University of Stavanger

FACULTY OF SCIENCE AND TECHNOLOGY

MASTER'S THESIS

Study programme / specialisation: Engineering Structures and Materials Specialization Civil Engineering Structures	Spring semester, 2023 Open access
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Thesis title:

Influence of Span-to-Depth Ratio in Designing Concrete Bridges

Credits (ECTS): 30

Keywords: Parametric Design Reinforced Concrete Post-tensioning Span-to-depth ratio	Pages: 166 + appendix: 42 Stavanger, 14.06.2023

Preface

This thesis is written as a finishing part of a two-year master's degree in constructions and materials at the University of Stavanger.

First and foremost, I would like to extend my sincere appreciation to my supervisor at Sweco, Engineer Erik Tveiten. His expertise, valuable insights, and continuous support have been instrumental in shaping the direction of this research. I would also like to thank Engineer Jo Gatue Fornes and Sweco Norge AS | Stavanger for providing me with the opportunity to be a part of their working environment. The fruitful discussions and my own desk space have greatly contributed to the development and completion of this thesis.

I would also like to express my deepest gratitude to my supervisor at the University of Stavanger, Associate Professor in Structural Engineering Samindi Samarakoon. Her extensive knowledge, guidance, and constructive feedback have been invaluable throughout the research process.

Furthermore, I would like to extend my thanks to everyone who has been part of my academic journey. To my colleagues and friends who have provided encouragement, assistance, and insightful discussions, I am truly grateful. Your support has been a constant source of motivation.

Finally, I would like to acknowledge the unwavering support of my family. Their encouragement, love, and understanding have been the pillars of strength throughout this endeavor. I am indebted to them for their patience and belief in my abilities.

Completing this master's thesis has been a challenging yet rewarding experience, and I am truly grateful to all those who have contributed to its successful completion.

Abstract

The influence of the span-to-depth ratio in designing concrete bridges is a critical aspect of bridge engineering. This study focuses on investigating the span-to-depth ratios of reinforced concrete bridges and post-tensioned concrete bridges. The main objective of the research is to establish a new basis for determining the height of the bridge deck based on selected span lengths for three-span simply supported reinforced and post-tensioned plate bridges.

Nine bridge models with varying mid-span lengths ranging from 8 to 40m are analyzed to examine the relationship between the mid-span length and plate thickness. These models cover a range of mid-span to plate thickness ratios from 17.77 to 34.28. The side spans is set to 0.3 times the total bridge length (L), and the mid-span is set to 0.4 times the total bridge length (L). The transition point from reinforced to post-tensioned concrete is identified, and the optimal span-to-depth ratio for reinforced concrete bridges is determined. The bridge models are analyzed using the Sofistik software and its Teddy programming language (CADINP), considering design requirements for ultimate limit state (ULS) and serviceability limit state (SLS).

The research adopts a parameterized approach, utilizing code in Sofistik to automatically generate bridge models based on input parameters, such as bridge length and depth. This parameterization allows for efficient and automated generation of loads and actions, dynamically adjusted whenever new structural parameters are inputted. As a result, comprehensive analysis of various load and action scenarios can be performed, specifically tailored to the specific bridge length. This approach enables a thorough exploration of the bridge's behavior under different loading conditions, leading to a more informed and optimized design.

The analysis reveals that the ULS governs the design for bridge models 8, 12, and 16, while the SLS governs the design for bridge model 20. For post-tensioned bridge models, the SLS governs the design for all of them. The findings demonstrate that the required cross-section height of a bridge varies with the span length. Specifically, the transition from reinforced to post-tensioned concrete occurs at a bridge length of 60m or when the middle span length is 24m. Additionally, the deflection control meets the requirements specified by EC2 and N400, ensuring the structural functionality of the bridges.

Contents

List of Figures

List of Tables

List of Notations

EC0 Eurocode : Basis of structural design

EC1-2 Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges

EC1-4 Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions

EC1-5 Eurocode 1: Actions on structures - Part 1-5: General actions - Thermal actions

EC1-7 Eurocode 1: Action on structures, Part 1-7: General actions, Accidental actions

EC2 Eurocode 2: Design of concrete structures - Concrete bridges - Design and detailing rules

EC2-1 Eurocode 2 : Design of concrete structures — Part 1-1: General rules and rules for buildings

EC8-1 Eurocode 8, Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings

EC8-2 Eurocode 8: Design of structures for earthquake resistance Part 2: Bridges

NRA Norwegian Road Administration

PC Pre-stressed concrete

RC Reinforced concrete

SSD Sofistik Structural Desktop

SVV Statens Vegvesen

Chapter 1

Introduction

Concrete bridges are an integral part of the infrastructure that provides a secure and efficient mode of transportation over roads, rivers, valleys, and other obstacles. To withstand the forces of gravity, wind, and traffic, concrete bridges must be designed to function reliably for several years with minimum maintenance. The design process of concrete bridges necessitates a thorough comprehension of structural principles, materials science, and construction techniques.

The span-to-depth ratio is a critical consideration in the design of concrete bridges. It defines the relationship between the span length and the depth of the superstructure and plays a crucial role in determining the structural behavior and construction cost of the bridge. The span-to-depth ratio is typically established during the early design phase, allowing for a preliminary analysis to evaluate the feasibility, cost-effectiveness, and aesthetic aspects in comparison to alternative design concepts. The selection of this ratio is often based on experience and conventional values derived from successful projects. Utilizing a high span-to-depth ratio reduces the required volume of concrete, thereby decreasing construction costs. However, it increases the need for prestressing and simplifies the construction process due to the lighter superstructure. Additionally, the span-to-depth ratio significantly impacts the visual appearance of the bridge.

In this review, the objective is to examine the importance of the span-to-depth ratio during the conceptual design stage of reinforced and prestressed concrete bridges. The focus of the study will be on a three-span bridge that has a side span to main span ratio of 0.75 (side span is 0.75 of center span), so that $L = 0.3L + 0.4L + 0.3L$ where the height of the bridge will be fixed while the width will be 12m. Various bridge lengths ranging from 20m to approximately 100m will be explored to determine the ideal height of the cross-section. This determination will be based on criteria related to the ultimate limit state (ULS) and serviceability limit state (SLS). Furthermore, this thesis will explore the transition from reinforced bridges to pre-stressed bridges for different span lengths. Typically, reinforced bridges are more suitable for shorter spans, while pre-stressed bridges are preferable for longer spans. Several factors influence this transition, including the amount of prestress required, the size and type of tendons used, and the construction method. Understanding these factors is critical to ensure that the most appropriate type of bridge is chosen for a specific span length.

Chapter 2

Literature Review

"We build too many walls and not enough bridges" is a well-known quote attributed to Isaac Newton, the English physicist, mathematician, and philosopher. It succinctly captures a long-standing pattern in human communication, where a preference for closure and isolation prevails over openness and connection. However, it is undeniable that bridges constructed by humans since ancient times have played a pivotal role in challenging this tendency and enriching societies across generations.

Bridges and aqueducts have served as vital links between cities and nations, facilitating seamless travel across vast stretches of the Earth for countless years. While passing over one of these structures today may seem commonplace to some, it is important to recognize that throughout history, the construction of bridges was a monumental achievement. It conquered geographical obstacles such as reefs, valleys, and bays, making previously arduous journeys attainable. Civilizations have always taken pride in their bridges, as they played a role in winning wars, inspiring writers and poets alike.

Human beings quickly recognized the utility of natural resources, particularly stones and sturdy tree trunks, for traversing small streams. Over time, these humble beginnings led to the development of rudimentary crossings, which gradually evolved into wooden-beamed archways and bridges. Such structures remained prevalent until the sixth century BC.

The Roman civilization played a significant role in the advancement of bridge construction. Engineers of this ancient civilization incorporated artistic elements by employing arches and monolithic stones in their designs. These bridges aided the Romans in establishing extensive networks of roads, which bolstered their commercial influence and military campaigns.

From the second century BC, the Romans utilized a type of mortar consisting of lime, sand, and volcanic rock powder. This mortar greatly enhanced the durability of their bridges, some of which have endured to this day. One of the most renowned examples is the Pont du Gard in France, constructed in the middle of the first century AD. This bridge served as a remarkable aqueduct, transporting water from the springs of the city of Uzès to supply the city of Nîmes with water.

After the Roman era, there was a significant hiatus in the development of bridge construction. The world entered a phase known as the "Dark Ages" where all construction activities came to a halt for approximately 500 years. It was during the Middle Ages, with the strengthening influence of the Church in Europe, that efforts to spread Christianity and extend bridges to remote areas resumed. However, these endeavors predominantly relied on Roman construction techniques without notable advancements, particularly in technical terms.

This state of affairs persisted until the Renaissance period in the late 17th and early 18th centuries when there was an increased focus on the aesthetics and geometry of bridges. Nonetheless, the most significant development emerged with the renowned French engineer of arches, Jean-Rodolphe Perronet, in the mid-18th century. Perronet is regarded as the father of modern engineering. He dedicated his research to studying the arches of stone bridges and successfully reduced the thickness of bridge stones by more than half. His innovative construction technique involved distributing the pressure on a single arch without impacting the other arches. This breakthrough allowed for the construction of longer-spanning bridges.

The scientific and technological advancements during the Second Industrial Revolution greatly enhanced the efficiency and lifespan of bridges. A significant milestone in bridge construction came with the discovery of artificial cement by the French engineer Louis Vicat in 1840. Although Portland Cement had been introduced since 1824, it was the establishment of French cement factories in 1850 that marked a crucial turning point. This development enabled France to make significant progress in concrete production, which was first utilized in 1853.

The incorporation of iron into cement revolutionized construction approaches as the 20th century approached. The use of reinforced concrete, known for its exceptional durability and the ability to mold it into unique geometric shapes, began to replace traditional stone structures. François Hennebique, a French engineer, demonstrated this innovation by constructing a bridge over the Châteleu River between 1896 and 1907. The bridge spanned over 100m and remained the largest reinforced concrete structure until 1911.

Concrete bridges refer to bridge structures where the primary support system is constructed using concrete. Concrete is commonly reinforced with either tension reinforcement or compression reinforcement. In reinforced concrete elements, the cross-sectional area of the reinforcing bars must meet the minimum requirements. Prestressed concrete, first used in the 1950s involves applying compressive stress to the concrete, designed to counteract the effects of external loads to a certain extent. This feature allows for even higher span-to-depth ratios and results in bridges that are more durable, stronger, and have longer lifespans.

2.1 Reinforced Concrete

Reinforced concrete (RC) bridges have emerged as a popular and widely chosen option for contemporary bridge projects due to their exceptional durability, strength, and versatility. These bridges are meticulously crafted using a combination of concrete and steel reinforcement bars (rebar), resulting in a robust structure capable of withstanding substantial loads and adverse weather conditions.

Although the utilization of reinforced concrete in bridge construction can be traced back to the late 19th century, it wasn't until the early 20th century that it gained widespread recognition as a preferred material (Nick Gromicko and Shepard 2023). One of the earliest instances of a reinforced concrete bridge is Joseph Monier's innovative RC bridge, which was built in 1875, employing a patented system of iron reinforcement bars.

Reinforced concrete bridges offer several significant advantages, primarily due to their ability to withstand heavy loads. While concrete is highly effective in handling compression forces, it has limited capacity to endure tensile forces. However, by implementing reinforcement, concrete gains the necessary strength to withstand tensile stresses, enabling it to accommodate moments. Reinforced concrete bridges leverage steel reinforcement bars to provide additional strength and support to the concrete structure. This reinforcement empowers the bridge to bear the weight of various vehicles, trains, and even heavy construction equipment.

Furthermore, reinforced concrete bridges can be engineered to withstand natural disasters such as earthquakes, floods, and other extreme events. This resilience is achieved through careful design, considering factors like seismic activity, water flow, and potential impacts. By integrating reinforcing elements strategically, these bridges can effectively absorb and distribute the forces generated during such events.

Another benefit of reinforced concrete bridges is its adaptability. They can be used for a wide range of bridge types, including arches, beams, and trusses. They can also be used for different types of spans and cross sections

On the other hand, reinforced concrete bridges can be costly to construct. The materials and labor required to build a reinforced concrete bridge can be expensive, and the process can take longer than other types of bridge construction. Additionally, reinforced concrete bridges can be difficult to repair or replace if they become damaged.

There are several different types of reinforced concrete bridges that can be used depending on the specific needs of the project:

1. Cast-in-Place Bridges: These bridges are constructed by pouring concrete into on site forms. The steel reinforcement bars are are positioned within the forms before the concrete is poured, and once cured, the forms are removed. Cast-in-place bridges are particularly suitable for shorter spans designed for road, pedestrian, or bicycle traffic. Victor (2022) classifies, cast-in-place concrete bridges into four basic types based on construction methodology:

- Solid or voided slabs: Solid slabs typically span 5-20m, while voided slabs span 20-40m. The span-to-depth ratios for highway bridges usually range from 18 to 24, depending on whether the spans are simple or continuous.
- Twin Rib Bridges, Span by Span Box Bridges, and Balanced Cantilever Bridges: These types fall under pre-stressed concrete bridges and will be discussed in the next section.

2. Box Girder Bridges: Constructed using precast concrete segments assembled on site, box girder bridges join the segments using steel connectors while incorporating steel reinforcement bars. They are commonly used for medium span bridges catering to vehicular traffic.

3. Pre-Cast Segmental Bridges: Similar to box girder bridges, pre-cast segmental bridges employ precast segments manufactured in a factory and transported to the site for assembly. The segments are connected using steel connectors and contain steel reinforcement bars. These bridges are often utilized for long-span structures accommodating trains or heavy vehicle traffic.

4. Arch Bridges: Reinforced concrete arches serve as the primary support for the roadway in arch bridges. These arches rely on piers or abutments for stability and are frequently employed

for spans spanning several hundred meters. Arch bridges find suitability in crossing large rivers or waterways.

2.2 Prestressed Concrete

The prestressing system has been developed over the years to produce stronger and more durable concrete structures. Prestressed concrete is a type of concrete that is reinforced with steel tendons by introducing internal stresses before the material hardens. This type of concrete is commonly used in the construction of bridges because it can withstand heavy loads and spans.
There are two main types of prestressing systems: pretensioning and post-tensioning. In the pretensioning system, the steel cables are tensioned before the concrete is cast. The tensioned cables are then anchored to the concrete forms to keep them in place while the concrete sets. Once the concrete has hardened, the cables are released, which creates compression forces that strengthen the concrete. Figure 2.1 illustrates how a concrete member is pretensioned. The post-tensioning system is used after the concrete has been cast in the formwork around the ducts. The steel tendons are placed in the ducts and left unstressed until the concrete has hardened and reached a significant strength. Once the concrete has achieved this strength, the tendons are tensioned using hydraulic jacks, which creates compression forces that strengthen the concrete (Gilbert et al. 2017). Figure 2.2 shows the post-tensioning procedure.

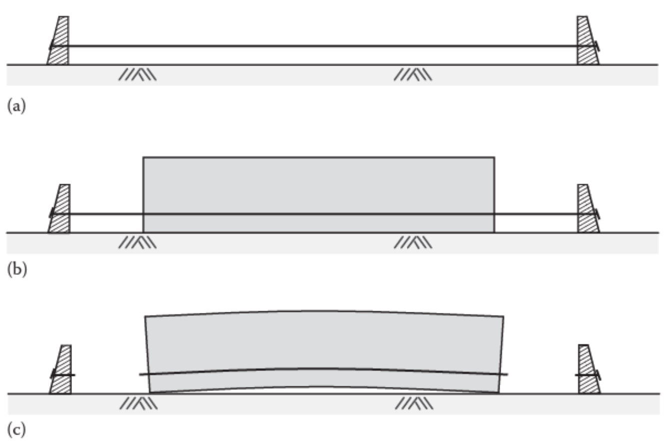


Figure 2.1: Pretensioning procedure. (a) Tendons stressed between abutments. (b) Concrete cast and cured. (c) Tendons released and prestress transferred. Figure is taken from (Gilbert et al. 2017)

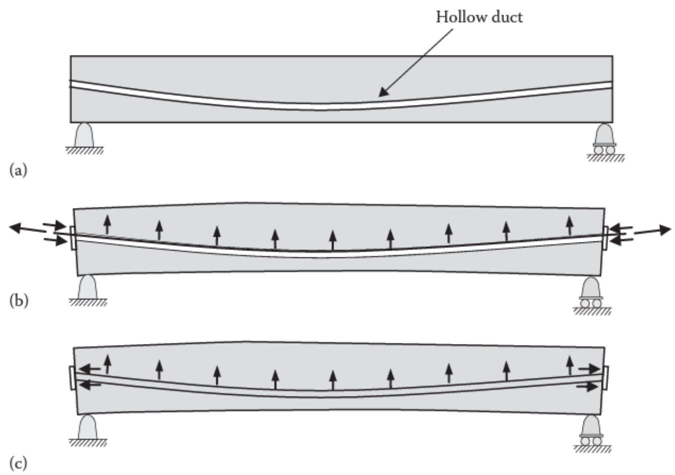


Figure 2.2: Post-tensioning procedure. (a) Concrete cast and cured. (b) Tendons stressed and prestress transferred. (c) Tendons anchored and subsequently grouted. Figure is taken from (Gilbert et al. 2017)

Pre-stressed concrete bridges offer a wide range of options to meet different project needs. There are various types, such as cast-in-place, box girder, and pre-cast segmental bridges. Structural engineers carefully consider factors like span length, site conditions, design specifications, and budget when choosing the right bridge type for a project. By evaluating these elements, engineers can make informed decisions and create strong and efficient pre-stressed concrete bridges that fit the project's requirements perfectly.

As previously mentioned, cast-in-place bridges are constructed on site using concrete that is poured into forms. The concrete is then pre-stressed using steel cables or rods that are placed in a specific pattern and tensioned to create the desired amount of stress. According to Victor (2022), this type of bridge is often used for short to medium span bridges, as the construction process is relatively quick and simple. Further, Victor (2022) mentioned in his article 3 types of pre-stressed cast-in-place bridges, as following:

- Twin Rib Bridges are considered to have a better efficient section compared with slab bridges and are employed for spans ranging from 20m -50m.
- Span by Span Box Bridges have a more efficient section compared to the slab and rib bridges.
- Balanced Cantilever Bridges have typical spans range from 40-300m and by definition,

are always continuous. span-to-depth ratios for highway bridges are typically 18-20 for constant depth schemes.

One example of a cast-in-place bridge is the San Francisco-Oakland Bay Bridge in California, USA. This bridge is a composite steel-concrete structure spanning across the San Francisco Bay, connecting the cities of San Francisco and Oakland. It was completed in 1936 and is approximately 8.4 miles long. The Bay Bridge includes the longest self-anchored suspension span in the world (Commission 2022)

Box girder bridges can be constructed using either cast-in-place concrete or precast concrete boxes that are assembled on-site. These boxes are typically made from pre-stressed concrete and are joined using a technique called post-tensioning, which involves the use of high-strength steel cables or rods. Box girder bridges are often used for medium to long span bridges and are known for their strength and stability. For highway bridges, typical spans range from 30-80m and span-to-depth ratios within the range of 16-22 is used as a first round of sizing (Victor 2022).

Pre-cast segmental bridges, also known as segmental bridges, are constructed using pre-cast concrete segments that are assembled on site. The segments are typically connected using post-tensioning, and the entire bridge is then pre-stressed to create the desired amount of stress. Pre-cast segmental bridges are often used for long span bridges and are known for their ability to span large distances without the need for intermediate supports (Rosignoli 2016). One example is the Governor Mario M. Cuomo Bridge in New York, USA. The bridge is a twin-span cable-stayed bridge that spans across the Hudson River, connecting the cities of Westchester and Rockland counties. The bridge deck is constructed of pre-cast, post-tensioned concrete segments that were prefabricated off-site and assembled on-site. The bridge was completed in 2017, and its main span is the longest cable-stayed bridge in the United States (Design 2019).

Additionally, pre-cast segmental bridges tend to be more economically feasible compared to the other types of pre-stressed concrete bridges. This is due to the fact that the pre-casting process allows for much of the work to be done off-site, which reduces the need for on-site labor and speeds up the construction process, solves problems of environmental restrictions, traffic interference, inaccessible terrain and many others (Barker 1981). Pre-cast segmental bridges also tend to have a longer lifespan compared to other types of bridges, as the pre-stressed concrete is resistant to cracking and deformation.

Cast-in-place bridges can also be economically feasible, especially for shorter span bridges where the construction process is relatively quick and simple. However, the need to form and pour the concrete on site can lead to higher labor costs and a longer construction period.

Even so, when considering the use of precast segments in concrete bridge construction, it is critical to compare the costs of form travelers for cast-in-place construction with the costs associated with the precasting yard, storage, transportation, and installation of the segments. This comparison is essential to determine the most cost-effective solution.

2.2.1 Bonded and Unbonded Post-tensioned Concrete

According to Gilbert et al. (2017), the use of bonded post-tensioning is preferred over unbonded post-tensioning in many countries due to the disadvantages of unbonded construction. Durability is a crucial consideration in all forms of construction, and the provision of active corrosion protection is of significant importance. Grouting the tendons provides an alkaline environment around the steel, which offers active corrosion protection. Bonded tendons are better than unbonded tendons for controlling cracking and for resisting progressive collapse in case of local failure. With appropriate design consideration, the prestressing forces in the unbonded tendons can theoretically be adjusted throughout the life of the structure. For these reasons, the main focus in this thesis is on bonded post-tensioning.

2.3 Reinforced Concrete versus Pre-stressed Concrete

Reinforced concrete (RC) and prestressed concrete (PC) are two types of concrete that are used in bridge construction. Both types use a combination of concrete and steel to create strong and durable structures. However, the way in which these materials are utilized differs between the two types of concrete.

RC is designed to address the weakness of concrete in tension. Concrete is strong in compression but weak in tension, and its tensile strength is only about 10% of its compressive strength. To overcome this, RC uses steel reinforcement bars to carry the internal tensile forces. The rebar is placed within the concrete and is anchored in place, and the concrete is poured over it and allowed to cure. The rebar provides extra strength and support to the concrete, which allows the structure to hold up under heavy loads. However, RC can crack under service loads due to varying tensile stresses in the steel caused by the bending moment. This can lead to several issues, including corrosion of the reinforcement, stiffness loss, excessive deflection, and reduced ability to resist shear stress. Additionally, a significant amount of steel is necessary to attain the same strength as a pre-stressed member, resulting in an increase in material costs for a traditionally reinforced member.

PC is a type of concrete that is designed to introduce artificial compressive forces into a structure before loading. This is achieved through the technique of pre-stressing, which involves introducing compressive forces into the concrete structure before it is loaded. The compressive forces help to reduce the tensile stresses within the structure and prevent cracking.

The steel is used primarily for inducing a prestress in concrete, and the stress in the steel is not dependent on the strain in the concrete. Unlike reinforced concrete, the stress in steel does not require to be restricted in order to control cracking of concrete. By using pre-stressed concrete, the risk of cracking is reduced, and the service life of the structure is increased. One of the methods used to introduce these compressive forces is Pre-tensioning, where wires or strands, called tendons, are stretched to a pre-determined amount between anchoring posts before the concrete is poured. The tendons are then bonded to the concrete throughout their length as the concrete sets. Once the concrete has hardened, the tendons are released from the anchoring posts and tend to regain their original length by shortening, transferring compressive stress to the concrete through bond. The tendons are typically stressed using hydraulic jacks. This method is particularly useful for structures that will experience heavy loads or will be exposed to harsh environmental conditions, as it increases the structural integrity and durability of the structure.

2.4 Advantages and Disadvantages of Different Types of Pre-stressed Concrete Bridges

The most notable advantage of pre-stressed concrete is that it eliminates or minimize the formation of cracks in the concrete. The use of pre-stressed concrete can lead to smaller sections in the structure, as the pre-stressing bending moment is counterbalanced by the dead load bending moments. As stated by Ray (2023), pre-stressing allows for longer span lengths, thinner slabs that decrease the weight of structure, and fewer joints required, resulting in reduced maintenance costs. However, pre-stressed concrete also has certain disadvantages. The process of pre-stressing can be complex, time-consuming and requires specialized equipment and materials. Additionally, it needs for better quality control, which can increase the cost of the project and it necessitates the use of high-strength concrete and steel wires with high tensile strength.

As been stated previously, there are different types of pre-stressed concrete bridges, each with their own advantages and disadvantages. One type of pre-stressed concrete bridge is the cast-in-place bridge. These bridges are constructed by forming and pouring the concrete on site. This method has the advantage of being relatively quick and simple, and can be customized to fit the specific site and design requirements. Additionally, they can be constructed using standard construction equipment and materials. However, cast-in-place bridges may be more susceptible to construction errors or delays due to the need to form and pour the concrete on site.

Another type of pre-stressed concrete bridge is the box girder bridge. These bridges have a strong and stable box-shaped cross section, and can span longer distances compared to cast-in-place bridges. They can also be constructed using standard construction equipment and materials. However, box girder bridges are more complex and time-consuming to construct compared to cast-in-place bridges, and may require specialized equipment for handling and installing the pre-cast boxes.

A third type of pre-stressed concrete bridge is the pre-cast segmental bridge. These bridges are constructed by precasting segments off-site and assembling them on site. This method has the advantage of being relatively quick, as much of the work can be done off-site, and requires less on-site labor compared to other types of bridges. They can also span longer distances compared to other types of pre-stressed concrete bridges. However, pre-cast segmental bridges may be more expensive due to the need for specialized equipment and the pre-casting process, and design options are limited.

2.4.1 Other Types of Bridge Cross Sections:

There are several other different types of cross sections that are commonly used in the construction of pre-stressed concrete bridges. Some of the most common types include:

- T-beam: This type of cross section consists of a T-shaped beam with a flange on top and a stem on the bottom. T-beams are often used in the construction of short to medium span bridges, and are known for their simplicity and ease of construction.

- I-beam: This type of cross section consists of an I-shaped beam with a top flange and a bottom flange. I-beams are often used in the construction of short span bridges, and are known for their simplicity and ease of construction.

- Composite beam: This type of cross section consists of a steel beam that is combined with a concrete slab or deck. Composite beams are often used in the construction of medium to long span bridges, and are known for their strength and stability.

2.5 Selection of Bridge Type

According to the guidelines outlined in Design and Abutments (2020), a good starting point for determining the depth of a deck in a bridge construction project is to aim for a span-to-depth ratio of 20. Additionally, using continuous decks over supports can reduce the number of expansion joints, decrease the maximum bending moment, and ultimately decrease the required depth or amount of materials used. However, it is important to note that utilizing continuous decks may increase the sensitivity to differential settlements. Table 2.1 shows recommended various types of deck based on span ranges.

Table 2.1: Useful span ranges of various types of deck, table from (Design and Abutments 2020)

Span	**Deck Type**
Up to 20m	Insitu reinforced concrete. Insitu prestressed post-tensioned concrete. Prestressed pre-tensioned inverted T beams with insitu fill.
16m to 30m	Insitu reinforced concrete voided slab. Insitu prestressed post-tensioned concrete voided slab. Prestressesd pre-tensioned Y and U beams with insitu slab. Prestressed pre-tensioned box beams with insitu topping. Prestressed post-tensioned beams with insitu slab. Steel beams with insity slab.
30m to 40m	Prestressed pre-tensioned SY beams with insitu slab. Prestressed pre-tensioned box beams with insitu topping. Prestressed post-tensioned beams with insity slab. Steel beams with insitu slab.
30m to 300m	Box girder bridges- As the span increases the construction tends to go from 'all concrete' to 'steel box / concrete deck' to 'all steel' Truss bridges - for spans up to 50m they are generally less economic than plate girders.
150m to 1000m	Cable stayed bridges.
350m to -	Suspension bridges.

2.6 Previous Studies on Span-to-Depth Ratio

The span-to-depth ratio, also known as the aspect ratio, is a measurement of the relationship between the length of a bridge's span and the depth of its structural elements. In reinforced concrete bridges, this ratio is typically used to determine the thickness of the deck, beams, and other structural components. The ideal span-to-depth ratio varies depending on the type of bridge and the loads it will bear. For example, a bridge with a longer span will typically have a higher span-to-depth ratio than a bridge with a shorter span.

There are different methods to determine the span-to-depth ratio, including analytical methods, experimental methods, and statistical methods. Analytical methods involve using mathematical formulas and computer models to predict the structural behavior of a bridge. Experimental methods involve building and testing physical models of a bridge to measure its performance. Statistical methods involve collecting data from existing bridges and using it to make predictions about new bridges.

The span-to-depth ratio is an important consideration in the early stages of bridge design. It can be used to determine the overall dimensions of the bridge, including the height of the deck, the size and spacing of the beams, and the thickness of the deck slab. It can also be used to estimate the amount of steel and concrete required for the bridge and to predict the structural behavior of the bridge under different loads. The proper selection of the span-to-depth ratio is important for ensuring the structural integrity and stability of the bridge and for achieving an economical solution.

There are many studies found in literature that investigates the span-to-depth ratio and its effect on the structure behavior and capacity, including a Master's thesis (Optimization of Span-to-Depth Ratios in High-Strength Concrete Girder Bridges) by Poon (2009), which provides a comprehensive overview of the different recommendations and guidelines for the span-to-depth ratio in concrete bridges, and serves as a useful reference for this research, shown in Table 2.2.

Table 2.2: Recommended span-to-depth ratios for reinforced and prestressed concrete bridges (Poon 2009)

Author	**Description**
Fritz Leonhardt, Professor of Civil Engineering at the University of Stuttgart, 1979	Suggests ratios based on values from previously constructed prestressed concrete bridges with good performance. For cast-in-place single-cell box-girder, a ratio of 21 is recommended. The suggested ratio is lowered to around 12 to 16 when incremental launching method is used due to the large negative construction moments associated with this construction method. For cast-in-place slab, he suggests values from 18 to 36, with the higher values used for longer spans and for bridges with lighter traffic.
ACI-ASCE, The American Concrete Institute-American Society of Civil Engineers, 1988	The recommendations are intended to provide general guidelines for preliminary design. For cast-in-place, post-tensioned multiple-cell box-girder, ACI-ASCE recommends ratios from 25 to 33. The recommended ratio for precast multiple-cell continuous box-girder is around 22. These ratios are higher than the ones for single-cell box-girder, because a multiple-cell box section has more webs to accommodate tendons compared to a single-cell section with similar width. The recommended range of ratios is between 24 and 40 for cast-in-place, post-tensioned slab.
Christian Menn, Professor of Structural Engineering at the Institute of Structural Engineering in Zurich, 1990	His suggestions are based on existing bridges with satisfactory performance in terms of structural behaviour, aesthetics, and economics. He recommends ratios between 17 and 22 for cast-in-place box-girders, because girders with ratios below 17 would appear too heavy. On the other hand, girders with ratios above 22 have substantial cost increase due to the significantly higher longitudinal prestressing demand. Menn also suggests a maximum practical limit of 25 for solid slab and a maximum cost-effective slab depth of 0.8m.
AASHTO, The American Association of State Highway and Transportation Officials, 1994	Defines optional criteria for span-to-depth ratios. These values are based on traditional maximum ratios of constant-depth continuous highway bridges with adequate vibration and deflection response. To ensure proper vibration and deflection behaviours, the maximum ratios are determined to be 25 for cast-in-place box-girder and 37 for cast-in-place slab.

M.Z. Cohn, Professor of Civil Engineering at the University of Waterloo, 1994	The span-to-depth ratios suggested in this paper are part of Ph.D. thesis prepared by Z. Lounis. These ratios are established from a systematic, multi-level optimization approach that determines the ideal cross-sectional dimensions, span layouts and superstructure system based on cost, material consumption, and aesthetics. For cast-in-place single-cell box-girder, the optimum ratio is found to range from 12 to 20. The ratio increases with span length and decreases with bridge width (e.g. a ratio of 12 corresponds to a span of 20m and a width of 16m while a ratio of 20 corresponds to a span of 50m and a width of 8m). This range of ratios is slightly lower relative to the ones from other publications, because this study investigates a simply-supported system while the ratios from other publications are mostly based on continuous systems. A simply-supported girder tends to be deeper since it experiences greater moments at midspan compared to a continuous structure. Cohn & Lounis also suggest the range of optimum ratios for voided and solid slabs are 22 to 29 and 28 to 33 respectively.
AASHTO-PCI-ASBI, The American Segmental Bridge Institute (ASBI), 1997	ASBI has established various standard precast sections for segmental construction to enhance uniformity and simplicity for forming and production methods. Using these standard sections generally lead to practical and cost-effective solutions. The ranges of span-to-depth ratios obtained from these standard sections are 17 to 19 for span-by-span method and 17 to 20 for balanced cantilever method.
Lian Duan, Senior Bridge Engineer with the California Department of Transportation and a Professor of Structural Engineering at Taiyuan University of Technology in China. 1999	A span-to-depth ratio of 25 is recommended for cast-in-place multiple-cell box-girder based on typical values from existing bridges. A range of ratios from 12.5 to 20 is recommended for precast segmental box-girder. This range is based on frequently used standard precast sections from Federal Highway Administration (FHWA).
Nigel Hewson, Recognized expert in the design and construction of prestressed bridges and is an Associate Lecturer at the University of Surrey, 2003	He suggested a span-to-depth ratio of 20 for cast-in-place single-cell box-girder and a maximum ratio of 20 for cast-in-place voided slab.
Paul Gauvreau, 2007	A span-to-depth ratio of 17 is recommended for precast segmental span-by-span constructed box-girder.

As shown in the Table 2.2, the research studies have presented a variety of recommended span-to-depth ratios for reinforced and prestressed concrete bridges, with variations based on factors such as the type of construction method, the type of bridge structure, and the intended use of the bridge. However, despite the advancements in material strengths and construction technologies, there has been little increase in the recommended span-to-depth ratio over the past decades. The use of high-strength concrete in bridge construction has the potential to lead to more slender structural components and longer span lengths, but the economic feasibility of this approach remains a significant challenge. Further research is needed to determine more efficient and cost-effective ways to utilize the enhanced properties of high-strength concrete in bridge design.

In the master's thesis written by Poon (2009), a review of 86 existing bridges with regard to the relationship between the span-to-depth ratios was conducted, also shown in Table 2.2. The review demonstrates the range of ratios that have been typically utilized over the past almost 60 years. Poon studies three types of bridges: cast-in-place box-girder, cast-in-place slab, and precast box-girder. The results of the study conducted by Poon are presented in the following Table 2.3.

Table 2.3: Summary of conventional span-to-depth ratios

Bridge type	**Range of span-to-depth ratios**	**Number of bridges within this range**	**Average ratio**	**Notes**
Cast-in-place box-girder	17.7 to 22.6	33 out of 44 (75%)	20	Range varies little between 1958 and 2002
Cast-in-place voided slab	19 to 35	13 out of 14 (92%)	27	Conventional ratio is closer to 20 for bridges completed after 1990
Cast-in-place solid slab	22 to 39	12 out of 14 (86%)	30	Used mainly from 1961 to 1975
Precast segmental box-girder	15.7 to 18.8	14 out of 14 (100%)	17	Range varies little between 1981 and 2007

The objective of the master thesis conducted by Poon (2009) is to highlight the importance of considering the optimal span-to-depth ratio when designing 8-span high-strength concrete girder bridges. Three different types of bridges were analyzed in the study, including solid slab bridges, precast segmental span-by-span box girder bridges, and cast-in-situ false work prestressed box girder bridges. A range of span lengths were considered for each type of bridge, including 35m to 75m for cast-in-place box girder bridges, 20m to 35m for solid slab bridges, and 30m to 50m for precast segmental box girder bridges. A variety of span-to-depth ratios were also evaluated for each type of bridge and span length, including 10 to 35 for cast-in-place box girder bridges, 30 to 50 for solid slab bridges, and 15 to 25 for precast segmental box girder bridges. The structural response and material consumption were evaluated for each combination of bridge type, span length, and span-to-depth ratio to determine the optimal ratio for each bridge system.

The research showed that using the traditional ratios from decades ago based on normal-strength concrete may not be economically feasible when using high-strength concrete. The study also found that the optimal ratios for the different bridge types and span lengths varied, with the most optimal ratios being 25 for cast-in-place box girders, 40 for solid slabs, and 20 for precast segmental box girders. The results of this research provide valuable insight for engineers and designers to consider when planning and constructing high-strength concrete girder bridges, ultimately leading to cost-effective, aesthetically pleasing, and structurally sound bridges.

2.7 Span-to-Depth Ratios By Norwegian Public Roads Administration

The Norwegian Public Roads Administration has published a bridge Handbook-4 for cast-in-place reinforced plate bridges (Statens Vegvesen 2000). This handbook contains standards with complete shape and reinforcement data for single- and three-span precast plate bridges. It gives recommendations for span-to-depth ratio. The handbook is developed for straight plate bridges and bridges with skews up to 10° and with span widths from 4 m to 20 m for single-span and 10 m to 20 m (mid-span) for three-span bridges. For three-span bridges, a side-span/mid-span ratio of 0.6 to 0.8 applies. The width of the bridge plate must be between 7 and 10 m for single-span bridges and between 5 and 10 m for three-span bridges (Statens Vegvesen 2000).
The handbook-4 is an outdated handbook based on previous Norwegian standards and not the Eurocodes.

2.7.1 Single Span Plate Bridges

When designing single-span slab bridges with mid-span length between 4 and 20m, the ratio of the span to the depth should be determined using Figure 2.3 as a reference.

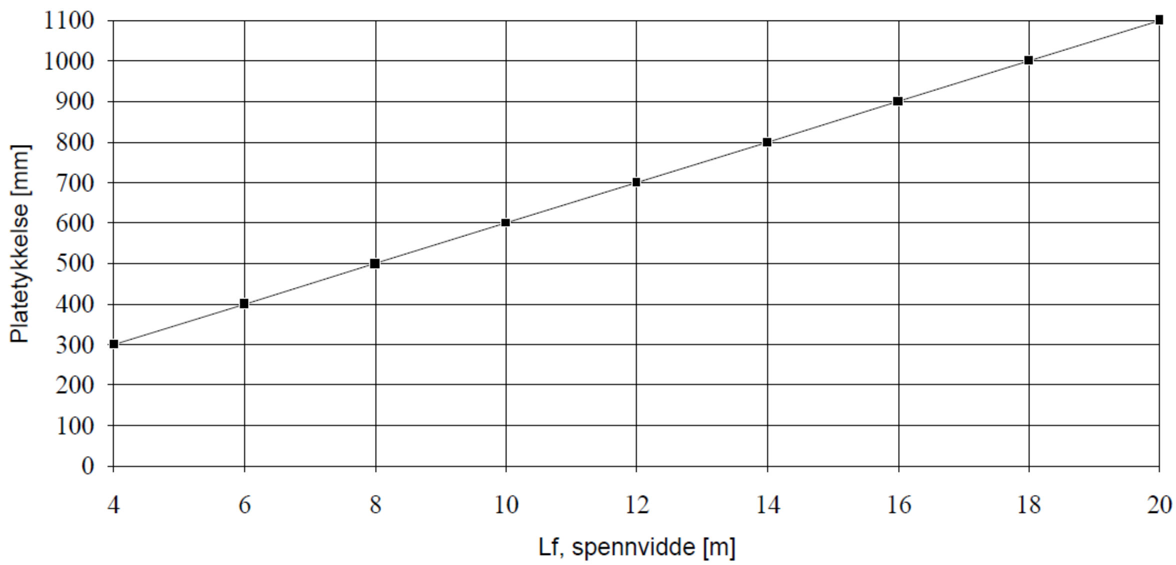


Figure 2.3: Slab thickness for single span slab bridges. Figure from (Statens Vegvesen 2000)

2.7.2 Three Span Plate Bridges

Handbook-4 (Statens Vegvesen 2000) provides recommendations for determining the thickness of a three-span plate bridge cross-section for mid-span lengths ranging from 10 to 20m. The total bridge length should be between 22 and 52m, and the total bridge width should be between 5 and 10m. Additionally, the skewness, or angle of deviation from a straight line, should not exceed 10 degrees. Figure 2.4 should be taken into account in the design phase.

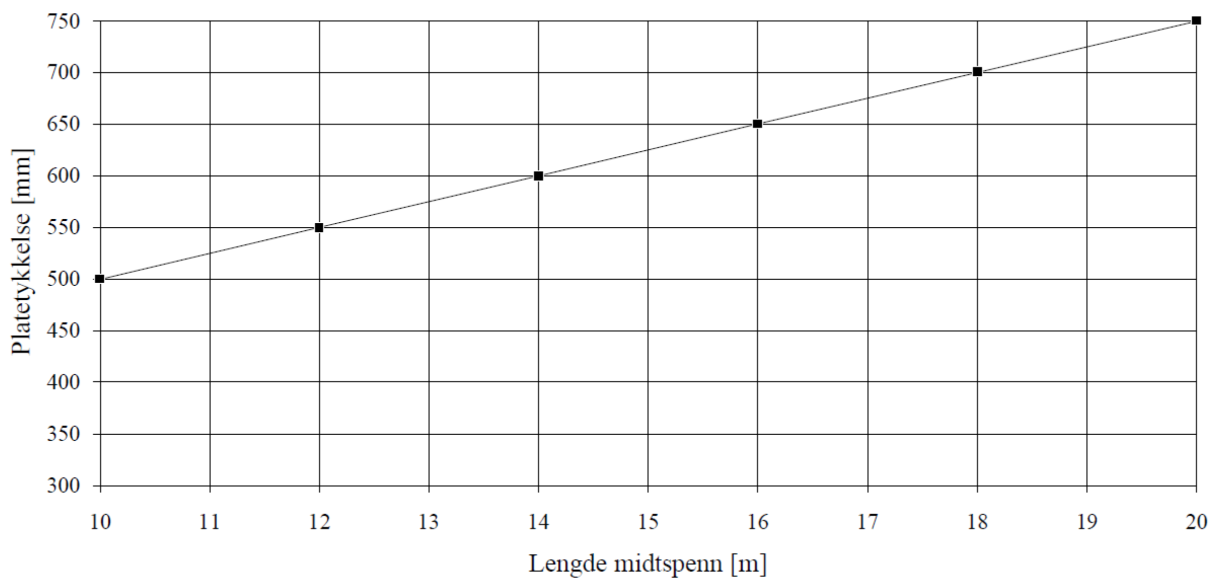


Figure 2.4: Plate thickness for three-span plate bridges. Figure from Handbook-4 (Statens Vegvesen 2000)

2.8 Recommendations in Eurocode 2

Eurocode 2 — Design of concrete structures — Part 1-1 (Standard Norge 2004a) provides guidelines for the relationship between the span and the effective height of reinforced beams and slabs, to guarantee requirements for limiting deflection are met. Deflections that could damage adjacent parts of the structure should be limited. For the deflection after construction, span/500 is normally an appropriate limit for quasi-permanent loads. The achieved span/height limits meet the requirements for limiting deflection, ensuring that the deflection for a beam or slab does not exceed the span/250. The limit state of deformation may be checked by limiting the span/depth ratio, according to clause 7.4.2 in Eurocode 2. The limiting span/depth ratio may be estimated using equations (Standard Norge 2004a).

$$l/d = K\left[11 + 1,5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3,2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{\frac{3}{2}}\right] if \rho \leq \rho_0 \tag{2.1}$$

$$l/d = K\left[11 + 1,5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] if \rho > \rho_0 \tag{2.2}$$

where:

l/d is the limit span/depth

K is the factor to take into account the different structural systems

ρ_0 is the reference reinforcement ratio = $\sqrt{f_{ck}}\ 10^{-3}$

ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

ρ' is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)

f_{ck} is in MPa units

Table 2.4 shows recommended basic ratios of span/effective depth for reinforced concrete members without axial compression, according to clause 7.4.2 in Eurocode 2.

Table 2.4: basic ratios of span/effective depth for reinforced concrete members without axial compression. Table from EC2-1 (Standard Norge 2004a)

Structural System	**K**	**Concrete highly stressed** ρ **= 1,5%**	**Concrete lightly stressed** ρ **= 0,5%**
Simply supported beam, one- or two-way spanning simply supported slab	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory

2.9 Regulations

The Norwegian regulations for concrete bridge design are based on the Eurocode 1 (EC1) and Eurocode 2 (EC2) standards, which are the European Union's harmonized standards for the design of concrete structures. The Norwegian regulations also include additional requirements and recommendations from the Norwegian Road Administration (NRA). The regulations ensure that the concrete is strong enough to resist the forces acting on the bridge, such as the weight of the deck and the live load of the vehicles and pedestrians crossing the bridge. Additionally, the regulations require that the design of the concrete bridge must consider the effects of temperature and shrinkage. The design must ensure that the bridge can withstand the thermal stresses caused by the expansion and contraction of the concrete due to temperature changes and the reduction in volume of the concrete due to shrinkage. A list over the relevant Handbooks, Standards and Eurocodes as following:

- Handbook N400, Bruprosjektering (Statens Vegvesen 2015)

- Eurocode 0: Basis of structural design (Standard Norge 2002)
- Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions (Standard Norge 2005a)
- Eurocode 1: Actions on structures - Part 1-5: General actions - Thermal actions (Standard Norge 2003a)
- Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges (Standard Norge 2003b)
- Eurocode 2 — Design of concrete structures — Part 1-1: General rules and rules for buildings (Standard Norge 2004a)
- Eurocode 2: Design of concrete structures - Concrete bridges - Design and detailing rules (Standard Norge 2005b)

Chapter 3

Research Methodology

The Research Methodology chapter of this study aims to investigate the material and sectional properties, loads, and loss of pre-stressing force in pre-stressed concrete bridges.

3.1 Work Plan

The objective of this research is to investigate the span-to-depth ratio of three-span reinforced and pre-stressed concrete bridges. The bridge models will have a length range of 20m to 100m, with a ratio of 0.75 between the span length of the side span and the main span. The total length of the bridges will be calculated as $L = 0.3L + 0.4L + 0.3L$, where 0.3L represents the length of the side span and 0.4L represents the length of the main span. This task aims to provide insight during the initial phase of a project and the results obtained can be utilized to establish a new foundation for determining the height of the bridge's cross-section and the amount of reinforcement required, based on the chosen span lengths.

In this research, 20 analysis models were created for the same type of bridge cross-section with varying heights, span lengths, and types of concrete (reinforced and pre-stressed). These models were designed and analyzed using the SOFiSTiK FEM program, and the necessary results were extracted. The design process is carried out in the ultimate limit state and the serviceability limit state. The reinforcement area is determined so that the requirements in all limit states are satisfied.

3.2 Parts of Bridge

In bridge design, the substructure and superstructure are distinguished by their geometry.

3.2.1 Superstructure

A bridge superstructure is the portion of a bridge that sits on top of the substructure and spans the gap between the two supporting foundations. It includes the roadway or deck, which is the surface that vehicles and pedestrians travel on, as well as any additional elements that are above the roadway, such as railings, sidewalks, and lighting. The superstructure also includes the structural elements that support the deck, such as beams, girders, and trusses. In this study, a constant plate cross-section width of 12m is employed, which is a widely adopted bridge width in Norway. This is illustrated in Figure 3.1.

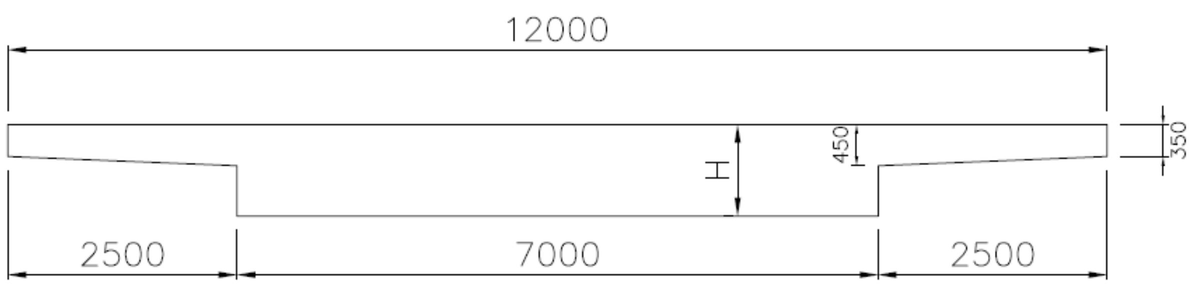


Figure 3.1: Illustration of the bridge cross section (Dimensions in mm), Figure is drawn in Auto-Cad

3.2.1.1 Barriers

Bridges should generally have barriers for controlled water drainage. This can be omitted if water drainage directly over the edge of the bridge does not cause problems underneath the bridge (Statens Vegvesen 2016). The barrier is designed in accordance with the minimum requirements outlined in chapter 4.4.3 of Handbook N400, and is placed on both sides of the bridge deck. The barrier has a general width and height of 540mm and 700mm, respectively. It has a clear height of 155mm over the asphalt layer. Figure 3.2 presents a side cross-sectional view of the barrier.

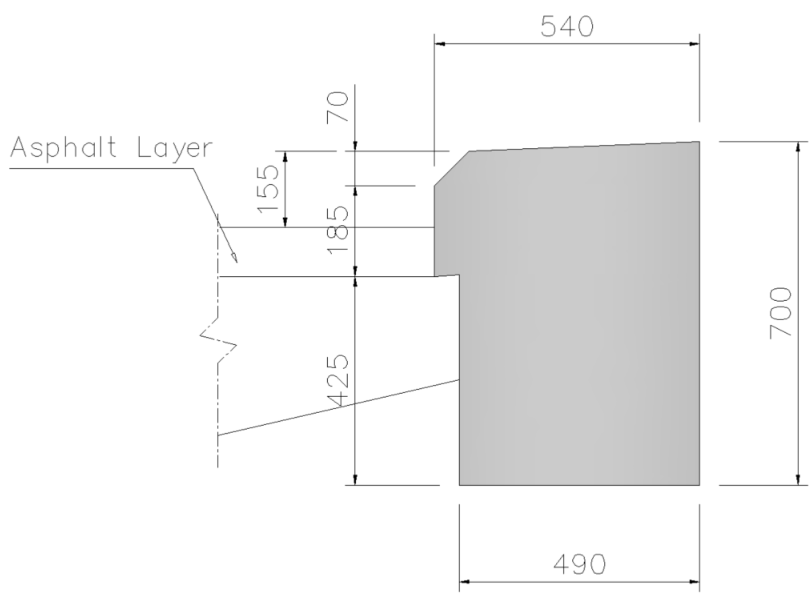


Figure 3.2: Illustration of barrier cross section (Dimensions in mm), Figure is drawn in AutoCad

3.2.2 Substructure

A bridge substructure is the portion of a bridge that sits below the roadway or deck and connects the bridge to its foundations. It includes the abutments, which are the supports at either end of the bridge, and the piers, which are the supports that are placed in the middle of the bridge. The substructure also includes any foundations that are required to support the piers, such as piles or footings. The substructure is responsible for transferring the load of the superstructure and the live load of the bridge (i.e. the weight of the vehicles and pedestrians crossing the bridge) to the underlying soil or rock (Midas Bridge 2023).

For our design, the substructure consists of two piers located at axes 2 and 3. Each pier has a rectangular shape, a cross-sectional dimension of 5000mm x 800mm and a height of 10m when free-standing, and is situated on land. The bridge models will have a range of bridge lengths from 20m to 100m, divided into three spans. The Lengths of spans and pier axes are L = 0.3L + 0.4L + 0.3L, where L is the total length, shown in Figure 3.3. The reason for the chosen span lengths is that generally, the side span should be between 0.6L to 0.8L of the main span. The main span is supported on both sides while a side span will be freely supported at the ends, which gives it a more balanced moment diagram. Therefore, it is statically favorable for the side span to be shorter than the main span. If the side span is too short, the end of the bridge could lift up under heavy load in the middle, which is not desirable. In this thesis, all bridge models are straight, without any curvature or skewed supports.

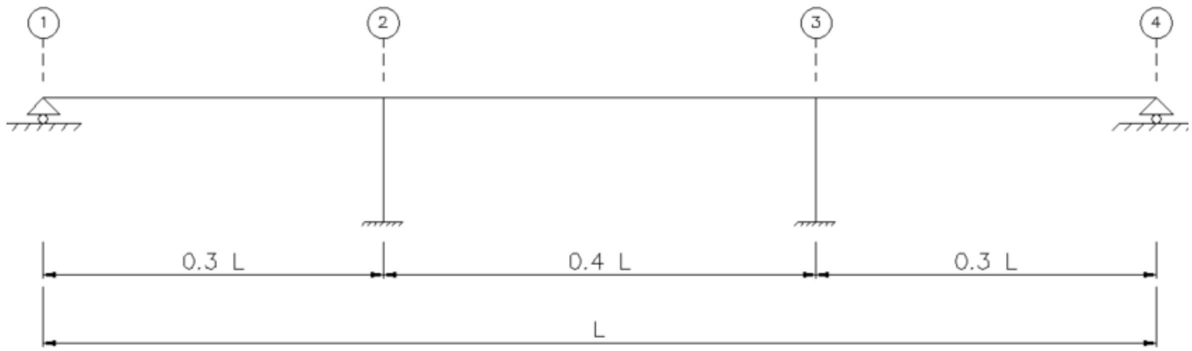


Figure 3.3: Illustration of bridge's substructure, Figure is drawn in AutoCad

3.3 Material and Sectional Properties

The main materials required for constructing reinforced and prestressed concrete bridges are concrete, reinforcement steel, and prestressing steel. This section provides an overview of the theoretical principles and key properties of these materials with regard to the relevant bridge type.

3.3.1 Concrete

Concrete consists of cement, sand, aggregate (coarse and fine), water and admixtures. Water to cement ratio (W/C) is an important factor of resulting concrete strength. Concrete has good properties such as high compressive strength, high stiffness and good volume stability. on the other hand, concrete is very weak in tension and needs steel reinforcement in order to withstand the tensile forces.

Both prestressed concrete bridges and reinforced concrete bridges use concrete as the primary structural material. In Norway, a high strength concrete B45 is commonly used for both types under normal conditions. A high strength concrete is made of low w/c mix design that has an advantage of reducing corrosion of the prestressing tendons. Concrete classes B35 – B55 are mostly used, but a concrete with another class in some cases is needed (Sørensen 2013). In this study, a concrete of class B45, which is commonly used for new bridges in Norway, is utilized.

3.3.1.1 Compressive Strength and Exposure Class

The compressive strength of concrete is denoted by concrete strength classes which relate to the characteristic (5%) cylinder strength f_{ck}, or the cube strength f_{ck},cube, in accordance with EN 206-1, EC2-1 (Standard Norge 2004a). As mentioned further in Eurocode 2-1, the strength classes are based on the characteristic cylinder strength f_{ck} determined at 28 days and the characteristic strengths for f_{ck} and the corresponding mechanical characteristics necessary for design, are given in Table 3.1 in Eurocode 2-1 as shown in Table 3.1. EC2-1 also shows how to specify the concrete compressive strength, $f_{ck}(t)$, at time different from 28 days as following equations:

$$f_{ck} = f_{cm}(t) - 8(MPa) \quad for \quad 3 < t < 28\,days \tag{3.1}$$

$$f_{ck}(t) = f_{ck} \qquad for \quad t \geq 28\,days \tag{3.2}$$

Table 3.1: Strength and deformation characteristics for concrete, Table is from Eurocode 2-1 (Standard Norge 2004a)

Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0{,}30\times f_{ck}^{(2/3)}$ ≤C50/60 $f_{ctm}=2{,}12\cdot\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0{,}7\times f_{ctm}$ 5% fractile
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1{,}3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 ε_{c1} (‰) $= 0{,}7\, f_{cm}^{0,31} < 2.8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1}(‰)$=2{,}8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2}(‰)$=2{,}0+0{,}085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2}(‰)$=2{,}6+35[(90-f_{ck})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n=1{,}4+23{,}4[(90-f_{ck})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{c3}(‰)$=1{,}75+0{,}55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3}(‰)$=2{,}6+35[(90-f_{ck})/100]^4$

Additionally, EC 2-1 shows the following relationship for the change of concrete strength at various ages f_{cm}(t):

$$f_{cm}(t) = \beta_{cc}(t)\, f_{cm} \tag{3.3}$$

with,

$$\beta_{cc}(t) = exp\left\{ s\left[1 - \left(\frac{28}{t}\right)^{\frac{1}{2}}\right]\right\} \tag{3.4}$$

where,

β_{cc} is time-dependent coefficient dependent on age of concrete, s is a factor depends on the cement strength class and t is age of concrete at which given in days.

The concrete design compressive strength f_{cd} as following equation :

$$f_{cd} = \frac{a_{cc} f_{cc}}{\gamma_c} \tag{3.5}$$

where α_{cc} equal to 0.85 is the coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied and γ_c is the partial safety factor for concrete equal to 1.5 and other material factors are taken from EC2-1 (Standard Norge 2004a), Table NA.2.1N. Strength and deformation characteristics for concrete are given in table 3.1 in EC2-1 and shown in Table 3.1. Design compressive in accordance with EC2-1 is shown in Table 3.2. Values are given in N/mm^2.

Table 3.2: Design Compressive Strength of Concrete

Concrete Class	**Characteristic Strength**	**f_{cd} at ULS**	**f_{cd} at SLS**	**f_{cd} at fatigue state**	**f_{cd} at ALS**
		$\gamma_c = 1.5$	$\gamma_c = 1.0$	$\gamma_c = 1.5$	$\gamma_c = 1.2$
B45	45	25.5	38.2	25.5	31.8

Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions (Standard Norge 2004a). Exposure classes are determined according to the EC2-1, table 4.1. (Standard Norge 2010) NA.4.2 (105) requires that concrete surfaces protected with a asphalt membrane be considered in exposure class XD1. In this thesis, exposure class XD1 is selected.

