CHALMERS
UNIVERSITY OF TECHNOLOGY

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Conceptual and structural design

Master's thesis in the Master's Programme Structural Engineering and Building Technology

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Photo of physical model from master thesis "A Pilgrims walk"

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ABSTRACT

A pedestrian bridge can give new ways of experiencing a site and create added value. When walking over a bridge the experience is different than when travelling by car or train. One moves in a slower pace and there is no vehicle in between. The experience is a combination of the feel of the materials, the environment with wind and sun, the changing views and the response of the bridge. That make pedestrian bridges extra interesting both from a structural point of view but also from an architectural.

This thesis aims to investigate how the design process of double curved timber suspension bridges could be performed,from conceptual to preliminary design according to Eurocode. Describing the how the structure work and how suitable it is for timber construction. As one example of a double curved pedestrian bridge, the design suggested in the master thesis in architecture "A pilgrims walk", performed by the author in spring 2012 at Chalmers, has been used.

To give a suggestion of how the design process of a double curved suspension timber bridge, one can begin by giving a background of the development of timber, suspension and curved bridges. This thesis start by explaining the development and statical system of the three bridge types and by doing so putting the example bridge in context.

The method given is a combination of sketches, physical models and computer models, both for concept development and structural and dynamical analysis. Using the 3D-modelling software Rhinoceros3D and the graphical programming plug-in Grasshopper to define the geometry, is a powerful way of creating a framework for trying out different configurations. The curvature of the deck, thickness and width, number of bridge segments or the location of the masts are among many different parameters that can easily be changed. The geometry of a curved suspension bridge is very important for how effective it is. A custom written component in Grasshopper has been created using the principle of dynamic relaxation to define the geometry of the suspension cable. The geometry of the whole bridge has then been exported to SAP2000 for preliminary structural and dynamical analysis. The results has been used for preliminary design according to Eurocode. The preliminary structural design show that it is possible to design a curved suspension bridge in timber. A timber suspension bridge is a light weight structure often sensitive to vibrations. A comparison of dynamical assessment according to Eurocode and SETRA show that the same bridge can have completely different outcomes.

This thesis finds out that to design a complicated structure it is very helpful to have a design process that enables an iterative process from the conceptual to structural design. Small adjustments of a complicated design can have large effects in the end. Using software that communicates well with each other enables one to use each for what they do best.

Keywords: Curved suspension bridges, timber bridges,

Design av svängda hängbroar i trä
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SAMMANFATTNING

En gångbro kan ge nya sätt att uppleva en plats och på det sättet skapa mervärde. När man promenerar över en bro så är det en annan upplevelse än när man kör bil eller sitter i ett tåg. Tempot är långsammare och det är inget fordon mellan som skapar distans. Upplevelsen är en kombination av känslan av materialen, omgivningen med vind och sol, vyerna som förändras och brons svängningar. Det gör gångbroar intressanta ur både ett strukturellt och ett arkitektoniskt perspektiv.

Målet med det här examensarbetet är att undersöka hur designprocessen av dubbensvängda hängroar i trä kan se ut, från konceptuell till preliminär dimensionering enligt Eurocode. Arbetet ska beskriva hur konstruktionen fungerar och hur lämpligt det är att bygga i trä. Som ett exempel på en dubbelsvängd hängbro i trä har designen i examensarbetet "A pilgrims walk" användts, genomfört av författaren vårterminen 2012 vid Chalmers Arkitektur.

För att ge ett förslag till hur designprocessen för en dubbelsvängd hängbro i trä kan se ut, så kan man börja med en bakgrund till hur träbroar, hängbroar och svängda broar utvecklats. Examensarbetet börjar med att beskriva utvecklingen och de statiska systemen hos de tre brotyperna för att på det sättet sätta exempelbron in en kontext.

Den föreslagna metoden är en kombination av skisser, fysiska modeller och datamodeller, både för koceptutveckling och för strukturell och dynamisk analys. 3D-modelleringsprogrammet Rhinoceros 3D tillsammans med tillägget Grasshopper har gjort det möjligt att definiera brogeometrien paramatriskt och gjort det möjligt att testa många olika konfigurationer. Geometrien av en dubbesvängd hängbro är väldgigt avgörande för hur effektiv den är. En skräddarsydd komponent, skriven i C# i Grasshopper, har använts för att formsöka geometrien av huvudkablen. Efter att geometrin blivit definerad exporteras den till SAP2000 för vidare analys. Resultatet används för en preliminär design enligt Eurocode. Bland resultaten av examensarbetet är att för att designa en komplex konstruktion så är det fördelaktigt att ha en designprocess som möjliggör en itterativ process från konceptuell design till dimensionering. Små förändringar i en komplicerad struktur kan ha stora effekter. Att använda program som kommunicerar väl med varandra gör det möjligt att använda dem till det de gör bäst.

Nyckelord: Svängda hängbroar, träbroar, Formsökning,

CONTENTS

Preface

This thesis were initiated by Professor Karl Gunnar Ohlsson at Chalmers University who proposed two theses concerning the design of a pedestrian timber bridge at Borgeby castle outside of Lund in southern Sweden. The two theses, one in architecture and one in structural engineering, were supposed to run parallel and contribute to each other during one year. As a potential client and contributing with information about Borgeby Castle was Eva Wängelin who a that time was working for for Lomma municipality.

The architectural thesis was conducted during spring 2012 at the "Matter and Space"-studio at Chalmers with Professor Morten Lund as examiner.

This thesis comprises 30 ECTS and began in the autumn semester of 2012 and after a break of three years was finished spring semester 2016. The thesis has been conducted with supervision from Professor Roberto Crocetti, LTH and Professor Karl Gunnar Ohlsson from Chalmers. Professor Robert Klieger from Chalmers is examiner.

I would like to give thanks to my two supervisors for discussion and guidance during the work. I would also like to give thanks to, at that time of 2012 fellow student, Vedad Alic for discussions and support during the thesis and through out our education. My family and my girlfriend Ingrid Endresen Haukeli deserves many thanks for support through out my education.

1 Introduction

A pedestrian bridge is a structure that is experienced through a close connection to the person walking over it. The experience includes a sense of the materials, the structure and the response of the bridge together with the environment surrounding the bridge. Pedestrian bridges does not have the same requirements as road and rail bridges when it comes to loads, inclinations and curvature of the deck since the speed is lower. However, pedestrian bridges are located in a context connecting foot paths where a straight and flat path in not always the best alternative for experiencing the surroundings. A pedestrian bridge is experienced in direct contact with the people walking and touching them. This put high demands on the architecture and design of the bridge. The design of bridges are the design of the structural system in large extent and in order to have a successful process it is important to see keep both engineering and design in mind.

Timber bridges were the first bridges ever constructed, by simple logs crossing a stream of water. Later in history timber was a popular material in bridge construction because it was a resource there were plenty of, it was easy to work with and enabled a fast constructions time. Timber had one problem though, it was and still is sensitive to water and moisture. How ever if designed correctly a timber structure can stand for hundreds of years. Today timber bridges are getting more popular again because of the environmental properties of timber, fast and economic construction and also because new products are available, like stress laminated timber decks, and new ways of treating timber.

1.1 Problem description

Pedestrian bridges are experienced at slow speed and with a close connection between the people walking over them and the environment around. That gives a possibility to create a new way of experience the site and the best path is in most cases not a straight and flat bridge. What does that mean for the design of the bridge?

There are examples of curved pedestrian bridges, many of them in Germany designed by firms like Schlaich Bergermann and Partner. Most of them are in steel and concrete, but would it be possible to build one in timber? Timber is an orthotropic material and what does that mean for a complicated three-dimensional structure?

Structural design according to Eurocode 5 is the usual way of designing timber structures. How can that be implemented in the design of a double curved suspension bridge? Which problems can occur and how does that effect the design?

1.2 Aim

The aim with this thesis is to investigate possible design procedures of curved timber suspension bridges. Further aims are to describe how curved suspension bridges behave and to illustrate problems with the design and giving methods of solving them. Going from conceptual to load bearing design of critical members in the bridge given in the master thesis "A pilgrims walk", the thesis aims to show different parts of the design process.

To be able to complete the task in this thesis several computer softwares have been used. The conceptual design and form finding was performed in Rhino3D/Grasshopper. After the conceptual design SAP2000 and Eurocode were used to continue the design and investigate the dynamical properties and the load bearing capacity. The aim is to investigate the potential of a more iterative design process where several software can be used in a efficient work flow.

1.3 Limitations

Compared to the bridge in the thesis "A pilgrims walk" this thesis is considering a curved suspension bridge without any inclination of the bridge deck. The thesis show a overall design procedure without going into deep on each stage from conceptual to load bearing design. Still going deep enough for giving a picture of the feasibility of the project.

1.4 Outline

This master thesis consists of one part focusing on the background and conceptual design of curved pedestrian bridges and one part focusing on the analysis and structural design of the bridge given in the master thesis "A pilgrims walk". In the end of the thesis reflections and discussion of the results from the previous chapters are presented. Finally a proposal for future work is given. The thesis is divided in following Chapters.

Background

In this chapter the bridge used as an example of a double curved suspension bridge is presented and how the process towards the final design were. The chapter also include the development of bridges, details and durability in timber constructions.

Static and dynamics of pedestrian bridges

This chapter describes the statics and dynamics properties of pedestrian bridges. The intention is to in the end describe the behaviour of a curved suspension bridge. In order to do that first the behaviour of a curved beam/bridge and a suspension bridge are presented separately. After that the behaviour of a curved suspension bridge can be described. The dynamic properties of pedestrian bridges are presented with two alternative methods of calculating dynamic response, according to Eurocode and according to SETRA.

Method

The method from conceptual design to a final finite elements model from which the results can be used to load bearing design is presented in this chapter.

Analysis and results

A closer look to the analysis and the results are given in this chapter. Beginning with a one curved suspension bridge going to a two curved bridge the analysis intend to show problems when modelling a curved suspension bridge and what is important to think about. The results from this chapter are used in the next chapter, Load bearing design according to Eurocode.

Load bearing design according to Eurocode

This chapter includes the load bearing design in ULS and SLS according to Eurocode. The load bearing design is limited to the design of the timber deck, cables, timber struts and the mast.

Discussion and future work

The conclusions from the previous chapters are presented alongside new questions and suggestions for future work in this chapter.

2 Background

Many road or rail bridges have to be constructed straight or just slightly curved because of the design loads and the speed of the vehicles and it would not be practical to build them in an other way. Pedestrian bridges are different, they connect to foot paths that are adapted to the landscape. The landscape have a certain character with different heights, directions, views and other qualities that should be taken in considerations when designing a bridge. A bridge could be a possibility to give a new experience to a site(Schlaich (2002)).

The first bridges ever constructed were probably made out of timber. Simple logs crossing a river or between two cliffs. Timber is a natural material with many qualities that we as human beings respond to, it has a genuine natural feel different from steel and concrete. Even though timber was the first material to construct bridges of, the use of timber encountered problems. Durability was a major concern and compared to other materials as stone, steel or concrete, timber had problems with detailing and protection of the load bearing structure. There are however examples of timber bridges that have lasted for a long time. Through the history of bridges, timber have been used when there was a need to build fast and economic bridges. When Ceasar invaded Germany he built a timber bridge crossing the river Rhine and when the railway in Great Britain were constructed the great engineers Brunell and Telford used timber in some of the bridges. Today the use of timber in bridges is increasing, much because of the environmental benefits, economic and fast construction and a growing knowledge of how to protect the load bearing parts from deterioration. The development of bridges has developed from the first log to a broad spectrum of structural concepts and materials. (SETRA (2006a))

In this chapter a summary of the thesis "A pilgrims walk" with presentation of the site, conceptual design and the final concept. Also a background to the design of timber, suspension and curved bridges is given. Finally, in this chapter, details and durability in timber design are presented.

2.1 Master thesis "A Pilgrims walk"

As an example of a double curved suspension bridge, that was designed by the author in the Master Thesis in architecture was used. The work was performed during the spring semester of 2012. Appendix A show a summary of the thesis and the site, Borgeby Castle. This section explains the process of the design of the final concept.

The bridge was designed as one part of a network of pilgrimage routes in Skåne in south of Sweden. The bridge will be located at Borgeby Castle outside Lund and will cross the river Lödde. The area is very beautiful and has a long history spanning from the Viking age up to modern days. In a context like Borgeby it is important to design a bridge that is respectful to its environment and still have an identity of its own.

2.1.1 Conceptual design

The design process during the conceptual design was an iterative process where first the landscape and topography were analysed in order to find out what possible routes exist and their pros and cons. Also important spots and views that should be kept were decided. After the analysis of the site, the process continued with many physical models and sketches to decide which route and concept was the most interesting, see Fig.2.1. Different structural concepts were studied and analysed in terms of how they fit in the context. When the desired route was established, the different concepts were evaluated according to the list of requirements that the bridge should fulfil. From all the concepts, one was chosen that fulfilled the requirements and had the most interesting structural system.

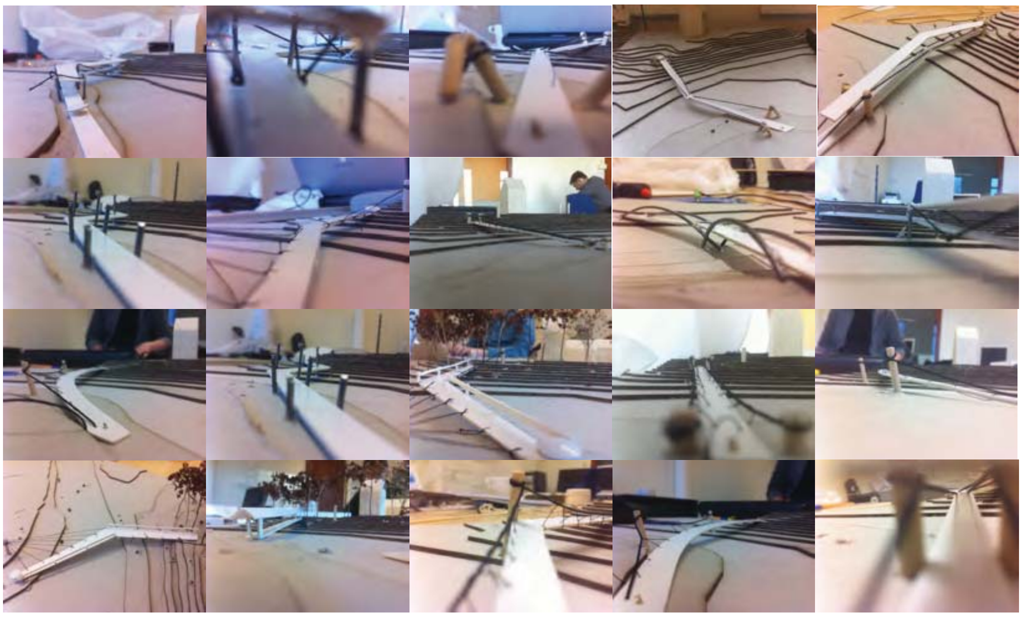


Figure 2.1: Concept development

2.1.2 Final Concept

The final concept was a double curved bridge that starts in the middle of the pathway down to the water and lands parallel to the water on the other side. This route makes it possible to walk under the bridge on the southern side and it is also possible to travel by small boats under the bridge. The height difference and inclination of the bridge is still less than 1:20 making the whole path between the churches accessible for people in wheelchairs. The viewpoint looking over the landscape is kept unchanged and is still possible to experience without having to walk to the bridge. The bridge fulfils the function in the way that it connects the two sides and adds a new way of experiencing the landscape. The proportions are respectful to the landscape with one pylon on the flat north side and that the bridge land parallel to the slope of landscape on the steep side. Since the bridge is low it does not over power the landscape, but it still has a strong character of its own. The incorporation of nature is present when walking over the bridge. The connection to the water and the way you walk in the forest when arriving to the bridge from the south, or in the reed when approaching the bridge the north.

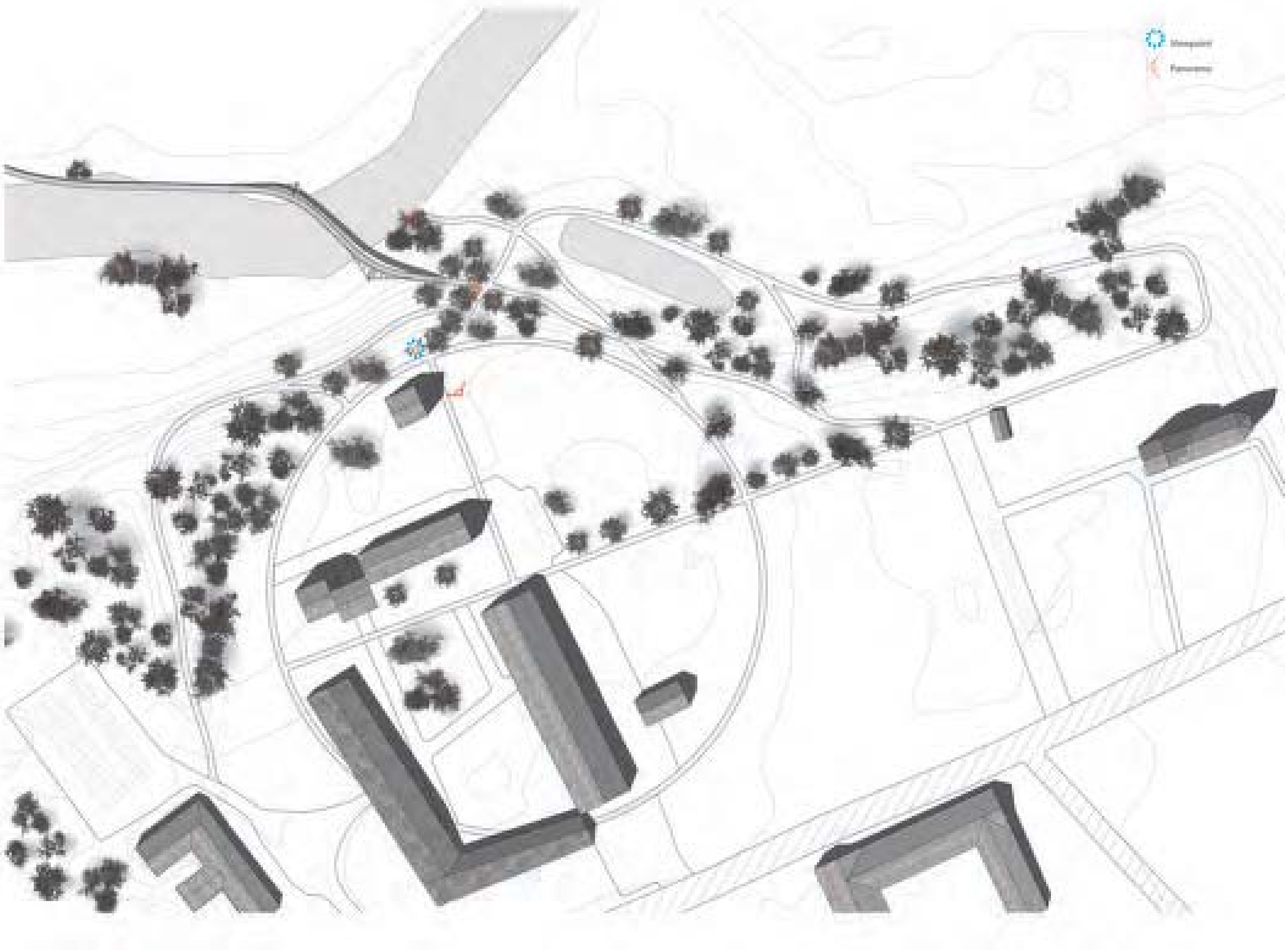


Figure 2.2: Site plan of Borgeby castle

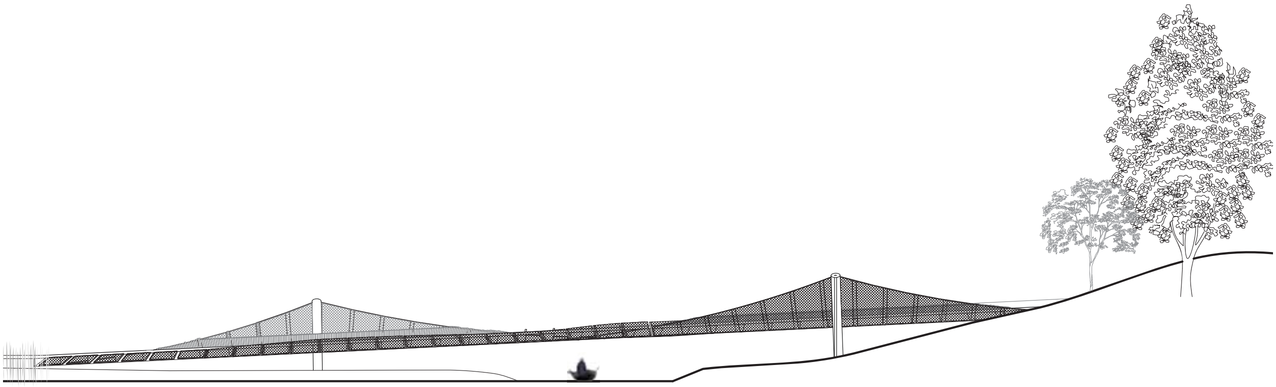


Figure 2.3: Elevation of the bridge

Figure 2.4: View from the bridge looking up towards the castle

2.2 Development of bridges

Bridges are a way of connecting two points and crossing a barrier. A barrier could be for instance a river, a valley or a motorway, something that is an obstacle. The first bridges were probably timber bridges constructed by simply cutting a three and putting it over a stream of water. Stone became later a common bridge material and together with timber have been the historically most common material. The last 150 years steel and concrete has been the dominating materials in bridges, however that is about to change. Today bridges are built in numerous materials and construction methods depending on the site and architectural ambitions. Timber has once again become a popular material because of the environmental benefits, aesthetics and rapid construction process. Also more knowledge about how to detail and new ways of treating timber has made it a durable option. (Pousette (2008))

2.2.1 Timber bridges

A timber bridge is a bridge with the main load bearing structure made out of timber. The development of new ways of producing timber elements like glulam, cross-laminated timber and plywood has opened new possibilities for timber construction.

About 17000 years ago the first bridge in timber were constructed and 2000 years ago there were structures in timber that made it possible to cross larger spans. The development of timber bridge design should be credited shipbuilders as shown in Figure 2.5. (SETRA (2006a))

Figure 2.5: Bridge made out of ships (SETRA (2006a))

When Ceasar was about to invade Germany 55 BC, he should reportedly said that he wanted to find the widest and nicest crossing. There he let construct the first bridge to ever cross the river Rhine. The bridge was constructed of timber beams and built with many supports along its length. The supports were made by timber piers which were piled down by winching a large stone and then droppings it on the pier pressing it down in the riverbed (SETRA (2006a)).

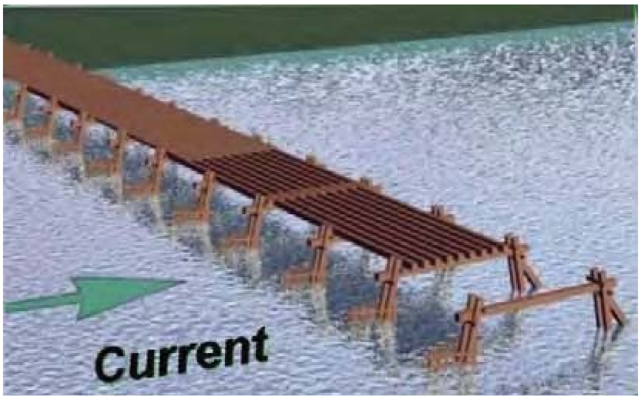


Figure 2.6: Ceasars bridge over river Rhine (SETRA (2006a))

150 years after the Romans crossed the river Rhine they built a bridge crossing Danube. They had developed a new construction method consisting of a number of arches and more complex joints and strut frames. The bridge was in total 1135 meters divided in spans of 38 meters. For 1000 years it was the longest bridge in the world, both total length and span length.

The development of timber bridges was not only confined to Europe, in Asia they were also using complex construction methods like arches, strut-frames and also suspension bridges. The suspension bridges in the island of Java was constructed out of vine.

Timber had been the main material to use in bridge construction because it was the only feasible material. Stone had been introduced as supports in the bridge at Trajan and was now becoming a more common material. (SETRA (2006a))

During the middle ages, timber got a bad reputation for not being durable and expensive to maintain. In Paris there were several accidents with many casualties that lead to the ban of timber in bridge design. In Italy, the engineer and architect Andre Palladio suggested a law saying that if timber were going to be used it had to be covered. That lead to many interesting bridge concepts, but there were still problems with durability and detailing so the use of timber decreased. There were though some good examples of timber-bridge design, many of them in Switzerland. Like the covered bridge in Lucerne built in 1333 and were standing until 1993 when it was destroyed in a fire. In 1994 it was soon reconstructed to its original state, see Figure 2.8.

Figure 2.7: Chapel bridge, Lucerne (Pousette (2008))

The Swiss carpenters Hans Ulrich and Jean Grubenmann should also be mentioned in the history of timber bridge development. The swiss family of carpenters designed and built large bridges like the Schaffhausen bridge built in 1758, see Figure 2.9. The covered bridge had two spans of 60 meters and the constructions was a mix of an arch and a truss system. Through long practical experience in timber construction the Grubermanns managed several bridge projects in Switzerland. (SETRA (2006a))

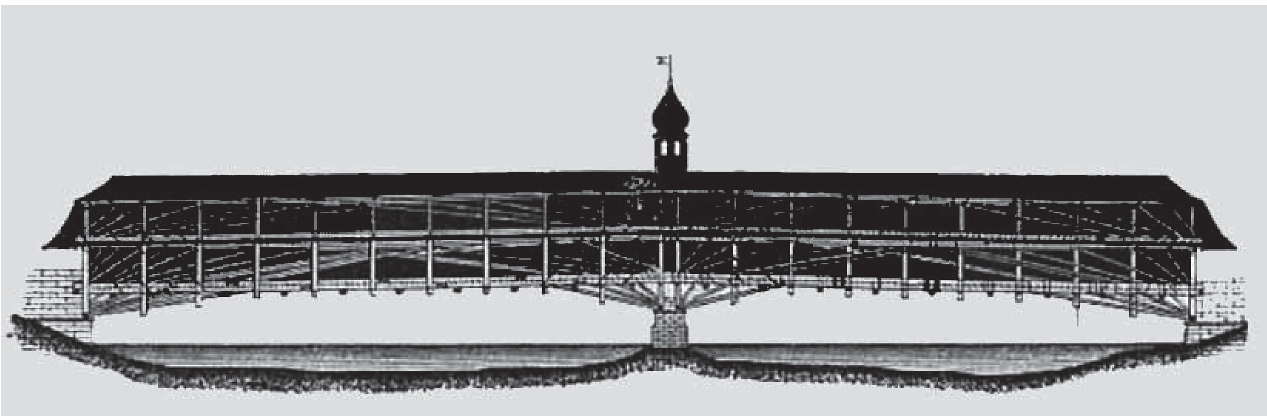


Figure 2.8: Elevation of Schaffhausen Brücke, finished in year 1758 (SETRA (2006a))

During the development of the Great Western Railway in Great Britain, there were many bridges and viaducts built in timber. It was not that the engineers did not know about problems with timber and durability and maintenance, it was more a question of economy and fast erection time. A vast infrastructure system was going to be built and it was a huge investment. Timber was therefore considered as a good alternative. Two of the most famous British engrinners Thomas Tellford and Isambard Kingdom Brunel built impressive structures. One of Brunells examples is the Cornwall viaducts built between 1848 and 1849, see Figure 2.10. (Lewis (2007))

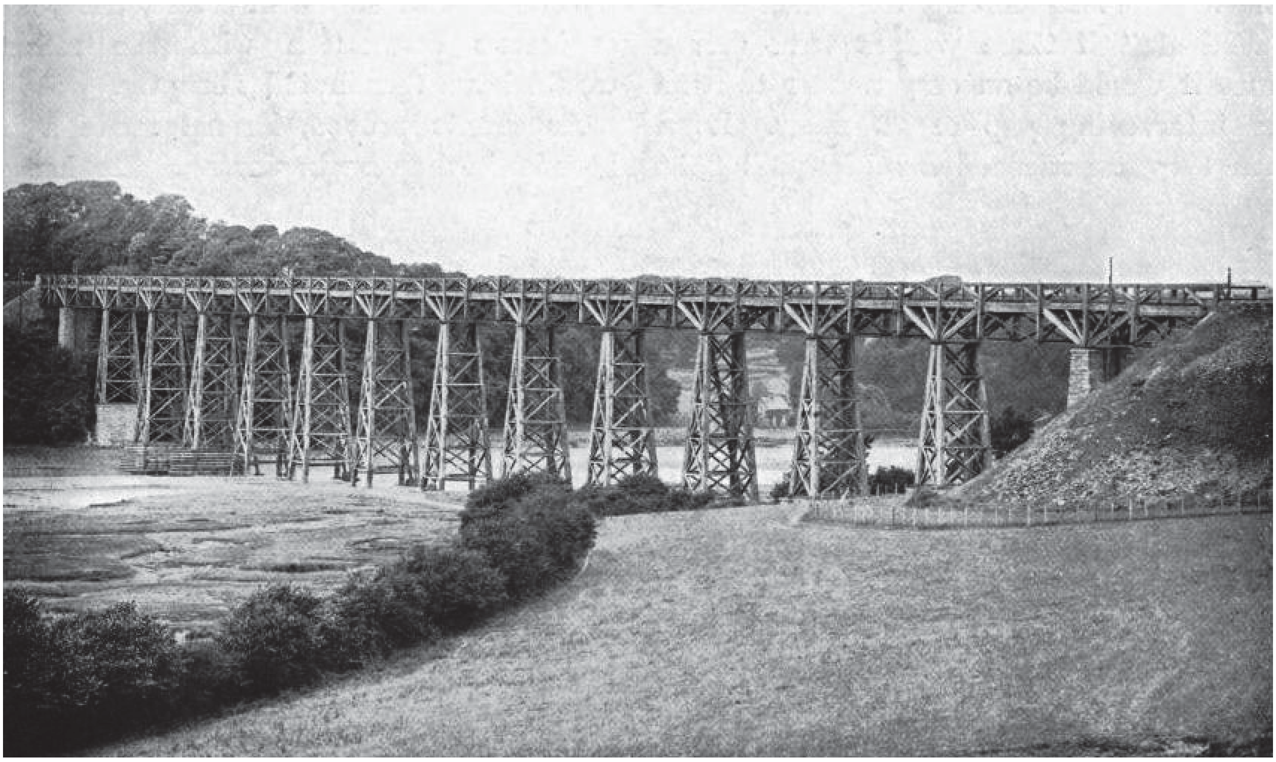


Figure 2.9: One of Brunell´s timber viaducts in Cornwall, (Lewis (2007))

2.2.2 Suspension bridges

As mentioned in previous section the first type of suspension bridges was constructed using vine or other suitable plants as load-bearing structure. It was first in the 19th century that the first suspension bridges appeared in Europe. Through the development of steel, suspension bridges were made possible. At the beginning, cast iron chains were used and when the production of steel cables were invented, the capacity was increased since it was possible to create long and continuous tension members. Timber was found to be a useful material for the bridge decks in the first suspension bridges. Timber was a light weight material and strong for it's weight. One example is the Tournon bridge built in France in 1845, see Figure 2.11.

Figure 2.10: Tournon bridge built in France 1845 (SETRA (2006a))

One example of suspension bridges in timber built in Sweden is Floda Kyrkälvbro in Dala-Floda, see Figure 2.11. The bridge was built in 1922 and the span is 158 m which makes it the longest timber suspension bridge for traffic load in Sweden. The bridge is restricted to one car of maximum 2 ton at a time and a maximum speed of 30 km/h. It is a interesting structure given the time it was built and that it is made out of timber. The bridge was destroyed in 1980 when a fire caused the collapse of the southern tower and the bridge collapsed. It was however rebuilt after the original drawings and was reopened in 1983.(Wikipedia (2016))

Figure 2.11: Floda Kyrkälvbro in Dala-Floda (Pousette (2008))

Today suspension bridges are common as pedestrian bridges when there is a need to cross longer distances. One contemporary example of a pedestrian timber suspension bridge is Traversiner steg 2 designed by the swiss engineer Jörg Conzett. The bridge replaced in 2005 the first Traversiner steg bridge built in 1996, also designed by Jörg Conzett. Both bridges are examples of the freedom you could have when designing bridges if you understand how the structures work, see Figure 2.13. Because of the remote location, Traversiner steg 1 was built off site and placed by use of a helicopter. The bridge is a combination of a truss and suspension bridge that instead of hanger cables in tension uses timber trusses. In 1999 the bridge was destroyed when a boulder fell down from the mountain side crushing the bridge. Travesiner steg 2 was later built on a place further down the valley. It is a suspension bridge with a arch shaped deck put up side down. The suspension cable is post tensioned, thus creating compression in the timber deck.

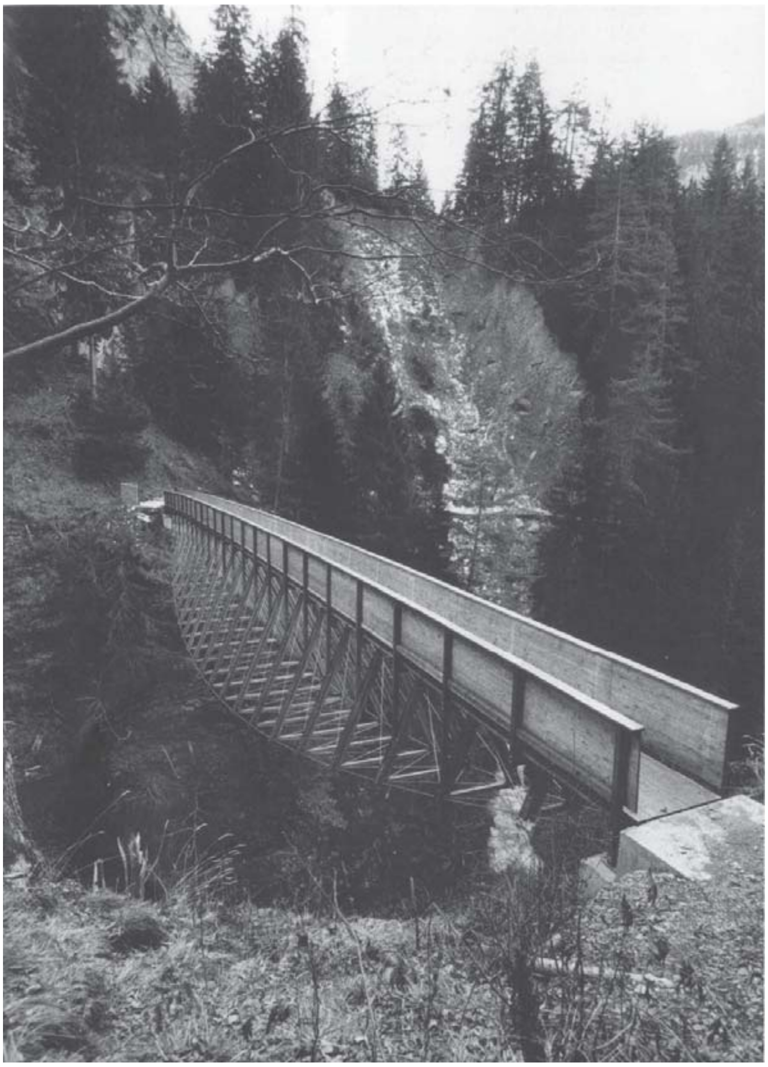


Figure 2.12: Traversiner steg 1 (Baus and Schlaich (2007))

2.2.3 Curved bridges

The development of curved bridges does not have a long history compared to the development of bridges. Robert Maillart, the famous Swiss engineer began to experiment with curved shapes in the beginning of the 1920th. He was an expert in concrete construction and saw the potential in casting concrete in free shapes. The Schandbach bridge was the first curved bridge that Maillart designed. (Keil (2012))

Figure 2.13: Schwandbachbrücke, Robert Maillart (Keil (2012))

After the Second World War the development of curved concrete bridges were expanding but they were kept to small span bridges. It was not until the 1980th there were a change in the development. Thanks to new computer software more complex structures could be developed and analysed and creative engineers began to design longer bridges and also curved suspension bridges.

