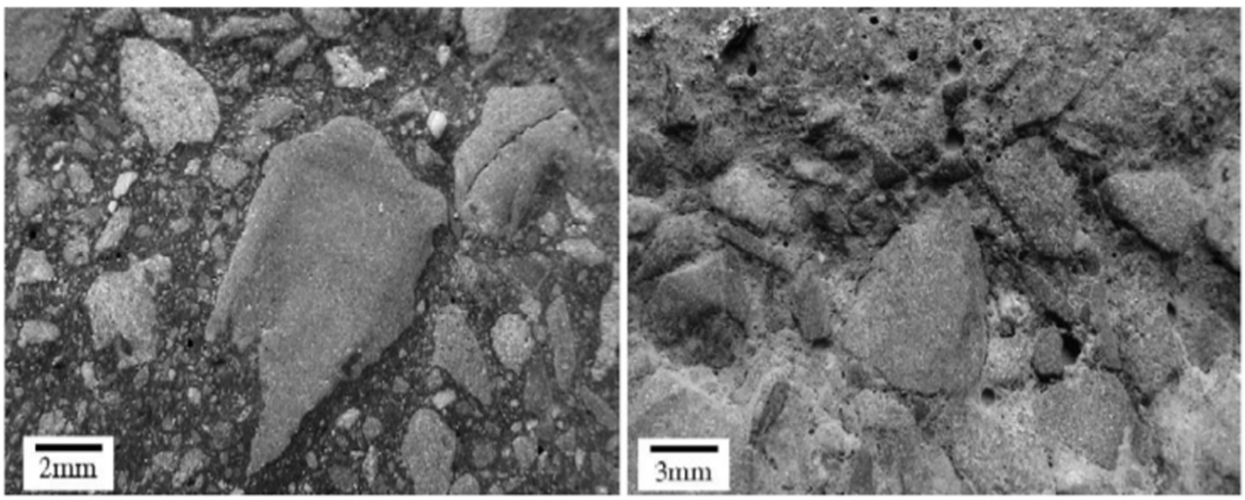



*Figure 9.3.1.2.1 - Abraded surface of concretes with varying w/cm ratios - 0.28 (left) and 0.36 (right) [8]*

### 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]*

## 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.<br>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.<br>Examples: garage floors, porches, steps, pavements, sidewalks, curbs, and gutters. |
| C-3 | Continuously submerged concrete exposed to chlorides, but not to freezing and thawing.<br>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.<br>Examples: underground parking slabs on grade. |
| F-1 | Concrete exposed to freezing and thawing in a saturated condition, but not to chlorides.<br>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.<br>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.<br>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.<br>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.<br>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.<br>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.<br>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.<br>Examples: interior slabs on grade. |

*(Continued)*

*Figure 9.3.2.2.1 - Classes of Exposure [14]*

## 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.

### 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]*

## 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.)

| | | | | Curing type (see Table 19) | | | |
|---|---|---|---|---|---|---|---|
| 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 | Normal concrete | HVSCM-1 | HVSCM-2 | Chloride on penetrability requirements and age at test‡ |
| 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]*

**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 | Performance requirements§,§§ — Maximum expansion when tested using CSA A3004-C8 Procedure A at 23 °C, % — At 12 months†† | Performance requirements§,§§ — 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.