3.3.1.2 Tensile Strength

The tensile strength of concrete is relatively low compared to its compressive strength. It can be improved by using steel reinforcement or by adding fibers to the concrete mix. The design tensile strength f_{ctd} is defined in EC2-1 (Standard Norge 2004a) as following equation :

$$f_{ctd} = \alpha_{ct} \frac{f_{ctk,0,05}}{\gamma_c} \tag{3.6}$$

where, α_{ct} is a coefficient taking account of long term effects on the tensile strength set to 0.85. Design tensile strength and safety factors in accordance with EC2-1 (Standard Norge 2004a) is shown in Table 3.3. Values are given in N/mm^2.

Table 3.3: Design Tensile Strength of Concrete

Concrete Class	**Characteristic Strength**	**f_{ctd} at Ultimate Limit States (ULS)**	**f_{ctd} at Serviceability Limit States (SLS)**	**f_{ctd} at Fatigue States**	**f_{ctd} at Accidental States**
		γ_c =1.5	γ_c =1.0	γ_c =1.5	γ_c =1.2
B45	2.7	1.53	2.3	1.53	1.91

3.3.1.3 Modulus of Elasticity

The elastic modulus of concrete E_{cm} also known as Young's modulus is shown in Figure 3.4 and defined as the ratio of applied stress to corresponding strain within the elastic limit of the material. The ranges for the modulus of elasticity are given in table 3.1 in EC2 and shown in Figure 3.1. According to EC2-1 (Standard Norge 2004a), the exact value depends on factors such as the type and amount of aggregates, water-cement ratio, and curing conditions. For limestone and sandstone aggregates the value should be reduced by 10% and 30% respectively. For basalt aggregates the value should be increased by 20% (Standard Norge 2004a). As it can be seen in Figure 3.4 that the concrete has a nonlinear behaviour called material non-linearity. A Linear approximation for the elastic modulus can be made from the slope line where the secant value must be between σ_c = 0 and 0,4f_{cm}. Where f_{cm} is the mean compressive strength after 28 days for concrete (Standard Norge 2004a).

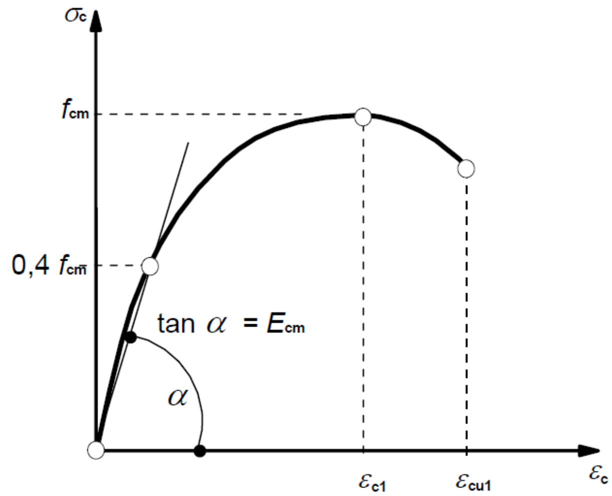


Figure 3.4: Schematic representation of the stress-strain relation, Figure is taken from figure 3.2 EC2-1 (Standard Norge 2004a)

3.3.1.4 Creep

According to Haseeb Jamal (2017), creep is a gradual deformation or change in shape of a structure under sustained load over time. This deformation occurs in the direction the force is being applied and when concrete is loaded, there will be an immediate contraction. Creep can have significant impacts on the performance and service life of concrete structures, specially structures that are subjected to long-term loads. Aggregate type, mix proportions, age of concrete and load types are factors that affect creep. Similarly, EC2 states that the creep of concrete depends on the ambient humidity, element dimensions, compressive strength and concrete composition. The creep deformation of concrete $\varepsilon_{cc}(\infty, t_0)$ at time $t = \infty$ for a constant compressive stress σ_c applied at the concrete age t_0, is given in EC2-1 (Standard Norge 2004a) by :

$$\varepsilon(\infty, t_0) = \varphi(\infty, t_0) \cdot \frac{\sigma_c}{E_c} \tag{3.7}$$

Where,

$\varphi(\infty, t_0)$ is the creep coefficient. E_c is the tangent modulus that may be determined as 1.05 $E_{cm}(t)$. According to EC2-1 (Standard Norge 2004a), if the concrete is not subjected to compressive strength greater than 0.45 $E_{ck}(t_0)$, the creep coefficient can be taken from Figure 3.5.

E_{cm} is the E-modulus for concrete with age t < 28 days and can be found by the following equation :

$$E_{cm}(t) = (\frac{f_{cm}(t)}{f_{cm}})^{0.3} * E_{cm} \tag{3.8}$$

Where, $f_{cm}(t)$ is the compression strength at age t days, E_{cm} and f_{cm} are determined at 28 days.

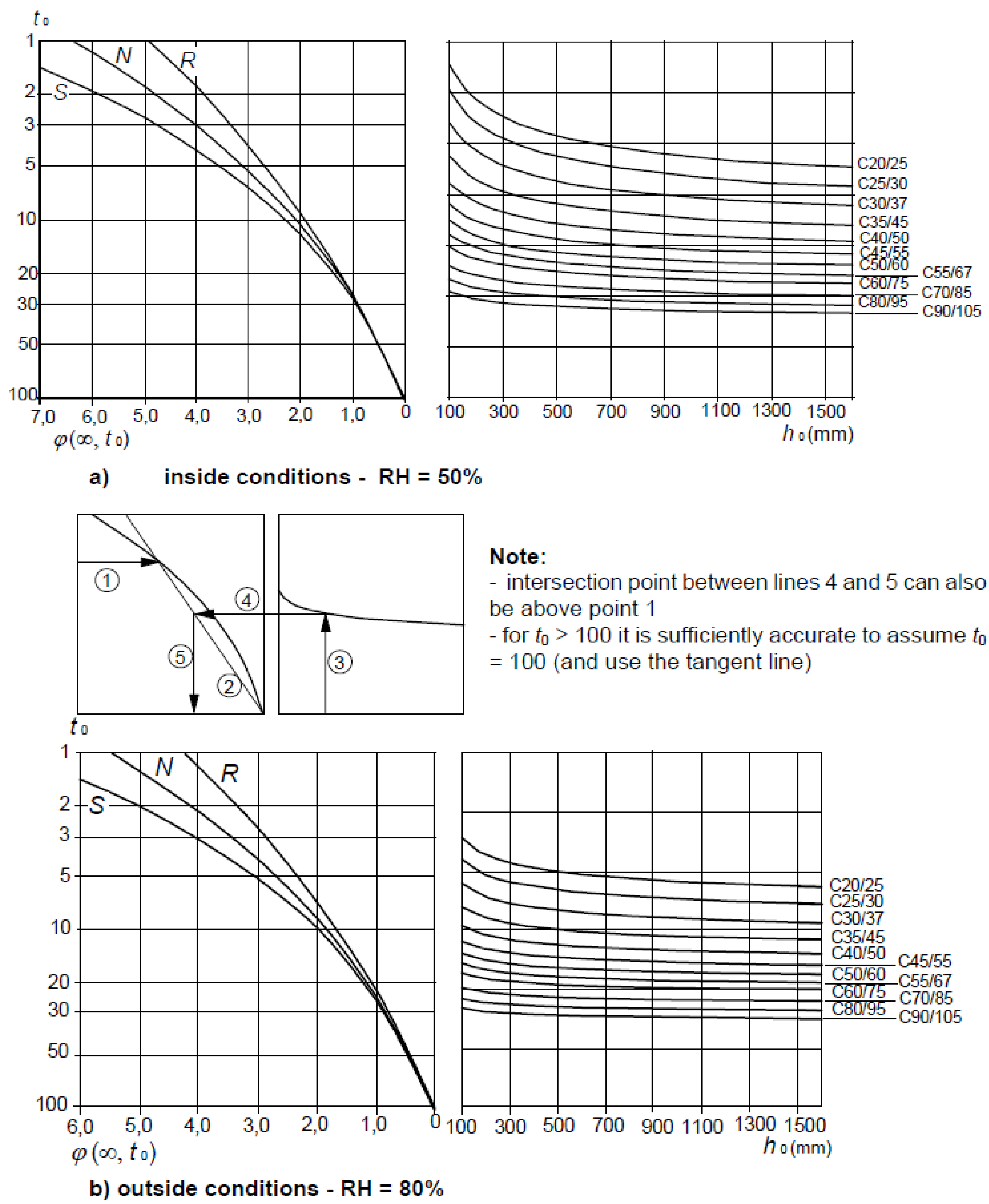


Figure 3.5: Determination of creep coefficient $\varepsilon_{cc}(\infty,t_0)$ for concrete, Figure is taken from figure 3.1 EC2-1 (Standard Norge 2004a)

3.3.1.5 Shrinkage

Concrete shrinkage is independent of stress and refers to the reduction in volume that occurs in concrete as it dries and hardens over time. This shrinkage can result in cracking, especially in large concrete structures. Factors such as the mix design, moisture content, temperature changes and curing conditions can all impact the amount of shrinkage that occurs in concrete. Both concrete shrinkage and creep can cause damages to concrete structures, including curvature on reinforced and pre-stressed concrete cross-sections and loss of pre-stress, leading to deformation and potentially failure over time. Minimizing shrinkage is important for ensuring the durability and long-term performance of concrete structures. The total shrinkage strain ε_{cs} is made up of two parts: the drying shrinkage strain ε_{cd}, which slowly forms as a result of water loss, and the autogenous shrinkage strain ε_{ca}, which develops when the concrete hardens (Standard Norge 2004a). The total shrinkage strain is found as following:

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \tag{3.9}$$

3.3.2 Steel Reinforcement

Steel reinforcement, also known as rebar or mesh, is made from a raw material called iron ore and used in concrete to provide additional strength in areas where the concrete is weak in tension. Steel is strong in both tension and compression, making it ideal for reinforcing concrete structures. Carbon steel is one component of steel reinforcement, which is hot-rolled with patterns to create a strong bond with the concrete. Reinforcing steel bars with a circular cross-section and transverse ribs are utilized to create a bond in concrete. To accommodate sufficient capacity for the forces and ensure that cracks remain within an acceptable width, reinforcement is implemented for bridge decks in both longitudinal and transverse directions, along with stirrups. Table 3.4 provides key parameters for B500NC steel. In this thesis, steel reinforcement of type B500NC is used for all bridge models.

Table 3.4: B500NC steel parameters

Symbol	Value	Description
f_{yk}	500 MPa	Yield strength
$f_{yd} = \frac{f_{yk}}{\gamma_s}$	434.8 MPa	Design yield strength
E_s	200 000 MPa	Modulus of elasticity
ε_{uk}	0,035	Characteristic strain
ε_{ud}	0,03	Upper yield strain
$\varepsilon_u = \frac{f_{yd}}{E_s}$	0,00217	Design strain

3.3.3 Prestressing Steel

Prestressing steel is used to provide high tensile strength to the concrete structure. To achieve full prestressing, it is important to use steel with high strength. However, over time, the prestressing forces will be reduced due to the effects of creep and shrinkage in concrete and relaxation in prestressing steel. Creep and shrinkage cause shortening in concrete, which in turn affects the prestressing steel attached to it. Due to the similarity in modulus of elasticity between reinforcing steel and high-strength prestressing steel, the reduction of stress in prestressed reinforcement and corresponding reduction of compressive stress in concrete are mostly unaffected by the steel strength. As a result, stress loss is proportionally lower for high-strength prestressing steel compared to ordinary reinforcing steel (Sørensen 2013). Figure 3.6 shows a cross-section of a typical tendon.

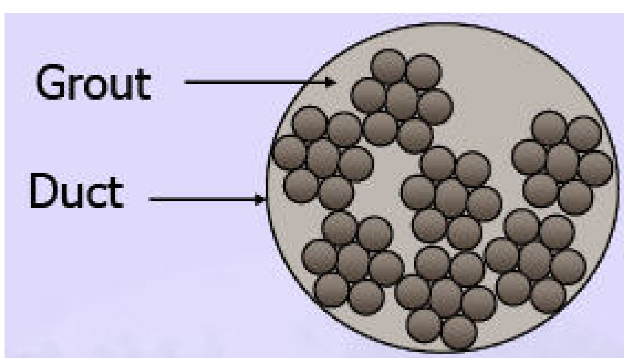


Figure 3.6: Cross section of a typical tendon, figure is from (Materia 2008)

According to EC2-1 (Standard Norge 2004a), section 3.3.2, the properties of prestressing steel are given in EN 10138 or European Technical Approval, ETA.

3.3.3.1 Strength and Ductility

Figure 3.7 shows the stress-strain curve for typical prestressing steel. The characteristic tensile strength is denoted as f_{pk}, while the yield strength is $f_{p0,1k}$, which is equivalent to 0.1% proof stress. The characteristic value of the 0.1% proof load and the characteristic maximum axial tension load are both divided by the nominal cross-sectional area, as specified by the EC2-1 (Standard Norge 2004a). Additionally, ϵ_{uk} represents the elongation at maximum load.

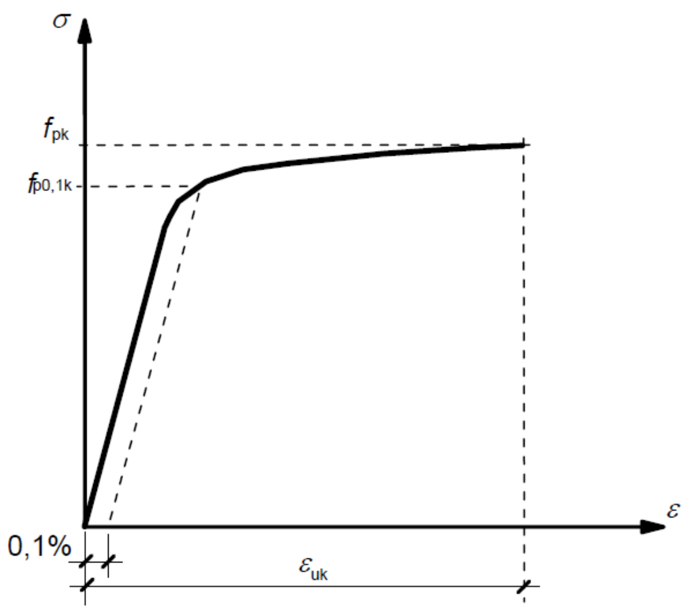


Figure 3.7: Stress-strain curve for typical prestressing steel, figure is from EC2-1 (Standard Norge 2004a)

Eurocode 2-1 (Standard Norge 2004a) states in section 3.3.4 (5) the following equation to ensure the adequate ductility of prestressing steel in tension:

$$\frac{f_{pk}}{f_{p0,1k}} \geq k \tag{3.10}$$

Where, 1.1 is the recommended value of k.

3.3.3.2 Elastic Modulus

According to Eurocode 2-1 section 3.3.6 (2) (Standard Norge 2004a), the stiffness of prestressing steel is determined by its modulus of elasticity. For the strand, the value assumed for the modulus of elasticity E_p is 195 GPa.

3.3.3.3 Steel Relaxation

In Eurocode 2-1 section 3.3.2 (Standard Norge 2004a), the classification of steel relaxation is divided into three categories. Class 1 and 2 are for wire or strand with ordinary relaxation and low relaxation, respectively. Class 3 is for hot-rolled and processed bars. The loss of steel relaxation is defined as the ratio of the change in prestressing stress to the initial prestressing stress as shown in following equations:

$$\text{Class 1} \quad \frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 5.39\rho_{1000}e^{6.7\mu}\left(\frac{t}{1000}\right)^{0.75(1-\mu)} 10^{-5} \tag{3.11}$$

$$\text{Class 2} \quad \frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 0.66\rho_{1000}e^{9.1\mu}\left(\frac{t}{1000}\right)^{0.75(1-\mu)} 10^{-5} \tag{3.12}$$

$$\text{Class 3} \quad \frac{\Delta\sigma_{pr}}{\sigma_{pi}} = 1.98\rho_{1000}e^{8\mu}\left(\frac{t}{1000}\right)^{0.75(1-\mu)} 10^{-5} \tag{3.13}$$

Where,

$\Delta\sigma_{pr}$ is absolute value of the relaxation losses of the prestress.

σ_{pr} for post-tensioning, σ_{pi} is the absolute value of the initial prestress $\sigma_{pi} = \sigma_{pm0}$

t is the time after tensioning (in hours)

$\mu = \frac{\sigma_{pi}}{f_{pk}}$, where fpk is the characteristic value of the tensile strength of the prestressing steel

ρ_{1000} is the value of relaxation loss (in %), at 1000 hours after tensioning and at a mean temperature of 20°C.

3.3.3.4 Material Properties

The prestressing system used in this thesis is the BBR VT CONA CMI BT 2706-150-1860 Internal Post-tensioning System, which utilizes a 27-wire prestressing steel strand with a nominal diameter of 15.7mm and nominal cross-sectional area of 150 mm^2. The total number of tendons used in each model is 10. This system has characteristic values listed in Table 3.5, and the tendon properties are shown in Table 3.6

Table 3.5: Characteristic values of post-tensioning system

System	**Characteristic values [MPa]**			**ULS [MPa]**		**SLS [MPa]**		**Fatique [MPa]**		**ALS [MPa]**	
1860	f_{pk}	$f_{p0,1k}$	E_{pk}	f_{pd}	E_{pd}	f_{pd}	E_{pd}	f_{pd}	E_{pd}	f_{pd}	E_{pd}
	1860	1640	195000	1426	-	1640	-	1426	-	1640	-

Table 3.6: Material properties of tendon

Tendon Properties		
Cross-sectional area of a single strand	A_p [mm^2]	150
Number of strands	n [pcs]	27
Cross-sectional area of a single tendon	n x A_p [mm^2]	4050
Char. value of maximum force	F_{pk} [KN]	7533
Char. value of 0.1% proof force	$F_{p0.1k}$ [KN]	6642
Max. prestressing force	0.90 $F_{p0.1k}$ [KN]	5977.8
Max. overstressing force	0.95 $F_{p0.1}$ [KN]	6310

3.3.4 Partial Factors for Materials

The partial factors for concrete refer to the values used to account for uncertainties in the design process, such as the variability in material strength, workmanship, and environmental effects. These factors are used in structural engineering to ensure a sufficient safety margin in the design, so that the structure can withstand the expected loads and environmental conditions. Partial factors for different limit states are given in Table 3.7 (Standard Norge 2004a).

Table 3.7: Partial factors at different limit states

Design situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Ultimate limit state (ULS)	1.50	1.15	1.15
Fatigue state	1.50	1.15	1.15
Accidental state (ALS)	1.20	1.00	1.00
Serviceability limit state (SLS)	1.00	1.00	1.00

3.4 Design Criteria

3.4.1 Design Service Life

The minimum design service life shall be 100 years according to EC0 (Standard Norge 2002), Table 2.1. The design service life means the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.

3.4.2 Reliability Classes

Reliability classes are used to classify the level of reliability required for a particular structure. They are used to ensure that structures are designed to withstand the expected loads and environmental conditions for their intended lifespan. Similarly, they also take into account uncertainties and variations in the loads and materials.
Reliability classes are determined according to EC0 (Standard Norge 2002), Table NA.A1(901). The bridge structure belongs to reliability class RC3.

3.4.3 Concrete Cover

According to Eurocode 2-1 (Standard Norge 2004a), Section 4.4.1.1, the concrete cover refers to the distance between the surface of the reinforcement, which is closest to the adjacent concrete surface, and that nearest concrete surface. This measurement includes links, stirrups, and any relevant surface reinforcement.

Concrete cover ensures that the reinforcement is protected from atmospheric conditions, preventing corrosion, fire protection and ensuring the durability and long-term performance of the reinforced concrete structure. After selecting the exposure class, the nominal cover C_{nom} is determined. The nominal cover is defined as the sum of the minimum cover C_{min} and the deviation tolerance ΔC_{dev} equal to 10mm according to EC2, NA.4.4.1.3(1)P. C_{nom} is calculated by the following equation:

$$C_{nom} = C_{min} + \Delta C_{dev} \tag{3.14}$$

Both Eurocode 2-1 (Standard Norge 2004a) and Handbook N400 (Statens Vegvesen 2015) provide specifications for the minimum concrete cover, C_{min}, required to ensure sufficient protection against atmospheric conditions and corrosion. C_{min} is given and described by the following equation:

$$C_{min} = max\{C_{min,b}\,;\, C_{min,dur}\,;\, 10mm\} \tag{3.15}$$

where,

$C_{min,b}$ is the minimum cover due to bond requirement, determined in EC2, Table NA.4.5N.

$C_{min,dur}$ is the minimum cover due to environmental conditions

Concrete cover according to EC2:

The largest assumed conservative bar diameter is bundle of 2ø32 and the largest nominal aggregate size is 22 mm

Minimum bond cover, $C_{min,b}$ = 45 mm

Minimum durability cover per EC2, $C_{min,dur,EC2}$ = 50 mm

Deviation tolerance, ΔC_{dev} = 10 mm

$C_{min,EC2}$ = max ($C_{min,b}$; $C_{min,dur,EC2}$; 10 mm) = 50 mm

Nominal cover $C_{nom,EC2} = c_{min,EC2} + \Delta C_{dev}$ = 50 mm + 10 mm = 60 mm

Concrete cover according to Handbook N400, Chapter 7.4 :

Minimum durability cover per N400, $C_{min,dur,N400}$ = 60 mm

Minimum cover $C_{min,N400}$ = max($C_{min,b}$; $C_{min,dur,N400}$) = 60 mm

Deviation tolerance, ΔC_{dev} = 15 mm

Nominal cover $C_{nom,N400} = C_{min,N400} + \Delta C_{dev}$ = 60 mm + 15 mm = 75 mm

The nominal cover used on both the top and bottom of the bridge is set to 75 mm

3.4.4 Crack Requirements

The crack,w_{max}, requirements are determined according to EC2-1 (Standard Norge 2004a), Table NA.7.1N as shown in Table 3.8. The crack load is calculated according to EC0 (Standard Norge 2002), Table NA.A2.6, quasi-permanent combination.

Table 3.8: Limiting values of w_{max} , Table is from Eurocode 2-1 (Standard Norge 2004a)

Exposure Class	**Reinforced members and prestressed members with unbonded tendons**		**Prestressed members with bonded tendons** [c)]	
	Load combination	**Limiting value**	**Load combination**	**Limiting value**
X0	Quasi-permanent	0,40[a)]	Frequent	0,30 k_c
XC1, XC2, XC3, XC4	Quasi-permanent	0,30 k_c	Frequent	0,20 k_c
XD1, XD2, XS1, XS2	Quasi-permanent	0,30 k_c	Frequent	0,20 k_c
			Quasi-permanent	Decompression[b)]
XD3, XS3	Frequent	0,30 k_c	Frequent	Decompression[b)]
XSA	Given special consideration [d)]		Given special consideration [d)]	

a For the X0 exposure class, crack width has no influence on durability, and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions, this limit may be relaxed.

b Where verification that decompression does not occur is required, the whole section of pre-stressing steel and where relevant the duct for post-tensioned tendons, should lie at least Δc_{dev} within the compression zone.

c Where the pre-stressing steel lies within a layer of ordinary reinforcement, the calculated crack width must meet the requirements for both ordinary reinforcement and pre-stressing steel. For verification against the requirement for prestressing steel the frequent load combinations are used, the calculated crack width may be adjusted with the formula $w_{2k} = w_k (\varepsilon_{s2} / \varepsilon_{s1})$, where ε_{s1} is the tensile strain in the reinforcement on the side with greatest strain, ε_{s2} is the tensile strain at the level of the prestressing steel and w_{2k} is an adjusted calculated crack width that is compared with the limiting values in the table.

d A total assessment is necessary in these cases in order to arrive at an appropriate combination of structural detailing, material composition, cover, limitation of crack width and other protective measures.

Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions (Standard Norge 2004a). In thesis thesis, the exposure class is XD1. This exposure class gets limiting value of the crack width w_{max} = 0.30 k_c in case of reinforced concrete . Where, k_c takes into account the effect of greater cover than the requirement for $C_{min,dur}$ and is determined by the formula:

$$k_c = \frac{C_{nom}}{C_{min,dur}} \leq 1.3 \tag{3.16}$$

Hence,

$$w_{max} = 0.3 \cdot k_c = 0.3 \cdot min\left(\frac{C_{nom,N400}}{C_{min,EC2}}; 1.3\right) mm = 0.39mm \tag{3.17}$$

Additionally, according to Table NA.7.1N in case of prestressing, the superstructure should be checked against decompression relief, which is done in SLS (Serviceability Limit State) quasi-permanent combination, and against crack width which often occurs in SLS frequent combination with an allowable crack width of $0.2 \cdot k_c = 0.26$mm.

3.5 Loads

Loads on a structure refer to any kind of external force or weight that a bridge must be able to support. The loads on a structure can be divided into several categories, such as: dead, live, wind, earthquake, snow, hydrostatic and dynamic loads. These loads on a structure result in internal forces within the structure, such as axial forces, bending moments, shear forces, and torsional forces. These internal forces must be taken into account in the design of the structure to ensure that the structure can withstand the expected loads and environmental conditions. The internal forces are calculated using the principles of structural mechanics and the properties of the materials used in the structure, such as the strength and stiffness of the concrete or steel. The structure must be designed such that the maximum internal forces do not exceed the capacity of the materials, and the structure must also be able to deform within acceptable limits to accommodate any deformations or movements caused by the loads. Loads on structures are classified by EC0 (Standard Norge 2002).

- Permanent loads (G): such as the weight of the structure itself, barrier, rails and paving.
- Live loads (Q): temporary loads such as wind load, snow load, the weight of people and vehicles.
- Accidental load (A): impact loads from vehicles, explosions, and falling objects are exam-

ples of accidental loads.

3.5.1 Permanent Loads

Permanent loads, also known as dead loads, are loads that are always present on a structure and do not change over time. They include the weight of the structure itself, any permanent fixtures such as barrier, rails, paving and any other features that are part of the structure. Permanent loads are typically estimated based on the weight of the materials used in the construction of the structure.

3.5.1.1 Self-Weight

According to Handbook N400 (Statens Vegvesen 2015), The self-weight of the bridge includes the weight of the bridge cross section, railing, and all other permanent installations. To calculate the self-weight, the volume of each component of the bridge is multiplied by its density, and the results are combined to obtain the total self-weight of the bridge. The density of concrete structural elements $\rho = 25$ KN/m^3. This is applies for bridge deck, barriers and columns.
Asphalt and railings are other permanent parts that contribute to the self-weight, but they are not made of concrete.

3.5.1.2 Bridge Deck

The self-weight of the bridge deck is calculated by multiplying the cross-sectional area by the density of reinforced concrete for each component. Table 3.9 presents the cross-sectional area and corresponding self-weight for four different bridge models out of a total of 20.

Table 3.9: Self-weight of bridge deck

Bridge model no.	**Deck Thickness [mm]**	**Cross Sectional Area [m^2]**	**Self-Weight [KN/m]**
Model 8-1	450	5.15	128.75
Model 16-3	550	5.85	146.25
Model P32-2	1100	9.7	242.5
Model p36-2	1550	12.85	321.25

3.5.1.3 Barriers

The self-weight of the barrier is calculated as a distributed load along each side of the bridge. The cross-sectional area of the barrier is estimated to be $A_{barrier}$ = 0.348 m^2. This results in a distributed load value of G_{bar} = 8.7 kN/m, which is calculated as follows:

$$G_{bar} = A_{barrier} \ \ x \ \ \rho_{concrete} = 8.7KN/m \tag{3.18}$$

3.5.1.4 Rails

Bridge railings are guardrail systems designed to prevent people or vehicles from falling off the bridge. They can be made of concrete or steel (The Constructor 2021). The type of railing being used is made of steel and has a self-weight G_{rails} of 0.5 kN/m, as specified in Handbook R412 (Statens Vegvesen 2014).

3.5.1.5 Paving

According to Handbook N400 (Statens Vegvesen 2015), the minimum requirements for dimensional paving weights in the roadway are determined based on the largest span shown in Table 3.10. The largest span length in this case is 0.4 * L = 40m, where L = 100m is the longest bridge model. As a result, an asphalt self-weight of 3.5 kN/m^2 is selected. The distributed load value of $G_{asphalt}$ in kN/m is calculated by multiplying the asphalt self-weight by the bridge width (12m), as follows:

$$G_{asphalt} = 3.5KN/m^2 \ \ x \ \ 12m = 42KN/m \tag{3.19}$$

Table 3.10: Minimum requirements for dimensional paving weights in the roadway

Largest span width l [m]		
$l \leq 50$	$50 < l \leq 200$	$l > 200$
3.5 kN/m^2	2.5 kN/m^2	2.0 kN/m^2

3.5.2 Variable Loads

Variable loads on a concrete bridge refer to loads that change in magnitude, direction, or location over time. These types of loads can include moving vehicles, wind, snow, and earthquakes. It is important to design concrete bridges to withstand these variable loads in addition to static loads. This can be done by considering load factors, load combinations, and load distributions in the design process. Traffic loads and natural loads will be discussed in this section.

3.5.2.1 Traffic Load

Loads due to the road traffic, consisting of cars, lorries and special vehicles, give rise to vertical and horizontal, static and dynamic forces (Standard Norge 2003b). Pedestrians and bicycles also contribute to the traffic loads. Traffic loads on road bridges are calculated according to Eurocode 1: Actions on structures, Part 2: Traffic loads on bridges (Standard Norge 2003b).
Calculations are performed by looking at various load models. The four different load models that act vertically on a bridge are:

- Load Model 1 (LM1) : Uniformly distributed loads (UDL) with Bogie load (TS).
- Load Model 2 (LM2) : A single axle load applied on specific tyre contact areas.
- Load Model 3 (LM3): Axle loads representing special vehicles.
- Load Model 4 (LM4) : UDL from crowds of people.

Load Models 2, 3, and 4 are not considered in this thesis. For Load Model 2, the maximum moment is calculated using the equation PL/4, where P = 400 kN (given in section 4.3.3 in EC1-2 (Standard Norge 2003b)), which does not result in a higher bending moment compared to Load Model 1. Furthermore, according to the Handbook (Beregningsveiledning for etteroppspente betongbruer), section 1.7.6 (Statens Vegvesen 2017), LM3 is not dimensioning, and LM4 will never be governing for small and medium-sized bridges. In addition, LM4 is not applicable in this thesis since there are no sidewalks on the bridge models.
Besides the four load models, braking and acceleration forces, centrifugal forces, and other transverse forces must be taken into account when designing the bridge. The acceleration forces

are considered along with the forces in the opposite direction of the braking forces, both with the same magnitude. Centrifugal forces and other transverse forces can be disregarded since the bridges are straight with no skewness at the supports and lack any curvature.

The divisions of the carriageway into notional lanes are defined in in EC1-2 and presented in Table 3.11 and lane numbering is shown in Figure 3.8

Table 3.11: Number and width of notional lanes

Carriageway width w	**Number of notional lanes**	**Width of a notional lane w_l**	**Width of the remaining area**
w <5,4 m	$n_1 = 1$	3 m	w - 3 m
5,4 m w ≤ <6 m	$n_1 = 2$	$\frac{w}{2}$	0
6 m ≤ w	$n_1 = \text{Int}\left(\frac{w}{3}\right)$	3 m	w - 3 x n_1

where,

n_1 is the greatest possible whole (integer) number of lanes.

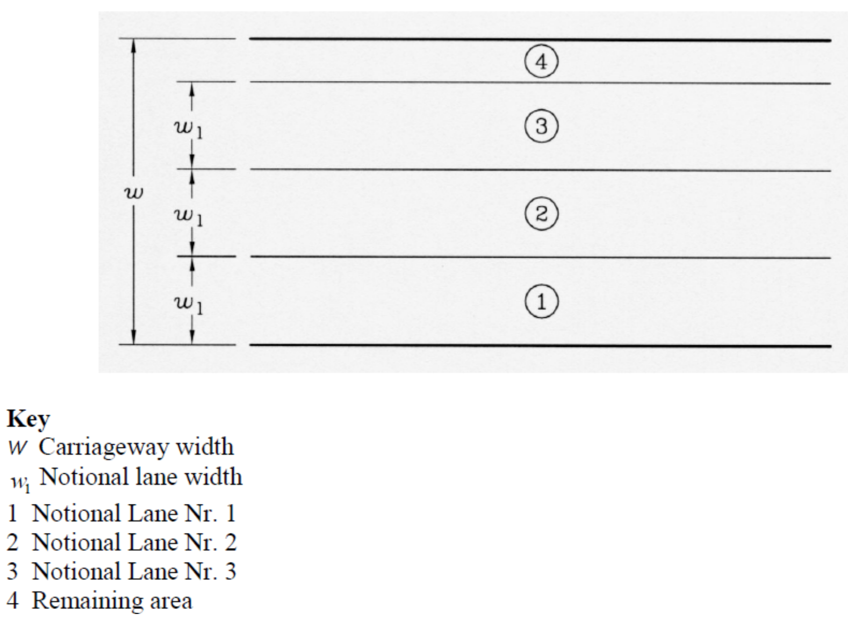


Figure 3.8: Lane numbering, Figure is taken from figure 4.1 in EC1-2 (Standard Norge 2003b)

Table 3.11 dictates that four traffic lanes will be present, each having a width of three meters. Thus, there will not be any unused space left over. The calculations are outlined as follows and summarized in Table 3.12.

$$W_{bridge} = 12m > 6m$$
$$n_l = Int(\frac{W_{bridge}}{3}) = 4$$
$$W_l = 3m$$
$$W_{res} = W_{bridge} - 3 \text{ x } n_l = 0m$$

Table 3.12: Summarize of number and width of lanes

Bridge width	12 m
Lane width	3 m
Number of lanes	4
Width of remaining area	0 m

3.5.2.1.1 Load Model 1

A double-axle load that covers most of the effects of the traffic of lorries and cars. It combines both of uniformly distributed loads (UDL), q_{ik}, and concentrated loads (TS) such as bogie loads,Q_{ik}. The characteristic values of q_{ik} and Q_{ik} are listed in Table 3.13, the adjustment factors α_{Qi} and α_{qi} as specified in EC1-2 (Standard Norge 2003b) and the load values for load model 1 can be found in Table 3.14.

Table 3.13: Load model 1 : characteristic values

Location	**Tandem system TS** **Axle loads Q_{ik} (KN)**	**UDL system** **q_{ik} (orq_{rk}) (KN/m^2)**
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

Table 3.14: Adjustment factors and load values for LM1

Location	**Adjustment factor**	**Axle load (KN)**	**UDL (KN/m^2)**	**Load**
Lane Number 1	$\alpha_{Q1} = 1{,}0$	$Q_{1k} = 300$	-	300 KN
	$\alpha_{q1} = 0{,}6$	-	$q_{1k} = 9{,}0$	5,4 KN/m^2
Lane Number 2	$\alpha_{Q2} = 1{,}0$	$Q_{2k} = 200$	-	200 KN
	$\alpha_{q2} = 1{,}0$	-	$q_{2k} = 2{,}5$	2,5 KN/m^2
Lane Number 3	$\alpha_{Q3} = 1{,}0$	$Q_{2k} = 100$	-	100 KN
	$\alpha_{q3} = 1{,}0$	-	$q_{3k} = 2{,}5$	2,5 KN/m^2
Lane Number 4	-	-	-	0 KN
	$\alpha_{q4} = 1{,}0$	-	$q_{4k} = 2{,}5$	2,5 KN/m^2
Remaining area	-	-	-	0 KN
	$\alpha_{qr} = 1{,}0$	-	$q_{rk} = 2{,}5$	2,5 KN/m^2

LM1 is applied over the entire width of the bridge. The train load is placed on 3 lanes, and the position of the lanes is varied in the transverse direction. The uniformly distributed load is applied as a line load in the center lane. Figure 3.9 presents load model 1.

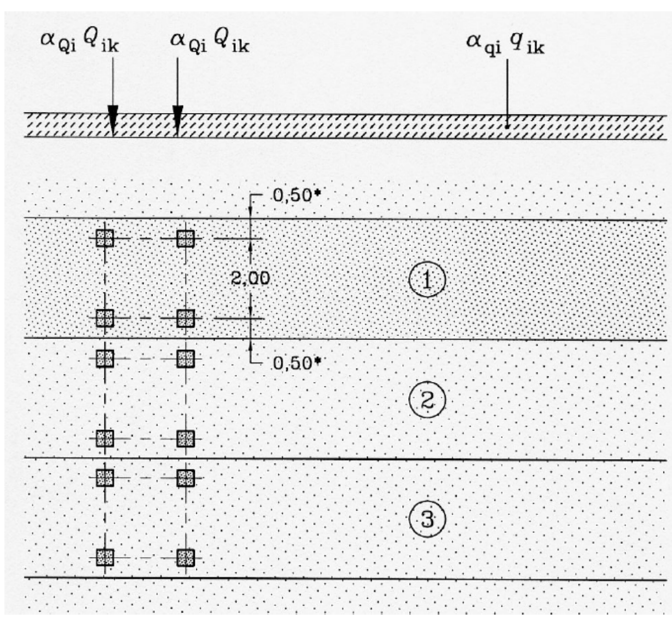


Figure 3.9: The details of Load Model 1. Figure is taken from EC1-2 (Standard Norge 2003b)

An example of the positioning of the train load is shown for bogie loads in Figure 3.10 and for uniformly distributed load in Figure 3.11. The load case number for LM1 in Sofistik is given in Table 3.15

Table 3.15: Load numbering for LM1 in Sofistik

Load Type	**Load Case in Sofistik**
Bogie Load	10000 till $\left(\frac{Bridge\,length}{2} + 10000\right)$
UDL	100 till 105

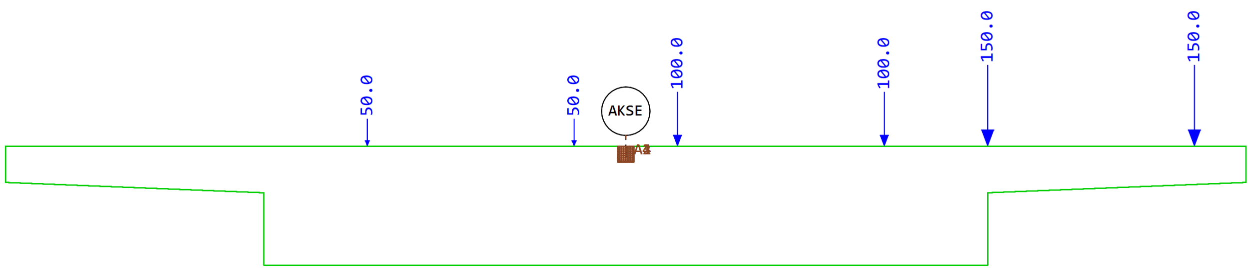


Figure 3.10: Bogie loads on the bridge are in [kN]

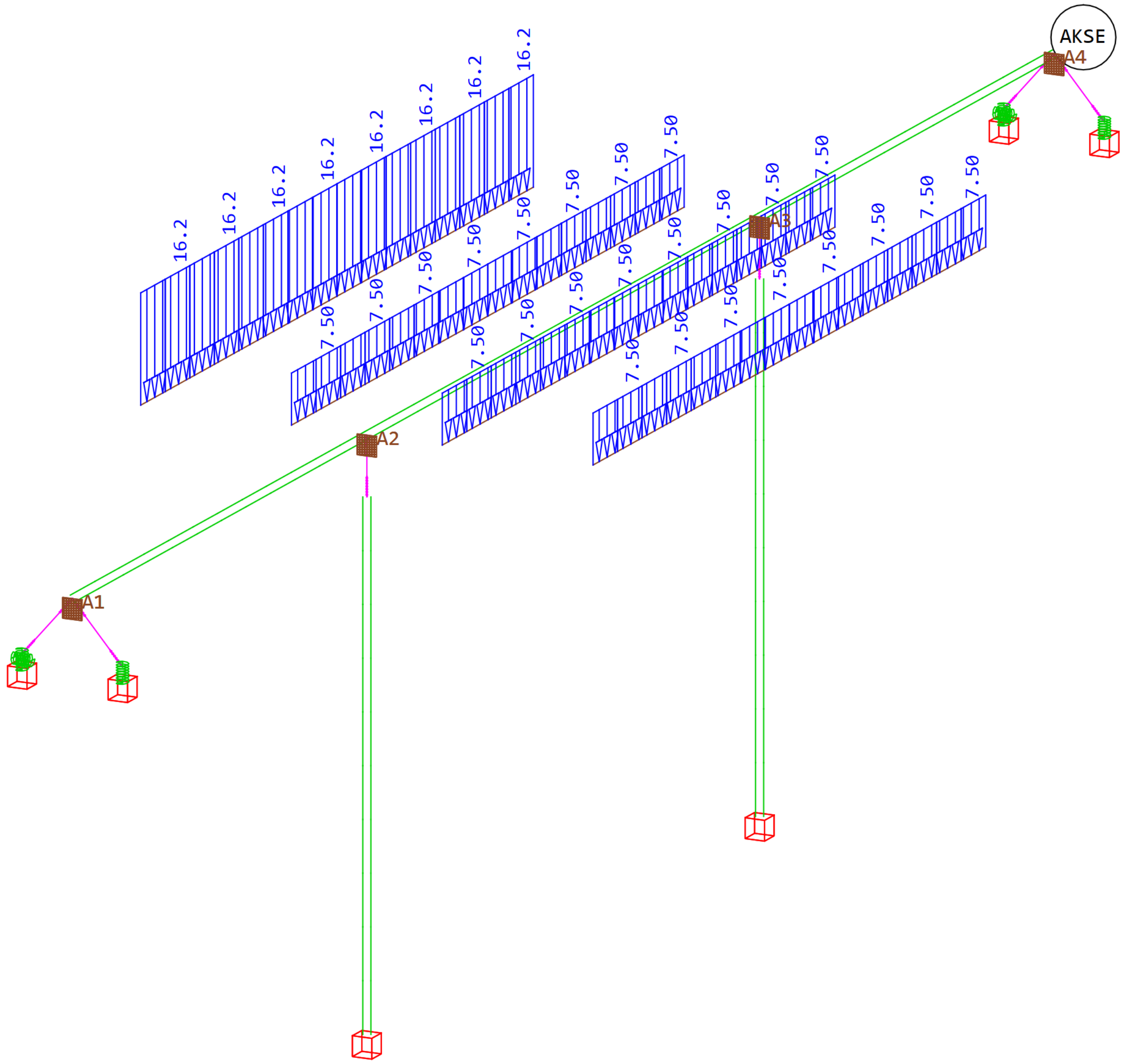


Figure 3.11: UDL on the bridge [kN/m]

3.5.2.1.2 Groups of Traffic Loads

The groups of traffic loads are determined according to table NA.4.4a in EC1-2 (Standard Norge 2003b) and presented in Table 3.16. It is noticeable that the footnote 'a' in the table appears to be inconsistent with the rest of the table's content. It states that horizontal forces should be included in load group gr1a, but the table does not list any horizontal forces for that same load group. It is established practice to follow the footnote and to include characteristic values of horizontal traffic loads in gr1a. The result of this is that gr2 is not relevant, and the assumption is conservative (Statens Vegvesen 2017).

Table 3.16: Groups of traffic loads

		KJØREBANE						GANGBANE OG FORTAU
Lastmodell		Vertikale krefter				Horisontale krefter		Bare vertikale krefter
Referanse		4.3.2	4.3.3	4.3.4	4.3.5	4.4.1	4.4.2	5.3.2-(1)
Lastsystem		LM1 (Boggilast og jevnt fordelt last)	LM2 (Enkel aksling)	LM3 (Spesial-kjøretøyer)	LM4 (Belastning i form av menneske-mengde)	Bremsekrefter og akselerasjons-krefter [a]	Sentrifugal-krefter og tverrkrefter [a]	Jevnt fordelt last
Lastgrupper	gr1a	Karakteristisk verdi						Kombinasjons-verdi [b]
	gr1b		Karakteristisk verdi					
	gr2	Ofte forekommende verdi				Karakteristisk verdi	Karakteristisk verdi	
	gr3 [d]							Karakteristisk verdi [c]
	gr4				Karakteristisk verdi			Karakteristisk verdi
	gr5	Se tillegg A		Karakteristisk verdi				
		Påvirkning fra dominerende komponent (betegnet som komponent som tilhører gruppen)						

[a] For gr1a brukes karakteristiske verdier for lastreferanse 4.4.1 (bremsekrefter og akselerasjonskrefter) og lastreferanse 4.4.2 (sentrifugalkrefter og tverrkrefter). Sentrifugalkrefter og bremsekrefter eller akselerasjonskrefter opptrer ikke samtidig i gr1a.

[b] For gr1a brukes kombinasjonsverdi for lastreferanse 5.3.2(1) (jevnt fordelt last på gangbane/fortau), dvs. 2,5 kN/m^2.

[c] Se 5.3.2.1(2) Ved tosidig gangbane/fortau regnes det ene belastet der det er ugunstigere enn at begge er belastet samtidig.

[d] Denne lastgruppen er ikke aktuell der gr4 benyttes.

As previously stated, Load Models 2, 3, and 4 are not relevant for design because they will never result in a larger bending moment than Load Model 1. For road bridges, gr1a is dimensioning group for global analysis, while gr1b is dimensioning group for local plate controls. gr1a becomes the relevant load group since the focus in this thesis is on the global analysis.

3.5.2.2 Wind Load

The wind load on the bridge deck is determined in accordance with Eurocode 1-4 actions on structures for wind actions (Standard Norge 2005a) and Handbook N400 section 5.4.3 (Statens Vegvesen 2015). The wind load is classified into three classes, referred to as Wind Classes 1-3, as outlined in Handbook N400 as follows:

- Wind class I: bridge structures with insignificant dynamic load from wind. Wind load class I encompasses all bridges where the highest natural frequency period is < 2 seconds.
- Wind class II: bridge structures with dynamic wind load that cannot be neglected. Wind load class II encompasses all bridge structures where one of the following is satisfied:

· the highest natural frequency period is ≥ 2 seconds and the span width is < 300m,

· the highest natural frequency period is < 2 seconds and the span width is ≥ 300m

- Wind class III: Bridge structures with pronounced dynamic wind load. Wind load class III encompasses all bridges where the following two conditions are satisfied:
  · the highest natural frequency period is ≥ 2 seconds,
  · the span width is ≥ 300m.

Plate bridges are categorized as Wind Class I in accordance with Handbook 400. The wind load is calculated in accordance with EC1 based on the mean wind velocity, V_m, and the velocity pressure q_p. The wind velocity should be determined from the basic wind velocity,V_b, which depends on the wind climate and the height variation of the wind determined from the terrain roughness and orography (Standard Norge 2005a). The location selected for the bridge in this research is Oslo. According to table NA.4(901) in EC1-4 the fundamental value of the basic wind velocity,$V_{b,0}$, is set to 22 m/s.

The basic wind velocity V_b is given by the following equation:

$$V_b = C_{dir} \cdot C_{season} \cdot C_{alt} \cdot C_{prob} \cdot V_{b,0} \tag{3.20}$$

where,

C_{dir} is the fundamental value of the basic wind velocity = 1.0

C_{season} is the season factor = 1.0

C_{alt} is the level factor = 1.0

C_{prob} is the probability factor = 1.0

$V_{b,0}$ is the fundamental value of the basic wind velocity given in table NA.4(901)

The mean wind velocity, V_m, is calculated using the following equation:

$$V_m(z) = C_r(z) \cdot C_0(z) \cdot V_b \tag{3.21}$$

where,

C_r(z) is the roughness factor, given as C_r(z) = k_r x ln($\frac{z}{z_0}$), where k_r, z and z_0 are given in table NA.4.1

C_0(z) is the orography factor, taken as 1.0

Terrain category II is selected which is area with low vegetation and isolated obstacles (trees, buildings).

The turbulence intensity I_v(z) at height z is defined as the standard deviation σ_v of the turbulence divided by the mean wind velocity as following:

$$I_v(z) = \frac{\sigma_v}{V_m(z)} = \frac{k_l}{c_0(z) \cdot ln(\frac{z}{z_0})} \tag{3.22}$$

where, k$_l$ is the turbulence factor = 1.0

The peak velocity pressure q_p and the wind gust velocity V_p are found by the following equations, respectively:

$$q_p(z) = \left[1 + 2k_p I_v(z)\right] \cdot 0.5 \rho v_m^2(z) \tag{3.23}$$

$$V_p(z) = v_m(z) \cdot \sqrt{1 + 2k_p I_v(z)} \tag{3.24}$$

where,

k$_p$ is a peak factor = 3.5

ρ = is the air density = 1.25 kg/m^3

The wind velocity pressure on the bridge deck is calculated with and without traffic and parameterized using the Teddy programming language in Sofistik for each bridge model. The wind load calculations for Model 20-5 are included in Appendix A.

Table 3.17: Wind load in y-direction of some models (y-direction in Sofistik is the direction parallel to the deck width, perpendicular to the span)

Model No.	**Wind load without traffic [kN/m]**	**Wind load with traffic [kN/m]**
Model 8-1	1.129	2.42
Model 20-5	1.591	2.985
Model p32-2	1.730	3.248
Model p36-2	2.146	4.035

The wind load cases in Sofistik are given in Table 3.18:

Table 3.18: Wind load cases in Sofistik

Load Type	Load Case in Sofistik
+ Axial on the bridge (eccentric) / -Vertical (eccentric)	20
- Axial on the bridge (eccentric) / -Vertical (eccentric)	21
+ Along the bridge / - Vertical	22
- Along the bridge / - Vertical	23
+ Vertical	24

3.5.2.3 Snow Load

Snow load is the downward force on a bridge's deck by the weight of accumulated snow and ice. According to Handbook N400 (Statens Vegvesen 2015), snow load is not considered to occur simultaneously with traffic load on road bridges, ferry docks or pedestrian and bicycle bridges. Snow load is smaller than traffic load and is not intended to be combined with it. As a result, it is neglected in this thesis.

3.5.2.4 Temperature Load

Temperature changes occur periodically and depend on the structure's geographical location, mass, and orientation. These changes result in a type of load called temperature load, which causes expansion and contraction of materials and can lead to stresses on the structure that can change its shape, strength, and stability.
Temperature load shall be calculated based on Eurocode 1: Actions on structures, Part 1-5: General actions, Thermal actions (Standard Norge 2003a). EC1-5 divides the temperature distribution in structural components into four main parts, as also shown in Figure 3.12

- a) a uniform temperature component, ΔT_u
- b) a horizontal linearly varying temperature difference component, ΔT_{MY}
- c) a vertical linearly varying temperature difference component, ΔT_{MZ}

- d) a vertical non-linear temperature difference component,ΔT_E

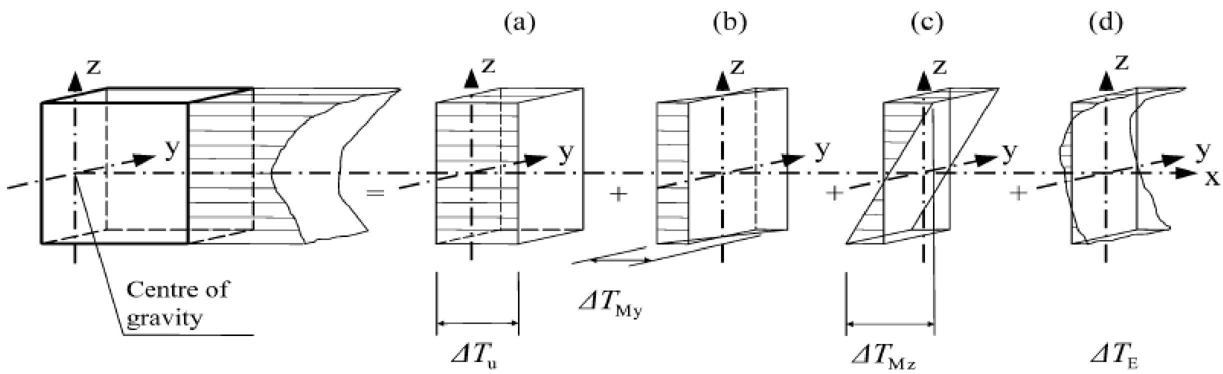


Figure 3.12: Temperature distribution in a structural component, Figure 3.12 is the same as figure 4.1 in EC1-5 (Standard Norge 2003a)

In most cases, it is common to take into account the effects of the uniform temperature component (a), the vertical linearly varying temperature difference component (c), and the vertical non-linear temperature difference component (d). According to EC1-5 Section 6.1.4.3, the horizontal temperature difference component should be considered when the orientation of the bridge results in one side being more exposed to sunlight than the other side (Standard Norge 2003a). However, it is assumed that this is not applicable in our case. Therefore, the horizontal temperature difference component ΔT_{MY} is neglected.

3.5.2.4.1 Uniform Temperature Component ΔT_u

The uniform temperature component of the bridge is based on its location and the minimum and maximum air temperatures, as indicated in Figures NA.A1 and NA.A2 of EC1-5 (Standard Norge 2003a). Changes in temperature result in expansions and contractions within the bridge, which generate forces in the supports. However, these forces do not have a significant effect on the deck plate, except in the case where the deck plate is fully restrained and leads to compressive forces and buckling.

The maximum and minimum temperature for Oslo according to Figures NA.A1 and NA.A2 are as following:

T_{max} = 36°C

T_{min} = -35°C

According to NA.6.1.2 in EC1-2 (Standard Norge 2003b), plate bridges are classified as type 3 bridge superstructure. Hence, the minimum and maximum uniform bridge temperature components $T_{e,min}$, $T_{e,max}$ are as following equations :

$$T_{e,max} = T_{max} - 3 = 33°C \tag{3.25}$$

$$T_{e,min} = T_{min} + 8 = -27°C \tag{3.26}$$

The characteristic value of the maximum contraction range $\Delta T_{N,con}$ and the maximum expansion range $\Delta T_{N,exp}$ of the uniform bridge temperature component should be taken as :

$$\Delta T_{N,con} = T_0 - T_{e,min} = -37°C \tag{3.27}$$

$$\Delta T_{N,exp} = T_{e,max} - T_0 = 23°C \tag{3.28}$$

Where T_0 = 10°C is the initial bridge temperature at the time that the structure is restrained.

3.5.2.4.2 Vertical Linearly Varying Temperature Difference Component ΔT_{MZ}

The component of the temperature difference that varies linearly vertically means that over a period of time, the heating and cooling of the top and bottom surfaces of a bridge deck will result in a maximum variation in temperature, which in turn leads to a vertical curvature of the bridge deck (Standard Norge 2003a). The recommended values for the linear temperature difference component $\Delta T_{M,heat}$ (for the top being warmer than the bottom) and $\Delta T_{M,cool}$ (for the bottom being warmer than the top) are obtained from Table NA.6.1 in EC1-5 (Standard Norge 2003a). These values are based on a paving thickness of 50 mm. However, in this thesis, the thickness of the asphalt is 100 mm. Hence, both $\Delta T_{M,heat}$ and $\Delta T_{M,cool}$ must be multiplied by factors $K_{sur,top}$ = 0.7 and $K_{sur,bottom}$ = 1, which are obtained from Table NA.6.2 in EC1-5 (Standard Norge 2003a).

The vertical linearly varying temperature difference component is as following:

Temperature component for the warmer top surface:

$$\Delta T_{M,heat} = K_{sur,top} \cdot T_{M,heat} = 10.5°C \tag{3.29}$$

Temperature component for the cooler top surface

$$\Delta T_{M,cool} = K_{sur,bottom} \cdot T_{M,cool} = 8.0°C \tag{3.30}$$

3.5.2.4.3 Combination of Temperature Loads

The combination of temperature loads is given in accordance with Clause 6.1.5 EC1-5 (Standard Norge 2003a). The following expression can be used:

$$\Delta T_{M,heat}(or \Delta T_{M,cool}) + \omega_N \Delta T_{N,exp}(or \Delta T_{N,con}) \tag{3.31}$$

or

$$\omega_M \Delta T_{M,heat}(or \Delta T_{M,cool}) + \Delta T_{N,exp}(or \Delta T_{N,con}) \tag{3.32}$$

where the most adverse effect should be chosen. The recommended values for ω_N and ω_M are:

$\omega_N = 0.35$

$\omega_M = 0.75$

For columns, the temperature difference $\Delta T_{M,S}$ is set to +/-5°C according to EC1-5 (Standard Norge 2003a), Section N.A.6.2.2. Bearings and expansion joints are checked according to EC1-5 (Standard Norge 2003a), Section N.A.6.1.3.3. It specifies the maximum expansion range of the uniform bridge temperature component $\Delta T_{N,exp}$+20°C, and the maximum contraction range of the uniform bridge temperature component $\Delta T_{N,con}$+20°C. Equations 3.31 and 3.32 result in 16 combinations of temperature loads as shown in Table 3.19:

Table 3.19: Temperature load combinations

Load Case in Sofistik	$\Delta T_{M,heat}$	$\Delta T_{M,cool}$	$\Delta T_{N,exp}$	$\Delta T_{N,con}$	$\Delta T_{M,S}$
80	10.5	-	8.1	-	5
81	7.9	-	23	-	5
82	10.5	-	-	-12.95	5
83	7.9	-	-	-37	5
84	-	8	8.1	-	5
85	-	6	23	-	5
86	-	8	-	-12.95	5
87	-	6	-	-37	5
88	10.5	-	8.1	-	-5
89	7.9	-	23	-	-5
90	10.5	-	-	-12.95	-5
91	7.9	-	-	-37	-5
92	-	8	8.1	-	-5
93	-	6	23	-	-5
94	-	8	-	-12.95	-5
95	-	6	-	-37	-5

3.5.2.5 Deformation Load

3.5.2.5.1 Creep and Shrinkage

The creep deformation of concrete, $\varepsilon_{cc}(\infty,t_0)$, is calculated according to equation 3.7 in Section 3.3.1.4. The total shrinkage strain, ε_{cs}, is found by equation 3.9 as it stated in Section 3.3.1.5. For calculations of creep and shrinkage, it is assumed that a relative humidity of 70% is used for the superstructure and a relative humidity of 80% is used for columns above water, as specified in Handbook N400 (Statens Vegvesen 2015), chapter 7.2.3. The decision was made to use a relative humidity of 70% for all construction elements in order to reduce the scope of the calculations.