The first bridge of the new kind was a bridge over Rhein-Main-Donau Channel designed by the German engineer Jörg Schlaich, Figure 2.14. Schlaich was one of the pioneers in designing curved suspension bridges and his office Schlaich, Bergerman und Partner is still among those offices who take the development of curved bridges further. One example is The Riphorst Bridge, Figure 2.15, of how the office of Schlaich, Bergermann and Partner has developed the concept of curved bridges from suspension bridges to curved arch bridges.

Figure 2.14: Footbridge over Rhine-Main-Danube Channel, Schlaich, Bergerman und Partner. (1987) (Holgate (1997))

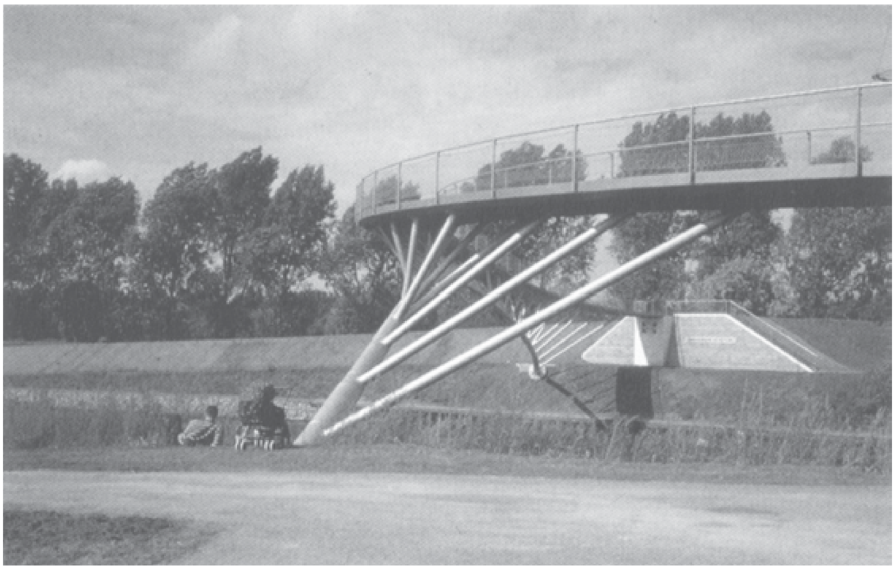


Figure 2.15: The Riphorst Bridge, Schlaich, Bergerman und Partner. (1997) (Baus and Schlaich (2007))

2.3 Details and Durability

Detailed correct, timber is a very valid alternative material to steel or concrete due to its low weight, fast construction time, low price and sustainable properties.

Moisture and rot is the main problem regarding durability in timber construction. When protected timber structures can stand for hundreds of years. If the moisture content is kept under 20 percent, wood will not be attacked by harmful organisms. One might achieve that by covering the load-bearing parts or by chemical treatment.

2.3.1 Protection of timber

As mentioned in the section about development of timber bridges it is crucial to protect timber from standing water, direct sunshine and organic materials. If timber is detailed and protected it can function for a very long time. Otherwise there is a high risk of mould, rot and deterioration which leads to structural failure.

There are different ways of protecting timber and they can be divided in two categories, chemical treatment of timber and constructive protection. Chemical treatment is when the timber is treated so that rot and mould is prevented or is very slowly developed. Constructive protection is when the load bearing timber parts are covered with timber panels or metal sheets to serve as protection.

Chemical treatment

Timber is not attacked by rot, mould or other organisms that damage the timber if the moisture content is kept under 20%. That is sometimes difficult to ensure if the timber is in direct contact with water or with the ground. It could also be that it is located in an area with a lot of rain and wind over a long time. In those cases treatment is needed if the structure should last over longer time.

One method of chemical treatment is impregnation. The timber is treated with a substance that is poisonous for the rot fungus that damage the wood. The impregnation, often a combination of copper and other substances is pressed into the wood to a certain depth depending on how long the protection should last. Impregnated wood has often a moisture content higher than 20 % but is not attacked by rot. The timber get in most cases a characteristic green color because of the copper in the substance. There are other types of impregnation that are more poisonous and give a better protection. Those substances often contain crome, arsenic or creosote, but they are forbidden in Sweden for bridge construction.

There are also methods of changing the chemical structure of timber and thereby increase its durability. When the chemical structure is modified the properties change. One method is heat treatment in an climate without oxygen. The timber is heated to 160-240 °C which makes it more resistant to rot, less swelling due to moisture or heat but it also lowers the structural properties making it more brittle. Therefore it is not recommended for structural parts. The treatment gives the timber a darker color. Other methods is using chemicals which react with the timber in a heated environment. Examples of that is acetylated wood which makes the wood harder, heavier, less swelling under influence of water and heat but it also makes it more brittle. The most common treatment of wood is surface treatment. The purpose is to protect the surface from UV-light and moisture which breaks down the surface. In many cases it is also an aesthetic purpose for the treatment. There are different kinds of paint, oils and varnish that are used. To protect against sunlight pigments are often added which do not let the destructive sunlight through. The pigment can also influence the surface temperature which changes the moisture content. In order to protect against moisture from the surroundings a water repellent layer can be applied keeping the water away from the timber. Another method is to use a surface treatment that does not let the water through. A more or less diffusion tight layer will keep the moisture content stable and lower the effect of swelling.

Constructive protection

Besides treating timber or covering it there are some things to keep in mind when designing a timber structure.

- Limit standing water on timber surfaces by making sure it has correct inclination
- Limit openings and slots where water can accumulate and not get out.
- Limit direct absorption though capillary suction when in contact with other materials
- Limit exposure of the end grain
- Make sure that the details and structure enables natural ventilation of all timber parts.

Timber bridges can be divided in open and covered bridges. The bridges in Luzern and in Shaffhausen mentioned before in this chapter are good examples of covered bridges. A bridge is considered to be properly protected if the angle from the part to the protecting cover is 30° or more in relation to the vertical plane. In an open bridge without a roof the bridge deck need to be treated or protected in some other way. Under the bridge deck there is a protected area as if the deck would have been a roof.

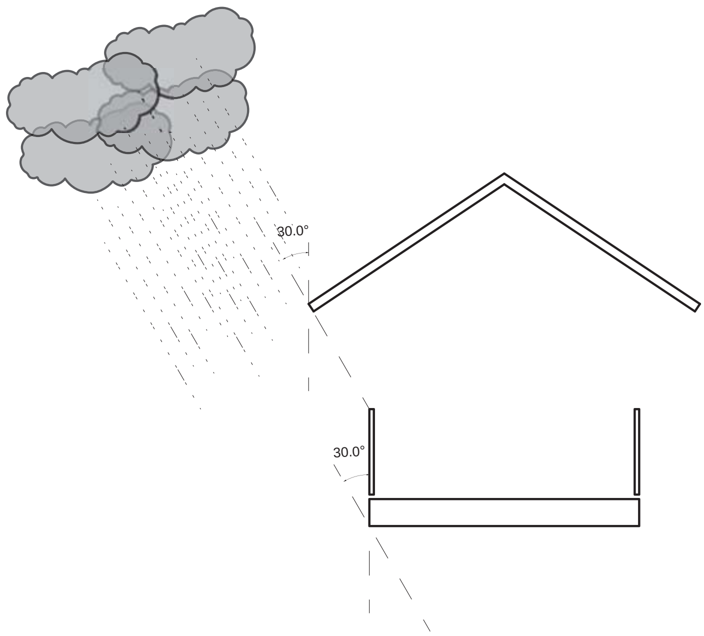


Figure 2.16: Area protected from rain

2.3.2 Joints

Traditional joint technique in timber structures only utilized wood and mainly transferred loads in compression. The Japanese carpenters/architects perfected the way to construct wood-wood joints. Also the Scandinavian way of building timber houses and churches used joints mainly working in compression. Examples of wood-wood joints are dovetail joints, mortise and tenon, splayed indent joints. These techniques are impressing and for a proper design they need craftsmanship and knowledge about how they work. In exterior joints as in bridges it is important to minimize the risk of water traps since that would damage the wood and thereby the joint.

Today the most common method to join wood members subjected to large loads are by metal-wood joining. Some examples are nails, spike nails, screws, bolts and notched rings. These joints can transfer the stresses through metal plates or through wood-metal-wood. Depending on the type of connector used, the joints can transfer compression, tension, shear and bending. Between the metal fastener and the the joint will slip a little when loaded. Just as important as the detailing with traditional joints the detailing of wood-metal is important. The wood should be protected against standing water that could lead to durability problems and failure. Multi-part joints are often more safe than mono-part ones since if one part fails there could still be capacity to transfer the stress. Timber joints fail either in a brittle or ductile manner. Brittle failure is when the timber component fails and could lead to a complete collapse of the whole structure. Ductile failure is when the metal part yields and is plastically deformed. Ductile failure is often a better failure mode since the connection deforms but the integrity of the structure can be maintained.

2.3.3 Stress laminated timber decks

Stress laminated timber decks was developed to be able to handle larger point loads. The technique was first developed in Canada and are now used all over the world. In stress laminated timber decks planks or beams are pressed together by high-strength steel rods that run through the deck and are post-tensioned. The compression creates friction between the elements and the deck will work as a orthotropic material. The concentrated load can then be distributed in the transversal direction of the deck.

Timber decks can be produced in straight or curved shapes and the length of the deck is theoretically endless. Ekholm (2013)

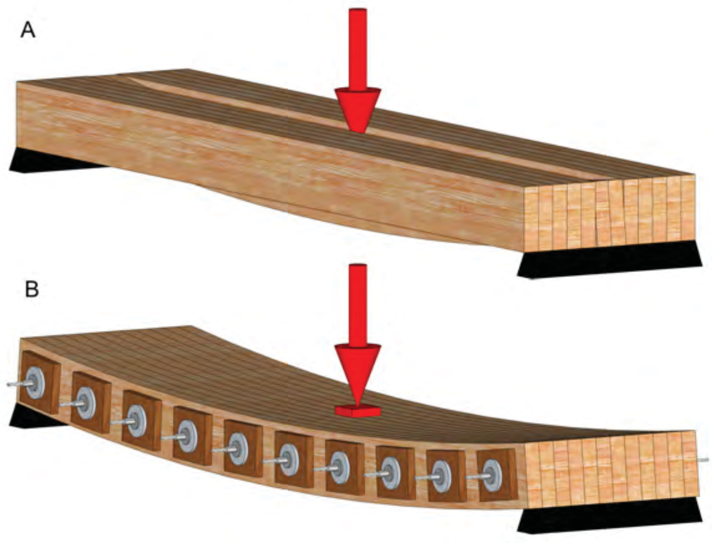


Figure 2.17: Timber deck and stress laminated timber deck, (Ekholm (2013))

Figure 2.18: Production of a curved stress laminated timber deck at Martinsons Träbroar (Pousette (2008))

3 Statics and dynamics of pedestrian bridges

It is important to understand the behaviour of structures when interpreting the results from the analysis. A double curved suspension bridge is a complicated structure that at first glance looks unstable. In order to understand and explain how it actually works it could be smart to begin with curved beams and suspension bridges.

Pedestrian timber suspension bridges are in most cases light weight structures which can be sensitive to vibrations. In most cases this might be a problem. It is therefore important to understand the dynamic behaviour and what to do if the analysis shows there are problems.

This chapter begin with explaining the principle of curved beams/bridges, suspension bridges and then curved suspension bridges. This chapter also explain the principle of dynamic behaviour in pedestrian bridges.

3.1 Statics of curved bridges/beams

A curved bridge could be simplified as a curved beam. Using the same dimensions as one of the curves in the double curved bridge as an example.

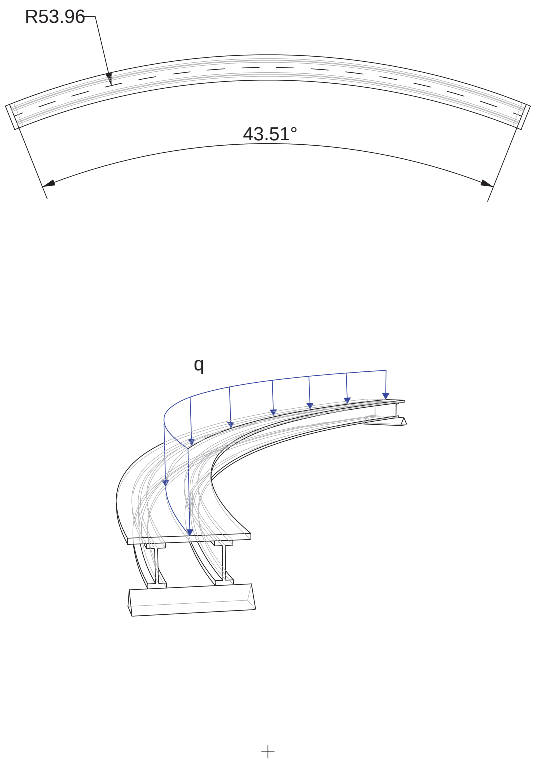


Figure 3.1: Illustration of a curved beam/bridge with a uniformly distributed load

The length of the bridge, see Eq. 3.1, can be calculated if the radius and angle are known. The bridge is subjected to a uniformly distributed load q.

$$l = \alpha_{rad} R \tag{3.1}$$

Where:

α_{rad}, *is the angle in radians*
R, is the radius
l, is the length

In the case of a simply supported straight beam bending will cause a compression force in the top flange and tension in the bottom one. That is also the case in a curved beam. However the curvature has an other effect, it creates radial forces in the top and bottom flange. These radial forces creates a uniformly distributed torsion. In Figure 3.2, it is possible to see why torsion occurs and how to calculated the magnitude of it. Considering one small segment and looking at the top flange where there are compression due to bending. In the bottom flange there will be tension instead and the forces will have opposite direction.

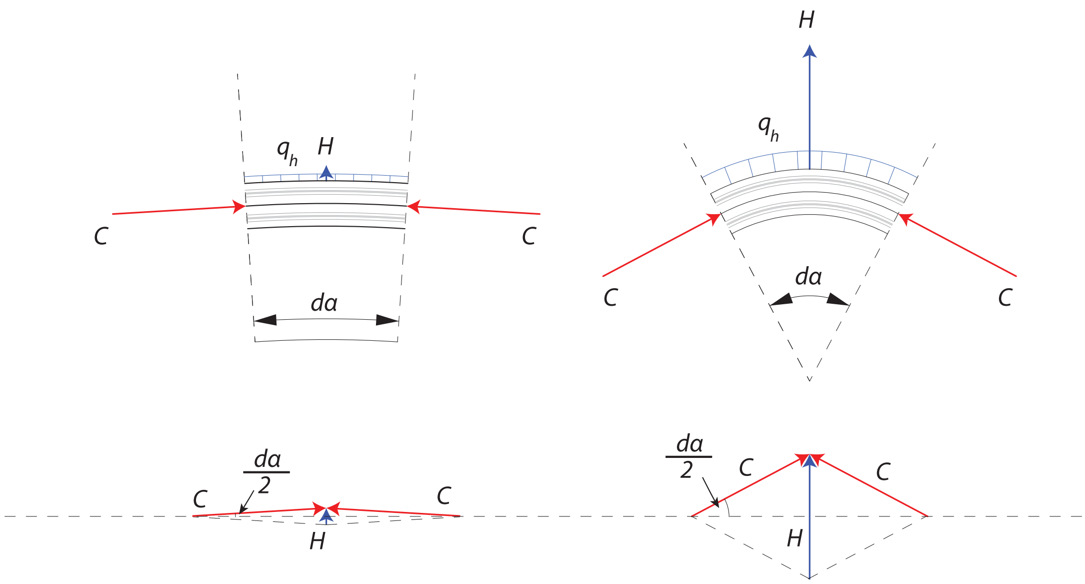


Figure 3.2: Forces in the top flange of a curved beam segment subjected to a uniformly distrubuted load

$$H = q_h * dl \tag{3.2}$$

$$dl = d\alpha * R \tag{3.3}$$

Where:

C, is the compression due to bending
H, is the horizontal resultant force
$d\alpha$, is the angle of the segment in radians
R, is the radius
dl, is the length of the segment

The force equilibrium shows that the resultant force can be expressed as,

$$H = C * \sin(\frac{d\alpha}{2}) \tag{3.4}$$

For $\alpha \ll 1$ $\sin(\alpha) \approx \alpha$. That means that we can rewrite the resultant force H as

$$H \approx C * d\alpha = C * \frac{dl}{R} \tag{3.5}$$

Figure 3.2 shows the effect of the radius and the angle of a curved segment. The compression C in top flange results into a resultant tangential force H that can be expressed as a uniform load q_h along the length of the segment. The same principle is true for the bottom flange, but instead of compression there is tension and the resultant forces are directed towards the center of the curvature.

The radial forces that is the result of tension and compression over the cross-section of the beam influence the beam in the way that it wants to rotate. To resist that the beam needs to be stiff and to handle the torsional moment that is created.

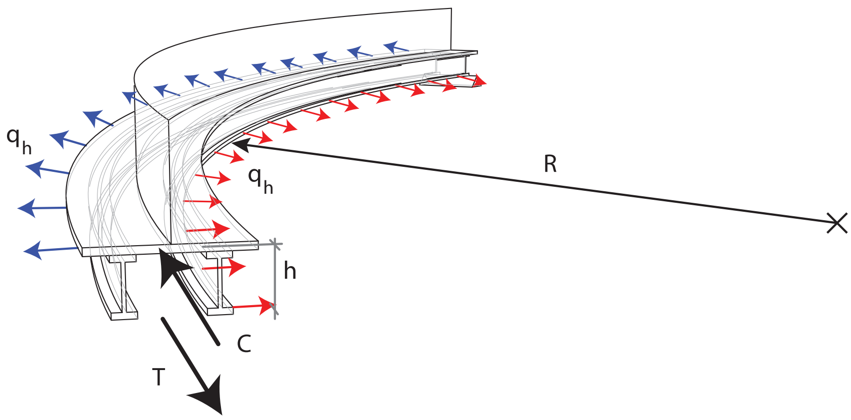


Figure 3.3: Forces in the top flange of a curved beam segment subjected to a uniformly distributed load

$$C = T \tag{3.6}$$

$$q_h = \frac{C}{R} \tag{3.7}$$

The compression in the top flange is equal to the tension in the bottom flange. That makes it possible to write the bending moment as,

$$M_y = C * h \tag{3.8}$$

$$m_t = q_h * h = \frac{C}{R} * \frac{M_y}{C} = \frac{M_y}{R} \tag{3.9}$$

Where:

q_h, *is the distributed horizontal load*
M_y, *is the bending moment in the longitudinal direction of the bridge*
m_t, *is the uniformly distributed torsion*

From equations above it is possible to see that the uniformly distributed torsion is depending on the bending moment and the radius of curvature of the bridge.

3.2 Statics of suspension bridges

A suspension bridge is a bridge where the deck is supported by hangers connected to a main suspension cable. The suspension cable is in most cases connected to one or two masts and the ends are connected to supports in the ground.

If a suspension bridge works as it is intended each hanger carries its corresponding part of the bridge deck and the load upon it. The load is then carried through the hangers to the suspension cable and then to the masts and the ground.

Cables are elements that can carry loads only in tension due to their very low bending stiffness. Since the cable only transfer loads in tension it will find a shape that enables it to transfer loads. If the load is a moving point load the shape will change. How much of load that distributes to the neighbouring cables depends on the stiffness of the deck.

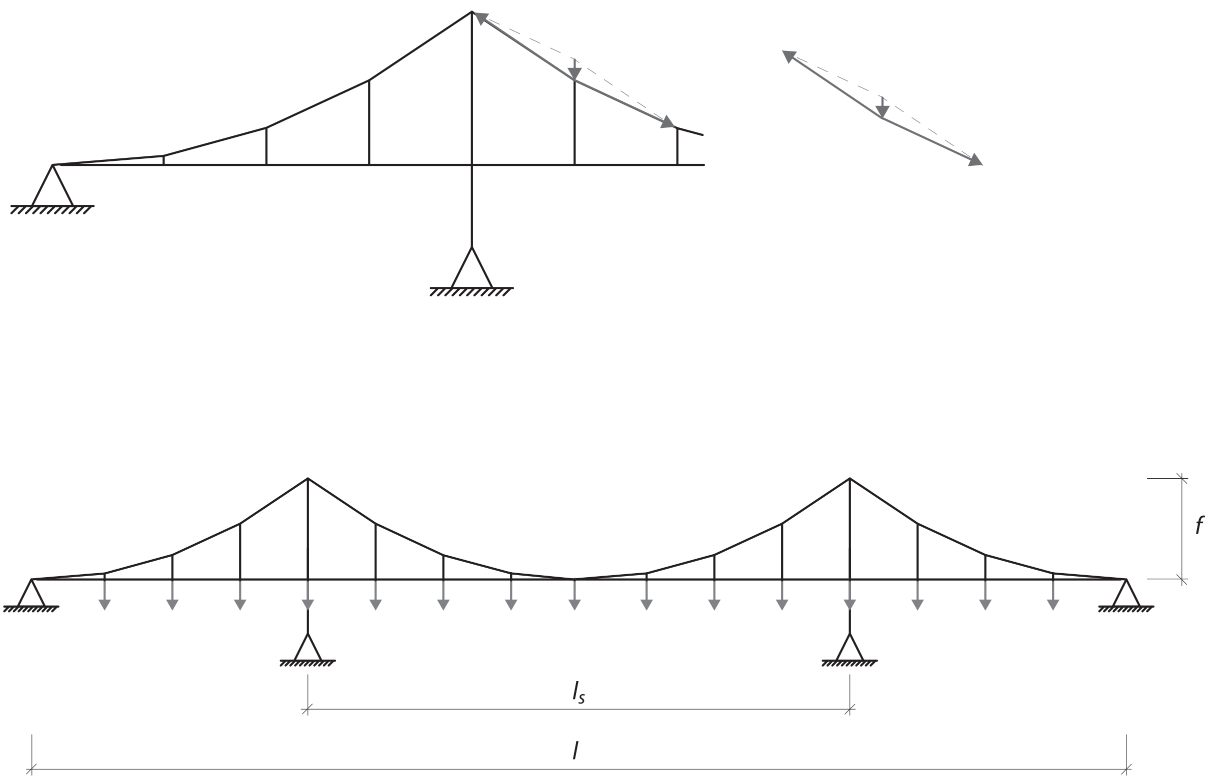


Figure 3.4: Principle of a suspension bridge

The tension in the suspension cable is depending on the load, the length of the span l_s and the sag f.

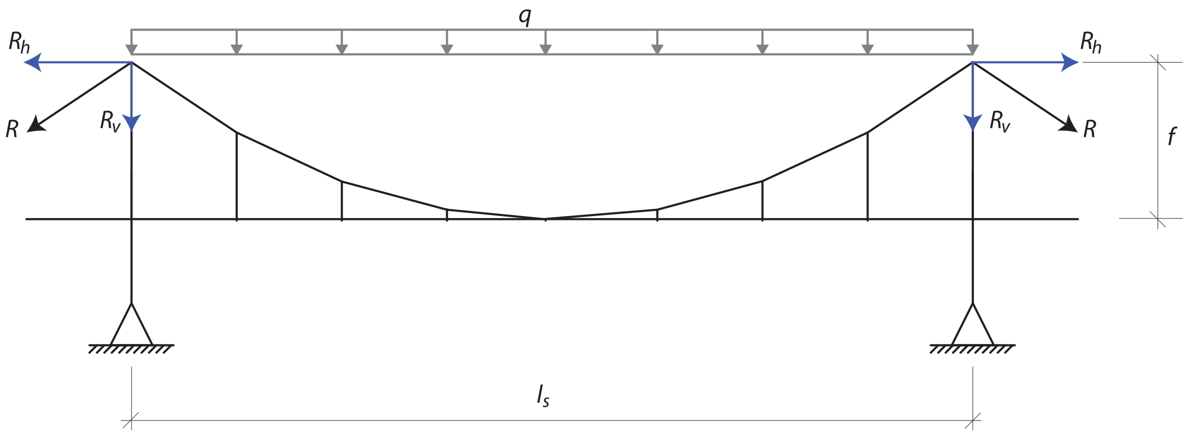


Figure 3.5: Moment distribution of the desired ratio EI/k

The vertical component of the force from the suspension cable is the sum of the forces from the bridge. The horizontal component depends on the other hand on the sag of the bridge.

$$R_h \approx \frac{q * l_s^2}{8 * f} \tag{3.10}$$

Where:

R_h, *is the resultant horizontal force*
q, is the uniformly distributed load on the bridge, including self-weight
f, *is the sag of the suspension cable*
l, is the length between the supports

The horizontal force from the cable can in cases with a low sag/length ratio be very large. The force can be transferred over the mast to another suspension cable and then anchored to the ground. In cases of self anchored bridges the cable is anchored in the bridge deck which leads to compression in the deck.

3.2.1 Stiffness of the bridge deck v.s. suspension system

The structural system is structurally indeterminate since the system of hangers connect to the stiff timber deck. The deck can be simplified to a beam on a bed of springs where the springs represent the stiffness of the hangers connected to the suspension cable, see Figure 3.6. The stiffness of suspension system and the stiffness of the deck govern how the stress distribution is in the deck.

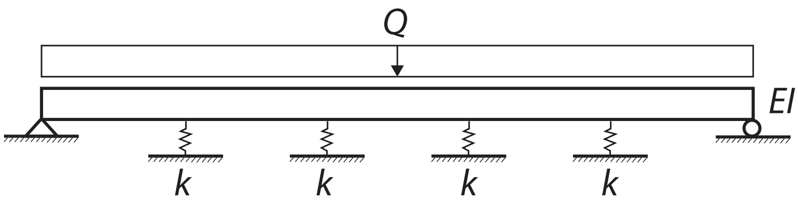


Figure 3.6: Simplification of the structural system as a simply-supported beam supported by a bed of springs

Depending on the ratio between the stiffness of the suspension system and the bending resistance in the deck, the moment and torsion distribution in the deck will differ. Spanning from a simply-supported beam when the stiffness in the wires is much smaller than the bending stiffness in the beam to a indeterminate beam with six supports when the stiffness in the wires is much greater than the bending stiffness in the beam.

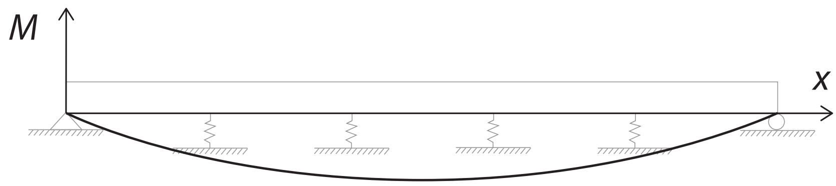


Figure 3.7: EI much larger k

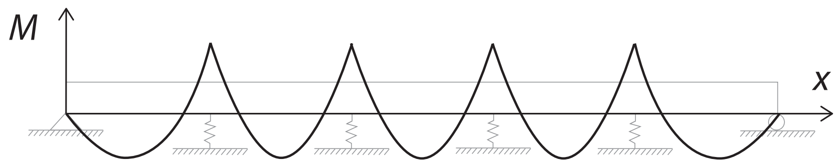


Figure 3.8: EI much smaller k

3.2.2 What happens if the hanger can not carry the corresponding load?

When the forces from the hanger cables are transferred to the suspension cable the cable finds a shape where it transfer the loads only in tension. If the length of the suspension cable and hanger cables are correct and under the right tension each hanger will carry its corresponding part of the bridge deck. If the length or the tension is incorrect, the suspension cable will again find a shape where it only transfer loads in tension. The hanger cables will on the other hand not carry its intended load of the bridge deck. The deck will be under more bending stress and with larger deflection. To evaluate if the length and tension of the cables is correct the tension in the hangers and the bending moment in the bridge deck can be used.

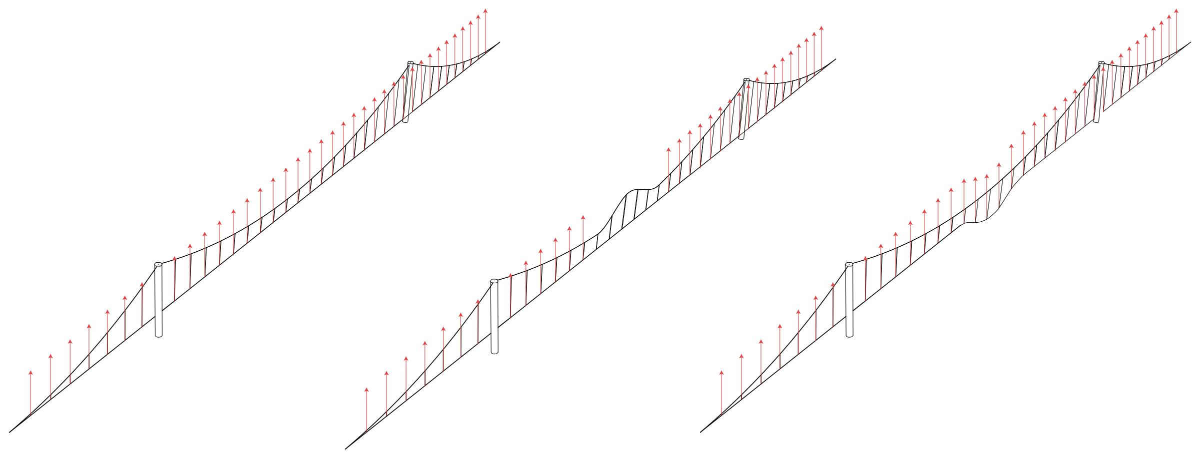


Figure 3.9: Suspension bridge, left: Each hanger carrying corresponding part of the bridge deck. middle: Suspension cable with incorrect length. right: Bridge where the suspension cable found a new shape and the bridge deck under more stress.

3.3 Statics of curved suspension bridges

The bridge from the mater thesis "A pilgrims walk" is constructed as a double curved suspension bridge where the bridge deck is hanged eccentrically. The system can be described as a suspension bridge where the pylons are shifted with respect to the longitudinal direction of the bridge, thus creating the S-shape when the bridge is looked from upon. The suspension cable, hanger and a cable representing the deck create a system in tension. On that system cross-laminated timber beams are laid on top. The fact that the beams are eccentrically placed on the tension system will result in a moment that needs to be counteracted with a radial force. That can be obtained through the shape of the bridge deck since it is shaped as an arch. The deck works as an arch that takes the forces in compression. It works in the same way as a vertical arch that takes vertical forces except that it is placed in the horizontal plane and counteract the moment that is created from the eccentricity of the load.

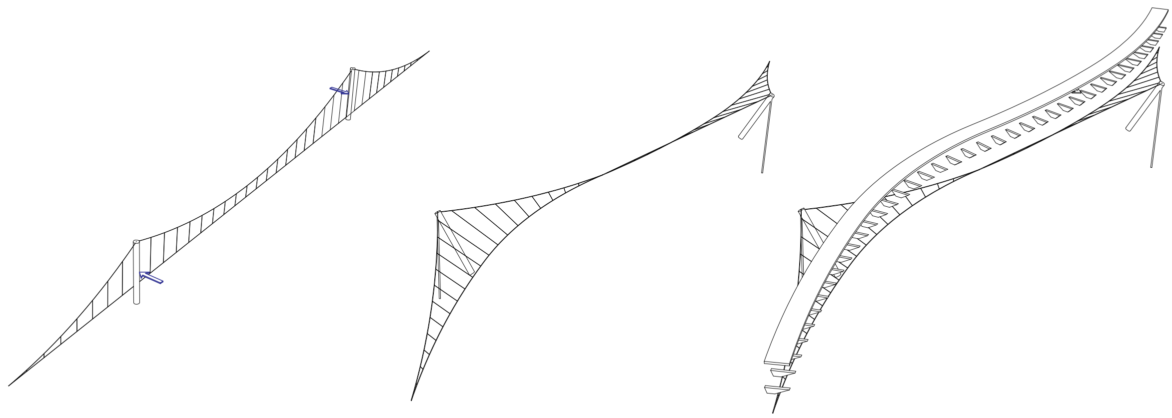


Figure 3.10: Princple of from the bridge in the thesis "A pilgrims walk"

That was how the bridge from the conceptual design was intended to work. The development of this thesis showed that it was complicated to model a system where the suspension cable runs under the bridge connecting to the tension cable. Much of the behaviour is still the same but more on that in the Analysis chapter.

3.3.1 Equilibrium of forces

To understand the structure and how it works one can first consider a cross section and then zooming out looking at the whole structure. Looking at the plane of the cross section the forces can be divided in vertical loads and loads radial to the curvature of the deck.

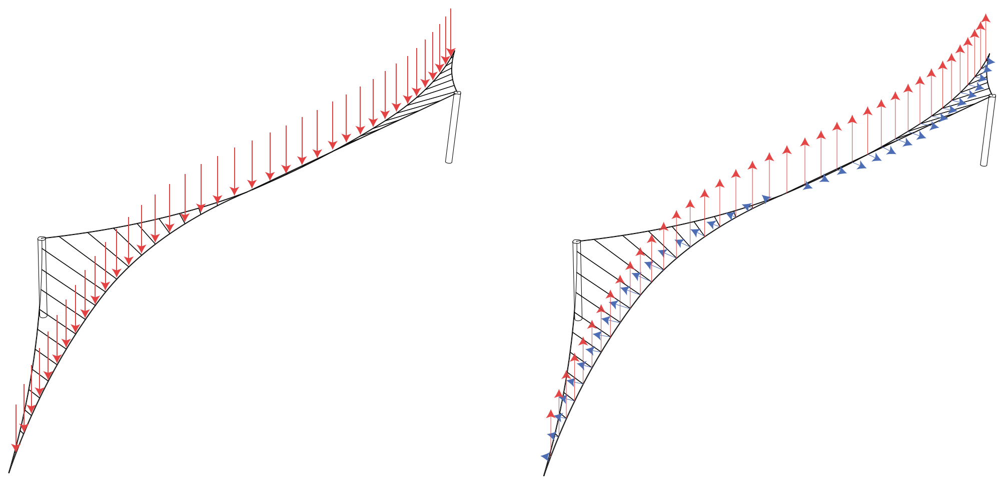


Figure 3.11: Right: Forces acting on hangers, Left: Force components in the hangers

The geometry and initial stresses in the cables were designed in such a way that the vertical components of the hanger forces balance the dead load. The horizontal/radial component of the hanger force is a result of the angle of the hanger.

Since the deck is hanged on only one side the cross section would tend to rotate around point A, see Figure 3.12. To counteract the rotation a radial force, T_x can be introduced by a horizontal cable running under the bridge with the same curvature as the deck. The force in the hanger cable is depending on the geometry of the cross section, width, depth and thickness and the angle of the hanger cable. In most cases the horizontal component of the hanger force is smaller than the radial force from the supporting cable under the bridge. That mean that there is a need for an other radial force counteracting the difference. That force can be introduced by the bridge deck which also has a curved shape. Now if we look at whole structure we have a structure that is in balance in each cross section and the forces effecting the global structure is the force from the inclined hanger and radial forces in the supporting cable and the timber deck. Since the bridge has a curvature the forces in the cable will be in tension and the forces in the deck will be in compression. Since the bridge is double curved and rotational symmetrical the compression and tension from each curve will balance out in the middle and also be transferred to the supports.