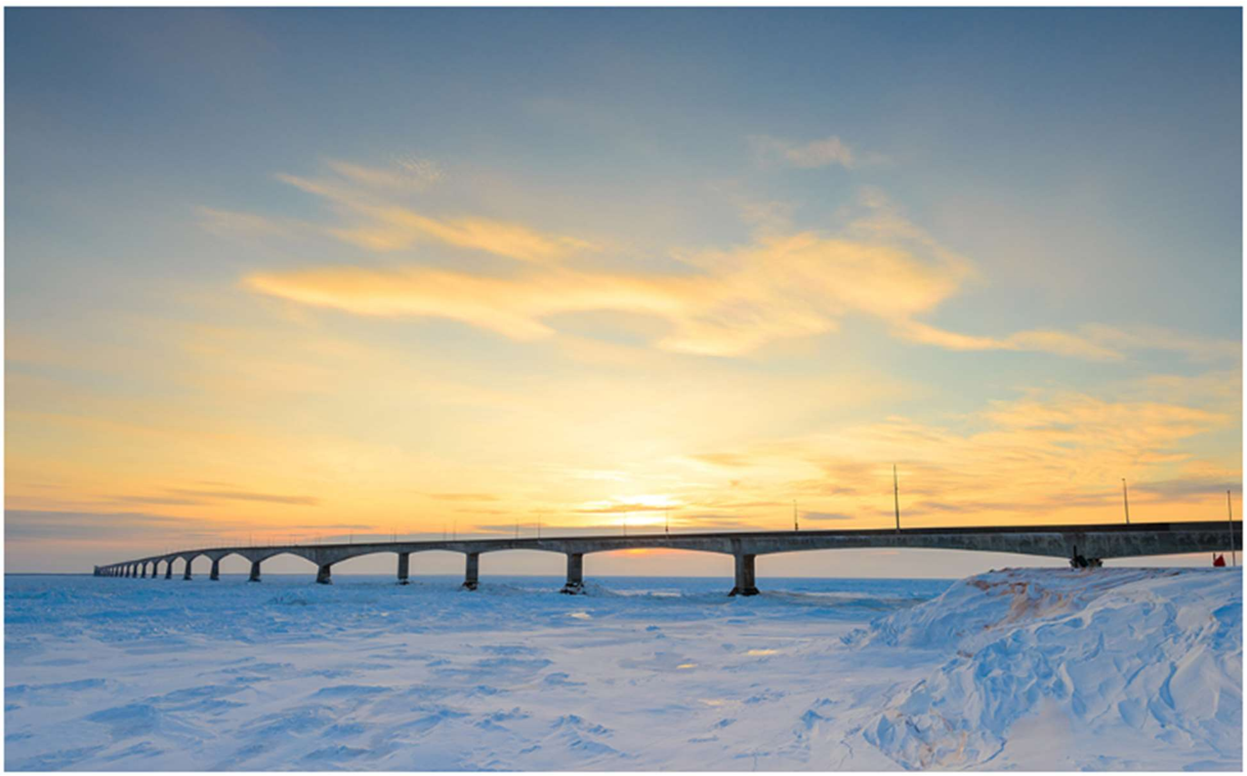


*Figure 9.7.1 - Confederation Bridge over a frozen Northumberland Strait [16]*

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.

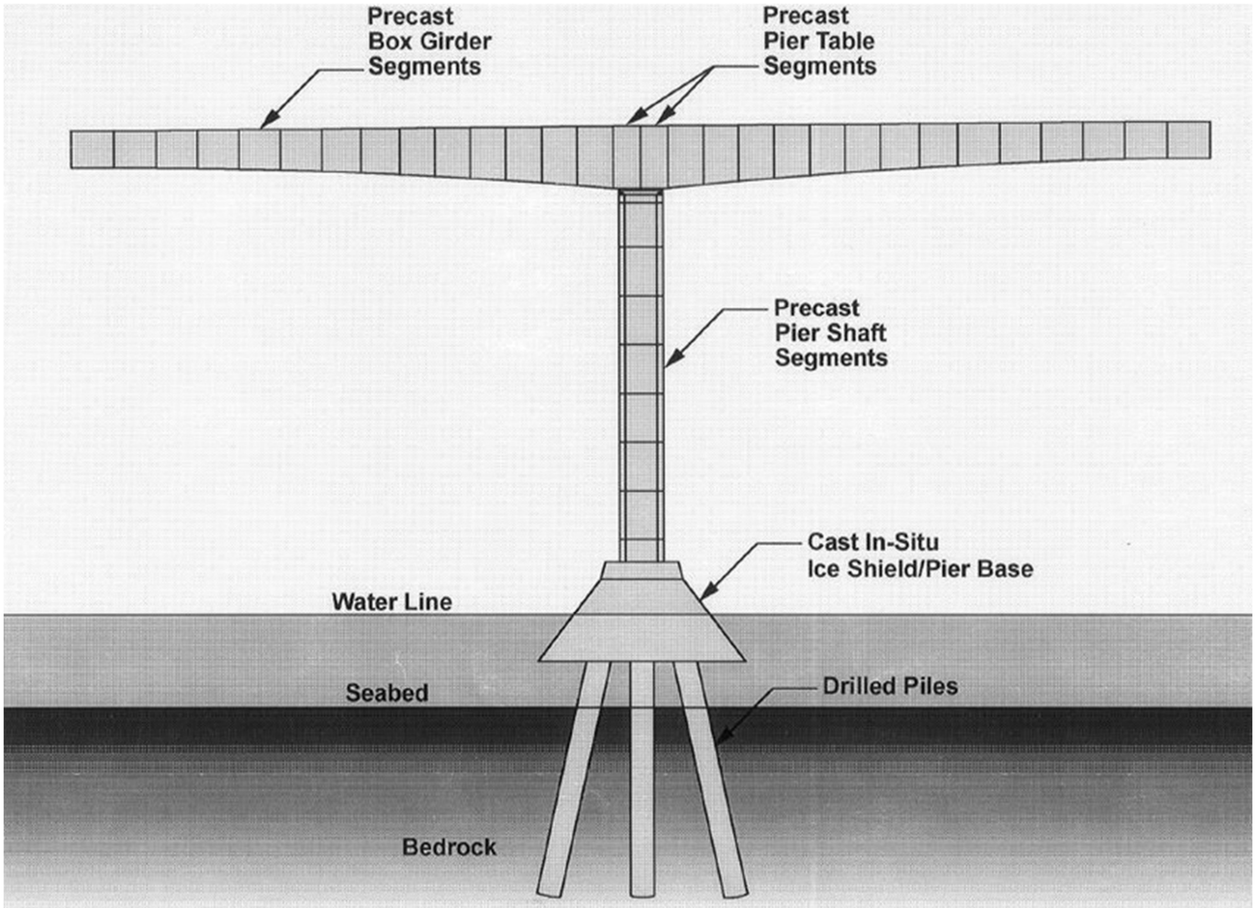



*Figure 9.7.2 - Typical Elevation View of Confederation Bridge [19]*

## 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


[1]. Concrete Network, “Aggregate in Concrete,” *The Concrete Network*, 15-Oct-2018. [Online]. Available: https://www.concretenetwork.com/aggregate/. [Accessed: 06-Apr-2020].