Calculations for shrinkage and creep are performed in Sofistik for a period of 100 years, and are automatically updated for each bridge model. The load cases are shown in Table 3.20.

Table 3.20: Creep and Shrinkage load cases in Sofistik

Load Case	**Load Number in Sofistik**
Creep and Shrinkage of Substructure	5006
Creep and Shrinkage til 100 years	5030 till 5032

3.5.2.6 Accidental Load

Accidental loads are extreme loading conditions that a bridge may be subjected to. These loads may include impact loads from vehicles, earthquakes and ice. Impact loads from vehicles are considered one type of accidental load. These loads can occur on various bridge components, such as guardrails, the substructure, and the superstructure.

3.5.2.6.1 Impact Load on Guardrails

The impact load on guardrails is determined according to EC1-2 (Standard Norge 2003b), Chapter 4.7.3.3 and consists of a horizontal and a vertical force. The vertical load is equal to 75% of the horizontal load. The horizontal load acts 100 mm below the top of the guardrail or 1 m above the roadway/sidewalk, whichever value is lowest. The distribution length for both loads is 0.5 m for concrete guardrails and two guardrail posts for steel guardrails. The horizontal load value depends on the type of guardrail and is determined according to EC1-2 (Standard Norge 2003b), Table N.A. 4.9(n) and the Handbook V161 (Statens Vegvesen 2016), Chapter 3.4.5 as following :

- Impact load on concrete guardrails, horizontal: Q_{Ar} = 200 kN
- Impact load on concrete guardrails, vertical (if unfavorable): $Q_{Ar,v}$ = 150 kN
- Impact load on steel guardrails, horizontal: Q_{Ar} = 100 kN
- Impact load on steel guardrails, vertical (if unfavorable): $Q_{Ar,v}$ = 75 kN

These loads are lower than other variable loads in the ultimate limit state when considering global effects. Vehicle impact on the bridge is therefore omitted in the global analysis.

3.5.2.6.2 Impact load on the superstructure

In this thesis, the focus is only on impact loads on the superstructure as these have the greatest significance for the dimensioning of the bridge deck.

The impact load is determined according to EC1-7 (Standard Norge 2008), Table NA.4.2, Section 4.3.2 based on speed limits. The highest category of traffic, with a speed limit equal to or greater than 80 km/h, results in an equivalent static design force of 500 KN.

According to the Handbook N100 (Statens Vegvesen 2022), the minimum clear height for bridges over roads or bridges with overhead support systems must be at least 4.90m. In this thesis, the clear height, h, of the bridge is set to 10 m. The value of the clearance between the road surface and the underside of the bridge deck is defined as h_1, which is recommended to be 6.0 m according to EC1-7 (Standard Norge 2008). If h_1 is greater than h, the reduction factor, r_F, is equal to 0, meaning that the impact load on the superstructure is neglected.

3.5.2.7 Seismic Load

Earthquakes can occur suddenly and at any time, especially when tectonic plates collide, resulting in wave-like movements in the Earth's crust. The severity of the impact depends on soil conditions and the type of construction. Structures should be checked for the influence of seismic loads. However, only a few cases will have an impact on the structure's design. According to the Handbook N400 (Statens Vegvesen 2015), seismic impact is considered as an abnormal natural load. The reference peak ground acceleration ,α_{gR} m/s^2, with a return period of 475 years is characterized in Eurocode 8 ,EC8-1, (Standard Norge 2004b), Table NA.3.2 (901 - 911).

The reference peak ground acceleration,α_{gR}, for Oslo is equal to 0.30 m/s^2.

In this thesis, seismic class II is selected for the bridge models according to Eurocode 8, EC8-2, (Standard Norge 2014) Table NA.2(901). As a result, the seismic factor, γ_1, is equal to 1.0 as given in Table NA.2(903) in EC8-2 (Standard Norge 2014). The design ground acceleration can then be determined according to the following equation:

$$a_g = \gamma_1 \, x \, \alpha_{gR} = 0.30 m/s^2 \tag{3.33}$$

The bridge models are founded on gravel fill in all axes, which corresponds to ground type B according to Eurocode 8-1 (Standard Norge 2004b), Table NA.3.1. As a result, the amplification factor $S = 1.3$, Table 3.3 in EC8-1 (Standard Norge 2004b). The behaviour factor, q equal to 1.5, is determined from EC8-1, Table NA6.1 (Standard Norge 2004b). Additionally, the models will be designed in accordance with provisions that apply to low seismicity according to NA.3.2.1(4), EC8-1 (Standard Norge 2004b) after the following equation:

$$\alpha_g \, x \, S = 0.39 m/s^2 \; < \; 0.1g = 9.81 m/s^2 \tag{3.34}$$

No seismic assessment is required for structures in seismic class II if α_g S < 0.49 m/s^2 or S$_d$ < 0.49m/s^2 and with a behaviour factor,q, ≥ 1.5. As a result, seismic load calculations are not required for our design since it meets the specified criteria for seismic class II.

3.6 Loss of Prestressing Force

According to EC2-1, section 5.10.2.1 (1) (Standard Norge 2004a), the maximum stressing force applied to tendon P$_{max}$ shall not exceed the following value :

$$P_{max} = A_p \cdot \sigma_{p,max} \tag{3.35}$$

where, A$_p$ is the cross-sectional area of the tendon

$\sigma_{p,max}$ is the maximum stress applied to the tendon = min ($k_1 \cdot f_{pk}$; $k_2 \cdot f_{p0,1k}$)

f$_{pk}$ is the characteristic tensile strength of prestressing steel

f$_{p0,1k}$ is the 0.1% proof stress of prestressing steel

k$_1$ is a factor to be set equal to 0.8

k$_2$ is a factor to be set equal to 0.9

The prestressing force along the tendon will over time experience force reduction due to friction, anchoring, and elastic shortening. These types of losses can be categorized into two groups: immediate losses and time-dependent losses.

3.6.1 Immediate Loss of Prestressing

According to EC2-1, section 5.10.3 (Standard Norge 2004a), the immediate losses of the prestressing force at transfer (immediately after jacking) $\Delta P_i(x)$ are defined by the difference between the maximum imposed force P_{max} at the active end and the value of the initial prestress force $P_{m0}(x)$ that appears immediately after tensioning or transfer at a distance of x, as shown in the following equation:

$$\Delta P_i(x) = P_{max} - P_{m0}(x) \tag{3.36}$$

where the initial prestressing force $P_{m0}(x)$ is found as following:

$$P_{m0}(x) = A_p \cdot \sigma_{pm0}(x) \tag{3.37}$$

where σ_{pm0} is the immediate stress in tendon after tensioning calculated as following:

$$\sigma_{pm0}(x) = min\left(0.75 f_{pk}\,;\, 0.85 f_{p0.1k}\right) \tag{3.38}$$

According to EC2-1, section 5.10.3 (3) (Standard Norge 2004a), the following immediate influences should be considered, where relevant, when determining the immediate losses $\Delta P_i(x)$:

- losses due to elastic deformation of concrete ΔP_{el}
- losses due to short term relaxation ΔP_r
- losses due to friction ΔP_μ(x)
- losses due to anchorage slip ΔP_{sl}

3.6.1.1 Elastic Shortening Losses

According to Section 5.10.5.1 of Eurocode 2-1 (Standard Norge 2004a), the deformation in concrete caused by instantaneous elastic shortening results in a loss of tendon force, denoted as ΔP_{el}. This loss is assumed to be the mean loss in each tendon, and can be calculated as follows:

$$\Delta P_{el} = A_p \cdot E_p \Sigma \left[\frac{j \cdot \Delta\sigma_c(t)}{E_{cm}(t)}\right] \tag{3.39}$$

where:

$\Delta\sigma_c(t)$ is the variation of stress at the center of the gravity of the tendons applied at time t.

j is a coefficient equal to $\frac{(n-1)}{2n}$ where n is the number of identical tendons successively prestressed. As an approximation j may be taken as $1/2$

E_p is the design value of modulus of elasticity of prestressing steel

$E_{cm}(t)$ is the elastic modulus of concrete.

3.6.1.2 Losses Due to Friction

The loss of prestress due to friction between the duct and cable is described in Section 5.10.5.2 of Eurocode 2-1 (Standard Norge 2004a). When the tendons are being stressed, their surface area comes into contact with the duct, resulting in friction between the strands and the duct. This friction leads to a loss of stress ΔP_μ(x) found as follows:

$$\Delta P_\mu(x) = P_{max}\left(1 - e^{-\mu(\theta + kx)}\right) \tag{3.40}$$

where,

θ is the sum of the angular displacements over a distance x

μ is the coefficient of fiction between the tendon and its duct. μ is set equal to 0.18

k is an unintentional angular displacement for internal tendons (per unit length). k is set equal to 0.005

x is the distance along the tendon from the point where the prestressing force is equal to P_{max}

3.6.1.3 Losses at Anchorage

During the prestressing process, tendons are jacked and then released, causing a force to be applied to the concrete through the anchor. However, during this process, there may be a slip or draw-in of the strands, resulting in a loss of prestress. This loss is known as the anchorage loss, and is described in Eurocode 2-1, section 5.10.5.3 (Standard Norge 2004a). The anchorage loss can be quantified using the following equation:

$$\Delta P_s = 2\beta L_d \tag{3.41}$$

$$L_d = \sqrt{\frac{\Delta_s E_p A_p}{\beta}} \tag{3.42}$$

where,

L_d is the draw-in length

β is the slope of the friction loss line

E_p is modulus of elasticity of steel

A_p is cross-sectional area of steel

Δ_s is the anchorage slip

3.6.2 Time Dependent Losses

Both post-tensioned and pre-tensioned members experience time-dependent losses due to two reasons: the long-term shortening of tensioned cable caused by creep and shrinkage in concrete, and the reduction of stress in steel due to relaxation. To accurately calculate time-dependent losses in a structure, factors such as concrete creep and shrinkage, and steel relaxation must be taken into account. These losses can be determined using a simplified method given in EC2-1, section 5.10.6 (Standard Norge 2004a). The estimated losses can be calculated using the following equation:

$$\Delta P_{c+s+r} = A_p \Delta\sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(t,t_0)\cdot\sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}}\frac{A_p}{A_c}\left(1 + \frac{A_c}{I_c} Z_{cp}^2\right)\left[1 + 0.8\varphi(t,t_0)\right]} \tag{3.43}$$

where,

ΔP_{c+s+r} is the absolute value of the variation of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t

ε_{cs} is the estimated shrinkage strain

E$_p$ is the modulus of elasticity for the prestressing steel

E$_{cm}$ is the modulus of elasticity for the concrete

$\Delta\sigma_{pr}$ is the absolute value of the variation of stress in the tendons at location x, at time t, due to the relaxation of the prestressing steel.

$\varphi(t,t_0)$ is the creep coefficient at a time t and load application at time t$_0$

$\sigma_{c,QP}$ is the stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant.

A_p is the area of all the prestressing tendons at the location x

A_c is the area of the concrete section.

I_c is the second moment of area of the concrete section.

Z_{cp} is the distance between the centre of gravity of the concrete section and the tendons.

3.7 Design Limit States

EC0 (Standard Norge 2002) provides the basis for the load combinations. Loads on structure must not exceeded the structure capacity to ensure safety of the structure at any design situation. There are three types of the design limit states:

- Ultimate Limit State, ULS.
- Serviceability Limit State, SLS.
- Accidental Limit State, ALS.

3.7.1 Ultimate Limit State - ULS

It is a requirement that the structure must withstand the maximum loading without collapsing, including safety factors to ensure the safety of the structure and the safety of people. In the ultimate limit state, the capacity of the structure is calculated and the structure is checked for failure. The ULS load is compared to the design strength, and the design is considered safe if the design strength is greater than the ULS load. The ultimate limit state divides into three basic situations and should be considered where it is relevant:

- Control of static equilibrium, EQU (Set A).
- Control of failure in the structure, STR/GEO (Set B)
- Control of failure in the ground, STR/GEO (Set C)

In this thesis, only load combinations in STR (Set B) are relevant for capacity checks of the superstructure. EC0 (Standard Norge 2002) provides Table NA.A2.4(B), presented as Table 3.21, as a reference for the design values of actions for Set B, which includes two equations (6.10a and 6.10b) that are particularly useful for construction design. The evaluation of the construction's capacity to withstand failure in this load combination, which results in the largest forces on the superstructure, making it the decisive factor for the design of the bridge deck.

Table 3.21: Design values of actions (STR/GEO) Set B

Persistent and transient design situations	**Permanent actions**		**Prestress**	**Leading variable action (*)**	**Accompanying variable actions (*)**
	Unfavourable	**Favourable**			
(Eq. 6.10 a)	$\gamma_{G,j,sup}\ G_{k,j,sup}$	$\gamma_{G,j,inf}\ G_{k,j,inf}$	$\gamma_P\ P$	$\gamma_{Q,1}\ \psi_{0,1}\ Q_{k,1}$	$\gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$
(Eq. 6.10 b)	$\xi\ \gamma_{G,j,sup}\ G_{k,j,sup}$	$\gamma_{G,j,inf}\ G_{k,j,inf}$	$\gamma_P\ P$	$\gamma_{Q,1}\ Q_{k,1}$	$\gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$

(*) Variable actions are those considered in Tables NA.A2.1 to NA.A2.3.

NOTE 1 For the ultimate limit state it shall be verified using Equations 6.10a and 6.10b that the the structural behavior of bridges is in accordance with the assumed calculation model, other than minor changes (e.g. bearing uplift, disregarded tensile piles, plastic hinges, etc.) which is acceptable provided that the structure is designed in accordance with the changed conditions.

NOTE 2 The following set of γ and ξ values shall be used in Equations 6.10a and 6.10b:

$\gamma_{G,sup}$	=	1,35	for permanent actions, except those listed below [1) 2)]
		1,00	for irreversible imposed loads [3)]
		1,20	for permanent part of water pressure
$\gamma_{G,inf}$	=	1,0	for permanent actions [1) 2)]
		0	for irreversible deformation loads [3)]
		1,0	for water pressure
ξ	=	0,89	for selfweight [1)]
γ_P	=	value defined in the relevant Eurocode [4)]	
γ_Q	=	1,35	for road and pedestrian traffic, when unfavourable (0 when favourable)
γ_Q	=	1,5	for railway traffic, when unfavourable (0 when favourable). γ_Q = 1,2 shall be used for load model SW/2
γ_Q	=	1,60	for actions from wind, waves and currents, when unfavourable (0 when favourable)
γ_Q	=	1,20	for thermal actions, when unfavourable (0 when favourable)
γ_Q	=	1,35	for actions from bearing friction and variable part of water pressure, when unfavourable (0 when favourable)
γ_Q	=	1,50	for other variable actions, when unfavourable (0 when favourable)

1) These values applies for self-weight of structural and non-structural elements, ballast, soil, removable loads, etc.

2) The characteristic values of actions from one source, e.g. self-weight, are multiplied by $\gamma_{G,sup}$ when the resulting total load effect is unfavourable, and by $\gamma_{G,inf}$ when the resulting total load effect is favourable. This also applies when different materials are involved.

3) Irreversible imposed loads can among others be uneven settlements, creep and shrinkage. Reference is made to NS-EN 1992-1-1 for γ values for shrinkage. See also NS-EN 1991 to NS-EN 1999 for any other γ values to be used for applied deformations. Irreversible imposed loads shall always be taken into account when the effects are unfavourable.

4) Where relevant, the values are applicable also for indirect effects of prestressing, i.e. as constraining forces in statically indeterminate structures.

NOTE 3 See above footnote 2. See also A2.3.1(2).

NOTE 4 For particular verifications, the values for γ_G and γ_Q may be subdivided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} in the range 1,0 – 1,15 may normally be used.

More detailed rules may be given for the individual project.

NOTE 5 For actions from water not covered in this table or by NS-EN 1997 (e.g. running water), may the individual project specify the load combinations to be used.

3.7.2 Serviceability Limit State - SLS

Serviceability limit state (SLS) that represent criteria governing normal functional or operational use (Paik and Thayamballi 2007). It also represent human comfort and the appearance of the construction in terms of factors such as deflection, vibration, and durability. During the serviceability limit state, the EC0 (Standard Norge 2002), section 6.5.1, demonstrate that the governing value for the actions for the relevant serviceability criteria is less than or equal to the governing limit value for the relevant serviceability criteria. EC0 refers to deformation, stress and strain limitation and crack width as serviceability criteria.

According to EC0 (Standard Norge 2002), the combinations of action for serviceability limit states are defined by the following:

- a) Characteristic combination
- b) Frequent combination
- c) Quasi-permanent combination

According to the Handbook N400 (Statens Vegvesen 2015), section 7.7.1, crack width shall be checked during the serviceability limit state, which is a combination that occurs frequently and is quasi-permanent. Table NA.A2.6 given in EC0 (Standard Norge 2002), presented as Table 3.22 shows the different load combinations for serviceability limit state. This thesis assesses the crack width and deflection of the bridge deck for Quasi-permanent combination.

Table 3.22: Design values of actions in the serviceability limit state

Combination	Permanent actions G_d		Pre-stress	Variable actions Q_d	
	Unfavour-able	Favourable		Leading action	Other actions
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Infrequent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{1,infq}Q_{k,1}$	$\psi_{1,i}Q_{k,i}$
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

3.7.3 Accidental Limit State - ALS

Accidental limit states represent situations in which a structure may be subjected to accidental events, such as earthquakes, explosions, or impact loads. The limit states define the minimum safety requirements for the structure during these events. However, even if these events are short-lived, they can result in significant impacts. Accidental and seismic combinations are design situations for the Accidental Limit State.
The Accidental loads (Impact Load on Guardrails and Impact load on the superstructure) discussed in section 3.5.2.6 and the Seismic load presented in section 3.5.2.7 are neglected in this thesis.

3.8 Combination Factors and Load Factors

3.8.1 Combination Factor ψ

The ψ values for road bridges and similar structures are determined according to EC0 (Standard Norge 2002), Table NA.A2.1 which is presented as Table 3.23. It is important to note that ψ is a reduction factor that is only applicable to variable loads and takes into account the reduced probability that variable loads occur with their maximum value simultaneously. Whereas, ξ is a reduction factor that only applicable to permanent loads for cases in which the permanent loads act favorably. There are three types of combination factors:

- a) ψ_0 - combination value
- b) ψ_1 - frequently value
- c) ψ_2 - quasi-permanent value

Table 3.23: Values of ψ factors for road bridges

Action	Symbol		ψ_0	ψ_1	ψ_2 5)	$\psi_{1,infq}$ 1)
Traffic loads (see EN 1991-2, Table 4.4)	gr1a (LM1 + horizontal loads + pedestrian or cycle-track loads)	Tandem system (TS)	0,7 3)	0,7	0,2/0,5	0,8
		Uniformly distributed load (UDL)	0,7 3)	0,7	0,2/0,5	0,8
		Horizontal loads	0,7 3)	0,7	0,2/0,5	0,8
		Pedestrian+cycle-track loads 2)	0,7 3)	0,7	0,2/0,5	0,8
	gr1b (single axle)		0,7 3)	0,7	0,2/0,5	0,8
	gr2 (horizontal forces)		0,7 3)	0,7	0,2/0,5	0,8
	gr3 (pedestrian loads)		0,7 3)	0,7	0,2/0,5	0,8
	gr4 (LM4 – crowd loading))		0,7 3)	0,7	0,2/0,5	0,8
	gr5 (LM3 – special vehicles))		-	-	-	-
Wind forces	F_{wk} - Persistent design situations		0,7	0,6	0/0,5	0,8
	F_{wk} – During execution		0,8	-	-	-
	F_w^*		0,7	0,6	0/0,5	0,8
Thermal actions 4)	T_k		0,7	0,6	0/0,5	0,8
Snow loads	$Q_{Sn,k}$ - on roofs, etc.		0,7	0,6	0,2/0,5	0,8
	$Q_{Sn,k}$ - during execution		0,8	-	-	-
Construction loads	Q_c		1,0	-	1,0	-
Ice pressure	-		0,7	0,6	0/0,5	0,8
Waves and currents	-		0,7	0,6	0/0,5	0,8
Water pressure, variable part	-		0,7	0,6	0/0,5	0,8
Earth pressure, variable part	-		0,7	0,6	0/0,5	0,8
Friction actions from bearings	-		0,7	0,6	0/0,5	0,8
Other variable actions	-		0,7	0,6	0/0,5	0,8

1) $\psi_{1,infq}$ is a factor for defining infrequent values of actions

2) The combination value of the pedestrian and cycle-track load, mentioned in Table 4.4a of EN 1991-2, is a "reduced" value, and ψ factors are applicable to this value.

3) Where wind loads are regarded as leading variable action represented by F_{wk}, should the value of ψ_0 for traffic loads be set equal to 0, see also A2.2.2(5).

4) Temperature actions shall be taken into account in all load combinations, also in ultimate limit states, if the effects are unfavourable.

5) When calculating crack widths according to NS-EN 1992 for load combination "quasi-permanent", the value 0,5 shall be used. The values 0,2 and 0 may be used, respectively, when calculating the long-term effects of time-dependent properties.

3.8.2 Partial Factors for Actions (ULS)

Partial factors for actions, load factors or safety factors, are used in the ultimate limit state design approach to ensure that the structure can withstand the loads it is exposed to during its design life with a sufficient level of safety. In the ULS design approach, the loads acting on the structure are multiplied by a load factor to obtain the ULS load. In the same manner, the strength of the structural elements is multiplied by a resistance factor to obtain the design strength. The associated load factors are determined according to EC0 (Standard Norge 2002), Table NA.A2.4(B) and presented in Table 3.21.

3.8.3 Partial Safety Factor γ

The partial safety factor γ for limit states is as presented in Section 3.3.4 and Table 3.7.

3.9 Load Combinations

Load combinations are used to check the strength and stability of a structure under different loading scenarios. For each limit state, different load combinations are defined based on the types of loads that the structure may experience, the load factors to be applied to each load, and the criteria used to determine the maximum expected loading on the structure. The load combinations for the ULS are designed to ensure that the structure can resist the most loading conditions that it is exposed to. While, the load combinations for the SLS, are designed to ensure that the structure can meet the requirement performance criteria under normal operating conditions. The design load value becomes the most unfavorable combination of loads with their associated load factors.

3.9.1 Load Combinations at ULS

EQU static equilibrium for structures should be verified using the design values of action in the following equation:

$$\sum_{j\geq 1} \gamma_{Gj} G_{kj} + \gamma_p P_k + \gamma_{Q1} Q_{k1} + \sum_{i>1} \gamma_{Qi} \psi_{0i} Q_{ki} \ (6.10) \tag{3.44}$$

STR/GEO the design values of actions in the persistent and transient design situations (the relevant design combination for the bridge deck):

$$\sum_{j\geq 1}\gamma_{Gj}G_{kj} + \gamma_p P_k + \sum_{i>1}\gamma_{Qi}\psi_{0i}Q_{ki} \ (6.10a) \tag{3.45}$$

$$\sum_{j\geq 1}\xi_j\gamma_{Gj}G_{kj} + \gamma_p P_k + \gamma_{Q1}Q_{k1} + \sum_{i>1}\gamma_{Qi}\psi_{0i}Q_{ki} \ (6.10b) \tag{3.46}$$

Accidental situation:

$$\sum_{j\geq 1}G_{kj} + P_k + A_d + (\psi_{11}\ or\ \psi_{21})Q_{k1} + \sum_{i>1}\psi_{2i}Q_{ki} \ (6.11b) \tag{3.47}$$

Seismic situation:

$$\sum_{j\geq 1}G_{kj} + P_k + A_{Ed} + \sum_{i>1}\psi_{2i}Q_{ki} \ (6.12b) \tag{3.48}$$

SOFiSTiK FEM software for structural engineers uses the following equation of the superposition of actions from eurocode, resulting load cases type ULS fundamental combination

$$E_d = E\left\{\sum_{j\geq 1}\gamma_{Gj}G_{kj} + \gamma_p P_k + \gamma_{Q1}Q_{k1} + \sum_{i>1}\gamma_{Qi}\psi_{0i}Q_{ki}\right\} \tag{3.49}$$

3.9.2 Load Combinations at SLS

Characteristic combination:

$$\sum_{j\geq 1}G_{kj} + P_k + Q_{k1} + \sum_{i>1}\psi_{0i}Q_{ki} \ (6.14) \tag{3.50}$$

Sofistik uses the following equation for characteristic combination:

$$E_{d,rare} = E\left\{\sum_{j\geq 1}G_{kj} + P_k + Q_{k1} + \sum_{i>1}\psi_{0i}Q_{ki}\right\} \tag{3.51}$$

Frequent combination :

$$\sum_{j\geq 1}G_{kj} + P_k + \psi_{11}Q_{k1} + \sum_{i>1}\psi_{2i}Q_{ki} \ (6.15) \tag{3.52}$$

Sofistik uses the following equation for frequent combination:

$$E_{d,frequ} = E\left\{\sum_{j\geq 1} G_{kj} + P_k + \psi_{1,1}Q_{k1} + \sum_{i>1}\psi_{2i}Q_{ki}\right\} \tag{3.53}$$

Quasi-permanent combination :

$$\sum_{j\geq 1} G_{kj} + P_k + \sum_{i\geq 1}\psi_{2i}Q_{ki} \ \ (6.16) \tag{3.54}$$

Sofistik uses the following equation for quasi-permanent combination:

$$E_{d,frequ} = E\left\{\sum_{j\geq 1} G_{kj} + P_k + \psi_{1,1}Q_{k1} + \sum_{i>1}\psi_{2i}Q_{ki}\right\} \tag{3.55}$$

The same equation as for frequently occurring combinations is used, but with two values per ψ_2. In the combination, the value of ψ_1 is directly overwritten with the ψ_2 value, which deviates from the values defined in the table. Additionally, a new value of 0.5 for ψ_1 is used to replace the values for temperature loads and traffic loads.

3.9.3 Load Combinations at ALS

Load combinations and associated load factors are determined according to EC0 (Standard Norge 2002), Table NA.A2.5. This does not apply to the global model of our design.

3.9.4 Load Groups and Load Combinations in Sofistik

In Sofistik's calculation model, the loads are categorized into different groups or "Actions" that are used in load combinations. The specific groups used in the model are shown in Table 3.24

Table 3.24: Load groups in Sofistik

Type	Part	Designation
C_1	G	C+S till traffic opening
C_2	G	C+S after traffic opening
G_1	G	dead load g_1
G_2	G	dead load g_2
ZW	Q	wind load
C	P	creep + shrinkage
P	P	prestress
GR_1	Q	load group gr1a
GR_2	Q	load group gr1a
T	Q	temperature load
Y_1	Q	rare without gpc
Y_3	Q	freq. without gpc
Y_4	Q	prem. without gpc
Y_D	Q	(B) desi. 6.10b wihtout gpc
Y_E	Q	earq. wihtout gpc
Y_H	Q	(B) desi. 6.10a wihout gpc

Load cases generated in force combinations are shown in Table 3.25. For each series, the results are set up as presented in Table 3.26.

Table 3.25: Load cases generated in force combinations in Sofistik

Series	Load Case
700-series	LM1 traffic
1300-series	SLS quasi-permanent
1400-series	SLS frequent
1500-series	SLS characteristic
2200-series	ULS set (B) 6.10 a
2300-series	ULS set (B) 6.10 b
3500-series	SLS characteristic without self-weight, prestressed, creep and shrinkage
3550-series	SLS frequent without self-weight, prestressed, creep and shrinkage
3600-series	SLS quasi-permanent without self-weight, prestressed, creep and shrinkage
3650-series	ULS set (B) 6.10b without self-weight, prestressed, creep and shrinkage
3700-series	ULS set (B) 6.10a without self-weight, prestressed, creep and shrinkage

Table 3.26: Result cases in Sofistik

Force Type	Result Case in Sofistik
Max. Moment My	XX01
Min. Moment My	XX02
Max. Shear Vz	XX03
Min. Shear Vz	XX04
Max. Torsion Mt	XX05
Min. Torsion Mt	XX06
Max. Axial force N	XX07
Min. Axial force N	XX08
Max. Shear Vy	XX09
Min. Shear Vy	XX10
Max. Moment Mz	XX11
Min. Moment Mz	XX12

This system means that the maximum bending moment in SLS quasi-permanent will have a load case of LC1301.
For the design of bridge elements, the AQB module in Sofistik is used. In this module, combination results are used from the combination without self-weight, restraint, creep, and shrinkage. Self-weight, restraint, creep, and shrinkage are obtained from calculations done in the CSM module in Sofistik. This generates new force result. The following results are presented in Table 3.27

Table 3.27: New force results in Sofisitk

Series	**Load Case**	
	Reinforced Concrete	**Prestressed Concrete**
8200-series	ULS set (B) 6.10a	ULS set (B) 6.10a
8300-series	ULS set (B) 6.10b	ULS set (B) 6.10b
9100-series	SLS quasi-permanent, design for cracks in columns 0.39mm	SLS quasi-permanent, design for cracks in columns 0.39mm
9200-series	SLS frequent, design for cracks in superstructure 0.39mm	SLS frequent, design for cracks in superstructure 0.26mm
9300-series	SLS characteristic, for checking stresses in concrete and reinforcement	SLS characteristic, for checking stresses in concrete and reinforcement
9400-series	-	SLS quasi-permanent check for pressure relief around the prestressed cable

For each series, the results are set up from the AQB module as presented in Table 3.28

Table 3.28: Result cases from AQB mdoule - Sofistik

Force Type	**Result Case in Sofistik**
Max. Moment My at opening	XX01
Min. Moment My at opening	XX02
Max. Shear Vz at opening	XX03
Min. Shear Vz at opening	XX04
Max. Torsion Mt at opening	XX05
Min. Torsion Mt at opening	XX06
Max. Axial force N at opening	XX07
Min. Axial force N at opening	XX08
Max. Shear Vy at opening	XX09
Min. Shear Vy at opening	XX10
Max. Moment Mz at opening	XX11
Min. Moment Mz at opening	XX12
Max. Moment My at 100 years	XX13
Min. Moment My at 100 years	XX14
Max. Shear Vz at 100 years	XX15
Min. Shear Vz at 100 years	XX16
Max. Torsion Mt at 100 years	XX17
Min. Torsion Mt at 100 years	XX18
Max. Axial force N at 100 years	XX19
Min. Axial force N at 100 years	XX20
Max. Shear Vy at 100 years	XX21
Min. Shear Vy at 100 years	XX22
Max. Moment Mz at 100 years	XX23
Min. Moment Mz at 100 years	XX24

Chapter 4

Modelling

4.1 Modelling in Sofistik

SOFiSTiK program is a powerful finite element analysis software used in civil and structural engineering. It provides advanced capabilities for modeling and analyzing complex structures and is widely used in the design of buildings, bridges, tunnels, and other large-scale projects. SOFiSTiK offers a range of features for modeling and analyzing structures, including advanced geometry modeling tools, meshing capabilities for generating high-quality finite element meshes, material and section libraries for a wide range of materials, and advanced analysis tools for linear and nonlinear analysis, dynamic analysis, and more. It also has comprehensive post-processing capabilities for visualizing and interpreting analysis results and integration with other design software such as AutoCAD, Revit, and Rhino. Cloud-based computing options are also available for large-scale simulations (Sofistik AG 2023).

4.1.1 General Workflow and Application

Sofistik is a modular-based software that enables different modules to communicate back and forth with the database, as depicted in Figure 4.1. The software comprises three distinct module types: pre-processing types, processing types, and post-processing types, which operate behind various Sofistik programs, including Sofistik Structural Desktop (SSD), Sofiplus (-X), System Visualization, Report Browser, Wingraf, and Result Viewer.

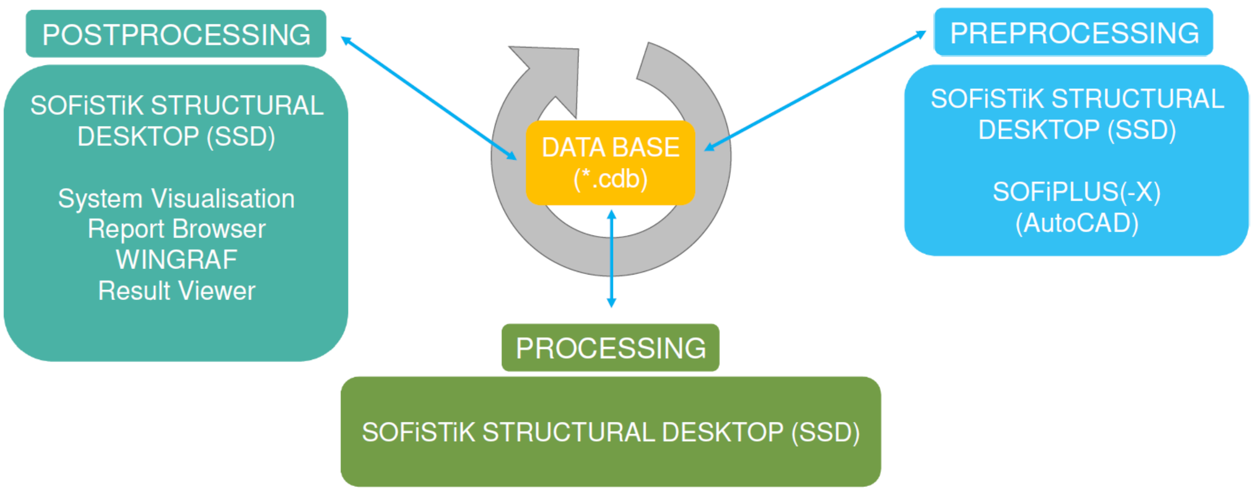


Figure 4.1: Schematic representation of Sofistik database, the figure is taken from (Sofistik AG 2020)

The general workflow in Sofistik, using the aforementioned modules, is presented in Figure 4.2. The pre-processing phase involves creating a structural model using the Text Editor "TEDDY" which utilizes its own programming language called the CADINP-command language. This process includes executing specific commands or functions within the Text Editor to construct the model, or using the Sofiplus (-X) program which is integrated into the AutoCad interface and allows the user to draw and define the geometric structures, loads, and support conditions in a CAD environment. During the pre-processing stage of the structural analysis, various tasks must be completed, including the selection of materials and cross-sections, defining the geometric system, prestressing system, loads, and construction stages.
The SSD with help of TEDDY task files applies the loads to the structure and analyzes the structure using the General Static Analysis of Finite Element Structures ASE model. The results of the analysis can be superimposed using, for example, the MAXIMA module in Sofistik. Following that, the design of the structural element can be carried out in the system using AQB module (Design of Cross Sections). Lastly, the results can be evaluated, and final documentation can be created.

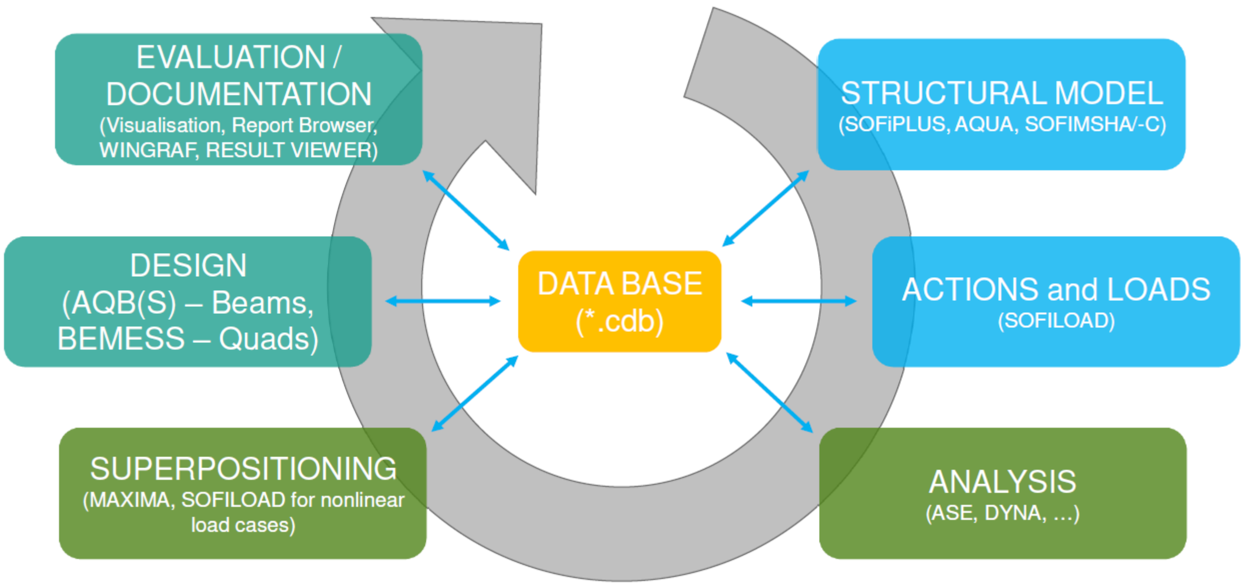


Figure 4.2: Schematic representation of Sofistik general workflow, the figure is taken from (Sofistik AG 2020)

Sofistik Structural Desktop provides its most important feature which is basically to communicate between the most important Sofistik programs such as Graphic, Result Viewer, Report browser and the Text Editor (TEDDY).

4.2 Modeling Overview

The purpose of the study is to investigate the effect of span-to-depth ratio on the behavior and design of reinforced and prestressed concrete bridges. The study aims to evaluate the impact of different span lengths and cross section heights on the structural response and efficiency of the bridge design. The findings of this study can provide valuable insights for bridge engineers and designers in selecting optimal span-to-depth ratios for concrete bridges.

For this study, several reinforced and prestressed concrete bridge models were created in Sofistik with different span-to-depth ratios. The bridge models vary in length, with span lengths ranging from 20m to 100m and cross-section heights of various sizes ranging from 0.45m to 2.5m. The minimum height of 0.45m was chosen to prevent the bridge deck from becoming nearly a flat plate, considering that the flange height is 0.35m refer to Figure 3.1. On the other hand, the maximum height of 2.5m was selected to assess the bridge's ability to withstand the applied load at the maximum chosen bridge length in this study. Each bridge model is composed of three spans, with a total length L and side spans with a span length of 0.3L, while the middle span has a span length of 0.4L. All of the bridge models have the same cross section width of 12m, but the cross section height varies between models to achieve different span-to-depth ratios. The purpose of varying the span-to-depth ratio in the models is to evaluate the impact of this parameter on the structural behavior and design of the bridges.

The most critical hogging bending moments that occur at the middle supports, specifically at axis 2 and 3 (refer to Figure 3.3), as well as the sagging bending moment in the mid-span, which will occur at the middle of the span, have been chosen to be focused on. In the design of the bridges, a steel reinforcement with a diameter of 32mm and a center-to-center spacing of 100mm has been used. This combination ensures the maximum reinforced steel area (as defined in this thesis) to withstand the critical bending moments, while complying with the highest allowable diameter and minimum allowable center-to-center spacing. Figure 4.3 shows an example of a bridge model in Sofistik.

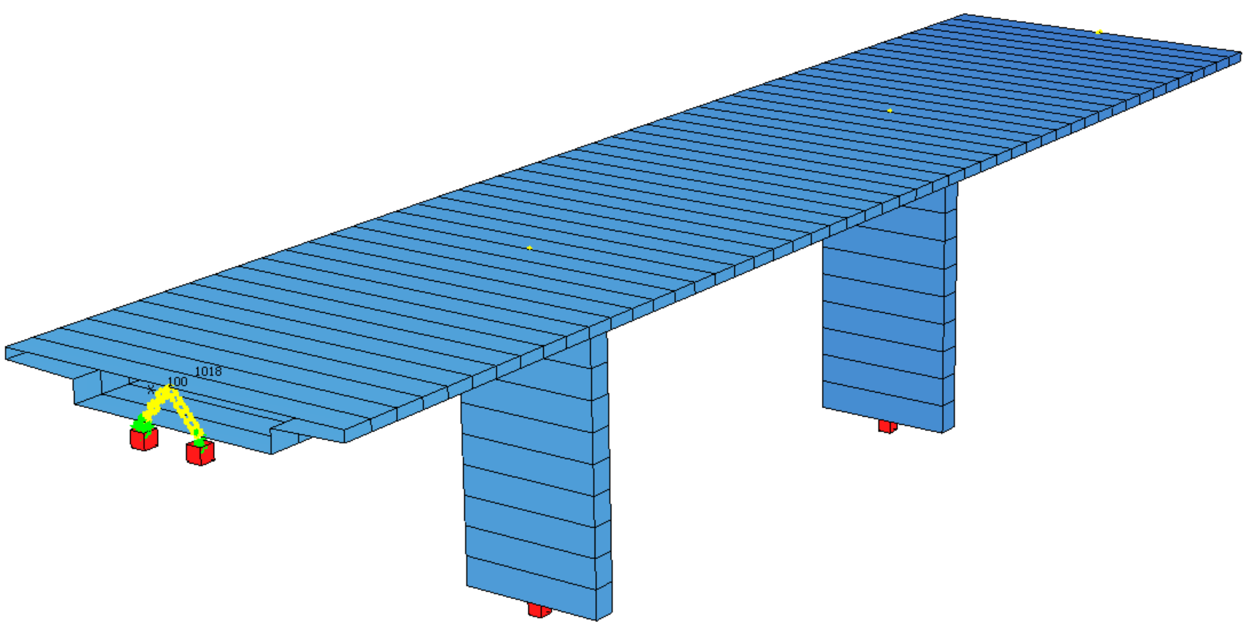


Figure 4.3: Example of a bridge model

4.2.1 Modelling Process

The sequence of the general workflow, utilizing SSD and SOFiPLUS as depicted in Figure 4.4.

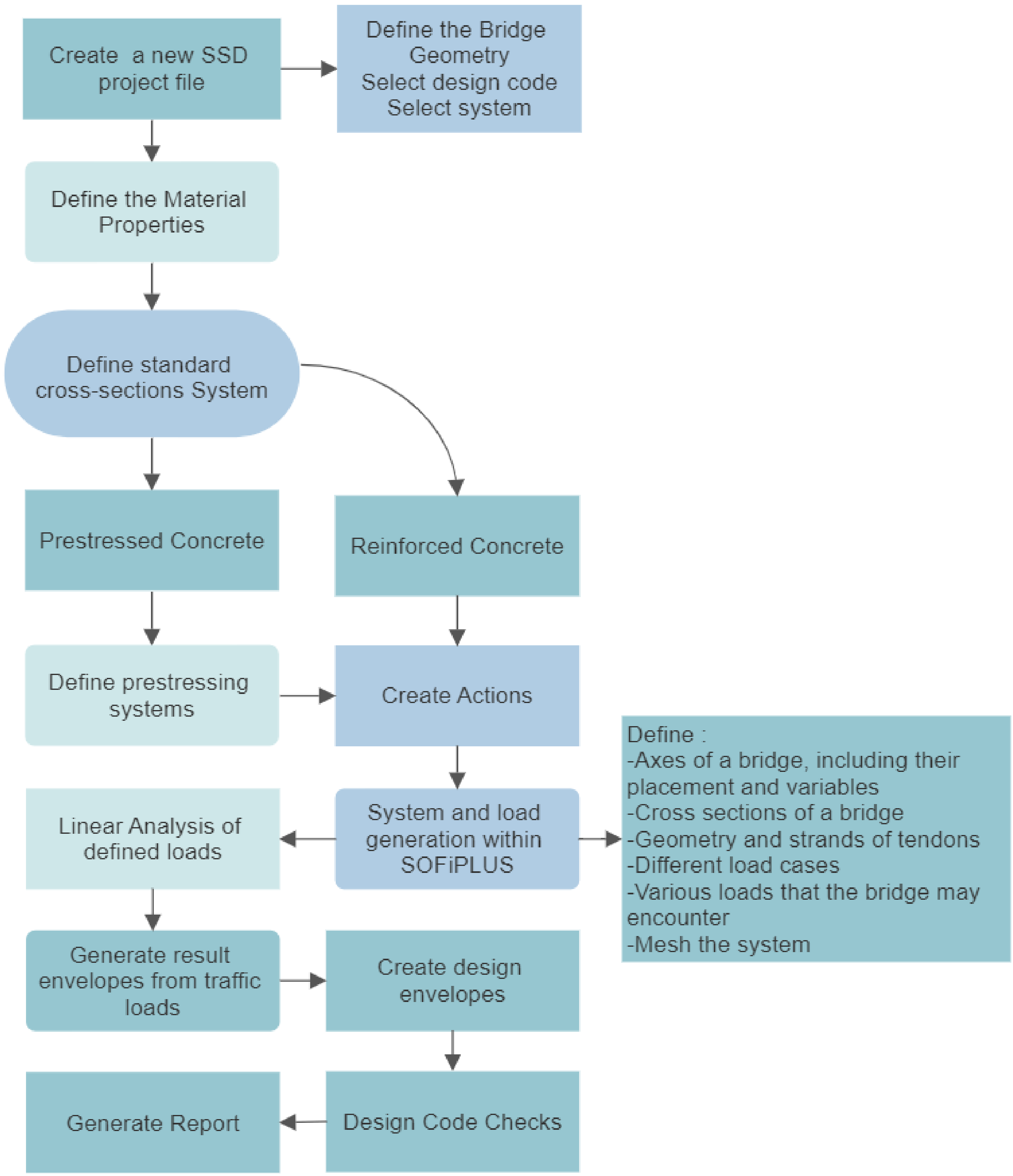


Figure 4.4: General workflow

In this thesis, a parametric approach was employed to simplify and streamline the process of inputting data for the bridge models. The input data for the bridge models were written and parameterized as TEDDY task files to simplify the data input process. All tasks were executed using TEDDY's own programming language. By utilizing TEDDY's interactive worksheets, the data input process was further simplified as only the parameters of the structure need to be inputted, and from these given parameters, the geometry of the bridge model, loads calculation, and design of the model were automatically generated. This approach streamlines the data input process, reduces the potential for manual errors in the model setup, and allows for efficient design iteration to explore various span-to-depth ratios. Of particular note is the parametric setup of the traffic load in the study. It is adjusted according to the total length of the bridge, providing flexibility and adaptability to different bridge configurations. Figure 4.6 shows an example of SSD-project navigation that includes most of the used TEDDY tasks (All tasks are attached in Appendix C) and presented in the following list:

- **Input**: parameters for the bridge model, such as the bridge length, cross-sectional height, column height, and starting load case, are defined in the input task file.
- **Cross Section** : the geometric cross-sections of the bridge master and columns are parameterized in the cross-section task file.
- **Static system**: the coordinates of axis 1-4 and the superstructure for the static system group are parameterized in the static system task file. Then, the structural system is meshed into beam elements in the Mesh task file.
- **Prestressing**: the prestressing system, the geometric and the type of the post-tensioning tendons are defined and parameterized in the prestressing group.
- **Actions**: in this task file, current actions are defined with corresponding rules for combination and factors.
- **Loads** : the load cases for permanent, temperature, wind and traffic loads are parameterized in task files.

- **Static Analysis**: the pre-defined loads are applied and the structure is analyzed using the ASE model. The Traffic Envelope module creates the bending moment envelope of the most critical traffic load cases
- **Construction Stages**: The construction stage manager is responsible for setting up and organizing the various stages of the structure's construction process, as well as conducting analyses on creep and shrinkage.
- **Load Combinations**: describes the superposition of actions.
- **Design**: Limit states, ULS and SLS, design of beams are defined in task files.

Figure 4.5 displays some of the most commonly used defined parameters in the Input-task file. The **sto** command as shown in the figure, which stands for "store," is utilized to store new variables in the system. For example, the **#lengdeB** parameter is used to define the bridge's total length in meters, while **#Htverr** specifies the height of the cross-section. Whenever new values are assigned to these parameters, a new bridge model is automatically generated, along with all other related Teddy tasks shown in Figure 4.6, such as Actions, Loads, Load combinations, and model design. These tasks are parameterized to follow the new parameters, resulting in new load distributions and model designs. This feature enables efficient exploration of various span-to-depth ratios for reinforced and prestressed concrete bridges, leading to a more informed and optimal design decision. The TEDDY task files contain coding language, as depicted in Figure 4.7, which demonstrates the use of the defined parameters from the Input task file to create the reinforced cross-section of the bridge. This illustrates the powerful capabilities of parametric design in generating complex models based on predefined variables.

```
+prog template urs:27.1
head
$Bridge Parameters:
sto#lengdeB 50  $ Bridge Total Length L in meter
sto#Htverr 1.15  $ Height of Cross Section in meter
sto#Slen 10      $ Columns Height in meter
sto#Cen2Arm 0.120  $ Length from concrete cover to center of Øl
```

Figure 4.5: Parameters defined in Input task file

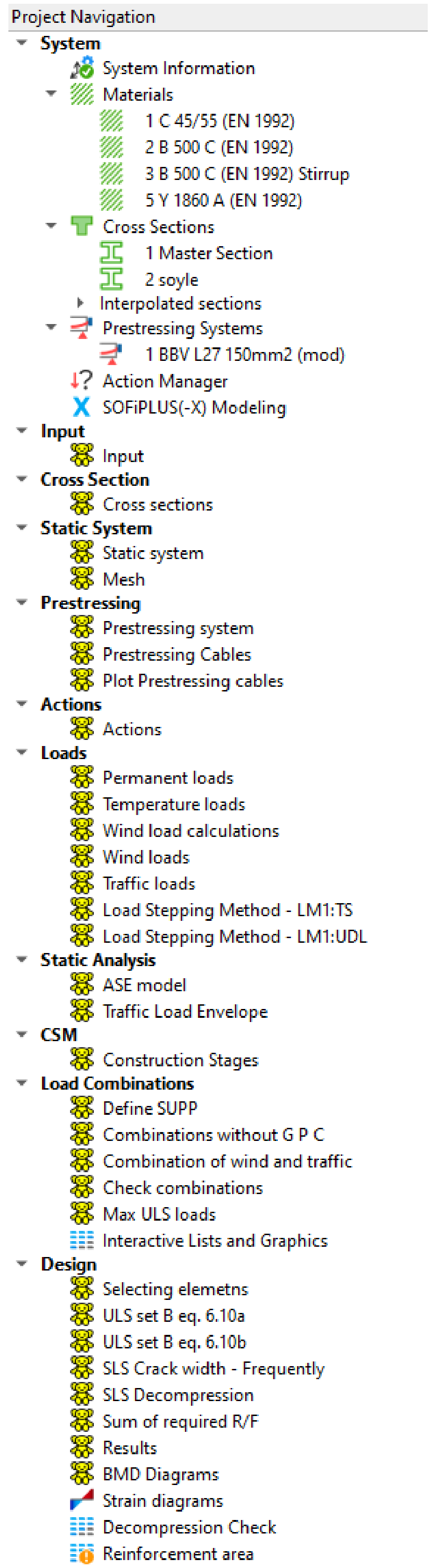


Figure 4.6: Overview of SSD-project navigation

```
PROG AQUA urs:34.1
HEAD Masteroppgaven
UNIT 5
CTRL
CTRL RFCS 0
CTRL FACE -1
CTRL REFD 0
CTRL STYP  FEM
CTRL SCUT 0
CTRL PLAS 1
SECT 1 MNO 1 MRF 2 MRFL 3 FSYM NONE BTYP BEAM TITL "Master Section"
SV   IT 100[o/o] LEVY 85[o/o] LEVZ 85[o/o]
TVAR 'NEFF' VAL 0[mm]
LAY  1 'BOT' TYPE  MIN MRF 2
LAY  2 'TOP' TYPE  MIN MRF 2
LAY  3 'TLEFT' TYPE  MIN MRF 2
LAY  3 'TRIGH' TYPE  MIN MRF 2
$LAY  5 'TORS' TYPE  OPT MRF 2
_
//lengdearmering
LRF  'TOP' YB -3500+#Cen2Arm*1000 ZB #Cen2Arm*1000 YE 3500-#Cen2Arm*1000 ZE #Cen2Arm*1000 AS 26.80[cm2/m] LAY   M2 TORS ACTI D 32 A 300 DIST EVEN         //konstant
LRF  'TLEFT' YB -3500+#Cen2Arm*1000 ZB #Cen2Arm*1000 YE -5950 ZE #Cen2Arm*1000 AS 26.80[cm2/m] LAY   M2 TORS PASS D 32 A 300 DIST EVEN
LRF  'TRIGH' YB 3500-#Cen2Arm*1000 ZB #Cen2Arm*1000 YE 5950 ZE #Cen2Arm*1000 AS 26.80[cm2/m] LAY   M2 TORS PASS D 32 A 300 DIST EVEN
LRF  'BOT' YB 3500-#Cen2Arm*1000 ZB (#Htverr-#Cen2Arm)*1000 YE -3500+#Cen2Arm*1000 ZE (#Htverr-#Cen2Arm)*1000 AS 26.80[cm2/m] LAY   M1 TORS ACTI D 32 A 300 DIST EVEN

//tverrsnitt
POLY TYPE    O MNO 1
VERT '0100' Y -3500 Z 450 EXP 1      //konstant
VERT '0101' Y -6000 Z 350 EXP 1      //konstant
VERT '0102' Y -6000 Z 0 EXP 1        //konstant
VERT '0103' Y 6000 Z 0 EXP 1         //konstant
VERT '0104' Y 6000 Z 350 EXP 1      //konstant
VERT '0105' Y 3500 Z 450 EXP 1      //konstant
VERT '0106' Y 3500 Z #Htverr*1000 EXP 1
VERT '0107' Y -3500 Z #Htverr*1000 EXP 1
VERT '0100' Y -3500 Z 450 EXP 1      //konstant
```

Figure 4.7: Teddy task that create reinforced cross-section based on some parameters

4.3 Placement of Tendon Profiles

As a simplification, the tendon profiles are positioned across the cross-section based on the expertise and calculations of Sweco engineers. A total of 10 tendons are utilized, with a duct diameter of 120mm. Each tendon comprises 27 strands, each having an area of 150mm^2. The arrangement of the pre-stressing tendons is depicted in Figure 4.8.

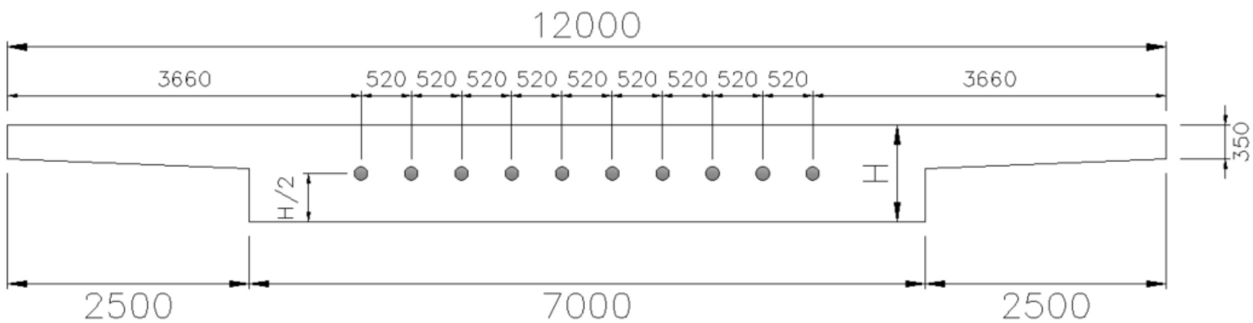


Figure 4.8: Arrangement of pre-stressing tendons over axis 1 and 4 (Dimensions given in mm)

The eccentricities of the parabolic tendons are parameterized in Sofistik, ensuring that the top and bottom points align with the height of the cross-section. The eccentricities are presented in Table 4.1 and illustrated in Figure 4.9. Figure 4.10 shows the distribution of tendons along the cross-section of the p36-2 model discussed in Section 5.3.4.2. The parameterization code utilized to define the parabolic tendons is depicted in Figure 4.11.