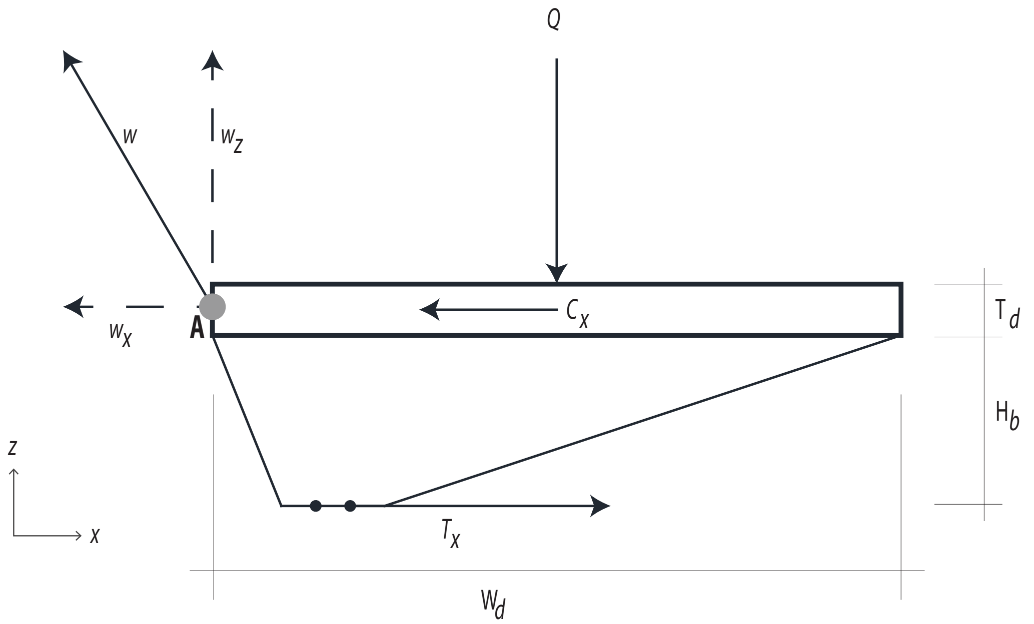


Figure 3.12: Priciple equilibrium of forces in a cross-section of the bridge

Where:

***A**, is the anchorage of the hanger cable*
$\boldsymbol{W}$, is the force from the wire, $\boldsymbol{W_x}$ and $\boldsymbol{W_y}$ are the corresponding components in x and z-direction
***Q**, is the self-weight of the bridge*
$\boldsymbol{T_x}$, is the radial force from the horizontal cable
$\boldsymbol{C_x}$, is the stabilizing radial force from the bridge deck
$\boldsymbol{h_b}$, is the height of the supporting structure for the bridge deck
$\boldsymbol{t_d}$, is the thickness of the timber bridge deck

3.3.2 Effect of connection point of the hangers

The forces in the deck and horizontal cable are depending on where the hangers connect to the bridge, in the bottom of the beam or at the timber deck, see Figure 3.13. For structural reasons, alternative A, where the connection is located at the deck is to prefer since horizontal component of the hanger is contributing to balancing the moment created from the eccentric suspension.

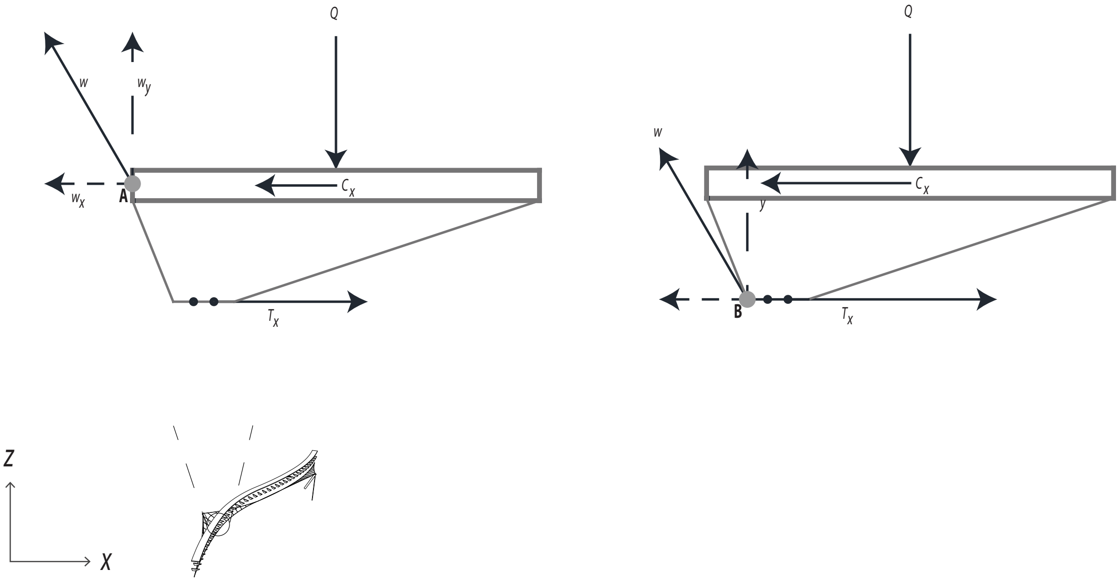


Figure 3.13: Different connection points for the hanger and the bridge.
A) Connecting to the bridge deck
B) Connecting to the horizontal wire

There are some problems though, the forces need to be transferred from the connection to the bottom of the beam and the horizontal wire. That means the timber beam will be under tension. That is not by it self such a big problem, however combined with a more exposed connection that could result in problems with durability. Important is also to note that the tension force from the main wire need to be anchored on different sides of the bridge in a case of a double curved bridge. This anchorage will lead to moment and shear forces in the middle of the bridge.

The benefits with this composition is that it is simpler to anchor the wires since they do not intersect with the bridge deck. Since the suspension system and the horizontal wire is not directly connected it is less complicated to find the right tension in the wires.

In alternative B, where the hangers connect to the horizontal wire, the horizontal component is instead increasing the stress in the deck and the horizontal wire. Depending on the angle and where under the deck it connects the stress change. The smaller the angle between the hanger and x-axis the larger will the horizontal component be. However, the more centred the connection is, the smaller will the moment caused by the eccentric load be.

This alternative is much more complicated to find good location for the wires. However, since the bridge is double curved the tension force in the suspension cable can balance each other in the middle of the bridge and need not to be anchored on different sides of bridge deck.

3.3.3 Derivation of cross-section forces

Again looking at at the cross section of the bridge, see Figure 3.12. When knowing the geometry of the section, inclination of the hanger cable and the load applied equilibrium equations can be formulated.

$$\uparrow: W_z - Q = 0 \tag{3.11}$$

$$\leftarrow: W_x + Cx - T_x = 0 \tag{3.12}$$

$$\curvearrowright_A: Q * b/2 - T_x * (h_b + \frac{t_b}{2} = 0 \tag{3.13}$$

Going from the cross section to whole bridge we end up with radial forces affecting the bridge deck and the horizontal wire. Also the force from the hanger which depends on the angle and the load that is applied from one bridge segment.

The resultant forces from the radial forces can be approximated using the Boiler formula. The boiler formula was first used to calculate the forces caused by the pressure in water boilers. The formula can be used to calculate the forces in cables which are radially loaded using the radius of the center line and the distributed radial load. The same formula is valid for compression.

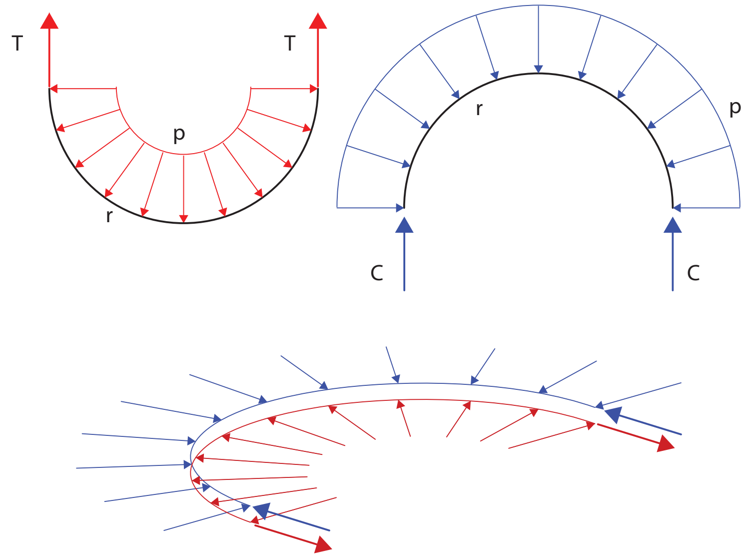


Figure 3.14: Tension and compression ring

$$C = p * r \tag{3.14}$$

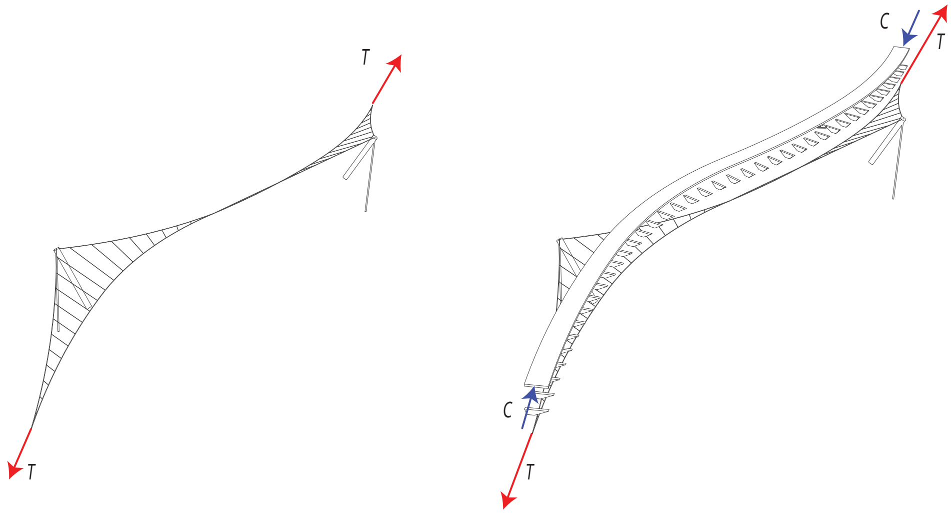


Figure 3.15: Priciple equilibrium of forces in a cross-section of the bridge

3.3.4 Shape of the suspension cable: what happens if it is not correct?

A normal suspension bridge with a straight deck has often two suspension cables on each side and the cables lies in a vertical plane. As mentioned before the shape of the cable is important if the bridge deck and hanger cables should be under the correct stresses. The shape of the cable for a evenly distributed load is not that complicated to find since the cables only take forces in tension and the force in each hanger is the same. In a curved suspension bridge it is a more complicated system. Each force in the hangers is different and have a different angle. That means the suspension cable needs to be a three dimensional funicular shape. Before computers were available complicated tension structures were designed with large physical models by engineers/architects like Frei Otto and Antoni Gaudi amongst others. Today it is possible to calculate the shape by means of adequate computer software and some knowledge in programming. The process is called Form-finding and there are different methods. In this thesis a method called Dynamic relaxation have been used. More on how dynamic relaxation has been used is shown in Chapter 4 Method.

But what happens if the suspension cable does not have the correct shape? We saw before that in a normal suspension bridge the result will be that bridge deck will be subjected to more bending stress and that the suspension cable will find a path where it can carry loads. It is the same for a curved suspension bridge. The deck will be deformed and the cable will find a path where it can transfer loads. Since the bridge deck is curved in will will act more like a curved beam. That is there will be more bending stress and also torsion in the cross section.

3.4 Dynamics of pedestrian bridges

Pedestrian bridges are often light-weight structures that can be sensitive to vibrations. Timber bridges are no exception and dynamics is a important subject to consider during design of bridges. The dynamic vibrations of bridges can effect the user comfort. How people react to vibrations depends on many parameters. Who are causing the vibrations, is it your self or some one else that you can't control. Is the bridge of the type that you would expect to be vibrating? A long thin suspension bridge in the countryside might be expected to vibrate and have large deflections and might not give a bad user comfort. Compared to a more solid bridge in a urban environment which could be felt unsafe even with small vibrations.

There are two types of loads that cause vibrations in pedestrian bridges. That is vibrations caused by people walking on the bridge and vibrations caused by wind. The eigegenfrequencies and eigenmodes for the bridge is good to know to understand how it works. A beam bridge with a certain length simply supported on each side will vibrate with half-sine wave length over the whole span and with a certain frequency. The frequency mainly depends on the bending stiffness of the beam mass. Just like a guitar string, that has one main frequency but also vibrate in other frequencies crating the specific sound, a bridge or any structure has several eigenfrequencies and eigenmodes.

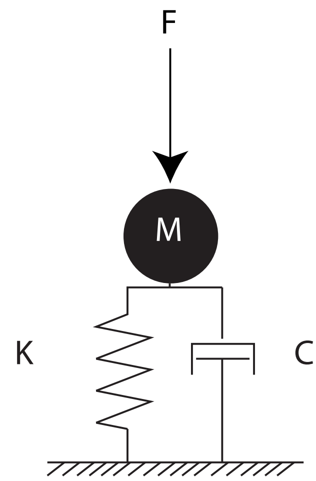


Figure 3.16: Dynamic system

To predict the dynamic properties of a pedestrian bridge is difficult. The rules for what is acceptable is not always helping. The frequencies for a bridge depend on its mass, stiffness and damping of the bridge. Especially the damping is difficult to predict since it depends on the connections and things like the railings and deck surfacing influence the vibrations and frequencies. Many pedestrian bridges which through analysis show that the eigenfrequencies are with in the range of what can be caused by pedestrians moving over the bridge, show no problems when built. If the dynamic properties are crucial in a bridge project it could be smart to prepare for some actions if measurements show that the there is a problem when the bridge is built. Some of these actions could be installing dampers, increase the mass or the stiffness (Schlaich).

3.4.1 Vibrations caused by pedestrians

Vibrations caused by pedestrians can be divided in vertical and horizontal vibrations. If eigenfrequencies of a bridge are within a range of 2,5-5.0 Hz then the bridge can be set in motion by people walking. There are also horizontal loads from pedestrians and if the horizontal eigen frequiencies is between 0,5 and 2,5 Hz then that is also within the range of what could induce vibrations. If a bridge would have eigen frequencies within that range then controls should be made that the accelerations do not exceed the limits.

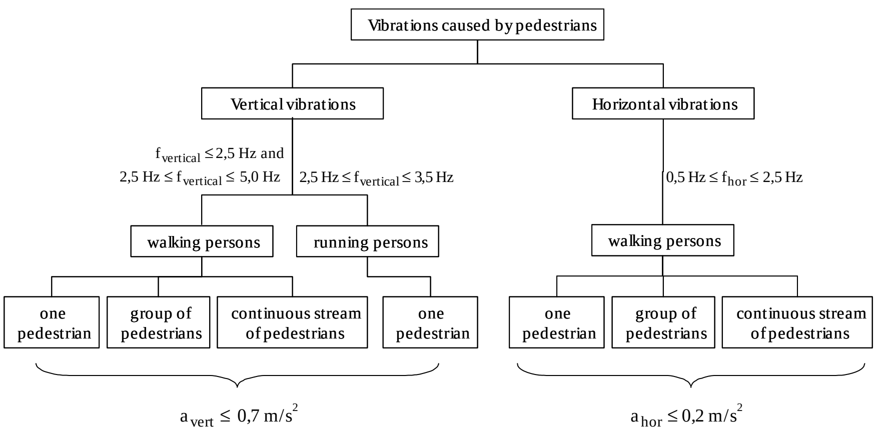


Figure 3.17: Classifying frequencies from pedestrian loading (Hamm (-))

As mentioned before, if the eigen frequencies is within the range of pedestrian loading the size of accelerations should controlled. That is possible to do through finite element software or through a simplified method. The method begin with calculating the response from one person walking or running.

$$a_{vert,1,walking} = \frac{200}{M' * 2\zeta} \tag{3.15}$$

$$a_{vert,1,running} = \frac{600}{M' * 2\zeta} \tag{3.16}$$

$$a_{hor,1,walking} = \frac{50}{M' * 2\zeta} \tag{3.17}$$

The value for generalized mass M' can be extracted from SAP2000 or other FEM-software and the damping ratio ζ has to be estimated.

Testing of 20 timber bridges has shown that the value of damping factor for different types of bridges,see Table 3.1

Table 3.1: Recomended damping ratios ζ based on measurements on 20 timber bridges(Hamm (-))

Static system of beam/bridge	Damping factor ζ
Glued laminated beam	0,50 %
Bending beam bridges made of glued laminated timber	1,20 %
Suspended beam bridge	0,90 %
Frame work bridges	0,80 %
Cable stayed bridges with short cables	1,00 %
Cable stayed bridges with long cables	0,30 %

When the response from one pedestrian is known the response for a group of people or a stream can be calculated. A group of people is considered to be 13 people and a stream of people is calculated as 0,6 $persons/m^2$.

$$a_{vert,n,walking} = 0,23 * n * k_{vert} * a_{vert,1,walking} \tag{3.18}$$

$$a_{hor,n,walking} = 0,18 * n * k_{hor} * a_{hor,1,walking} \tag{3.19}$$

$$a_{vert,stream,walking} = 0,23 * (0,6 * w * l) * k_{vert} * a_{vert,1,walking} \tag{3.20}$$

$$a_{hor,stream,walking} = 0,18 * (0,6 * w * l) * k_{hor} * a_{hor,1,walking} \tag{3.21}$$

The coefficients k_{vert} and k_{hor} are used to account for how the accelerations relate to the natural frequencies.

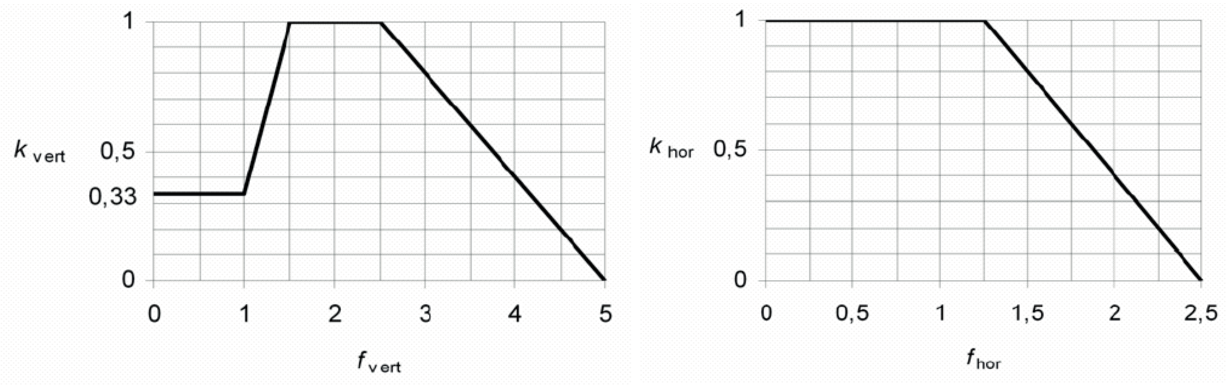


Figure 3.18: Relationship between k_{vert} and k_{hor} and natural frequencies for horizontal an vertical vibrations

As mentioned before is the user comfort depends on many parameters. Is the bridge located in a urban environment the probability for the bridge to have a stream of people walking over is larger. Also the expectations of how the bridge should behave has a large part in how the user comfort is. In the country side it is not often that that a full stream of people will walk over the bridge and also the suspended structure has a expectation to behave a little lively.

3.4.2 Alternative method to calculate accelerations from pedestrian loading

The procedure for calculating accelerations in pedestrian bridges according to the standards of Eurocode has been under debate. As an alternative method to verify the dynamic behaviour is a method proposed by SETRA, Service d'études sur les transports, les routes et leurs aménagements. (SETRA (2006b))

The method is based on four stages:

- Stage 1: Determination of footbridge class
- Stage 2: Choice of comfort level by the owner
- Stage 3: Determination of frequencies and of the need to perform dynamic load case calculations or not
- Stage 4: Stage 4 if necessary: calculation with dynamic load cases.

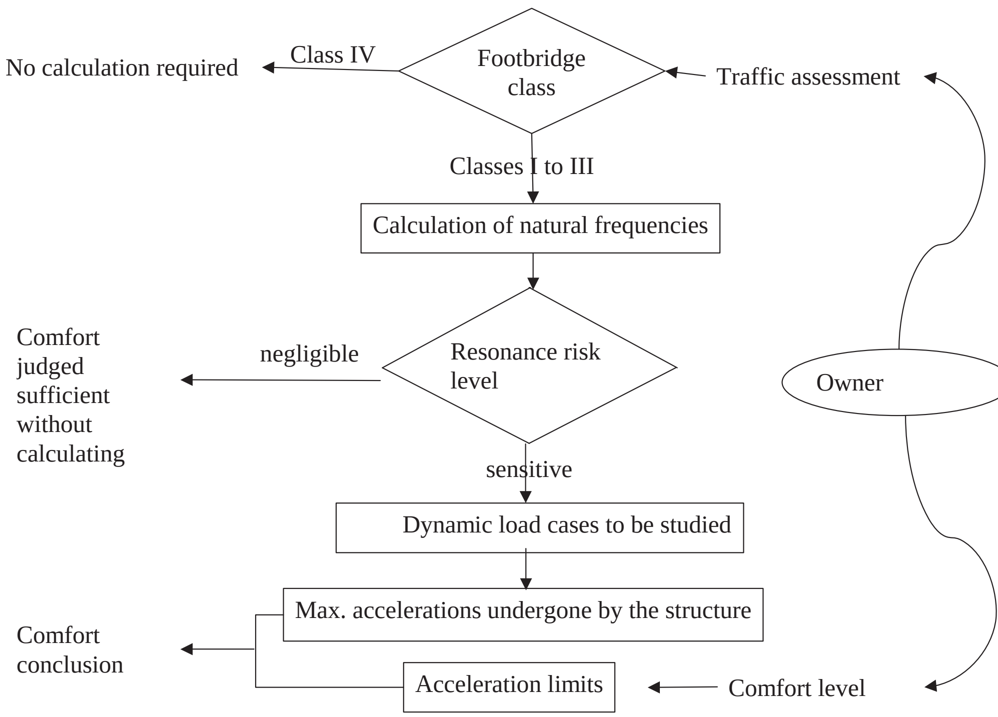


Figure 3.19: Design procedure according to SETRA (SETRA (2006b))

Stage 1

To be able to determine the level of pedestrian loading on a bridge SETRA have defined four classes. Below is the classes and how they are classified according to SETRA. It is the owner who are responsible to determine which class should be applied and taking in account changes in use. Very light suspension bridges are recommended to at least be in Class III.

- Class I: urban footbridge linking up high pedestrian density areas (for instance, nearby presence of a rail or underground station) or that is frequently used by dense crowds(demonstrations, tourists, etc.), subjected to very heavy traffic.

- Class II: urban footbridge linking up populated areas, subjected to heavy traffic and that may occasionally be loaded throughout its bearing area.

- Class III: footbridge for standard use, that may occasionally be crossed by large groups of people but that will never be loaded throughout its bearing area.

- Class IV: seldom used footbridge, built to link sparsely populated areas or to ensure continuity of the pedestrian footpath in motorway or express lane areas.

Stage 2

Definition of comfort levels is also for the owner to determine. How vibrations are perceived is highly subjective and these levels should not be seen as absolute. Who are using the bridge should also be taken into consideration.

- Maximum comfort: Accelerations undergone by the structure are practically imperceptible to the users.

- Average comfort: Accelerations undergone by the structure are merely perceptible to the users.

- Minimum comfort: under loading configurations that seldom occur, accelerations undergone by the structure are perceived by the users, but do not become intolerable.

SETRA has given four ranges as guidelines for which levels of accelerations corresponding to which comfort level. The fourth range is what can be seen as acceptable.

	Vertical accelerations (Hz)	Horizontal accelerations (Hz)
Range1(Max)	0-0,5	0-0,15
Range2(Medium)	0,5-1	0,15-0,3
Range3(Minimum)	1-2,5	0,3-0,8

Table 3.2: Ranges of accelerations according to SETRA

Stage 3

In stage 3 the frequency range should be determined for two mass cases. One with an empty bridge and one with a fully-loaded bridge with a load of 700 N/m^2 representing one person per square meter. The risk of resonance can then be determined with corresponding frequencies.

Range 1: Maximum risk of resonance
Range 2: Medium risk of resonance
Range 3: Low risk of resonance for standard load situations
Range 4: Negligible risk of resonance

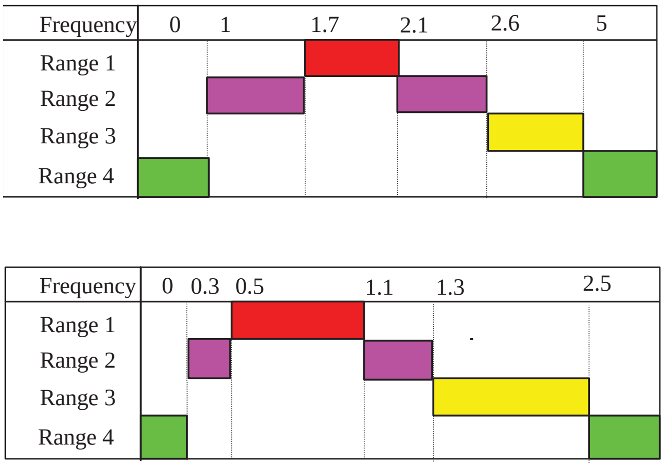


Figure 3.20: Frequency ranges (Hz) of the vertical and longitudinal vibrations (top), Frequency ranges (Hz) of the transverse horizontal vibrations(bottom) (SETRA (2006b))

		Load cases to select for acceleration checks		
		Natural frequency range		
Traffic	Class	1	2	3
Sparse	III	Case1	Nil	Nil
Dense	II		Case 1	Case 3
Very dense	I	Case 2	Case 2	Case 3

Case No. 1: Sparse and dense crowd
Case No. 2: Very dense crowd
Case No. 3: Crowd complement (2nd harmonic)

Figure 3.21: Load cases under consideration depending on bridge class and frequency range (**SETRA2006**)

Stage 4-Calculation with dynamic load cases

If the verification in the first three stages show that there is a need for dynamic calculations. The three dynamic load cases according to SETRA are:

Case 1 Spare and dense
Case 2 Very dense
Case 3 2nd harmonic effect

A main difference in terms of design according to Eurocode is that the calcutaion of the dynamic loads are depending on the bridge class.

The dynamic load per square meter for the first and second load case is presented in Table 3.3 and 3.4.

Direction	Load per square meter
Vertical	$d * (280N) * \cos(2 * \pi * f_v * t) * 10,8 * \sqrt{\frac{\epsilon}{n}} * \psi$
Longitudinal	$d * (140N) * \cos(2 * \pi * f_l * t) * 10,8 * \sqrt{\frac{\epsilon}{n}} * \psi$
Transversal	$d * (35N) * \cos(2 * \pi * f_l * t) * 10,8 * \sqrt{\frac{\epsilon}{n}} * \psi$

Table 3.3: Load per square meter for Case I

Direction	Load per square meter
Vertical	$1,0 * (280N) * \cos(2 * \pi * f_v * t) * 1,85 * \sqrt{\frac{1}{n}} * \psi$
Longitudinal	$1,0 * (140N) * \cos(2 * \pi * f_l * t) * 1,85 * \sqrt{\frac{1}{n}} * \psi$
Transversal	$1,0 * (35N) * \cos(2 * \pi * f_l * t) * 1,85 * \sqrt{\frac{1}{n}} * \psi$

Table 3.4: Load per square meter for Case II

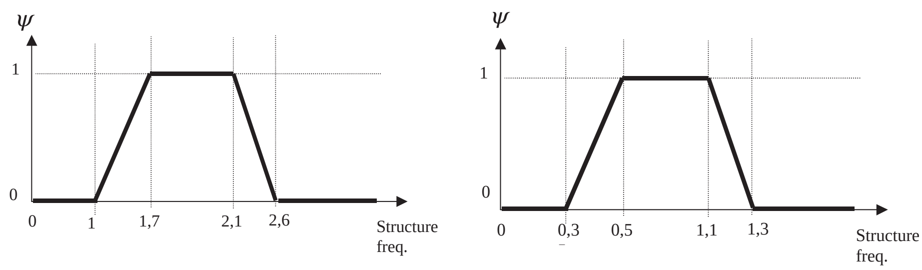


Figure 3.22: Value of the minus factor ψ for Case 1 and Case 2. Vertical and longitudinal vibrations on the left and horizontal on the right. (SETRA (2006b))

The third load case take in consideration the effect of second harmonic loading when the frequency from the pedestrians is double the first natural frequency.

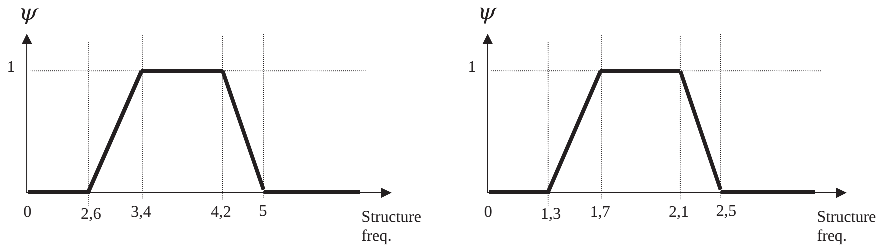


Figure 3.23: Value of the minus factor ψ for Case 3. Vertical vibrations on the left and lateral on the right. (SETRA (2006b))

After the dynamic load cases has been determined, analysis performed through a Finite element software and the frequencies and accelerations obtained it is possible to evaluate if the bridge is ok or not.

Stage 5- Modification of the project

If the accelerations should be considered to be to high in relation to the demands of the owner, the bridge need to be modified. Going back to the basic of what influence dynamic behaviour there are four options.

- Changing the stiffness of the structure
- Increasing the mass/tuned mass damper
- Installing dampers to increase the damping factor

4 Method

To be able to have a analytical approach to the design of the bridge several tools were used. Starting with sketches, physical models, hand calculations and computer models in thesis "A pilgrims walk" to then develop the geometry and export it to SAP2000 for further analysis. The results from the analysis have then been interpreted and compared with hand calculations of how the structure should have been behaving. The structure has then been modified and once again imported for further analysis. The results from SAP2000 have then been used to dimensioning of members according to Eurocode 5.

4.1 Sketches, physical models and hand calculations

In the Chapter 2, some of the process from this thesis. The process was a mix of methods going parallel to each other.

4.2 Rhino3d and grasshopper

Rhino3d and Grasshopper have been used to find the geometry of the bridge. The combination of these two software enables you to build a parametric 3D-model that is easily changed, which was important while developing the concept.

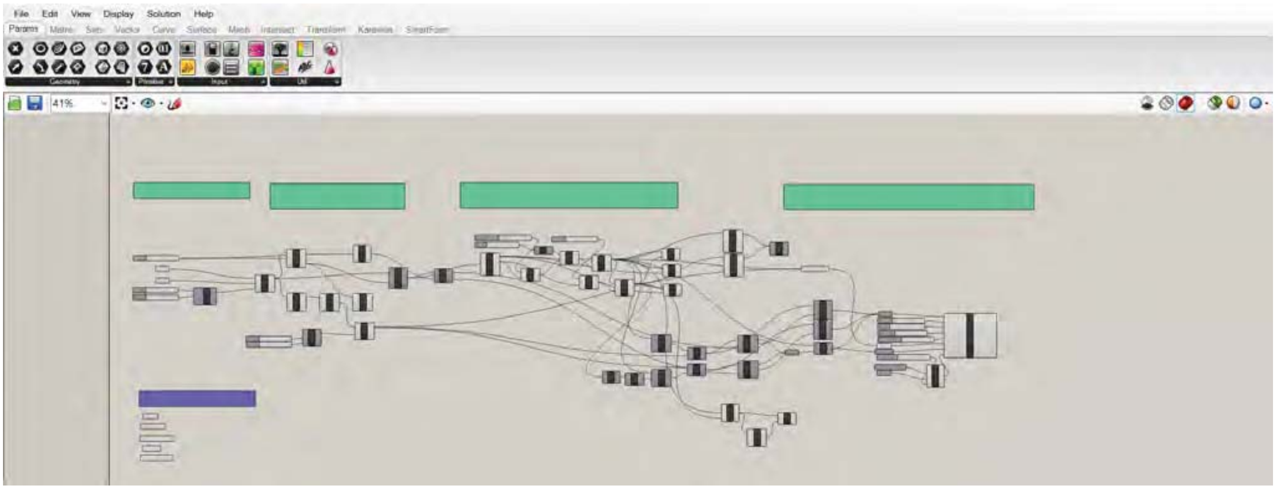


Figure 4.1: View of graphical programming in Grasshopper

Rhino3D is a 3D-modelling software that is used for architectural- and product design. It has been developed to be a effective and exact modelling tool that communicates well with other software. Rhino can also be used to produce drawings, illustrations and renderings. From Rhino3d it is possible to export a .dxf file to SAP2000 and use as a start for building a FEM-model.

Grasshopper is a graphical programming plug-in for Rhino3D. Graphical programming is a way of building a 3D-model by connecting different components or functions. The functions that is used is the same the ones that are in Rhino3d and the result is visible in the views in Rhino. By building up a parametric model in Grasshopper and Rhino it is possible to change parameters for curvature of the deck, thickness, division in segments and so on.

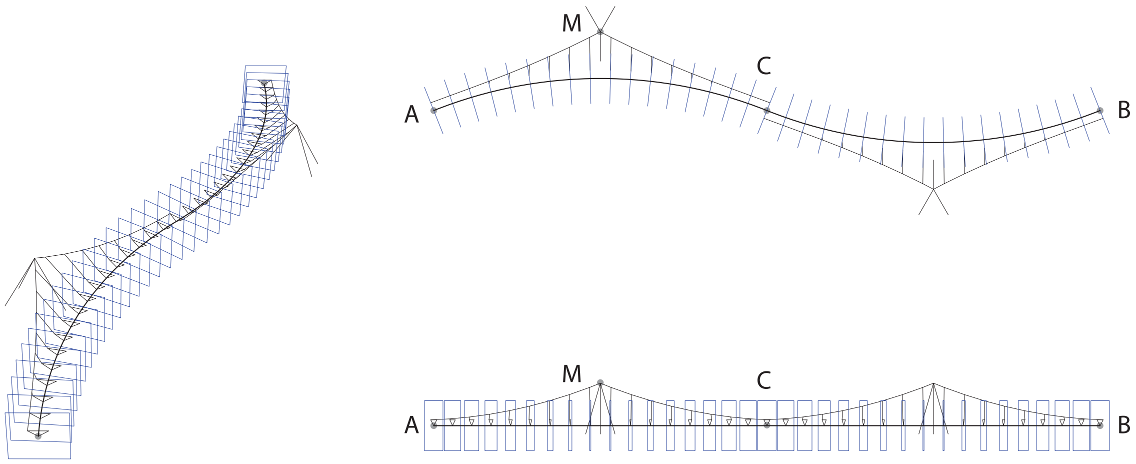


Figure 4.2: Parametric model

The parametric model for this thesis has two points, A and M, as input from Rhino3D to Grasshopper, see Figure 4.2. Since the bridge is rotational symmetric around point M first curve of the bridge is rotated to create the whole bridge. Between these points a curve with a parametric radius as a center line is drawn. The center curve is then divided in a number of segments with perpendicular planes in each point. Grasshopper works well to rationalise modelling of many objects with the same basic build-up but with small differences. It is therefore useful to continue modelling from the planes in created along the center curve. Parameters for the width, hight of the bridge and parameters for the struts and beams is also added.

Parameters for where and how high the mast is was also added creating the point M.

The suspension cable and hangers are created as line-segments to be able to used to later find the correct path of the cable. By creating lines from point A-M and M-C and creating new points in the intersection between the planes and the new lines. The hangers are modelled as lines connecting to points on the bridge and the suspension cable. The lines symbolising the suspension cable and hangers are the used in a customized component for form finding.

4.3 Form Finding of the suspension cable

As stated in the previous chapter each section point and cross-section should be in equilibrium. The wires can only withstand forces in tension and it is also desirable that the deck is under pure compression during dead load.

The shape and the curvature of the deck and the wires are deciding weather the system behaves as intended or not. By assuming that the entire vertical load is taken by the suspension system, i.e assuming that the stiffness of the suspension system is considerably larger than the vertical stiffness in the deck, an iterative process where the resultant force vector in each node is calculated and the node is the moved.

4.3.1 Form finding through dynamic relaxation

The simplest way of form finding through dynamic relaxation is an explicit way of finding the statical shape of a tensile structure. The method is based on the movement of each node during a small time-step, Δt. The method is based on Newtons second law.

$$F = m * a \tag{4.1}$$

The aim of the method to find a static solution and not to find the dynamic response of the structure as the name could indicate. The mass is therefore not the real nodal mass but a fictitious mass that is set to enable fast convergence.