[2]. “Classification of Aggregates: Grain Size, Density, Shape, Origin,” *Hinton's Waste*, 13-Nov-2018. [Online]. Available: https://www.hintonswaste.co.uk/news/classification-of-aggregates/. [Accessed: 06-Apr-2020].

[3]. Concrete Network, “Cement - Information about Components of Concrete,” *The Concrete Network*, 13-Mar-2017. [Online]. Available: https://www.concretenetwork.com/cement.html. [Accessed: 06-Apr-2020].

[4]. “What are the Proper Concrete Mix Proportions?,” *BN Products*, 25-Sep-2017. [Online]. Available: http://www.bnproducts.com/blog/what-are-the-proper-concrete-mix-proportions/. [Accessed: 06-Apr-2020].

[5]. Hearns, “Improve Compressive Strength of Concrete with These Curing Techniques,” *Giatec Scientific Inc*, 31-Jan-2020. [Online]. Available: https://www.giatecscientific.com/education/curing-techniques-for-measuring-the-compressive-strength-of-concrete/. [Accessed: 06-Apr-2020]

[6]. Pepin, “Understanding the Process of Corrosion in Reinforced Concrete: Giatec,” *Giatec Scientific Inc*, 04-Nov-2019. [Online]. Available: https://www.giatecscientific.com/education/understanding-concrete-corrosion/. [Accessed: 06-Apr-2020].

[7].“Freeze Thaw Damage of Concrete - Effects & Preventive Measures,” *CivilDigital*, 16-Mar-2017. [Online]. Available: https://civildigital.com/freeze-thaw-damage-concrete-effects-preventive-measures/. [Accessed: 06-Apr-2020].

[8]. Safiuddin and B. J. Scott, “ABRASION RESISTANCE OF CONCRETE – DESIGN, CONSTRUCTION AND CASE STUDY,” *Semantic Scholar*, 01-Jan-1970. [Online]. Available: https://www.semanticscholar.org/paper/ABRASION-RESISTANCE-OF-CONCRETE-–-DESIGN,-AND-CASE-Safiuddin-Scott/4d6f22d1dad91af5928d7ad310b4270bf6f05139. [Accessed: 06-Apr-2020]

[9]. “Sulfate Attack,” *Sulfate Attack - an overview | ScienceDirect Topics*. [Online]. Available: https://www.sciencedirect.com/topics/engineering/sulfate-attack. [Accessed: 06-Apr-2020].

[10].
*Alkali-Aggregate Reaction*. [Online]. Available: https://www.cement.org/learn/concrete-technology/durability/alkali-aggregate-reaction. [Accessed: 06-Apr-2020].

[11]. C. D. Posted in Concrete Cracks, “Types of Chemical Attacks on Concrete Structures,” *The Constructor*, 04-Aug-2018. [Online]. Available: https://theconstructor.org/concrete/chemical-attacks-types-concrete-structures/7237/. [Accessed: 06-Apr-2020].

[12] “Corrosion of structural steel,” *www.steelconstruction.info*. [Online]. Available: https://www.steelconstruction.info/Corrosion_of_structural_steel. [Accessed: 06-Apr-2020].

[13]. “Corrosion of Embedded Metals,” *Corrosion of Embedded Materials*. [Online]. Available: https://www.cement.org/learn/concrete-technology/durability/corrosion-of-embedded-materials. [Accessed: 06-Apr-2020].

[14]. CSA A23.1

[15]. “Mr. Lowell Croken (Chief Electoral Officer and Chief of Protocol, Legislative Assembly, Elections P.E.I.) at the Procedure and House Affairs Committee,” *openparliament.ca*. [Online]. Available: https://openparliament.ca/committees/house-affairs/40-2/29/lowell-croken-1/only/. [Accessed: 06-Apr-2020].

[16].“Confederation Bridge,” *The Canadian Encyclopedia*. [Online]. Available: https://thecanadianencyclopedia.ca/en/article/confederation-bridge. [Accessed: 06-Apr-2020].