Table 4.1: Eccentricities of the parabolic tendons (Dimensions given in m)

	Axis 1	**Span 1-2**	**Axis 2**	**Span 2-3**	**Axis 3**	**Span 3-4**	**Axis 4**
Eccentricity	H/2	H-0.2	0.2	H-0.2	0.2	H-0.2	H/2

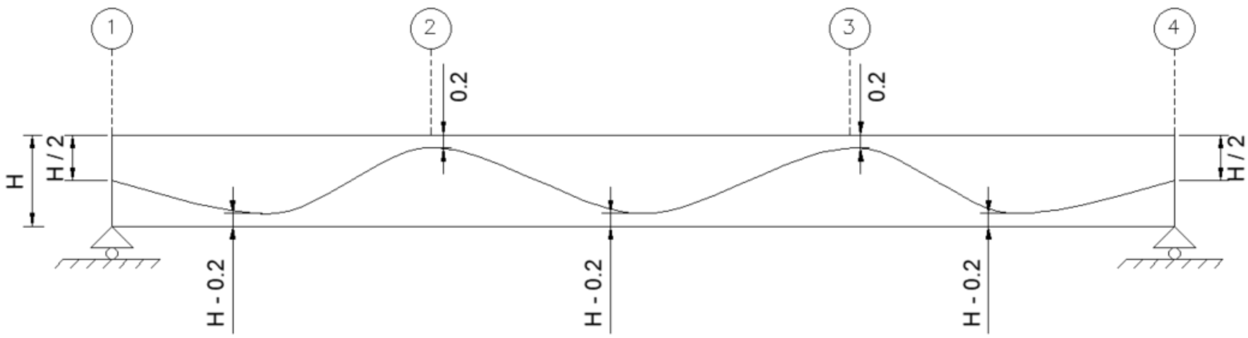


Figure 4.9: Eccentricities of the parabolic tendons

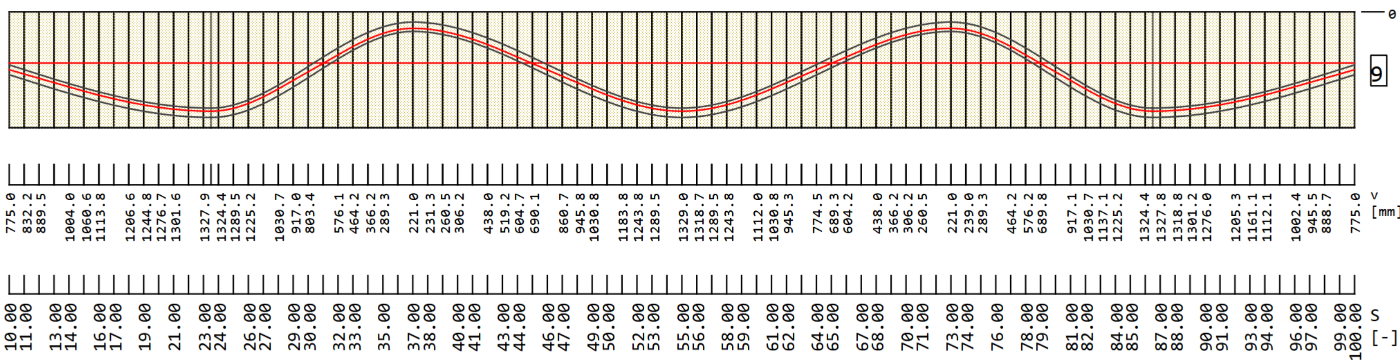


Figure 4.10: Elevation view of the distribution of tendons for model p36-2. The first number line represents the height of tendons measured in [mm] from the top of the cross-section and the second line number represents the length of the model in [m]

```
#define Spennfoering
TGEO NOG #tendonNr   NOH  1   NOPS  1  TITL  'Forste spenn'
  PTUV TYPE refx s  #l1              U #B1(#1)  V #Htverr/2   KIND PRFX $Start point at axis 1 = (H/2)
  PTUV TYPE refx s  (#l2+#l1)/2      U #B1(#1)  V #Htverr-0.2  dvs 0.0  $ Bottom point at span 1-2 =(H-0.2m)
  PTUV TYPE refx s  #l2              U #B1(#1)  V 0.2 dvs 0.0  $Top point at axis 2 = 0.2m from the top
  PTUV TYPE refx s  (#l2+#l3)/2      U #B1(#1)  V #Htverr-0.2  dvs 0.0  $ Bottom point at span 2-3  = (H-0.2m)
  PTUV TYPE refx s  #l3              U #B1(#1)  V 0.2  dvs 0.0  $ Top point at axis 3 = 0.2m from the top
  PTUV TYPE refx s  (#l4+#l3)/2      U #B1(#1)  V #Htverr-0.2 dvs 0.0   $ Botton point at axis 3-4  = (H-0.2m)
  PTUV TYPE refx s  #l4              U #B1(#1)  V #Htverr/2  $ End point at axis 4  = (H/2)
#enddef
```

Figure 4.11: Parameterization code of tendons placement in Sofistik

Chapter 5

Results and Discussion

The following sections present the results for the bending moments of the bridge models under ultimate limit state (ULS) and serviceability limit state (SLS) conditions. The maximum hogging and sagging bending moments that occur at the mid-supports and mid-span are the primary focus in ULS. Additionally, the load combinations used in the design cases are presented.

All models have a cross-sectional width of 12m. The amount of reinforcement in the top and bottom layers is inputted as a minimum reinforcement in Sofistik, using steel reinforcement with a diameter of ø32. The amount of reinforcement in each layer is increased as needed when designing the bridge models in Sofistik. Each model is checked to ensure that the designed reinforcement area in each layer does not exceed the maximum allowable areas of 957 cm^2 (+/- 5%) at the top layer and 546.88 cm^2 (+/- 5 %) at the bottom layer, made of steel reinforcement ø32c100mm. Table 5.1 displays the maximum allowable reinforcement areas and the maximum number of distributed longitudinal reinforcements in each layer. Figure 5.1 shows a cross section of the models with top and bottom reinforcement layers. In addition to the previous checks, a check of the maximum decompression strain will be carried out for post-tensioned models, as mentioned in Section 3.4.4 and given in EC2-1 (Standard Norge 2004a), Table NA.7.1N note *b*. This note states that for sections where it is required to verify that decompression does not occur, the prestressing steel and the duct for post-tensioned tendons should lie at least ΔC_{dev} within the compression zone.

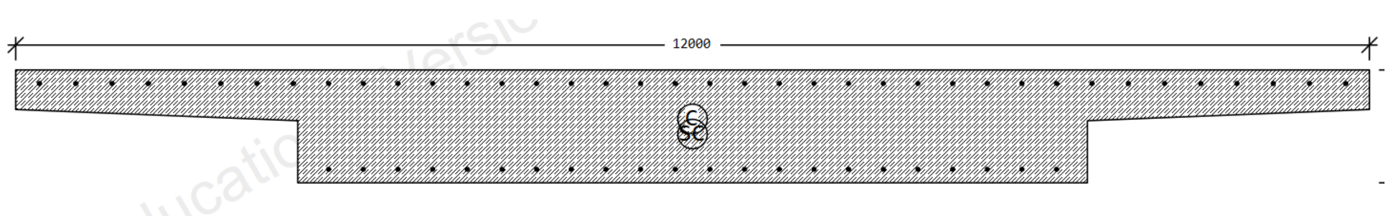


Figure 5.1: Cross section with its R/F layers

Table 5.1: Maximum of allowable R/F area and number of longitudinal R/F in each layer

	Width [m]	**Spacing C-C [mm]**	**Number of R/F**	**Total Max. Area [cm^2]**	**± 5% of Total Area**
Top layer ASL2	11.9	100	119	957	47.85
Bottom Layer ASL1	6.76	100	68	546.88	27.34

5.1 Design Values of Reinforced Concrete

5.1.1 Model 8

The Bridge model 8 is composed of a central span of 8m length and two side spans each of 6m length, resulting in a total length of 20m. Table 5.2 summarizes the sub-model of model 8 and its corresponding efficiency percentages in using the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.2: Sub-model of model 8

Model	**Mid span length [m]**	**Total length [m]**	**Cross section depth [mm]**	**Span-to-depth ratio**	**ASL1**	**ASL2**	**% ASL1**	**% ASL2**
Model 8-1	8	20	450	17.778	Ok	Ok	42.28 %	57.97 %

5.1.1.1 Model 8-1

Model 8-1 has a mid-span length of 8m, cross section height of 450mm, and a span-to-depth ratio of 17.778. Figure 5.2 and Figure 5.3 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

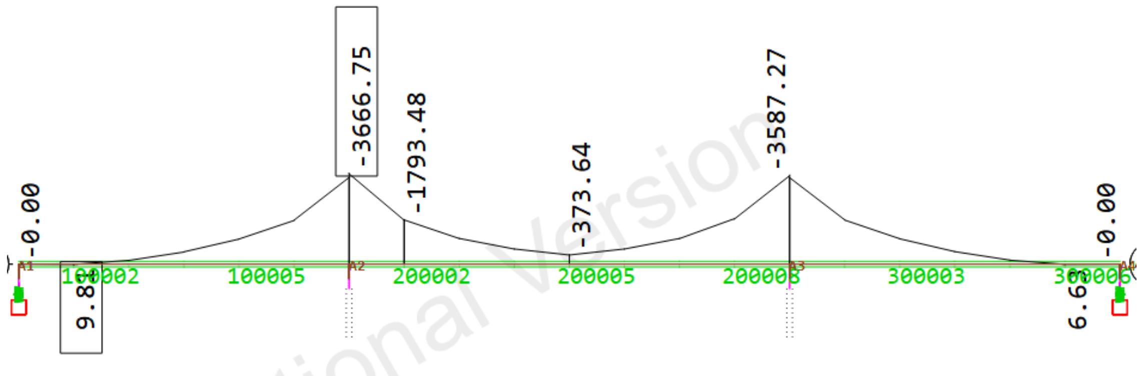


Figure 5.2: BMD My [kNm] for load combinations at middle supports, model 8-1

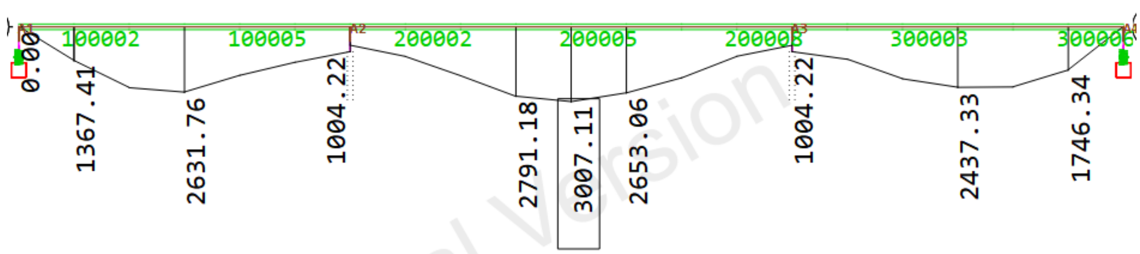


Figure 5.3: BMD My [kNm] for load combinations at middle span, model 8-1

Table 5.3 and Table 5.4 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.3: Load combination due to different load cases "LC" returning highest My at middle supports, model 8-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200001	0.000	G_1	5010	Overbygning	1.200	-674.16
		G_2	5020	Superegenvekt	1.200	-282.76
		-	10004	LM1:TS Lane 10/11/12 Step5	1.350	-1062.00
			100	LM1:UDL Lane 10/11/12 Span1	1.350	-55.34
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-178.03
		T	84	Tsummer+negdt +wn*TN+DTZ	0.840	-862.67
		ZW	24	+Vertikalt	1.120	-40.21
200001	0.000		2302	MIN-MY BEAM ULSB_b		-3666.75

Table 5.4: Load combination due to different load cases "LC" returning highest My at middle span, model 8-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200005	0.000	C	5030	Creep until t-infinite	1.350	69.02
			5031	Creep until t-infinite	1.350	8.91
			5032	Creep until t-infinite	1.350	5.20
		G_1	5010	Overbygning	1.200	355.84
		G_2	5020	Superegenvekt	1.200	149.24
		-	10004	LM1:TS Lane 10/11/12 Step5	1.350	1044.52
			101	LM1:UDL Lane 10/11/12 Span2	1.350	131.57
		T	82	Twinter+posdt -wn*TN-DTZ	0.840	806.26
		ZW	24	+Vertikalt	1.120	21.26
200005	0.000		2301	MAX-MY BEAM ULSB_b		3007.11

Table 5.5 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.5: Required steel reinforcement, Model 8-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		360.44	545.39	-	161.00
	200005	0.000	0.000	101	tens	tens		184.64	318.92	-	161.00
		1.000	1.000	101	tens	tens		231.23	334.46	-	161.00
	300001	0.000	0.000	101	tens	tens		361.22	554.78	-	161.00

DC design case   Xi Relative distance
BEAM beam element   NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 554.78 = 402.22 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 231.23 = 315.65 cm^2$$

As the remaining areas are positive, it indicates that the design reinforcement areas do not exceed the maximum allowable areas. Additionally, a span length of 8m with a minimum cross-section depth of 450mm, resulting in a span-to-depth ratio of 17.778, is acceptable when designing a reinforced concrete bridge that is 20m in length.

The design case is over middle supports where,

$$\frac{\text{requirement R/F area } (554.78\ \text{cm}^2)}{\text{the maximum allowable R/F area } (957\ \text{cm}^2)} * 100 = 57.97\% \text{ of the max. allowable R/F area.}$$

5.1.2 Model 12

The Bridge model 12 is composed of a central span of 12m length and two side spans each of 9m length, resulting in a total length of 30m. Table 5.6 displays sub-model 12-1 and the corresponding efficiency percentages for utilizing the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.6: Sub-models of model 12

Model	Mid span length [m]	Total length [m]	Cross section depth [mm]	span-to-depth ratio	ASL1	ASL2	% ASL1	% ASL2
Model 12-1	12	30	450	26.667	Ok	Ok	60.66 %	81 %

5.1.2.1 Model 12-1

Model 12-1 has a mid-span length of 12m, cross section height of 450mm, and a span-to-depth ratio of 26.667. Figure 5.4 and Figure 5.5 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

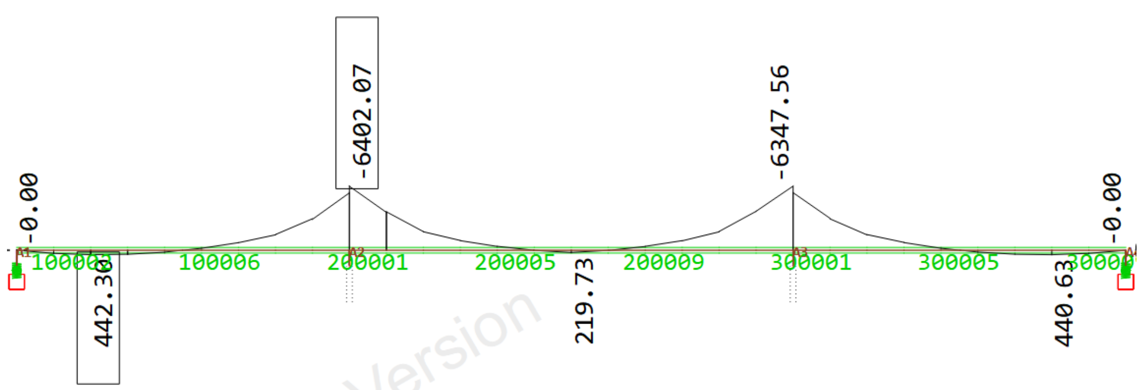


Figure 5.4: BMD My [kNm] for load combinations at middle supports, model 12-1

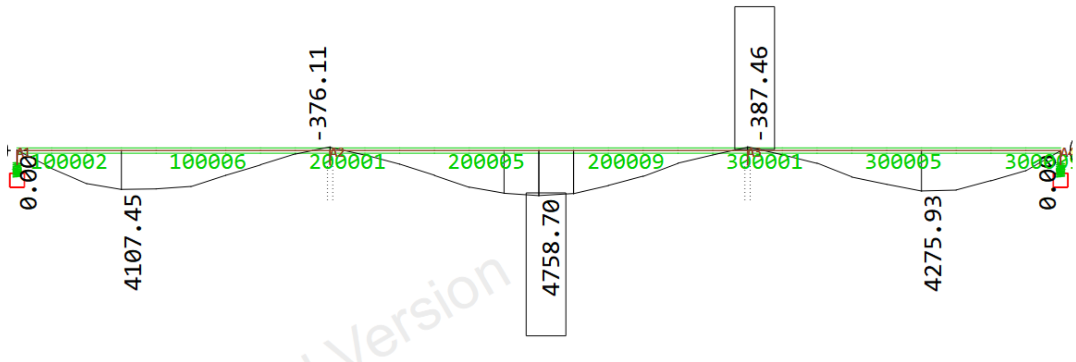


Figure 5.5: BMD My [kNm] for load combinations at middle span, model 12-1

Table 5.7 and Table 5.8 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.7: Load combination due to different load cases "LC" returning highest My at middle supports, model 12-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200001	0.000	G_1	5010	Overbygning	1.200	-1521.66
		G_2	5020	Superegenvekt	1.200	-638.21
		-	10006	LM1:TS Lane 10/11/12 Step7	1.350	-1712.48
			100	LM1:UDL Lane 10/11/12 Span1	1.350	-109.46
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-415.94
		T	84	Tsummer+negdt +wn*TN+DTZ	0.840	-818.38
		ZW	24	+Vertikalt	1.120	-90.76
200001	0.000		2302	MIN-MY BEAM ULSB_b		-6402.07

Table 5.8: Load combination due to different load cases "LC" returning highest My at middle span, model 12-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200006	1.000	C	5030	Creep until t-infinite	1.350	53.14
			5031	Creep until t-infinite	1.350	-0.30
			5032	Creep until t-infinite	1.350	4.96
		G_1	5010	Overbygning	1.200	795.84
		G_2	5020	Superegenvekt	1.200	333.79
		-	10507	LM1:TS Lane 20/21/22 Step8	1.350	1659.85
			104	LM1:UDL Lane 20/21/22 Span2	1.350	280.66
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	776.36
		ZW	24	+Vertikalt	1.120	47.57
200006	1.000		2301	MAX-MY BEAM ULSB_b		4758.70

Table 5.9 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.9: Required steel reinforcement, Model 12-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	1.000	1.000	101	tens	tens		344.87	524.30	-	161.00
	200006	0.000	0.000	101	tens	tens		330.30	404.04	-	161.00
	200007	0.000	0.000	101	tens	tens		331.74	366.67	-	161.00
	300001	0.000	0.000	101	tens	tens		424.51	774.49	-	161.00

DC design case Xi Relative distance
BEAM beam element NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 774.49 = 182.51 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 331.74 = 215.14 cm^2$$

As the remaining areas are positive, it indicates that the design reinforcement areas do not exceed the maximum allowable areas. Additionally, a span length of 12m with a minimum cross-section depth of 450mm, resulting in a span-to-depth ratio of 26.667, is acceptable when designing a reinforced concrete bridge that is 30m in length.
The design case is over middle supports where,

$$\frac{\text{requirement R/F area (774.49 cm}^2\text{)}}{\text{the maximum allowable R/F area (957 cm}^2\text{)}} * 100 = 80.09\% \text{ of the max. allowable R/F area.}$$

5.1.3 Model 16

Model 16 of the bridge comprises a central span measuring 16m in length, flanked by two side spans each measuring 12m in length, resulting in a total length of 40m. Table 5.10 provides an overview of the various sub-models that make up Model 16 and their corresponding efficiency percentages in utilizing the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.10: Sub-models of model 16

Model	Mid span length [m]	Total length [m]	Cross section depth [mm]	span-to-depth ratio	ASL1	ASL2	% ASL1	% ASL2
Model 16-1	16	40	450	35.56	Not Ok	Not Ok	108.16%	132.76%
Model 16-2	16	40	500	32	Ok	Not Ok	92.90%	113.45%
Model 16-3	16	40	550	29.1	Ok	Ok	86.97%	95.54%

5.1.3.1 Model 16-1

Model 16-1 has a mid-span length of 16m, cross section height of 450mm, and a span-to-depth ratio of 35.556. Figure 5.6 and Figure 5.7 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

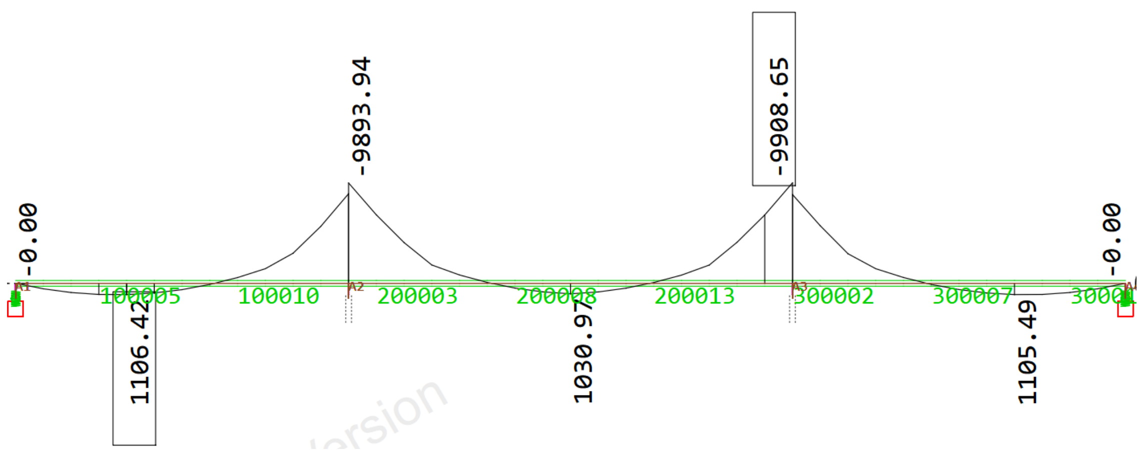


Figure 5.6: BMD My [kNm] for load combinations at middle supports, model 16-1

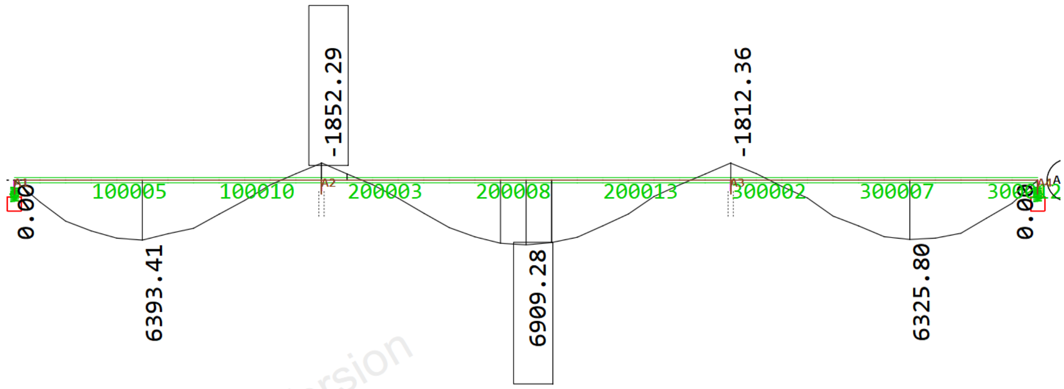


Figure 5.7: BMD My [kNm] for load combinations at middle span, model 16-1

Table 5.11 and Table 5.12 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.11: Load combination due to different load cases "LC" returning highest My at middle supports, model 16-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200016	1.000	G_1	5010	Overbygning	1.200	-2712.49
		G_2	5020	Superegenvekt	1.200	-1137.67
		-	10010	LM1:TS Lane 10/11/12 Step11	1.350	-2357.53
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-755.75
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-176.04
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-793.65
		ZW	24	+Vertikalt	1.120	-161.80
200016	1.000		2302	MIN-MY BEAM ULSB_b		-9908.65

Table 5.12: Load combination due to different load cases "LC" returning highest My at middle span, model 16-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200008	1.000	C	5030	Creep until t-infinite	1.350	48.54
			5031	Creep until t-infinite	1.350	-2.62
			5032	Creep until t-infinite	1.350	4.99
		G_1	5010	Overbygning	1.200	1407.51
		G_2	5020	Superegenvekt	1.200	590.33
		-	10509	LM1:TS Lane 20/21/22 Step10	1.350	2262.34
			104	LM1:UDL Lane 20/21/22 Span2	1.350	482.65
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	765.71
		ZW	24	+Vertikalt	1.120	84.12
200008	1.000		2301	MAX-MY BEAM ULSB_b		6909.28

Table 5.13 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.13: Required design steel reinforcement, Model 16-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		749.00	1270.51	-	161.00
	200008	1.000	1.000	101	tens	tens		535.48	378.44	-	161.00
	200009	1.000	1.000	101	tens	tens		591.53	418.98	-	161.00
	300001	0.000	0.000	101	tens	tens		446.75	1160.03	-	161.00

DC design case Xi Relative distance
BEAM beam element NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 1270.51 = -313.51 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 591.53 = -44.65 cm^2$$

Since the remaining areas are negative, it is unlikely that a cross section with a height of 450mm will be able to withstand the design loads. This indicates that the design reinforcement areas exceed the maximum allowable reinforcement areas. Therefore, a span length of 16m with a minimum cross-section depth of 450mm, resulting in a span-to-depth ratio of 35.56, is not acceptable when designing a reinforced concrete bridge that is 40m in length.

5.1.3.2 Model 16-2

Model 16-2 has a mid-span length of 16m, cross section height of 500mm, and a span-to-depth ratio of 32. Figure 5.8 and Figure 5.9 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

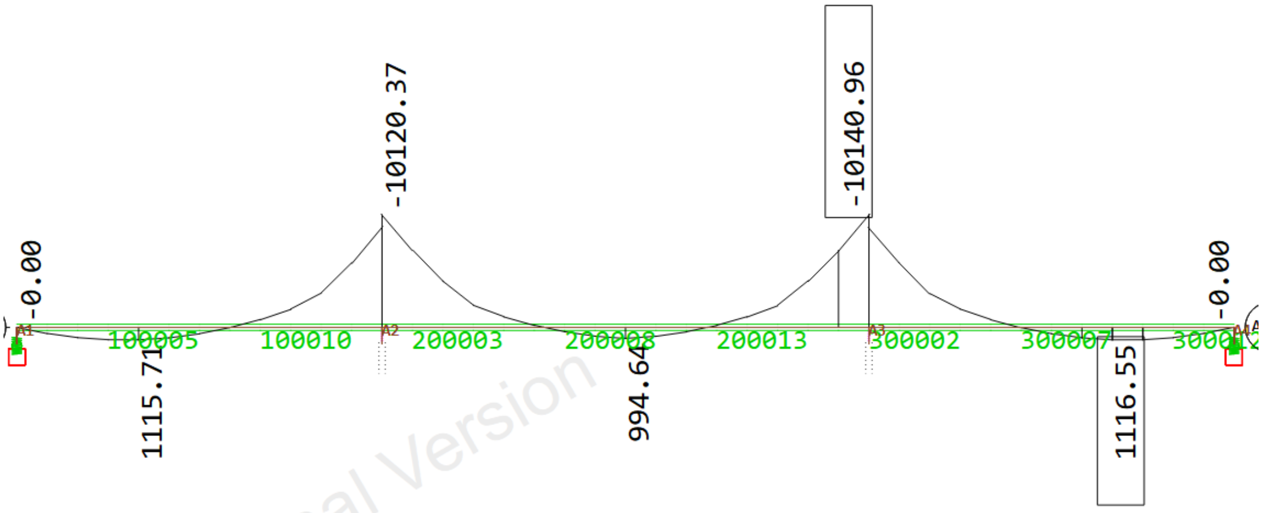


Figure 5.8: BMD My [kNm] for load combinations at middle supports, model 16-2

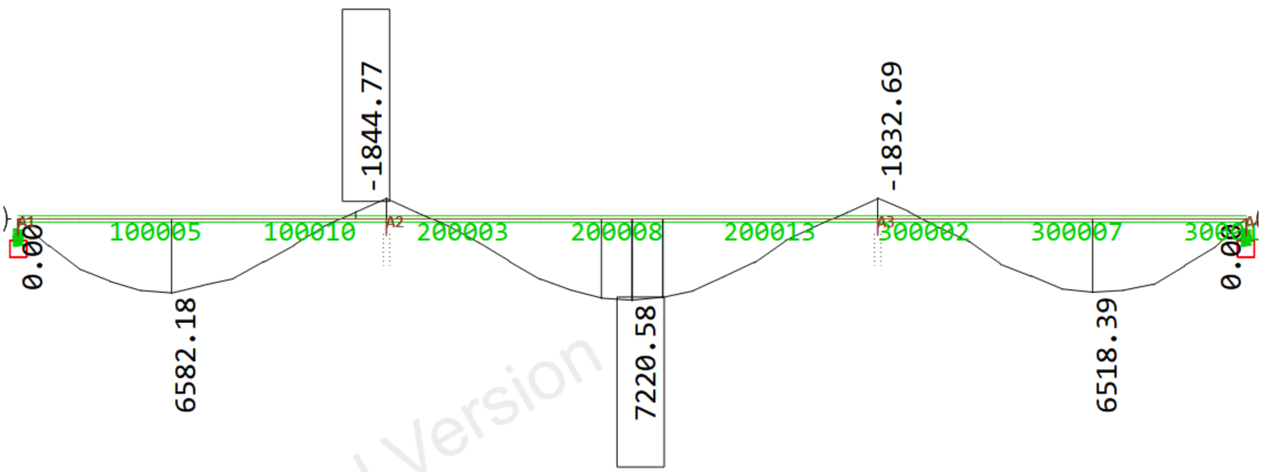


Figure 5.9: BMD My [kNm] for load combinations at middle span, model 16-2

Table 5.14 and Table 5.15 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.14: Load combination due to different load cases "LC" returning highest My at middle supports, model 16-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200016	1.000	G_1	5010	Overbygning	1.200	-2889.57
		G_2	5020	Superegenvekt	1.200	-1134.81
		-	10010	LM1:TS Lane 10/11/12 Step11	1.350	-2307.23
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-742.95
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-190.48
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-900.10
		ZW	24	+Vertikalt	1.120	-161.37
200016	1.000		2302	MIN-MY BEAM ULSB_b		-10140.96

Table 5.15: Load combination due to different load cases "LC" returning highest My at middle span, model 16-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200008	1.000	C	5030	Creep until t-infinite	1.350	57.21
			5031	Creep until t-infinite	1.350	0.55
			5032	Creep until t-infinite	1.350	4.75
		G_1	5010	Overbygning	1.200	1510.43
		G_2	5020	Superegenvekt	1.200	593.19
		-	10509	LM1:TS Lane 20/21/22 Step10	1.350	2299.17
			104	LM1:UDL Lane 20/21/22 Span2	1.350	495.45
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	886.21
		ZW	24	+Vertikalt	1.120	84.54
200008	1.000		2301	MAX-MY BEAM ULSB_b		7220.58

Table 5.16 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.16: Required design steel reinforcement, Model 16-2

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		451.12	1085.74	-	161.00
	200008	1.000	1.000	101	tens	tens		474.57	370.52	-	161.00
	200009	1.000	1.000	101	tens	tens		508.00	401.67	-	161.00
	300001	0.000	0.000	101	tens	tens		407.73	997.76	-	161.00

DC design case
Xi Relative distance
BEAM beam element
NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 1085.74 = -128.74cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 508 = 38.88cm^2$$

The negative remaining area over support indicates that the design reinforcement area exceed the maximum allowable area. Therefore, a span length of 16m with cross-section depth of 500mm, resulting in a span-to-depth ratio of 32, is not acceptable when designing a reinforced concrete bridge of 40m total length. However, the design case is at middle span. where,

$$\frac{\text{requirement R/F area (508 cm}^2\text{)}}{\text{the maximum allowable R/F area (546.88 cm}^2\text{)}} * 100 = 92.9\% \text{ of the max. allowable R/F area (ASL1)}$$

5.1.3.3 Model 16-3

Model 16-3 has a mid-span length of 16m, cross section height of 550mm, and a span-to-depth ratio of 29.1. Figure 5.10 and Figure 5.11 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

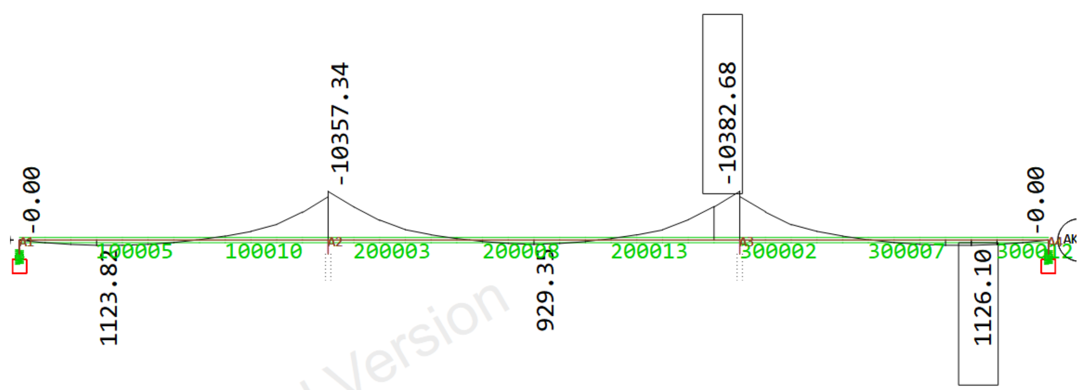


Figure 5.10: BMD My [kNm] for load combinations at middle supports, model 16-3

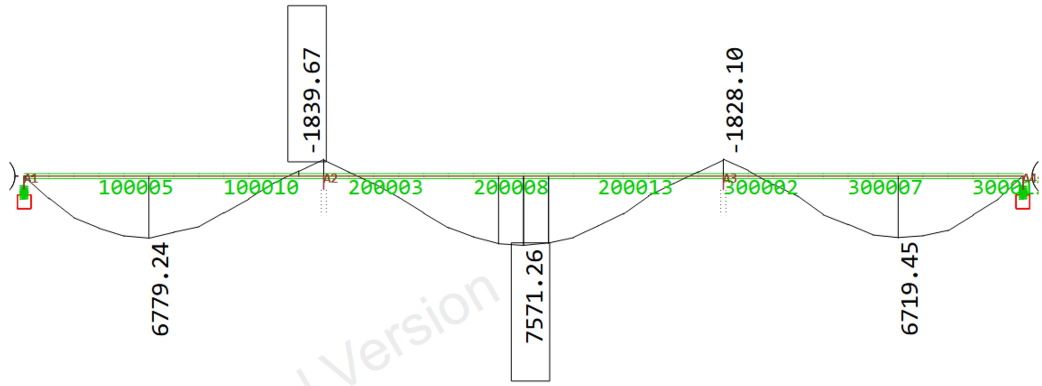


Figure 5.11: BMD My [kNm] for load combinations at middle span, model 16-3

Table 5.17 and Table 5.18 show the load combinations that return highest BMD over middle supports and at middle span, respectively.

Table 5.17: Load combination due to different load cases "LC" returning highest My at middle supports, model 16-3

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200016	1.000	G_1	5010	Overbygning	1.200	-3064.68
		G_2	5020	Superegenvekt	1.200	-1131.57
		-	10010	LM1:TS Lane 10/11/12 Step11	1.350	-2254.38
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-728.73
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-204.33
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-1028.48
		ZW	24	+Vertikalt	1.120	-160.90
200016	1.000		2302	MIN-MY BEAM ULSB_b		-10382.68

Table 5.18: Load combination due to different load cases "LC" returning highest My at middle span, model 16-3

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200008	1.000	C	5030	Creep until t-infinite	1.350	67.46
			5031	Creep until t-infinite	1.350	4.93
			5032	Creep until t-infinite	1.350	4.37
		G_1	5010	Overbygning	1.200	1615.32
		G_2	5020	Superegenvekt	1.200	596.43
		-	10509	LM1:TS Lane 20/21/22 Step10	1.350	2340.07
			104	LM1:UDL Lane 20/21/22 Span2	1.350	509.67
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	1037.09
		ZW	24	+Vertikalt	1.120	85.02
200008	1.000		2301	MAX-MY BEAM ULSB_b		7571.26

Table 5.19 shows the required longitudinal steel reinforcement to withstand the design BMD.

Table 5.19: Required design steel reinforcement, Model 16-3

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		392.99	914.31	-	161.00
	200008	1.000	1.000	101	tens	tens		475.62	362.57	-	161.00
	200009	0.000	0.000	101	tens	tens		475.62	363.00	-	161.00
	300001	0.000	0.000	101	tens	tens		355.06	846.36	-	161.00

DC design case; Xi Relative distance
BEAM beam element; NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 914.31 = 42.69cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 475.62 = 71.26cm^2$$

As the remaining areas are positive, it indicates that the design reinforcement areas do not exceed the maximum allowable areas. Therefore, a span length of 16m with a cross-section depth of 550mm, resulting in a span-to-depth ratio of 29.1, is acceptable when designing a reinforced concrete bridge that is 40m in length.

The design case is over middle supports where,

$$\frac{\text{requirement R/F area (914.31 cm}^2\text{)}}{\text{the maximum allowable R/F area (957 cm}^2\text{)}} * 100 = 95.5\% \text{ of the max. allowable R/F area (ASL2)}$$

5.1.4 Model 20

The Bridge model 20 is composed of a central span of 20m length and two side spans each of 15m length, resulting in a total length of 50m. Table 5.20 summarizes the different sub-models of model 20 and their corresponding efficiency percentages in using the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.20: Sub-models of model 20

Model	Mid span length [m]	Total length [m]	Cross section depth [mm]	Span to depth ratio	ASL1	ASL2	% ASL1	% ASL2
Model 20-1	20	50	550	36.36	Not ok	Not ok	137.00	132.33
Model 20-2	20	50	600	33.34	Not ok	Not ok	131.19	157.68
Model 20-3	20	50	750	26.67	Not ok	ok	114.77	99.49
Model 20-4	20	50	900	22.23	Not ok	ok	106.71	86.48
Model 20-5	20	50	950	21.05	ok	ok	104.86	83.37

5.1.4.1 Model 20-1

Model 20-1 has a mid-span length of 20m, cross section height of 550mm, and a span-to-depth ratio of 36.36. Figure 5.12 and Figure 5.13 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

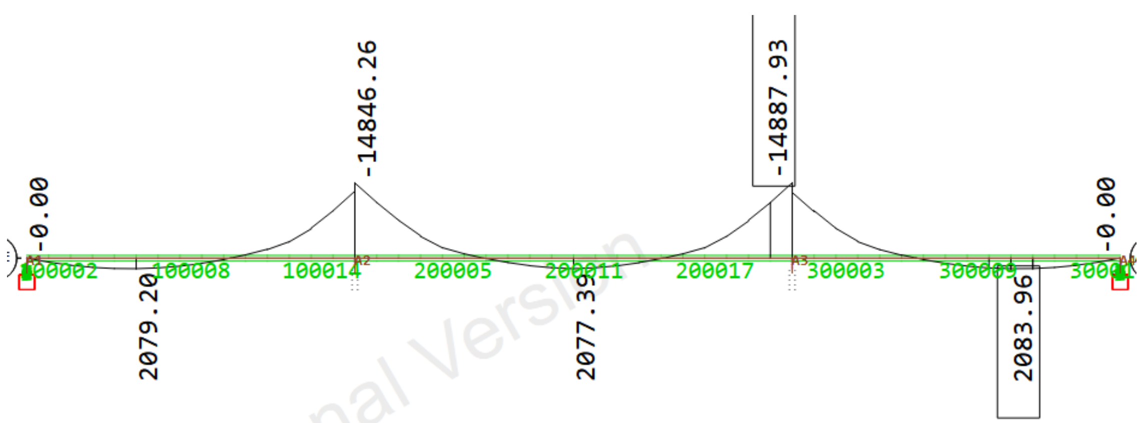


Figure 5.12: BMD My [kNm] for load combinations at middle supports, model 20-1

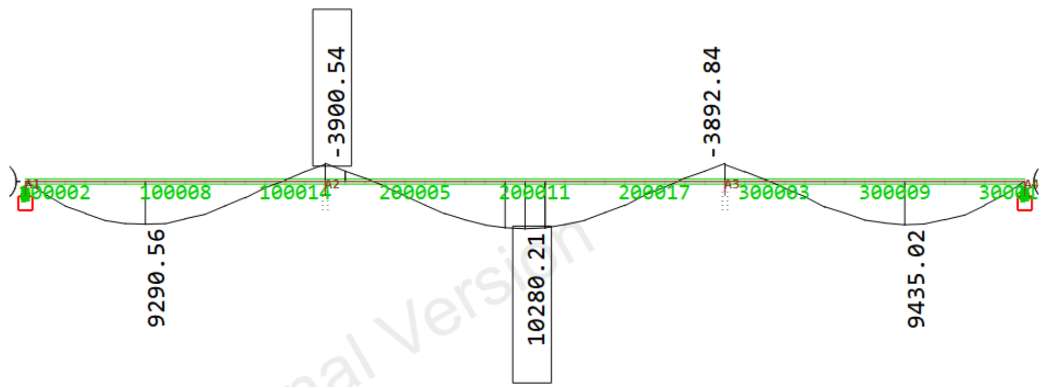


Figure 5.13: BMD My [kNm] for load combinations at middle span, model 20-1

Table 5.21 and Table 5.22 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.21: Load combination due to different load cases "LC" returning highest My at middle supports, model 20-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200020	1.000	G_1	5010	Overbygning	1.200	-4799.00
		G_2	5020	Superegenvekt	1.200	-1771.94
		-	10013	LM1:TS Lane 10/11/12 Step14	1.350	-2892.29
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-1160.15
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-296.84
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-1010.80
		ZW	24	+Vertikalt	1.120	-251.96
200020	1.000		2302	MIN-MY BEAM ULSB_b		-14887.93

Table 5.22: Load combination due to different load cases "LC" returning highest My at middle span, model 20-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200011	0.000	C	5030	Creep until t-infinite	1.350	61.93
			5031	Creep until t-infinite	1.350	3.94
			5032	Creep until t-infinite	1.350	4.10
		G_1	5010	Overbygning	1.200	2513.50
		G_2	5020	Superegenvekt	1.200	928.06
		-	10512	LM1:TS Lane 20/21/22 Step13	1.350	2961.27
			104	LM1:UDL Lane 20/21/22 Span2	1.350	774.85
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	1028.52
		ZW	24	+Vertikalt	1.120	132.29
200011	0.000		2301	MAX-MY BEAM ULSB_b		10280.21

Table 5.23 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.23: Required steel reinforcement, Model 20-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		611.60	1266.41	-	161.00
	200010	1.000	1.000	101	tens	tens		749.28	392.67	-	161.00
	200011	0.000	0.000	101	tens	tens		749.28	377.48	-	161.00
	300001	0.000	0.000	101	tens	tens		372.85	1108.77	-	161.00

DC design case Xi Relative distance
BEAM beam element NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 1266.41 = -309.41 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 749.28 = -202.4 cm^2$$

The resulting remaining areas are negative, it is unlikely that a cross section with a height of 550mm will be able to withstand the design loads. This indicates that the design reinforcement areas exceed the maximum allowable reinforcement areas. Therefore, a span length of 20m with a minimum cross-section depth of 550mm, resulting in a span-to-depth ratio of 36.36, is not acceptable when designing a reinforced concrete bridge that is 50m in length.

5.1.4.2 Model 20-2

Model 20-2 has a mid-span length of 20m, cross section height of 600mm, and a span-to-depth ratio of 33.334. Figure 5.14 and Figure 5.15 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

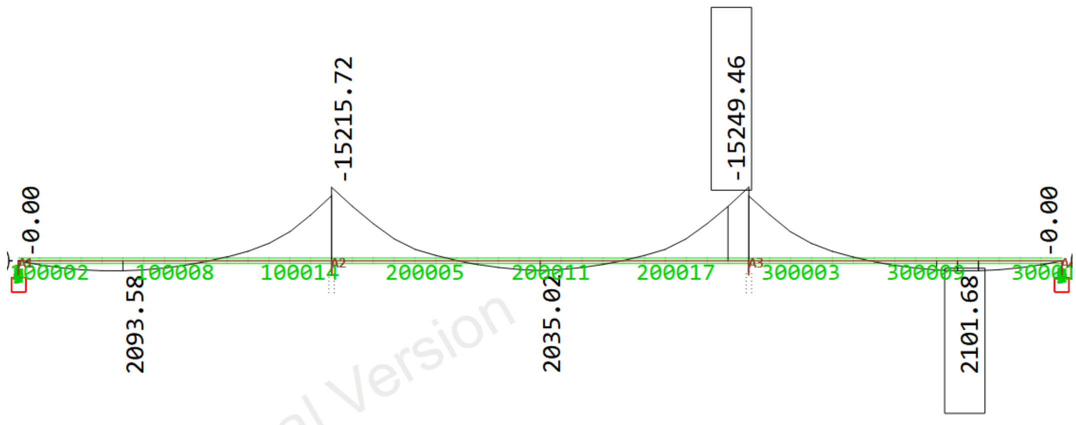


Figure 5.14: BMD My [kNm] for load combinations at middle supports, model 20-2

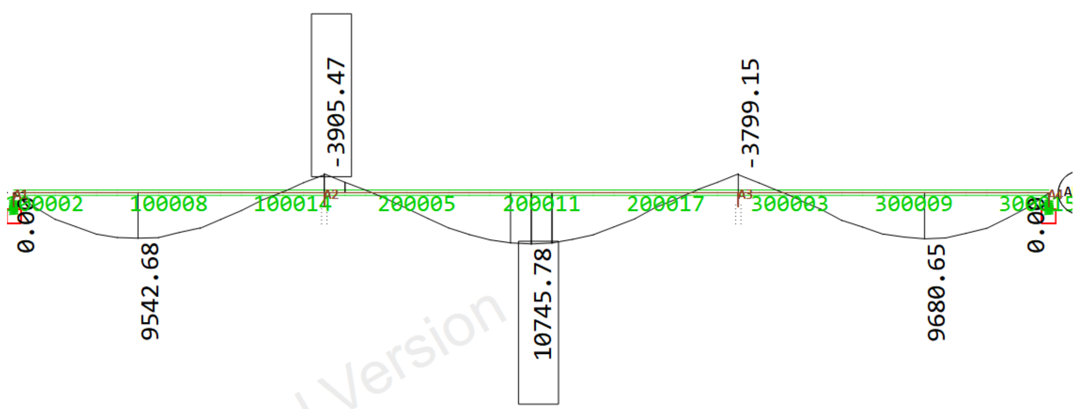


Figure 5.15: BMD My [kNm] for load combinations at middle span, model 20-2

Table 5.24 and Table 5.25 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.24: Load combination due to different load cases "LC" returning highest My at middle supports, model 20-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200020	1.000	G_1	5010	Overbygning	1.200	-5074.37
		G_2	5020	Superegenvekt	1.200	-1767.85
		-	10013	LM1:TS Lane 10/11/12 Step14	1.350	-2825.86
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-1138.49
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-320.20
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-1158.48
		ZW	24	+Vertikalt	1.120	-251.37
200020	1.000		2302	MIN-MY BEAM ULSB_b		-15249.46

Table 5.25: Load combination due to different load cases "LC" returning highest My at middle span, model 20-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200011	0.000	C	5030	Creep until t-infinite	1.350	73.27
			5031	Creep until t-infinite	1.350	8.46
			5032	Creep until t-infinite	1.350	3.71
		G_1	5010	Overbygning	1.200	2675.63
		G_2	5020	Superegenvekt	1.200	932.15
		-	10512	LM1:TS Lane 20/21/22 Step13	1.350	3011.22
			104	LM1:UDL Lane 20/21/22 Span2	1.350	796.51
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	1204.57
		ZW	24	+Vertikalt	1.120	132.88
200011	0.000		2301	MAX-MY BEAM ULSB_b		10745.78

Table 5.26 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.26: Required steel reinforcement, Model 20-2

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		310.20	1187.26	-	161.00
	200010	1.000	1.000	101	tens	tens		717.50	376.14	-	161.00
	200011	0.000	0.000	101	tens	tens		717.50	361.91	-	161.00
	300001	0.000	0.000	101	tens	tens		372.01	1509.03	-	161.00

DC design case   Xi Relative distance
BEAM beam element   NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 1509.03 = -552.03cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 717.50 = -171.5cm^2$$

The resulting remaining areas are negative, it is unlikely that a cross section with a height of 600mm will be able to withstand the design loads. This indicates that the design reinforcement areas exceed the maximum allowable reinforcement areas. Therefore, a span length of 20m with a cross-section depth of 600mm, resulting in a span-to-depth ratio of 33.34, is not acceptable when designing a reinforced concrete bridge that is 50m in length.

5.1.4.3 Model 20-3

Model 20-3 has a mid-span length of 20m, cross section height of 750mm, and a span-to-depth ratio of 26.667. Figure 5.16 and Figure 5.17 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

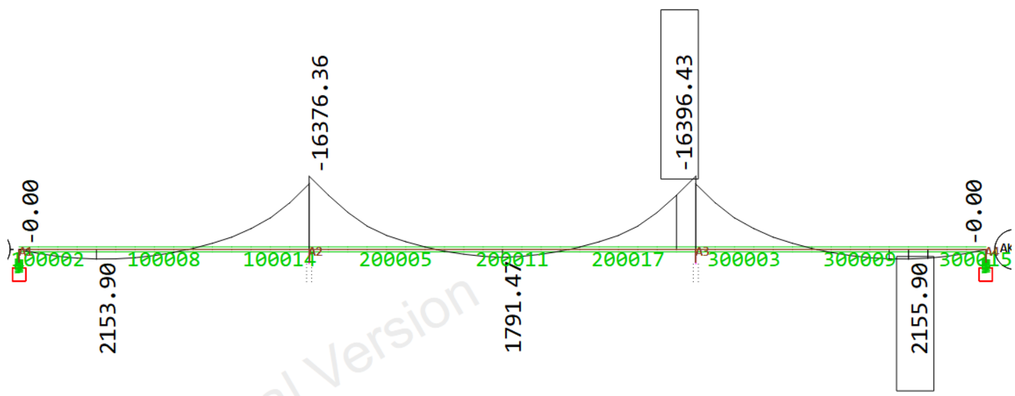


Figure 5.16: BMD My [kNm] for load combinations at middle supports, model 20-3

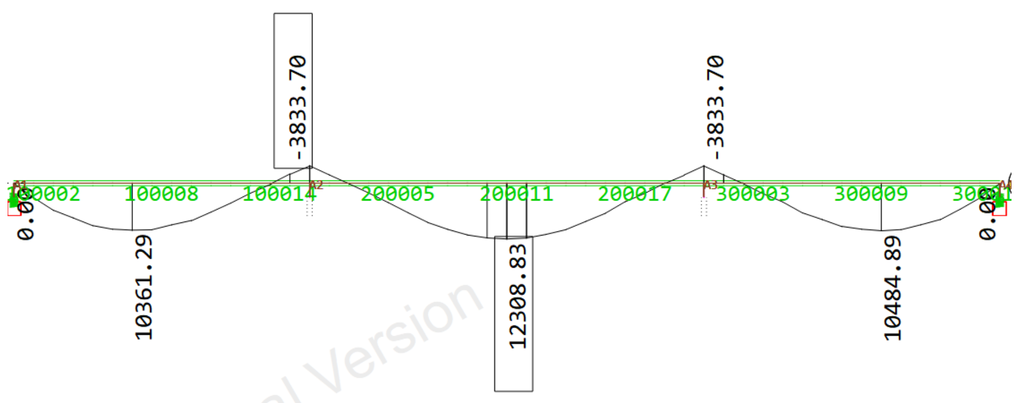


Figure 5.17: BMD My [kNm] for load combinations at middle span, model 20-3

Table 5.27 and Table 5.28 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.27: Load combination due to different load cases "LC" returning highest My at middle supports, model 20-3

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200020	1.000	G_1	5010	Overbygning	1.200	-5884.40
		G_2	5020	Superegenvekt	1.200	-1753.14
		-	10013	LM1:TS Lane 10/11/12 Step14	1.350	-2642.74
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-1072.75
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-373.96
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-1704.12
		ZW	24	+Vertikalt	1.120	-249.24
200020	1.000		2302	MIN-MY BEAM ULSB_b		-16396.43

Table 5.28: Load combination due to different load cases "LC" returning highest My at middle span, model 20-3

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200011	0.000	C	5030	Creep until t-infinite	1.350	112.47
			5031	Creep until t-infinite	1.350	30.25
			5032	Creep until t-infinite	1.350	1.67
		G_1	5010	Overbygning	1.200	3178.11
		G_2	5020	Superegenvekt	1.200	946.86
		-	10512	LM1:TS Lane 20/21/22 Step13	1.350	3162.67
			104	LM1:UDL Lane 20/21/22 Span2	1.350	862.25
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	1879.88
		ZW	24	+Vertikalt	1.120	135.01
200011	0.000		2301	MAX-MY BEAM ULSB_b		12308.83

Table 5.29 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.29: Required steel reinforcement, Model 20-3

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		288.52	952.20	-	161.00
	200010	1.000	1.000	101	tens	tens		627.66	349.95	-	161.00
	200011	0.000	0.000	101	tens	tens		627.66	340.87	-	161.00
	300001	0.000	0.000	101	tens	tens		311.00	873.42	-	161.00

DC design case   Xi Relative distance
BEAM beam element   NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 952.20 = 4.8cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 627.66 = -80.78cm^2$$

The positive remaining area over the middle support is acceptable for design purposes as it is within the allowable limit of ± 5% of the maximum allowable area. However, the remaining area at the middle span is negative, which indicates that the cross-section with a height of 750mm may not be able to withstand the design loads over the middle span. It is unlikely to use a cross-section with a depth of 750mm for a reinforced concrete bridge with a span length of 20m and a total length of 50m. This span-to-depth ratio of 26.67 is not acceptable for designing a reinforced concrete bridge. However, the design case over the middle supports is

$$\frac{\text{requirement R/F area (952.20 cm}^2)}{\text{the maximum allowable R/F area (957 cm}^2)} * 100 = 99.49\% \text{ of the max. allowable R/F area (ASL2)}$$

5.1.4.4 Model 20-4

Model 20-4 has a mid-span length of 20m, cross section height of 900mm, and a span-to-depth ratio of 22.223. Figure 5.18 and Figure 5.19 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

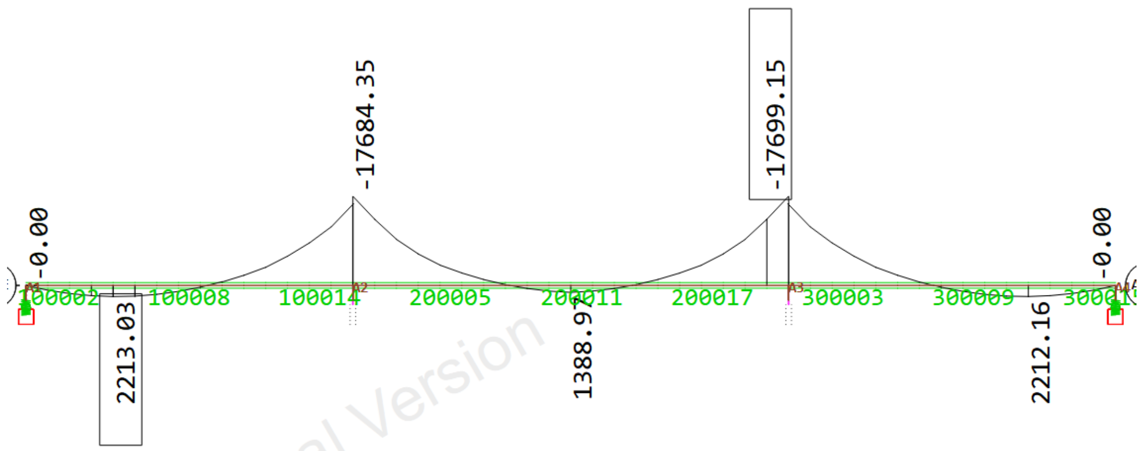


Figure 5.18: BMD My [kNm] for load combinations at middle supports, model 20-4

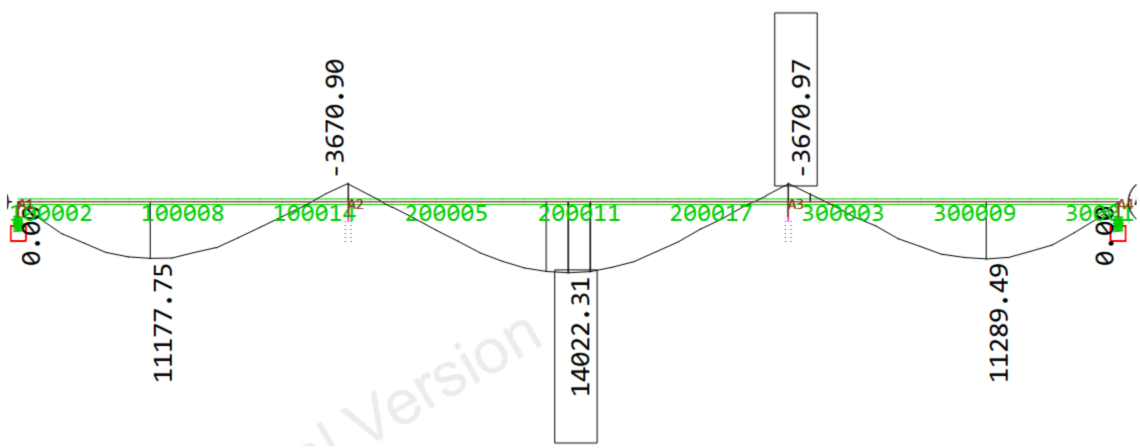


Figure 5.19: BMD My [kNm] for load combinations at middle span, model 20-4

Table 5.30 and Table 5.31 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.30: Load combination due to different load cases "LC" returning highest My at middle supports, model 20-4

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200020	1.000	G_1	5010	Overbygning	1.200	-6690.38
		G_2	5020	Superegenvekt	1.200	-1741.11
		-	10013	LM1:TS Lane 10/11/12 Step14	1.350	-2496.00
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-1015.77
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-413.23
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-2387.32
		ZW	24	+Vertikalt	1.120	-247.54
200020	1.000		2302	MIN-MY BEAM ULSB_b		-17699.14

Table 5.31: Load combination due to different load cases "LC" returning highest My at middle span, model 20-4

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200011	0.000	C	5030	Creep until t-infinite	1.350	162.25
			5031	Creep until t-infinite	1.350	60.63
			5032	Creep until t-infinite	1.350	0.50
		G_1	5010	Overbygning	1.200	3684.62
		G_2	5020	Superegenvekt	1.200	958.89
		-	10512	LM1:TS Lane 20/21/22 Step13	1.350	3293.64
			104	LM1:UDL Lane 20/21/22 Span2	1.350	919.23
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	2747.67
		ZW	24	+Vertikalt	1.120	136.70
200011	0.000		2301	MAX-MY BEAM ULSB_b		14022.31

Table 5.32 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.32: Required steel reinforcement, Model 20-4

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		271.26	827.69	-	134.16
	200010	1.000	1.000	101	tens	tens		583.61	328.32	-	134.16
	200011	0.000	0.000	101	tens	tens		583.61	319.26	-	134.16
	300001	0.000	0.000	101	tens	tens		281.21	770.14	-	134.16

DC design case   Xi Relative distance
BEAM beam element   NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 827.69 = 129.31cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 583.61 = -36.73cm^2$$

The remaining area at the middle span is negative, which indicates that the cross-section with a height of 900mm may not be able to withstand the design loads over the middle span. It is unlikely to use a cross-section with a depth of 900mm for a reinforced concrete bridge with a span length of 20m and a total length of 50m. This span-to-depth ratio of 22.23 is not acceptable for designing a reinforced concrete bridge. However, the design case over the middle supports is

$$\frac{\text{requirement R/F area } (827.69 \text{ cm}^2)}{\text{the maximum allowable R/F area } (957 \text{ cm}^2)} * 100 = 86.48\% \text{ of the max. allowable R/F area (ASL2)}$$

5.1.4.5 Model 20-5

Model 20-5 has a mid-span length of 20m, cross section height of 950mm, and a span-to-depth ratio of 21.05. Figure 5.20 and Figure 5.21 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

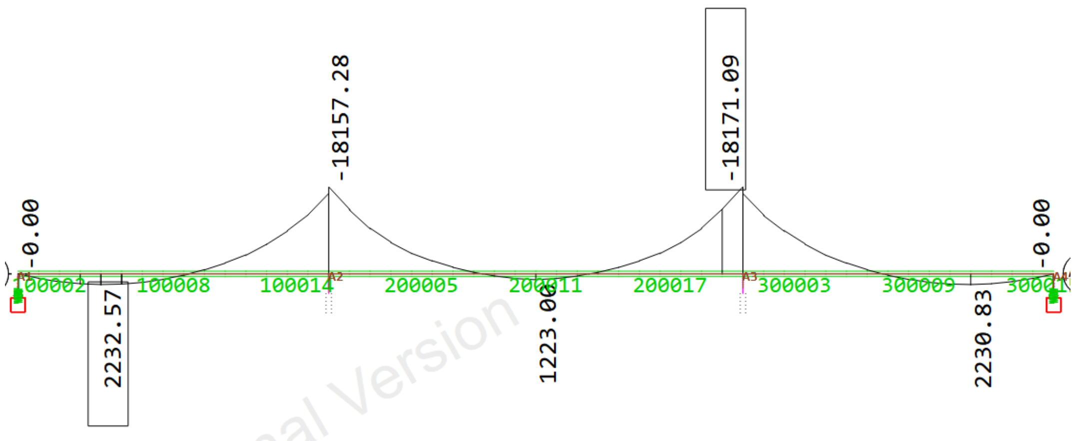


Figure 5.20: BMD My [kNm] for load combinations at middle supports, model 20-5

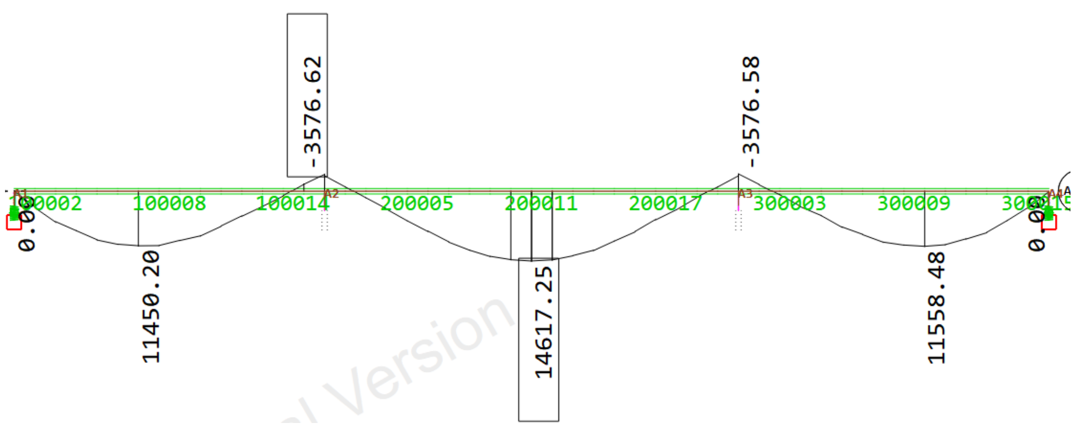


Figure 5.21: BMD My [kNm] for load combinations at middle span, model 20-5

Table 5.33 and Table 5.34 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.33: Load combination due to different load cases "LC" returning highest My at middle supports, model 20-5

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200020	1.000	G_1	5010	Overbygning	1.200	-6960.02
		G_2	5020	Superegenvekt	1.200	-1737.99
		-	10013	LM1:TS Lane 10/11/12 Step14	1.350	-2455.36
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-999.59
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-423.84
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-2643.25
		ZW	24	+Vertikalt	1.120	-247.11
200020	1.000		2302	MIN-MY BEAM ULSB_b		-18171.09

Table 5.34: Load combination due to different load cases "LC" returning highest My at middle span, model 20-5

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200011	0.000	C	5030	Creep until t-infinite	1.350	180.99
			5031	Creep until t-infinite	1.350	72.60
			5032	Creep until t-infinite	1.350	1.00
		G_1	5010	Overbygning	1.200	3852.48
		G_2	5020	Superegenvekt	1.200	962.01
		-	10512	LM1:TS Lane 20/21/22 Step13	1.350	3330.74
			104	LM1:UDL Lane 20/21/22 Span2	1.350	935.41
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	3075.33
		ZW	24	+Vertikalt	1.120	137.13
200011	0.000		2301	MAX-MY BEAM ULSB_b		14617.25

Table 5.35 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.35: Required steel reinforcement, Model 20-5

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		268.69	797.93	-	119.64
	200010	1.000	1.000	101	tens	tens		573.49	321.99	-	119.64
	200011	0.000	0.000	101	tens	tens		573.49	318.92	-	119.64
	300001	0.000	0.000	101	tens	tens		279.93	745.99	-	119.64

DC design case — Xi Relative distance
BEAM beam element — NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 797.93 = 159.07 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 573.49 = -26.61 cm^2$$

The negative remaining R/F area at the middle span is acceptable for design purposes as it falls within the allowable limit of $\pm$ 5% of the maximum allowable area ASL1. Furthermore, the remaining area at the middle span is positive, which suggests that a cross-section with a height of 950mm may be able to withstand the design loads over the middle span. Therefore, it is possible to use a cross-section with a depth of 950mm for a reinforced concrete bridge with a span length of 20m and a total length of 50m. This span-to-depth ratio of 21.05 is considered acceptable for designing a reinforced concrete bridge.