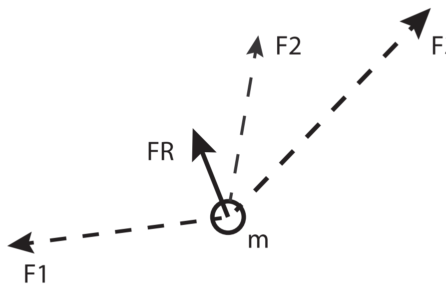


Figure 4.3: Forces affecting one point

The force F is the resultant of all forces in the node. These forces can be tension forces from connecting cables, gravity effecting the nodal mass and applied loads as some examples. The resultant force F_R is:

$$F_R = F_1 + F_2 + F_3 \tag{4.2}$$

The acceleration of the node i at time t can be approximated as:

$$a_i^t = \frac{v_i^{t+\Delta t} - v_i^{t-\Delta t}}{\Delta t} \tag{4.3}$$

Combining Equations 4.1 and 4.2 produces the equation for the velocity in node i.

$$v_i^{t+\Delta t} = v_i^{t-\Delta t} + \frac{\Delta t}{m} * F_R \tag{4.4}$$

The new node position is then calculated, see Equation 4.5.

$$p_i^{t+\Delta t} = p_i^t + \Delta t * v_i^{t+\Delta t} \tag{4.5}$$

When the new geometry is obtained, the velocities are set to zero and the new nodal resultant forces is calculated from the strain of the cables connecting the node, applied loads and gravity.

The process is then iterated until the static solution has converged to static equilibrium.

Dynamic relaxation in this thesis

There are many types on dynamic relaxation schemes using different input and approximations in order to optimize convergence to the specific problem. The script in this thesis uses also a damping factor to improve the stability and fast convergence of the static solution. First the typology is set up. It is important to know which cables are hangers, which nodes are locked and which cables are connected to which node.

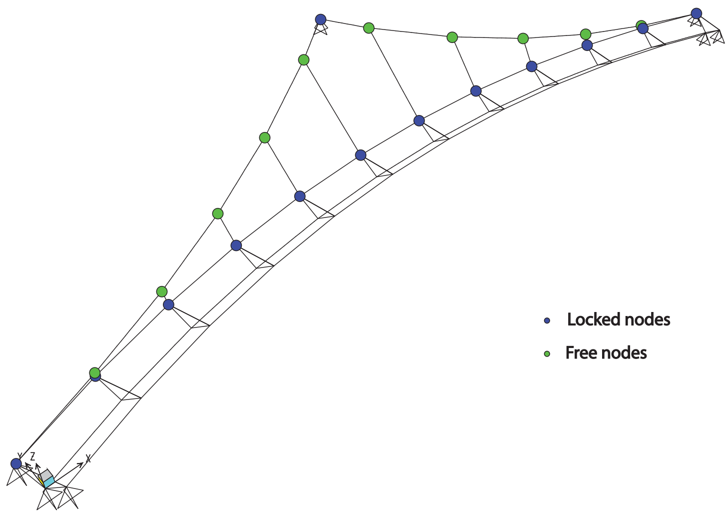


Figure 4.4: Typology for the dynamic relaxation

After the typology is set up the iteration scheme is:
1, Set all force vectors to zero vectors
2, Calculate the cable forces
3, Calculate the node force 4, Calculate the node acceleration
5, Calculate the new node position

In the second step, calculating the cable force by use of stiffness (EA) and slack length (sl) or the resultant with vertical component equals the prescribed vertical load.

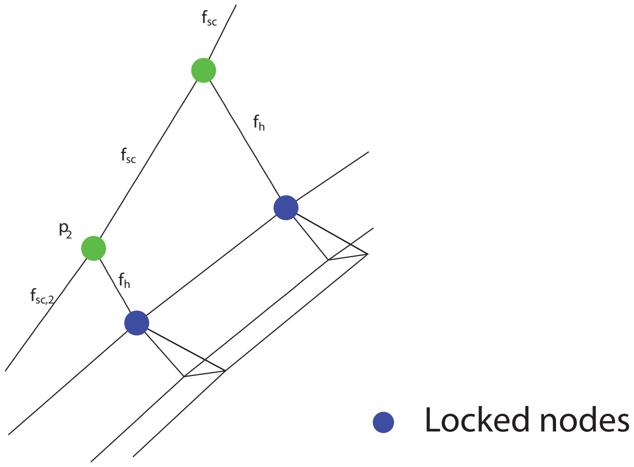


Figure 4.5: Nodes and nodal forces

$$\overrightarrow{f_{sc}} = \overrightarrow{ab} * sl * EA \tag{4.6}$$

$$\overrightarrow{f_h} = \overrightarrow{ab} * \frac{\overrightarrow{V_{load}}}{\overrightarrow{ab}_z} \tag{4.7}$$

In the third step, after the cable forces been calculated the resultant node force, $\overrightarrow{f_n}$ is calculated using the typology and cable forces.

In the fourth step the node acceleration is calculated using the nodal mass and nodal force.

$$\overrightarrow{a_i} = \frac{\overrightarrow{f_n}}{m_n} \tag{4.8}$$

When the acceleration is know, the new position can be calculated using the nodal velocity vector from the iteration before, $\overrightarrow{v_n^{i-1}}$ and the prescribed time step **dt** to calculate the new nodal velocity vector, $\overrightarrow{v_n}$.

$$\overrightarrow{v_n} = \overrightarrow{v_n^{i-1}} * \mathbf{c} + \overrightarrow{a_n} * \mathbf{dt} \tag{4.9}$$

With the new nodal velocity vector know the new position can be calculated for the nodes that are not locked.

$$p_n^{new} = p_n^{old} \overrightarrow{v_n} * \delta t \tag{4.10}$$

4.3.2 Dynamic relaxation through grasshopper

When implementing dynamic relaxation scheme in Grasshopper it i possible to write a custom component which can be programmed in C#, JAVA or Phyton. In this thesis it is written in C#.

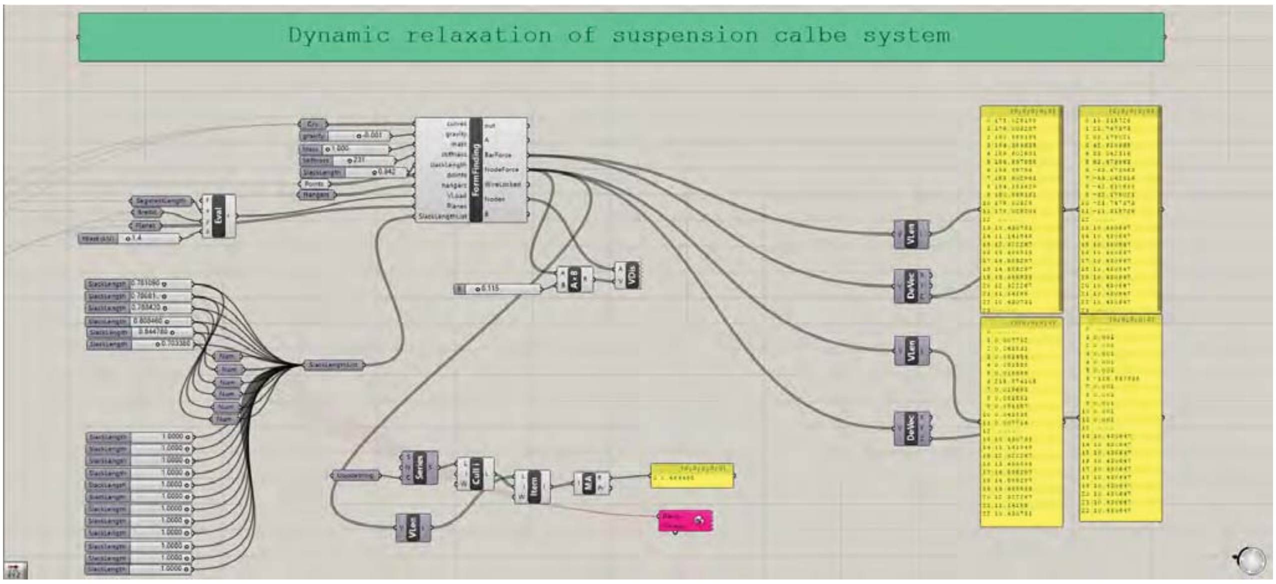


Figure 4.6: Dynamic relaxation scheme in Grasshopper

The input to the component is the curves representing the cables, an input for gravity, the fictitious value for the nodal mass, a value for the stiffness of the cables which influences the force in the suspension cable, which nodes are locked and which cables representing the hanger cables.

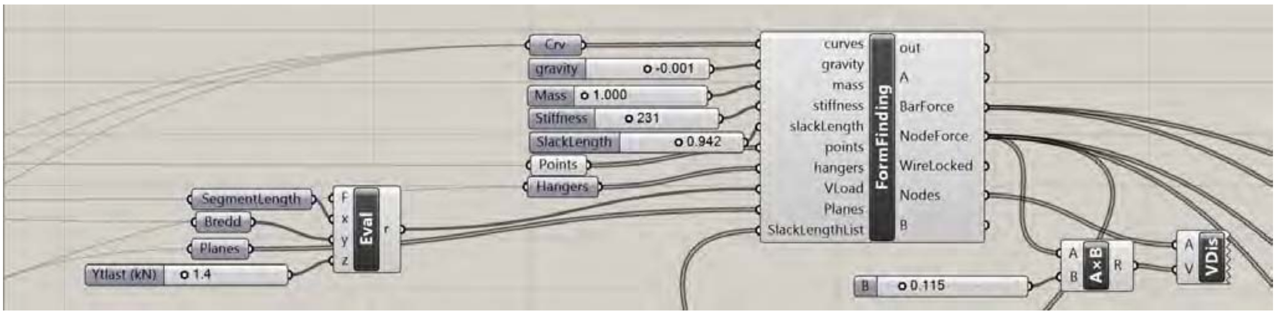


Figure 4.7: Form finding in Grasshopper

The shape of the suspension cable in this thesis will be mostly depending on the forces coming from the hanger cables. The gravity is therefore set to a small number, 0,001 m/s^2. The stiffness input is controlling the sag/force in the suspension cable. The value is like the mass a fictitious value used to find a static solution. The sag of the cable can also be controlled by the input slack length. It decides the length of the cable as a factor times the original length of the curve.
As mentioned in the section of curved bridges in the Chapter 3, the hanger cable is supposed to be perpendicular to the curvature of the bridge. If the initial length of the cables is defined and controlled by

the same slack length and a straight line from the support to the pylon and divided by the cross section planes of the bridge deck the hanger will not be perpendicular to the curvature. The movement of the nodes between the hanger and suspension cable can be restrained to only move in the cross section plane, but that would not converge in a static solution. The length of each cable segment needs therefore to be individually modified. By setting one slack length value per each cable segment they can be adjusted. It would be a practically impossible task to manually change each cable segment so that the initial length is correct. In Grasshopper it is possible to control the parameters and evaluate the result through a evolutionary solver called Galapagos. The solver analyses the results from many different settings and comes up with a configuration that get closer and closer to the correct shape.

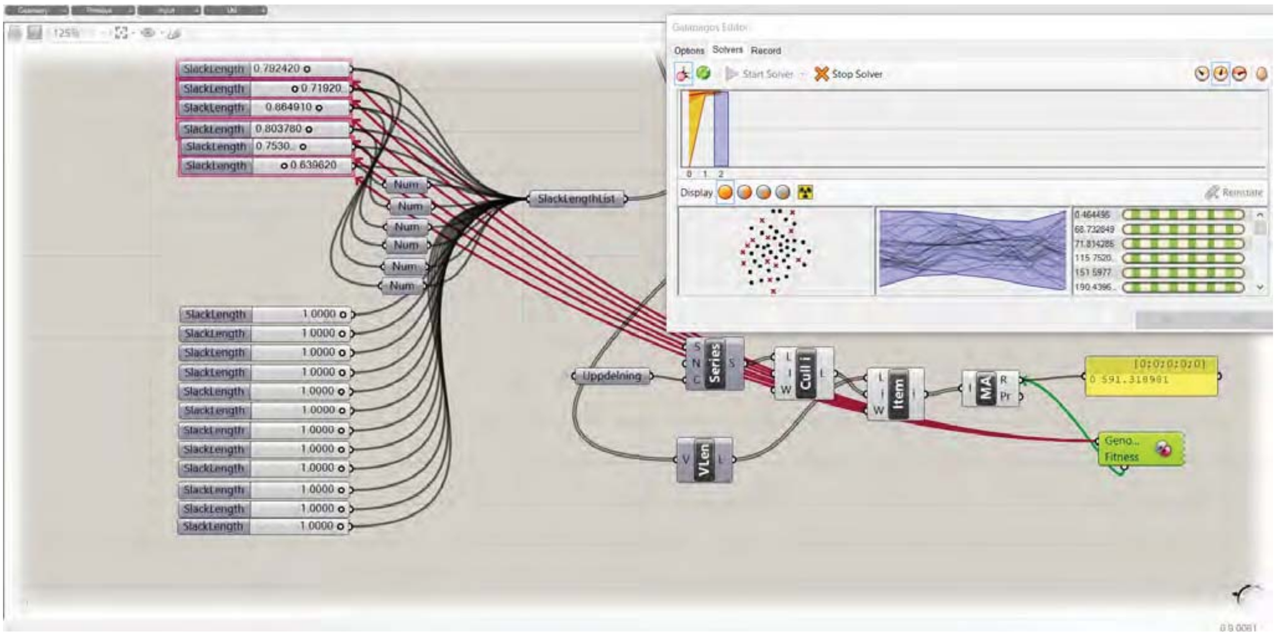


Figure 4.8: Modefication of slack length with help from an evolutionary solver

When the script has found a static solution each node should be in equilibrium. As output from the script it is possible to get the force vectors corresponding to each cable. The information can be used to decide weather or not the sag should be changed, curvature of the deck or position of the pylon.

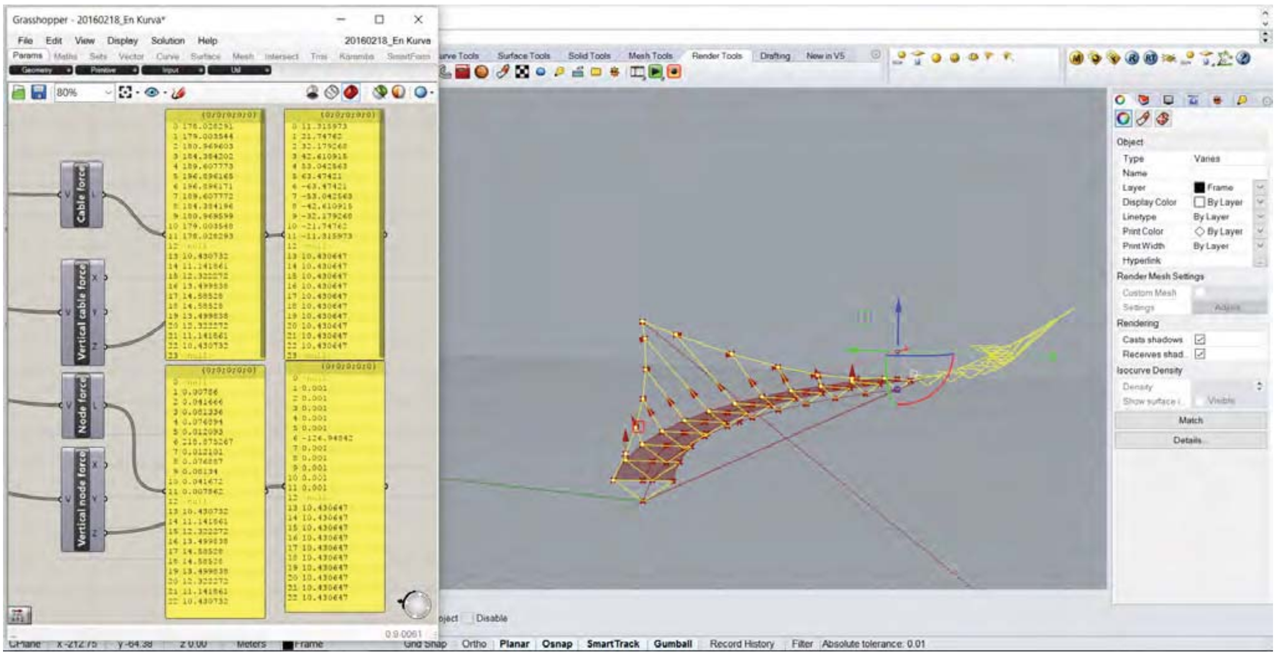


Figure 4.9: Output from Grasshopper and Rhnino 3D

Table 4.1: Forces in the hanger cables after form finding in Rhino3D/Grasshopper

Hanger	13	14	15	16	17	18	19	20	21	22
Force [kN]	10,11	10,74	11,96	13,10	14,10	14,10	13,10	11,96	10,74	10,11

Table 4.2: Forces in the segments of the suspension cable after form finding in Rhino3D/Grasshopper

Cable segment	1	2	3	4	5	6	7	8	9	10	11	12
Force [kN]	143,7	144,4	146,2	149,6	154,9	162,2	162,2	154,9	149,6	146,2	144,4	143,7

4.4 Finite elements modelling

SAP2000 is one of the leading finite element modelling software in the world. It is able to do analysis and design of simple cases like simple supported beams to complicated 3D non-linear models like suspension bridges. It is also easy to communicate and transfer geometry from Rhino3D.

In SAP2000 one can model beams, cables, shell elements and has been used to further analysis in this thesis.

5 Analysis and results

The analysis in this chapter is performed in order to understand how a curved suspension bridge works. A combination of simple hand calculations and a procedure of building more and more complex models give a method of how to model and interpreted the results.

5.1 Hand calculations

Simple hand calculations can be good to get some grasp of how the bridge should be modelled and help verify the results.

5.1.1 One bridge segment

To be able to first understand the basics about how the stresses should be distributed in the bridge deck we look at one bridge segment. The segment has a length of 3,72 meter, a width of 2 meter and a thickness of 0,215 meter. The segment is supported on two beams. The timber deck i assumed to have a specific weight of, $\gamma_{bs} = 4,7\frac{kN}{m^3}$
.

The bridge segment can be considered as a beam fixed in both ends. To estimate the size of the forces and bending moments from one part of the bridge. The result can be used to verify the results when looking at the whole bridge.

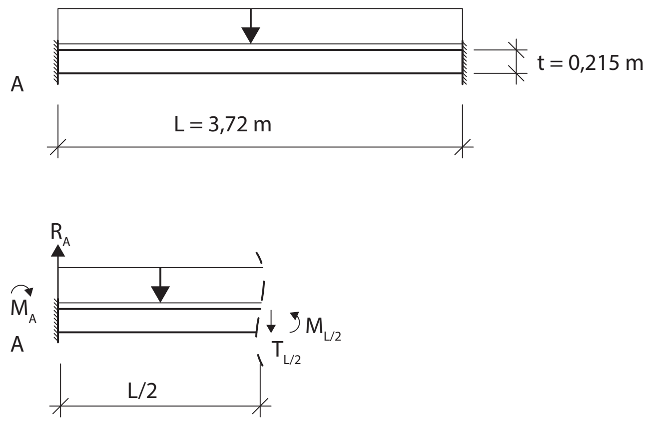


Figure 5.1: Fixed beam on both sides representing one bridge segment

The volume and self weight of one bridge segment can be calculated as V_{bs} and G_{bs}:

$$V_{bs} = B * L * t = 2,0 * 3,72 * 0,215 = 1.6\,\mathrm{m}^3 \quad (5.1)$$

$$G_{bs} = V_{bs} * \gamma_{bs} = 1,6 * 4,7 = 7,5\text{kN} \tag{5.2}$$

$$R_A = R_B = \frac{G}{2} \Rightarrow \tag{5.3}$$

$$M_A : -\frac{G * L}{12} = -\frac{7,5 * 3,72}{12} = -2,35kNm \tag{5.4}$$

$$M_{L/2} = \frac{G * L}{24} = \frac{7,5 * 3,72}{24} = 1,16kNm \tag{5.5}$$

Tension in the horizontal cable under dead load

Using the Boiler formula refereed to in chapter 3 the tension in the horizontal cable can be estimated. The radius of one curve is 54 meter. Calculating the radial force effecting the horizontal cable from one bridge segment with a length of 3,72 meter. The dead load is the weight of the deck + the weight from the timber struts, the horizontal cable, flooring and connections. The self weight of on segment was previously calculated to 7,5 kN and approximating the other loads to 2,5 kN gives a total dead load of 10 kN.

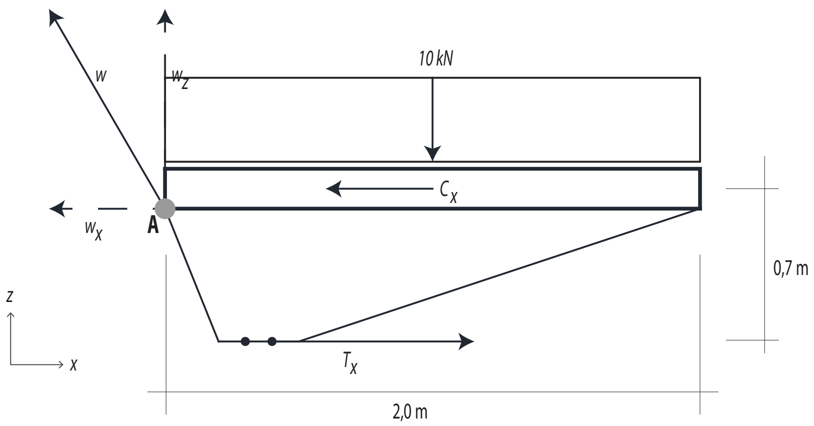


Figure 5.2: Equilibrium of forces for one segment

$$M_A : 10 * \frac{2}{2} - T_x * 0,7 = 0 \tag{5.6}$$

$$T_x = \frac{10 * 2}{2 * 0,7} = 14,3kN \tag{5.7}$$

Each radial force T_x over the length of the bridge can be approximated to a distributed load along the length.

$$p \approx \frac{T_x}{3,72} = 3,84kN/m \tag{5.8}$$

Using the boiler formula then give the pre-stress needed to stabilize the bridge deck under dead load.

$$T = p * r = 3,84 * 54 = 207,4kN \tag{5.9}$$

5.2 Pros and cons of using beam or shell elements.

To decide which method to model the bridge deck one needs to consider the type of geometry and forces acting on it. There are pros and cons with modelling by means of beam or shell elements. Modelling by means of beam elements is a fast, easy and a safe way of interpreting the results. One con is that the deck in reality will have a behaviour more like a plate and complicated forces connected to it. On the other hand the results from shell elements can be difficult to interpret and there are mistakes to be made. When modelling area-elements in SAP2000 it is important that the local axes are in the direction of the element and not the global coordinates as is the default setting. The decision of what to use depends on the structure one is modelling. A curved suspension bridge is a complicated three dimensional structure and before deciding how to model it it could be smart to model one curve with both methods.

5.2.1 Modelling one bridge segment

Beginning with one bridge segment and comparing the results from hand calculations. The dead load of one bridge segment, G=7,5 kN.

$$R_A + R_B - G = 0 \tag{5.10}$$

$$R_A = R_B = \frac{G}{2} = 3,75kN \Rightarrow \tag{5.11}$$

$$M_{L/2} = G/2 * L/4 = 3,5\mathrm{kN\,m} \tag{5.12}$$

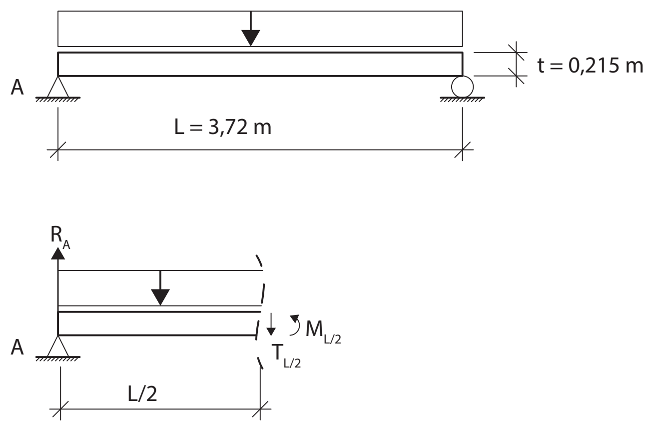


Figure 5.3: Simply supported beam representing one bridge segment

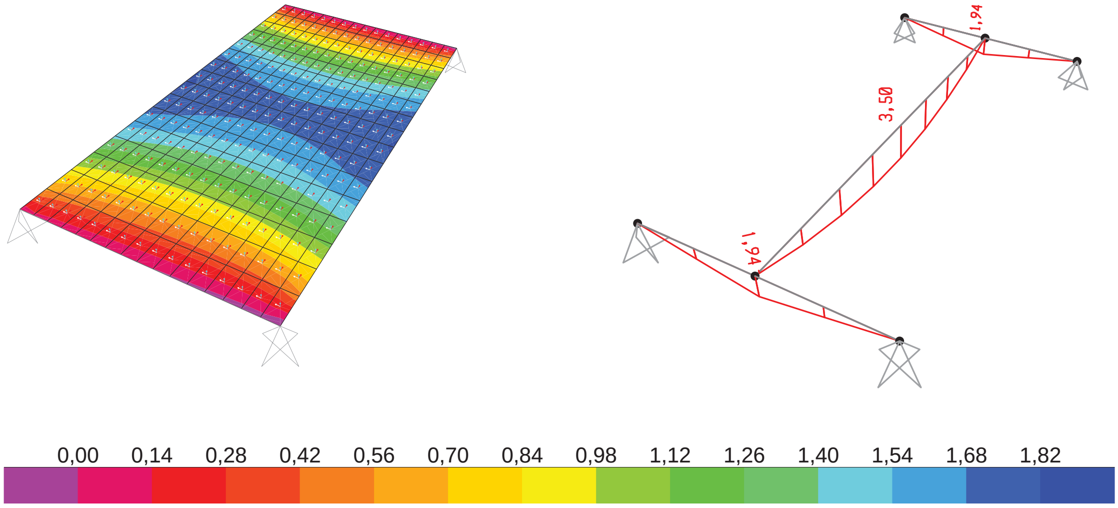


Figure 5.4: Moment distribution in one segment

Table 5.1: Results from one segment modelled with beam and shell elements

	Shell	Beam
Max moment	3,6 kNm	3,5 kNm
Reaction force in one support	2,0 kN	2,0 kN

The reaction force in one support is calculated as the self weight of the bridge deck and the supporting beam hence the difference in result from the hand calculations.
When modelling with shell elements it is important to note that the resultant forces is presented per unit length. Moment is presented as kNm/m as an example. Important to note when modelling with shell elements is the rotation of the local axis. If the element is not in the direction of the global coordinate system they need to be rotated to produce a result that is easy to interpret.

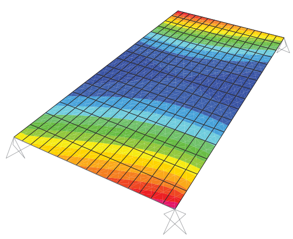


Figure 5.5: M11 modelled with shell elements and default local axis

5.2.2 Analysis of a one curved bridge

A double curved suspension bridge is a complicated three dimensional structure and before deciding how to model it it could be smart to model one half of the bridge by means of both beam and shell elements. The models are only modelled with the self weight of the bridge as load case since that is what the shape should be optimised for.

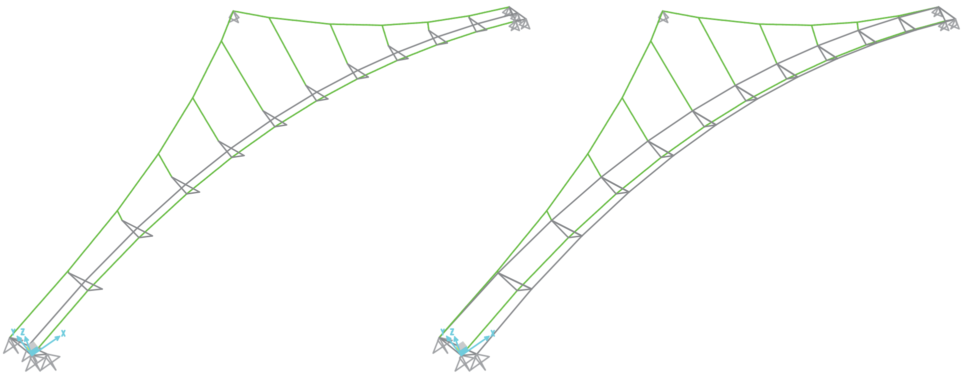


Figure 5.6: One curve modelled with beam and shell elements

One curve without suspension cables

Comparing the beam and shell models they are first modelled without the suspension cable and hangers. Instead the deck is connected to rolling supports at each connection point.

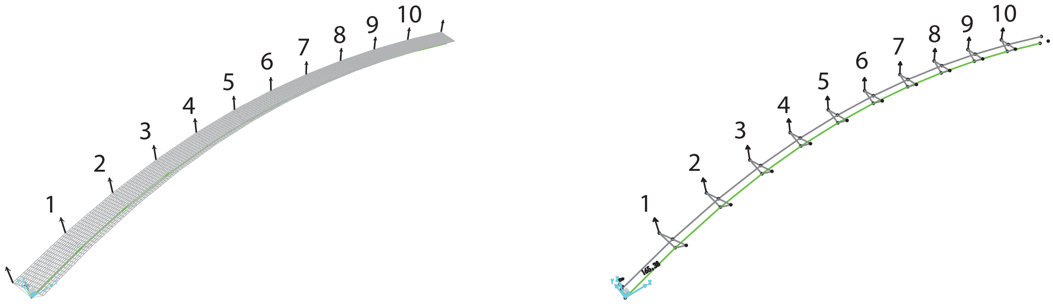


Figure 5.7: Joint reaction forces

Table 5.2: Joint reaction forces

Joint	1	2	3	4	5	6	7	8	9	10
Beam model, reaction force (kN)	8,90	9,44	9,65	9,69	9,68	9,68	9,69	9,65	9,44	8,90
Shell model, reaction force (kN)	10,26	9,39	9,53	9,51	9,51	9,51	9,51	9,53	9,39	10,26

The moment distribution along the one curved bridge show different behaviour for the beam and shell model. The difference is quite significant and comparing to the result from the hand calculations of one segment with a maximum moment of 2,35 kNm and minimum of -1,15 kNm. Converted to kNm/m gives a maximum of 1,175 and minimum of -0,575.

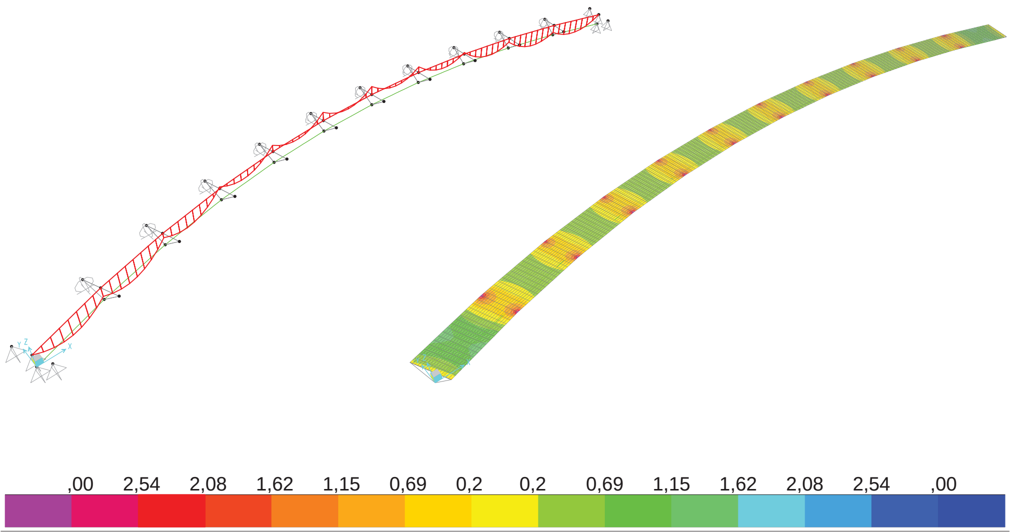


Figure 5.8: M11, Shell model on rolling supports

Table 5.3: Max and min moment

Location	Max	Min
Center line	0,51 kNm/m	-0,54 kNm/m
Edge line	0,66 kNm/m	-2,8 kNm/m
Beam model	2,13 kNm/m	-1,35 kNm/m

5.2.3 One curved bridge with suspension cables

The suspension system is influencing the decision of how the deck should be modelled. The complete bridge is a more complicated system compared to when modelling just with rolling supports. The stiffness of the deck and the connections to the timber struts decides how forces from the deck distributes to the hangers.
The axial forces in the cables, timber struts and timber deck is showed in Table 5.4.

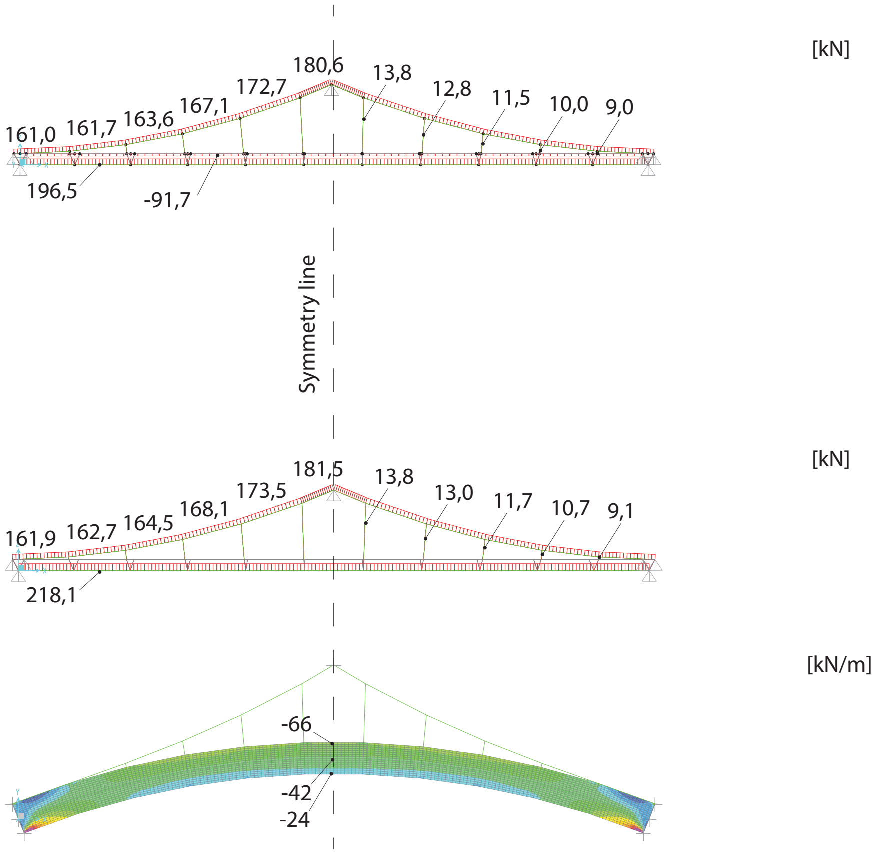


Figure 5.9: Axial forces in beam and shell model

Table 5.4: Comparison of one curved models.

	Beam model	Shell model
Hanger	13,8 kN	13,8 kN
Horizontal cable	196 kN	218 kN
Suspension cable	181 kN	182 kN
Bridge deck, (middle of the deck)	92 kN	90 kN
Compression strut	4,2 kN	7,7 kN
Tension strut	12,3 kN	10,6 kN

The bending moment in the longitudinal direction of the bridge are different in the two models. Both models have the same suspension system and horizontal cable with the same initial tension. The largest positive moment is for the beam model whilst the largest negative moment is for the shell model. The behaviour is the same for both model and small changes have rather large influence on the bending moment.