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

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

[19].Md Safiuddin “Typical elevation view of The Confederation Bridge pier and deck”, Research Gate. [Online]. Available: https://www.researchgate.net/figure/Typical-elevation-view-of-The-Confederation-Bridge-pier-and-deck-28_fig2_281594259

# Chapter 10 - Construction Issues

## Table of Contents

## 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.

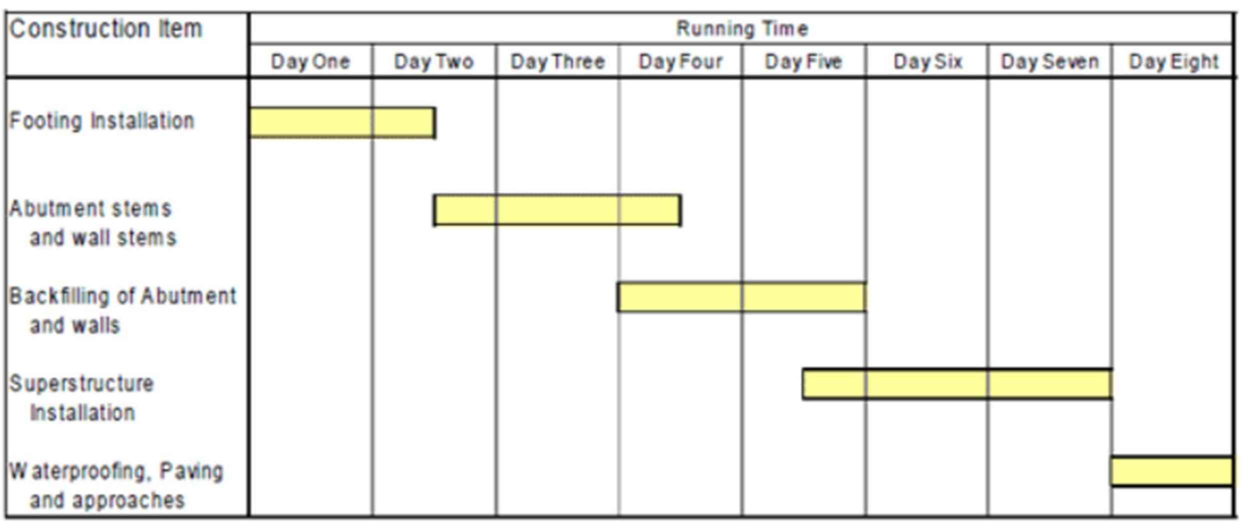



*Figure 10.4.2.1 - Construction Timeline Template [8]*

### 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].

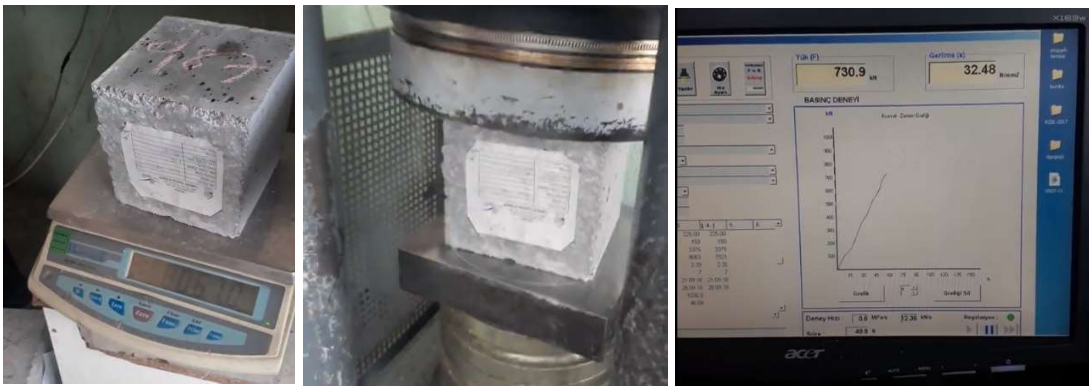



*Figure 10.5.1.3.1 – Compression test on a cubic C30/37 concrete sample*

### 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.

### 10.8 References

[1]. Aviva Insurance, The 5 most common workplace accidents on construction sites, 2018
Available Online:
https://www.aviva.ca/en/business/blog/5-most-common-workplace-accidents-on-construction-sites/

[2]. United States Department of Labor, Worker Safety Series: Construction, 2005
Available Online:
https://www.osha.gov/Publications/OSHA3252/3252.html

[3]. Occupational Health and Safety Act, R.S.O. 1990, c. O.1
Available Online:
https://www.ontario.ca/laws/statute/90o01

[4]. Canadian Centre for Occupational Health and Safety, WHMIS FAQ and general info
Available Online:
https://www.ccohs.ca/oshanswers/chemicals/whmis_ghs/general.html

[5]. Minnesota Local Road Research Board, personal protection equipment poster & website
Available Online:
https://lrrb.org/personal-protection-equipment-poster-website/

[6]. PNGKey, construction safety signs – Signs in a workplace
Available Online:
https://www.pngkey.com/detail/u2t4y3t4t4i1e6u2_construction-safety-signs-signs-in-a-workplace/

[7].Stonemark Construction Management, What is Procurement in Construction Management?
Available Online:
https://stonemarkcm.com/blog/procurement-in-construction-management

[8].US Department of Transportation, Federal Highway Administration, Connection Details for PBES
Available Online:
https://www.fhwa.dot.gov/bridge/prefab/if09010/appd.cfm

[9].DesigningBuildingsWiki, Budget for building design and construction projects, February 2020
Available Online:

https://www.designingbuildings.co.uk/wiki/Budget_for_building_design_and_construction_projects