The design case over the middle span is:

$$\frac{\text{requirement R/F area } (573.49\text{cm}^2)}{\text{the maximum allowable R/F area } (546.88\text{ cm}^2)} * 100 = 104.86\% \text{ of the max. allowable R/F area (ASL1)}$$

5.1.5 Model 24

The Bridge model 24 is composed of a central span of 24m length and two side spans each of 18m length, resulting in a total length of 60m. Table 5.36 summarizes the sub-model of model 24 and its corresponding efficiency percentages in using the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.36: Sub-model of model 24

Model	Mid span length [m]	Total length [m]	Cross section depth [mm]	Span to depth ratio	ASL1	ASL2	% ASL1	% ASL2
Model 24-1	24	60	1500	16	Not ok	ok	122.34	93.12

5.1.5.1 Model 24-1

Model 24-1 has a mid-span length of 24m, cross section height of 1500mm, and a span-to-depth ratio of 16. Figure 5.22 and Figure 5.23 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

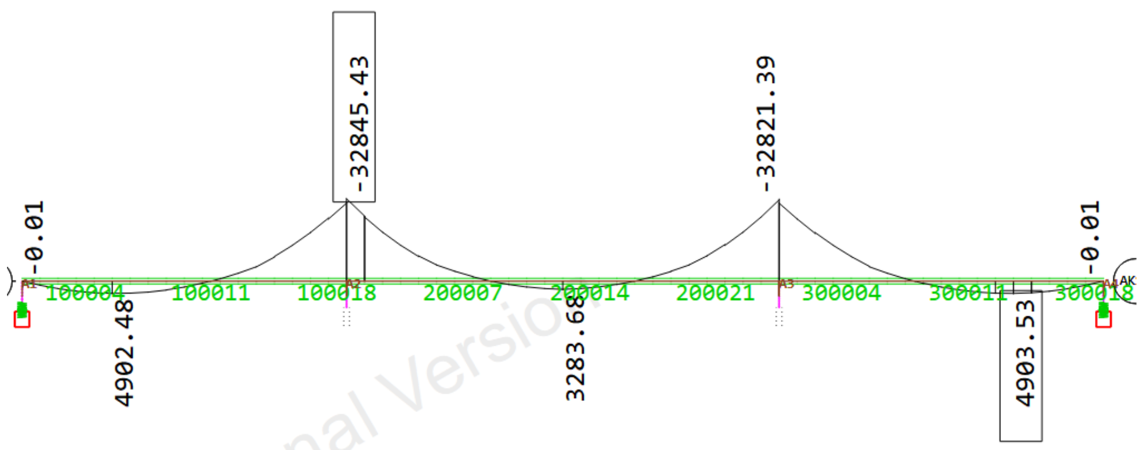


Figure 5.22: BMD My [kNm] for load combinations at middle supports, model 24-1

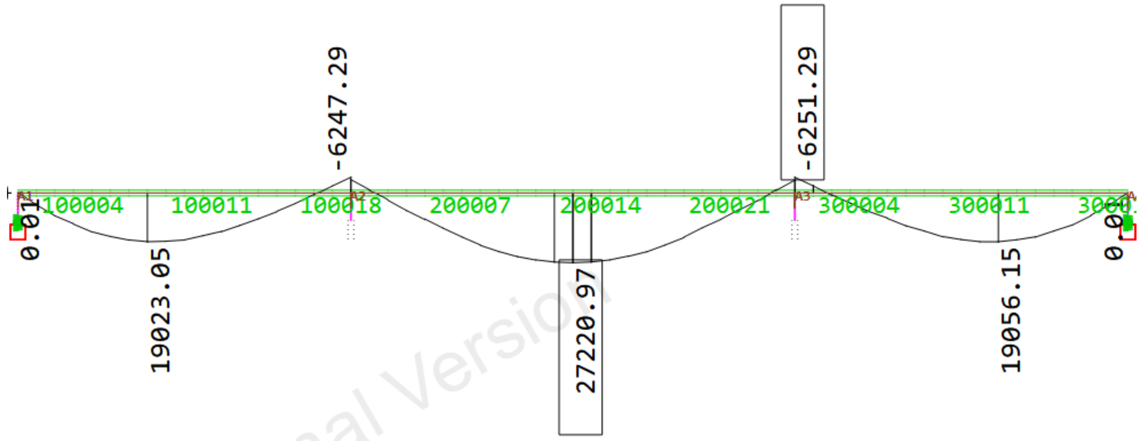


Figure 5.23: BMD My [kNm] for load combinations at middle span, model 24-1

Table 5.37 and Table 5.38 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.37: Load combination due to different load cases "LC" returning highest My at middle supports, model 24-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200001	0.000	G_1	5010	Overbygning	1.350	-14385.56
		G_2	5020	Superegenvekt	1.350	-2485.82
		-	10013	LM1:TS Lane 10/11/12 Step14	0.945	-2691.41
			100	LM1:UDL Lane 10/11/12 Span1	0.945	-700.03
			101	LM1:UDL Lane 10/11/12 Span2	0.945	-1308.10
		T	84	Tsummer+negdt +wn*TN+DTZ	0.840	-6228.40
		ZW	24	+Vertikalt	1.120	-353.71
200001	0.000		2202	MIN-MY BEAM ULSB_a		-32845.43

Table 5.38: Load combination due to different load cases "LC" returning highest My at middle span, model 24-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200013	0.000	C	5030	Creep until t-infinite	1.350	398.14
			5031	Creep until t-infinite	1.350	218.12
			5032	Creep until t-infinite	1.350	17.58
		G_1	5010	Overbygning	1.200	8114.45
		G_2	5020	Superegenvekt	1.200	1402.18
		-	10514	LM1:TS Lane 20/21/22 Step15	0.945	4309.79
			104	LM1:UDL Lane 20/21/22 Span2	0.945	1478.30
		T	90	Twinter+posdt -wn*TN-DTZ	1.200	7710.03
		ZW	24	+Vertikalt	1.120	199.61
200013	0.000		2301	MAX-MY BEAM ULSB_b		27220.97

Table 5.39 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.39: Required steel reinforcement, Model 24-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		238.02	891.19	-	93.91
	200012	0.000	0.000	101	tens	tens		662.12	318.92	-	93.91
	200013	0.000	0.000	101	tens	tens		669.02	318.92	-	93.91
	300001	0.000	0.000	101	tens	tens		241.86	856.96	-	93.91

DC design case — Xi Relative distance
BEAM beam element — NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 891.19 = 65.81 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 669.02 = -122.14 cm^2$$

The negative remaining area at the middle span suggests that a cross-section with a height of 1500mm may not be able to withstand the design loads over the middle span. Therefore, it is unlikely to use a cross-section with a depth of 1500mm for a reinforced concrete bridge with a span length of 24m and a total length of 60m. A span-to-depth ratio of 16 is deemed unacceptable for designing a reinforced concrete bridge. Therefore, any other span-to-depth ratios with cross-section heights ranging from 450mm to 1500mm are also considered unsuitable for designing the bridge in this manner. For these reasons, it is advisable to explore the use of a different type of cross-section or introduce prestressing to meet the design requirements.

5.2 Summary of Reinforced Concrete

Table 5.40 presents the results for mid-span length and the corresponding span-to-depth ratio. Meanwhile, Figure 5.24 showcases the findings on the depth of three-span plate reinforced concrete bridge models, considering mid-span lengths ranging from 20m to 50m. The x-axis of the figure represents the mid-span length in meters, while the y-axis represents the depth of the cross-section in millimeters.

The results demonstrate that the thinnest cross-section height that can withstand the applied loads is 450mm for short mid-spans ranging from 8m to 12m with span-to-depth ratios of 17.78 and 26.67. However, for longer mid-spans exceeding 12m, such as 16m and 20m with span-to-depth ratios of 29.1 and 21.05, respectively, the thinnest cross-section heights that can withstand the applied loads are 550mm and 950mm. Notably, the results show a non-linear slope between spans of 8m to 20m, which differs from the recommendations given by the Norwegian Road Administration in Handbook 4 (Statens Vegvesen 2000) and provided in Section 2.7.2. Nevertheless, the obtained ratios for middle span length between 8m and 16m are within the recommended ratios provided by Handbook 4, while the ratio obtained for span length of 20m differ from the recommended ratios. For a mid-span length of 24m, the cross-section with a height of 1500mm was unable to support its applied load, suggesting the need to consider a different type of cross-section or the implementation of a prestressing mechanism as an alternative solution. It should be mentioned that there is a significant difference in cross-section height for spans of 16m and 20m, which may introduce a source of error.

Table 5.40: Span length and span-to-depth ratio of reinforced concrete bridges

	Span length [m]	span-to-depth ratio
Model 8	8	17.778
Model 12	12	26.667
Model 16	16	29.10
Model 20	20	21.05

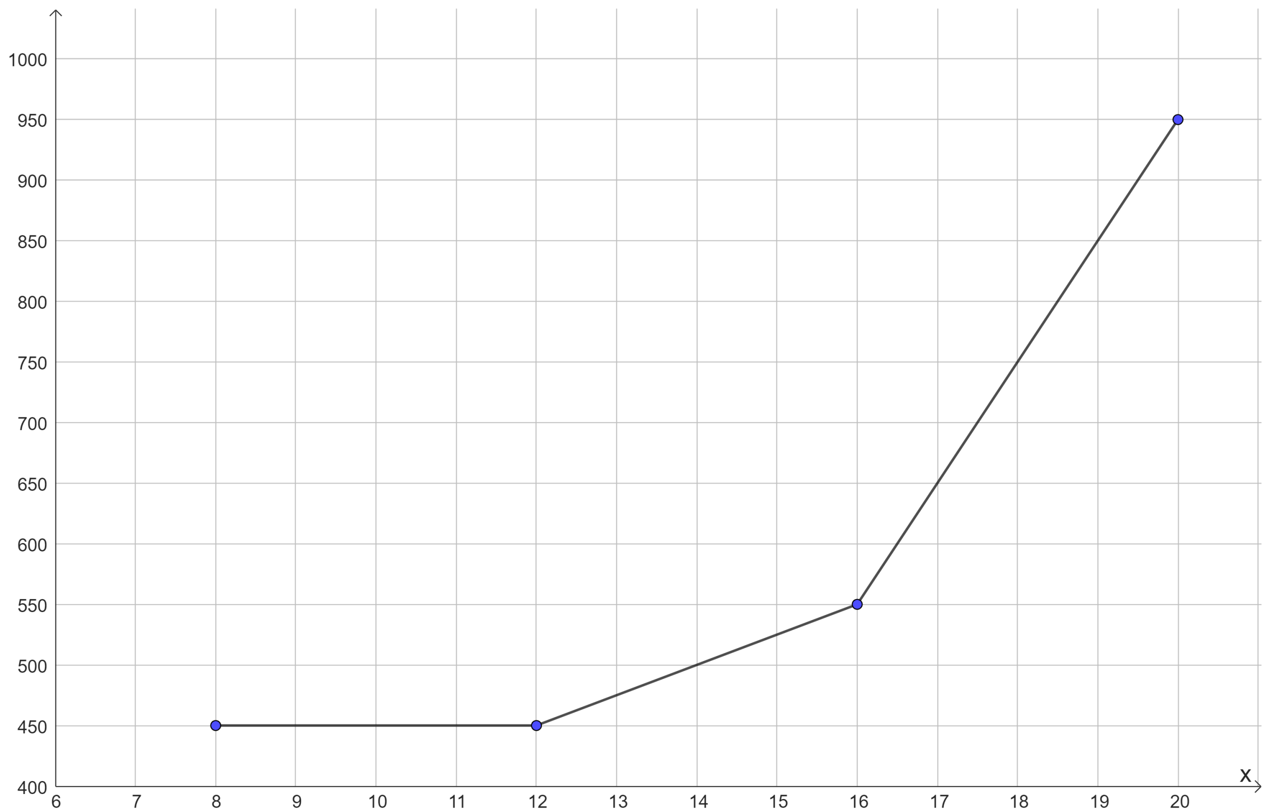


Figure 5.24: Mid-span length VS. depth of reinforced concrete bridges. The x-axis shows the mid-span length in meters, and the y-axis shows the depth of the cross-section in millimeters.

5.3 Design Values of Post-tensioned Concrete

All bridge models in this section are post-tensioned using the BBR VT CONA BT 2706-150-1860 prestressing system. The system uses 10 cables made up of 27 strands, each with a nominal cross-sectional area of 150 mm^2. The post-tensioned models have lengths ranging from 60m to 100m, starting with Model P24, which was previously modeled without prestressing and failed to carry its loads. Figure 5.25 depicts the prestressing model with tendon cables.

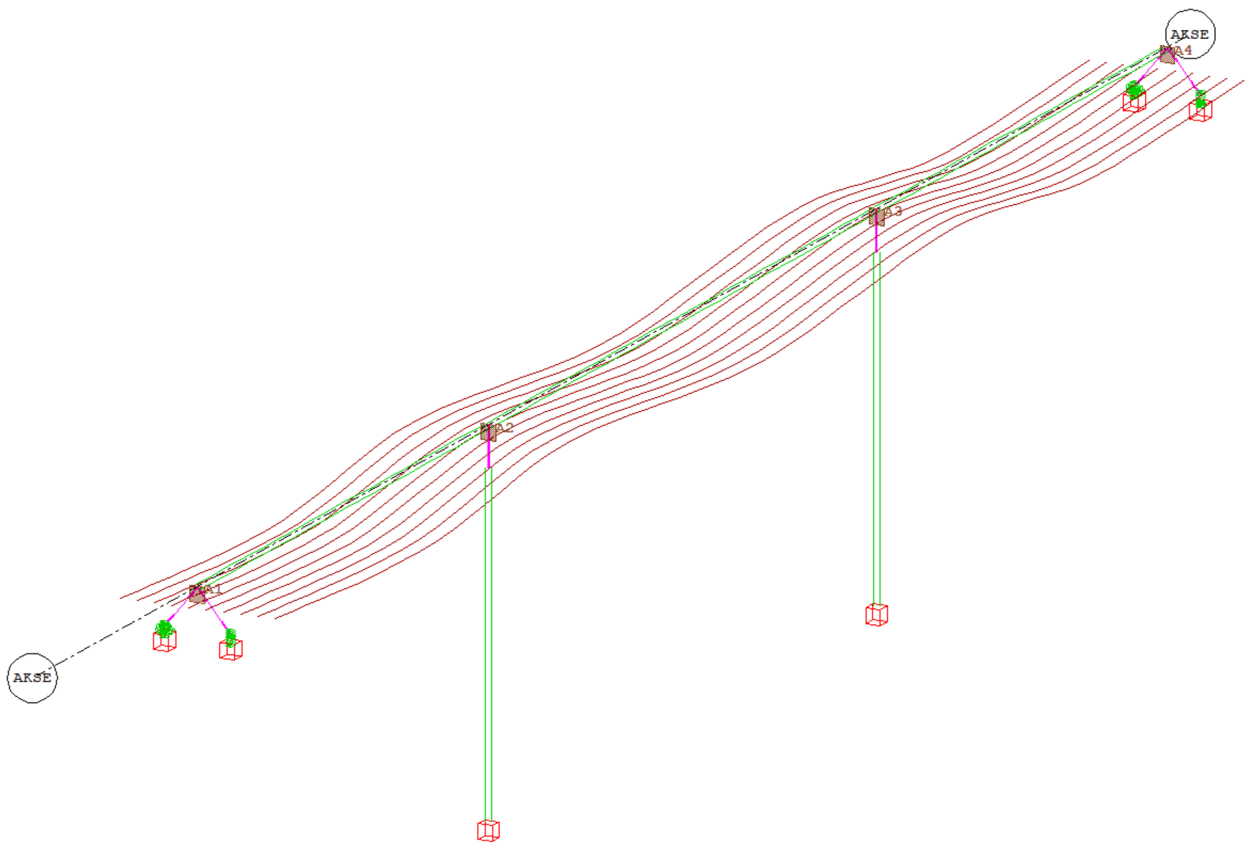


Figure 5.25: A bridge model with Post-tensioning tendons

5.3.1 Model P24

The prestressed bridge model P24 consists of a central span with a length of 24m and two side spans, each measuring 18m, resulting in a total length of 60m. Table 5.41 provides an overview of the sub-model for P24, including the efficiency percentages obtained when using longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Bridge models with cross-section heights ranging from 500mm to 650mm were tested for their applied load but failed to withstand it. For the sake of simplicity in presenting the results, these models are not included in this section. Consequently, there is a noticeable difference in cross-section height between 450mm and 750mm for Model P24.

Table 5.41: Sub-models of model p24

Model	**Mid span Length [m]**	**Total length [m]**	**C/S depth [mm]**	**L/H**	**ASL1**	**ASL2**	**% ASL1**	**% ASL2**	**Deco. Status**
Model p24-1	24	60	450	53.34	Not ok	Not ok	147.65	276.47	Not ok
Model p24-2	24	60	700	34.28	Ok	Ok	33.13	47.48	Ok

5.3.1.1 Model P24-1

Model P24-1 is a post-tensioned model that has a mid-span length of 24m, cross section height of 450mm, and a span-to-depth ratio of 53.34. Figure 5.26 and Figure 5.27 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

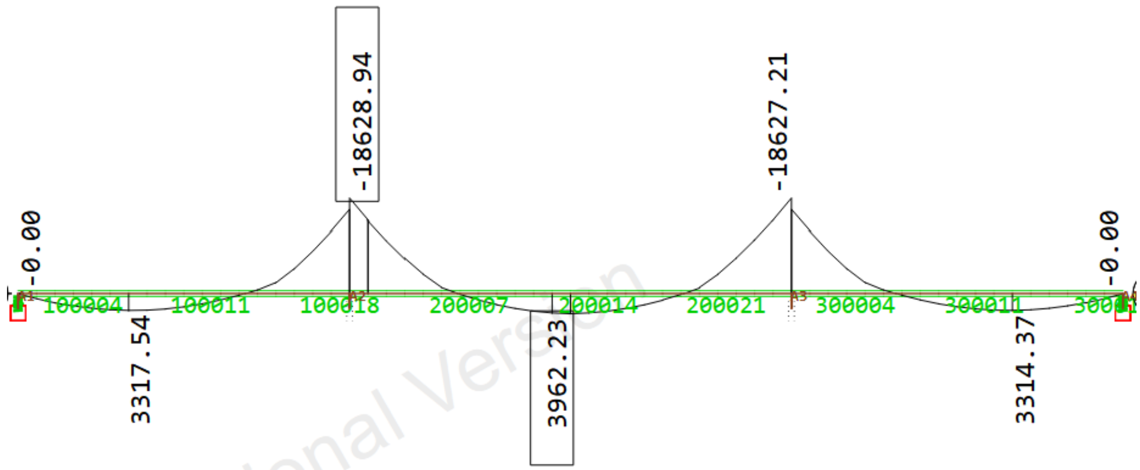


Figure 5.26: BMD My [kNm] for load combinations at middle supports, model p24-1

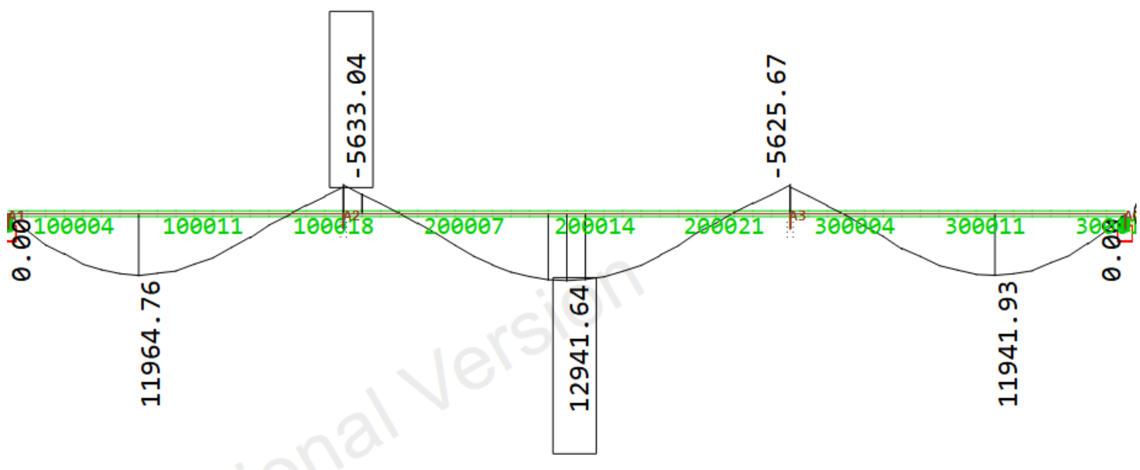


Figure 5.27: BMD My [kNm] for load combinations at middle span, model p24-1

Table 5.42 and Table 5.43 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.42: Load combination due to different load cases "LC" returning highest My at middle supports, model p24-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200001	0.000	C	5030	Creep until t-infinite	1.350	22.06
			5031	Creep until t-infinite	1.350	-32.05
			5032	Creep until t-infinite	1.350	-4.45
		G_1	5010	Overbygning	1.200	-6123.86
		G_2	5020	Superegenvekt	1.200	-2568.45
		P	16015	Secondary effect CS 15	1.000	633.69
		-	10013	LM1:TS Lane 10/11/12 Step14	1.350	-3671.70
			100	LM1:UDL Lane 10/11/12 Span1	1.350	-336.02
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-1743.24
		T	84	Tsummer+negdt +wn*TN+DTZ	0.840	-761.18
		ZW	24	+Vertikalt	1.120	-365.33
200001	0.000		2302	MIN-MY BEAM ULSB_b		-18628.94

Table 5.43: Load combination due to different load cases "LC" returning highest My at middle span, model p24-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200013	0.000	G_1	5010	Overbygning	1.200	3146.14
		G_2	5020	Superegenvekt	1.200	1319.55
		P	16015	Secondary effect CS 15	1.000	637.22
		-	10514	LM1:TS Lane 20/21/22 Step15	1.350	3474.47
			104	LM1:UDL Lane 20/21/22 Span2	1.350	1043.16
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	757.43
		ZW	24	+Vertikalt	1.120	187.99
200013	0.000		2301	MAX-MY BEAM ULSB_b		12941.64

Table 5.44 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.44: Required steel reinforcement, Model p24-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		1808.74	2645.83	-	161.00
	200013	1.000	1.000	101	tens	tens		807.48	849.97	-	161.00
	300001	0.000	0.000	101	tens	tens		1440.61	2155.29	-	161.00

DC design case
BEAM beam element
X distance from start
Xi Relative distance
NQ section number
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 2645.83 = -1688.83cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 807.48 = -260.6cm^2$$

The negative remaining areas at the middle span and over supports suggest that a cross-section with a height of 450mm will not be able to withstand the design loads over the bridge model. Therefore, the cross-section with a depth of 450mm wouldn't be suitable for a post-tensioned concrete bridge with a middle span length of 24m and a total length of 60m. The span-to-depth ratio of 53.34 is considered unacceptable for designing a post-tensioned concrete bridge.

Figure 5.28 shows the maximum decompression strain around the prestressing steel and the duct for the post-tensioned tendons. It appears that there is positive decompression strain observed over the middle supports and at the middle span, indicating that the duct for post-tensioned tendons is not located 10mm within the compression zone. This finding confirms that a cross-section with a depth of 450mm would not have the necessary strength to withstand the applied load after post-tensioning.

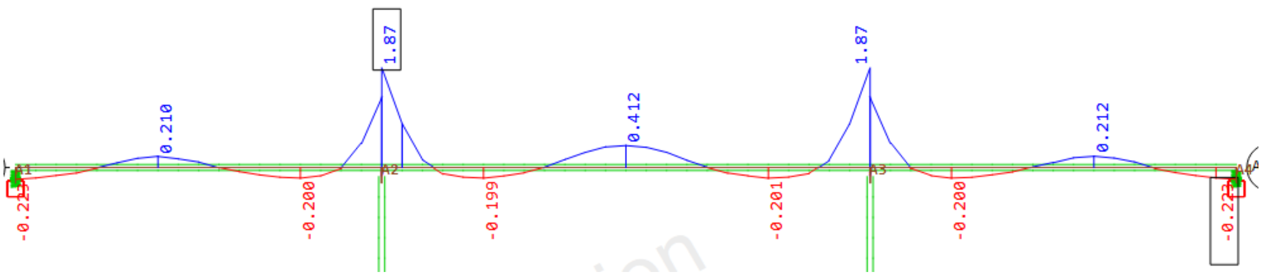


Figure 5.28: Maximum decompression strain at middle supports and at the middle span, model p24-1

5.3.1.2 Model P24-2

Model P24-2 is a post-tensioned model that has a mid-span length of 24m, cross section height of 700mm, and a span-to-depth ratio of 34.28. Figure 5.29 and Figure 5.30 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

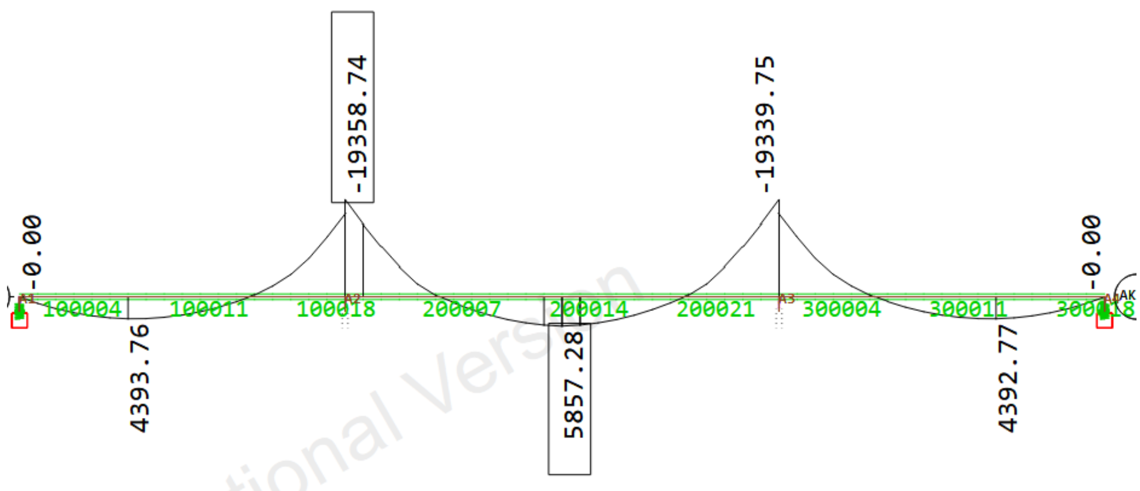


Figure 5.29: BMD My [kNm] for load combinations at middle supports, model p24-2

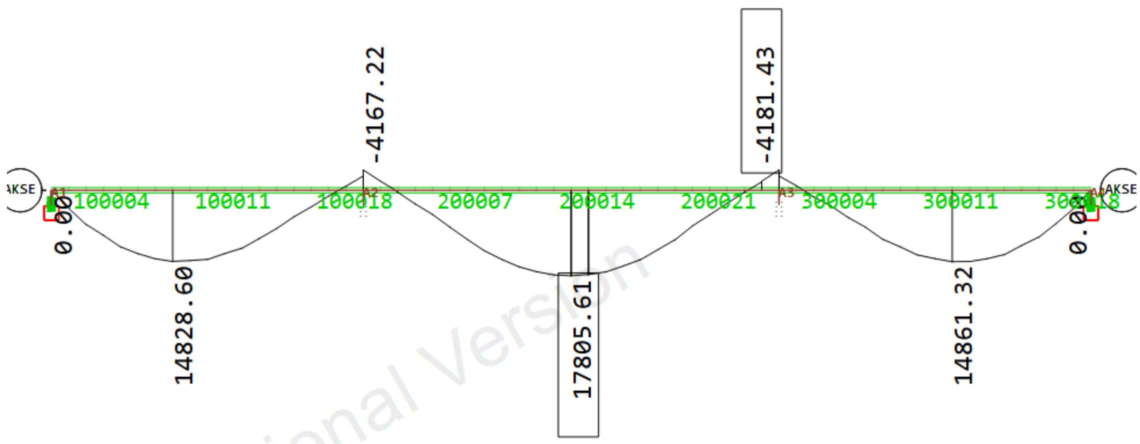


Figure 5.30: BMD My [kNm] for load combinations at middle span, model p24-2

Table 5.45 and Table 5.46 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.45: Load combination due to different load cases "LC" returning highest My at middle supports, model p24-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200001	0.000	C	5030	Creep until t-infinite	1.350	-153.43
			5031	Creep until t-infinite	1.350	-70.95
			5032	Creep until t-infinite	1.350	-78.73
		G_1	5010	Overbygning	1.200	-8107.04
		G_2	5020	Superegenvekt	1.200	-2537.85
		P	16015	Secondary effect CS 15	1.000	2790.07
		-	10013	LM1:TS Lane 10/11/12 Step14	1.350	-3315.36
			100	LM1:UDL Lane 10/11/12 Span1	1.350	-494.04
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-1604.16
		T	84	Tsummer+negdt +wn*TN+DTZ	0.840	-1492.06
		ZW	24	+Vertikalt	1.120	-360.83
200001	0.000		2302	MIN-MY BEAM ULSB_b		-19358.73

Table 5.46: Load combination due to different load cases "LC" returning highest My at middle span, model p24-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200013	0.000	G_1	5010	Overbygning	1.200	4312.97
		G_2	5020	Superegenvekt	1.200	1350.15
		P	16015	Secondary effect CS 15	1.000	2783.09
		-	10514	LM1:TS Lane 20/21/22 Step15	1.350	3742.78
			104	LM1:UDL Lane 20/21/22 Span2	1.350	1182.24
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	1621.92
		ZW	24	+Vertikalt	1.120	192.49
200013	0.000		2301	MAX-MY BEAM ULSB_b		17805.61

Table 5.47 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.47: Required steel reinforcement, Model p24-2

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		181.17	454.44	-	161.00
	200013	1.000	1.000	101	tens	tens		181.17	318.92	-	161.00
	300001	0.000	0.000	101	tens	tens		181.17	404.48	-	161.00

DC design case Xi Relative distance
BEAM beam element NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 454.44 = 502.56 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 181.17 = 365.71 cm^2$$

The remaining area at the middle span and at supports is positive, which indicates that a cross-section with a height of 700mm is able to withstand the design loads over the bridge model. Therefore, it is possible to use a cross-section with a depth of 700mm for a post-tensioned concrete bridge with a span length of 24m and a total length of 60m. This span-to-depth ratio of 34.28 is considered acceptable for designing a post-tensioned concrete bridge.

The design case is at mid-supports:

$$\frac{\text{requirement R/F area } (454.44\text{cm}^2)}{\text{the maximum allowable R/F area } (957\text{ cm}^2)} * 100 = 47.48\% \text{ of the max. allowable R/F area (ASL2)}$$

Figure 5.31 shows the maximum decompression strain around the prestressing steel and the duct for the post-tensioned tendons. It can be seen that the decompression strain over the middle supports and at the middle span is negative, indicating that the duct for post-tensioned tendons is located 10mm within the compression zone. This confirms that a cross-section with a depth of 700mm would be able to withstand the applied load after post-tensioning.

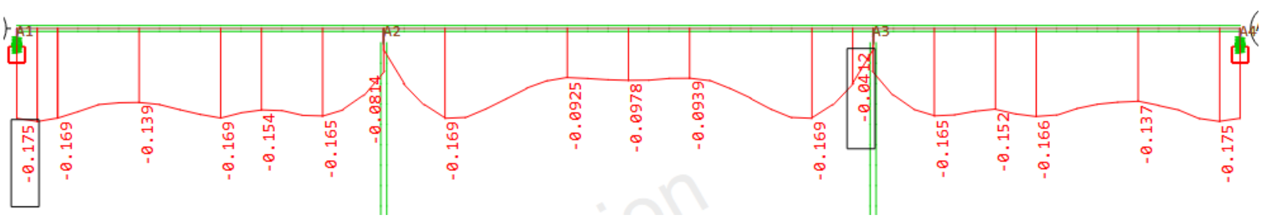


Figure 5.31: Maximum decompression strain at middle supports and at the middle span, model p24-2

5.3.2 Model P28

The prestressed bridge model P28 is made of a central span of 28m length and two side spans each of 21m, resulting in a total length of 70m. Table 5.48 summarizes the sub-models for model p28 and their corresponding efficiency percentages in using the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.48: Sub-models of model p28

Model	**Mid span Length [m]**	**Total length [m]**	**C/S depth [mm]**	**L/H**	**ASL1**	**ASL2**	**% ASL1**	**% ASL2**	**Deco. Status**
Model p28-1	28	70	800	35	Ok	Ok	33.12	43.62	Not ok
Model p28-2	28	70	850	32.94	Ok	Ok	33.13	37.82	Ok

5.3.2.1 Model p28-1

Model p28-1 is a post-tensioned model that has a mid-span length of 28m, cross section height of 800mm, and a span-to-depth ratio of 35. Figure 5.32 and Figure 5.33 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

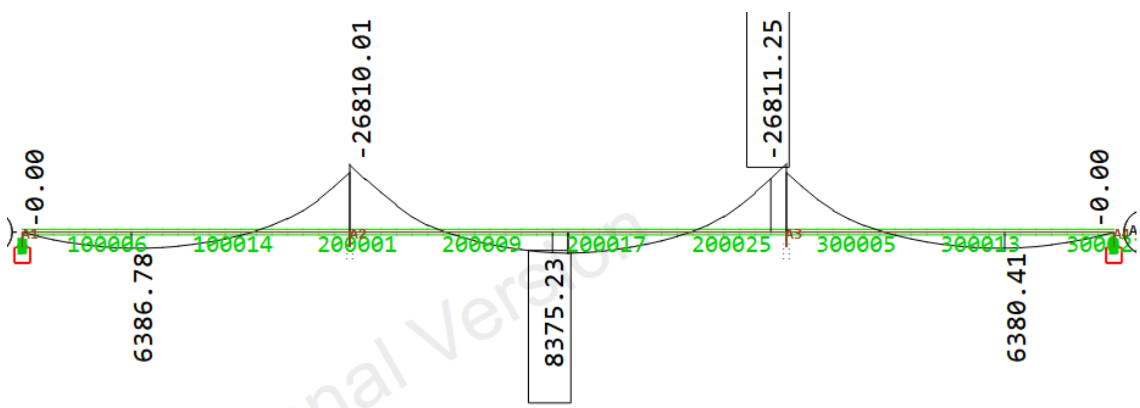


Figure 5.32: BMD My [kNm] for load combinations at middle supports, model p28-1

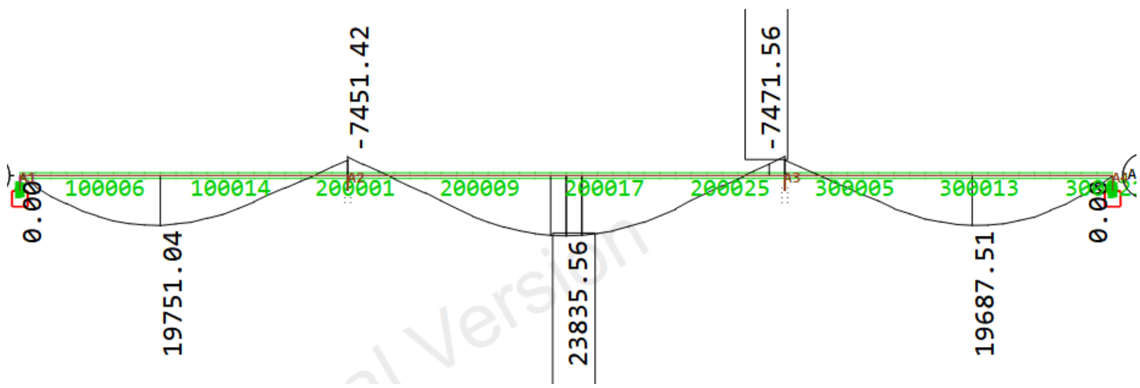


Figure 5.33: BMD My [kNm] for load combinations at middle span, model p28-1

Table 5.49 and Table 5.50 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.49: Load combination due to different load cases "LC" returning highest My at middle supports, model p28-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200028	1.000	C	5030	Creep until t-infinite	1.350	-167.47
			5031	Creep until t-infinite	1.350	-81.70
			5032	Creep until t-infinite	1.350	-98.21
		G_1	5010	Overbygning	1.200	-12118.61
		G_2	5020	Superegenvekt	1.200	-3444.24
		P	16015	Secondary effect CS 15	1.000	3414.16
		-	10018	LM1:TS Lane 10/11/12 Step19	1.350	-3776.52
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-2136.89
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-712.39
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-1890.19
		ZW	24	+Vertikalt	1.120	-489.68
200028	1.000		2302	MIN-MY BEAM ULSB_b		-26811.25

Table 5.50: Load combination due to different load cases "LC" returning highest My at middle span, model p28-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200015	0.000	G_1	5010	Overbygning	1.200	6501.40
		G_2	5020	Superegenvekt	1.200	1847.77
		P	16015	Secondary effect CS 15	1.000	3421.33
		-	10517	LM1:TS Lane 20/21/22 Step18	1.350	4503.09
			104	LM1:UDL Lane 20/21/22 Span2	1.350	1655.71
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	2125.96
		ZW	24	+Vertikalt	1.120	263.44
200015	0.000		2301	MAX-MY BEAM ULSB_b		23835.56

Table 5.51 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.51: Required steel reinforcement, Model p28-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		181.17	417.45	-	161.00
	200015	0.000	0.000	101	tens	tens		181.17	318.92	-	161.00
	200028	1.000	1.000	101	tens	tens		181.17	410.91	-	161.00

DC design case
Xi Relative distance
BEAM beam element
NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 417.45 = 539.55cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 181.17 = 365.71cm^2$$

The remaining area in the middle span and at the supports is positive, indicating that a post-tensioned cross-section with a height of 800mm could withstand the design loads on the bridge model. However, Figure 5.34 illustrates the maximum decompression strain around the prestressing steel and duct for the post-tensioned tendons. It is evident that the decompression strain at the middle supports is positive, indicating that the prestressing steel and duct for post-tensioned tendons are not situated 10mm within the compression zone. Therefore, the cross-section of height 800 is not suitable for withstanding the applied loads.

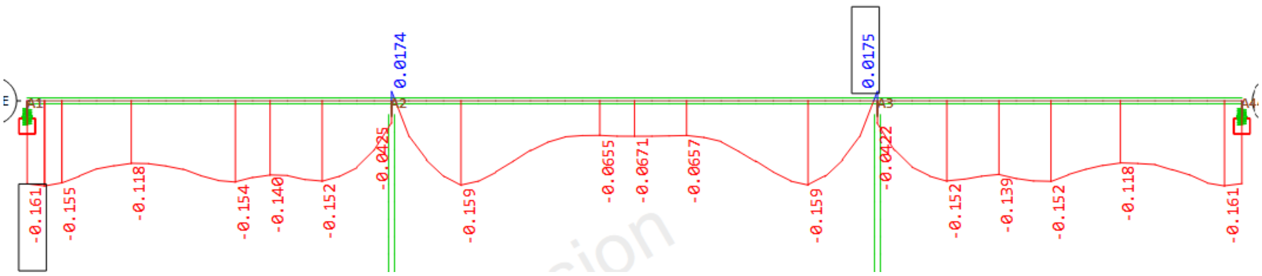


Figure 5.34: Maximum decompression strain at middle supports and at the middle span, model p28-1

5.3.2.2 Model p28-2

Model p28-2 is a post-tensioned model that has a mid-span length of 28m, cross section height of 850mm, and a span-to-depth ratio of 32.94. Figure 5.35 and Figure 5.36 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

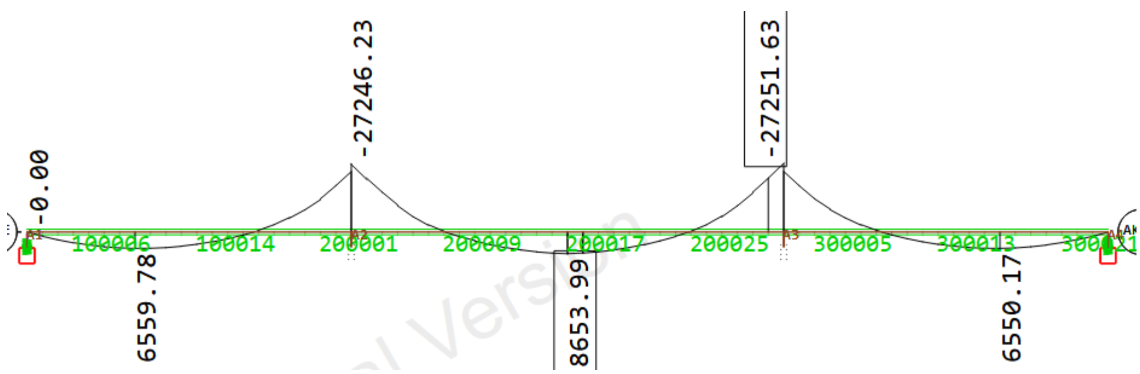


Figure 5.35: BMD My [kNm] for load combinations at middle supports, model p28-2

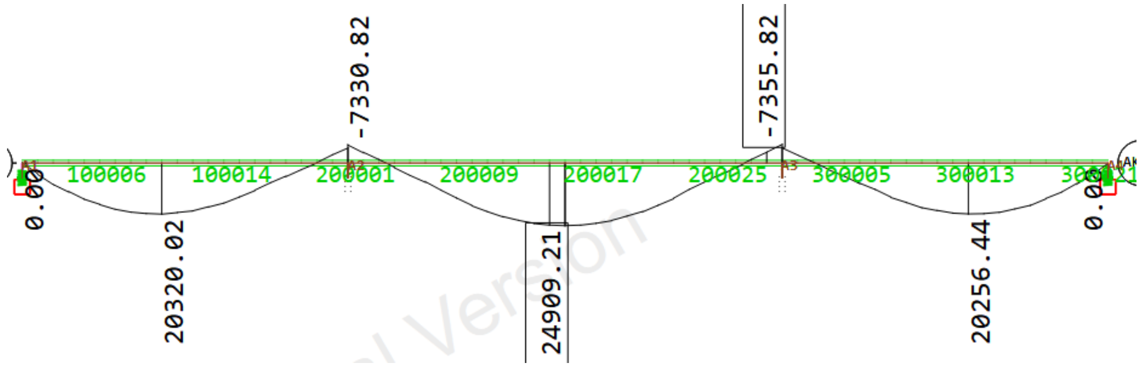


Figure 5.36: BMD My [kNm] for load combinations at middle span, model p28-2

Table 5.52 and Table 5.53 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.52: Load combination due to different load cases "LC" returning highest My at middle supports, model p28-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200028	1.000	C	5030	Creep until t-infinite	1.350	-169.68
			5031	Creep until t-infinite	1.350	-89.83
			5032	Creep until t-infinite	1.350	-106.79
		G_1	5010	Overbygning	1.200	-12646.92
		G_2	5020	Superegenvekt	1.200	-3436.14
		P	16015	Secondary effect CS 15	1.000	3707.43
		-	10018	LM1:TS Lane 10/11/12 Step19	1.350	-3708.02
			101	LM1:UDL Lane 10/11/12 Span2	1.350	-2100.01
			102	LM1:UDL Lane 10/11/12 Span3	1.350	-740.34
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-2116.01
		ZW	24	+Vertikalt	1.120	-488.52
200028	1.000		2302	MIN-MY BEAM ULSB_b		-27251.63

Table 5.53: Load combination due to different load cases "LC" returning highest My at middle span, model p28-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200015	0.000	G_1	5010	Overbygning	1.200	6830.58
		G_2	5020	Superegenvekt	1.200	1855.86
		P	16015	Secondary effect CS 15	1.000	3716.98
		-	10517	LM1:TS Lane 20/21/22 Step18	1.350	4564.04
			104	LM1:UDL Lane 20/21/22 Span2	1.350	1692.59
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	2411.53
		ZW	24	+Vertikalt	1.120	264.60
200015	0.000		2301	MAX-MY BEAM ULSB_b		24909.21

Table 5.54 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.54: Required steel reinforcement, Model p28-2

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		181.17	359.56	-	148.48
	200015	0.000	0.000	101	tens	tens		181.17	318.92	-	148.48
	200028	1.000	1.000	101	tens	tens		181.17	361.93	-	148.48

DC design case — Xi Relative distance
BEAM beam element — NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 361.93 = 595.07 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 181.17 = 365.71 cm^2$$

The remaining area in the middle span and at the supports is positive, indicating that a post-tensioned cross-section with a height of 850mm could withstand the design loads on the bridge

model. Therefore, it is possible to use a cross-section with a depth of 850mm for a post-tensioned concrete bridge with a middle span length of 28m and a total length of 70m. This span-to-depth ratio of 32.94 is considered acceptable for designing a post-tensioned concrete bridge.
The design case is at mid-supports:

$$\frac{\text{requirement R/F area } (361.93\text{cm}^2)}{\text{the maximum allowable R/F area } (957\text{ cm}^2)} * 100 = 37.82\% \text{ of the max. allowable R/F area (ASL2)}$$

Figure 5.37 shows the maximum decompression strain around the prestressing steel and the duct for the post-tensioned tendons. It can be seen that the decompression strain over the middle supports and at the middle span is negative, indicating that the prestressing steel and duct for post-tensioned tendons are situated 10mm within the compression zone. This confirms that a cross-section with a depth of 850mm would be able to withstand the applied load after post-tensioning.

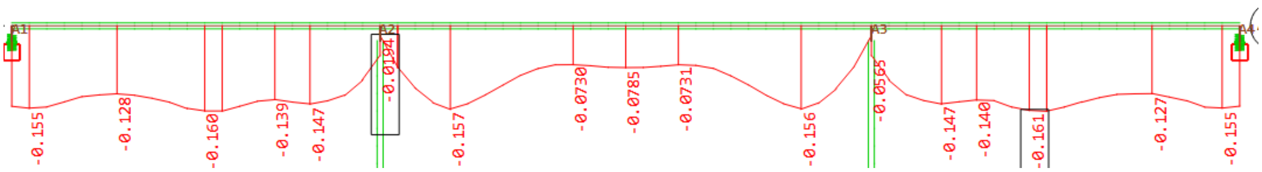


Figure 5.37: Maximum decompression strain at middle supports and at the middle span, model p28-2

5.3.3 Model P32

The prestressed bridge model P32 is made of a central span of 32m length and two side spans each of 24m, resulting in a total length of 80m. Table 5.55 summarizes the sub-models for model p32 and their corresponding efficiency percentages in using the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.55: Sub-models of model p32

Model	Mid span Length [m]	Total length [m]	C/S depth [mm]	L/H	ASL1	ASL2	% ASL1	% ASL2	Deco. Status
Model p32-1	32	80	1050	30.47	Ok	Ok	33.12	39.10	Not ok
Model p32-2	32	80	1100	29.1	Ok	Ok	33.13	43.14	Ok

5.3.3.1 Model p32-1

Model p32-1 is a post-tensioned model that has a mid-span length of 32m, cross section height of 1050mm, and a span-to-depth ratio of 30.47. Figure 5.38 and Figure 5.39 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

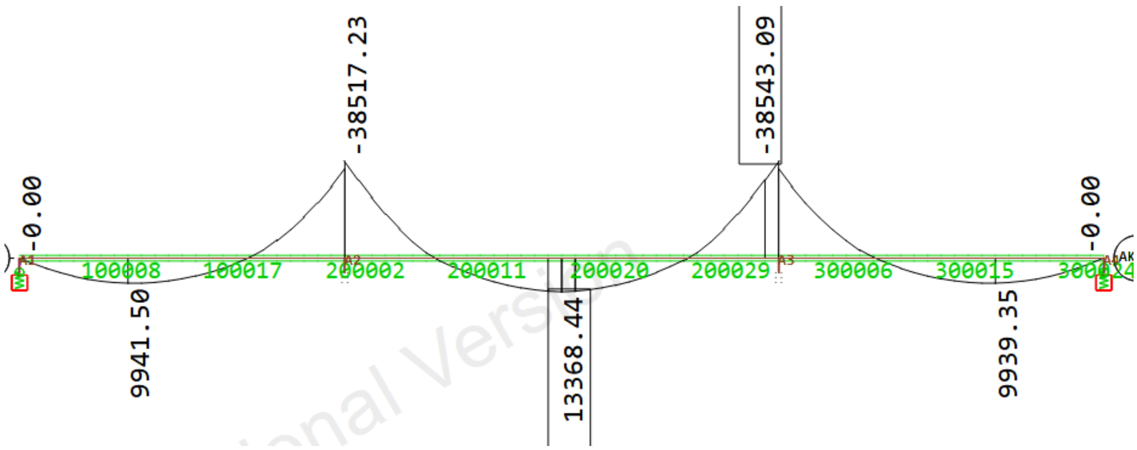


Figure 5.38: BMD My [kNm] for load combinations at middle supports, model p32-1

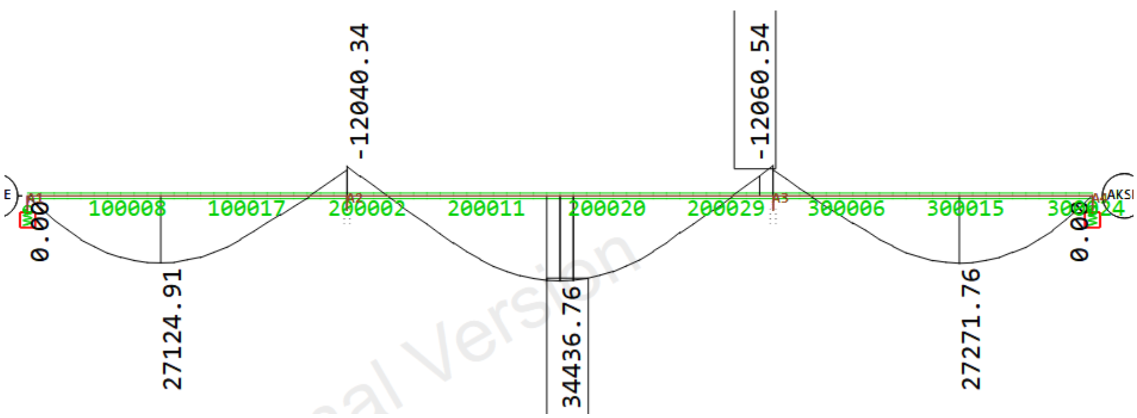


Figure 5.39: BMD My [kNm] for load combinations at middle span, model p32-1

Table 5.56 and Table 5.57 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.56: Load combination due to different load cases "LC" returning highest My at middle supports, model p32-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200017	0.000	G_1	5010	Overbygning	1.200	10610.83
		G_2	5020	Superegenvekt	1.200	2451.27
		P	16015	Secondary effect CS 15	1.000	4731.46
		-	10519	LM1:TS Lane 20/21/22 Step20	1.350	5446.17
			104	LM1:UDL Lane 20/21/22 Span2	1.350	2338.39
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	3726.46
		ZW	24	+Vertikalt	1.120	349.45
200017	0.000		2301	MAX-MY BEAM ULSB_b		34436.76

Table 5.57: Load combination due to different load cases "LC" returning highest My at middle span, model p32-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200032	1.000	C	5030	Creep until t-infinite	1.350	-165.54
			5031	Creep until t-infinite	1.350	-93.38
			5032	Creep until t-infinite	1.350	-135.46
		G_1	5010	Overbygning	1.350	-19309.17
		G_2	5020	Superegenvekt	1.350	-4460.72
		P	16015	Secondary effect CS 15	1.000	4723.86
		-	10021	LM1:TS Lane 10/11/12 Step22	0.945	-4047.80
			101	LM1:UDL Lane 10/11/12 Span2	0.945	-2615.21
			102	LM1:UDL Lane 10/11/12 Span3	0.945	-1057.82
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-3141.28
		ZW	24	+Vertikalt	1.120	-634.22
200032	1.000		2202	MIN-MY BEAM ULSB_a		-38543.09

Table 5.58 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.58: Required steel reinforcement, Model p32-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		181.17	365.68	-	114.85
	200017	0.000	0.000	101	tens	tens		181.17	318.92	-	93.91
	300001	0.000	0.000	101	tens	tens		181.17	374.19	-	106.66

DC design case
Xi Relative distance
BEAM beam element
NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 374.19 = 582.81 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 181.17 = 365.71 cm^2$$

The remaining area in the middle span and at the supports is positive, indicating that a post-tensioned cross-section with a height of 1050mm could withstand the design loads on the bridge model. However, Figure 5.40 illustrates the maximum decompression strain around the prestressing steel and duct for the post-tensioned tendons. It is evident that the decompression strain at the middle supports is positive, indicating that the prestressing steel and duct for post-tensioned tendons are not situated 10mm within the compression zone. Therefore, the cross-section of height 1050 is not suitable for withstanding the applied loads.