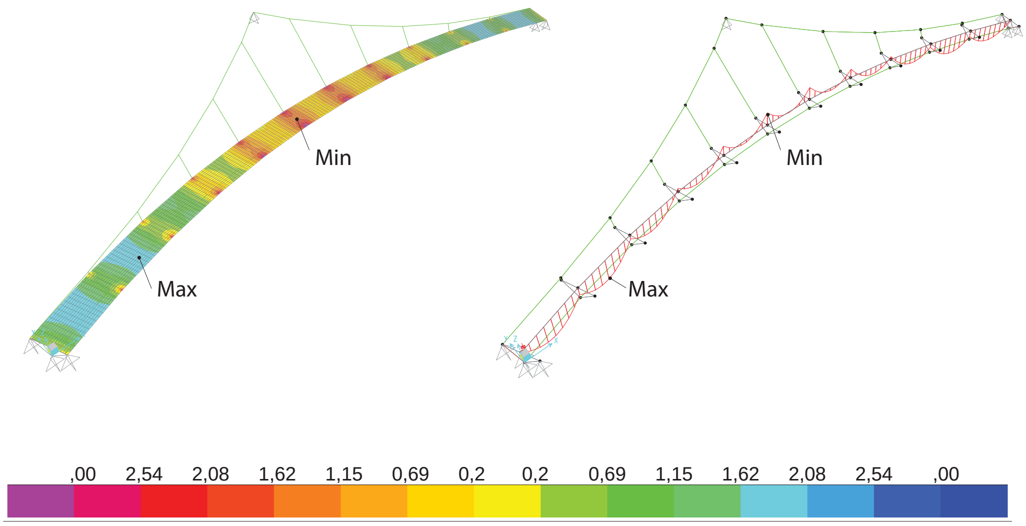


Figure 5.10: Moment distribution in beam and shell model

Table 5.5: Max and min moment

	Max	Min
Shell (center line)	3,6 kNm	-3,86 kNm
Beam	5,2 kNm	-2,4 kNm

The deck in a curved suspension bridge will be subjected to radial forces as a result of the eccentric suspension. The radial forces results in a moment acting in the plane of the deck. The moment distribution depends on the boundary conditions in the ends of the beam model, if it is fixed or pinned. To compare with the beam model and closest to reality is when the deck is fixed to rotate in the plane of the deck.

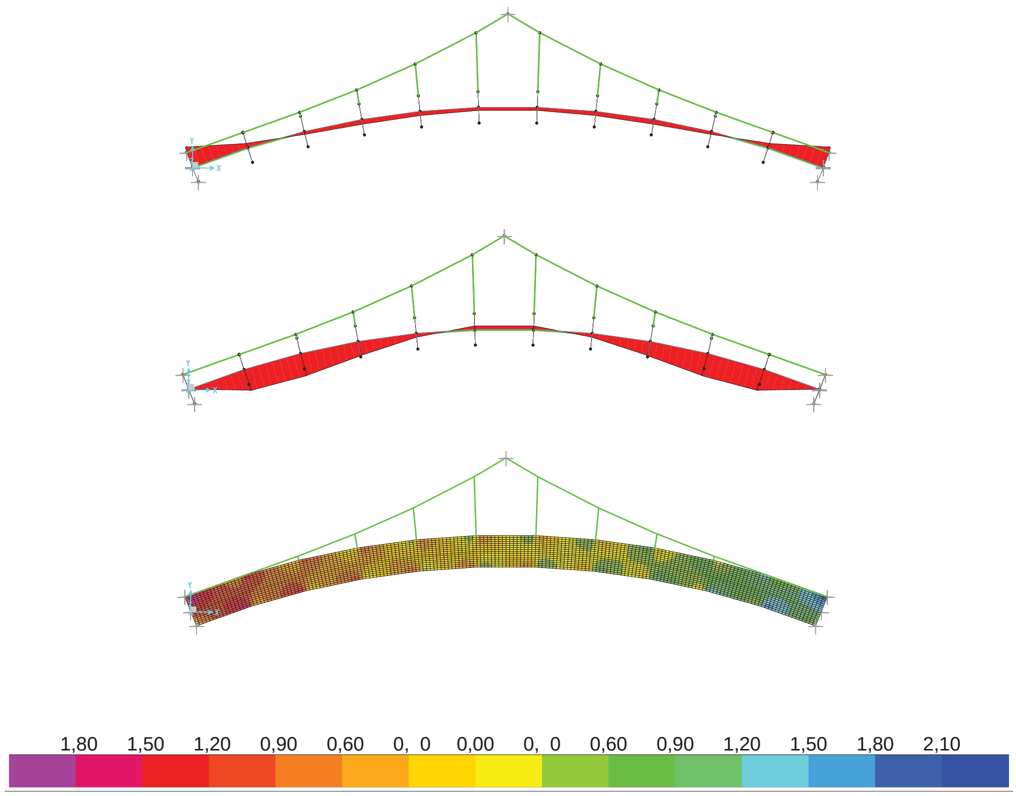


Figure 5.11: M2-2

Table 5.6: Max and min moment

Location	Max	Min
Center line	2,09 kNm/m	-1,97 kNm/m
Edge line	2,25 kNm/m	-3,7 kNm/m

5.2.4 Conclusions for further analysis

The beam and shell model are modelled in the same manner except for the bridge deck. They have the same total mass and show much of the same behaviour. The forces in suspension system and anchor point is more or less the same. The axial compression in the bridge deck is the same if one look in the center line of the deck. In the shell model the axial compression span from 24-66 kN/m, which is not visible in the beam model. Recalculated from the kN/m to kN the axial force in the center line is more or less the same in the two models. The largest difference between the the two models are the forces in the horizontal cable and the timber struts. The difference is due to the different connection to the bridge deck. Since a double curved suspension bridge is a complicated three dimensional and the the behaviour of the deck is more like a plate the conclusion is to continue modelling with shell elements.

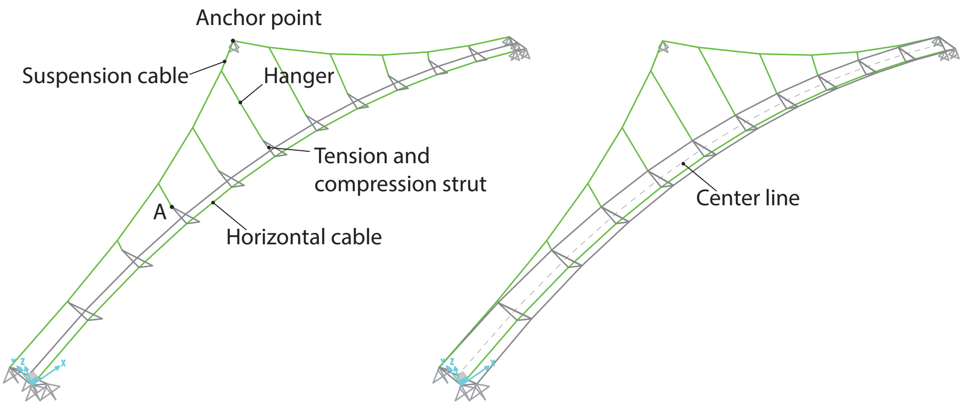


Figure 5.12: Points for comparison.

Table 5.7: Comparison of one curved models.

	Beam model	Shell model	Ratio
Ancorage point, vertical	126 kN	127 kN	0,99
Ancorage point, horizontal	164 kN	165 kN	0,99
Hanger	13,8 kN	13,8 kN	1,0
Horizontal cable	196 kN	218 kN	0,90
Suspension cable	181 kN	182 kN	0,99
Bridge deck, (middle of the deck)	92 kN	90 kN	1,02
Compression strut	4,2 kN	7,7 kN	0,55
Tension strut	12,3 kN	10,6 kN	1,16
Deflection (U3) in point A	-0,5 mm	-0,2 mm	2,5
Mass	12718 kg	12718 kg	1,0

5.3 Tuning the cable system and tensioning the cables

When the geometry is imported to SAP2000 from Rhino3D the cables are then modelled. Modelling cables with SAP2000 means that you specify the tension by specifying the deformed length, tension in either end, or sag.

5.3.1 What happens if the suspension cable has the wrong shape?

To illustrate the importance of the correct shape of the suspension cables three analysis have been performed. All three are only analysed using the self-weight of the bridge deck and members. The first analysis is one without any suspension system, the second with an erroneous shape and the third with the correct shape that converged in the form finding process. To compare the three analysis the force in hanger H1 and H2, deflection in A and B and the maximum and minimum moments M11 in the bridge are shown in table 5.7.

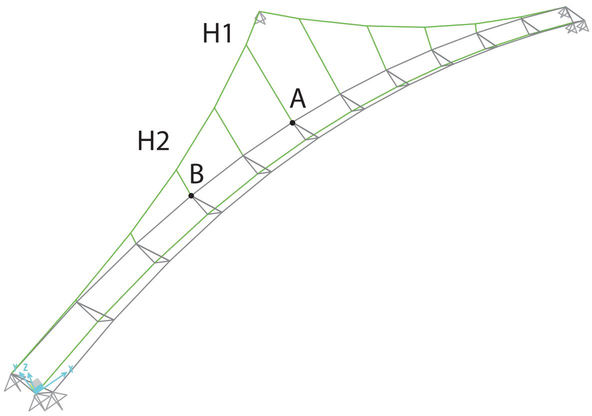


Figure 5.13: One curve modelled with shell elements, point and hanger for comparison

Table 5.8: Effects of bad suspension system

	H1 kN	H2 kN	Max M11 kNm/m	Min M11 kNm/m	$u3_A mm$	$u3_B mm$
No suspension	-	-	131	0	-335	-435
Wrong shape	25,3	5,8	9	-17	2,2	-1,9
Correct shape	13,9	11,8	2,1	-3,0	0,13	-0,29

The results show that the shape is crucial in order to have correct behaviour. The forces in the hangers as well as the moment distribution along the length of the bridge deviate greatly even if the deflection is relatively small. Even the model with the correct shape show a different result compared to the result from the analysis on rolling supports. That would indicate that the suspension system is complicated to model to achieve the exact correct behaviour. That is due to the elasticity of the cables and also the pre-tension of the system.

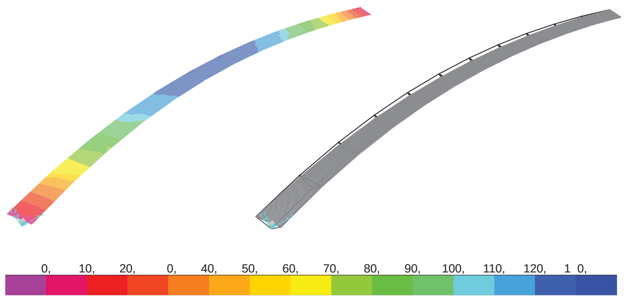


Figure 5.14: M11 without suspension system, left: longitudinal moment distribution, right: Deformation

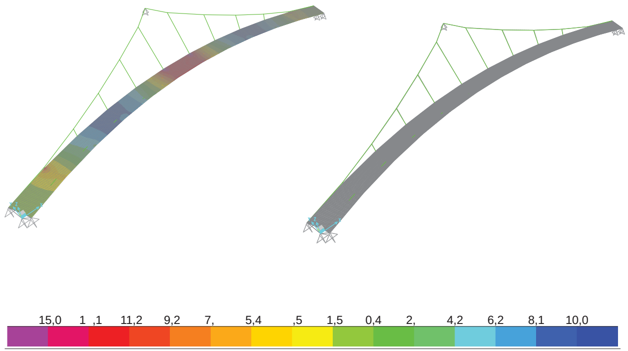


Figure 5.15: M11 with a suspension system with the wrong shape, left: longitudinal moment distribution, right: Deformation

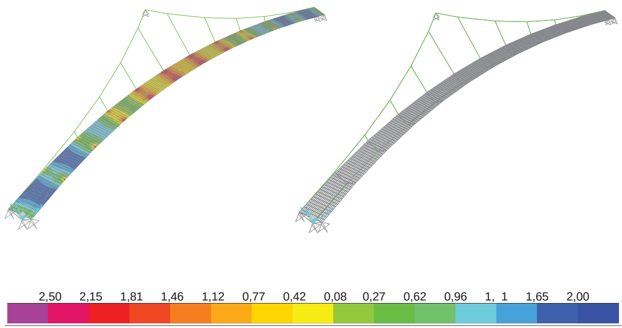


Figure 5.16: M11 with a suspension system with the correct shape, left: longitudinal moment distribution, right: Deformation

5.3.2 Effects of tensioning the horizontal cable

The horizontal cable runs under the bridge deck, stabilizes the eccentric suspension together with the curved deck. The cable can be tensioned to increase the moment from the eccentricity. In SAP200 you can introduce pretension to cables in different ways, one way is to specify the undeformed length. By specifying the undeformed length to 3,7244 m instead of 3,7245 m a pretension force of 175 kN is introduced. The effect is visible in Figure 5.17.

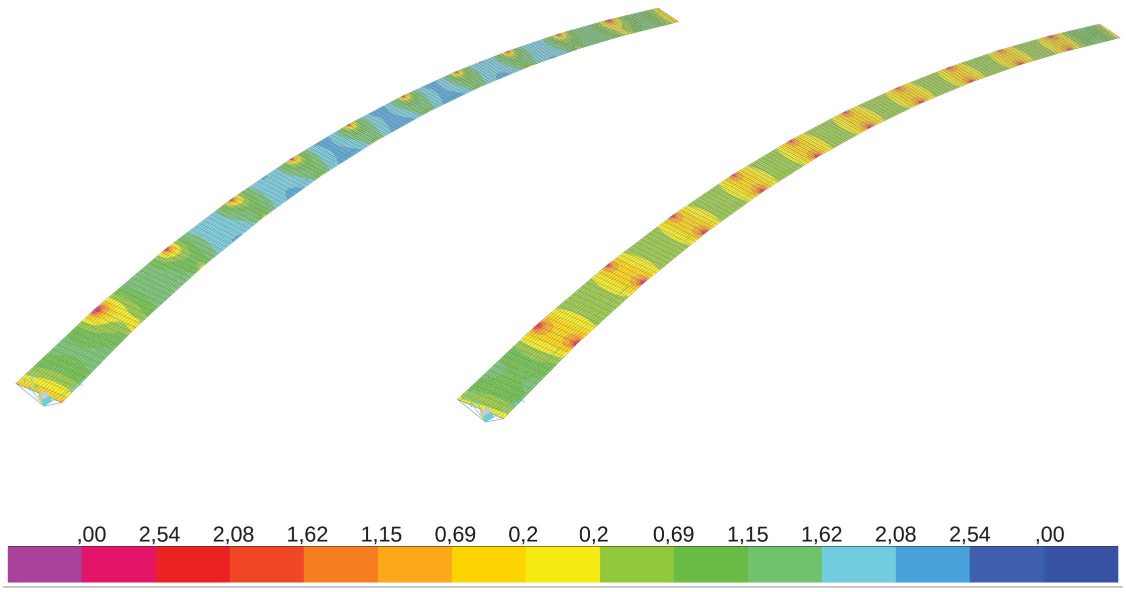


Figure 5.17: M11 with (right) and without (left) post tension in the horizontal cable

Table 5.9: Maximum and minimum longitudinal moment

	Max M11	Min M11
No extra post tension	2,3	-2,8
175 kN extra post tension	0,67	-2,8

5.3.3 Effects of tuning the suspension cable and hangers

The system for suspending the deck is more complicated then the one horizontal cable. The reason is that it is a system of cables that influence each other, when one is tensioned the stresses change in the others as well. One way to see how well the system is functioning is by looking at the longitudinal moment distribution M11 and the displacements in connection points. The

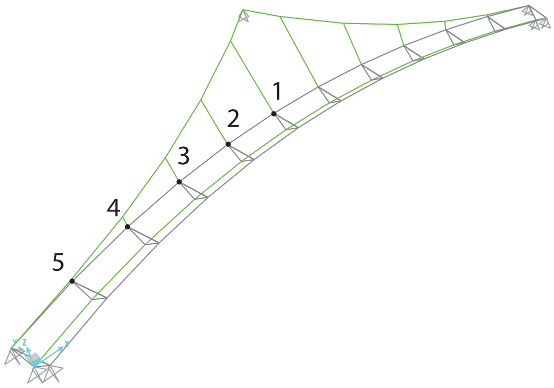


Figure 5.18: One curve with reference points

Table 5.10: Deflection in connection points

Point	Tuned system	Not tuned system
1	0,1 mm	1,2 mm
2	0 mm	0,9 mm
3	-0,3 mm	0,5 mm
4	-0,5 mm	0,1 mm
5	-0,4 mm	-0,1 mm

5.4 From one curve to two

The bridge from the thesis "A pilgrims walk" was a s-shaped bridge that were intended to work in a certain way. The suspension cable was intended to be a continuous cable running under the bridge to the other side. However when modelling the bridge and analysing it in SAP2000 it was shown to be difficult to configure the tension in the suspension cable and the horizontal cable since they were connected. In the earlier concepts he hangers were also connected to to the horizontal cable creating a complete system of cables. The benefit with that system was that the suspension cable was continuous and not anchored in the bridge deck. That system had a simplicity to it but were proved to be difficult to model correct in SAP2000. However, it could be interesting for further work and with more knowledge of SAP200 to investigate ways of modelling that type of bridge.

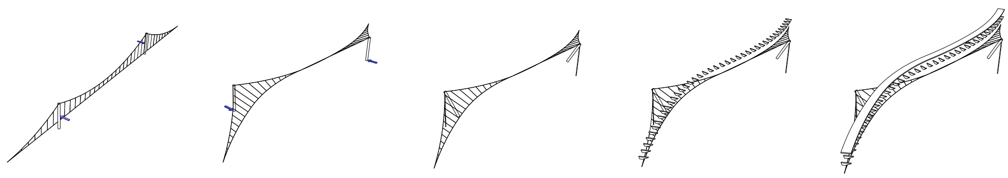


Figure 5.19: Construction principle from master thesis "A pilgrims walk"

Regardless of if the suspension cable is anchored in the bridge deck or continuous under the deck, the angle between the cable and the horizontal plane need to be right. That is because if the angle is to large, the vertical component of the force in the suspension cable will lift the middle section of the bridge if the force is larger than the weight of the corresponding bridge segment. That will create a bending moment in the bridge deck and also slack the hangers a little since the model in Rhino3D and Grasshopper was configured so that there were supposed to be no deflections in the connecting joints.

An effect of having the suspension cables anchored in the bridge is that the force needs to be handled in the bridge deck. The best way for the bridge deck to handle the force is if the cable anchors tangential to the curvature of the deck. In that way the width of the deck handles the loads in bending.

5.5 Final FE-model

The final model of the double curved bridge consists of cross sections located 3,72 meters apart. The total length of the bridge is 82 meter and curvature lies in the horizontal plane.

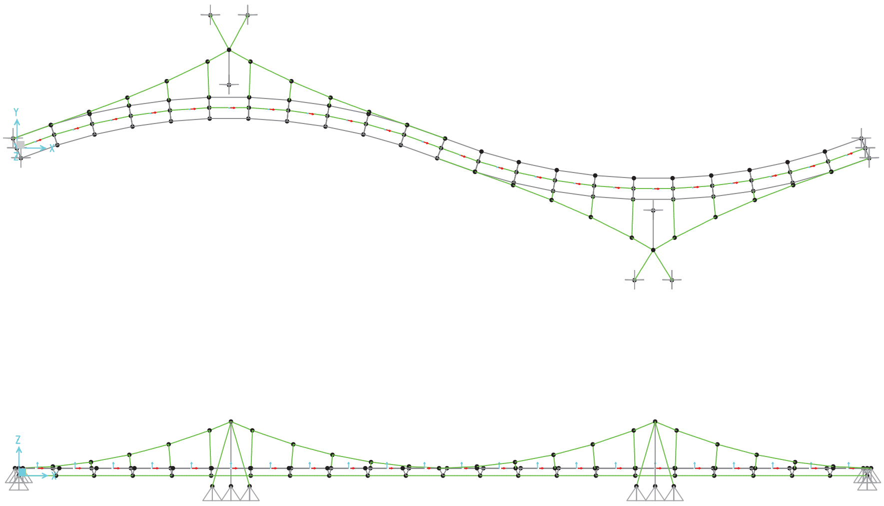


Figure 5.20: Static system of the final model

To be able to make sense of the results from the analysis the local axes needs to be following the curvature of the deck. Timber is a orthotropic material so the stresses in the right directions are what is interesting. As shown in Figure 5.20, the local axis 1 is running parallel with the deck, axis 2 is perpendicular and axis 3 is the the same as global z axis.

5.5.1 Mesh size of shell elements

The size of the mesh is important when modelling with shell elements. They should be so small that the results is good enough but as large as possible to not demand to much processing power from the computer. In SAP2000 it is possible to decide which resolution the mesh should have by specifying it in "auto mesh". In the model each segment is modelled as a shell element and it is possible to decide how many elements there should be along each edge. After comparing results from meshes division of each bridge segment in 8*8, 16*16, 32*32 and 64*64 elements the conclusion is that 16*16 seems to have enough accuracy.

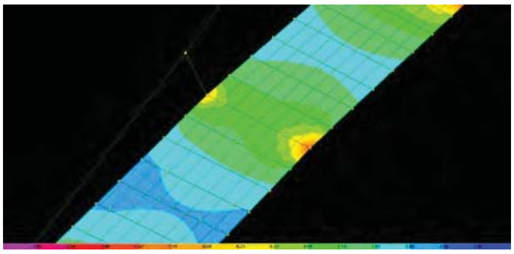


Figure 5.21: Mesh with each bridge segment divided in 8*8 elements

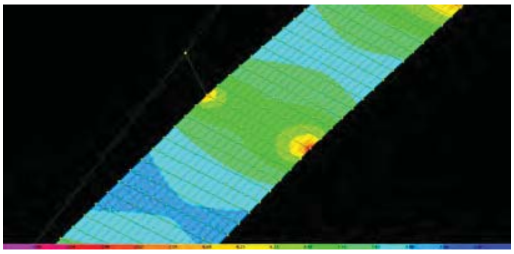


Figure 5.22: Mesh with each bridge segment divided in 16*16 elements

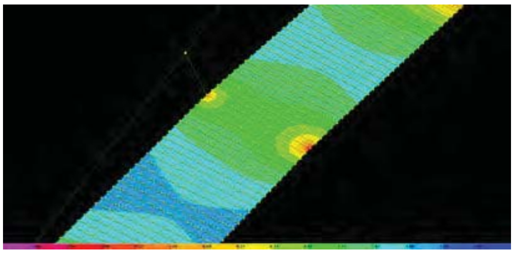


Figure 5.23: Mesh with each bridge segment divided in 32*32 elements

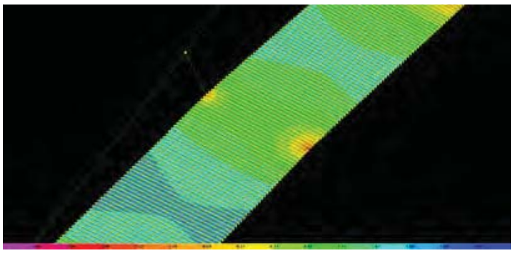


Figure 5.24: Mesh with each bridge segment divided in 64*64 elements

5.5.2 Material properties

The final model of the double curved bridge consists of two types of steel cables, timber struts, timber deck, steel beams and steel masts. The model is used to get the section forces for cables, beams, struts and masts. The section forces and stresses in the bridge deck are used for the preliminary structural design in the next chapter.

Material properties used in the model are shown in Table 5.10. The material properties for structural timber and glulam are the same in the model, since the aim was to perform a preliminary structural design it was considered to be sufficient.

Table 5.11: Material properties

	Dimension	p (kg/m^3)	E(GPa)	v
Suspension and horizontal cable	80 mm	7850	210	0,3
Hanger cable	40 mm	7850	210	0,3
Glulam	2000 * 215 mm	470	13	0,4
Structural timber	150 * 150 mm	470	13	0,4
Steel		7850	210	0,3

5.5.3 Loads and load combinations

The loads can be divided in permanent and variable. Loads that are considered as permanent loads are self weight and other loads that will be present during the whole lifespan of the structure, such as railing and flooring. Variable loads are loads that during a limited time span affects the structure. They are divided in short, medium and long term actions. Depending on which load duration class different safety factors are used.

According to Eurocode 5, loads should be divided in favourable and unfavourable loads.

The dead load is considered to be a permanent load and is calculated through using SAP2000 with the self weight of the bridge elements in the model plus added mass for the flooring and railing. According to Eurocode the EN1991-2 actions on footbridges the loads acting on a footbridge shall be defined as,

$$2,5 < q_k < 2,0 * 120/(L + 30) \leq 5,0kN \tag{5.13}$$

Where:

L, *is the length of the bridge*
q_k,, *is the distributed vertical live load*

5.5.4 Load cases

The load cases chosen in this thesis is intended to give a picture of how the bridge behaves in different situations. The suspension system is designed for when the bridge is under dead load which is a more or less evenly distributed load. The same is for when the bridge is fully loaded which could imply that it should work well for that case. Uneven loads are of interest to analyse to see how the structure behaves in other cases. A group of pedestrians walking over the bridge is an example of an unevenly distributed load.

Table 5.12: Load cases

Loadcase	Type	Placement	Magnitude
1	Dead	-	Obtained from SAP2000
2	Live	Whole span	3 kN/m2
3	Live	Half span	3 kN/m2
4	Live	Group at 1/4 of the span	3 kN/m2
5	Live	Group in the middle of the span	3 kN/m2

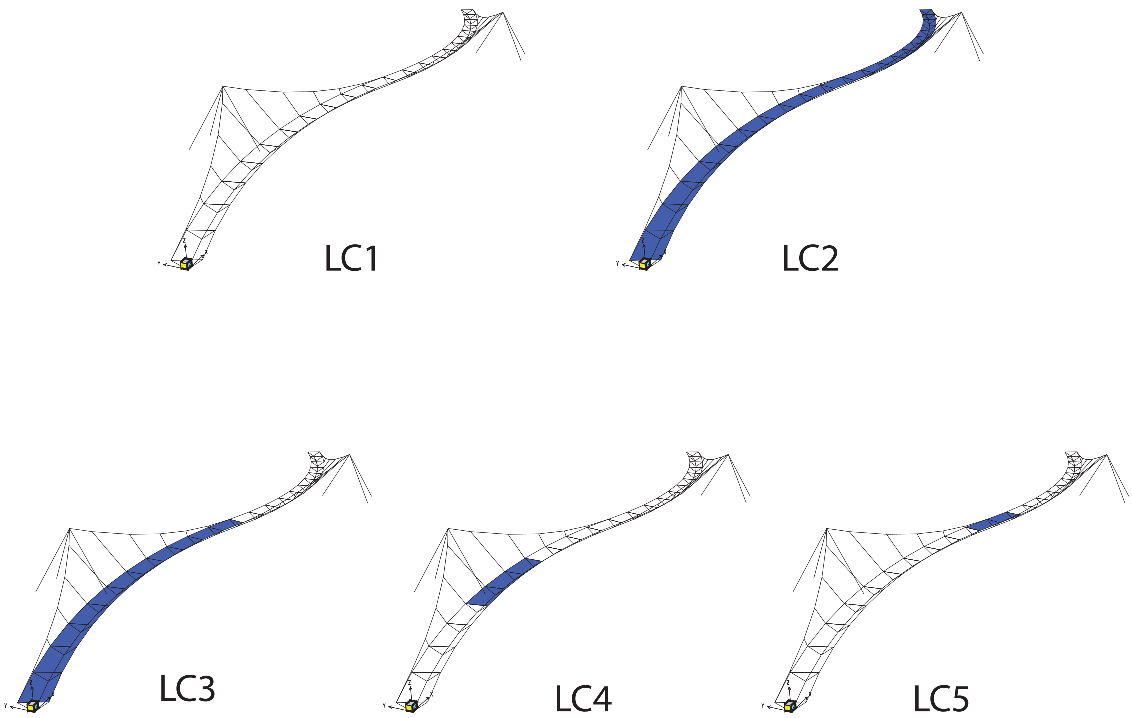


Figure 5.25: Load cases

5.6 Results from SAP2000

In the section below the results from the static and dynamic analysis is presented in short. For more detailed information see Appendix.

5.6.1 Static results

The results from the static analysis are used to understand the behaviour of the bridge and later for the load bearing design.

Timber struts

The timber struts connect to the horizontal cable and the bridge deck are some of the most complicated parts on this bridge. They are connected using steel plates and are subjected to different types of forces. In Table 5.20, are the forces acting on the struts during the different load cases presented. P is the axial force, V2 is the shear in the cross section of the bridge, V3 is the shear in parallel to the curvature of the bridge, T is the torsion, M2 is moment parallel to the curvature and M3 in moment in the cross section. The forces acting parallel to the curvature depends on how the horizontal cable is connected to the struts. Table 5.12 shows the results from a FE-model with fixed connections between the struts, deck and horizontal cable. A timber-metal connection is far from fixed. The dowel is fitted in a hole that are a little bit larger than the dowel and the timber deforms a little when loaded. That would imply that the connection is more like a hinge than a fixed connection. In table 5.13 the results most loaded strut is shown with a hinged connection.

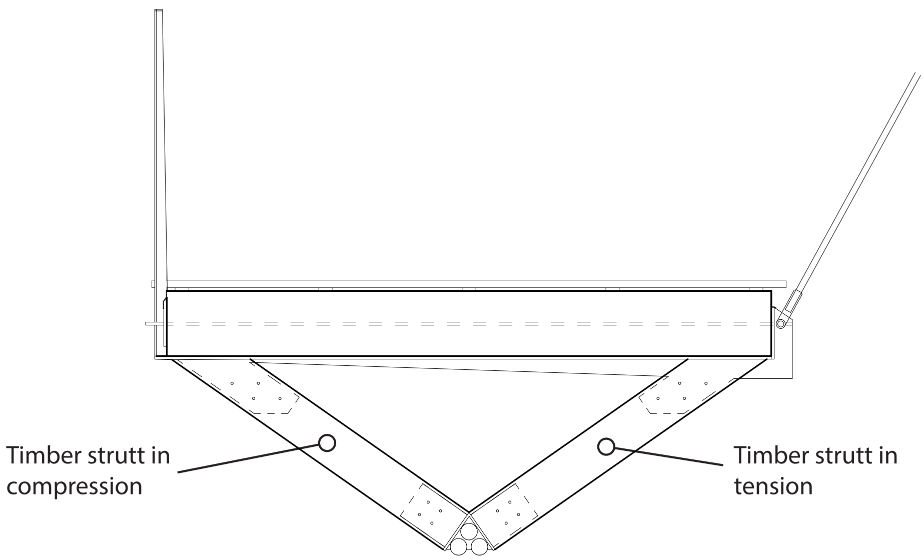


Figure 5.26: Timber struts in tension and compression

Table 5.13: Maximum and minimum results for the most loaded timber struts, modelled with fixed connections

Loadcase	P (kN)	V2 (kN)	V3 (kN)	T(kNm)	M2 (kNm)	M3 (kNm)
1 Max	10,6	0,3	1,2	0,5	0,8	0,2
1 Min	-7,5	-0,02	-1,26	-0,5	-0,9	-0,1
2 Max	36,4	1,3	7,9	2,7	6,1	1,0
2 Min	-30,9	0,1	-7,42	-3,2	-5,3	-0,5
3 Max	36,3	1,8	31,6	10,1	24,3	1,4
3 Min	-34,6	-1,0	-28,2	-9,56	-20,5	-0,9
4 Max	26,8	2,1	27,5	9,0	20,6	1,7
4 Min	-27,9	-2,4	-27,3	-9,4	-20,0	-2,0
5 Max	28,6	1,7	21,8	7,2	16,3	1,3
5 Min	-29,6	-2,15	-21,5	-7,4	-15,8	-1,8

The forces acting in the cross section is mostly dominated by the axial force. As described in the section of statics of curved suspension bridges one strut is in tension and the other is in compression. Since the connection of a wood-metal connection with dowels is not a fixed connection and that the connection between the horizontal cable and the struts is not fixed the results from the model with hinged connections is used.

Table 5.14: Maximum and minimum results for the most loaded timber struts, modelled with hinged connections

Loadcase	P (kN)
1 Max	10,4
1 Min	-7,6
2 Max	35,2
2 Min	-31,6
3 Max	31,2
3 Min	-27,5
4 Max	24,7
4 Min	-21,2
5 Max	28,8
5 Min	-25,3

Cables

In the final model there are three types of cables the suspension cable, hanger cables and the horizontal cable. For the suspension and horizontal cable load case 2 is the most demanding but for the hanger cables load case 3 is slightly more with 68,0 kN instead of 65,2 kN. The pre-tension in the cables are modelled so that they balance the dead load of the bridge.

Table 5.15: Maximum tension (kN) in cables

Loadcase	Hagers	Suspension cable	Horizontal cable
1	14,0	178,1	213,7
2	65,8	783,9	833,2
3	68,0	765,7	833,2
4	36,9	426,9	426,9
5	25,3	322,4	765,7

Mast

The two masts located on each side of the of the bridge are constructed of an inclined pylon and two back stays which stabilize the mast. The forces in the back stays differ from each other because they are placed symmetrical to the curvature of the deck. If the mast were to be rotated so that the pylon were to be inclined away from the difference in force magnitude would be smaller. However for a load case which is not uniformly distributed the forces will be different so the final position is based on architectural considerations.

Table 5.16: Maximum tension/compression (kN) in the masts

Loadcase	Pylon	Back stay 1	Back stay 2
1	-244,5	48,3	47,4
2	-1059,0	239,0	196,4
3	-990,2	339,2	102,7
4	-564,7	163,2	64,0
5	-431,9	65,2	107,1

Timber deck

The timber deck is the part of the bridge with most different types of forces acting on it. In this thesis the loads are from people walking over the bridge and the dead load of the bridge, which causes bending stress in the longitudinal direction of the bridge and shear forces as in a straight bridge. The deck is also subjected to the forces coming from the hanger cables which are inclined and with components in the vertical and horizontal plane. The eccentric suspension causes radial forces in the plane of the bridge which results in compression in the deck. The length and tension of the suspension cable is optimised for the dead load case. Figure 5.27 shows how compression and tension is distributed in the first three load cases. The figure shows that for the dead load case the stress is dominated by compression due to the eccentric suspension, with compression in both the top and bottom of the deck. One exception is where the suspension cables anchor to the deck on the two side causing tension in the middle of the bridge. In load case 2, when the bridge has a uniformly distributed load over the whole bridge, the magnitude of bending stress is larger causing tension in the bottom of the deck. In load case 3 only half of the bridge is loaded by pedestrians. It results in tension in the bottom and compression in the top of the loaded side of the deck. On the other side the only has dead load the result is opposite.