[10]. Zeina Hazem, PlanRadar, The modern way of ensuring quality control in construction,2018
Available Online:
https://www.planradar.com/quality-control-in-construction/

[11].PlanGrid, Building 101: What's the Difference Between Quality Assurance and Quality Control in Construction?, January 31, 2018
Available Online:
https://blog.plangrid.com/2018/01/building-101-whats-the-difference-between-quality-assurance-and-quality-control-in-construction/

[12]. Suryakanta, CivilBlog, QUALITY CONTROL IN CONCRETE CONSTRUCTION WORK, October 4, 2016
Available Online:
https://civilblog.org/2016/10/04/quality-control-concrete-construction-work/

[13]. BNBarchitects + Builders, Importance of Slump Test in the Construction, APRIL 29, 2018
Available Online:
http://bnbarchitects.blogspot.com/2018/04/importance-of-slump-test-in-construction.html

[14]. Noel Mades, 4 Steel Bar Reinforcement Tests That You Should Know, January 2, 2018
Available Online:
https://www.qualityengineersguide.com/4-steel-bar-reinforcement-tests-that-you-should-know

[15]. The Constructor, Types of Formwork (Shuttering) for Concrete Construction and its Properties
Available Online:
https://theconstructor.org/building/formwork-shuttering/types-of-formwork-shuttering/3767/