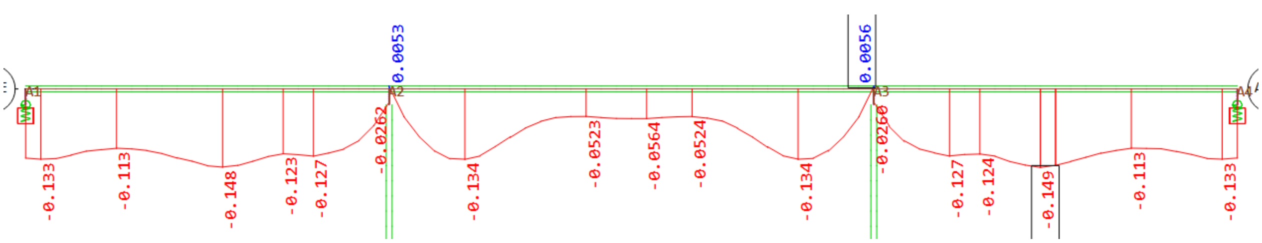


Figure 5.40: Maximum decompression strain at middle supports and at the middle span, model p32-1

5.3.3.2 Model p32-2

Model p32-2 is a post-tensioned model that has a mid-span length of 32m, cross section height of 1100mm, and a span-to-depth ratio of 29.1. Figure 5.41 and Figure 5.42 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

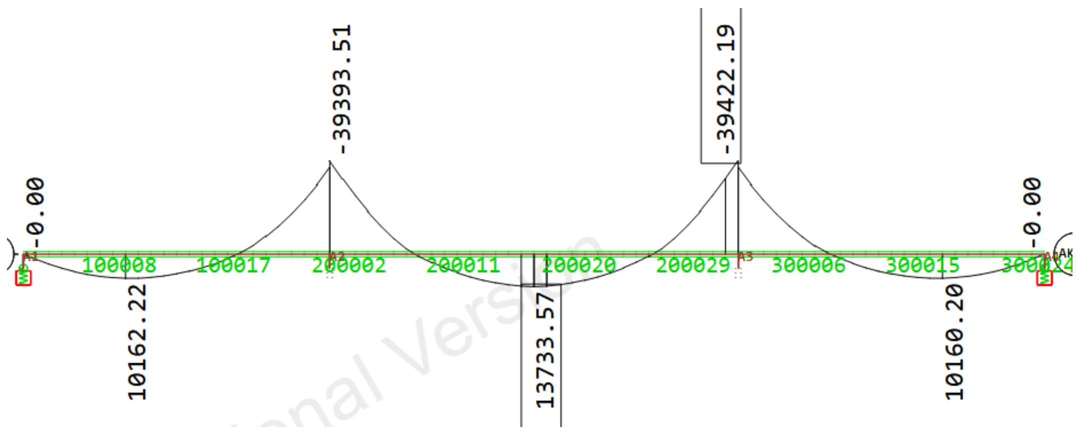


Figure 5.41: BMD My [kNm] for load combinations at middle supports, model p32-2

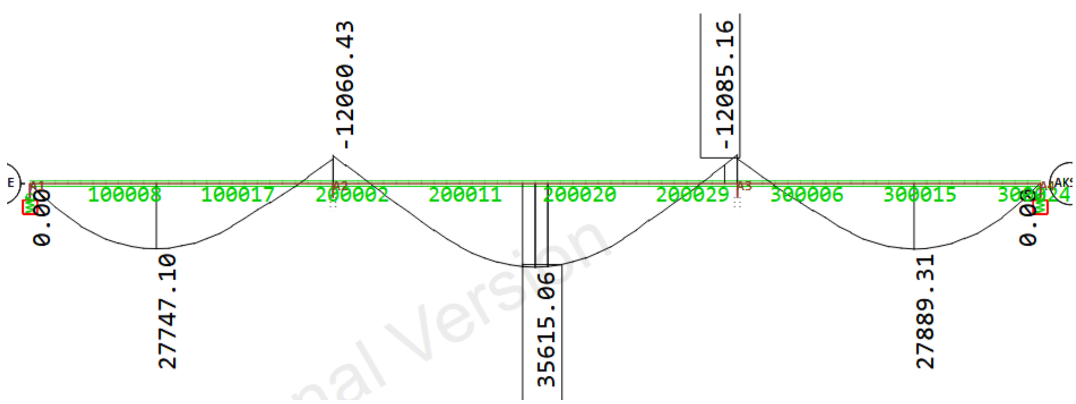


Figure 5.42: BMD My [kNm] for load combinations at middle span, model p32-2

Table 5.59 and Table 5.60 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.59: Load combination due to different load cases "LC" returning highest My at middle supports, model p32-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200032	1.000	C	5030	Creep until t-infinite	1.350	-160.27
			5031	Creep until t-infinite	1.350	-93.80
			5032	Creep until t-infinite	1.350	-140.56
		G_1	5010	Overbygning	1.350	-19999.45
		G_2	5020	Superegenvekt	1.350	-4453.49
		P	16015	Secondary effect CS 15	1.000	4947.55
		-	10021	LM1:TS Lane 10/11/12 Step22	0.945	-3994.48
			101	LM1:UDL Lane 10/11/12 Span2	0.945	-2580.87
			102	LM1:UDL Lane 10/11/12 Span3	0.945	-1080.13
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-3430.82
		ZW	24	+Vertikalt	1.120	-633.22
200032	1.000		2202	MIN-MY BEAM ULSB_a		-39422.20

Table 5.60: Load combination due to different load cases "LC" returning highest My at middle span, model p32-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200017	0.000	G_1	5010	Overbygning	1.200	11040.55
		G_2	5020	Superegenvekt	1.200	2458.52
		P	16015	Secondary effect CS 15	1.000	4957.51
		-	10519	LM1:TS Lane 20/21/22 Step20	1.350	5495.83
			104	LM1:UDL Lane 20/21/22 Span2	1.350	2372.73
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	4099.53
		ZW	24	+Vertikalt	1.120	350.46
200017	0.000		2301	MAX-MY BEAM ULSB_b		35615.06

Table 5.61 shows the required longitudinal steel reinforcement to withstand the design BMD.

Table 5.61: Required steel reinforcement, Model p32-2

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		181.17	333.47	-	112.30
	200017	0.000	0.000	101	tens	tens		181.17	318.92	-	93.91
	300001	1.000	1.000	101	tens	tens		266.76	412.88	-	93.91

DC design case
Xi Relative distance
BEAM beam element
NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 412.88 = 544.12cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 181.17 = 365.71cm^2$$

The remaining area in the middle span and at the supports is positive, indicating that a post-tensioned cross-section with a height of 1100mm could withstand the design loads on the bridge model. Therefore, it is possible to use a cross-section with a depth of 1100mm for a post-tensioned concrete bridge with a middle span length of 32m and a total length of 80m. This span-to-depth ratio of 29.1 is considered acceptable for designing a post-tensioned concrete bridge.

The design case is at mid-supports:

$$\frac{\text{requirement R/F area } (412.88\text{cm}^2)}{\text{the maximum allowable R/F area } (957\text{ cm}^2)} * 100 = 43.14\% \text{ of the max. allowable R/F area (ASL2)}$$

Figure 5.43 shows the maximum decompression strain around the prestressing steel and the duct for the post-tensioned tendons. It can be seen that the decompression strain over the middle supports and at the middle span is negative, indicating that the prestressing steel and duct for post-tensioned tendons are situated 10mm within the compression zone. This confirms that a cross-section with a depth of 1100mm would be able to withstand the applied load after post-tensioning.

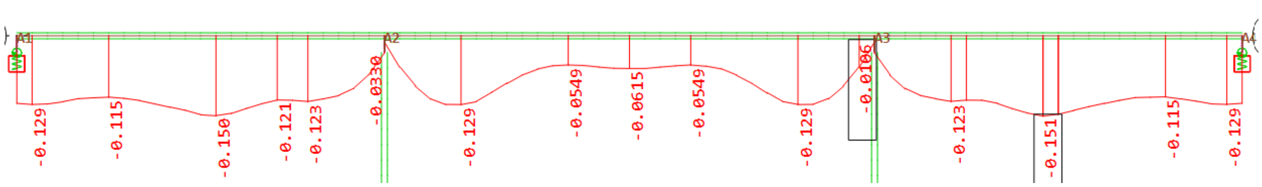


Figure 5.43: Maximum decompression strain at middle supports and at the middle span, model p32-2

5.3.4 Model P36

The prestressed bridge model P36 is made of a central span of 36m length and two side spans each of 27m, resulting in a total length of 90m. Table 5.62 summarizes the sub-models of model p36 and their corresponding efficiency percentages in using the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.62: Sub-models of model p36

Model	Mid span Length [m]	Total length [m]	C/S depth [mm]	L/H	ASL1	ASL2	% ASL1	% ASL2	Deco. Status
Model p36-1	36	90	1500	24	Ok	Ok	33.13	33.33	Not ok
Model p36-2	36	90	1550	23.22	Ok	Ok	33.13	33.33	Ok

5.3.4.1 Model p36-1

Model p36-1 is a post-tensioned model that has a mid-span length of 36m, cross section height of 1500mm, and a span-to-depth ratio of 24. Figure 5.44 and Figure 5.45 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

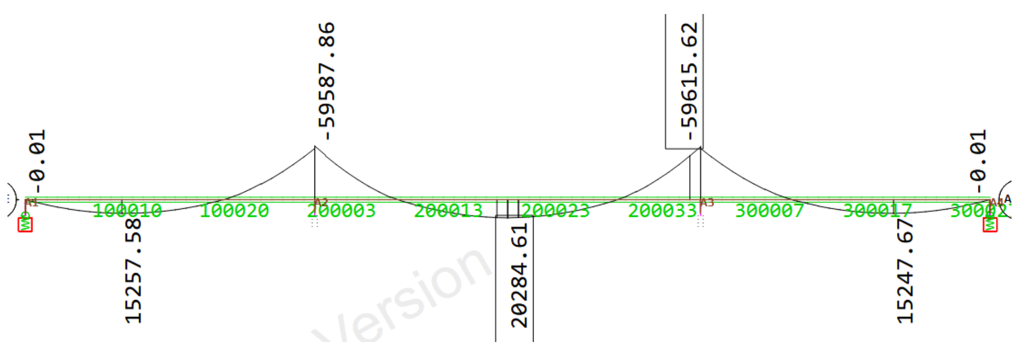


Figure 5.44: BMD My [kNm] for load combinations at middle supports, model p36-1

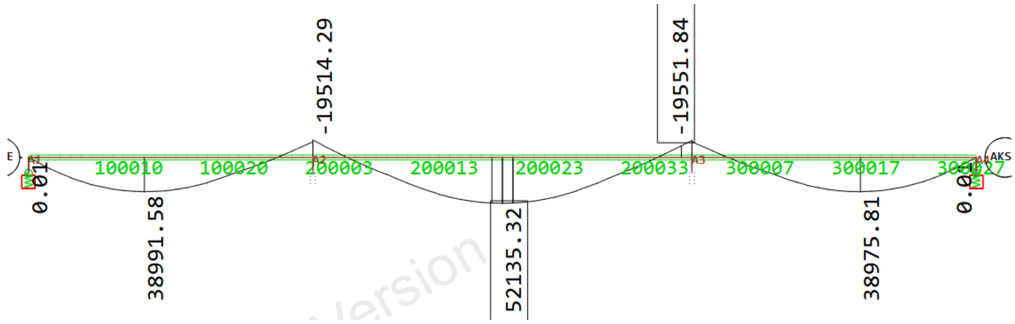


Figure 5.45: BMD My [kNm] for load combinations at middle span, model p36-1

Table 5.63 and Table 5.64 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.63: Load combination due to different load cases "LC" returning highest My at middle supports, model p36-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200036	1.000	C	5030	Creep until t-infinite	1.350	-86.33
			5031	Creep until t-infinite	1.350	-64.45
			5032	Creep until t-infinite	1.350	-168.86
		G_1	5010	Overbygning	1.350	-32380.11
		G_2	5020	Superegenvekt	1.350	-5595.28
		P	16015	Secondary effect CS 15	1.000	6421.53
		-	10024	LM1:TS Lane 10/11/12 Step25	0.945	-4207.47
			101	LM1:UDL Lane 10/11/12 Span2	0.945	-3056.38
			102	LM1:UDL Lane 10/11/12 Span3	0.945	-1499.48
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-6150.25
		ZW	24	+Vertikalt	1.120	-795.82
200036	1.000		2202	MIN-MY BEAM ULSB_a		-59615.62

Table 5.64: Load combination due to different load cases "LC" returning highest My at middle span, model p36-1

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200019	0.000	G_1	5010	Overbygning	1.200	18244.90
		G_2	5020	Superegenvekt	1.200	3152.71
		P	16015	Secondary effect CS 15	1.000	6440.04
		-	10522	LM1:TS Lane 20/21/22 Step23	1.350	6497.94
			104	LM1:UDL Lane 20/21/22 Span2	1.350	3213.02
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	7625.38
		ZW	24	+Vertikalt	1.120	449.14
200019	0.000		2301	MAX-MY BEAM ULSB_b		52135.32

Table 5.65 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.65: Required steel reinforcement, Model p36-1

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		181.17	318.92	-	104.30
	200019	0.000	0.000	101	tens	tens		181.17	318.92	-	93.91
	200036	1.000	1.000	101	tens	tens		181.17	318.92	-	104.83

DC design case   Xi Relative distance
BEAM beam element   NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 318.92 = 638.08 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 181.17 = 365.71 cm^2$$

The remaining area in the middle span and at the supports is positive, indicating that a post-tensioned cross-section with a height of 1500mm could withstand the design loads on the bridge model. However, Figure 5.46 illustrates the maximum decompression strain around the prestressing steel and duct for the post-tensioned tendons. It is evident that the decompression strain at the middle supports is positive, meaning that the prestressing steel and duct for post-tensioned tendons are not situated 10mm within the compression zone. Therefore, the cross-section of height 1500 is not suitable for withstanding the applied loads.

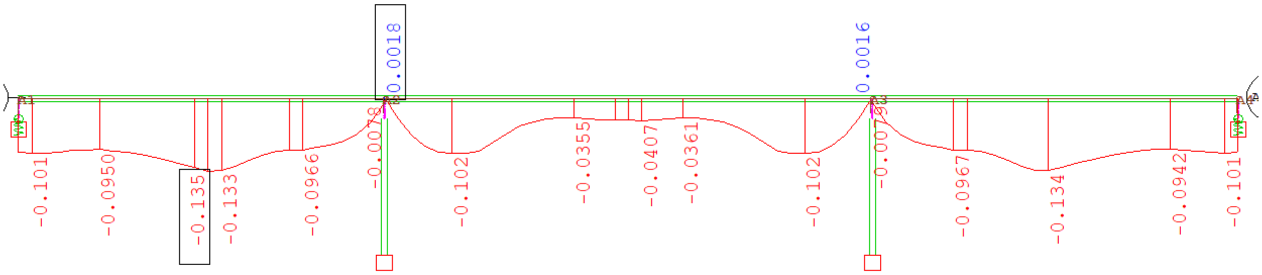


Figure 5.46: Max. decompression strain at mid. supports and at the mid. span, model p36-1

5.3.4.2 Model p36-2

Model p36-2 is a post-tensioned model that has a mid-span length of 36m, cross section height of 1550mm, and a span-to-depth ratio of 23.22. Figure 5.47 and Figure 5.48 show the maximum bending moments that occur at the middle supports and at the middle span, respectively.

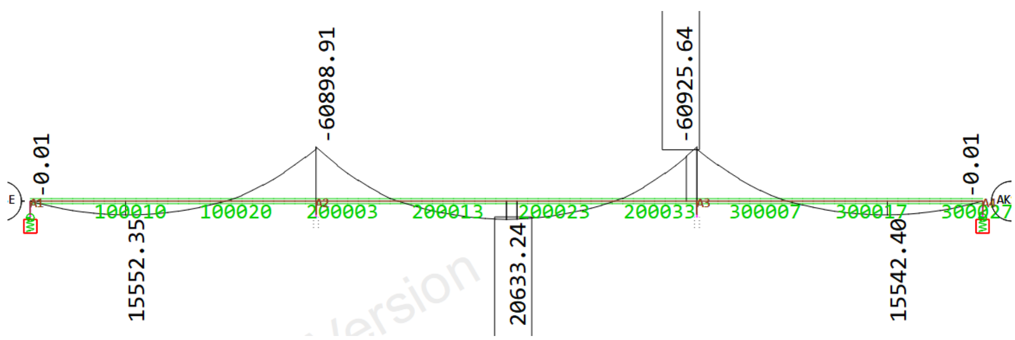


Figure 5.47: BMD My [kNm] for load combinations at middle supports, model p36-2

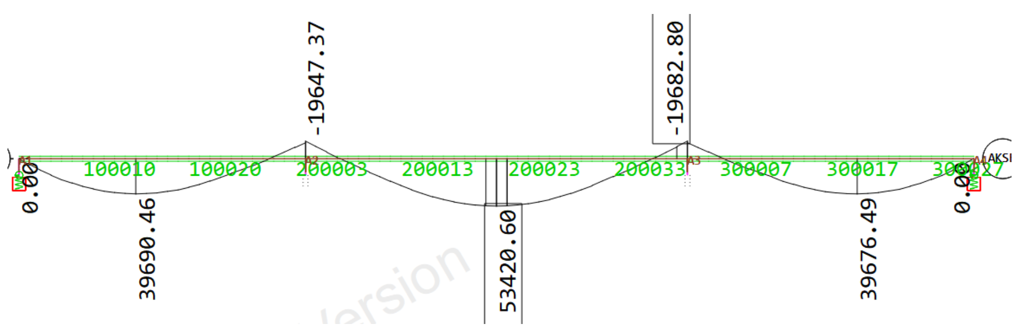


Figure 5.48: BMD My [kNm] for load combinations at middle span, model p36-2

Table 5.66 and Table 5.67 show the load combinations that return highest bending moment over middle supports and at middle span, respectively.

Table 5.66: Load combination due to different load cases "LC" returning highest My at middle supports, model p36-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200036	1.000	C	5030	Creep until t-infinite	1.350	-72.37
			5031	Creep until t-infinite	1.350	-59.97
			5032	Creep until t-infinite	1.350	-170.84
		G_1	5010	Overbygning	1.350	-33266.10
		G_2	5020	Superegenvekt	1.350	-5591.82
		P	16015	Secondary effect CS 15	1.000	6573.89
		-	10024	LM1:TS Lane 10/11/12 Step25	0.945	-4179.06
			101	LM1:UDL Lane 10/11/12 Span2	0.945	-3035.27
			102	LM1:UDL Lane 10/11/12 Span3	0.945	-1512.53
		T	92	Tsummer+negdt +wn*TN+DTZ	0.840	-6540.90
		ZW	24	+Vertikalt	1.120	-795.36
200036	1.000		2202	MIN-MY BEAM ULSB_a		-60925.64

Table 5.67: Load combination due to different load cases "LC" returning highest My at middle span, model p36-2

BEAM	x [m]	Act	LC	Designation	factor [-]	MY [kNm]
200019	0.000	G_1	5010	Overbygning	1.200	18776.40
		G_2	5020	Superegenvekt	1.200	3156.19
		P	16015	Secondary effect CS 15	1.000	6591.40
		-	10522	LM1:TS Lane 20/21/22 Step23	1.350	6524.92
			104	LM1:UDL Lane 20/21/22 Span2	1.350	3234.13
		T	90	Twinter+posdt -wn*TN-DTZ	0.840	8133.12
		ZW	24	+Vertikalt	1.120	449.60
200019	0.000		2301	MAX-MY BEAM ULSB_b		53420.61

Table 5.68 shows the required longitudinal steel reinforcement to withstand the design bending moments.

Table 5.68: Required steel reinforcement, Model p36-2

DC	BEAM	X [m]	Xi [-]	NQ	typ1	typ2	typ3	ASL1 [cm2]	ASL2 [cm2]	ASL3 [cm2]	ASB_m1 [cm2/m]
10	200001	0.000	0.000	101	tens	tens		181.17	318.92	-	102.59
	200019	0.000	0.000	101	tens	tens		181.17	318.92	-	93.91
	200036	1.000	1.000	101	tens	tens		181.17	318.92	-	103.09

DC design case — Xi Relative distance
BEAM beam element — NQ section number
X distance from start
typ1 Type of longitudinal reinforcement layer 1
typ2 Type of longitudinal reinforcement layer 2
typ3 Type of longitudinal reinforcement layer 3
ASL1 Longitudinal reinforcement layer 1
ASL2 Longitudinal reinforcement layer 2
ASL3 Longitudinal reinforcement layer 3
ASB_m1 Shear reinforcements of layer 1

Subtracting the design reinforcement area from the maximum allowable reinforcement area (Table 5.1) yields the following results:

$$\text{Remaining R/F area over middle supports} = 957 - 318.92 = 638.08 cm^2$$

$$\text{Remaining R/F area at middle span} = 546.88 - 181.17 = 365.71 cm^2$$

The remaining area in the middle span and at the supports is positive, indicating that a post-tensioned cross-section with a height of 1550mm could withstand the design loads on the bridge model. Therefore, it is possible to use a cross-section with a depth of 1550mm for a post-tensioned concrete bridge with a middle span length of 36m and a total length of 90m. This span-to-depth ratio of 23.22 is considered acceptable for designing a post-tensioned concrete bridge.

The design case is at mid-supports:

$$\frac{\text{requirement R/F area } (318.92\text{cm}^2)}{\text{the maximum allowable R/F area } (957\text{ cm}^2)} * 100 = 33.33\% \text{ of the max. allowable R/F area (ASL2)}$$

Figure 5.49 shows the maximum decompression strain around the prestressing steel and the duct for the post-tensioned tendons. It can be seen that the decompression strain over the middle supports and at the middle span is negative, meaning that the prestressing steel and duct for post-tensioned tendons are situated 10mm within the compression zone. This confirms that a cross-section with a depth of 1550mm would be able to withstand the applied load after post-tensioning.

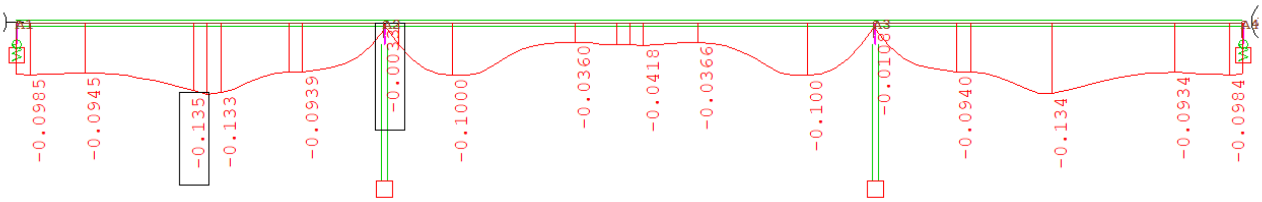


Figure 5.49: Maximum decompression strain at middle supports and at the middle span, model p36-2

5.3.5 Model P40

The prestressed bridge model P40 is made of a central span of 40m length and two side spans each of 30m, resulting in a total length of 100m. Table 5.69 summarizes the sub-model for model P40 and its corresponding efficiency percentages in using the longitudinal bottom (ASL1) and top (ASL2) reinforcement areas.

Table 5.69: Sub-models of model p40

Model	**Mid span Length [m]**	**Total length [m]**	**C/S depth [mm]**	**L/H**	**ASL1**	**ASL2**	**% ASL1**	**% ASL2**	**Deco. Status**
Model p40-1	40	100	2500	16	-	-	-	-	Not ok

5.3.5.1 Model p40-1

Model p40-1 is a post-tensioned bridge design with a mid-span length of 40m, a cross-section height of 2500mm, and a span-to-depth ratio of 16. The maximum decompression strain around the prestressing steel and duct for the post-tensioned tendons, shown in Figure 5.50, reveals that the decompression strain at the middle supports is positive. This indicates that the prestressing steel and duct for post-tensioned tendons are not situated within the compression zone, making the cross-section of height 2500mm unsuitable for withstanding the applied loads. Therefore, for bridge designs with lengths of 100m or more, it is recommended to use a higher number of prestressing tendons/strands or a different type of cross-section such as hollow box girder.

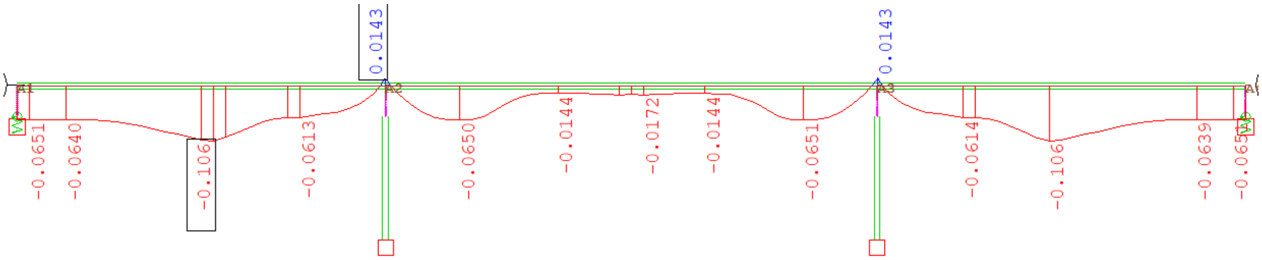


Figure 5.50: Maximum decompression strain at middle supports and at the middle span, model p40-1

5.4 Summary of Post-tensioned Concrete

Table 5.70 displays the results of mid-span length and the corresponding span-to-depth ratio for a three-span plate post-tensioned concrete bridge. On the other hand, Figure 5.51 illustrates the ratio between the mid-span length and the depth of the cross-section for the same type of bridge. The results show that the transition from reinforced to post-tensioned concrete occurs when the bridge length reaches 60m or when the middle span length is 24m. When the prestressing system is introduced, the required cross-section height is reduced from 1500mm to 700mm. However, for longer bridge lengths, a higher cross-section is still required. For middle span lengths of 28m, 32m, and 36m, the thinnest post-tensioned cross-sections that can withstand loads are 850mm, 1100mm, and 1550mm, respectively. The thesis shows that as the cross-section height increases, the span-to-depth ratio decreases. For the previously mentioned spans and cross-section heights, the following ratios are obtained: 34.28 for 28m, 32.94 for 32m, 29.1 for 36m, and 23.22 for 40m. In the case of bridge model p40-1, which has a middle span length of 40m and a total length of 100m, a maximum cross-section height of 2500mm was tested but failed to carry the applied loads. Therefore, for a bridge with a length of 100m or longer, a higher number of prestressing tendons or a different type of cross-section such as a hollow box girder would be required to withstand the loads.

Table 5.70: Mid-span length VS. span-to-depth ratio of post-tensioned concrete bridges

	Span length [m]	**span-to-depth ratio**
Model P24	24	34.28
Model P28	28	32.94
Model P32	32	29.1
Model P36	36	23.22

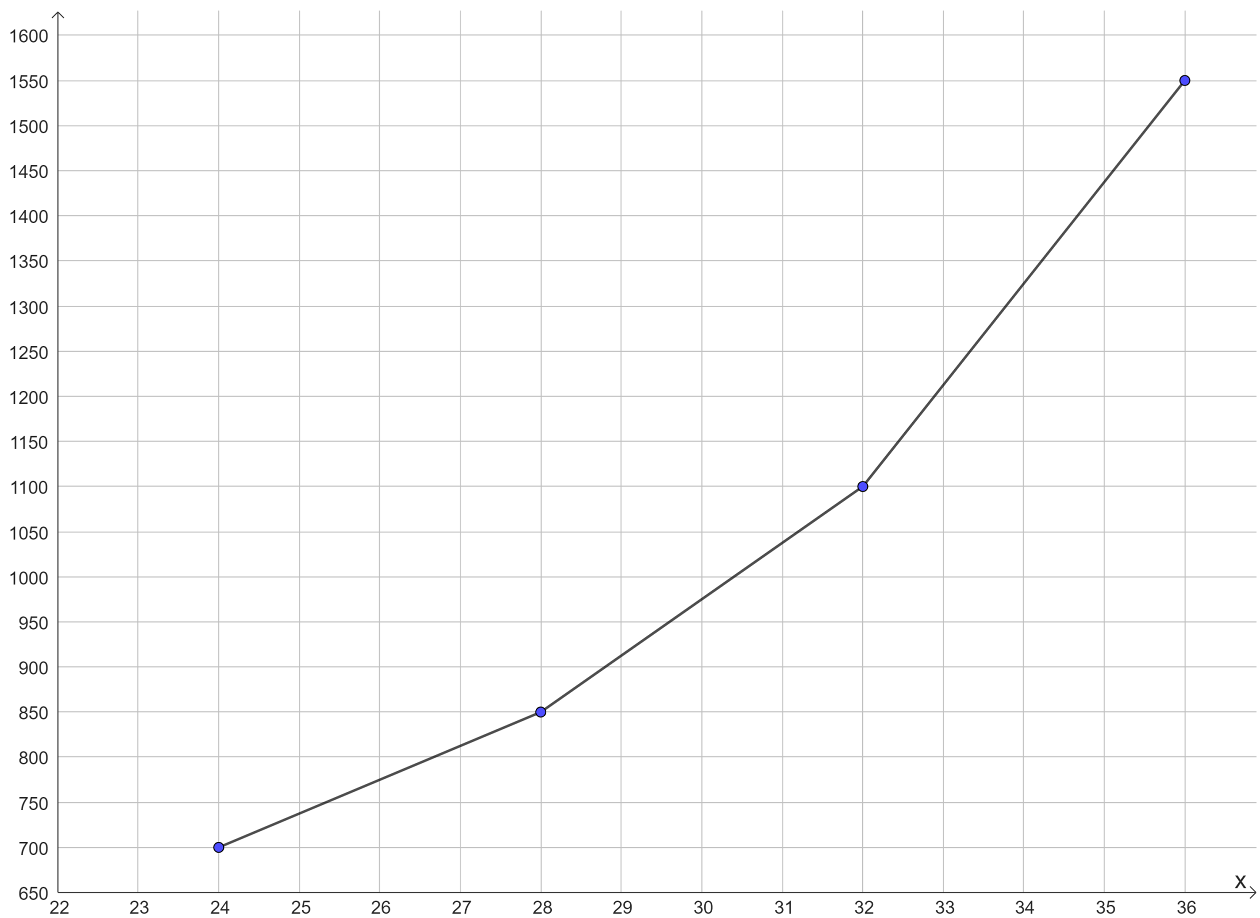


Figure 5.51: span-to-depth ratios of post-tensioned concrete bridges. The x-axis represents the mid-span length in meters, and the y-axis represents the depth of the cross section in millimeters.

5.5 Deflection Control

According to section 3.6.1 in Handbook N400 (Statens Vegvesen 2015), the deflection shall be limited to L/350 for characteristic traffic load only. The largest deflection from traffic load comes from LM1 at the middle span. However, EC2-1 (Standard Norge 2004a) has different criteria and states in section 7.4.1 (4) that the deflection should not exceed the limit of L/250, where L is the span length.

In reinforced concrete bridges, the cross-section may not always remain uncracked. Therefore, a linear calculation won't be accurate because the stiffness of the cross-section reduces when it is cracked. To account for this, a nonlinear analysis is necessary to obtain a more precise cracked stiffness matrix. This calculation is performed after the design is complete since the required reinforcement is used to obtain the cracked stiffness.

Initially, a non-linear deflection analysis of self-weights (permanent loads) is conducted in Sofistik. Subsequently, a cracked stiffness matrix is generated based on the permanent loads. Next, the cracked stiffness matrix is used to perform a linear calculation of the traffic load LM1 at the position that causes the most deformation.

Since traffic loading is considered a short-term load, this calculation is deemed to be sufficiently accurate. Furthermore, a non-linear calculation of both permanent loads and traffic loads was conducted, and the values were subtracted from the non-linear calculation of only permanent loads. Although the results were similar, this approach was much more time-consuming. Therefore, a decision was made to opt for a single non-linear calculation using permanent loads, and the resulting stiffness matrix was utilized to perform a linear calculation of the traffic load.

Table 5.71 and Figure 5.52 show the deflection values for all reinforced concrete models that have sufficient cross section depth.

Table 5.71: Deflections of reinforced concrete

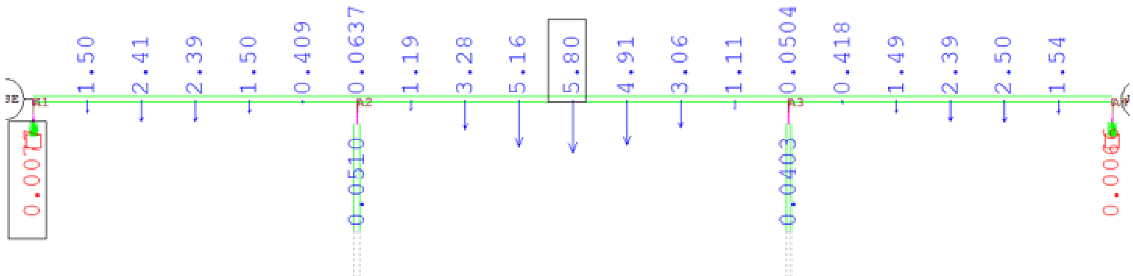
Model	δ_{EC2} **[mm]**	δ_{N400} **[mm]**	$\delta_{Sofistik}$ **[mm]**
12-1	32	22.86	5.80
16-3	48	34.3	20.5
20-5	64	45.71	34.5
24-1	80	57.14	23.3

In post-tensioned concrete bridges, the cross-section is always in compression. As a result, a linear calculation is sufficient to control the deflection due to LM1. Table 5.72 and Figure 5.53 show the deflection values for all post-tensioned models that have sufficient span-to-depth ratio and are in decompression.

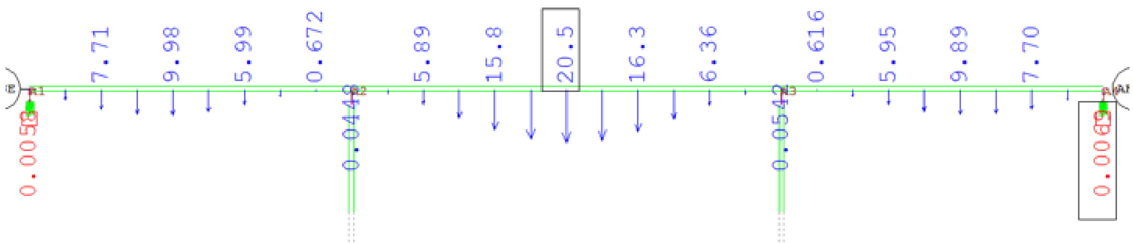
Table 5.72: Deflections of post-tensioned concrete

Model	δ_{EC2} **[mm]**	δ_{N400} **[mm]**	$\delta_{Sofistik}$ **[mm]**
p24-2	96	68.57	19.4
p28-2	112	80.0	19.4
p32-2	128	91.43	15.5
p36-2	144	102.85	9.26

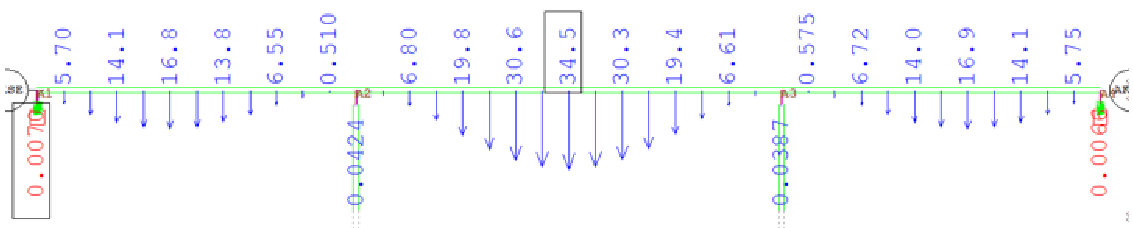
Since all deflections are below the limits specified in both EC2 and N400, they are deemed to be within safe limits.



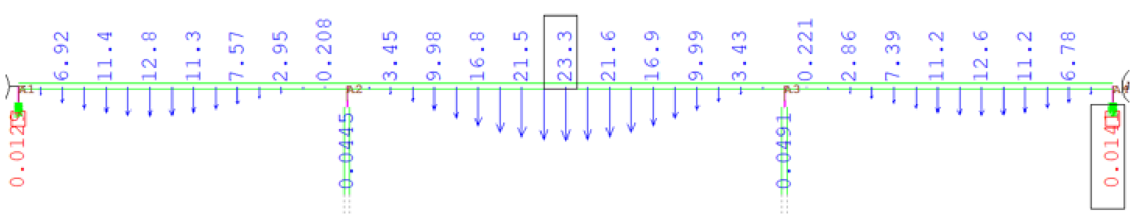
(a) Model 8-1



(b) Model 12-1

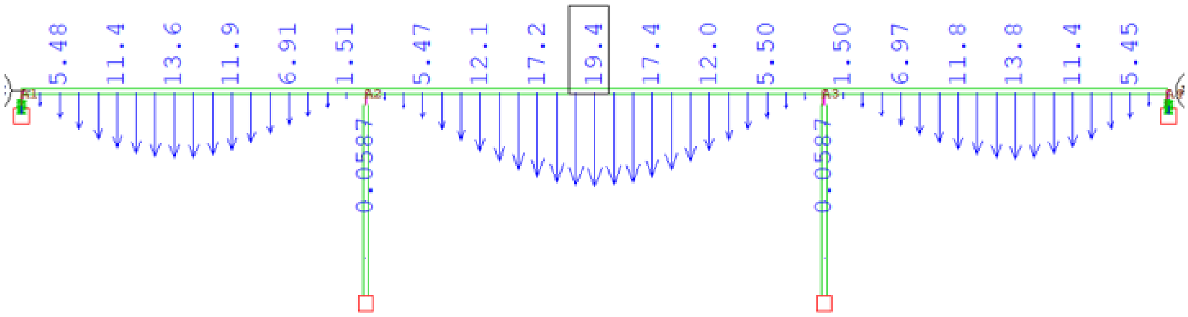


(c) Model 16-3

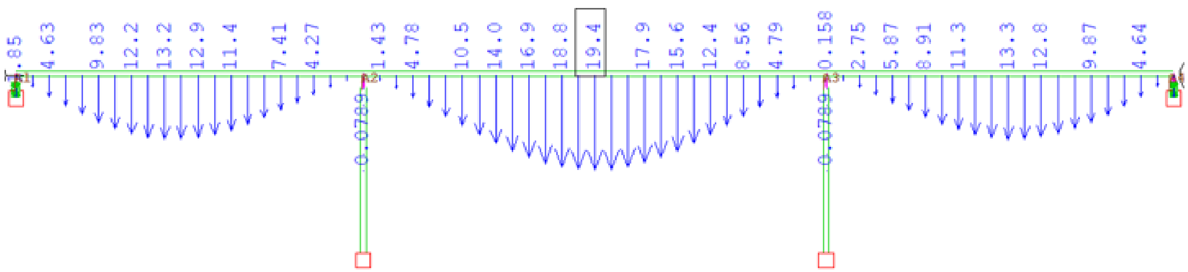


(d) Model 20-5

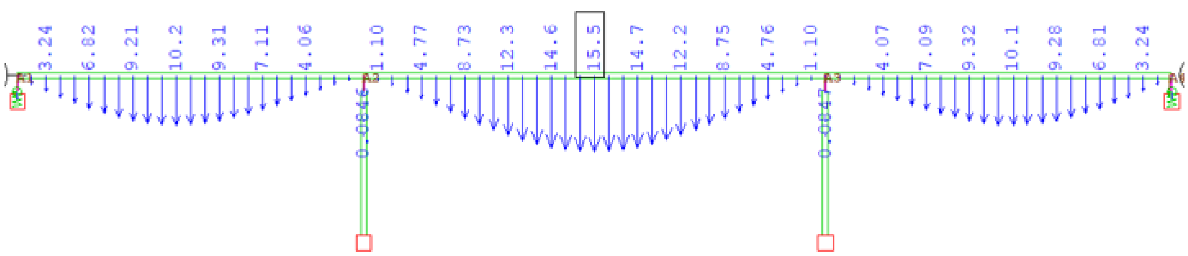
Figure 5.52: Deflection diagrams of reinforced concrete in [mm]



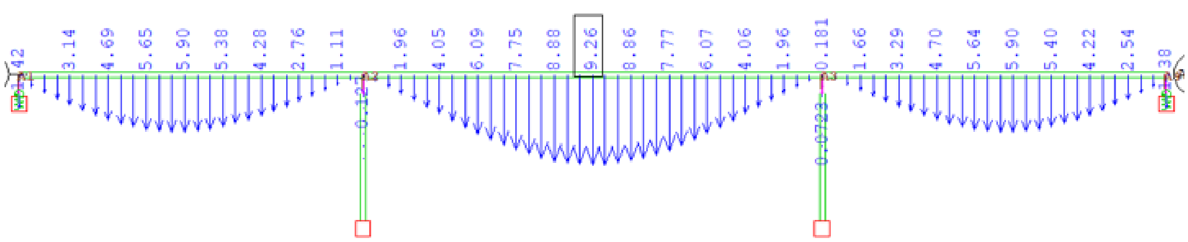
(a) Model p24-2



(b) Model p28-2



(c) Model p32-2



(d) Model p36-2

Figure 5.53: Deflection diagrams of post-tensioned concrete in [mm]

5.6 Verification of Model in Sofistik

To verify that the output of Sofistik is within reasonable values, the software program "K-bjelke" is used. "K-bjelke" is a program specifically designed for static dimensioning of continuous reinforced concrete beams or concrete slabs with adjacent walls/columns (Sogelink - Focus Software AS). Load values without load factors, i.e., the characteristic values, are used. A verification model of the bridge model 20-5 is developed with a total length of 50m and a cross-section depth of 950 mm.

5.6.1 Control of Self-Weight

In this thesis, it is assumed that the reinforced and post-tensioned concrete bridges have a self-weight of 25 kN/m^3. Figure 5.54 illustrates the cross-section of model 20-5, which has a cross-sectional area of 8.65 m^2. This results in a uniformly distributed load (UDL) q_{sw}, as shown in equation 5.1. Figure 5.55 displays the UDL calculated by Sofistik. This UDL is used to calculate the bending moments due to permanent load (self-weight).

$$25\,\frac{kN}{m^3}\cdot 8.65m^2 = 216.25kN/m \tag{5.1}$$

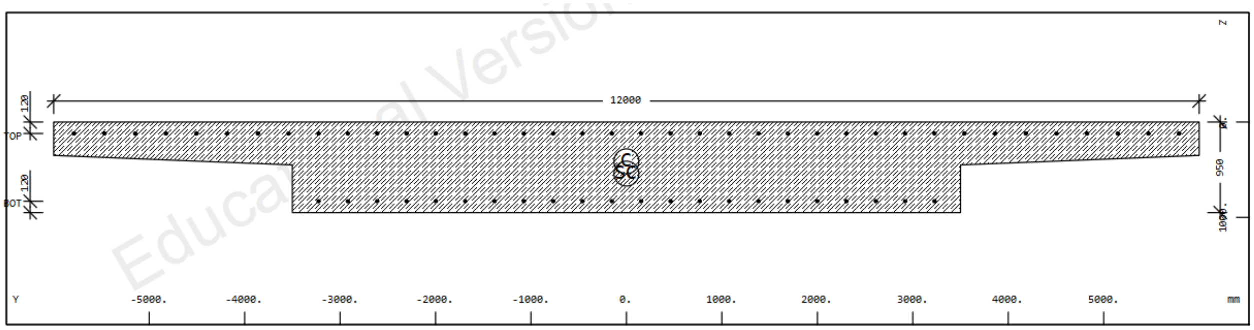


Figure 5.54: Cross-section of model 20-5. Dimensions given in [mm]

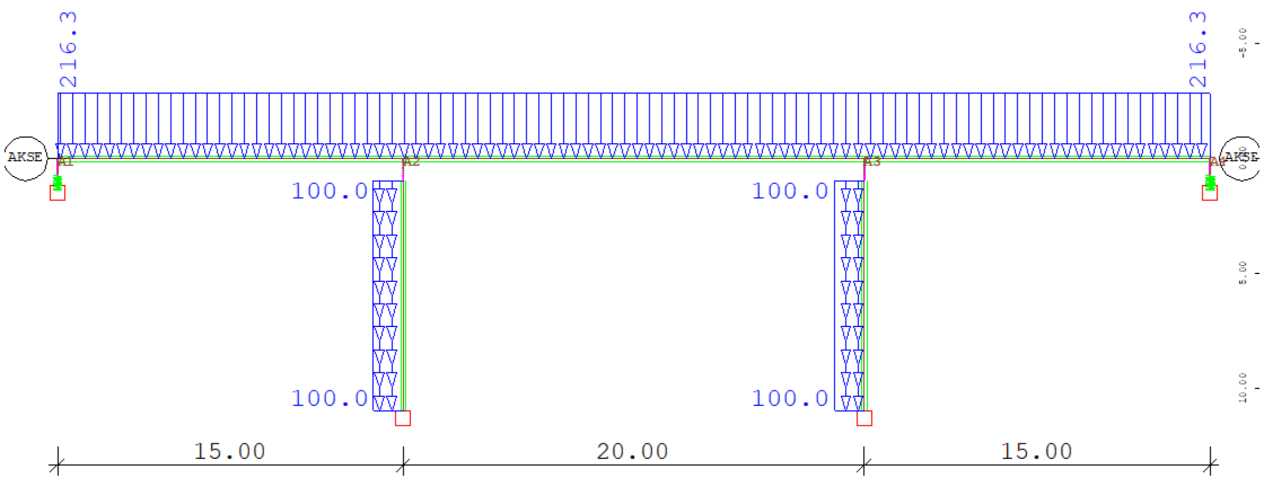


Figure 5.55: UDL of model 20-5 given in kN/m by Sofisitk. Dimensions are given in [m]

The bending moment due to self-weight calculated by Sofistik is illustrated in Figure 5.56 while Figure 5.57 displays the verification of the bending moment due to self-weight calculated by K-bjelke. Full K-bjelke calculations are given in Appendix B.

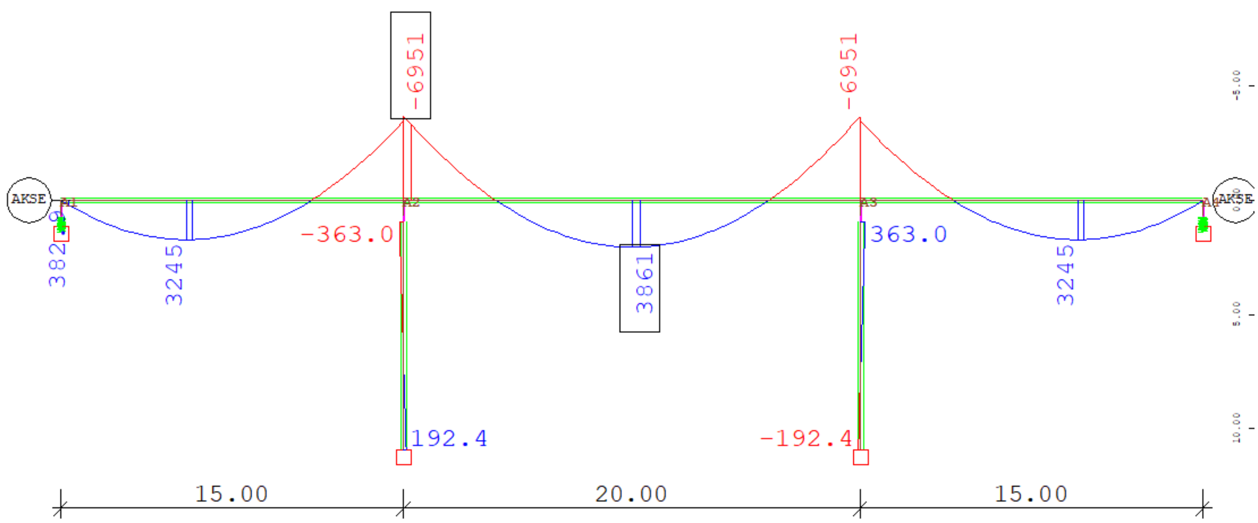


Figure 5.56: BMD due to self-weight in kN/m by Sofistik. Dimensions are given in [m]

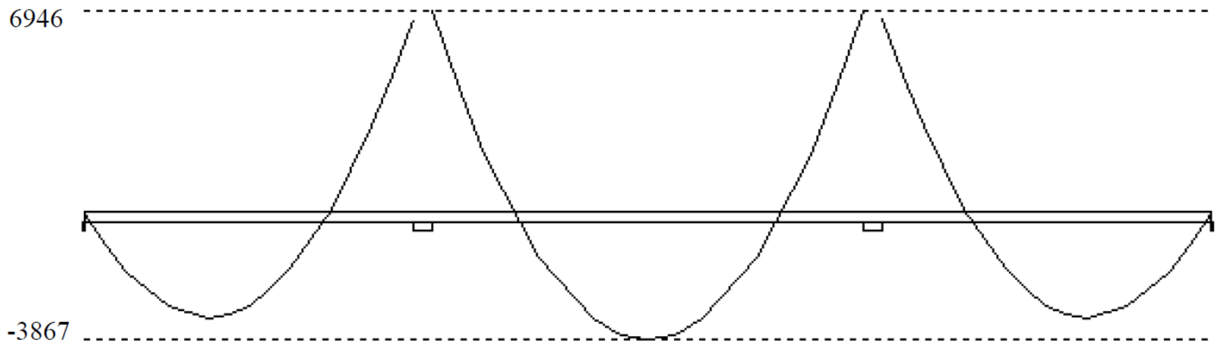


Figure 5.57: Verification of BMD due to self-weight and 0.01% of variable load given in kN/m by K-bjelke.

The bending moment results are presented in Table 5.73.

Table 5.73: Verification of bending moment due to self-weight

	Span 1-2	**Support 2**	**Span 2-3**	**Support 3**	**Span 3-4**
Sofistik [kNm]	3245	6951	3861	6951	3245
K-bjelke [kNm]	3245	6945	3865	6945	3245
Deviation [%]	0	0	0	0	0

According to the verification calculations, there are negligible variations in Sofistik's model for bending moments caused by self-weight, implying that it is safe to assume the accuracy of Sofistik's results.

Chapter 6

Conclusion

This thesis analyzed the span-to-depth ratio of reinforced and post-tensioned three-span plate concrete bridge models designed for environmental conditions in Oslo, Norway. The bridge models ranged in length from 20m to 100m and consisted of four axes with three spans of 0.3L + 0.4L + 0.3L. All models had a cross-section width of 12m but varying depths.

For the superstructure, a concrete of class B45 was chosen due to its high strength and suitability for use in concrete bridges. The prestressing system used was the BBR VT CONA BT 2706-150-1860, which employed 10 cables made up of 27 strands each with a nominal cross-sectional area of 150 mm^2. The total tendon area was therefore 4050 mm^2.

To study the behavior of the bridge models, in total 20 models were modeled in Sofistik using the text editor Teddy, which utilizes its own programming language called the CADINP-command language. The study revealed that the moments at the Ultimate Limit State (ULS) were considerably higher compared to those at the Serviceability Limit State (SLS). Furthermore, the analysis determined that the ULS governs the design for bridge models 8, 12, and 16, while the SLS governs the design for bridge model 20. In the case of post-tensioned bridge models, the SLS governs the design for all of them. The self-weight of the superstructure, traffic load LM1, and temperature were found to be the dimensioning loads over middle supports and middle spans at ULS, while creep and shrinkage loads contributed to the moment forces at the middle span. Wind loads have minimal impact on the dimensioning of the bridge deck, but in certain cases, wind load can contribute more significantly compared to creep and shrinkage. In cases of prestressing, the secondary effect arises at the Ultimate Limit State (ULS) due to the internal resistance of the prestressing cables. The secondary effect of prestressing significantly contributes to the moment force at the middle span. Self-weight had a greater influence on longer spans. For moments over the supports and middle span, self-weight was dominant for all bridge models.

Based on the assumptions set in this study, the findings indicate that the required cross-section height of a bridge varies depending on the span length. It is observed that the transition from reinforced to post-tensioned concrete occurs when the bridge length reaches 60m or when the middle span length is 24m. The results also show that for short spans ranging from 8m to 12m with span-to-depth ratios of 17.78 and 26.67, the thinnest reinforced cross-section height that can withstand the applied loads is 450mm. While, for spans exceeding 12m, such as 16m and 20m with span-to-depth ratios of 29.1 and 21.05, respectively, the thinnest cross-section heights that can withstand the applied loads are 550mm and 950mm. The study also highlights that there is a non-linear slope between spans of 8m to 20m, which differs from the recommendations given by the Norwegian Road Administration in Handbook 4. However, the obtained ratios for middle span lengths between 8m and 16m are within the recommended ratios provided by Handbook 4, while the ratio obtained for span length of 20m differ from the recommended ratios.

As the cross-section height increases, the span-to-depth ratio decreases. The study reveals that for middle span lengths of 28m, 32m, and 36m, the thinnest post-tensioned cross-sections that can withstand loads are 850mm, 1100mm, and 1550mm, respectively. For longer bridge lengths, a higher cross-section is still required. For instance, for a bridge model with a middle span length of 40m and a total length of 100m, a maximum cross-section height of 2500mm was tested but failed to carry the applied loads. Based on the findings of this study, it is advisable to utilize a higher number of prestressing tendons or consider an alternative cross-section design for bridges that exceed a length of 100m to ensure that they can support the applied loads effectively.

The deflection control produced satisfactory results, as all deflections remained within the limitations specified by EC2 and N400.

This thesis offers valuable insights into the thinnest cross-section heights required to withstand various spans and loads for reinforced and post-tensioned concrete bridges. Nevertheless, further research could investigate other factors that may affect the bridge's structural integrity, such as the choice of materials and the number of cables or strands in the prestressing system. By varying the number of cables or strands and utilizing different concrete classes, the load-carrying capacity of the bridge can be influenced. This provides designers with crucial information to choose an optimal prestressing system and achieve a more cost-effective design.