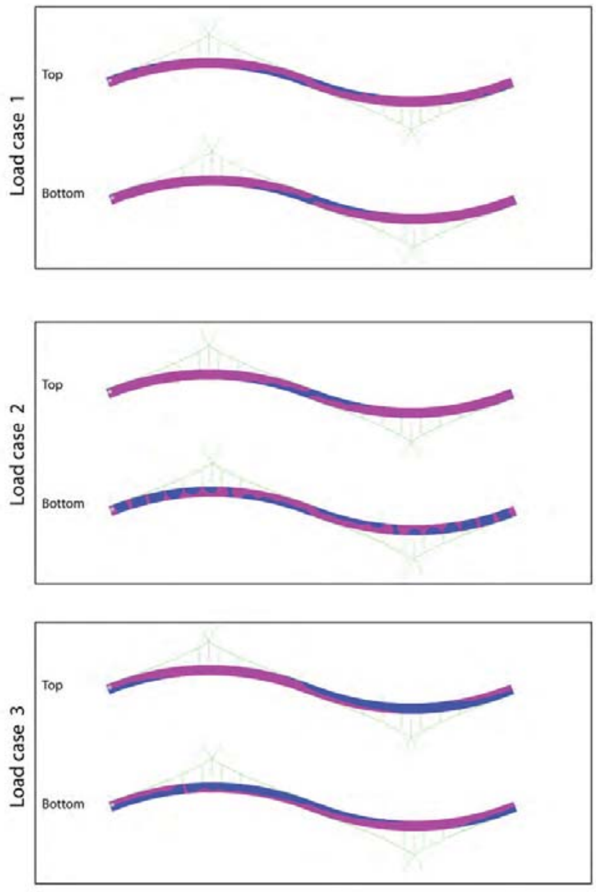


Figure 5.27: Compression (purple) and tension (blue) in LC1, LC2 and LC3

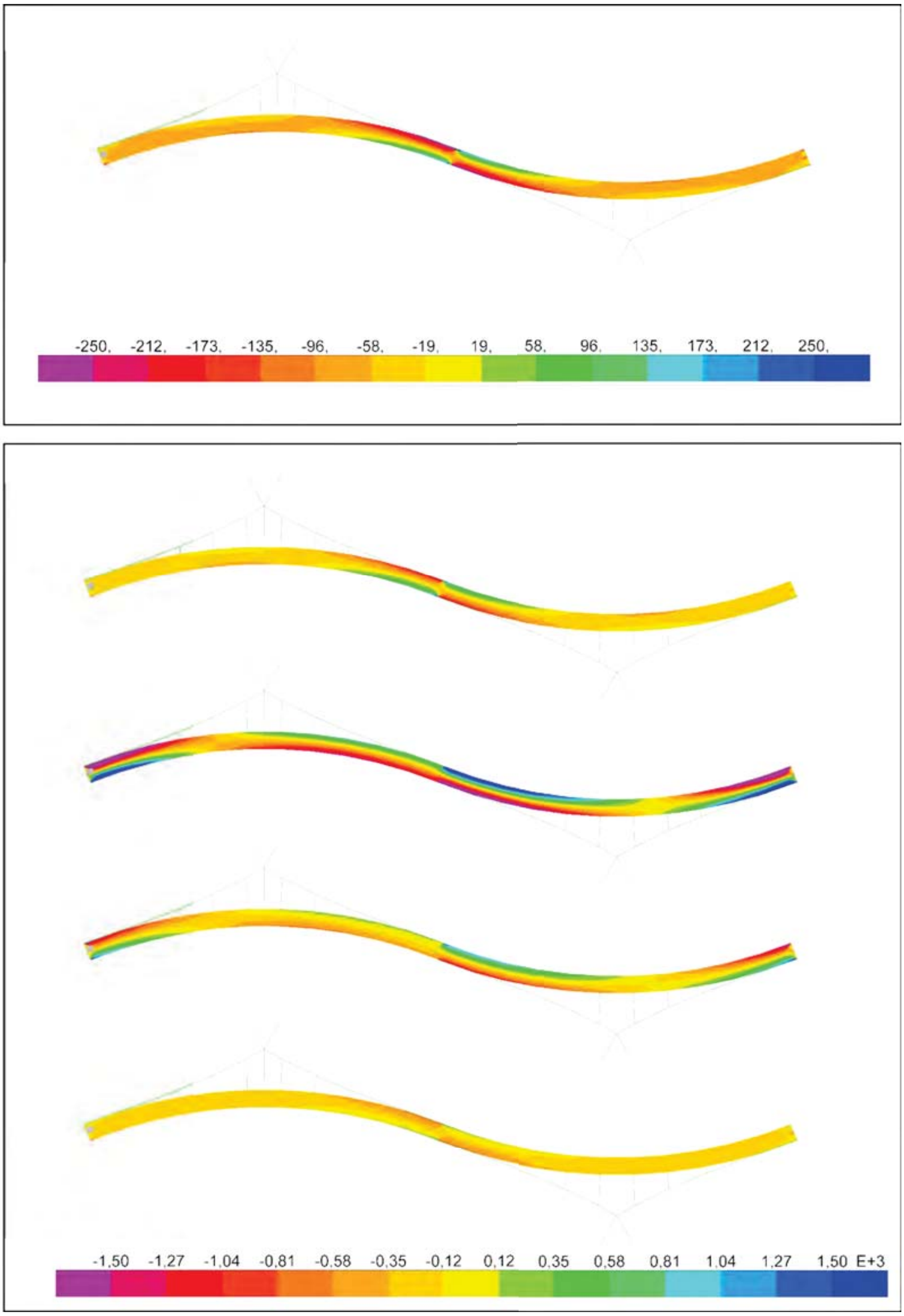


Figure 5.28: Stress perpendicular to the grain, F11. From the top: LC1-LC5

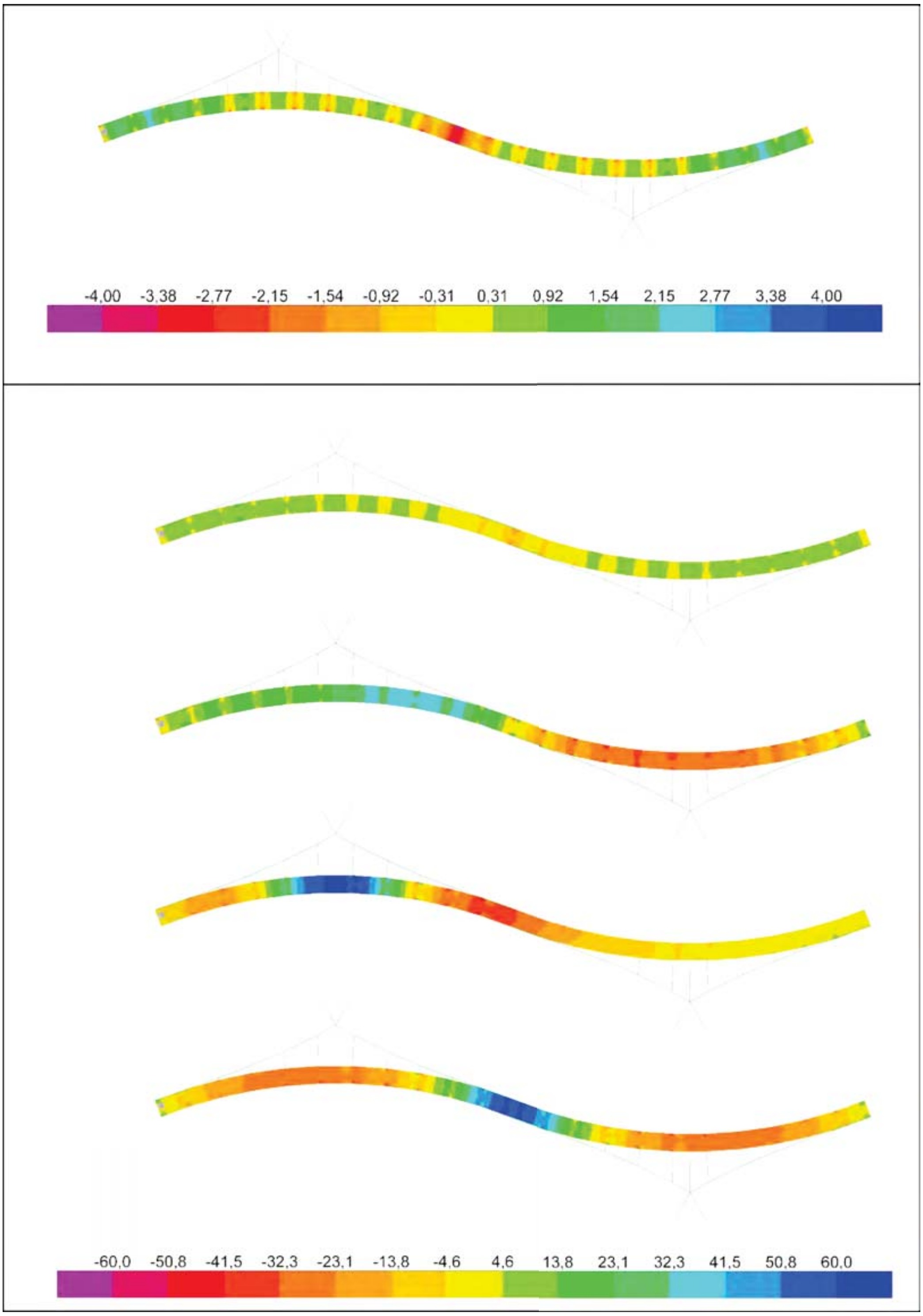


Figure 5.29: Bending moment in the longitudinal direction of the deck, M11. From the top: LC1-LC5

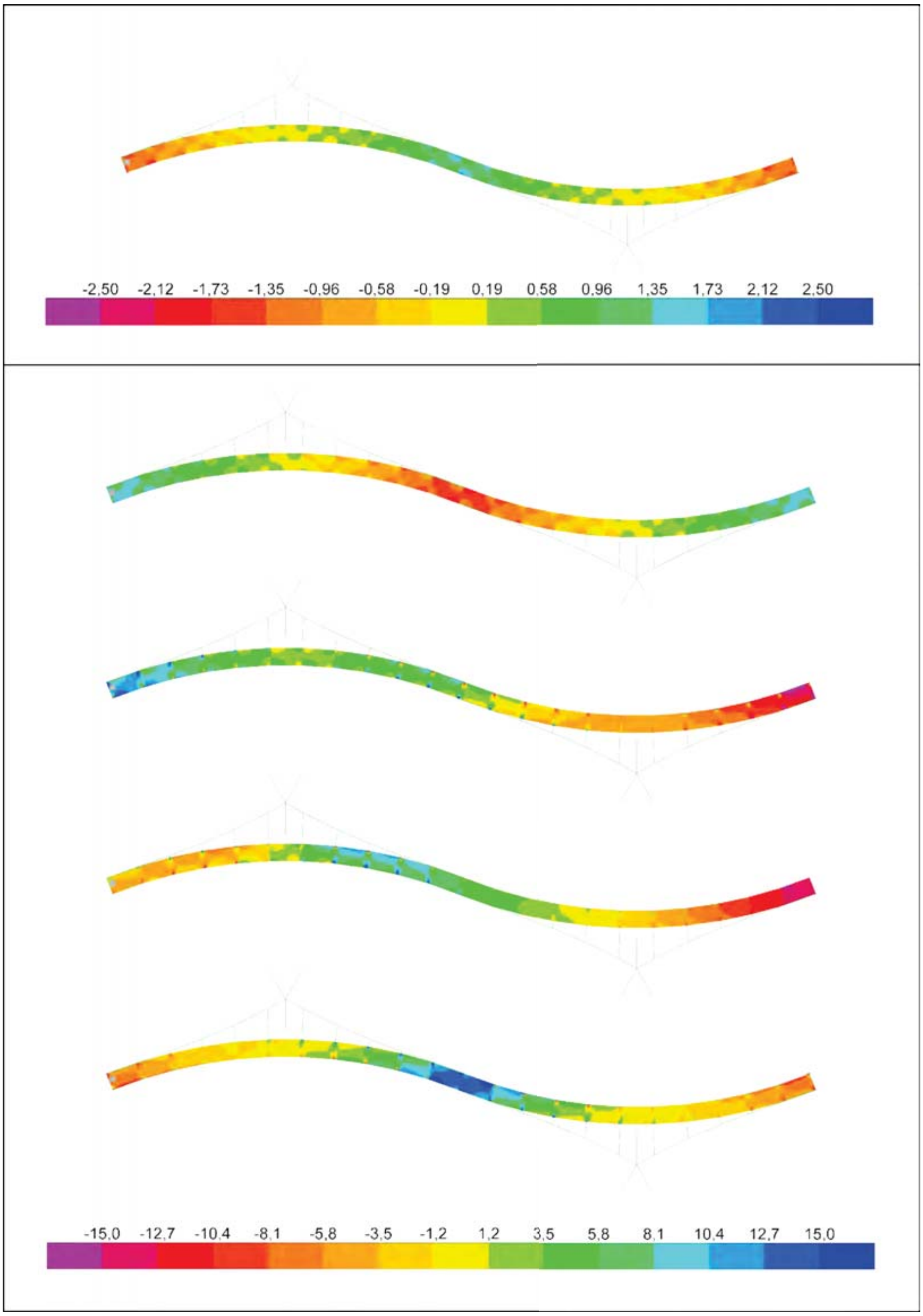


Figure 5.30: Bending moment in the plane of the deck, M12. From the top: LC1-LC5

5.6.2 Dynamic results

The dynamic behaviour of complex structures is difficult to predict but it is important to understand the structure and its weaknesses. It is therefore interesting to modify and use the FEM-model for calculating the modal frequencies. SAP2000 can not do a dynamic analysis with a model with cables. The cables are instead modelled as steel rods with the same properties as the cables. Through a eigen vector modal analysis the natural frequencies can be calculated. Masses representing the flooring and railing are also added as a mass of $18\frac{kg}{m^2}$. In SAP2000 you add in the current units so if the units are kN, m, s then the added mass is $0,018\frac{kNs^2/m}{m^2}$. Is is also possible to obtain the generalized mass from SAP2000 by exporting all joint masses and adding them.

$M' = 30672,8kg$

The results from the modal analysis show that the two first modes is within the frequency range of what a pedestrian can induce. The first mode is a combination of vertical and horizontal movement. The second one is only vertical and just below the pedestrian induced frequencies. The third which is higher than the range is a twisting mode.

Table 5.17: Relevant eigenmodes and frequencies

Eigenmode	Eigenfreqiency (Hz)	Type of eigenmode
1	2,65	vertical and horizontal
2	4,75	vertitcal
3	5,80	twisting

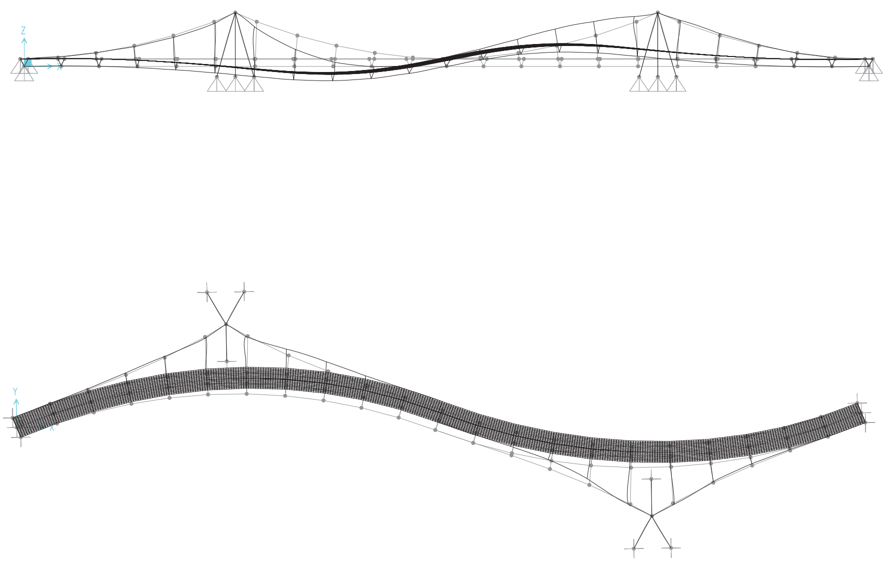


Figure 5.31: First eigenmode, f=2,65 Hz

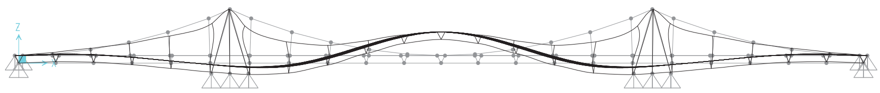


Figure 5.32: Second eigenmode, f=4,75 Hz

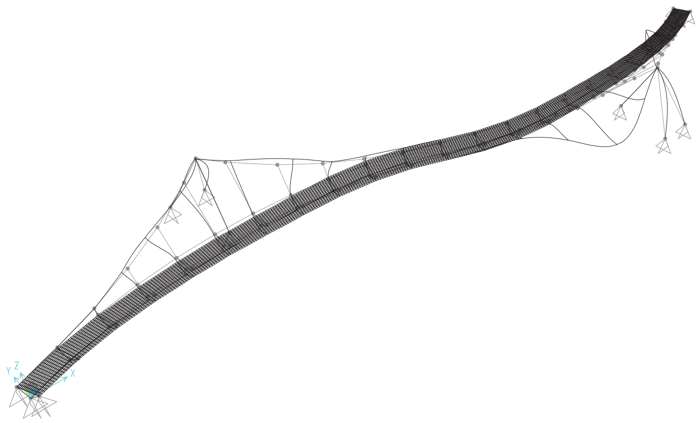


Figure 5.33: Third eigenmode, f=5,80 Hz

6 Load-bearing design and detailing of members

This chapter aims to give a overall picture on the preliminary load bearing design of the bridge. Focus is on main parts like the bridge deck, timber struts, cables, the mast and timber connections. The design is performed as ULS and SLS for the entire bridge with the load cases in Chapter Analysis and results.

The maximum and minimum loads and stresses are taken from the analysis performed in in SAP2000. Four different load cases has been analysed with the dead load, live load over the whole span and half of the bridge. A crowd of people standing in the middle and at 1/4 of the bridge. For more information about the analysis see chapter 5, Analysis and results.

According to Eurocode shall the load-bearing capacity or design value R_d be calculated as:

$$R_d = k_{mod} * \frac{R_k}{\gamma_m} \tag{6.1}$$

Where k_{mod} is the modification factor, R_k is the characteristic value and γ_m is the partial factor for the specific material. (Porteous and Kermani (2007)

Elements and calculations not performed in this thesis

The supports for the pylons and the ends of the bridge are considered to be in concrete and hidden in the ground. Design calculations of the supports is not in the scope of this thesis and would be something to develop in future work.

Also the steel details are one thing that in not in the scope of this thesis.

Fatigue and more detailed load bearing design is also something that could be performed in future work.

6.1 Design of the timber members according to Eurocode 5

The bridge is constructed with solid timber in the struts and gluelam in the bridge deck. In Eurocode there are different different material partial factors for solid timber and glulam. For both materials the same basic requirement is the same when designing structures.

$$f_d = k_{mod} * \frac{f_k}{\gamma_m} \tag{6.2}$$

Because of the production method of glulam it is considered to be more safe than solid wood. That is why the partial safety factor is higher for solid wood.

Material	γ_m
Solid wood	1,3
Glulam	1,25

Table 6.1: Partial safety factor γ_m for timber according to Eurocode 5

The durability and strength over time in timber structures depends widely on the surrounding environment. The temperature, exposure to climate and moisture are among the things that have a large impact. To take that into account during the load bearing design Eurocode has introduced Service classes.

6.1.1 Bridge deck

The bridge deck will be constructed as a stress-laminated timber deck.

When designing structures in gluelam. Gluelam can be produced as homogeneous or combined sections. For a normal beam the outer laminations can be produced with a higher strength class where the stresses are highest. The Timber deck will be subjected to axial stress, bending along M11 and M22. Because of the different stresses a homogeneous cross section is used.

$$f_{r,d} = \frac{k_{mod} * k_h * k_{sys} * f_{r,g,k}}{\gamma_M} \tag{6.3}$$

k_h is the size effect factor and apply to gluelam sections that have a smaller height than 600 mm and a thickness of 150 mm. For sections larger than 600 mm, k_h=1 For the bridge deck the hight is different for bending in different directions. However calculating with the lowest value of $k_h = 1,0$ is the most conservative choice.

$$k_h = min\{(600/h)^{0,1}; 1,1\} \tag{6.4}$$

$$k_{215} = min\{(600/215)^{0,1}; 1,1\} = 1,1 \tag{6.5}$$

- $k_{mod,short} = 1,1$
- $k_h = 1,0$

- $k_{sys} = 1,0$
- $\gamma_M = 1,25$

$$f_{r,d} = \frac{k_{mod,short} * k_h * k_{sys} * f_{r,g,k}}{\gamma_M} = \frac{1,1 * 1,0 * 1,0 * f_{r,g,k}}{1,25} \tag{6.6}$$

Strength class	Bending $f_{m,d}$	Tension 0 $f_{t,0,d}$	Tension 90 $f_{t,90,d}$	Compression 0 $f_{c,0,d}$	Compression 90 $f_{c,90,d}$	Shear $f_{v,d}$
GL24	21,1	14,5	0,4	21,1	2,4	2,4
GL32	28,2	19,8	0,4	25,5	2,9	3,3

Table 6.2: Design strength value of glulam

Stress parallel to the grain

The stresses parallel to the grain in the bridge deck comes from both bending and axial compression. Looking at the stress at the first load case where it is only dead load, the deck is mostly under compression except in the middle where the suspension cable connect. In load case 2 where the whole deck is loaded the top side is still in compression but the bottom is partly in tension due to bending between the bridge segments. In load case 3 the deck is fully loaded on half of the bridge creating compression in the top of the half which is loaded and tension in the bottom. On the side which is not loaded tension is instead in the top and compression in the bottom.

Figure 6.2 and 6.3 show the stresses parallel to the grain, S11, for the top and bottom face of the bridge deck. The figures are plotted with the design values for GL32 which show that the stress levels are OK according to Eurocode 5.

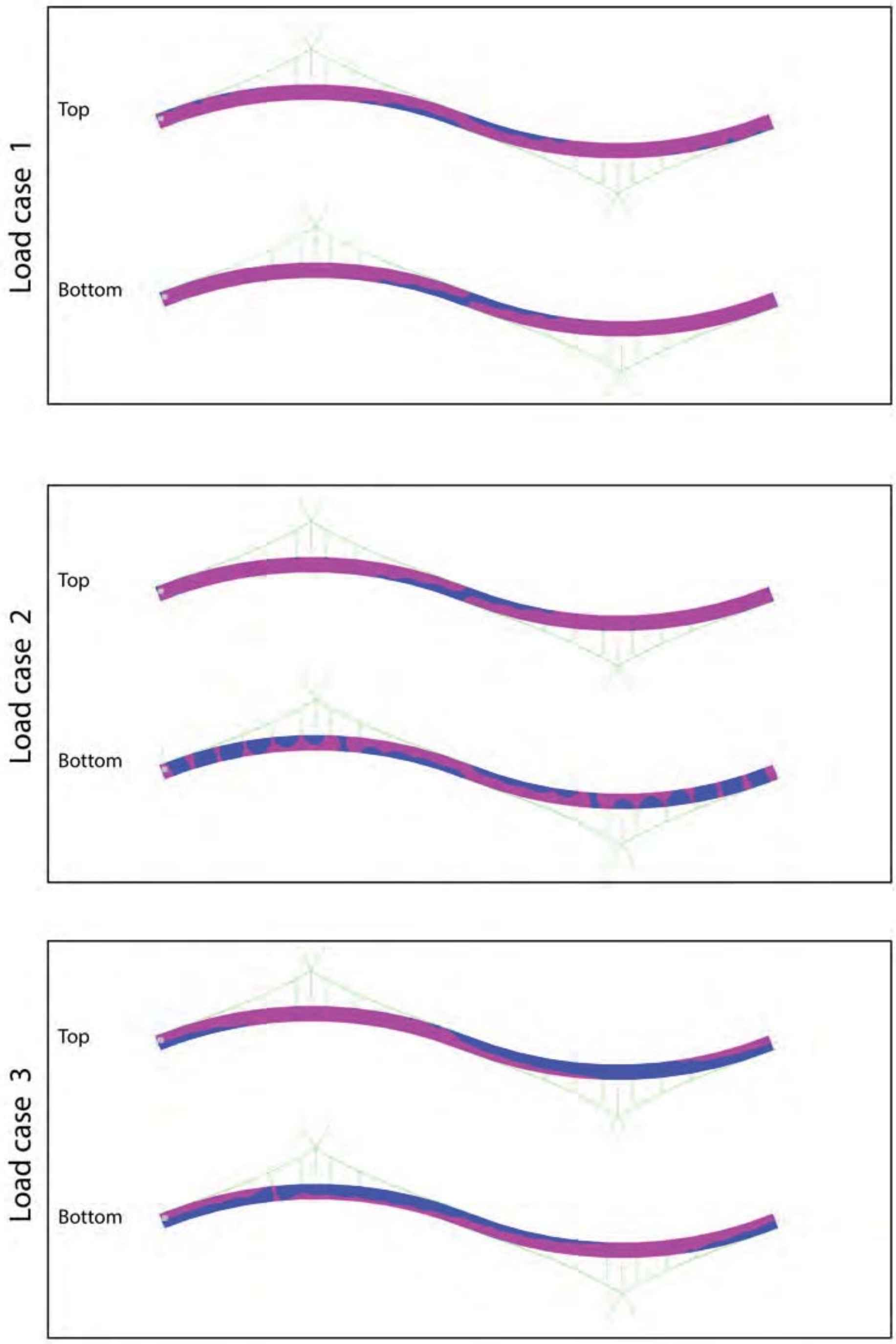


Figure 6.1: Compression and tension in LC1, LC2 and LC3

Figure 6.2: S11, bottom face

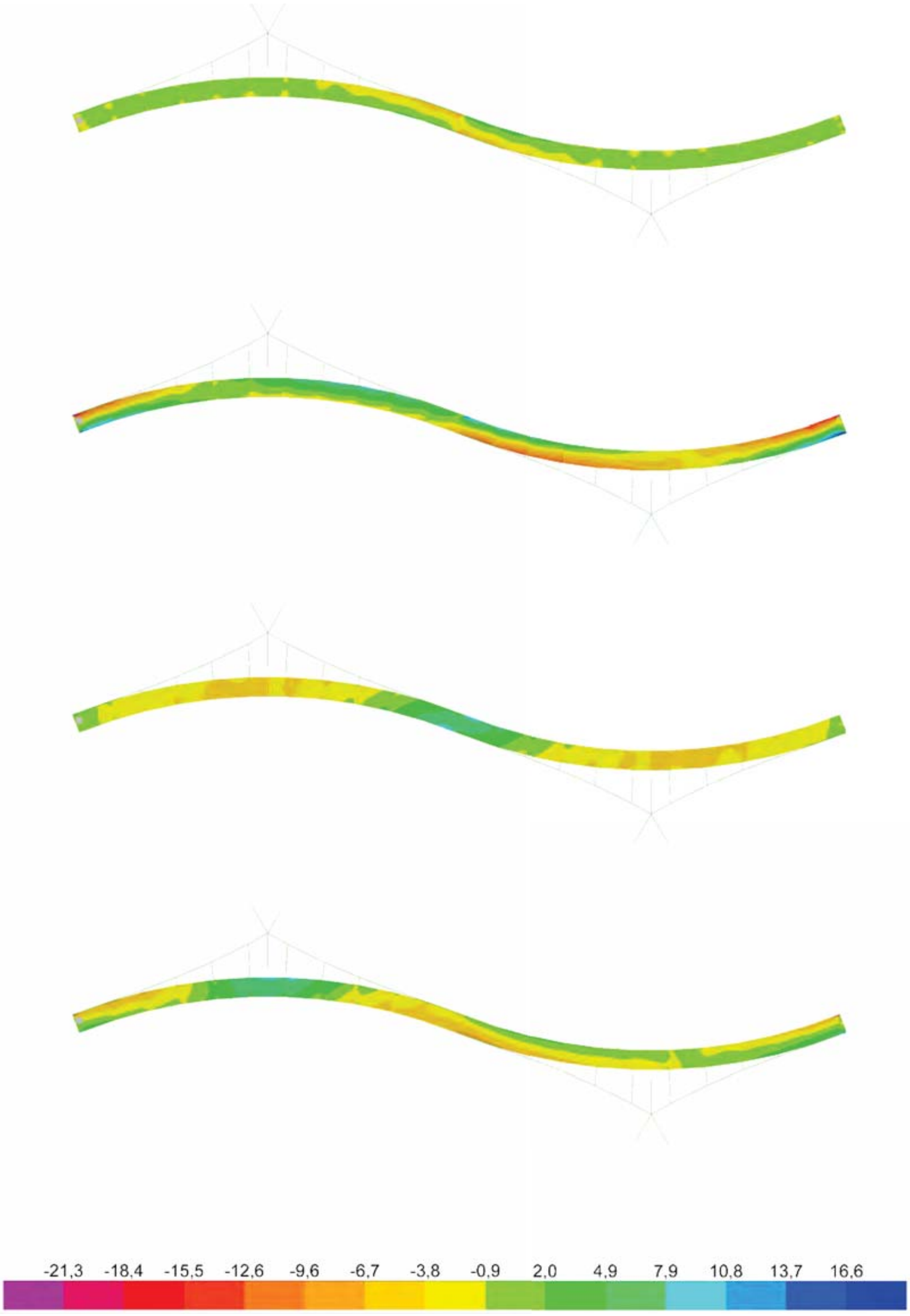


Figure 6.3: S11, top face

Stresses perpendicular to the grain

Stress perpendicular to the grain is a weakness in timber structures and especially tension. The stresses generated in the model in SAP2000 show that the stresses are to high according to the limit for GL32. The stresses is due to the forces transferred from the timber struts. The timber struts are connected to a steel beam that is connected to the timber deck. The modelled connection is so that the steel beam is connected to along its length causing the stress perpendicular to the grain in the timber deck. It is important that the detail let the stress be taken by the steel beam instead of the timber deck. Such a detail could be that the beam is connected with holes with gaps so that the force is not transferred and instead a hold that transfer the force from the edge of the deck. The deck is also stress laminated with stress bars each 60 cm making it possible to limit the tension perpendicular the the grain.

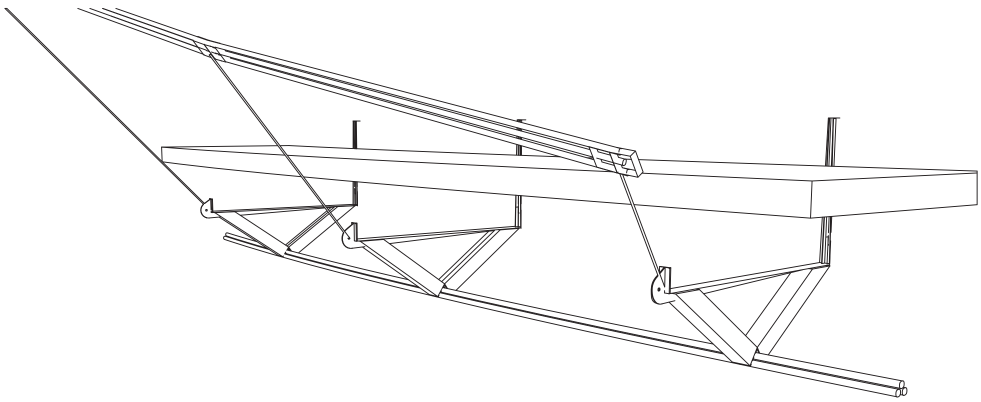


Figure 6.4: Assembly view of the connection between the bridge deck and steel beam

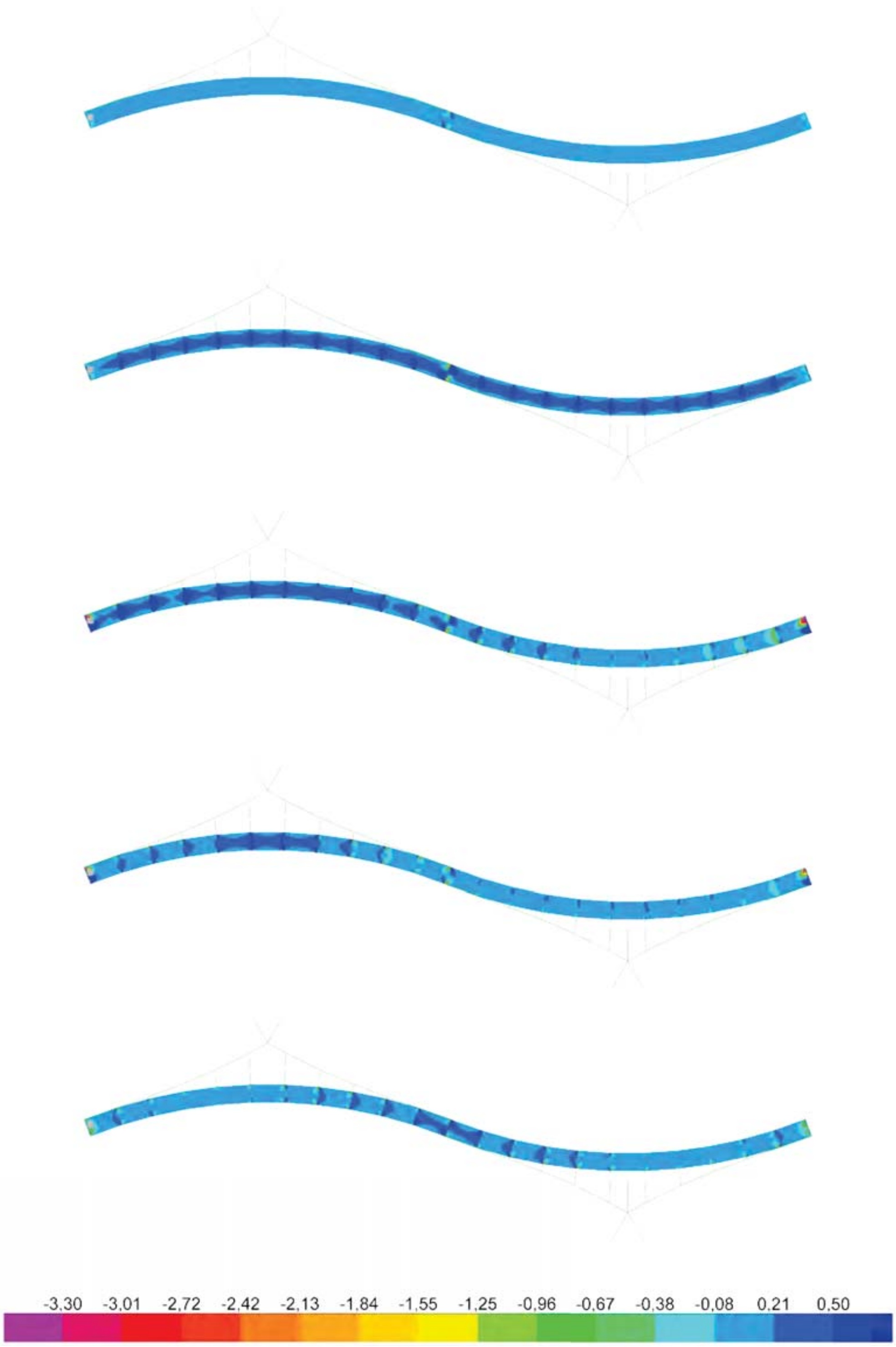


Figure 6.5: S22, bottom face, limits according GL32

Shear force

The shear stress S12 is shear acting in along the length of the bridge. Much of the stress is due to boundary conditions at the ends of the bridge and the suspension cable that connect to different sides. The shear stresses are presented in figure on the next page with the design values according to gluelam G32. A simple detail that could help handling the the stress from the connection of the suspension cable is show below. The detail would help transfer the forces in the cable to the other and less would be transferred through the deck. The moment would be the same but it would help the shear.