# Chapter 11 – Plant Life Management

## Table of Contents

## 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.

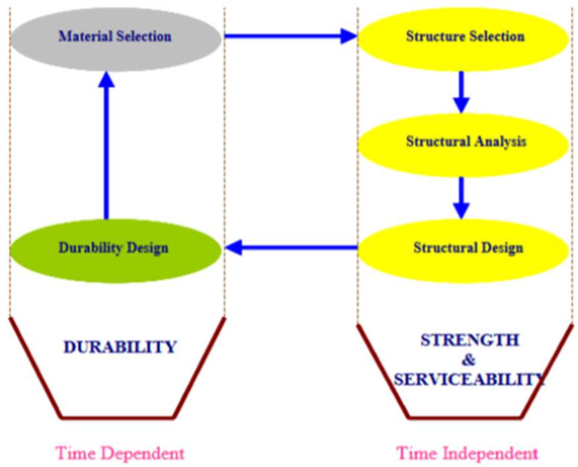



*Figure 11.3.1 - Components of Design Methodology [2]*

### 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.

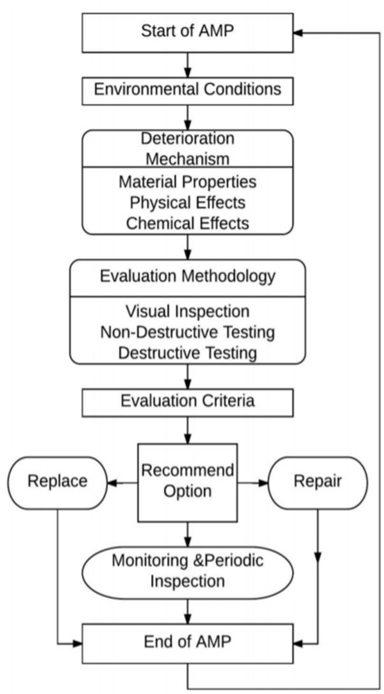



*Figure 11.5.1 - Ageing Management Program Flow Chart [4]*

### 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.

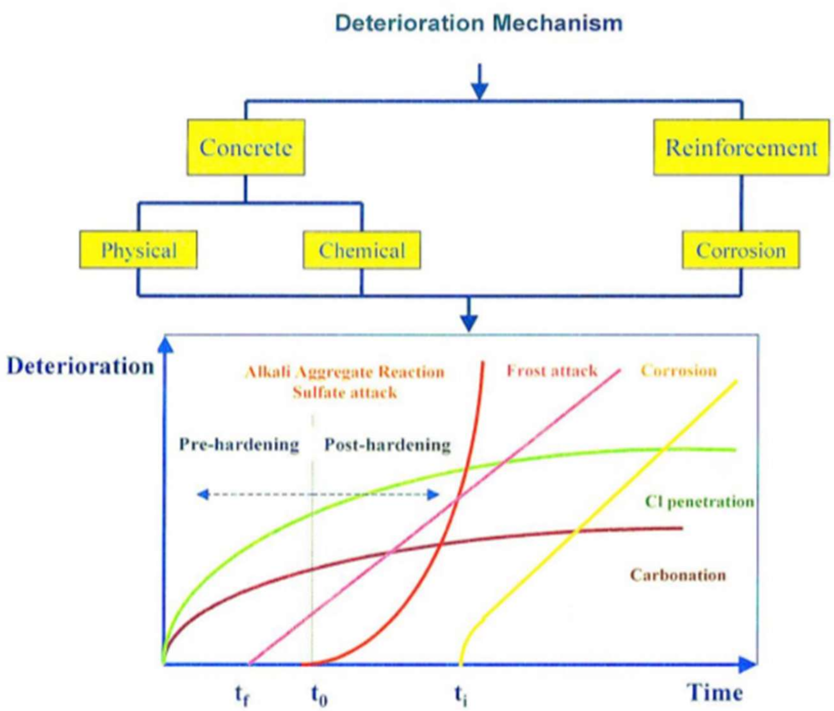



*Figure 11.5.2.1 - Different Deterioration Mechanisms [4]*

### 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.

## 11.7 References

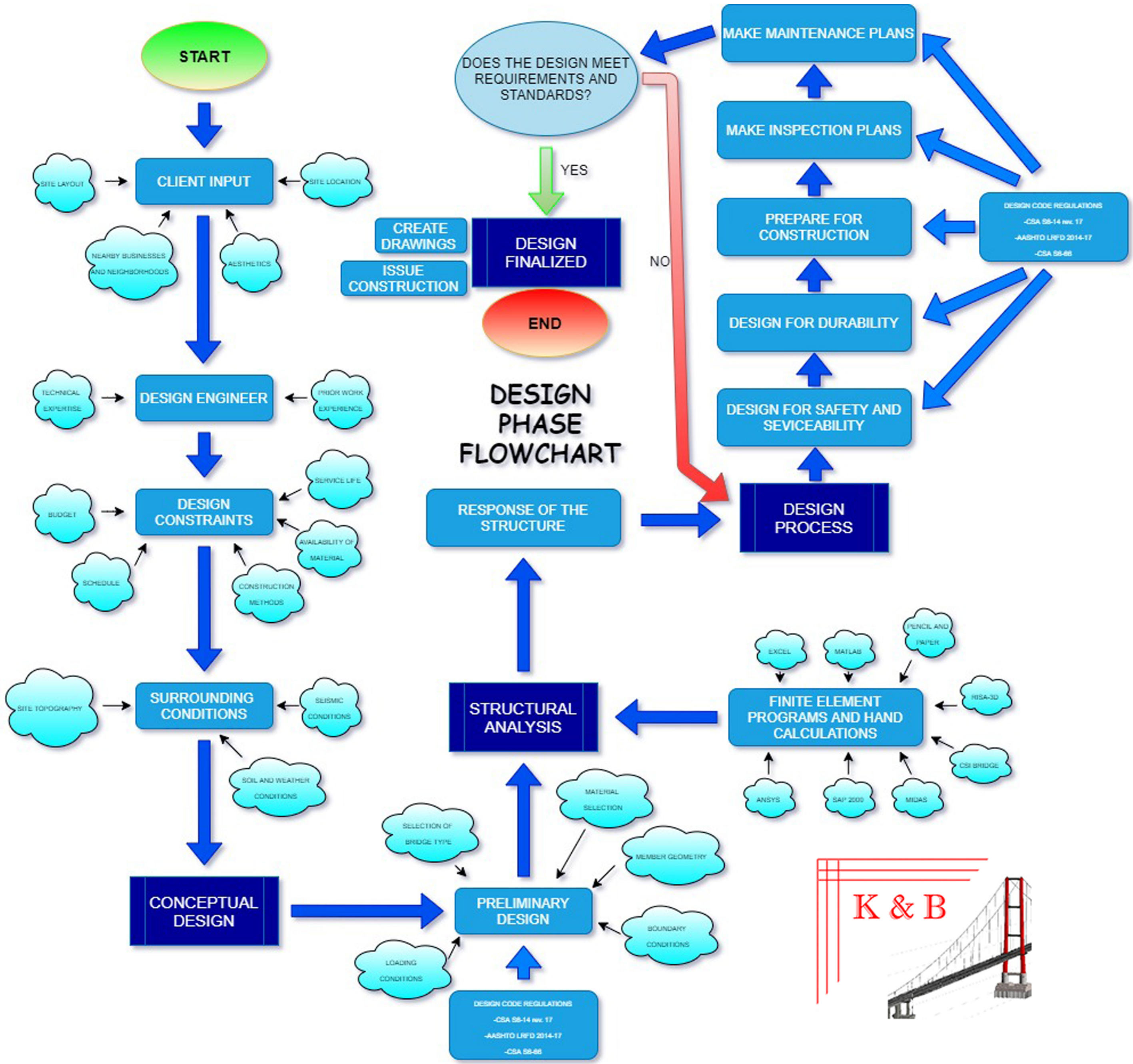
[1]. “Plant life management for safe long-term operation,” *IAEA*, 31-Jan-2017. [Online]. Available: https://www.iaea.org/topics/nuclear-power-plant-life-cycle/plant-life-management. [Accessed: 06-Apr-2020]

[2]. H.H. Abrishami (2014). A New Approach of Plant Life Management of Containment Structure.