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Appendix

- Appendix A - Wind load calculations
- Appendix B - Control of self-weight
- Appendix C - Teddy input task files

Appendix A

Wind load Calculations

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VINDLASTER

Variabler for vindlast (iht. NS-EN 1991-1-4):

Vindlastklasse I, iht. HB N400 pkt. 5.4.3:

Basisvindhastighet, v_b:

Fra NS-EN 1991-1-4, tab. NA.4(901.1):

$v_{b0} := 22\,\frac{m}{s}$ Oslo

Returperiode :=

10år (byggefase)
50år (ferdigtilstand)

$c_{dir} := 1.0$ $c_{season} := 1.0$

$c_{alt} := 1.0$

$$c_{prob} := \left(\frac{1 - 0.2 \cdot \ln(-\ln(1-p))}{1 - 0.2 \cdot \ln(-\ln(0.98))}\right)^{0.5} \qquad c_{prob} = 1 \qquad (4.2)$$

$$v_b := c_{dir} \cdot c_{season} \cdot c_{alt} \cdot c_{prob} \cdot v_{b0} \qquad (NA.4.1)$$

$v_b = 22\,\frac{m}{s}$

Ruhetsfaktor, $C_r(z)$:

Velger terrengruhetsklasse, tab. NA.4.1:

Terrengruhetsklasse :=

0
I
II
III
IV

$k_r = 0.19$ $z_0 = 0.05\,m$ $z_{min} = 4\,m$

Topp bru (høyde målt fra laveste punkt i terreng til sentrum av brudekket, Jfr. NS-EN 1991-1-4 8.3.1 (6)) :

$z := 10m$

$$c_r(z) := \begin{vmatrix} k_r \cdot \ln\left(\frac{z}{z_0}\right) & \text{if } z \ge z_0 \wedge z \le 200m \\ c_r(z_{min}) & \text{if } z < z_0 \end{vmatrix} \qquad (4.4)$$

$c_r(z) = 1.01$

Figure A.1: Wind load calculation of model 20-5, page 1

Stedsvindhastighet, $v_m(z)$:

$$c_0(z) := 1.0$$

$$v_m(z) := c_r(z) \cdot c_0(z) \cdot v_b \qquad (4.3)$$

$$v_m(z) = 22.15 \frac{m}{s}$$

Vindkasthastighetstrykket, q_p:

Luftens densitet: $\rho := 1.25 \frac{kg}{m^3}$

$$k_l := 1.0$$

$$I_{v.}(z) := \begin{cases} \dfrac{k_l}{c_0(z) \cdot \ln\left(\dfrac{z}{z_0}\right)} & \text{if } z \geq z_{min} \\ I_{v.}(z_{min}) & \text{if } z < z_{min} \end{cases} \qquad (4.7)$$

$$I_{v.}(z) = 0.19$$

$$k_{p.} := 3.5$$

$$q_p(z) := \left(1 + 2 \cdot k_{p.} \cdot I_{v.}(z)\right) \cdot \frac{1}{2} \cdot \rho \cdot v_m(z)^2 \qquad (NA.4.8)$$

$$q_p(z) = 0.71 \cdot \frac{kN}{m^2}$$

Vindkasthastighetstrykket på trafikk, $q_{p\_trafikk}$:

HB N400, pkt. 5.4.3.3 gir en øvre grense for kastvindhastigheten på trafikk på 35 m/s.

$$v_{m\_trafikk\_max}(z) := \frac{35 \frac{m}{s}}{\sqrt{1 + 2 \cdot k_{p.} \cdot I_{v.}(z)}}$$

$$v_{m\_trafikk}(z) := \min\left(v_m(z), v_{m\_trafikk\_max}(z)\right)$$

$$v_{m\_trafikk}(z) = 22.15 \frac{m}{s}$$

$$q_{p\_trafikk}(z) := \left(1 + 2 \cdot k_{p.} \cdot I_{v.}(z)\right) \cdot \frac{1}{2} \cdot \rho \cdot v_{m\_trafikk}(z)^2$$

Figure A.2: Wind load calculation of model 20-5, page 2

$$q_{p\_trafikk}(z) = 0.71 \cdot \frac{kN}{m^2}$$

A. Vindlast på brubane u/trafikk og m/rekkverk:

Rekkverk :=
- Åpent - Begge sider
- Åpent - Én side
- Tett - Begge sider
- Tett - Én side

Brurekkverk := ☑

Betongrekkverk := ☐

Kraftfaktor i x-retning [8.3.1]:

$b_{bru} := 12m$ $d_{bru} := 0.95m$ $d_{kant} := 0.17m$ $d_{rek} := 1.2m$ Tab.8.1

d_{tot} = "d + 0,6 m" $\frac{b_{bru}}{d_{tot.}} = 6.98$ gir $C_{fx0} = 1.3$ Fig. 8.3

Kraft i y-retning: tverretning definert som x-retning i eurocode, mens i Sofistik er tverretning definert som y-retning

$$q_{Bjelke\_y\_Brigade} := C_{fx0} \cdot q_p(z).dtot \ = 1.591 \ \frac{kN}{m^2}$$

Vindlaster i x-retning er 25% av y-retning for bjelkebruer (iht. 8.3.4):

$$q_{Bjelke\_y\_Brigade} := C_{fx0} \cdot q_p(z) \cdot 0.25 = 0.397 \cdot \frac{kN}{m^2}$$

Kraft i z-retning:

$C_{fz} := 0.9$

$$q_{Bjelke\_z} := C_{fz} \cdot q_p(z) \cdot b_{bru} = 7.68 \cdot \frac{kN}{m} \qquad e := \frac{b_{bru}}{4} = 3\ m$$ 8.3.3 (5)

Figure A.3: Wind load calculation of model 20-5, page 3

B. Vindlast på brubane m/trafikk:

Kraftfaktor i x-retning [8.3.1]:

$b_{bruT} := b_{bru}$ $d_{bruT} := d_{bru}$ $d_{kantT} := d_{kant}$ $d_{bilT} := 2m$

$d_{totT} :=$ "d + d1" $\frac{b_{bruT}}{d_{totT.}} = 3.85$ gir $C_{fx0T} = 1.35$ Fig. 8.3

Kraft i y-retning: tverretning definert som x-retning i eurocode, mens i Sofistik er tverretning definert som y-retning

$$\boxed{q_{BjelkeT\_y\_Brigade.} := C_{fx0T} \cdot q_{p\_trafikk}(z).dtotT = 2.985\ \frac{kN}{m^2}}$$

Vindlaster i x-retning er 25% av y-retning for bjelkebruer (iht. 8.3.4):

$$\boxed{q_{BjelkeT\_y\_Brigade.} := qBjelkeT\_y\_Brigade. \cdot 0.25 = 0.746\ \frac{kN}{m^2}}$$

Kraft i z-retning:

$C_{fz} = 0.9$

$$\boxed{q_{BjelkeT\_z} := C_{fz} \cdot q_{p\_trafikk}(z) \cdot b_{bruT} = 7.68 \cdot \frac{kN}{m}} \quad e := \frac{b_{bruT}}{4} = 3\ m$$ 8.3.3 (5)

Figure A.4: Wind load calculation of model 20-5, page 4

```
Masteroppgave

+PROG TEMPLATE
$ Dat : C:\...\Sem 4\Main Model\RF design\L50\L50.dat  (#001)      06/05/2023
$ Job : DESKTOP-3DIQ7PP:001107                                          14:48
HEAD
LET#Z        10     $ meter
PRT#Z $ meter
---- CADINT VARIABLE Z              (     0) =       10.00
LET#CDIR      1
LET#CSEASON   1
LET#CALT      1
LET#CPROB     1                              $for 50 years period
LET#VB_0     22       $ m/s
LET#VB       #CDIR*#CSEASON*#CALT*#CPROB*#VB_0

PRT#VB $ m/s
---- CADINT VARIABLE VB             (     0) =       22.00

$Stedvindhatighet Vm(z)
$terrenkategori 2
LET#KR        0.19
LET#Z_MIN     4     $ meter
LET#Z0        0.05
LET#C0_Z      1
$ for z_min < z < z_max
LET#CR_Z     #KR*LOG(#Z/#Z0)
LET#VM_Z     #CR_Z*#C0_Z*#VB
PRT#CR_Z
---- CADINT VARIABLE CR_Z           (     0) =       1.007
PRT#VM_Z
---- CADINT VARIABLE VM_Z           (     0) =       22.15

$Turbulensintensitet Iv(z)     $ for z_min < z < z_max
LET#K1     1
LET#IV_Z   #K1/(#C0_Z*LOG(#Z/#Z0))
PRT#IV_Z
---- CADINT VARIABLE IV_Z           (     0) =      0.1887

$Vindhastighetstryket qp(z)
LET#QM_Z  (0.5*1.25*(#VM_Z)**2)/1000  $ KN/m2
PRT#QM_Z
---- CADINT VARIABLE QM_Z           (     0) =      0.3066
LET#KP   3.5
LET#QP_Z (1+2*#KP*#IV_Z)*#QM_Z         $ KN/m2
PRT#QP_Z
---- CADINT VARIABLE QP_Z           (     0) =      0.7116

$Vindhastighetstrykket på trafikk, qp_trafikk

LET#VM_TRAFIKK_MAX_Z 35/SQR(1+2*#KP*#IV_Z)
PRT#VM_TRAFIKK_MAX_Z
---- CADINT VARIABLE VM_TRAFIKK_MAX_Z(     0) =       22.97
LET#VM_TRAFIKK_Z MIN(#VM_Z,#VM_TRAFIKK_MAX_Z)        $ m/s
PRT#VM_TRAFIKK_Z
---- CADINT VARIABLE VM_TRAFIKK_Z   (     0) =       22.15

 $Stedvindhastighetstrykket på trafikk
 LET#Q_P_TRAFIKK_Z (1+2*#KP*#IV_Z)*0.5*1.25*#VM_TRAFIKK_Z**2/1000  $ KN/m2
 PRT#Q_P_TRAFIKK_Z
---- CADINT VARIABLE Q_P_TRAFIKK_Z  (     0) =      0.7116

$Vindlast på brubane uten trafikk og med rekkverk
```


Figure A.5: Calculation of wind loads in Sofistik of model 20-5, page 1

Masteroppgave

```
LET#B_BRU  12
LET#D_BRU  #HTVERR
LET#D_KANT 0.17
LET#D_REK 1.2
LET#D_TOT #D_BRU+#D_KANT+0.6
PRT#D_TOT
---- CADINT VARIABLE D_TOT          (     0) =       1.720
LET#RIO #B_BRU/#D_TOT
PRT#RIO
---- CADINT VARIABLE RIO            (     0) =       6.977

IF ( 4<#RIO) ! CONDITION 1
   LET#CFX0 1.3
ELSE  ! OR CONDITION 2
   LET#CFX0 -0.29*#RIO+2.46
ENDIF
PRT#CFX0
---- CADINT VARIABLE CFX0           (     0) =       1.300
LET#CFZ 0.9
PRT#CFZ 0.9
---- CADINT VARIABLE CFZ            (     0) =      0.9000


$kraft i y-retning         tverretning definert som x-retning i eurocode, mens i Sofistik er tverre
LET#QY #CFX0*#QP_Z*#D_TOT      $ KN/m
PRT#QY
---- CADINT VARIABLE QY             (     0) =       1.591

$kraft i x-retning
LET#QX 0.25*#QY           $ KN/m
PRT#QX
---- CADINT VARIABLE QX             (     0) =      0.3978

$ kraft i z-retning
LET#QZ #CFZ*#QP_Z*#B_BRU
PRT#QZ
---- CADINT VARIABLE QZ             (     0) =       7.685
LET#E #B_BRU/4

$Vindlast på brubane m/trafikk
$kraftfaktor i x-retning (8.3.1)

LET#B_BRUT #B_BRU
LET#D_BRUT #D_BRU
LET#D_KANTT #D_KANT
LET#D_BILT 2
LET#D_TOTT  #D_BRU+#D_KANTT+#D_BILT
PRT#D_TOTT
---- CADINT VARIABLE D_TOTT         (     0) =       3.120
LET#RIOT #B_BRUT/#D_TOTT

IF ( 4<#RIOT)
   LET#CFX0T 1.3
ELSE
   LET#CFX0T -0.29*#RIOT+2.46
ENDIF
PRT#CFX0T
---- CADINT VARIABLE CFX0T          (     0) =       1.345

$kraft i y-retning                   tverretning definert som x-retning i eurocode, mens i Sofistik
LET#QYT #CFX0T*#Q_P_TRAFIKK_Z*#D_TOTT     $ KN/m
```


Figure A.6: Calculation of wind loads in Sofistik of model 20-5, page 2

Masteroppgave

```
PRT#QYT
---- CADINT VARIABLE QYT            (     0) =       2.985
$kraft i x-retning
LET#QXT 0.25*#QYT          $ KN/m
PRT#QXT
---- CADINT VARIABLE QXT            (     0) =      0.7463
$ kraft i z-retning
LET#QZT #CFZ*#Q_P_TRAFIKK_Z*#B_BRU  $ KN/m
PRT#QZT
---- CADINT VARIABLE QZT            (     0) =       7.685
LET#E #B_BRU/4
PRT#E
---- CADINT VARIABLE E              (     0) =       3.000
$max kraft av med eller uten trafikk
STO#QNYX MAX(#QX,#QXT)
STO#QNYY MAX(#QY,#QYT)
STO#QNYZ MAX(#QZ,#QZT)
PRT#QNYX
---- CADINT VARIABLE QNYX           (     0) =      0.7463
PRT#QNYY
---- CADINT VARIABLE QNYY           (     0) =       2.985
PRT#QNYZ
---- CADINT VARIABLE QNYZ           (     0) =       7.685
$ -----------------------------------------------------------------------------------------------
STO#VINDVERT #QNYZ         $ Linjelast, overbygning. Global Z-retning i Sofistik [kN/m]
STO#VINDTVER #QNYY            $ Linjelast, overbygning. Global Y-retning i Sofistik [kN/m]
STO#VINDMOM 40.6    $ må sjekkes        $ Linjemoment, overbygning. Bidrag fra eksentrisk vind i Y
STO#VINDLANG #QNYX          $ Linjelast, overbygning. Global X-retning i Sofistik [kN/m]
END
```


Figure A.7: Calculation of wind loads in Sofistik of model 20-5, page 3

```
+prog Sofiload urs:64.1
Head Vindlaster
$ + Normalt på bru (eksentrisk)/ - Vertikalt (eksentrisk)
$ -----------------------------------------------------------------------------------------------
LC 20 TYPE ZW TITL '+Normalt (eks.)/ -Vertikalt (eks.)'
beam grp (100 300 100)                TYPE PYY  #VindTver        $ Vind i Y-retning
beam grp (100 300 100)                TYPE MX   -#VindMom        $ Vind i Y- og Z-retning (momentbidrag)
beam grp (100 300 100)                TYPE PZZ  -#VindVert       $ Vind i Z-retning
beam grp (20 30 10)                   TYPE PY   #VindSoyT        $ Vind i Y-retning

$ - Normalt på bru (eksentrisk)/ - Vertikalt (eksentrisk)
$ -----------------------------------------------------------------------------------------------
LC 21 TYPE ZW TITL '-Normalt (eks.)/ -Vertikalt (eks.)'
beam grp (100 300 100)                TYPE PYY  -#VindTver        $ Vind i Y-retning
beam grp (100 300 100)                TYPE MX   #VindMom          $ Vind i Y- og Z-retning (momentbidrag)
beam grp (100 300 100)                TYPE PZZ  -#VindVert        $ Vind i Z-retning
beam grp (20 30 10)                TYPE PY   -#VindSoyT       $ Vind i Y-retning

$ + Langs bru/ - Vertikalt
$ -----------------------------------------------------------------------------------------------
LC 22 TYPE ZW TITL '+Langs bru/ -Vertikalt'
beam grp (100 300 100)                TYPE PXX  #VindLang         $ Vind i x-retning
beam grp (100 300 100)                TYPE PZZ  -#VindVert        $ Vind i Z-retning
beam grp (20 30 10)                TYPE PZ   #VindSoyL        $ Vind i lokal Z-retning

$ - Langs bru/ - Vertikalt
$ -----------------------------------------------------------------------------------------------
LC 23 TYPE ZW TITL '-Langs bru/ -Vertikalt '
beam grp (100 300 100)                TYPE PXX  -#VindLang        $ Vind i x-retning
beam grp (100 300 100)                TYPE PZZ  -#VindVert        $ Vind i Z-retning
beam grp (20 30 10)                TYPE PZ   -#VindSoyL       $ Vind i lokal Z-retning

$ + Vertikalt
$ -----------------------------------------------------------------------------------------------
LC 24 TYPE ZW TITL '+Vertikalt'
beam grp (100 300 100)                TYPE PZZ  #VindVert        $ Vind i Z-retning
end
```

Figure A.8: Wind loads parameterized in Teddy coding language

Appendix B

Control of Self-Weight

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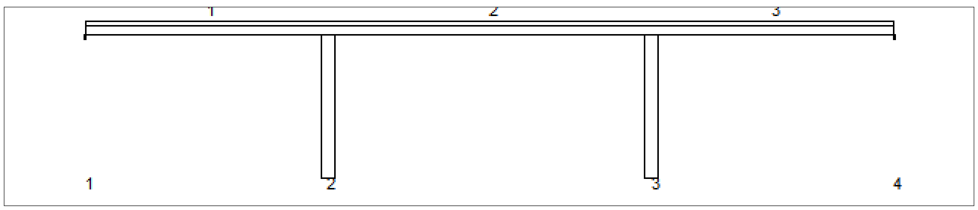
Self-weight control

Tittel Appendix			Side 1
Prosjekt Model 20-5	Ordre	Sign Yousef	Dato 12-04-2023

Dataprogram: K-Bjelke versjon 7.3.1 Laget av sivilingeniør Ove Sletten
Beregningene er basert på NS-EN 1992-1-1:2004 + NA:2008 og NS-EN 1990:2002
Data er lagret på fil: C:\Users\Yousef\OneDrive - Universitetet i Stavanger\Desktop\UIS\Master\Sem 4\Main Model\Control av egenvekt\Sjekk_egen model 20-5.kbj

INNHOLD

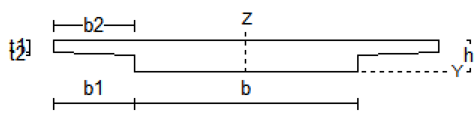
1.0 BJELKE MED 4 OPPLEGGSPUNKTER



1.1 SPENNVIDDER [mm], OG TVERRSNITTYPER

Felt nr	v.utkr.	1	2	3	h.utkr.
Spennvidde	0	15000	20000	15000	0
Tverrsnittype		1	1	1	

Tverrsnittype 1



b	7000	mm
b1	2500	mm
b2	2500	mm
h	950	mm
t1	350	mm
t2	100	mm
Zt	538	mm
Areal	8.65E+06	mm2
Iy	6.43E+11	mm4

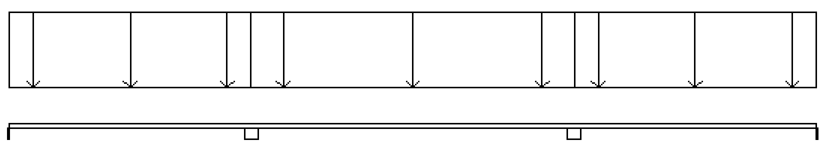
Figure B.1: Calculation of bending moment due to self-weight of model 20-5 by K-bjelke, page 1

Tittel Appendix			Side 2
Prosjekt Model 20-5	Ordre	Sign Yousef	Dato 12-04-2023

1.2 SØYLER OG OPPLEGGSPUNKT [mm]

Opplegg	Søyler på bjelkens underside				Søyler på bjelkens overside			
nr	kode	lengde	h/diameter	b(tverretn)	kode	lengde	h/diameter	b(tverretn)
1	Fri		100					
2	Rektangel	10000	800	5000				
3	Rektangel	10000	800	5000				
4	Fri		100					

1.3 LASTBILDE



Lastfaktorer (brukervalgte)

	Nedbøyning	Risskontroll	Bruddgrense
Permanent last	1.00	1.00	1.00
Variabel last	0.30	1.00	1.00

PSI-Faktor Kategori G :trafikk- parkeringsareal for mellomstore kjøretøy (30kN<kjøretøyvekt<160kN på to akslinger)
Krav maks.nedbøyning Konstruksjoner med alminnelige brukskrav eller estetiske krav

Pålitelighetsklasse: 3	Bjelkens romvekt: 2500 kg/m3

Jevnt fordelt last (kN/m)

Felt nr	Egenvekt	Permanent last	Variabel last
1	216.25	0.01	0.10
2	216.25	0.01	0.10
3	216.25	0.01	0.10

Figure B.2: Calculation of bending moment due to self-weight of model 20-5 by K-bjelke, page 2

Tittel			Side
Appendix			3
Prosjekt	Ordre	Sign	Dato
Model 20-5		Yousef	12-04-2023

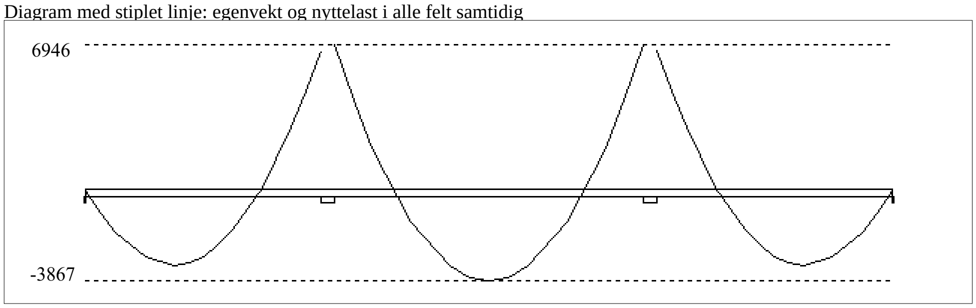
1.4 MATERIALDATA

Korreksjonsfaktor for Emodul pga tilslag	1	Eksponeringsklasse	XD3	XD3
Materialkoeffisient betong	1.5	Korrosjonsømfintlig armering		
Materialkoeffisient stål	1.15	Dimensjonerende levetid		100
Betongkvalitet	B45 (C45/55)			
Tilslagets spesifikke tyngde (kg/m3)	2400			
Sement i fasthetsklasse (R / N / S)	N	**Min. overdekning**	**uk**	**ok**
Armering flytegrense	500	Min krav	60	60
Bøyler flytegrense	500	Toleransekrav +/-	15	15
Relativ fuktighet %	40	Min. nominell overdekning	75	75
Betongens alder ved pålastning (døgn)	28			
Effektiv høyde, h0 (EN 1992-1-1 3.1.4(5))	673			
største tilslagsstørrelse, dg(mm)	22	Kryptall, FI 28_5000		1.47
Korttids Emodul, Ecm	36300	Svinntøyning, FI 0_28		-0.00007
Trykkfasthet, fcd	25.5	Svinntøyning, FI 28_5000		-0.00028
Middel verdi av strekkfasthet, fctm	3.8			
Strekkfasthet, fctd	1.51			

NA.6.2.2(1) Følgende krav til tilslag i betongen er oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

2.1 MOMENTDIAGRAMMER FOR MAKS OG MIN MOMENT I BRUDDGRENSETILSTAND, MED NYTTELAST I UGUNSTIGE FELT

Diagram med stiplet linje: egenvekt og nyttelast i alle felt samtidig



Største negative feltmomenter (strekk i uk)(kNm)

	Bruksgrense		Bruddgrense	
Felt	Mg	Mg+Mp	Mg	Mg+Mp
1	-3245	-3246	-3245	-3246
2	-3865	-3867	-3865	-3867
3	-3245	-3246	-3245	-3246

Mg: permanent last Mp: variabel last

Største positive momenter ved kant av opplegg (kNm)

	Bruksgrense		Bruddgrense	
Opplegg	Mg	Mg+Mp	Mg	Mg+Mp
1	0	0	0	0
2	6945	6946	6945	6946
3	6945	6946	6945	6946
4	0	0	0	0

Figure B.3: Calculation of bending moment due to self-weight of model 20-5 by K-bjelke, page 3