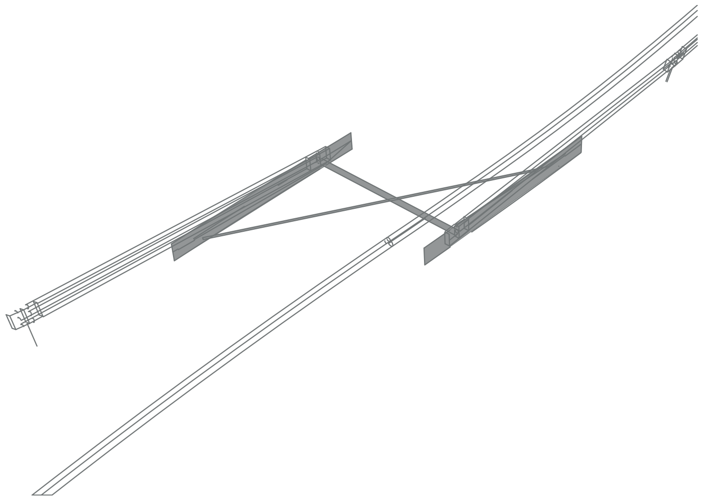


Figure 6.6: Detail of the connection between the two suspension cables

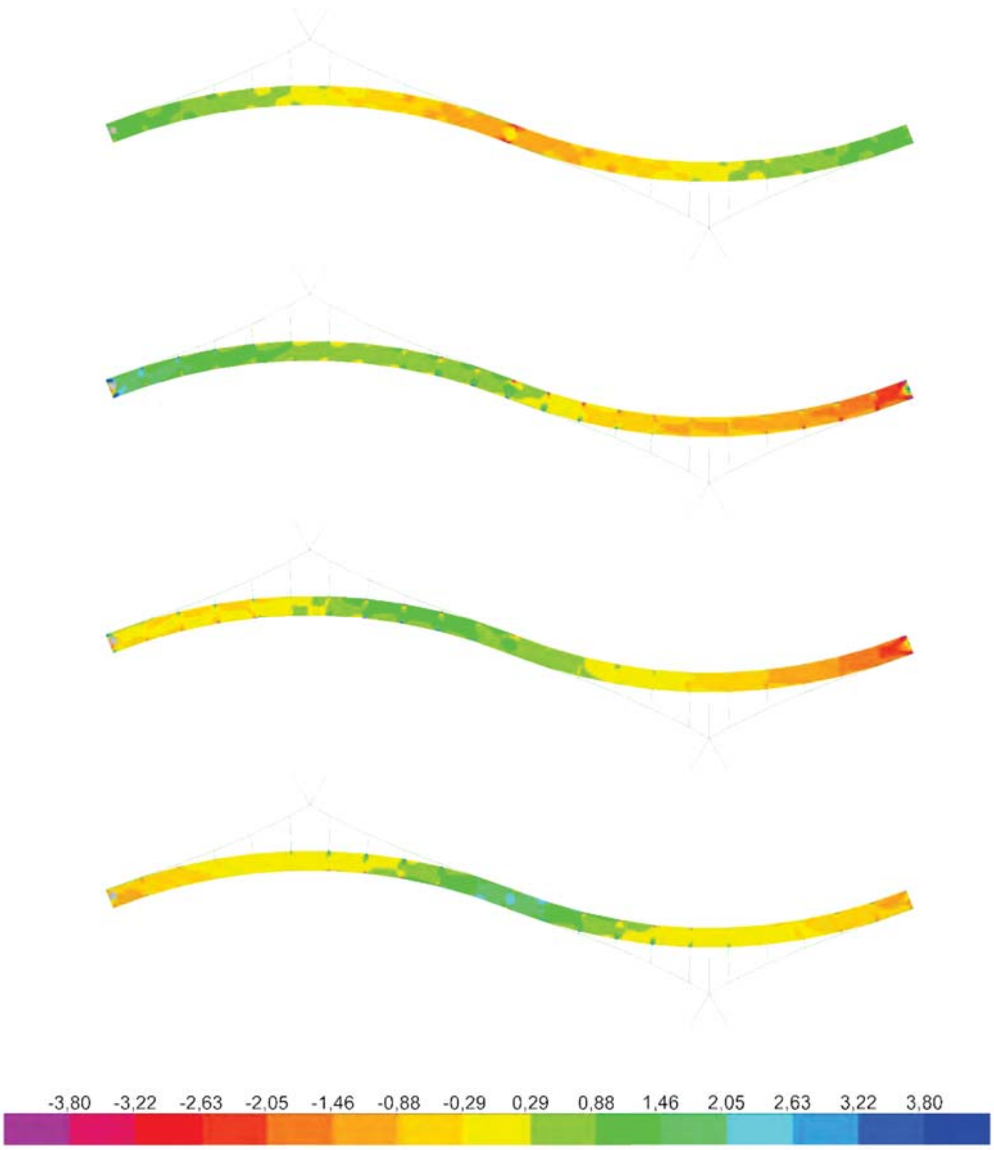


Figure 6.7: S12, bottom face, limits according GL32

6.1.2 Timber struts

The timber struts are constructed out of 150*150 mm solid timber C24.

	Hight	Witdth	Length
Tension strut	150 mm	150 mm	1220 mm
Compression strut	150 mm	150 mm	1220 mm

Table 6.3: Geometrical properties of timber struts

The section forces generated from SAP2000 are used for the load-bearing design. Load case 2 is the one that generates the largest section forces both for the compression and the tension strut. In the table below is the results shown. Both struts are subjected to axial force, moments and shear. Even if the

$$I_y = \frac{t * h^3}{12} \tag{6.7}$$

$$I_z = \frac{h * t^3}{12} \tag{6.8}$$

	A	I_y	I_z
Tension strut	22500 mm^2	42,18 * $10^6 mm^4$	42,18 * $10^6 mm^4$
Compression strut	22500 mm^2	42,18 * $10^6 mm^4$	42,18 * $10^6 mm^4$

Table 6.4: Section properties

	Axial force	Load case
Tension strut(88)	35,2 kN	2
Compression strut (25)	31,6 kN	2

Table 6.5: Actions in timber struts

According to Eurocode 5, the stresses caused by the forces in the timber struts should smaller than the design stresses calculated for the chosen material and environment.

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} \leq 1 \tag{6.9}$$

$$\frac{\sigma_{c,0,d}}{f_{c,0,d}} \leq 1 \tag{6.10}$$

Tension strut

$$f_{t,0,d} = \frac{k_{mod,short} * k_{sys} * k_h * f_{t,0,k}}{\gamma_M} = \frac{1,1 * 1 * 1 * 14}{1,3} = 11,85N/mm^2 \tag{6.11}$$

$$\sigma_{t,0,d} = \frac{N_d}{A} = \frac{35,2 * 10^3}{22500} = 1,56N/mm^2 \tag{6.12}$$

Now when all stresses and design values are know the design criteria can be checked.

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} = 0,13 \leq 1 \Rightarrow \textbf{OK!} \tag{6.13}$$

Compression strut

$$f_{c,0,d} = \frac{k_{mod,short} * k_{sys} * k_h * f_{c,0,k}}{\gamma_M} = \frac{1,1 * 1 * 1 * 21}{1,3} = 17,77N/mm^2 \tag{6.14}$$

$$\sigma_{c,0,d} = \frac{N_d}{A} = \frac{31,6 * 10^3}{22500} = 1,40N/mm^2 \tag{6.15}$$

Members under compression might fail from bucking if the design buckling strength is larger than the design stress. To verify that buckling will not occur the relative slenderness should first be calculated.

$$\lambda_y = \frac{L_{e.y}}{i_y} = \frac{0,7 * L}{\sqrt{\frac{I_y}{A}}} = \frac{0,7 * 1220}{\sqrt{\frac{42,15*10^6}{22500}}} = 19,72 \tag{6.16}$$

$$\lambda_{rel.y} = \frac{\lambda_y}{\pi} * \frac{f_{c.0.k}}{E_{0,05}} = \frac{19,72}{\pi} * \frac{21}{7,4} = 0,334 \tag{6.17}$$

The slenderness ratio is greater than 0,3 which mean that the strut will fail in buckling if it would fail. The condition below need to be fulfilled.

$$\frac{\sigma_{c.0.d}}{k_{c.y} * f_{c.0.d}} \leq 1 \tag{6.18}$$

Where,

$$k_{c.y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel.y}^2}} \tag{6.19}$$

$$k_y = 0,5 * (1 + \beta_c * (\lambda_{rel.y} - 0,3) + \lambda_{rel.y}^2 = 0,5576 \tag{6.20}$$

$$k_{c.y} = \frac{1}{0,5576 + \sqrt{0,5576^2 - 0,334^2}} = 0,9961 \tag{6.21}$$

$$\frac{\sigma_{c.0.d}}{k_{c.y} * f_{c.0.d}} = \frac{1,378}{0,9961 * 17,77} = 0,107 \rightarrow \textbf{OK!} \tag{6.22}$$

Knowing that the stress parallel to the grain is not large enough to buckle the strut control of the design criteria can be made.

$$\frac{\sigma_{c,0,d}}{f_{c,0,d}} = \frac{1,40}{17,77} = 0,08 \rightarrow \textbf{OK!} \tag{6.23}$$

Comments

The calculations show that the timber struts have some extra capacity which is not needed for design in ULS according to Eurocode 5. That mean that the dimensions could be smaller. However because of the connections which will be verified below the dimensions are kept at 150*150 mm.

6.1.3 Connections

As a preliminary design of the connection and the number of dowles need the axial force in the struts has used.

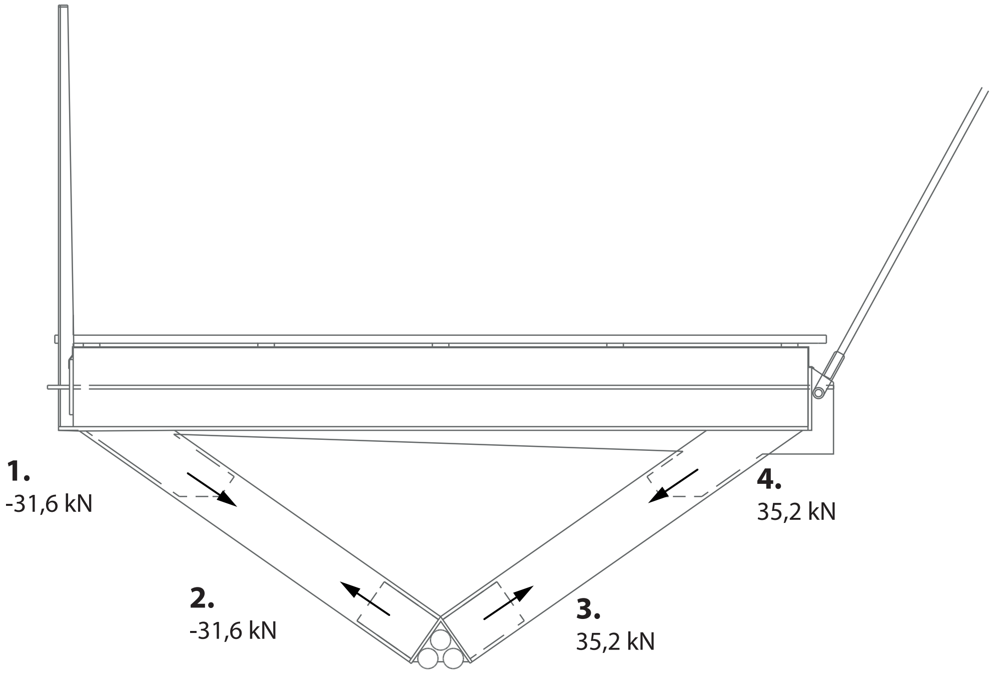


Figure 6.8: Axial force (P) transferred by the connections.

Table 6.6: Forces acting on the connections

	max axial force
1	-34,6 kN
2	-34,6 kN
3	36,4 kN
4	36,4 kN

The analysis showed that it is the load case with an uniformly distributed load over the whole bridge, LC2, that produce the largest axial force.

The most loaded connection is connection number 4. It is therefore the most interesting to analyse further.The connection is loaded at an angle to the grain and the connection is through a 8 mm slotted in metal plate and 8 mm metal dowels. The minimum distance from the outer sides is 3*d, where d is the diameter of the dowel. For a connection with four dowels of 8 mm diameter like the figure below the distance between the dowels will be.

$$a_{max} = 150 - 2 * 3 * 8 = 102mm \tag{6.24}$$

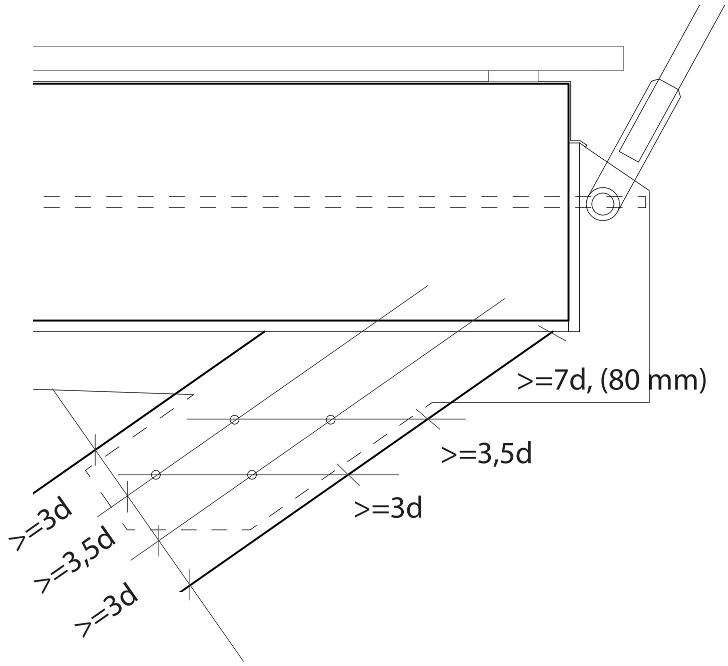


Figure 6.9: Connection 4

A timber-steel connection can have a brittle or ductile failure. A brittle failure is when the timber part breaks while a ductile failure is when the dowel of fastener deforms creating plastic hinges. A ductile failure is preferred because it is not the same unsuspecting failure. A ductile failure deforms the connection without total failure in the whole structure.

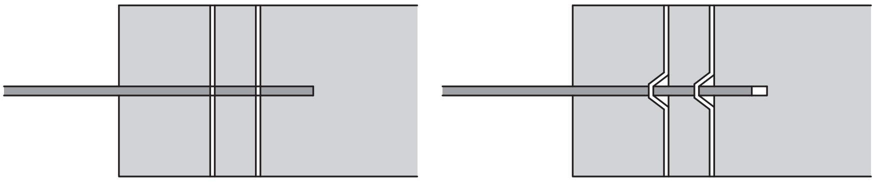


Figure 6.10: Preferred ductile failure mode

The forces distributed to the dowels is a result from the axial force in the strut. The most demanding load case for connection 4 is LC2 which results in an axial force of 35,2 kN. The axial force is distributed to the four dowels which are placed according to Figure 6.11.

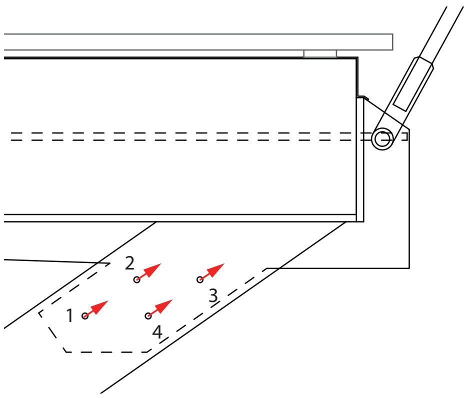


Figure 6.11: Distribution of forces to the dowels

$$F = \frac{P}{4} = \frac{35,2}{4} = 8,8kN \tag{6.25}$$

The force that a dowel connection can transfer depends on the length and diameter of the dowel and the material properties of the steel and timber. For a fastener with pre-drilled holes the embedment strength is determined by Eq. 6.26.

$$f_{h,\alpha,k} = \frac{0,082 * (1 - 0,01 * d) * \rho_k}{(1,35 + 0,015 * d) * \sin^2\alpha + \cos^2\alpha} \tag{6.26}$$

The characteristic yield moment for the dowel, Eq. 6.27, is when the dowel deforms and creates a plastic hinge.

$$M_{y,k} = 0,3 * f_{u,k} * d^{2,6} \tag{6.27}$$

The embedment strength for for C24 timber with a density of 350 kg/m^3 and a dowel with diameter of 8 mm and steel grade S355 is calculated, Eq. 6.28.

$$f_{h,0,k} = \frac{0,082 * (1 - 0,01 * 8) * 350}{(1} = 26,4N/mm^2 \tag{6.28}$$

$$M_{y,k} = 0,3 * 355 * 8^{2,6} = 23735Nmm \tag{6.29}$$

$$R_k = 2 * 1,15 * \sqrt{2} * \sqrt{2 * M_{y,k} * f_{h,k} * d} = 10,3kN \tag{6.30}$$

$$R_d = k_{mod} * \frac{R_k}{\gamma_m} = 0,9 * \frac{10,3}{1,3} = 7,1kN \Rightarrow \textbf{NOT OK!} \tag{6.31}$$

The force that the connection is able to transfer is to small with a 8 mm dowel. If the dowel would instead be a 10 mm would give the distance to the edge also increases making the distance between the dowels 90 mm.

$$f_{h,0,k} = \frac{0,082 * (1 - 0,01 * 10) * 350}{1} = 25,8N/mm^2 \tag{6.32}$$

$$M_{y,k} = 0,3 * 355 * 10^{2,6} = 42398Nmm \tag{6.33}$$

$$R_k = 2 * 1,15 * \sqrt{2} * \sqrt{2 * M_{y,k} * f_{h,k} * d} = 15,2kN \tag{6.34}$$

$$R_d = k_{mod} * \frac{R_k}{\gamma_m} = 0,9 * \frac{15,2}{1,3} = 10,5kN \tag{6.35}$$

$$F = \frac{P}{4} = \frac{35,2}{4} = 8,8kN \Rightarrow \textbf{OK!} \tag{6.36}$$

6.2 Load-bearing design of steel members according to Eurocode 3

Design of steel structures according to Eurocode 3 means that the design values should be calculated according to Eqs. 6.37 and 6.38.

$$f_{y,d} = \frac{f_y}{\gamma_{M0}} \tag{6.37}$$

$$f_{u,d} = \frac{f_u}{\gamma_{M2}} \tag{6.38}$$

Where,

$f_{y,d}$ is the design yield strength
$f_{u,d}$ is the design ultimate tensile strength
f_y is the characteristic yield strength
$f_{u,d}$ is the characteristic ultimate tensile strength
γ_{M0} is the partial safety factor for resistance of the cross section for overall yielding
γ_{M2} is the partial safety factor for resistance of the net section and for connections

The material properties for the steel parts in this thesis are shown in Table 6.7

Table 6.7: Material properties of the steel parts in this thesis

Modulus of elasticity	E=210 GPa
Shear modulus	G=81 GPa
Poisson's ratio	ν=0,3
Density	$\rho = 7850 kg/m^3$

6.2.1 Steel beam

The steel beams is there to create a connection between the timber struts, timber deck and the hangers. In the load bearing design of the steel beam considered to be out of the scope of this thesis. For future work it would been interesting to analyse the that detail and how the stresses distributes for the different load cases.

Table 6.8: Highest load in steel beams and which load case

	max axial force	Load case
Steel beam	3,9 kN	2

6.2.2 Cables

The load bearing capacity of cables are strongly dependent on the material, construction and connections. For this project the manufacturer Pfeifer is seen as a possible producer. They have calculated the load bearing capacity for their cables according to Eurocode. In their data sheets they have given the characteristic breaking load and also the tension limit, which is the limit for when the cable stop showing elastic behaviour. The tension limit is therefore the value that has to be larger then the forces in the cables.

Table 6.9: Highest load in cables and which load case

	max load	Load case
Suspension cable	768 kN	2
Hanger cable	74 kN	2
Horizontal cable	812 kN	2

Table 6.10: Cable load and corresponding type

	max load	Type of cable	Characteristic breaking load	Tension limit
Suspension cable	768 kN	PV150	1520 kN	921 kN
Hanger cable	74 kN	PE15	141 kN	86 kN
Horizontal cable	812 kN	3 * PG 55	3 * 537 kN	3 * 326 kN

The forces in the suspension and horizontal cable are rather large leading to relative large dimensions. A diameter of 150 mm for the horizontal cable is difficult to solve in a successful way regarding to detailing so it is proposed that the forces are divided in three cables with a diameter of 55 mm. The forces in the suspension cable are depending on the sag of the cable. Raising the mast with 1 meter would reduce the load significantly. Modifying the script in Grasshopper showed a decrease on 18% of the force in the suspension cable. There are also one alternative to divide the forces in more cables but that leads to a complicated detail and more difficult to active the the correct tension in the cables.

One alternative for the hanger cables would be to instead make them as a tension rods. That is a cheaper and more flexible way since it can easily be adjusted to the right length and tension by turning the connection and thereby shortening the rod. A tension rod with a diameter of 16 mm would have enough load bearing capacity for the hangers.

6.2.3 Mast

The masts are constructed as a compression pylon that are inclined towards the bridge and held back by back stays. The mast and the back stays are connected to concrete supports in the ground. Since the bridge is rotational symmetric the two mast are the same. Back stay 1, further from the middle of the the bridge, has the most load of the back stays since the load in the part of the suspension cable towards the middle is larger. A development of the bridge could be to rotate the pylons to make them more the same. The masts was first considered to be made out of timber but since the water level in Lödde river rises from time to time, steel and concrete was a more durable option.

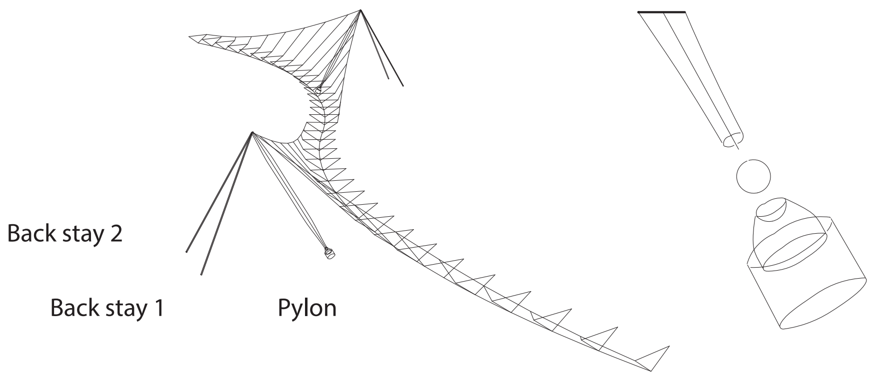


Figure 6.12: Mast and support

The end of the pylons are put on a steel sphere to create connection that does not transfer any moment.

Table 6.11: Highest load in the members of the mast

	max load	Load case
Compression pylon	1043 kN	2
Back stay 1	173 kN	2
Back stay 2	246 kN	2

Pylon

The compression pylon i made of a steel pipe with a diameter of 273 mm. The ends are tapered and the connection to the ground is placed on a steel sphere in order to have as small moment in the pylon.

Table 6.12: Section properties of the pylon

L	D	t	A	I	W_{el}	W_{pl}
7040	273	10	8260	7154	524	592

$$N_{cr} = \frac{\pi * E * I}{L^2} = \frac{\pi * 210 * 10^3 * 7154 * 10^4}{7040^2} = 2989kN \tag{6.39}$$

$$\lambda = \sqrt{\frac{A * f_y}{N_{cr}}} = \sqrt{\frac{8260 * 210}{2989 * 10^3}} = 0,87 \tag{6.40}$$

For a cold formed closed cross section the imperfection factor for buckling curves, $\alpha = 0,49$, can be taken from table and buckling curve c.

$$\Phi = 0,5 * [1 + \alpha * (\lambda - 0,2) + \lambda^2] = 1,044 \tag{6.41}$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0,617 \tag{6.42}$$

The design value for the normal force can then be calculated with the reductions for buckling.

$$N_{Rd} = \chi * \frac{A * f_y}{\gamma_{M1}} = 0,617 * \frac{8260 * 275}{1,0} = 1275kN \tag{6.43}$$

The normal force for the dimensioning load case LC2,

$$N = 1090kN \leq N_{Rd} \rightarrow \textbf{OK!} \tag{6.44}$$

Back stays

The back stays supporting the pylon will be made out of steel. Pfeifer is one manufacturer who deliverers steel tension rods with connections. The load bearing capacity of these rods are much depending on the connections and how they are produced. To help designer choosing the right dimensions Pfeifer has calculated the design values for their elements according to Eurocode. The type of rod suggested for this project is "tension rod type 860". It is made out of S460N steen and has a value of $E = 210+-10N/mm^2$.

Table 6.13: Normal forces, design values and diameter for the back stays.

	Normal force (ULS)	Design value	Diameter
Back stay 1	173 kN	278,4 kN	30 mm
Back stay 2	246 kN	278,4 kN	30 mm

6.3 Serviceability limit state (SLS)

Verifying timber structures for serviceability limit state (SLS) means that checks deformation and vibrations should be performed.

6.3.1 Deformation

As load cases for serviceability limit state design the same distribution of loads is used. However the safety facors i.e. the size of the loads are not the same.

$$u3_{max} \leq \frac{l}{400} \tag{6.45}$$

$$u3_{max,segemnt} \leq \frac{3720}{400} = 9,3mm \tag{6.46}$$

$$u3_{max,bridge} \leq \frac{82000}{400} = 205mm \tag{6.47}$$

It could be interesting to see the deformation over the whole length of the bridge and also the deformation of one bridge segment between two connections to the hanger cables.

Table 6.14: Vertical deformations for different load cases

Load case	max u3	min u3	max u3 in one bridge segment	
SLS 2	-9	<0	ca 1	**OK!**
SLS 3	-31	25	ca 1	**OK!**
SLS 4	-22	14	ca 1	**OK!**
SLS 5	-23	11	ca 1	**OK!**

6.3.2 Vibrations

If the natural frequencies of the bridge are within the range of what can be induced by pedestrians the size of accelerations should be investigated. The dynamic analysis showed that the two first eigen modes had frequencies within the critical range. For more info, see Chapter 4.

Table 6.15: Relevant eigenmodes and frequencies

Eigenmode	Eigenfreqiency (Hz)	Type of eigenmode
1	2,65	vertical and horrizontal
2	4,75	vertical
3	5,80	twisting

In Chapter 3, the method for calculating the accelerations was shown. With values of the mass of the bridge, the natural frequencies and damping factor the accelerations can be estimated.

$$a_{vert,1,walking} = \frac{200}{M' * 2\zeta} \tag{6.48}$$

$$a_{vert,1,running} = \frac{600}{M' * 2\zeta} \tag{6.49}$$

$$a_{hor,1,walking} = \frac{50}{M' * 2\zeta} \tag{6.50}$$

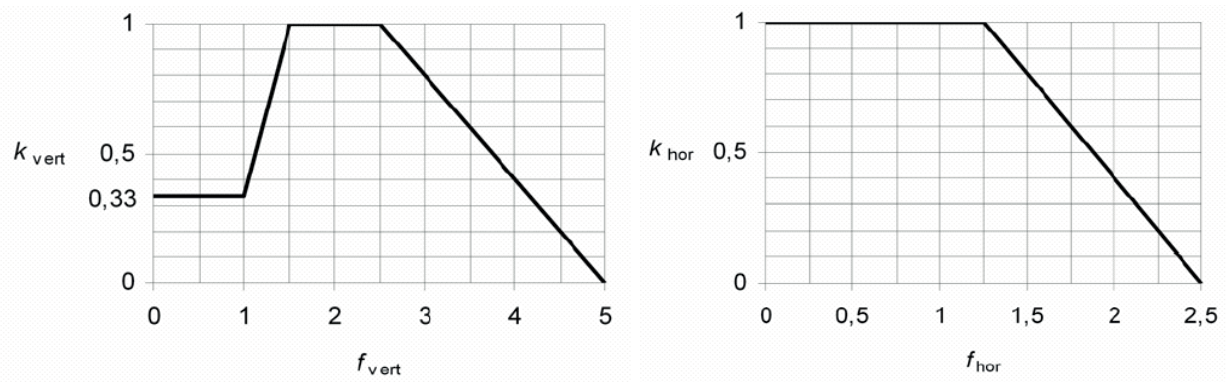


Figure 6.13: Relationship between k_vert and k_hor and natural frequencies

Where,

- $M' = 30672,8kg$
- $\zeta = 0,9\%$
- $k_{vert,2,65} = 0,97$

- $k_{vert,4,75} = 0,10$

The conditions that should be met according to Eurocode are,

$$a_{vert} \leq 0,7m/s^2$$
and
$$a_{hor} \leq 0,2m/s^2$$

Since the natural frequencies are outside of range for horizontal vibrations caused by pedestrians only control of vertical vibrations are made. First the response from one pedestrian who are walking or running is calculated.

$$a_{vert,1,walking} = \frac{200}{30672,8 * 2 * 0,009} = 0,36m/s^2 \longrightarrow \textbf{OK!} \tag{6.51}$$

$$a_{vert,1,running} = \frac{600}{30672,8 * 2 * 0,009} = 2,17m/s^2 \longrightarrow \textbf{NOT OK!} \tag{6.52}$$

After the response from one person has been obtained, the response for more people can be calculated. According to Eurocode a group of people consist of 13 individuals and a stream of people are $0,6pers/m^2$. In the case of this bridge it equals almost 100 individuals.

$$a_{vert,n,walking} = 0,23 * n * k_{vert} * a_{vert,1,walking} \tag{6.53}$$

$$a_{vert,stream,walking} = 0,23 * (0,6 * w * l) * k_{vert} * a_{vert,1,walking} \tag{6.54}$$

$$a_{vert,1,13,walking} = 0,23 * 13 * 0,97 * 0,36 = 1,05m/s^2 \longrightarrow \textbf{NOT OK!} \tag{6.55}$$

$$a_{vert,1,13,running} = 0,23 * 13 * 0,97 * 0,217 = 6,30m/s^2 \longrightarrow \textbf{NOT OK!} \tag{6.56}$$

$$a_{vert,stream,walking} = 0,23 * (0,6 * 2 * 82) * 0,97 * 0,36 = 7,95m/s^2 \longrightarrow \textbf{NOT OK!} \tag{6.57}$$

The conclusions from this is that the verifications according Eurocode show that the accelerations is only OK for when one person walking over the bridge. The fact that the bridge is lightweight has a large influence on the results from the verifications.

Dynamic analysis according to SETRA

If the bridge were going to be designed according to the check list from SETRA, the bridge would probably be a Class III bridge. A pedestrian bridge for standard use, that may occasionally be crossed by large groups of people but will never be loaded throughout its bearing area. In stage 2 according to SETRA is the definition of comfort level. In this case the vibrations can be something that enhance the experience walking over the bridge. The bridge is located in the countryside and the structure looks a little unstable. It is almost expected that the bridge will move when walking over it. The minimum comfort level according to SETRA is defined as "Under loading configurations that seldom occur, accelerations undergone by the structure are perceived by the users, but do not become intolerable." The corresponding frequency range is according to SETRA 1-2,5 Hz for vertical accelerations and 0,3-0,8 Hz for horizontal. In Stage 3 the natural frequencies should be determined for two load cases, one with and empty bridge and one with a fully loaded bridge with one pedestrian per square meter.

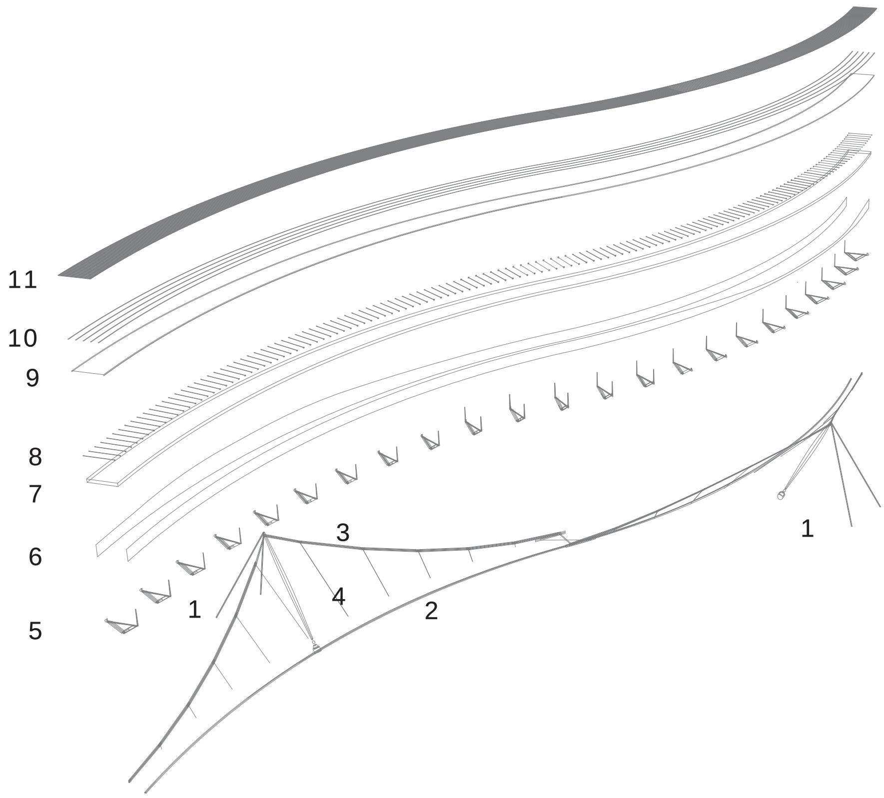
Table 6.16: Natural frequencies for Stage 3 according to SETRA

Load case	Eigen mode	Natural frequency	Frequency range
Empty bridge	1	2,65 Hz	3
Empty bridge	2	4,75 Hz	4
Full bridge	1	2,28 Hz	2
Full bridge	2	4,09 Hz	3

The natural frequency for the different load cases are shown in Table 6.15. The risk of resonance is depending on the frequency range in which the natural frequencies lie. In the case of the bridge in this thesis, it lays in range 2-4 which means that there are medium to negligible risk of resonance. Since the bridge is a Class III bridge there are no more analysis needed according to SETRA. If the bridge were to be within the first range of frequencies, with maximum risk of resonance, the analysis should continue to Stage 4 regarding dynamic load analysis. See Chapter 3 for more information about the procedure according to SETRA.

6.4 Details and Durability

Well thought through details are important for both the design and experience of a pedestrian bridge. The details should fit well with the concept of the bridge and enhance the experience. How the railing feels, the flooring looks and the slenderness of the pylons are some of the things that creates the overall picture. A timber bridge is not only made out of timber. Timber is a good material for construction in many cases, but in some cases steel is much more suitable.



1. Mast
2. Horizontal cable
3. Suspension cable
4. Hager cable
5. Timber struts + steel beam
6. Wire mesh railing
7. Stress laminated timber deck
8. Post tension bars
9. Metal sheet plate
10. Timber sluts
11. Timber flooring

Figure 6.14: Assembly of the different elements of the bridge

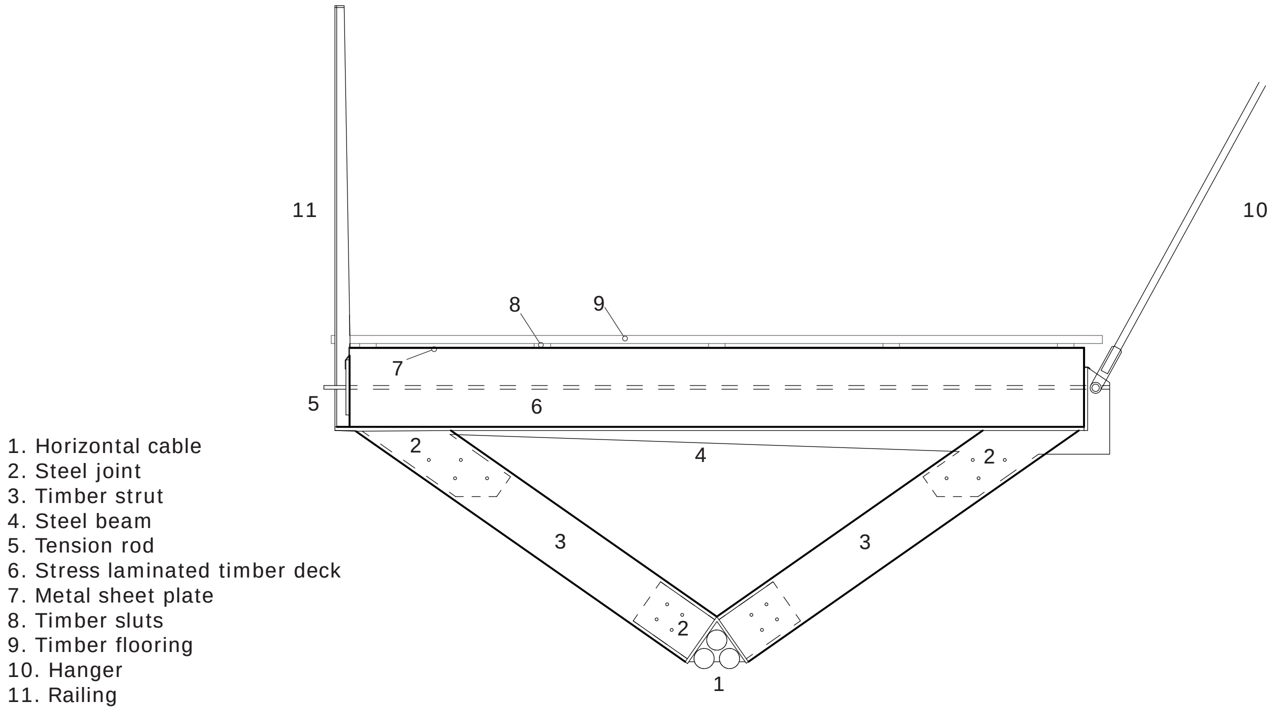


Figure 6.15: Detail 1:20 of the cross section

As mentioned in Chapter 2, it is crucial to think of the details when designing a timber bridge. They should take in consideration protection of the timber and enabling the bridge to be safe during its lifespan. The bridge in this thesis is an open bridge without a roof protecting the timber. If it should be able to be functional for its intended lifespan, it needs to be protected. The load bearing part of the bridge deck is a curved stress laminated beam with stress bars spaced 60 cm. It can be sensitive to both standing water on top of the beam and water that gets trapped where the stress bars are connected. It is important pay special attention to these details. In order to protect a metal sheet covering is used. It is covers the timber beam with an inclination of 2 % This metal covering is passing the connection of the stress bars. That makes the water run of the bridge and past the crucial details. A slippery metal sheet is not a good flooring for a pedestrian bridge and it need to be protected from being damaged. If the metal sheet is perforated, water could get in under the sheet and get trapped which would lead to serious problems with durability. The metal sheet is therefore protected by a plank deck which serves as flooring for the bridge. If the metal sheet still should be damaged it is ventilated under so that the moisture can get out. This detail should enable the load bearing deck to be protected. Connections to the timber struts is protected by the bridge deck since they are located inside of the 30 degrees from the outer sides of the bridge deck. That is good for the durability of the connections. In the beginning and end of the bridge it is located close to vegetation, reed on one side and bushes and grass on the other. I could be a problem if the vegetation grows and lead water and organic materials to the connections. It is important that the joints are inspected and that surrounding is kept down if it should be a problem.