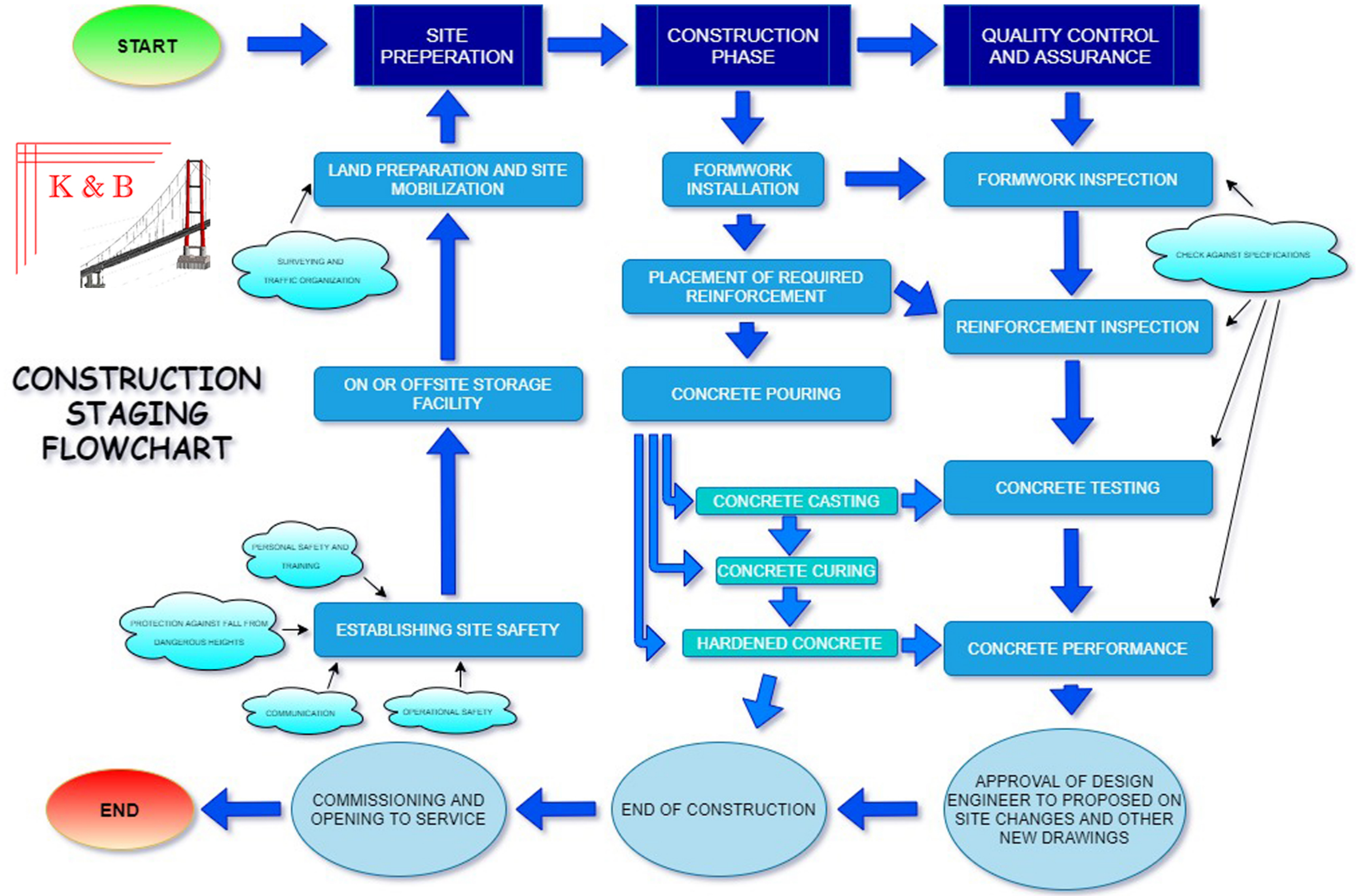
[3]. “Rebar Rust Stock Photos and Images,” *Alamy*. [Online]. Available: https://www.alamy.com/stock-photo/rebar-rust.html: [Accessed: 06-Apr-2020].