Appendix C

Teddy Input Task Files

This page is kept empty with purpose.

```
! SOFiSTiK Structural Desktop, Group:+1 [System]
! SOFiSTiK Structural Desktop, Task:+3 [Materials]
+PROG AQUA urs:3 $ Materials
HEAD Materials
PAGE UNII 0
ECHO FULL no
ECHO MAT FULL
END
! SOFiSTiK Structural Desktop, Task:-3 [Materials]
! SOFiSTiK Structural Desktop, Task:+4 [Cross Sections]
+PROG AQUA urs:4 $ Cross Sections
HEAD Sections
PAGE UNII 0
ECHO SECT FULL
ECHO PICT YES
ECHO REFP NO
ECHO WIND YES
CTRL REST 3
END
! SOFiSTiK Structural Desktop, Task:-4 [Cross Sections]
! SOFiSTiK Structural Desktop, Task:+5 [Prestressing Systems]
+PROG TENDON urs:5 $ Prestressing Systems
HEAD Prestressing systems
PAGE UNII 0
ECHO FULL NO
ECHO SYSP FULL
END
! SOFiSTiK Structural Desktop, Task:-5 [Prestressing Systems]
! SOFiSTiK Structural Desktop, Task:+6 [Action Manager]
+PROG TEMPLATE urs:6 $ Action Manager
HEAD Actions
PAGE UNII 0
ECHO ACT
END

! SOFiSTiK Structural Desktop, Task:+27 [Input]
+prog template urs:27.1
head
$Bridge Parameters:
sto#lengdeB 90  $ Bridge Total Length L in meter
sto#Htverr 1.55  $ Height of Cross Section in meter
sto#Slen 10      $ Columns Height in meter
sto#Cen2Arm 0.120  $ Length from concrete cover to center of Øl                      cnom = 75+25+40/2

!!trafikk
sto#l_bridge #lengdeB
sto#n_steps  #lengdeB/2              $ Steps for discrete load-postions

$ Loadcase numbers
sto#lc_min1  10000              $ Starting Loadcase for location in carriageway CASE A
sto#lc_min2  10500              $ Starting Loadcase for location in carriageway CASE A

sto#lc_max1  0                 $ Last Loadcase
sto#lc_max2  0                 $ Last Loadcase

!!UDL loads
sto#lc_minu1  100                $ Starting Loadcase
sto#lc_minu2  103                $ Starting Loadcase

!*!Label Vindlast på søyler
$ -----------------------------------------------------------------------------------------------------
sto#VindSoyT 1.21        $ Linjelast, søyle på tvers av bru. Lokal y-retning i Sofistik [kN/m]
sto#VindSoyL 2.91        $ Linjelast, søyle langs bru.  Lokal Z-retning i Sofistik [kN/m]
end
```

```
$ -------------------------------------------------------------------------------------------------------
! SOFiSTiK Structural Desktop, Task:-27 [Input]
! SOFiSTiK Structural Desktop, Group:+32 [Cross Section]
+PROG AQUA urs:34.1
HEAD Masteroppgaven
UNIT 5
CTRL
CTRL RFCS 0
CTRL FACE -1
CTRL REFD 0
CTRL STYP  FEM
CTRL SCUT 0
CTRL PLAS 1
SECT 1 MNO 1 MRF 2 MRFL 3 FSYM NONE BTYP BEAM TITL "Master Section"
SV   IT 100[o/o] LEVY 85[o/o] LEVZ 85[o/o]
TVAR 'NEFF' VAL 0[mm]
LAY  1 'BOT' TYPE  MIN MRF 2
LAY  2 'TOP' TYPE  MIN MRF 2
LAY  3 'TLEFT' TYPE  MIN MRF 2
LAY  3 'TRIGH' TYPE  MIN MRF 2
$LAY  5 'TORS' TYPE  OPT MRF 2
//lengdearmering
LRF  'TOP' YB -3500+#Cen2Arm*1000 ZB #Cen2Arm*1000 YE 3500-#Cen2Arm*1000 ZE #Cen2Arm*1000 AS 26.80[cm2/m] LAY   M2 T
LRF  'TLEFT' YB -3500+#Cen2Arm*1000 ZB #Cen2Arm*1000 YE -5950 ZE #Cen2Arm*1000 AS 26.80[cm2/m] LAY   M2 TORS PASS D 
LRF  'TRIGH' YB 3500-#Cen2Arm*1000 ZB #Cen2Arm*1000 YE 5950 ZE #Cen2Arm*1000 AS 26.80[cm2/m] LAY   M2 TORS PASS D 32
LRF  'BOT' YB 3500-#Cen2Arm*1000 ZB (#Htverr-#Cen2Arm)*1000 YE -3500+#Cen2Arm*1000 ZE (#Htverr-#Cen2Arm)*1000 AS 26.
prt#Cen2Arm

//shearcut web
CUT  'ZS' ZB 'S' NS 0 MS 0 MNO 1 MRF 2 LAY 1 TYPE  WEB INCL 90

//stress points
SPT  'BOTL' Y -3500 Z #Htverr*1000 MNO 1 FIX FREE
SPT  'BOTR' Y 3500 Z #Htverr*1000 MNO 1 FIX FREE
SPT  'TOPL' Y "=-#NEFF+10,-6000.000192" Z 0 MNO 1 FIX FREE        //konstant
SPT  'TOPR' Y "=#NEFF-10,6000.000192" Z 0 MNO 1 FIX FREE           //konstant

//tverrsnitt
POLY TYPE    O MNO 1
VERT '0100' Y -3500 Z 450 EXP 1      //konstant
VERT '0101' Y -6000 Z 350 EXP 1      //konstant
VERT '0102' Y -6000 Z 0 EXP 1        //konstant
VERT '0103' Y 6000 Z 0 EXP 1         //konstant
VERT '0104' Y 6000 Z 350 EXP 1      //konstant
VERT '0105' Y 3500 Z 450 EXP 1      //konstant
VERT '0106' Y 3500 Z #Htverr*1000 EXP 1
VERT '0107' Y -3500 Z #Htverr*1000 EXP 1
VERT '0100' Y -3500 Z 450 EXP 1      //konstan

//søyletverrsnitt
CTRL
CTRL RFCS 0
CTRL FACE -1
CTRL REFD 0
CTRL STYP  FEM
CTRL SCUT 0
CTRL PLAS 1
SECT 2 MNO 1 MRF 2 FSYM NONE BTYP BEAM TITL "soyle"
SV   IT 100[o/o]
LAY  0 TYPE  MIN MRF 2
LAY  1 TYPE  MIN MRF 2
LAY  2 TYPE  MIN MRF 2
POLY TYPE    O MNO 1
VERT '0100' Y 2500 Z -400 EXP 1
```

```
VERT '0101' Y 2500 Z 400 EXP 1
VERT '0102' Y -2500 Z 400 EXP 1
VERT '0103' Y -2500 Z -400 EXP 1
VERT '0100' Y 2500 Z -400 EXP 1
LRF  '0200' YB -2385 ZB -285 YE 2385 ZE -285 AS 32.72000[cm2/m] LAY   M1 TORS ACTI D 25 A 150 DIST EVEN
LRF  '0400' YB -2385 ZB 285 YE 2385 ZE 285 AS 32.72000[cm2/m] LAY   M2 TORS ACTI D 25 A 150 DIST EVEN
CUT  'ZS' ZB 'S' NS 0 MS 0 MNO 1 MRF 2 LAY 1 TYPE  WEB INCL 90
END
END

! SOFiSTiK Structural Desktop, Task:-34 [Cross sections ]
! SOFiSTiK Structural Desktop, Group:+26 [Static System]
+PROG SOFIMSHc urs:8.1 $ Text Interface for Model Creation
HEAD Masteroppgave
SYST 3D gdiv 1000 posz
ctrl TOPO GAXP 1+2
ECHO geom full


let#l1 10
let#l2 10+0.3*#lengdeB
let#l3 10+0.7*#lengdeB
let#l4 10+1*#lengdeB

GAX  'AKSE' TYPC AXIS
GAXA S    0 X       0.0       0.0 SX   1.00000       0.0
GAXA L    500 R       0.0 RA       0.0 RE       0.0

//overbygning

Gaxp 'AKSE' IDS 0 S #l1 Type 'S' IDP 'A1' spt 100 grp 100 ncs 1
Gaxp 'AKSE' IDS 0 S #l2 Type 'S' IDP 'A2' spt 200 grp 200 ncs 1
Gaxp 'AKSE' IDS 0 S #l3 Type 'S' IDP 'A3' spt 300 grp 300 ncs 1
Gaxp 'AKSE' IDS 0 S #l4 Type 'S' IDP 'A4' spt 400 grp 400 ncs 1

//Akse 1
//Venstre side
 coor GAXP id AKSE idp A1
SPT       1 X 0  -1  #Htverr
 sptp f ref 100 grp 10
SPTS NO 102 REF 2 TYPE 'CYY' CP 1000000 GRP 10 AR 1
SPTS NO 103 REF 2 TYPE 'CZZ' CP 1000000 GRP 10 AR 1
SPT       2 X   0   -1  #Htverr+0.2   FIX PPMM

 //Høyre side
 coor GAXP id AKSE idp A1
SPT       3 X 0  +1  #Htverr
 sptp f ref 100 grp 10
$SPTS NO 104 REF 4 TYPE 'CYY' CP 1000000 GRP 10 AR 1
SPTS NO 105 REF 4 TYPE 'CZZ' CP 1000000 GRP 10 AR 1
SPT       4 X   0   +1  #Htverr+0.2   FIX PPMM

//Akse 4
//Venstre side
 coor GAXP id AKSE idp A4
SPT       5 X 0  -1  #Htverr
 sptp f ref 400 grp 40
SPTS NO 402 REF 6 TYPE 'CYY' CP 1000000 GRP 40 AR 1
SPTS NO 403 REF 6 TYPE 'CZZ' CP 1000000 GRP 40 AR 1
SPT       6 X   0   -1  #Htverr+0.2   FIX PPMM

 //Høyre side
 coor GAXP id AKSE idp A4
SPT       7 X 0  +1  #Htverr
```

```
 sptp f ref 400 grp 40
$SPTS NO 404 REF 8 TYPE 'CYY' CP 1000000 GRP 40 AR 1
SPTS NO 405 REF 8 TYPE 'CZZ' CP 1000000 GRP 40 AR 1
SPT       8 X    0    +1  #Htverr+0.2   FIX PPMM

//Akse 2
coor GAXP id AKSE idp A2 $ Bestemmer hvor jeg skal jobbe, altså langs linjen "AKSE" ved referanse pkt P3
 spt 20 x 0 0 #Htverr $FIX F->#noder $lager node som skal kobles til overbygning i akse 3 med uendelig stiv kobling(
  sptp f ref 200 grp 20
coor GAXP id AKSE idp A2 $ Bestemmer hvor jeg skal jobbe, altså langs linjen "AKSE" ved referanse pkt P3
 spt 21 x 0 0 #Slen+#Htverr FIX PPMM  $Lager node i bunn av søylen, søylen er gitt på å være fastinnspent med PPMM
 sln no 20 npa 20 npe 21 grp 20 styp 'N' sno 2 $lager element mellom nodene jeg har laget under ref P2. Type 'N' bet

//Akse 3
coor GAXP id AKSE idp A3 $ Bestemmer hvor jeg skal jobbe, altså langs linjen "AKSE" ved referanse pkt P3
 spt 30 x 0 0 #Htverr $FIX F->#noder $lager node som skal kobles til overbygning i akse 3 med uendelig stiv kobling(
  sptp f ref 300 grp 30
coor GAXP id AKSE idp A3 $ Bestemmer hvor jeg skal jobbe, altså langs linjen "AKSE" ved referanse pkt P3
 spt 31 x 0 0 #Slen+#Htverr FIX PPMM  $Lager node i bunn av søylen, søylen er gitt på å være fastinnspent med PPMM
 sln no 30 npa 30 npe 31 grp 30 styp 'N' sno 2 $lager element mellom nodene jeg har laget under ref P2. Type 'N' bet

end
$ -----------------------------------------------------------------------------------------------------
! SOFiSTiK Structural Desktop, Task:-29 [Static system]
! SOFiSTiK Structural Desktop, Task:+30 [Mesh]
+prog sofimshc urs:2
head definition of supports and system generation
syst rest
ctrl mesh 1   $ activate meshing of structrual system into beam elements
ctrl hmin 1 $ mesh size: maximum length of beam elements
end

+prog aqua urs:3
head interpolation of sections along axis
echo full yes
inte 0
end

! SOFiSTiK Structural Desktop, Task:-30 [Mesh]
! SOFiSTiK Structural Desktop, Group:+78 [Prestressing]
+PROG TENDON urs:16 $ Prestressing systems
HEAD Prestressing systems
PAGE UNII 0
ECHO FULL NO
ECHO SYSP FULL
echo load yes
SYSP NOPS 1 COMP BBV TAPP ETA NO 0 MAT 5 ZV 6642  AZ 4050 LITZ 27 MINR 7.4 BETA 0.29 BETG 0.29 MUE 0.18 ECC 21 SP 6 

END

! SOFiSTiK Structural Desktop, Task:-79 [Prestressing system]
! SOFiSTiK Structural Desktop, Task:+80 [Prestressing Cables ]
+prog tendon urs:34.2
head
$ ------------------------------------------------------------------------------------------------------
$ I denne filen slettes all tidligere input av spennkabler, slik at ingenting henger igjen ved ny kjøring.
$ ------------------------------------------------------------------------------------------------------
tdel 0 0 0
end

+prog tendon  urs:176.3 $ Lage kabelføring

  head Kabelføring
let#l1 10
```

```
let#l2 10+0.3*#lengdeB
let#l3 10+0.7*#lengdeB
let#l4 10+1*#lengdeB

AXES NOH     1       TYPE refx 'AKSE'       kind beam         $24nr of element in each span
Topp 1 kind refx S #l1 SP 1
     1 kind refx s #l2 SP 2
     1 kind refx s #l3 SP 3
     1 kind refx s #l4 SP -4


#define Spennfoering
TGEO NOG #tendonNr   NOH  1   NOPS  1  TITL  'Forste spenn'
  PTUV TYPE refx s  #l1            U #B1(#1)  V #Htverr/2   KIND PRFX $start punkt
  PTUV TYPE refx s  (#l2+#l1)/2    U #B1(#1)  V #Htverr-0.2  dvs 0.0  $ Bunnpunkt
  PTUV TYPE refx s  #l2            U #B1(#1)  V 0.2 dvs 0.0   $ Toppunkt
  PTUV TYPE refx s  (#l2+#l3)/2    U #B1(#1)  V #Htverr-0.2  dvs 0.0  $ Bunnpunkt
  PTUV TYPE refx s  #l3            U #B1(#1)  V 0.2  dvs 0.0  $ Toppunkt
  PTUV TYPE refx s  (#l4+#l3)/2    U #B1(#1)  V #Htverr-0.2 dvs 0.0   $ Bunnpunkt
  PTUV TYPE refx s  #l4            U #B1(#1)  V #Htverr/2  $ endepunkt
#enddef

let#B1 -2.34,-1.82,-1.3,-0.78,-0.26,0.26,0.78,1.3,1.82,2.34
let#tendonNr 1
Loop#1 B1
#include Spennfoering
CS      ICS1     15 16  0       $ ICS1 = tensioning tendon, ICS2 = Grouting

$Psig kind #O1(#1) anws 'TS'
Psig kind 'RILE' anws 'TS'
TEND NOT #tendonNr  NOG  #tendonNr  NTEN 1  TITL  'Forste spenn' LC 50  Type REFx  From #l1 to  #l4
let#tendonNr #tendonNr+1
endloop
END

! SOFiSTiK Structural Desktop, Task:-80 [Prestressing Cables ]
! SOFiSTiK Structural Desktop, Task:+81 [Plot Prestressing cables]

+PROG TENDON urs:34.1   $Post-tensioning system definition
HEAD Post-tensioning system definition
$ ---------------------------------------------------------------------------------------------------
$ I denne filen defineres plot (output) av spennkablene (tendons).
$
$ !Forklaring!: Plot av spennkablene er nyttig for å kontrollere om de er lagt inn riktig.
$              OBS: Alle mål MÅ være i meter
$              Riktig antall kabler inkluderes med å kopiere/fjerne
$ ---------------------------------------------------------------------------------------------------
let#antallkabler 18

ECHO PLOT FULL
SCHH h2 0.18 h5 0 h6 0
loop#1 #antallkabler
let#nr 1
PLOT GEOE NO #nr+#1 FACH 5 TYPG DUTE DIA - PCS 1
endloop
loop#2 #antallkabler
let#nr 1
PLOT FACT NO #nr+#2 FACH 50
endloop

end
! SOFiSTiK Structural Desktop, Task:-81 [Plot Prestressing cables]

! SOFiSTiK Structural Desktop, Group:+35 [Actions]
```

```
! SOFiSTiK Structural Desktop, Task:+37 [Actions]
+PROG SOFILOAD urs:33.1 $ Definisjon av lasttyper (Actions)
HEAD Definisjon av lasttyper (Actions)

$ -------------------------------------------------------------------------------------------------------
$ I denne filen defineres aktuelle ACTIONS med tilhørende regler for kombinering og faktorer.
$
$ !Forklaring!: Se forklaring nedenfor av de forskjellige kommando-valgene.
$              Aktiver aktuelle ACTIONS.
$ --------------------------------------------------------------------------------------------------------
ctrl warn 12285
PAGE UNII 0
echo full yes

$ action factors :
$
ACT 'G_1'   GAMU 1.35 GAMF 1.0  PSI0 1 1 1 1           PART 'G'        SUP PERM TITL 'dead load g_1'
ACT 'G_2'   GAMU 1.35 GAMF 1.0  PSI0 1 1 1 1           PART 'G'        SUP PERM TITL 'dead load g_2'
ACT 'G_9'   GAMU 1.0 GAMF 1.0  PSI0 1 1 1 1            PART 'G'        SUP PERM TITL 'horisontalt jordtrykk, aktivt
ACT 'P'     GAMU 1.1 0.9                               PART P                  TITL 'prestress  '
ACT 'C_1'   GAMU 1 1  - SUP PERM                       PART G                  TITL 'C+S till traffic opening'
ACT 'C_2'   GAMU 1 1  - SUP cond                       PART G                  TITL 'C+S after traffic opening'
ACT 'F'     GAMU 1 0           PSI0 1 1 1 1           PART Q          SUP COND TITL 'settlement'
ACT 'FR'     GAMU 1 0           PSI0 0.7 0.7 0.5 0.8   PART Q          SUP COND TITL 'Friksjonskrefter fra lager'
ACT 'T'      GAMU 1.2 0         PSI0 0.7 0.6 0 0.8     PART Q          SUP EXCL TITL 'temperature loading'
ACT 'QT'     GAMU 1.0 0         PSI0 0.7 0.6 0 0.8     PART Q          SUP EXCL TITL 'temperature for lager deforma
ACT 'B'     GAMU 1.35 0         PSI0 1 1 1             PART Q          SUP cond TITL 'erection load'
ACT 'ZW' GAMU 1.6 0 PSI0 0.7 0.6 0 0.8                 PART Q          SUP EXCL  TITL 'wind on traffic'
ACT 'E'    GAMU 1 0 PSI0 1 1 1 1                       PART E          SUP USEX              Titl 'Jordskjelv'
ACT 'EX' TITL 'Seismic loading in X' GAMU 1 0 PSI0 1 PSI1 1 PS1S 1 GAMA 1 PART 'E' SUP EXCL
ACT 'EY' TITL 'Seismic loading in Y' GAMU 1 0 PSI0 1 PSI1 1 PS1S 1 GAMA 1 PART 'E' SUP EXCL
ACT 'EZ' TITL 'Seismic loading in Z' GAMU 1 0 PSI0 1 PSI1 1 PS1S 1 GAMA 1 PART 'E' SUP EXCL
$ -------------------------------------
$ Traffic Load Actions
$ -------------------------------------
ACT 'GR_1'  GAMU 1.35 0 PSI0 0.7 0.7 0.5 0.8           PART Q EXEX TITL "LM1/LM2/Service vehicle/LM4"         $ Last
ACT 'GR_2'  GAMU 1.35 0 PSI0 0.7 0.7 0.5 0.8           PART Q EXEX TITL "gr2 Horizontal Forces"    $ Freq. LM1 + Bral
ACT 'GR_5'  GAMU 1.1  0 PSI0 0    0   0   0            PART Q EXEX TITL 'LM3 spesialtransport'   $ LM3 spesialtrans
ACT 'GR_4'  GAMU 1.35 0 PSI0 0.7 0.7 0.2 0.8           PART Q Exex TITL "gr4 Crowd load"          $ Crowd load + Fo
ACT 'M'    GAMU 1.0 0 PSI0 1 0 0 0                     PART Q    EXEX  TITL 'Eksentrisitet moment'
ACT Y_1 GAMU 1.0 0 1 1 1 - SUP EXEX TITL ' rare without gpc'
ACT Y_3 GAMU 1.0 0 1 1 1 - SUP EXEX TITL ' freq without gpc'
ACT Y_4 GAMU 1.0 0 1 1 1 - SUP EXEX TITL ' perm without gpc'
ACT Y_D GAMU 1.0 0 1 1 1 - SUP EXEX TITL ' (B)desi 6.10b without gpc'     $ same as set B eq 6.10b and sett A eq 6.1
ACT Y_H GAMU 1.0 0 1 1 1 - SUP EXEX TITL ' (B)desi 6.10a without gpc'     $ added for set B eq 6.10a
ACT Y_E GAMU 1.0 0 1 1 1 - SUP EXEX TITL ' earq without gpc'
END

! SOFiSTiK Structural Desktop, Task:-37 [Actions]
! SOFiSTiK Structural Desktop, Group:+36 [Loads]
! SOFiSTiK Structural Desktop, Task:+38 [Permanent loads]

+prog sofiload urs:44.1
Head Laster fra superegenvekt

lc 1 dlz 1 type none titl 'egenlast'

$Egenvekt av asfalt
$-------------------------------------------------------------------------------------
lc 2 type none titl 'asfalt'
  beam grp (100 300 100) type PG Pa 3.5*12 $5m*3kN/m2

$$Egenvekt av kantdrager
$$-------------------------------------------------------------------------------------
```

```
lc 3 type none titl 'kantdrager'
    beam grp (100 300 100) type PG Pa 5 eYA 12/2*1000 $5m*3kN/m2
    beam grp (100 300 100) type PG Pa 5 eYA -12/2*1000 $5m*3kN/m2

$$Egenvekt av rekkverk
$$-------------------------------------------------------------------------------------------
lc 4 type none titl 'Rekkverk'
    beam grp (100 300 100) type PG Pa 1 eYA 12/2*1000 $5m*3kN/m2
    beam grp (100 300 100) type PG Pa 1 eYA -12/2*1000 $5m*3kN/m2

end
! SOFiSTiK Structural Desktop, Task:-38 [Permanent loads]
! SOFiSTiK Structural Desktop, Task:+39 [Temperature loads]
+PROG SOFILOAD urs:bri.load2
HEAD Temperatur last
PAGE UNII 0
let#w_N 0.35        $ Kombinasjonsfaktor for jevnt fordelt andel
let#TNexp 33
let#TNcon -37
let#TMheat 10.5
let#TMSoyle 5     $ Lineær varierende temperatur i søyle
let#w_M 0.75         $ Kombinasjonsfaktor for kurvaturandel
let#TMcool 8.0

$ Positiv søylegradient
$ ------------------------------------------------------------------------------------------------------------
LC 80 TYPE T TITL 'Tsummer+posdt +wn*TN-DTZ'
 BEAM GRP (100 300 100)  TYPE DT   #w_N*#TNexp
 BEAM GRP (20 30 10)     TYPE DT   #w_N*#TNexp
 BEAM GRP (100 300 100)  TYPE DTZ -#TMheat
 BEAM GRP (20 30 10)     TYPE DTZ  #TMSoyle

LC 81 TYPE T TITL 'Tsummer+posdt +TN-wm*DTZ'
  BEAM GRP (100 300 100) TYPE DT   #TNexp            $ center-temperature bue
  BEAM GRP (20 30 10)    TYPE DT   #TNexp            $ center-temperature bue
  BEAM GRP (100 300 100) TYPE DTZ -#w_M*#TMheat      $ * -DTZ = upside warm bue
  BEAM GRP (20 30 10)    TYPE DTZ  #TMSoyle          $ Gradient på søyle

LC 82 TYPE T TITL 'Twinter+posdt -wn*TN-DTZ'
 BEAM GRP (100 300 100)  TYPE DT   #w_N*#TNcon
 BEAM GRP (20 30 10)     TYPE DT   #w_N*#TNcon
 BEAM GRP (100 300 100)  TYPE DTZ -#TMheat
 BEAM GRP (20 30 10)     TYPE DTZ  #TMSoyle

LC 83 TYPE T TITL 'Twinter+posdt -TN-wm*DTZ'
 BEAM GRP (100 300 100)  TYPE DT   #TNcon
 BEAM GRP (20 30 10)     TYPE DT   #TNcon
 BEAM GRP (100 300 100)  TYPE DTZ -#w_M*#TMheat
 BEAM GRP (20 30 10)     TYPE DTZ  #TMSoyle

LC 84 TYPE T TITL 'Tsummer+negdt +wn*TN+DTZ'
 BEAM GRP (100 300 100)  TYPE DT   #w_N*#TNexp
 BEAM GRP (20 30 10)     TYPE DT   #w_N*#TNexp
 BEAM GRP (100 300 100)  TYPE DTZ  #TMcool
 BEAM GRP (20 30 10)     TYPE DTZ  #TMSoyle

LC 85 TYPE T TITL 'Tsummer+negdt +TN+wm*DTZ'
 BEAM GRP (100 300 100)  TYPE DT   #TNexp
 BEAM GRP (20 30 10)     TYPE DT   #TNexp
 BEAM GRP (100 300 100)  TYPE DTZ   #w_M*#TMcool
 BEAM GRP (20 30 10)     TYPE DTZ  #TMSoyle

LC 86 TYPE T TITL 'Twinter+negdt -wn*TN+DTZ'
 BEAM GRP (100 300 100)  TYPE DT   #w_N*#TNcon
```

```
 BEAM GRP (20 30 10)      TYPE DT    #w_N*#TNcon
 BEAM GRP (100 300 100)  TYPE DTZ  #TMcool
 BEAM GRP (20 30 10)      TYPE DTZ  #TMSoyle

LC 87 TYPE T TITL 'Twinter+negdt -TN+wm*DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #TNcon
 BEAM GRP (20 30 10)      TYPE DT    #TNcon
 BEAM GRP (100 300 100)  TYPE DTZ  #w_M*#TMcool
 BEAM GRP (20 30 10)      TYPE DTZ  #TMSoyle
$ Negativ søylegradient
$ -----------------------------------------------------------------------------------------------------------
LC 88 TYPE T TITL 'Tsummer+posdt +wn*TN-DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #w_N*#TNexp
 BEAM GRP (20 30 10)      TYPE DT    #w_N*#TNexp
 BEAM GRP (100 300 100)  TYPE DTZ -#TMheat
 BEAM GRP (20 30 10)      TYPE DTZ  -#TMSoyle

LC 89 TYPE T TITL 'Tsummer+posdt +TN-wm*DTZ'
  BEAM GRP (100 300 100) TYPE DT    #TNexp            $ center-temperature bue
  BEAM GRP (20 30 10)     TYPE DT    #TNexp            $ center-temperature bue
  BEAM GRP (100 300 100) TYPE DTZ -#w_M*#TMheat       $ * -DTZ = upside warm bue
  BEAM GRP (20 30 10)     TYPE DTZ  -#TMSoyle          $ Gradient på søyle

LC 90 TYPE T TITL 'Twinter+posdt -wn*TN-DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #w_N*#TNcon
 BEAM GRP (20 30 10)      TYPE DT    #w_N*#TNcon
 BEAM GRP (100 300 100)  TYPE DTZ -#TMheat
 BEAM GRP (20 30 10)      TYPE DTZ  -#TMSoyle

LC 91 TYPE T TITL 'Twinter+posdt -TN-wm*DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #TNcon
 BEAM GRP (20 30 10)      TYPE DT    #TNcon
 BEAM GRP (100 300 100)  TYPE DTZ -#w_M*#TMheat
 BEAM GRP (20 30 10)      TYPE DTZ  -#TMSoyle

LC 92 TYPE T TITL 'Tsummer+negdt +wn*TN+DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #w_N*#TNexp
 BEAM GRP (20 30 10)      TYPE DT    #w_N*#TNexp
 BEAM GRP (100 300 100)  TYPE DTZ  #TMcool
 BEAM GRP (20 30 10)      TYPE DTZ  -#TMSoyle

LC 93 TYPE T TITL 'Tsummer+negdt +TN+wm*DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #TNexp
 BEAM GRP (20 30 10)      TYPE DT    #TNexp
 BEAM GRP (100 300 100)  TYPE DTZ   #w_M*#TMcool
 BEAM GRP (20 30 10)      TYPE DTZ  -#TMSoyle

LC 94 TYPE T TITL 'Twinter+negdt -wn*TN+DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #w_N*#TNcon
 BEAM GRP (20 30 10)      TYPE DT    #w_N*#TNcon
 BEAM GRP (100 300 100)  TYPE DTZ  #TMcool
 BEAM GRP (20 30 10)      TYPE DTZ  -#TMSoyle

LC 95 TYPE T TITL 'Twinter+negdt -TN+wm*DTZ'
 BEAM GRP (100 300 100)  TYPE DT    #TNcon
 BEAM GRP (20 30 10)      TYPE DT    #TNcon
 BEAM GRP (100 300 100)  TYPE DTZ  #w_M*#TMcool
 BEAM GRP (20 30 10)      TYPE DTZ  -#TMSoyle

END
! SOFiSTiK Structural Desktop, Task:-39 [Temperature loads]
! SOFiSTiK Structural Desktop, Task:+59 [Wind load calculations]
+prog template urs:45.1
head
```

```
let#z        10
let#cdir      1
let#cseason   1
let#calt      1
let#cprob     1                                $for 50 years period
let#vb_0     22
let#vb       #cdir*#cseason*#calt*#cprob*#vb_0
$Stedvindhatighet Vm(z)
$terrenkategori 2
let#kr        0.19
let#z_min     4
let#z0        0.05
let#c0_z      1
$ for z_min < z < z_max
let#cr_z     #kr*LOG(#z/#z0)
let#Vm_z     #cr_z*#c0_z*#vb

prt#Vm_z

$Turbulensintensitet Iv(z)     $ for z_min < z < z_max
let#k1     1
let#Iv_z   #k1/(#c0_z*LOG(#z/#z0))
prt#Iv_z

$Vindhastighetstryket qp(z)
let#qm_z  (0.5*1.25*(#Vm_z)**2)/1000  $ KN/m2
prt#qm_z
let#kp   3.5
let#qp_z (1+2*#kp*#Iv_z)*#qm_z         $ KN/m2
prt#qp_z

$Vindhastighetstrykket på trafikk, qp_trafikk

let#vm_trafikk_max_z 35/SQR(1+2*#kp*#Iv_z)
prt#vm_trafikk_max_z
let#vm_trafikk_z MIN(#Vm_z,#vm_trafikk_max_z)        $ m/s
prt#vm_trafikk_z

 $Stedvindhastighetstrykket på trafikk
 let#q_p_trafikk_z (1+2*#kp*#Iv_z)*0.5*1.25*#vm_trafikk_z**2/1000  $ KN/m2
 prt#q_p_trafikk_z

$Vindlast på brubane uten trafikk og med rekkverk
let#b_bru  12
let#d_bru  #Htverr
let#d_kant 0.17
let#d_rek 1.2
let#d_tot #d_bru+#d_kant+0.6
prt#d_tot
let#rio #b_bru/#d_tot
prt#rio

IF ( 4<#rio) ! condition 1
   LET#cfx0 1.3
ELSE  ! or condition 2
   LET#cfx0 -0.29*#rio+2.46
ENDIF
prt#cfx0
let#cfz 0.9
prt#cfz 0.9

$kraft i y-retning         tverretning definert som x-retning i eurocode, mens i Sofistik er tverretning definert sor
let#qy #cfx0*#qp_z*#d_tot      $ KN/m
prt#qy
```

```
$kraft i x-retning
let#qx 0.25*#qy           $ KN/m
prt#qx

$ kraft i z-retning
let#qz #cfz*#qp_z*#b_bru
prt#qz
let#e #b_bru/4

$Vindlast på brubane m/trafikk
$kraftfaktor i x-retning (8.3.1)

let#b_bruT #b_bru
let#d_bruT #d_bru
let#d_kantT #d_kant
let#d_bilT 2
let#d_totT  #d_bru+#d_kantT+#d_bilT
prt#d_totT
let#rioT #b_bruT/#d_totT

IF ( 4<#rioT)
   LET#cfx0T 1.3
ELSE
   LET#cfx0T -0.29*#rioT+2.46
ENDIF
prt#cfx0T

$kraft i y-retning                   tverretning definert som x-retning i eurocode, mens i Sofistik er tverretning de
let#qyT #cfx0T*#q_p_trafikk_z*#d_totT     $ KN/m
prt#qyT

$kraft i x-retning
let#qxT 0.25*#qyT           $ KN/m
prt#qxT

$ kraft i z-retning
let#qzT #cfz*#q_p_trafikk_z*#b_bru  $ KN/m
prt#qzT
let#e #b_bru/4
prt#e


$max kraft av med eller uten trafikk
sto#qnyx MAX(#qx,#qxT)
sto#qnyy MAX(#qy,#qyT)
sto#qnyz MAX(#qz,#qzT)
prt#qnyx
prt#qnyy
prt#qnyz


!*! Vindlast  bruoverbygning
$ ------------------------------------------------------------------------------------------------------

sto#VindVert #qnyz         $ Linjelast, overbygning. Global Z-retning i Sofistik [kN/m]
sto#VindTver #qnyy            $ Linjelast, overbygning. Global Y-retning i Sofistik [kN/m]
sto#VindMom 40.6    $ må sjekkes        $ Linjemoment, overbygning. Bidrag fra eksentrisk vind i Y og Z-retning i So
sto#VindLang #qnyx          $ Linjelast, overbygning. Global X-retning i Sofistik [kN/m]

end
! SOFiSTiK Structural Desktop, Task:-59 [Wind load calculations]
! SOFiSTiK Structural Desktop, Task:+51 [Wind loads]
+prog Sofiload urs:64.1
```

```
Head Vindlaster

$ ---------------------------------------------------------------------------------------------------------------
$ I denne filen defineres lasttilfellene for vindlast.
$
$ !Forklaring!: Vindlast defineres i X, Y og Z retning. Den verste av vind med eller uten trafikk velges. Eventuelt l
$               All input til vindlastene bør gjøres i filen "Input laster".
$ ---------------------------------------------------------------------------------------------------------------


$ + Normalt på bru (eksentrisk)/ - Vertikalt (eksentrisk)
$ ------------------------------------------------------------------------------------------------------------
LC 20 TYPE ZW TITL '+Normalt (eks.)/ -Vertikalt (eks.)'
beam grp (100 300 100)             TYPE PYY  #VindTver      $ Vind i Y-retning
beam grp (100 300 100)             TYPE MX   -#VindMom      $ Vind i Y- og Z-retning (momentbidrag)
beam grp (100 300 100)             TYPE PZZ  -#VindVert     $ Vind i Z-retning
beam grp (20 30 10)                TYPE PY   #VindSoyT      $ Vind i Y-retning

$ - Normalt på bru (eksentrisk)/ - Vertikalt (eksentrisk)
$ ------------------------------------------------------------------------------------------------------------
LC 21 TYPE ZW TITL '-Normalt (eks.)/ -Vertikalt (eks.)'
beam grp (100 300 100)             TYPE PYY  -#VindTver      $ Vind i Y-retning
beam grp (100 300 100)             TYPE MX   #VindMom        $ Vind i Y- og Z-retning (momentbidrag)
beam grp (100 300 100)             TYPE PZZ  -#VindVert      $ Vind i Z-retning
beam grp (20 30 10)             TYPE PY   -#VindSoyT      $ Vind i Y-retning


$ + Langs bru/ - Vertikalt
$ ------------------------------------------------------------------------------------------------------------
LC 22 TYPE ZW TITL '+Langs bru/ -Vertikalt'
beam grp (100 300 100)             TYPE PXX  #VindLang       $ Vind i x-retning
beam grp (100 300 100)             TYPE PZZ  -#VindVert      $ Vind i Z-retning
beam grp (20 30 10)             TYPE PZ   #VindSoyL       $ Vind i lokal Z-retning


$ - Langs bru/ - Vertikalt
$ ------------------------------------------------------------------------------------------------------------
LC 23 TYPE ZW TITL '-Langs bru/ -Vertikalt '
beam grp (100 300 100)             TYPE PXX  -#VindLang      $ Vind i x-retning
beam grp (100 300 100)             TYPE PZZ  -#VindVert      $ Vind i Z-retning
beam grp (20 30 10)             TYPE PZ   -#VindSoyL      $ Vind i lokal Z-retning

$ + Vertikalt
$ ------------------------------------------------------------------------------------------------------------
LC 24 TYPE ZW TITL '+Vertikalt'
beam grp (100 300 100)             TYPE PZZ  #VindVert      $ Vind i Z-retning


end
! SOFiSTiK Structural Desktop, Task:-51 [Wind loads]
! SOFiSTiK Structural Desktop, Task:+41 [Traffic loads]

+PROG SOFILOAD urs:21 $ Traffic Loader
  HEAD Definition OF LOAD TRAINS
  PAGE UNII 0
  echo lane extr
  $ECHO OPT LOAD VAL NO $ Loads

let#l2 10+0.3*#lengdeB             $10 is start staion, 0.3 is fisrt span length
let#l3 10+0.7*#lengdeB
let#l4 10+1*#lengdeB

  LANE Akse TYPE EC WL -6 WR 6 YLA -6 YRA 6 $$
  Sa 10  se #l2
```

```
  Sa #l2  se #l3
  Sa #l3  se #l4


$ Lm1 laster
  LC NO 1200 TYPE 'none' TITL 'EN 1991-2 Load model LM1 P 300'
  TRAI USER P5 2.5 P6 0 P7 3.5 P8 1 P9 0 PFAC 1 WIDT 3 $$
   XCON 0 V 0 FUGA 1 FRB 0 DAB 0 BOGI 0 FRBO 0 DABO 0 WHEE 0 FRWH 0 DAWH 0
  TRPL P 300 B 2 BW 0.4 LW 0.4
  TRPL P 300 B 2 BW 0.4 LW 0.4 A 1.2
  TRBL P 16.2 PHI 16
  $
$ Lm1 laster
  LC NO 1201 TYPE 'none' TITL 'EN 1991-2 Load model LM1 P 200'
  TRAI USER P5 2.5 P6 0 P7 3.5 P8 1 P9 0 PFAC 1 WIDT 3 $$
   XCON 0 V 0 FUGA 1 FRB 0 DAB 0 BOGI 0 FRBO 0 DABO 0 WHEE 0 FRWH 0 DAWH 0
  TRPL P 200 B 2 BW 0.4 LW 0.4
  TRPL P 200 B 2 BW 0.4 LW 0.4 A 1.2
  TRBL P 7.5 PHI 16
  $
$ Lm1 laster
  LC NO 1202 TYPE 'none' TITL 'EN 1991-2 Load model LM1 P 100'
  TRAI USER P5 2.5 P6 0 P7 3.5 P8 1 P9 0 PFAC 1 WIDT 3 $$
   XCON 0 V 0 FUGA 1 FRB 0 DAB 0 BOGI 0 FRBO 0 DABO 0 WHEE 0 FRWH 0 DAWH 0
  TRPL P 100 B 2 BW 0.4 LW 0.4
  TRPL P 100 B 2 BW 0.4 LW 0.4 A 1.2
  TRBL P 7.5 PHI 16
  $
  $ LM2 laster
  LC NO 1204 TYPE 'none' TITL 'Single Point Load LM2'
  TRAI USER P5 0 P6 0 P7 0 P9 0 PFAC 1 WIDT 3 XCON 0 V 0 FUGA 1 FRB 0 DAB 0 $$
       BOGI 0 FRBO 0 DABO 0 WHEE 0 FRWH 0 DAWH 0
  TRPL P 400 B 2 BW 0.6 LW 0.35

$ Bremselast lengderetning
  LC NO 1211 TYPE 'none' TITL 'EN 1991-2 Load model LM1 P 0'
  TRAI USER P5 2.5 P6 0 P7 3.5 P8 1 P9 0 PFAC 1 WIDT 3
  TRPL PB 457.2
  $
$ Bremselast tverretning
  LC NO 1212 TYPE 'none' TITL 'EN 1991-2 Load model LM1 P 0'
  TRAI USER P5 2.5 P6 0 P7 3.5 P8 1 P9 0 PFAC 1 WIDT 3
  TRPL Pw 114.3
  $
$Engangstransport
let#last 75
lc NO 1207 TYPE 'none' TITL 'SVV Load model LM3 nummer 1'
TRAI USER P5 0 P6 0 widt 3
loop#1 18
if #1==0
trpl p #last a 0    y 0.75   bw 1.2 lw 0.15
trpl p #last a 0    y -0.75  bw 1.2 lw 0.15
else
trpl p #last a 1.5  y 0.75   bw 1.2 lw 0.15
trpl p #last a 0    y -0.75  bw 1.2 lw 0.15
endif
endloop
lc NO 1208 TYPE 'none' TITL 'SVV Load model LM3 nummer 2'
TRAI USER P5 0 P6 0 widt 3
loop#1 30
if #1==0
trpl p #last a 0    y 0.75   bw 1.2 lw 0.15
trpl p #last a 0    y -0.75  bw 1.2 lw 0.15
elseif #1<15
```

```
trpl p #last a 1.5  y 0.75   bw 1.2 lw 0.15
trpl p #last a 0    y -0.75  bw 1.2 lw 0.15
elseif #1==15
trpl p #last a 12   y 0.75   bw 1.2 lw 0.15
trpl p #last a 0    y -0.75  bw 1.2 lw 0.15
else
trpl p #last a 1.5  y 0.75   bw 1.2 lw 0.15
trpl p #last a 0    y -0.75  bw 1.2 lw 0.15
endif
endloop
end

+PROG SOFILOAD urs:2
HEAD 'Load Stepping LM1 TS'
ECHO FULL NO
ECHO LOAD YES

sto#start 10


let#trai 1200,1201,1202



$ CASE 1 (right most +y local)
$ Laneset 10

  LOOP#j #n_steps                                                   $ Loop for loadcases
    LET#pos  1+#j
        LC #lc_min1 TYPE NONE  TITL 'LM1:TS Lane 10/11/12 Step#pos'
        COPY NO #trai(0) FACT 1.0 TYPE GR0 REF 'AKSE.10' DX #j*#l_Bridge/(#n_steps-1)+#start wide 1
        COPY NO #trai(1) FACT 1.0 TYPE GR0 REF 'AKSE.11' DX #j*#l_Bridge/(#n_steps-1)+#start wide 1
        COPY NO #trai(2) FACT 1.0 TYPE GR0 REF 'AKSE.12' DX #j*#l_Bridge/(#n_steps-1)+#start wide 1
        LET#lc_min1  #lc_min1+1
   ENDLOOP
 sto#lc_max1 #lc_min1-1

$ CASE 2 (left most -y local)
$ Laneset 20

  LOOP#j #n_steps                                                   $ Loop for loadcases
    LET#pos  1+#j
        LC #lc_min2 TYPE NONE  TITL 'LM1:TS Lane 20/21/22 Step#pos'
        COPY NO #trai(0) FACT 1.0 TYPE GR0 REF 'AKSE.20' DX #j*#l_Bridge/(#n_steps-1)+#start  wide 1
        COPY NO #trai(1) FACT 1.0 TYPE GR0 REF 'AKSE.21' DX #j*#l_Bridge/(#n_steps-1)+#start  wide 1
        COPY NO #trai(2) FACT 1.0 TYPE GR0 REF 'AKSE.22' DX #j*#l_Bridge/(#n_steps-1)+#start  wide 1
        LET#lc_min2  #lc_min2+1
   ENDLOOP
 sto#lc_max2 #lc_min2-1

+PROG SOFILOAD urs:6
HEAD 'Load Stepping LM1 UDL'
ECHO FULL NO
ECHO LOAD YES

sto#n_spans  3                   $ Number of spans
let#trai 1200,1201,1202             $ Load trains, defined in advanced

$ Loadcase numbers

$ sto#lc_minur  560                 $ Starting Loadcase

sto#lc_maxu1  0                  $ Last Loadcase
sto#lc_maxu2  0                  $ Last Loadcase
```

```
sto#lc_maxu3  0                  $ Last Loadcase
sto#lc_maxu4  0                  $ Last Loadcase
$ sto#lc_maxur  0                  $ Last Loadcase


$ CASE 1 (right most +y local)
$ Laneset 10

  LOOP#j #n_spans                                       $ Loop for loadcases
    LET#span  1+#j
        LC #lc_minu1 TYPE NONE  TITL 'LM1:UDL Lane 10/11/12 Span#span'
        COPY NO #trai(0) FACT 1.0 TYPE GRU REF 'AKSE.10' DX 0 FROM #span TO - INC 0 wide 1
        COPY NO #trai(1) FACT 1.0 TYPE GRU REF 'AKSE.11' DX 0 FROM #span TO - INC 0 wide 1
        COPY NO #trai(2) FACT 1.0 TYPE GRU REF 'AKSE.12' DX 0 FROM #span TO - INC 0 wide 1
        COPY NO #trai(2) FACT 1.0 TYPE GRU REF 'AKSE.13' DX 0 FROM #span TO - INC 0 wide 1
$        COPY NO #trai(2) FACT 1.0 TYPE GRU REF 'AKSE.13' DX 0 FROM #span TO - INC 0
        LET#lc_minu1  #lc_minu1+1
   ENDLOOP
 sto#lc_maxu1 #lc_minu1-1

$ CASE 2 (left most -y local)
$ Laneset 20

  LOOP#j #n_spans                                       $ Loop for loadcases
    LET#span  1+#j
        LC #lc_minu2 TYPE NONE  TITL 'LM1:UDL Lane 20/21/22 Span#span'
        COPY NO #trai(0) FACT 1.0 TYPE GRU REF 'AKSE.20' DX 0 FROM #span TO - INC 0 wide 1
        COPY NO #trai(1) FACT 1.0 TYPE GRU REF 'AKSE.21' DX 0 FROM #span TO - INC 0 wide 1
        COPY NO #trai(2) FACT 1.0 TYPE GRU REF 'AKSE.22' DX 0 FROM #span TO - INC 0 wide 1
        COPY NO #trai(2) FACT 1.0 TYPE GRU REF 'AKSE.23' DX 0 FROM #span TO - INC 0 wide 1
$        COPY NO #trai(2) FACT 1.0 TYPE GRU REF 'AKSE.23' DX 0 FROM #span TO - INC 0
        LET#lc_minu2  #lc_minu2+1
   ENDLOOP
 sto#lc_maxu2 #lc_minu2-1

END



! SOFiSTiK Structural Desktop, Task:-44 [Load Stepping Method - LM1:UDL]
! SOFiSTiK Structural Desktop, Group:+46 [Static Analysis]
! SOFiSTiK Structural Desktop, Task:+47 [ASE model]
+prog ase urs:47.1
head
lc all
end
! SOFiSTiK Structural Desktop, Task:-47 [ASE model]
! SOFiSTiK Structural Desktop, Task:+48 [Traffic Load Envelope]


!+!Kapitel Superposisjon av trafikk
$ *************************************************************
+PROG MAXIMA urs:41.2 $ Superposisjon av trafikk
  HEAD Superposition of traffic

  #define settings
     PAGE UNII 0  ! standard input units
     echo tabs yes
     ctrl warn 34 ;
     ctrl warn 2034 ;
     ctrl warn 2112 ;
     ctrl warn 2008 ;
     ctrl warn 12149 ;
  $   echo load,fact
```

```
  #enddef
#define SUPP
 $Beam elements
supp comb #co extr mami etyp 'BEAM*' type N    LC  #lc+1     titl  '#title'
supp comb #co extr mami etyp 'BEAM*' type VY   LC  #lc+3     titl  '#title'
supp comb #co extr mami etyp 'BEAM*' type VZ   LC  #lc+5     titl  '#title'
supp comb #co extr mami etyp 'BEAM*' type MT   LC  #lc+7     titl  '#title'
supp comb #co extr mami etyp 'BEAM*' type MY   LC  #lc+9     titl  '#title'
supp comb #co extr mami etyp 'BEAM*' type MZ   LC  #lc+11    titl  '#title'
$Fjærer
supp comb #co extr mami etyp 'SPRI' type P    LC  #lc+29     titl  '#title'
supp comb #co extr mami etyp 'SPRI' type PT   LC  #lc+31     titl  '#title'
$Noder
supp comb #co extr mami etyp 'NODE' type PX   LC  #lc+33    titl  '#title'
supp comb #co extr mami etyp 'NODE' type PY   LC  #lc+35    titl  '#title'
supp comb #co extr mami etyp 'NODE' type PZ   LC  #lc+37    titl  '#title'
supp comb #co extr mami etyp 'NODE' type UX   LC  #lc+39   titl  '#title'
supp comb #co extr mami etyp 'NODE' type UY   LC  #lc+41   titl  '#title'
supp comb #co extr mami etyp 'NODE' type UZ   LC  #lc+43   titl  '#title'
#enddef


  #include settings

    COMB 4 EXTR STAN BASE 0 TYPE none
    lc (#lc_minu1  #lc_maxu1 1) TYPE cond
    Let#co 4   $ Combination rule number
    let#lc 11000
    let#title 'UDL LM1'
    #include SUPP
    COMB 5 EXTR STAN BASE 0 TYPE none
     lc (#lc_minu2  #lc_maxu2 1) TYPE cond
    Let#co 5   $ Combination rule number
    let#lc 11100
    let#title 'UDL LM1'
    #include SUPP

End
+PROG MAXIMA urs:41.3 $ Superposisjon av trafikk
  HEAD Superposition of traffic
  #include settings
    COMB 1 EXTR STAN BASE 0 TYPE GR_1 $ kun vertikale laster fra LM1
    lc (#lc_min1  #lc_max1 1) TYPE A1
    lc (#lc_min2  #lc_max2 1) TYPE A1
    lc (11000  11050 1) TYPE A2
    lc (11100  11150 1) TYPE A2
$    lc (10900  10950 1) TYPE A2
$    lc (10950  11000 1) TYPE A2
    Let#co 1   $ Combination rule number
    let#lc 700
    let#title 'LM1'
    #include SUPP
    End
! SOFiSTiK Structural Desktop, Task:-48 [Traffic Load Envelope]
! SOFiSTiK Structural Desktop, Group:+53 [CSM]
! SOFiSTiK Structural Desktop, Task:+56 [Construction Stages]
+PROG CSM urs:54 $ Construction Stages
HEAD Calculation of Construction Stages
PAGE UNII 0
CTRL OPT DL VAL AUTO
CTRL OPT BEAM VAL -
CTRL OPT CREP VAL RCRE
CTRL OPT RELZ VAL AUTO
CTRL OPT CANT VAL 12
```

```
CTRL OPT CAST VAL 0
CTRL OPT STOR VAL 1
CTRL OPT PROB VAL LINE V2 80
CTRL OPT NMAT VAL NO
CTRL OPT EMOD VAL AUTO
CTRL OPT GPCS VAL 0


$ Table of Construction Stages   (hvordan bygges bruen)
CS 5 TYPE G_1 TITL 'Søyler'
CS 6 TYPE C_1 TITL 'Creep søyler' T 28 RH 70 TEMP 10 NCRE 1
CS 10 TYPE G_1 TITL 'Overbygning'
CS 15 TYPE P TITL 'oppspenning'
CS 20 TYPE G_2 TITL 'Superegenvekt'
CS 30 TYPE C_2 TITL 'Creep until t-infinite' T 36500 RH 70 TEMP 10 NCRE 3


$ Table of Groups
GRP 10 ICS1 10 ATIL - HFIX 99999 BEDD 10 SITU - T0 7 FAC1 1 PHIF 1
GRP 20 ICS1 5 ATIL - HFIX 99999 BEDD 10 SITU - T0 7 FAC1 1 PHIF 1
GRP 30 ICS1 5 ATIL - HFIX 99999 BEDD 10 SITU - T0 7 FAC1 1 PHIF 1
GRP 40 ICS1 10 ATIL - HFIX 99999 BEDD 10 SITU - T0 7 FAC1 1 PHIF 1
GRP 100 ICS1 10 ATIL - HFIX 99999 BEDD 10 SITU - T0 7 FAC1 1 PHIF 1
GRP 200 ICS1 10 ATIL - HFIX 99999 BEDD 10 SITU - T0 7 FAC1 1 PHIF 1
GRP 300 ICS1 10 ATIL - HFIX 99999 BEDD 10 SITU - T0 7 FAC1 1 PHIF 1
$ Table of Loads
Lc 2 ICS1 20 atil -
Lc 3 ICS1 20 atil -
Lc 4 ICS1 20 atil -
END

+apply "$(NAME)_csm.dat"
! SOFiSTiK Structural Desktop, Task:-56 [Construction Stages]
! SOFiSTiK Structural Desktop, Group:+42 [Load Combinations]
! SOFiSTiK Structural Desktop, Task:+50 [Define SUPP]
+prog maxima urs:142.1

#define SUPP
SUPP  ETYP   TYPE  LC    TITL    COMB=#co EXTR=MAMI  from=

$ Nodekrefter
$ ----------------------------------------------------------------------------------------------------------------
     'node'  PX  #lc+20  '#title'
     'node'  PY  #lc+23  '#title'
     'node'  PZ  #lc+25  '#title'
$     'node'  MX  #lc+7  '#title'
$     'node'  MY  #lc+9  '#title'
$     'node'  MZ  #lc+11 '#title'
$ Nodeforskyvninger
$ ----------------------------------------------------------------------------------------------------------------
     'node'  UX    #lc+33 '#title'
     'node'  UY    #lc+35 '#title'
     'node'  UZ    #lc+37 '#title'
     'node'  URX   #lc+39 '#title'
     'node'  URY   #lc+41 '#title'
     'node'  URZ   #lc+43 '#title'

$ Bjelkeelementer - rekkefølge tilpasset dimensjonering
$ ----------------------------------------------------------------------------------------------------------------
     'beam'  MY   #lc+1  '#title'
     'beam'  VZ   #lc+3  '#title'
     'beam'  MT   #lc+5  '#title'
     'beam'  N    #lc+7  '#title'
     'beam'  VY   #lc+9  '#title'
```

```
     'beam'  MZ   #lc+11 '#title'

#enddef

end
! SOFiSTiK Structural Desktop, Task:-50 [Define SUPP]
! SOFiSTiK Structural Desktop, Task:+63 [Combinations without G P C]


!+!Kapitel Presuperposition actions without GPC
$ *************************************************************
+PROG MAXIMA urs:csmmax_3 $ Presuperposition actions without GPC
HEAD Presuperposition actions without GPC

#define settings
   PAGE UNII 0  ! standard input units
$   echo full yes ; $echo fact yes
   ctrl warn 34 ; ctrl warn 44 ; ctrl warn 12283 ; ctrl warn 83 ; ctrl warn 2034
$   echo load,fact
$   echo load yes $ chck yes = load+fact yes $ activate this line for checks
#enddef

#include settings
$ *************************************************************
                              $ Presuperposition actions without GPC
COMB  31 rare BASE - TYPE Y_1 $ combination karakteristisk Y_1

 ACT GR_1     $LM1 og/eller LM2

 ACT ZW
 ACT T


$ *************************************************************
                              $ Presuperposition actions without GPC
COMB  33 freq BASE - TYPE Y_3 $ combination Ofte forekommende Y_3

  ACT GR_1      $LM1 og/eller LM2    $ Freq is used for crack checks, factors from N400 4.3.2.3

  ACT ZW
  ACT T


$ *************************************************************
                              $ Presuperposition actions without GPC
COMB  34 freq BASE - TYPE Y_4 $ combination tilnærmet permanent Y_4

  ACT GR_1     PSI1 0.5     $LM1 og/eller LM2

  ACT ZW       PSI1 0.5
  ACT T        PSI1 0.5


$ *************************************************************
                              $ Presuperposition actions without GPC
COMB  42 desi BASE - TYPE Y_D $ combination design Y_D (B)6.10b/(A)6.10

  ACT GR_1       $LM1 og/eller LM2 eller servicekjøretøy hvis gangbru

  ACT ZW   $GAMU 0.7*1.60  PSI0 1.0
  ACT T


$ *************************************************************
```

```
                              $ Presuperposition actions without GPC
COMB  43 desi BASE - TYPE Y_H $ combination design Y_H (B)6.10a

  ACT GR_1    GAMU 0.7*1.35  PSI0 1.0 $gamma*psi0

  ACT ZW      GAMU 0.7*1.60  PSI0 1.0
  ACT T       GAMU 0.7*1.20  PSI0 1.0



$-----------------------------------------------------------------------
$              SUPP er definert tidligere
$-----------------------------------------------------------------------


$***************************************************************
$ Maxima rare Y_1
$***************************************************************
#include settings
let#co 31   $ Combination rule number
let#lc 3500
let#title 'Y_1 temp.noGPC'
#include SUPP
$***************************************************************
$ Maxima frequent Y_3
$***************************************************************
#include settings
Let#co 33   $ Combination rule number
let#lc 3550
let#title 'Y_3 temp.noGPC'
#include SUPP
$***************************************************************
$ Maxima permanent Y_4
$***************************************************************
#include settings
Let#co 34   $ Combination rule number
let#lc 3600
let#title 'Y_4 temp.noGPC'
#include SUPP
$***************************************************************
$ Maxima design Y_D (B)6.10b/(A)6.10
$***************************************************************
#include settings
Let#co 42   $ Combination rule number
let#lc 3650
let#title 'Y_D temp.noGPC'
#include SUPP
$***************************************************************
$ Maxima design Y_H  (6.10a)
$***************************************************************
#include settings
Let#co 43   $ Combination rule number
let#lc 3700
let#title 'Y_H temp.noGPC'
#include SUPP

END


! SOFiSTiK Structural Desktop, Task:-63 [Combinations without G P C]
! SOFiSTiK Structural Desktop, Task:+57 [Combination of wind and traffic]
#define kombinering
#include $(project)_csmlf.dat
!+!Kapitel Superposition of actions WIND WITH TRAFFIC
```

```
$ *************************************************************
+PROG MAXIMA urs:csmmax_3 $ Superposition of actions WIND WITH TRAFFIC
HEAD Superposition of actions


#define settings
   PAGE UNII 0  ! standard input units
   echo tabs yes
   ctrl warn 34 ; ctrl warn 83 ; ctrl warn 2034
#enddef
#include settings

$ *************************************************************
COMB  11 rare BASE - TYPE - $ combination rare        (karakteristisk)
 #define comb_serv
 ACT G_1
 #include maxactg_1
 Act G_2
 #include maxactg_2
 ACT C
 #include max_act_c
$#if $(spennarm) == ja
 Act P
  #include max_act_p
$$ #endif
#enddef
#include comb_serv
 ACT Y_1
$ *************************************************************
COMB  13 freq BASE - TYPE freq $ combination quasi-permanent (N400 7.7.1)   (ofte forekommende)
  #include comb_serv
  ACT Y_3

$ *************************************************************
COMB  14 freq BASE - TYPE perm $ combination frequent (N400 7.7.1) (tilnærmet permanent)
  #include comb_serv
  ACT Y_4

$ *************************************************************

$ *************************************************************
COMB  21 desi BASE - TYPE - $ combination design (A)6.10

 ACT G_1  gamu 1 0.9
  #include maxactg_1
 Act G_2  gamu 1 0.9
  #include maxactg_2
 ACT C
 #include max_act_c
$ #if $(spennarm) == ja
 Act P
  #include max_act_p
$ #endif
  ACT Y_D

$ *************************************************************
COMB  22 desi BASE - TYPE - $ combination design (B)6.10a
ACT G_1
   #include maxactg_1
Act G_2
 #include maxactg_2
ACT C
 #include max_act_c
  Act P
```

```
  lc 16015
ACT Y_H
$ ************************************************************
COMB  23 desi BASE - TYPE - $ combination design (B)6.10b

ACT G_1  gamu 1.2 1
   #include maxactg_1
Act G_2  gamu 1.2 1
 #include maxactg_2
ACT C
 #include max_act_c
  Act P
  lc 16015
ACT Y_D

$ ************************************************************

COMB  27 rare BASE - TYPE - $ combination rare        (karakteristisk)
#include comb_serv

COMB  28 rare BASE - TYPE - $ combination rare        (karakteristisk)
 ACT C
 #include max_act_c

$*************************************************************
$ Maxima rare
$*************************************************************
#include settings
let#co 11   $ Combination rule number
let#lc 1500
let#title 'RARE'
#include SUPP

$*************************************************************
$ Maxima permanent
$*************************************************************
#include settings
Let#co 13   $ Combination rule number
let#lc 1300
let#title 'FREQ'
#include SUPP
$*************************************************************
$ Maxima frequent- ekstra
$*************************************************************
#include settings
Let#co 14   $ Combination rule number
let#lc 1400
let#title 'PERM'
#include SUPP
$*************************************************************

$ Maxima design (A)6.10
$*************************************************************
#include settings
Let#co 21   $ Combination rule number
let#lc 2100
let#title 'ULSA'
#include SUPP
$*************************************************************
$ Maxima design (B)6.10a
$*************************************************************
#include settings
Let#co 22   $ Combination rule number
let#lc 2200
```

```
let#title 'ULSB_a'
#include SUPP
$*************************************************************
#include settings
Let#co 23   $ Combination rule number
let#lc 2300
let#title 'ULSB_b'
#include SUPP
$*************************************************************
#include settings
Let#co 27  $ Combination rule number
let#lc 2700
$ #if $(spennarm) == ja
let#title 'G+C+P'
let#title 'G+C'
#include SUPP

END
#enddef
#include kombinering
! SOFiSTiK Structural Desktop, Task:-57 [Combination of wind and traffic]
! SOFiSTiK Structural Desktop, Task:+58 [Check combinations ]
+prog maxima urs:58.1
head
trac 2202 etyp beam elem 200036 x 1 opt if
trac 2301 etyp beam elem 200019 x 0 opt if
end
! SOFiSTiK Structural Desktop, Task:-58 [Check combinations ]
! SOFiSTiK Structural Desktop, Task:+52 [Max ULS loads]
+PROG MAXIMA  urs:68.7
HEAD 'ULStot envelope of all ULS (set A and B)'

$ EXTR= Kind of superposition
$ STAN= Standard combination (without safety factor and combination coefficients)
$ TYPE= Type of the result loadcases
COMB 20 EXTR STAN BASE - Type None
LC (2100+1 2100+12 1) TYPE AG1 $ ULS A6.10a
LC (2200+1 2200+12 1) TYPE AG1 $ combination design (B)6.10a
LC (2300+1 2300+12 1) TYPE AG1 $ combination design (B)6.10b

#include settings
let#lc 2000
let#co 20
let#title 'ULStot'
#include SUPP
END
! SOFiSTiK Structural Desktop, Task:-52 [Max ULS loads]
! SOFiSTiK Structural Desktop, Task:+76 [Interactive Lists and Graphics]
+PROG RESULTS urs:76.1
HEAD
$ Begin Page 1
SIZE TYPE "-URS" SPLI "2x1"
$ Begin Grafic/Table/Diagram 1
PICT SC DEFA W DEFA H DEFA SPLT   NO
GRP  NUMB - OPTI  YES
JOIN
DBO
FILT
FLT  ID 1 NAME "beam_for.nr" RULE "200001"
FLT  ID 1 NAME "beam_for.__xi" RULE "0"
FLT  ID 2 NAME "beam_for.nr" RULE "300001"
FLT  ID 2 NAME "beam_for.__xi" RULE "0"
LC   NO 2202
LC   NO 2302
```

```
$ Begin Result 1
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE   MY STYP BEAM REPR DLST
END
! SOFiSTiK Structural Desktop, Task:-76 [Interactive Lists and Graphics]
! SOFiSTiK Structural Desktop, Group:+62 [Design]
! SOFiSTiK Structural Desktop, Task:+70 [Selecting elemetns ]
+prog template urs:70.1
head  Velg elementer
$Sto#elementer 200001,300001,200000-1+0.2*#lengdeB,200000+0.2*#lengdeB,200001+0.2*#lengdeB,200002+0.2*#lengdeB,20001(
Sto#elementer 200016



prt#elementer

end
! SOFiSTiK Structural Desktop, Task:-70 [Selecting elemetns ]
! SOFiSTiK Structural Desktop, Task:+64 [ULS set B eq. 6.10a]
#include $(project)_csmlf.dat
#define cs_design=999


+PROG AQB urs:38.1 $ ULS Ultimate Limit State Design Beams
HEAD ULS Ultimate Limit State Design Beams

PAGE UNII 0  ! standard input units
$ Result-control: necessary reinforcement in following plot
ECHO FULL yes
Echo comb yes
$ Controlling the calculation
  CTRL SVRF   1.0       $ take into account reinforcement for C+S
  CTRL AXIA   -2        $ Biaxial bending, uniaxial extreme fibre stresses in y-z system of section
$  CTRL VM VAL 2 VAL2 2   $ Verdi for skjær og torsjon for lengdearm
$  CTRL DESV 5            $ flange shear design setting
$  ctrl rein fixl

$  ctrl imax 4000
$  ctrl etol 0.002

LC      TYPE 'Y_H' CST   999 REF GROS  gamu 1 gamf 0


REIN LCR   2 RMOD  sing   $
$loop#1 elementer
$BEAM #elementer(#1)  cs auto $
$endloop
BEAM grp 100,200,300  cs auto $

#define GAMU_G_1=GAMU 1.35 GAMF 1.00   $  'dead load'       take sum of all effects of G_1 with 1.35 or 1.00
#define GAMU_G_2=GAMU 1.35 GAMF 1.00   $  'add.dead'        take sum of all effects of G_2 with 1.35 or 1.00
$#define GAMU_G_8=GAMU 1.50 GAMF 0.90   $  'ballast'
$ #define GAMU_G_9=GAMU 1.00 GAMF 1.00   $  'cable stressing' here added in g_1 together with dead load !
                                        $  Please notice: also support lowering is defined as g_1 !
                                        $  This will first sum up dead load + cable stressing + support lowering
                                        $  and then take the sum 1.00 or 1.35 times !
#define GAMU_P  =GAMU 1.1  GAMF 0.9
#define GAMU_C_1=GAMU 1.00 GAMF 1.00
#define GAMU_C_2=GAMU 1.00 GAMF 0.00


#include stage_design
let#LCst 8200
```

```
$Sjekk ved bruåpning
$-----------------------------------------------------------------------------------------------------------
COMB MAXD MY TITL 'ULS lign 6.10a(Max:My)'  LCST #LCst+1   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MIND MY TITL 'ULS lign 6.10a(Min:My)'  LCST #LCst+2   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MAXD VZ TITL 'ULS lign 6.10a(Max:Vz)'  LCST #LCst+3   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MIND VZ TITL 'ULS lign 6.10a(Min:Vz)'  LCST #LCst+4   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MAXD MT TITL 'ULS lign 6.10a(Max:Mt)'  LCST #LCst+5   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MIND MT TITL 'ULS lign 6.10a(Min:Mt)'  LCST #LCst+6   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MAXD N  TITL 'ULS lign 6.10a(Max:N)'   LCST #LCst+7   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MIND N  TITL 'ULS lign 6.10a(Min:N)'   LCST #LCst+8   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MAXD VY TITL 'ULS lign 6.10a(Max:VY)'  LCST #LCst+9   LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MIND VY TITL 'ULS lign 6.10a(Min:VY)'  LCST #LCst+10  LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MAXD MZ TITL 'ULS lign 6.10a(Max:MZ)'  LCST #LCst+11  LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe
COMB MIND MZ TITL 'ULS lign 6.10a(Min:MZ)'  LCST #LCst+12  LC1 G  LC2   C_1 LC3 P LC4 Y_H 1.0 $ Y_ in MAXIMA presupe

$Sjekk etter 100 år
$-----------------------------------------------------------------------------------------------------------
COMB MAXD MY TITL 'ULS lign 6.10a_100(Max:My)'  LCST #LCst+13   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5 Y_H 1.0 $ Y_ in MAX
COMB MIND MY TITL 'ULS lign 6.10a_100(Min:My)'  LCST #LCst+14   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MAXD VZ TITL 'ULS lign 6.10a_100(Max:Vz)'  LCST #LCst+15   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MIND VZ TITL 'ULS lign 6.10a_100(Min:Vz)'  LCST #LCst+16   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MAXD MT TITL 'ULS lign 6.10a_100(Max:Mt)'  LCST #LCst+17   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MIND MT TITL 'ULS lign 6.10a_100(Min:Mt)'  LCST #LCst+18   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MAXD N  TITL 'ULS lign 6.10a_100(Max:N)'   LCST #LCst+19   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MIND N  TITL 'ULS lign 6.10a_100(Min:N)'   LCST #LCst+20   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MAXD VY TITL 'ULS lign 6.10a_100(Max:VY)'  LCST #LCst+21   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MIND VY TITL 'ULS lign 6.10a_100(Min:VY)'  LCST #LCst+22   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MAXD MZ TITL 'ULS lign 6.10a_100(Max:MZ)'  LCST #LCst+23   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA
COMB MIND MZ TITL 'ULS lign 6.10a_100(Min:MZ)'  LCST #LCst+24   LC1 G  LC2 C_1 LC3 C_2 LC4 P LC5  Y_H 1.0 $ Y_ in MA


COMB GMAX    TITL 'ULS lign 6.10a(MAX)' LCST 8000+3
COMB GMIN    TITL 'ULS lign 6.10a(MIN)' LCST 8000+4
DESI STAT ULTI SMOD yes $TANA 0.5 TANB 0.5 $ ULS design AMAX FIXL
TVAR NAME "TANMIN" VAL 0.5 SCOP DESI $Dette bør uansett gjøres siden N400 setter begrensening 1,0<=cot(theta)<=2,0 T
$TVAR NAME "TANMAX" VAL 0.5 SCOP DESI $ Default 1,0 (cot(theta)=1). Reduksjon her gir mer lengdearmering og mindre b


END
! SOFiSTiK Structural Desktop, Task:-64 [ULS set B eq. 6.10a]
! SOFiSTiK Structural Desktop, Task:+65 [ULS set B eq. 6.10b]
#include $(project)_csmlf.dat
#define cs_design=999

+PROG AQB urs:desi02 $ ULS Ultimate Limit State Design Beams
HEAD ULS Ultimate Limit State Design Beams

PAGE UNII 0  ! standard input units
$ Result-control: necessary reinforcement in following plot
ECHO FULL yes
Echo comb yes
$ Controlling the calculation
  CTRL SVRF   1.0        $ take into account reinforcement for C+S
  CTRL AXIA   -2         $ Biaxial bending, uniaxial extreme fibre stresses in y-z system of section

LC      TYPE 'Y_D' CST    999 REF GROS  gamu 1 gamf 0


REIN LCR    3 RMOD  sing $  p7 10

BEAM grp 100,200,300  cs auto $
```

```
#define GAMU_G_1=GAMU 1.35*0.89 GAMF 1.00   $  'dead load'       take sum of all effects of G_1 with 1.35 or 1.00
#define GAMU_G_2=GAMU 1.35*0.89 GAMF 1.00   $  'add.dead'        take sum of all effects of G_2 with 1.35 or 1.00
#define GAMU_P   =GAMU 1.1   GAMF 0.9
#define GAMU_C_1=GAMU 1.00 GAMF 1.00
#define GAMU_C_2=GAMU 1.00 GAMF 0.00

#include stage_design
let#LCst 8300

$Sjekk ved bruåpning
$-----------------------------------------------------------------------------------------------------------------
COMB MAXD MY TITL 'ULS lign 6.10b(Max:My)'  LCST #LCst+1   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MIND MY TITL 'ULS lign 6.10b(Min:My)'  LCST #LCst+2   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MAXD VZ TITL 'ULS lign 6.10b(Max:Vz)'  LCST #LCst+3   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MIND VZ TITL 'ULS lign 6.10b(Min:Vz)'  LCST #LCst+4   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MAXD MT TITL 'ULS lign 6.10b(Max:Mt)'  LCST #LCst+5   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MIND MT TITL 'ULS lign 6.10b(Min:Mt)'  LCST #LCst+6   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MAXD N  TITL 'ULS lign 6.10b(Max:N)'   LCST #LCst+7   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MIND N  TITL 'ULS lign 6.10b(Min:N)'   LCST #LCst+8   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MAXD VY TITL 'ULS lign 6.10b(Max:VY)'  LCST #LCst+9   LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MIND VY TITL 'ULS lign 6.10b(Min:VY)'  LCST #LCst+10  LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MAXD MZ TITL 'ULS lign 6.10b(Max:MZ)'  LCST #LCst+11  LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe
COMB MIND MZ TITL 'ULS lign 6.10b(Min:MZ)'  LCST #LCst+12  LC1 G  LC2 C_1  LC3 P LC4  Y_D 1.0 $ Y_ in MAXIMA presupe

$Sjekk etter 100 år
$-----------------------------------------------------------------------------------------------------------------
COMB MAXD MY TITL 'ULS lign 6.10b_100(Max:My)'  LCST #LCst+13   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MIND MY TITL 'ULS lign 6.10b_100(Min:My)'  LCST #LCst+14   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MAXD VZ TITL 'ULS lign 6.10b_100(Max:Vz)'  LCST #LCst+15   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MIND VZ TITL 'ULS lign 6.10b_100(Min:Vz)'  LCST #LCst+16   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MAXD MT TITL 'ULS lign 6.10b_100(Max:Mt)'  LCST #LCst+17   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MIND MT TITL 'ULS lign 6.10b_100(Min:Mt)'  LCST #LCst+18   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MAXD N  TITL 'ULS lign 6.10b_100(Max:N)'   LCST #LCst+19   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MIND N  TITL 'ULS lign 6.10b_100(Min:N)'   LCST #LCst+20   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MAXD VY TITL 'ULS lign 6.10b_100(Max:VY)'  LCST #LCst+21   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MIND VY TITL 'ULS lign 6.10b_100(Min:VY)'  LCST #LCst+22   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MAXD MZ TITL 'ULS lign 6.10b_100(Max:MZ)'  LCST #LCst+23   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M
COMB MIND MZ TITL 'ULS lign 6.10b_100(Min:MZ)'  LCST #LCst+24   LC1 G  LC2 C_1  LC3  C_2 LC4 P LC5 Y_D 1.0 $ Y_ in M


COMB GMAX    TITL 'ULS lign 6.10b(MAX)' LCST 8000+5
COMB GMIN    TITL 'ULS lign 6.10b(MIN)' LCST 8000+6
DESI STAT ULTI SMOD yes $TANA 0.5 TANB 0.5 $ ULS design AMAX FIXL
TVAR NAME "TANMIN" VAL 0.5 SCOP DESI $Dette bør uansett gjøres siden N400 setter begrensening 1,0<=cot(theta)<=2,0 T
$TVAR NAME "TANMAX" VAL 0.5 SCOP DESI $ Default 1,0 (cot(theta)=1). Reduksjon her gir mer lengdearmering og mindre b


END
! SOFiSTiK Structural Desktop, Task:-65 [ULS set B eq. 6.10b]
! SOFiSTiK Structural Desktop, Task:+66 [SLS Crack width - Frequently ]
#include $(project)_csmlf.dat
#define cs_design=999

PROG AQB urs:115.1 $ SLS Design Beams
HEAD SLS Design Beams
PAGE UNII 0  ! standard input units
ECHO FULL yes
ECHO Crac extr
$ Controlling the calculation
  CTRL SVRF    1.0        $ take into account reinforcement for C+S
  CTRL AXIA    -2         $ Biaxial bending, uniaxial extreme fibre stresses in y-z system of section
$  CTRL VM VAL 2 VAL2 2
$  CTRL DESV 5             $ flange shear design setting
$  ctrl imax 5000
```

```
$  ctrl etol 0.001
!*!Label Reinforcement 6.10b

LC      TYPE 'Y_3' CST  999 REF GROS gamu 1 gamf 0


REIN LCR   5 RMOD sing

BEAM grp 100,200,300  cs auto $
$loop#1 elementer
$BEAM #elementer(#1)  cs auto $
$endloop
#include stage_design
let#LCst 9200


$Sjekk ved bruåpning
$-----------------------------------------------------------------------------------------------------------
COMB MAXF MY TITL 'Freq(Max:My)Crack 0.26'  LCST #LCst+1   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MINF MY TITL 'Freq(Min:My)Crack 0.26'  LCST #LCst+2   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MAXF VZ TITL 'Freq(Max:Vz)Crack 0.26'  LCST #LCst+3   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MINF VZ TITL 'Freq(Min:Vz)Crack 0.26'  LCST #LCst+4   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MAXF MT TITL 'Freq(Max:Mt)Crack 0.26'  LCST #LCst+5   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MINF MT TITL 'Freq(Min:Mt)Crack 0.26'  LCST #LCst+6   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MAXF N  TITL 'Freq(Max:N)Crack  0.26'   LCST #LCst+7   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MINF N  TITL 'Freq(Min:N)Crack  0.26'   LCST #LCst+8   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MAXF VY TITL 'Freq(Max:VY)Crack 0.26'  LCST #LCst+9   LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MINF VY TITL 'Freq(Min:VY)Crack 0.26'  LCST #LCst+10  LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MAXF MZ TITL 'Freq(Max:MZ)Crack 0.26'  LCST #LCst+11  LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0
COMB MINF MZ TITL 'Freq(Min:MZ)Crack 0.26'  LCST #LCst+12  LC1 G LC2 C_1 LC3 P LC4 Y_3 1.0


$Sjekk etter 100 år
$-----------------------------------------------------------------------------------------------------------
COMB MAXF MY TITL 'Freq(Max:My)Crack 0.26_100'  LCST #LCst+13  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MINF MY TITL 'Freq(Min:My)Crack 0.26_100'  LCST #LCst+14  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MAXF VZ TITL 'Freq(Max:Vz)Crack 0.26_100'  LCST #LCst+15  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MINF VZ TITL 'Freq(Min:Vz)Crack 0.26_100'  LCST #LCst+16  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MAXF MT TITL 'Freq(Max:Mt)Crack 0.26_100'  LCST #LCst+17  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MINF MT TITL 'Freq(Min:Mt)Crack 0.26_100'  LCST #LCst+18  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MAXF N  TITL 'Freq(Max:N)Crack  0.26_100'   LCST #LCst+19  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MINF N  TITL 'Freq(Min:N)Crack  0.26_100'   LCST #LCst+20  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MAXF VY TITL 'Freq(Max:VY)Crack 0.26_100'  LCST #LCst+21  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MINF VY TITL 'Freq(Min:VY)Crack 0.26_100'  LCST #LCst+22  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MAXF MZ TITL 'Freq(Max:MZ)Crack 0.26_100'  LCST #LCst+23  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0
COMB MINF MZ TITL 'Freq(Min:MZ)Crack 0.26_100'  LCST #LCst+24  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_3 1.0


COMB GMAX    TITL 'Freq(MAX)Crack 0.26'     LCST 9000+3
COMB GMIN    TITL 'Freq(MIN)Crack 0.26'     LCST 9000+4
NSTR CRAC yes cw 0.26 $ crack width design

END
! SOFiSTiK Structural Desktop, Task:-66 [SLS Crack width - Frequently ]
! SOFiSTiK Structural Desktop, Task:+82 [SLS Decompression ]
#include $(project)_csmlf.dat
#define cs_design=999

PROG AQB urs:115.1 $ SLS Design Beams
HEAD SLS Design Beams

PAGE UNII 0  ! standard input units
ECHO FULL yes
ECHO Crac extr
```

```
$ Controlling the calculation
  CTRL SVRF   1.0       $ take into account reinforcement for C+S
  CTRL AXIA   -2        $ Biaxial bending, uniaxial extreme fibre stresses in y-z system of section
$  CTRL VM VAL 2 VAL2 2
$  CTRL DESV 5            $ flange shear design setting
$  ctrl imax 5000
$  ctrl etol 0.001
!*!Label Reinforcement 6.10b

LC      TYPE 'Y_4' CST  999 REF GROS gamu 1 gamf 0


REIN LCR   6 RMOD sing

BEAM grp 100,200,300  cs auto $
$loop#1 elementer
$BEAM #elementer(#1)  cs auto $
$endloop
#include stage_design
let#LCst 9300


$Sjekk ved bruåpning
$------------------------------------------------------------------------------------------------------------------
COMB MAXP MY TITL 'Deco(Max:My)'  LCST #LCst+1   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MINP MY TITL 'Deco(Min:My)'  LCST #LCst+2   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MAXP VZ TITL 'Deco(Max:Vz)'  LCST #LCst+3   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MINP VZ TITL 'Deco(Min:Vz)'  LCST #LCst+4   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MAXP MT TITL 'Deco(Max:Mt)'  LCST #LCst+5   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MINP MT TITL 'Deco(Min:Mt)'  LCST #LCst+6   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MAXP N  TITL 'Deco(Max:N)C'   LCST #LCst+7   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MINP N  TITL 'Deco(Min:N)C'   LCST #LCst+8   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MAXP VY TITL 'Deco(Max:VY)'  LCST #LCst+9   LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MINP VY TITL 'Deco(Min:VY)'  LCST #LCst+10  LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MAXP MZ TITL 'Deco(Max:MZ)'  LCST #LCst+11  LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0
COMB MINP MZ TITL 'Deco(Min:MZ)'  LCST #LCst+12  LC1 G LC2 C_1 LC3 P LC4 Y_4 1.0


$Sjekk etter 100 år
$------------------------------------------------------------------------------------------------------------------
COMB MAXP MY TITL 'Deco(Max:My)_100'  LCST #LCst+13  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MINP MY TITL 'Deco(Min:My)_100'  LCST #LCst+14  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MAXP VZ TITL 'Deco(Max:Vz)_100'  LCST #LCst+15  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MINP VZ TITL 'Deco(Min:Vz)_100'  LCST #LCst+16  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MAXP MT TITL 'Deco(Max:Mt)_100'  LCST #LCst+17  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MINP MT TITL 'Deco(Min:Mt)_100'  LCST #LCst+18  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MAXP N  TITL 'Deco(Max:N)_100'   LCST #LCst+19  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MINP N  TITL 'Deco(Min:N)_100'   LCST #LCst+20  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MAXP VY TITL 'Deco(Max:VY)_100'  LCST #LCst+21  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MINP VY TITL 'Deco(Min:VY)_100'  LCST #LCst+22  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MAXP MZ TITL 'Deco(Max:MZ)_100'  LCST #LCst+23  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0
COMB MINP MZ TITL 'Deco(Min:MZ)_100'  LCST #LCst+24  LC1 G LC2 C_1 LC3 C_2 LC4 P LC5 Y_4 1.0


COMB GMAX    TITL 'Deco(MAX)'     LCST 9000+5
COMB GMIN    TITL 'Deco(MIN)'     LCST 9000+6
NSTR serv crac deco  10 $ = the whole section of pre-stressing steel and where relevant the duct for post-tensioned 

END
! SOFiSTiK Structural Desktop, Task:-82 [SLS Decompression ]
! SOFiSTiK Structural Desktop, Task:+67 [Sum of required R/F]
+prog aqb urs:86.1
head Summert armeringsbehov
```

```
rein  lcr  2,3,5 rmod accu

rein lcr 10 rmod sing


end
! SOFiSTiK Structural Desktop, Task:-67 [Sum of required R/F]
! SOFiSTiK Structural Desktop, Task:+72 [Results ]
+PROG RESULTS urs:71.1
HEAD
$ Begin Page 1
SIZE TYPE "-URS" SPLI "2x1"
$ Begin Grafic/Table/Diagram 1
PICT SC DEFA W DEFA H DEFA SPLT   NO
GRP
JOIN
DBO
let#midl #elementer(2)
let#midl2 #elementer(3)
FILT
FLT  ID 1 NAME "beam_rfc.nr" RULE "200001"
FLT  ID 1 NAME "beam_rfc.x" RULE "0"
FLT  ID 2 NAME "beam_rfc.nr" RULE "300001"
FLT  ID 2 NAME "beam_rfc.x" RULE "0"
FLT  ID 3 NAME "beam_rfc.nr" RULE "#midl"
FLT  ID 3 NAME "beam_rfc.__xi" RULE "1"
FLT  ID 4 NAME "beam_rfc.nr" RULE "#midl2"
FLT  ID 4 NAME "beam_rfc.__xi" RULE "0"
LC   DESI 5
$ Begin Result 1
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS1 STYP BEAM REPR DLST
$ Begin Result 2
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS2 STYP BEAM REPR DLST
$ Begin Result 3
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS3 STYP BEAM REPR DLST
$ Begin Result 4
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE ASU1 STYP BEAM REPR DLST
END
! SOFiSTiK Structural Desktop, Task:-72 [Results ]
! SOFiSTiK Structural Desktop, Task:+77 [BMD Diagrams]
PROG WING urs:77.1 $ Interactive Graphics
HEAD $Beam Elements , Bending moment My  LC:      1301
PAGE UNII 0 $ default unit set
CTRL OPT GSTR VAL DEFA
CTRL OPT REPR VAL YES
$ DB   NUMB 1 TITL "l60.cdb"
CTRL OPT AXIS VAL DEFA
$ graphics    1 | picture    1 | layer    1 : Beam Elements , Bending moment My  LC:      1301
let#laster 1301,1302,1401,1402,2201,2202,2301,2302
loop#1 laster
PAGE LANO 1
SIZE TYPE URS SC 0 SPLI '1*1'
SIZ2 SPLI DEFA
HEAD Interactive Graphics
AND  POSI 1 POSL 0 POSR 100 POSD 0 POSU 100
SCHH H6 0.560000 ND 2
SCH2 DIRE DEFA
```

```
GRID TYPE NO DIRE DEFA OFFP 0 OFFC 0 OFF3 0 TOLA 15 TOLZ -5
COLO C13 1001 1001
LC   NO #laster(#1) DESI 1
BOX
GRP  NUMB NODE OPTI OFFL
GRP  NUMB ENOD OPTI OFFL
GRP  NUMB EDGE OPTI OFFL
GRP  LC NO
GRP  NUMB 10 YES SPRI,KINE
GRP  NUMB 20 OFF BEAM
GRP  NUMB 20 YES KINE,GLN
GRP  NUMB 30 OFF BEAM
GRP  NUMB 30 YES KINE,GLN
GRP  NUMB 40 YES SPRI,KINE
GRP  NUMB 100 YES BEAM,GLN
GRP  NUMB 200 YES BEAM,GLN
GRP  NUMB 300 YES BEAM,GLN
VIEW TYPE DIRE X 0 Y -1 Z 0 AXIS POSZ ROTA 0
DEFO TYPE NO EXPO 0 SMOV NO
SELE NUMB 0
BEAM TYPE MY UNIT DEFA SCHH YES STYP BEAM FILL NO REPR DLIN
$ graphics    1 | picture    1 | layer    2 : Beam Elements , Number of element
AND
SCHH H6 0.500000 ND NMAX
STRU NUME ENO NUMN BEAM FILL NO REPR DTXT UNIT DEFA SCHH YES
endloop
END
! SOFiSTiK Structural Desktop, Task:-77 [BMD Diagrams]
! SOFiSTiK Structural Desktop, Task:+84 [Strain diagrams]
$ Automatically generated by WING 2022-8.0 21/03/2023 09:39
$ Attention: Changes will be overwritten if the task is opened again!
+PROG WING urs:84.1 $ Interactive Graphics
HEAD Interactive Graphics
PAGE UNII 0
PAGE UNII 0 $ default unit set
CTRL OPT GSTR VAL DEFA
CTRL OPT REPR VAL YES
$ DB   NUMB 1 TITL "l50.cdb"
CTRL OPT AXIS VAL DEFA
$ graphics    1 | picture    1 | layer    1 : Beam Elements , Maximum decompression strain  DC:   9005
PAGE LANO 1
SIZE TYPE URS SC 0
SIZ2
AND  POSI 1 POSL 0 POSR 100 POSD 0 POSU 100
SCHH H6 0.350000
SCH2 DIRE DEFA
LC   NO 1 DESI 9005
BOX
VIEW TYPE DIRE X 0 Y -1 Z 0 AXIS POSZ ROTA 0
DEFO TYPE NO EXPO 0 SMOV NO
SELE NUMB 0
BEAM TYPE DCSX UNIT DEFA SCHH YES STYP BEAM FILL NO REPR DLIN
$ graphics    2 | picture    1 | layer    1 : Beam Elements , Utilisation level crack Longitudinal reinforcement for
SIZ2
AND  POSI 1 POSL 0 POSR 100 POSD 0 POSU 100
SCH2 DIRE DEFA
BEAM TYPE CCW UNIT DEFA SCHH YES STYP BEAM FILL NO REPR DLIN
END
! SOFiSTiK Structural Desktop, Task:-84 [Strain diagrams]
! SOFiSTiK Structural Desktop, Task:+83 [Decompression Check]
+PROG RESULTS urs:83.1
HEAD
SIZE TYPE "-URS" SPLI "2x1"
PICT SC DEFA W DEFA H DEFA SPLT   NO
```

```
VIEW TYPE DIRE X 0.358066 Y -0.930968 Z -0.071326 AXIS POSZ
GRP  NUMB - OPTI  YES
DBO
FILT
FLT
$ Begin Layer 1
LC   NO 9314
LC   ENO 100005 X 0
TXTP SHOW SIGN OVLP AMAX EXTR  YES
CROS TYPE RFSS ETYP BEAM RTYP NONL REPR DLIN FILL   NO SCHH  YES
$ Begin Grafic/Table/Diagram 2
PICT SC DEFA W DEFA H DEFA SPLT   NO
VIEW TYPE DEFA
GRP
JOIN
DBO
FILT
FLT
LC   NO 9005
$ Begin Result 1
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  CCW STYP BEAM REPR DLST
END
! SOFiSTiK Structural Desktop, Task:-83 [Decompression Check]
! SOFiSTiK Structural Desktop, Task:+71 [Reinforcement area ]
+PROG RESULTS urs:71.1
HEAD
$ Begin Page 1
SIZE TYPE "-URS" SPLI "2x1"
$ Begin Grafic/Table/Diagram 1
PICT SC DEFA W DEFA H DEFA SPLT   NO
GRP  NUMB - OPTI  YES
JOIN
DBO
FILT
FLT  ID 1 NAME "beam_rfc.nr" RULE "200001"
FLT  ID 1 NAME "beam_rfc.x" RULE "0"
FLT  ID 2 NAME "beam_rfc.nr" RULE "200019"
FLT  ID 2 NAME "beam_rfc.x" RULE "0"
FLT  ID 3 NAME "beam_rfc.nr" RULE "200036"
FLT  ID 3 NAME "beam_rfc.x" RULE "1"
LC   DESI 10
$ Begin Result 1
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS1 STYP BEAM REPR DLST
$ Begin Result 2
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS2 STYP BEAM REPR DLST
$ Begin Result 3
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS3 STYP BEAM REPR DLST
$ Begin Result 4
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE ASU1 STYP BEAM REPR DLST
END
! SOFiSTiK Structural Desktop, Task:-71 [Reinforcement area ]
! SOFiSTiK Structural Desktop, Task:+75 [Full list of R/F area]
+PROG RESULTS urs:75.1
HEAD
$ Begin Page 1
SIZE TYPE "-URS" SPLI "2x1"
```

```
$ Begin Grafic/Table/Diagram 1
PICT SC DEFA W DEFA H DEFA SPLT   NO
GRP  NUMB - OPTI  YES
JOIN
DBO
FILT
FLT
LC   DESI 10
$ Begin Result 1
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS1 STYP BEAM REPR DLST
$ Begin Result 2
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS2 STYP BEAM REPR DLST
$ Begin Result 3
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE  AS3 STYP BEAM REPR DLST
$ Begin Result 4
AND
TXTP SHOW SIGN OVLP AMAX EXTR  YES
BEAM TYPE ASU1 STYP BEAM REPR DLST
END


! Crack Check
+PROG SOFILOAD urs:82.1 $ Combine Loads
HEAD Combine Loads
PAGE UNII 0
LC 1001  TYPE (P) TITL '1.35G_2'
    $ 5020: G_2  Superegenvekt
COPY 5020 1
$COPY 5030 1
$COPY 5020 1
END
prog ase urs:83.2
head

lc 1001
end

+PROG ASE urs:22.1 $ Berechnung der kombinierten Lastfälle
HEAD Opprisset tilfelle egenvekt
PAGE UNII 0
CTRL OPT WARP VAL 0
syst prob nonl iter 800
REIN RMOD ACCU LCR 10
rein rmod sing lcr 20
NSTR KMOD S1 KSV SLD FMAX 0.80
LC 1002 facd  1
lcc 1001
END



-PROG ASE urs:81.2 $ Berechnung der kombinierten Lastfälle
HEAD Opprisset tilfelle  trafikk + permanente laster(egenvekt)
PAGE UNII 0
CTRL OPT WARP VAL 0
syst prob nonl iter 800
REIN RMOD ACCU LCR 10
rein rmod sing lcr 21
NSTR KMOD S1 KSV SLD FMAX 0.80
```

```
LC 1003 facd  1
lcc 1001
lcc 10012 $boggiloading LM1        10012 is max loadcase deflection  (should be changed for each model)
lcc 101
END


+PROG ASE urs:81.3 $ Berechnung der kombinierten Lastfälle
HEAD Opprisset tilfelle: kun trafikk last, bruker opprisset stivhetsmatrise beregnet i LC 1002
PAGE UNII 0
CTRL OPT WARP VAL 0
$syst prob nonl iter 800
$REIN RMOD ACCU LCR 10
$rein rmod sing lcr 21
$NSTR KMOD S1 KSV SLD FMAX 0.80
syst plc 1002
LC 1004

lcc 10012 $boggiloading LM1        10012 is max loadcase deflection  (should be changed for each model)
lcc 101
END

-PROG ASE urs:81.4 $ Berechnung der kombinierten Lastfälle
HEAD Upprisset tilfelle  trafikk
PAGE UNII 0
CTRL OPT WARP VAL 0
$syst prob nonl iter 800
$REIN RMOD ACCU LCR 10
$rein rmod sing lcr 21

LC 1005

lcc 10012 $boggiloading LM1
lcc 101
END
```