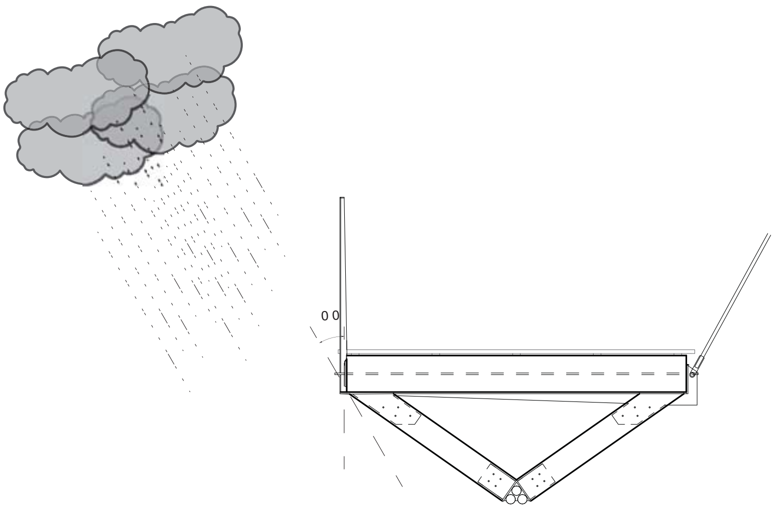


Figure 6.16: Area protected from rain

7 Discussion and conclusions

The design of a pedestrian bridge that aims for more than just connecting two points can be more complicated than just connecting them in a straight line. The result and value of the invested money can often more as well, if the bridge can serve as an attraction in its self. The design of a bridge should take in consideration the persons walking over and how they experience it. How does the handrail feel, does the bridge move when someone walk over it, does it feel safe or maybe a little thrilling? Does the bridge give new ways of experiencing the site and focus the attention to things you would not have noticed otherwise? A bridge is always a part of a larger picture connecting footpaths and should therefore be seen in its context.

Going from the conceptual design of the bridge in the Master thesis "A pilgrims walk" to the developed design in this thesis has shown a lot of potential in an integrated design process. During this process the combination of different software has been very good. Using Rhino3D and Grasshopper for the design and framework for the FEM model that verify the results. Going back and forth, adjusting and modifying, changing parameters, making new models and getting new results is important when designing a complicated structure. Small changes can have large influence on the results and the design. Like the change of the suspension cable that in the conceptual design ran under the bridge, later connected to different sides of the deck. The change gave a simpler solution for the suspension system that was not connected to the horizontal cable. However the first solution was interesting in the sense that the suspension cable was a continuous element and did not create any moment and shear in the deck. For further work it could have been interesting to see how a system like that could work. Another example of how the process has benefited with an integrated design process is the example of the pylon raised by 0,5 meters, the dimension of the suspension cable could have been changed to a smaller size.

The form finding in Grasshopper was helpful for fast response of changes like how many bridge segments should there be, curvature of the deck, connection point of the hangers and the height of the pylon. It was especially useful when designing the double curved bridge model. In that case the vertical component of the suspension cable anchoring in the middle of the bridge, should be half of the vertical component in the hangers. If it would not have been possible to see the approximate force in the cables it would have taken much more time to find the correct shape.

Creating a parametric model where variations of the same concept can be evaluated has been a good tool both for the architectural design of the bridge and structural evaluation. By creating the a simplified model for export to SAP2000 and creating a template file with correct properties enabled simple evaluation of different configurations.

Timber is a material that has a good potential for bridge building in the future but knowledge about its properties is very important. There are many things that can go wrong and that can scare some people from using timber. What the history and development of timber bridges show if designed correctly, timber bridges can stand for hundreds of years, but if the design is wrong the bridge can stand for only for a couple of years. Another benefit with timber is that it is easy to work with and parts can be replaced and maintained.

The fact that timber is an orthotropic material has some influences when designing complex structures. It is crucial to know which way timber should be designed in each scale of the project, from global system down to the details. Also knowing how to modify timber and creating new possibilities in the design like stress laminated timber decks helps when deciding if timber should be used.

One of the aims when working with this thesis was to see how much of the bridge could be built in timber and what it would mean for the design. In the conceptual design of the bridge the pylons were also made of timber, which was at a later stage changed to steel during the structural design. That decision was both based on the architectural design and durability issues. It happens that the water level in Lödde å rises which would lead to that the foundation detail was exposed to water. Since that is one of the most important connections, and if it would be damaged, the whole bridge would collapse. As a result steel chosen as a more safe solution. That was one reason, but the design was just as important. A more slender pylon in a different material connecting to the suspension cable gives a clearer design of the bridge. The timber bridge deck is more in focus and is a continuation of the foot path.

If the focus of this thesis would not have been to try to build the bridge in timber another solutions for the timber struts under the bridge would have been to use steel. The cross sections could have been prefabricate steel plates that were cut out using water jet. Steel would have been a solution with fewer joints and the alternative with timber still uses steel plates to connect. Using steel plates would probably be an OK alternative to timber for the architectural design but that is something that could be looked at in future work.

Load bearing design according to Eurocode is sometimes complicated to apply when designing a complex structure. As an example, the timber deck that is a three dimensional element which is subjected to bending in two directions and axial compression. When modelling the deck as an shell object in SAP2000, it is not as clear which stress is due to bending or axial compression. In Eurocode 5 there is a difference in which design stress, $f_r d$, should be used for the load bearing design if the stress is due to bending or axial compression/tension. A safe choice is to use the lower value.

The dynamic analysis final model showed that the bridge was within the range of what can be induced by a pedestrian. The bridge is located in the countryside and 82 meter long and 2 meters wide. The length and width of the bridge makes the dynamic load according Eurocode higher than what is expected for a bridge in that location. The verification for Serviceability Limit State shows that the bridge would not be ok. When designing according to SETRA, which takes more things in consideration and has another method of calculating the dynamic load, the bridge would be ok. The results are very different for two approved ways of evaluating a pedestrian bridge. It is interesting and raises questions of which method should be used when. Why are they so different and is it OK to choose the one that say the design is OK, if the other does not? If the bridge would become a real project or for future work a more detailed dynamic evaluation should be performed and also get the thoughts of the owner of what is the design criteria for the bridge.

Having looked at both design according to Eurocode and according to SETRA shows that the latter is more detailed and taking more parameters in consideration. There are many different kind of bridges in different locations and having the same and rules for all is a problem. When designing according to SETRA much is up to the owner to decide and that might also be a problem. The owner needs to be able to make an educated guess of how the area and the use could develop over the lifespan of the bridge. If the bridge should be there for 40 or 80 years that can change a lot, but on the other hand if it would change dramatically there might also be a need for a different kind of bridge.

The nest step in design of the bridge would be to go further into details and from a preliminary to a complete structural design. Detailing of the concrete supports and how the bridge land on the two sides of Lödde river is important for the appearance of the bridge.

This thesis shows that it is possible and also a good alternative to construct a double curved suspension bridge out of timber. A timber bridge is not just made out of timber, but also consist steel and concrete. Knowing when to use which material should be a decision based on both structural and architectural point of view. Timber is a natural material which many good properties which makes it a great choice for design of pedestrian bridges.

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A A: Summary of the master thesis "A pilgrims walk"

As the basis of this thesis, a master thesis in Architecture performed by the author, the design of a pedestrian bridge at Borgeby castle was given. The bridge was intended to help the development and give new ways of experiencing the site connecting to existing and new foot paths.

A pilgrim is a person who is travelling with a set goal in mind, but the goal is more than just the physical movement from start to finish. He stops and ponders over the questions and seeks guidance or enlightenment. The connection to nature is an important aspect to the journey as it gives the person a distance to the everyday life and enables new thoughts to be developed.

In Scania, southern Sweden, the Swedish Church has been developing pilgrim routes to enable the movement between churches and cities. The routes in Scania intended to be one part of a larger network of pilgrim routes connecting Trondheim in Norway with Santiago de Compostella in Spain. The bridge proposed in the thesis "A Pilgrims walk" could be one stop on the long journey.

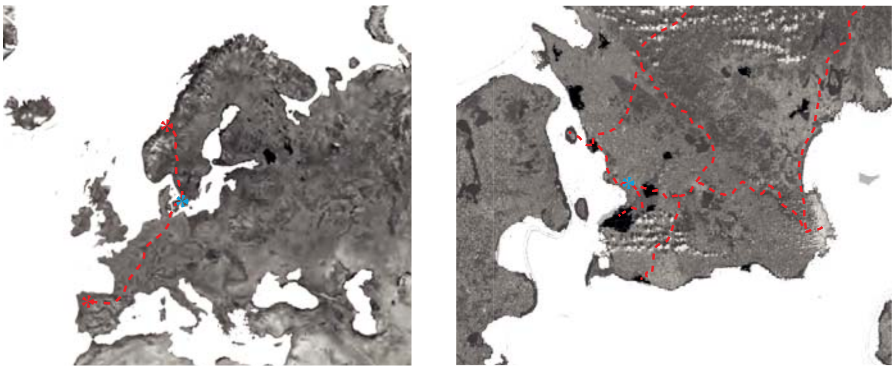


Figure A.1: Pilgrim routes in Scania and between Santiago de Compostella and Trondheim, (Asp (2012)

A.0.1 Borgeby Castle

The meaning of a pilgrim has changed through out history, at least in a more philosophical meaning, and Borgeby has always been a place worth visiting for people. Borgeby has experienced four different eras. First the time of the Vikings when the Danish king Harald Blåtand constructed a Viking ring fortress at the site. Later came the time of the church when the archbishop had Borgeby as one of his residences. When the reformation came to Sweden the castle changed owner and instead the time of the noble men began. It was not until the end of the 18th century that the first non noble owner lived at Borgeby. This was the start of the cultural era when Borgeby functioned as a meeting place for the cultural elite of northern Europe.(Rosborn (2013)

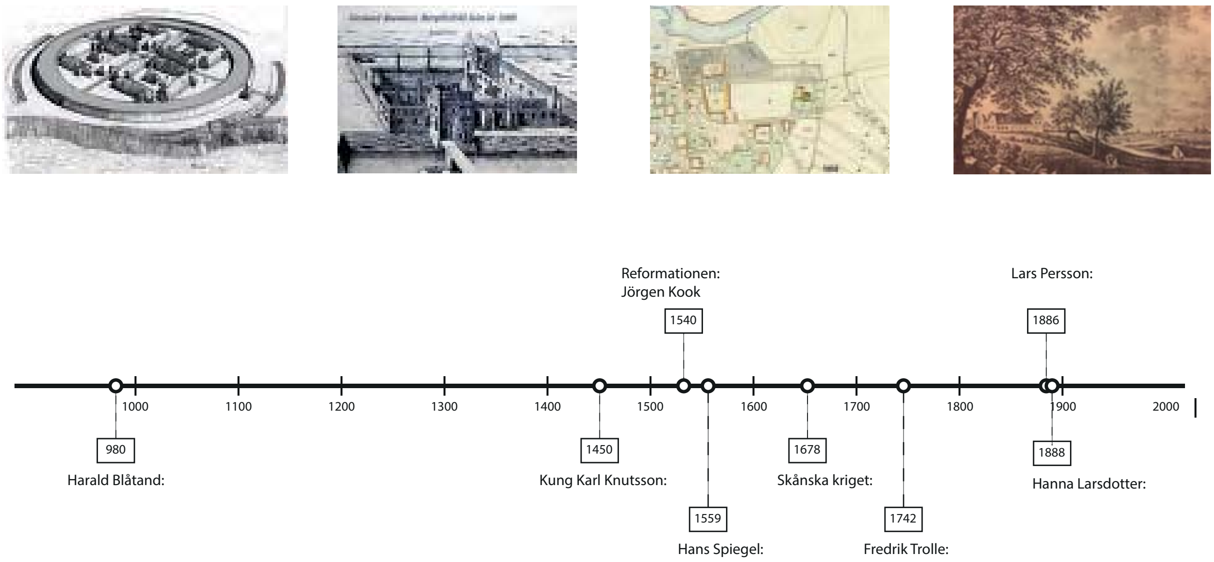


Figure A.2: Historic time line of Borgeby castle

The site and surrounding landscape

The castle is located on a plateau overlooking the landscape and Lödde river. Together with two older stable buildings they form a courtyard. In front of the castle there are a tower called Börjes Torn, once connected to the main building but destroyed during the 17th century. Close to the castle is Borgeby church located also with long history dating back to the 11th century.

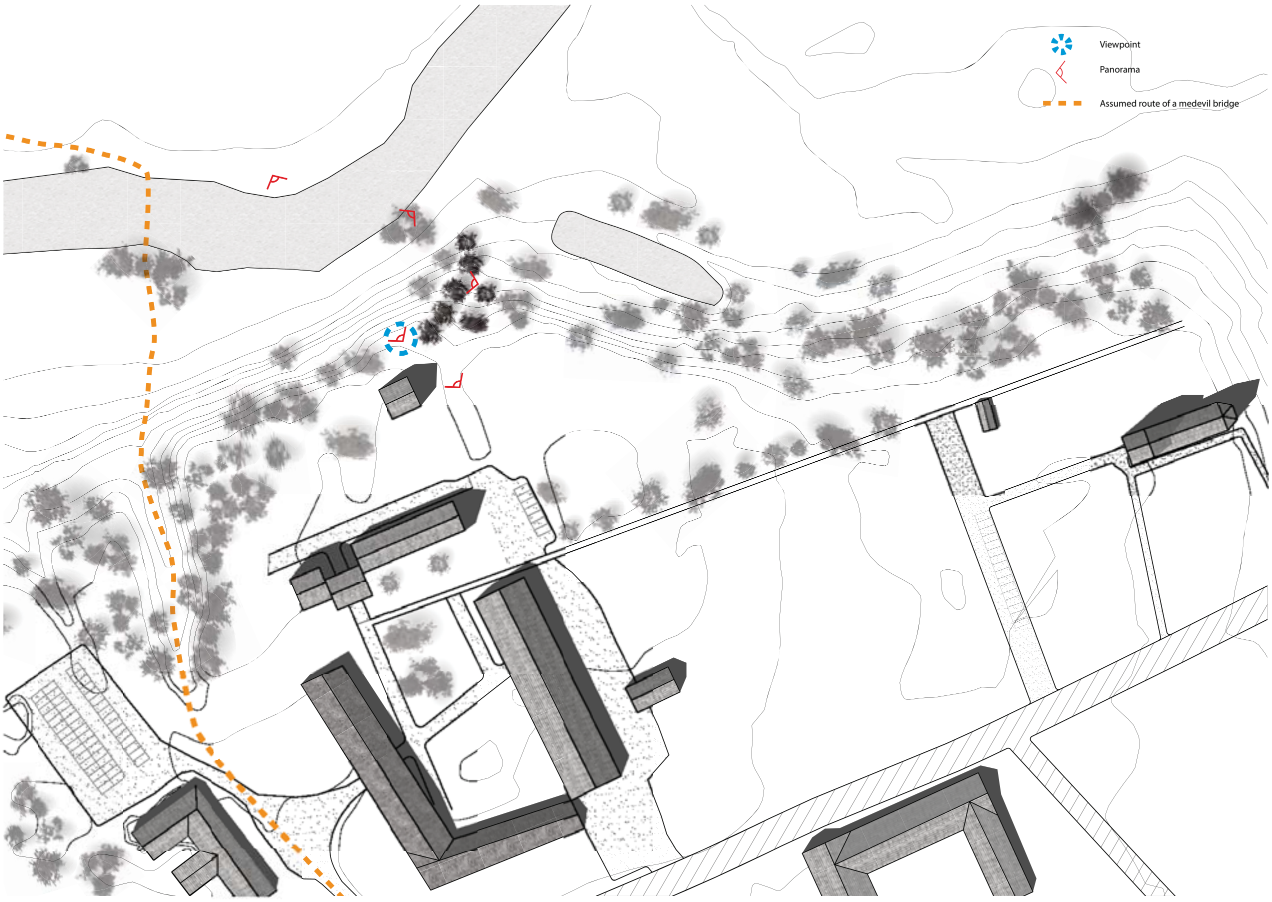


Figure A.3: Borgeby castle, (Asp (2012)

The landscape is beautiful and unusual in its topography from the otherwise flat Skåne. The castle, church and other surrounding buildings have a strong character and high materiality. It is therefore important that the bridge should be an expressive structure of the same quality and that it is able to stand out without overpowering its context. The bridge should add a new dimension of experiencing the landscape and the site.

After a analysis of the site some important spots were chosen. A view point where you can look over the whole landscape, Lödde river and the path towards the church in Löddeköpinge. The pathway down to the water was another important feature on the site. The pathway runs between two beautiful rows of trees which lead you down to the water. When walking back up the pathway it serves as a nice entry to the castle area as you approach it from an angle looking up on the tower and later the castle and stables. The connection to the water was important and it should be taken into consideration in the design. On the southern side of Lödde river it is easy to access the water where the pathway leads down to the bank and people stop there with small boats and canoes. On the northern side it is not as easy since the fields block access and the bank is soft. A bridge and connecting pathways could be a way to help people accessing the castle from two sides.

Figure A.4: Matrials on the site

Figure A.5: Panorama from Lödde å

Figure A.6: Panorama of the pathway down to the water

Figure A.7: Panorama from the view point looking over the landscape

Figure A.8: Panorama close to the water

B B: Data used in the preliminary design.

The material data used in the chapter preliminary design of the bridge.

B.1 Service classes

Service class 1 The average moisture content for most softwood species will not exceed 12 percent, which corresponds to an environment with temperature of 20 degrees Celsius and relative humidity exceeding 65 percent only a few weeks per year.

Service class 2 The average moisture content for most softwood species will not exceed 20 percent, which corresponds to an environment with temperature of 20 degrees Celsius and relative humidity exceeding 85 percent only a few weeks per year.

Service class 3 The average moisture content for most softwood species will exceed 20 percent, which gives a higher wood moisture content than that specified for service class 2.

B.2 Strength modification factor k_{mod}

Material	Service class	Permanent	Long term	Medium term	Short term	Instant
Solid wood	1	0,60	0,70	0,80	0,9	1,1
Solid wood	2	0,60	0,70	0,80	0,9	1,1
Solid wood	3	0,50	0,55	0,65	0,70	0,90
Gluelam	1	0,60	0,70	0,80	0,9	1,1
Gluelam	2	0,60	0,70	0,80	0,9	1,1
Gluelam	3	0,50	0,55	0,65	0,70	0,90

Table B.1: Strength modification factor k_{mod} for different service classes and load durations

B.3 Material properties of timber

Strength class	Bending $f_{m,k}$	Tension 0 $f_{t,0,k}$	Tension 90 $f_{t,90,k}$	Compression 0 $f_{c,0,k}$	Compression 90 $f_{c,90,k}$	Shear $f_{v,k}$
C24	24	14	0,5	21	2,5	2,5
GL24	24	16,5	0,4	24	2,7	2,7
GL32	32	22,5	0,5	29	3,3	3,8

Table B.2: Characteristic strength of timber

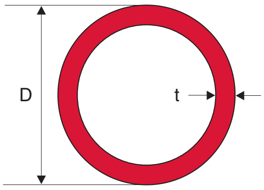
Strength class	Mean modulus of elasticity 0 $E_{0,mean}$	5% modulus of elasticity 0 $E_{0.05}$	Mean modulus of elasticity 90 $E_{90,mean}$	Mean shear modulus G_{mean}	Density ρ_k	Mean density ρ_{mean}
C14	7	4,7	0,23	0,44	290	350
C24	11	7,4	0,37	0,69	350	420
GL24	11,6	9,4	0,39	0,72	380	420
GL32	13,7	11,1	0,46	0,85	430	470

Table B.3: Stiffness properties and density of timber

B.4 Steel properties

Konstruktionsrör KCKR

Svetsade runda, kallformade
S355J2H enligt SS-EN 10 219



- F = mantelyta per m
- A = tvärsnittsarea
- g = massa per m
- I = tröghetsmoment
- W_{el} = elastiskt böjmotstånd
- W_{pl} = plastiskt böjmotstånd
- r = tröghetsradie
- I_t = vridstyvhetens tvärsnittsfaktor
- W_v = elastiskt vridmotstånd

Tvärsnittsdata

D	t	g	A	I	W_{el}	W_{pl}	r	I_t	W_v	F
42,4	3	2,91	371	7,25	3,42	4,67	14	14,5	6,84	0,133
	4	3,79	483	8,99	4,24	5,92	13,6	18,0	8,48	0,133
48,3	3	3,35	427	11,0	4,55	6,17	16,1	22,0	9,11	0,152
	4	4,37	557	13,8	5,70	7,87	15,7	27,5	11,4	0,152
60,3	3	4,24	540	22,2	7,37	9,86	20,3	44,4	14,7	0,189
	4	5,55	707	28,2	9,34	12,7	20,0	56,3	18,7	0,189
76,1	4	7,11	906	59,1	15,5	20,8	25,5	118	31,0	0,239
	5	8,77	1120	70,9	18,6	25,3	25,2	142	37,3	0,239
88,9	4	8,38	1070	96,3	21,7	28,9	30,0	193	43,3	0,279
	5	10,3	1320	116	26,2	35,2	29,7	233	52,4	0,279
101,6	4	9,63	1230	146	28,8	38,1	34,5	293	57,6	0,319
	5	11,9	1520	177	34,9	46,7	34,2	355	69,9	0,319
	6	14,1	1800	207	40,7	54,9	33,9	413	81,4	0,319
114,3	4	10,9	1390	211	36,9	48,7	39,0	422	73,9	0,359
	5	13,5	1720	257	45,0	59,8	38,7	514	89,9	0,359
139,7	4	13,4	1710	393	56,2	73,7	48,0	786	112	0,439
	6	19,8	2520	564	80,8	107	47,3	1129	162	0,439
	8	26,0	3310	720	103	139	46,6	1441	206	0,439
168,3	4	16,2	2060	697	82,8	108	58,1	1394	166	0,529
	6	24,0	3060	1009	120	158	57,4	2017	240	0,529
	8	31,6	4030	1297	154	206	56,7	2595	308	0,529
193,7	6	27,8	3540	1560	161	211	66,4	3119	322	0,609
	8	36,6	4670	2016	208	276	65,7	4031	416	0,609
	10	45,3	5770	2442	252	338	65,0	4883	504	0,609
	12,5	55,9	7120	2934	303	411	64,2	5869	606	0,609
219,1	6	31,5	4020	2282	208	273	75,4	4564	417	0,688
	8	41,6	5310	2960	270	357	74,7	5919	540	0,688
	10	51,6	6570	3598	328	438	74,0	7197	657	0,688
	12,5	63,7	8110	4345	397	534	73,2	8689	793	0,688
244,5	6	35,3	4500	3199	262	341	84,3	6397	523	0,768
	8	46,7	5940	4160	340	448	83,7	8321	681	0,768
	10	57,8	7370	5073	415	550	83,0	10146	830	0,768
	12,5	71,5	9110	6147	503	673	82,1	12295	1006	0,768
273	8	52,3	6660	5852	429	562	93,7	11703	857	0,858
	10	64,9	8260	7154	524	692	93,1	14308	1048	0,858
	12,5	80,3	10200	8697	637	849	92,2	17395	1274	0,858
323,9	10	77,4	9860	12158	751	986	111	24317	1501	1,02
	12,5	96,0	12200	14847	917	1213	110	29693	1833	1,02
Multipel Enhet	mm	kg/m	mm^2	$x10^4$ mm^4	$x10^3$ mm^3	$x10^3$ mm^3	mm	$x10^4$ mm^4	$x10^3$ mm^3	m^2/m

Figure B.1: Properties of KCKR steel pipe for the pylon from the manufacturer TIBNOR

PFEIFER
Seilbau Cable Structures

Das Zugstabsystem – Datenblätter
The Tension Rod System – Data Sheets

Zugstab Typ 860 und Typ 860-X
Tension Rod Type 860 and Type 860-X

Technische Daten

Material (Typ 860):
S460N, DIN EN 10025

Material (Typ 860-X):
S460N, modifizierter Stahl mit erhöhter Zugfestigkeit

Elastizitätsmodul:
210± 10 kN/mm²

Korrosionsschutz:
feuerverzinkt oder walzblank

Anwendungsgebiet

Aussteifungen:
Windverbände Dach, Wände, Stützen

Abspannungen:
Dachelemente, Pylone

Unterspannungen:
Stahl-, Holzbinder, Raumfachwerk

Technical Data

Material (Type 860):
S460N, DIN EN 10025

Material (Type 860-X):
S460N, modified steel with raised tensile strength

Modulus of Elasticity:
210± 10 kN/mm²

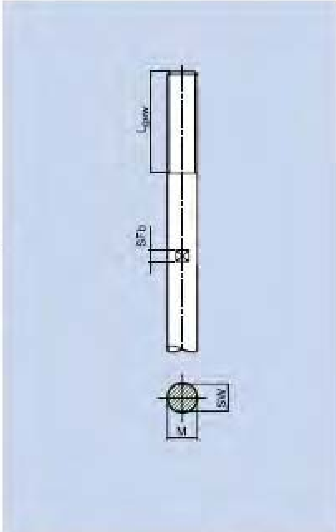
Corrosion Protection:
Hot dip galvanised or bright

Field of Application

Bracings for roofs, walls, girders

Stays for roof elements, pylons

Suspension of steel trusses, timber trusses and space frames



CE ETA

Größe *size* M mm	ETA/EC3 860 $N_{R,d}$* kN	EC3 860-X $N_{R,d}$* kN	L_{gew} mm	S_{Fb} mm	S_W mm	L_{max} mm	Gewicht *weight* kg/m
10	26,1	–	33	19	9	8000	0,61
12	37,9	41,8	38	19	10	8000	0,88
16	70,5	77,8	54	19	14	12000	1,58
20	110,1	121,6	67	19	18	12000	2,47
24	158,5	175,0	80	19	22	12000	3,55
27	208,6	228,1	90	19	25	12000	4,50
30	252,1	278,4	100	19	28	12000	5,55
36	367,3	405,5	120	19	33	12000	8,00
42	504,2	556,6	140	19	39	12000	10,9
48	658,6	731,5	159	19	45	15000	14,2
52	790,6	872,8	172	**	**	15000	16,7
56	913,0	1008,0	187	**	**	15000	19,3
60	1062,4	1172,8	199	**	**	15000	22,2
64	1203,6	1328,7	211	**	**	15000	25,3
70	1463,7	1615,9	233	**	**	15000	30,2
80	1953,8	2157,0	266	**	**	18000	39,5
90	2514,6	2776,1	297	**	**	18000	49,9
100	3146,0	3473,2	328	**	**	18000	61,7

Konstruktionsänderungen vorbehalten
* Anschlussblech S355
** Wir empfehlen die Verwendung einer Kettenrohrzange

Subject to technical modification
** Connecting plate S355*
*** We propose the use of a chain wrench*

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Figure B.2: Properties of tension rod, type 860 from the manufacturer Pfeifer

PFEIFER
Seilbau Cable Structures

Spiralseil DIN EN 12385 – Edelstahl
Spiral Strand DIN EN 12385 – Stainless Steel

CE 0769 EOTA
ETA-11/0160

PE

Die Seilsysteme – Datenblätter
The Cable Systems – Data Sheets

Technische Daten

Material:
gemäß Zulassung ETA-11/0160

Elastizitätsmodul:
130 ± 10 kN/mm²

Toleranz d_S:
+ 3%

Technical Data

Material:
According to technical approval ETA-11/0160

Modulus of Elasticity:
130 ± 10 kN/mm²

Tolerance d_S:
+ 3%

Größe *size*	Charakt. Bruchkraft *charact. breaking load* $Z_{B,k}$ DIN 18800* kN	Grenzzugkraft *limit tension* $Z_{R,d}$ DIN 18800 kN	Metall. Querschnitt *metallic cross section* ca. / *approx* mm²	Gewicht* *weight** ca. / *approx* kg	Konstruktion *construction*	Seil-Nenndurchmesser *nomin. strand dia.* d_S mm
PE 3	26	16	22	0,2	1 x 19	6,1
PE 5	47	28	38	0,3	1 x 19	8,1
PE 7	73	44	60	0,5	1 x 19	10,1
PE 10	101	61	83	0,7	1 x 19	11,9
PE 15	141	86	117	0,9	1 x 37	14,1
PE 20	195	118	161	1,3	1 x 37	16,6
PE 30	298	180	246	1,9	1 x 37	20,5
PE 45	409	248	338	2,7	1 x 61	24,1
PE 60	578	350	477	3,7	1 x 61	28,6
PE 75	730	442	602	4,7	1 x 91	32,1
PE 100	945	573	780	6,1	1 x 91	36,6

*nach EC 3 = F_{Uk} und nach ASCE 19-96 = S_0
Konstruktionsänderungen vorbehalten
Größere Abmessungen und Zwischengrößen auf Anfrage

**according EC 3 = F_{Uk} and according ASCE 19-96 = S_0*
Subject to technical modification
Bigger dimensions and intermediate dimensions upon request

10/2015

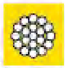
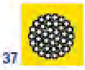
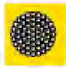
Figure B.3: Properties of spiral strand cables, type PE from the manufacturer Pfeifer

PFEIFER
Seilbau Cable Structures

Spiralseil DIN EN 12385 – GALFAN
Spiral Strand DIN EN 12385 – GALFAN

CE 0769 EOTA
ETA-11/0160

PG

1 x 19  1 x 37  1 x 61 

Technische Daten

Material:
unlegierter Qualitätsstahl

Elastizitätsmodul:
160 ± 10 kN/mm²

Toleranz d_S:
+ 3%

Korrosionsschutz:
GALFAN verzinkt ohne Innenverfüllung

Technical Data

Material:
Unalloyed quality steel

Modulus of Elasticity:
160 ± 10 kN/mm²

Tolerance d_S:
+ 3%

Corrosion Protection:
GALFAN coated without inner filling

Größe *size*	Charakt. Bruchkraft *charact. breaking load* $Z_{B,k}$ DIN 18800* kN	Grenzzugkraft *limit tension* $Z_{R,d}$ DIN 18800 kN	Metall. Querschnitt *metallic cross section* ca. / *approx.* mm²	Gewicht* *weight** ca. / *approx.* kg	Konstruktion *construction*	Seil-Nenndurchmesser *nomin. strand dia.* d_S mm
PG 5	59	36	39	0,3	1 x 19	8,1
PG 10	93	56	60	0,5	1 x 19	10,1
PG 15	134	81	87	0,7	1 x 19	12,2
PG 20	181	109	117	0,9	1 x 37	14,1
PG 25	260	158	168	1,3	1 x 37	17,0
PG 40	367	222	237	1,9	1 x 37	20,1
PG 55	537	326	347	2,7	1 x 37	24,4
PG 75	722	438	467	3,7	1 x 37	28,3
PG 90	884	536	572	4,5	1 x 61	31,3
PG 125	1189	721	769	6,1	1 x 61	36,3

*nach EC 3 = $F_{u,k}$ und nach ASCE 19-96 = S_g
Konstruktionsänderungen vorbehalten
Größere Abmessungen und Zwischengrößen auf Anfrage

*according EC 3 = $F_{u,k}$ and according ASCE 19-96 = S_g
Subject to technical modification
Bigger dimensions and intermediate dimensions upon request

Die Seilsysteme – Datenblätter
The Cable Systems – Data Sheets

Figure B.4: Properties of spiral strand cables, type PG from the manufacturer Pfeifer

PFEIFER
Seilbau Cable Structures

Die Seilsysteme – Datenblätter
The Cable Systems – Data Sheets

Vollverschlossenes Seil – GALFAN
Full Locked Cable – GALFAN

CE 0769 EOTA
ETA-11/0160

PV

VVS-1  VVS-2  VVS-3 

Technische Daten	*Technical Data*
Material: unlegierter Qualitätsstahl	*Material: unalloyed quality steel*
Elastizitätsmodul: 160 ± 10 kN/mm²	*Modulus of Elasticity: 160 ± 10 kN/mm²*
Toleranz d_S: + 3%	*Tolerance d_S: + 3%*
Korrosionsschutz: innere Lagen: feuerverzinkt mit Innenverfüllung; äußere Lagen: GALFAN verzinkt ohne Innenverfüllung	*Corrosion Protection: inner layers: Hot dip galvanised with inner filling; outer layers: GALFAN coated without inner filling*

Größe *size*	Charakt. Bruchkraft *charact. breaking load* $z_{B,k}$ DIN 18800* kN	Grenzzugkraft *limit tension* $Z_{R,d}$ DIN 18800 kN	Metall. Querschnitt *metallic cross section* ca. / *approx.* mm	Gewicht* *weight** ca. / *approx.* kg	Konstruktion *construction* **	Seil-Nenndurchmesser *nomin. strand dia.* d_S mm
PV 40	405	245	281	2,4	VVS-1	21
PV 60	621	376	430	3,6	VVS-1	26
PV 90	916	555	634	5,3	VVS-2	31
PV 115	1170	709	808	6,8	VVS-2	35
PV 150	1520	921	1060	8,9	VVS-2	40
PV 195	1930	1170	1340	11,2	VVS-2	45
PV 240	2380	1442	1650	13,8	VVS-2	50
PV 300	3020	1830	2090	17,2	VVS-3	55
PV 360	3590	2176	2490	20,5	VVS-3	60
PV 420	4220	2558	2920	24,1	VVS-3	65
PV 490	4890	2964	3390	27,9	VVS-3	70
PV 560	5620	3406	3890	32,1	VVS-3	75
PV 640	6390	3873	4420	36,4	VVS-3	80
PV 720	7210	4370	4990	41,1	VVS-3	85
PV 810	8090	4903	5600	46,2	VVS-3	90
PV 910	9110	5521	6310	52,0	VVS-3	95
PV 1010	10100	6121	6990	57,6	VVS-3	100
PV 1110	11100	6727	7710	63,5	VVS-3	105
PV 1220	12200	7394	8460	69,7	VVS-3	110
PV 1340	13400	8121	9240	76,2	VVS-3	115
PV 1450	14500	8788	10100	83,2	VVS-3	120
PV 1580	15800	9576	10900	89,8	VVS-3	125
PV 1730	17300	10485	11900	96,7	VVS-3	130
PV 1860	18600	11273	12900	104,8	VVS-3	135
PV 2000	20000	12121	13900	112,9	VVS-3	140

**VVS-1 = 1, VVS-2 = 2, VVS-3 = 3 und mehr Lagen Profildrähte
*nach EC 3 = $F_{u,k}$ und nach ASCE 19-96 = S_d
Unter Vorspannung und / oder Witterungseinflüssen ist der Austritt von Innenverfüllung möglich.
Konstruktionsänderungen vorbehalten
Größere Abmessungen und Zwischengrößen auf Anfrage

***VVS-1 = 1, VVS-2 = 2, VVS-3 = 3 and more layers z-profiled wires*
**according EC 3 = $F_{u,k}$ and according ASCE 19-96 = S_d*
Due to prestressing and / or differing weather conditions inner filling may escape to the surface.
Subject to technical modifications
Bigger dimensions and intermediate dimensions upon request

10/2015

Figure B.5: Properties of spiral strand cables, type PV from the manufacturer Pfeifer