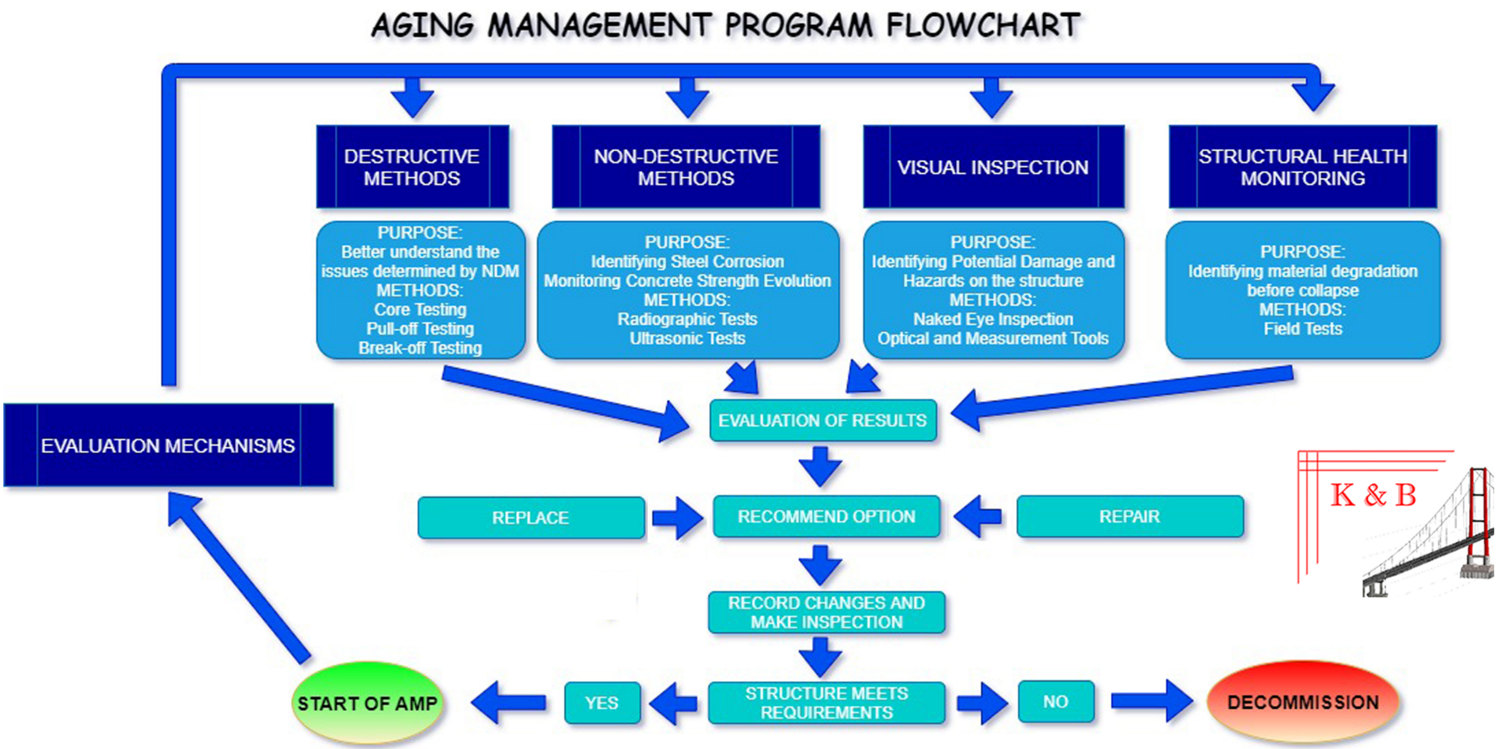
[4]. H. Abrishami, “Plant Life Management (PLiM),” Email: March 28, 2020.

[5]. “What is Non-Destructive Testing (NDT)? Methods and Definition,” *TWI*. [Online]. Available: https://www.twi-global.com/technical-knowledge/faqs/what-is-non-destructive-testing. [Accessed: 06-Apr-2020]



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 PHASE FLOWCHART
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


CONSTRUCTION STAGING FLOWCHART
K & B
START
SITE PREPERATION
CONSTRUCTION PHASE
QUALITY CONTROL AND ASSURANCE
LAND PREPARATION AND SITE MOBILIZATION
SURVEYING AND TRAFFIC ORGANIZATION
ON OR OFFSITE STORAGE FACILITY
ESTABLISHING SITE SAFETY
PERSONAL SAFETY AND TRAINING
PROTECTION AGAINST FALL FROM DANGEROUS HEIGHTS
COMMUNICATION
OPERATIONAL SAFETY
FORMWORK INSTALLATION
PLACEMENT OF REQUIRED REINFORCEMENT
CONCRETE POURING
CONCRETE CASTING
CONCRETE CURING
HARDENED CONCRETE
FORMWORK INSPECTION
CHECK AGAINST SPECIFICATIONS
REINFORCEMENT INSPECTION
CONCRETE TESTING
CONCRETE PERFORMANCE
APPROVAL OF DESIGN ENGINEER TO PROPOSED ON SITE CHANGES AND OTHER NEW DRAWINGS
END OF CONSTRUCTION
COMMISSIONING AND OPENING TO SERVICE
END


AGING MANAGEMENT PROGRAM FLOWCHART
DESTRUCTIVE METHODS
NON-DESTRUCTIVE METHODS
VISUAL INSPECTION
STRUCTURAL HEALTH MONITORING
PURPOSE: Better understand the issues determined by NDM METHODS: Core Testing Pull-off Testing Break-off Testing
PURPOSE: Identifying Steel Corrosion Monitoring Concrete Strength Evolution METHODS: Radiographic Tests Ultrasonic Tests
PURPOSE: Identifying Potential Damage and Hazards on the structure METHODS: Naked Eye Inspection Optical and Measurement Tools
PURPOSE: Identifying material degradation before collapse METHODS: Field Tests
EVALUATION MECHANISMS
EVALUATION OF RESULTS
REPLACE
RECOMMEND OPTION
REPAIR
RECORD CHANGES AND MAKE INSPECTION
START OF AMP
YES
STRUCTURE MEETS REQUIREMENTS
NO
DECOMMISSION
K & B