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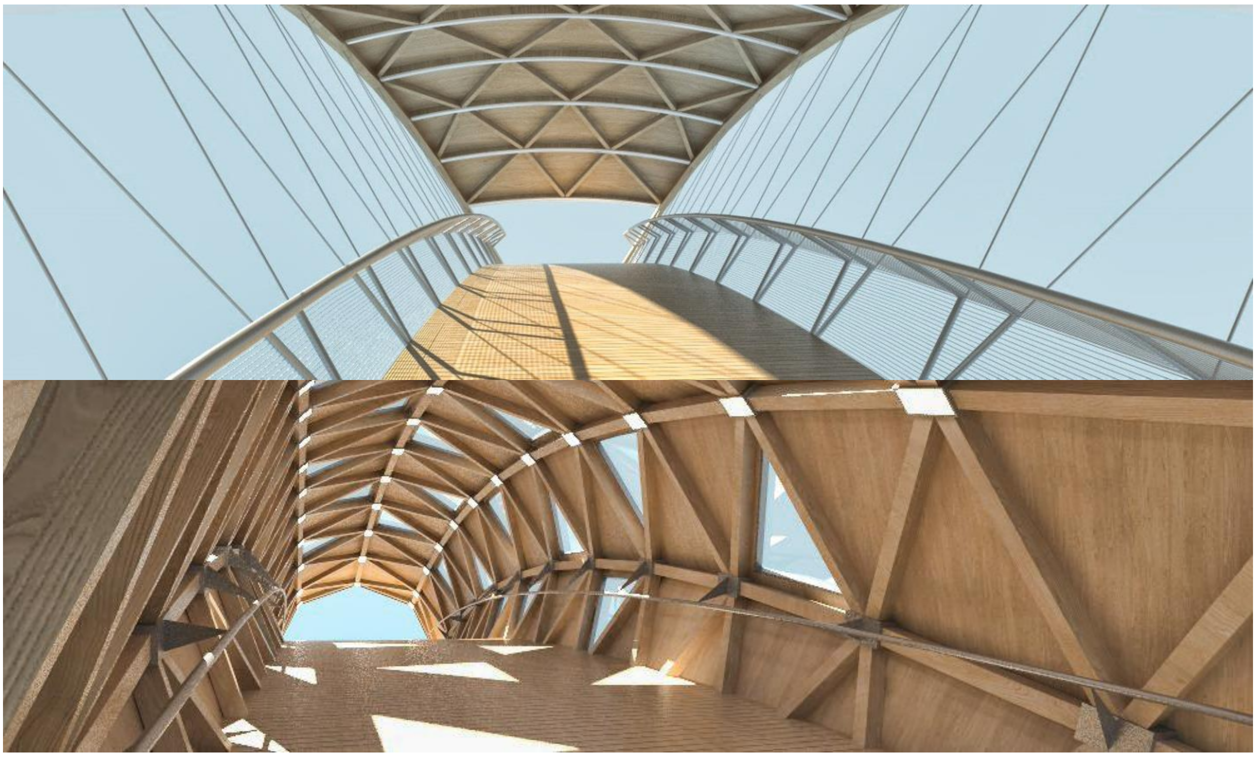

Covered Timber Bridges

Inspiration, conceptual and final design

Master of Science Thesis in the Master's Programme Structural Engineering and Building Technology

JACOB FLÅRBACK

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2015
Master's Thesis 2015:2

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Cover:
Rendered pictures of the two final bridge designs, further pictures and information can be found in Chapters 8 and 9.

Chalmers Reproservice
Department of Civil and Environmental Engineering Göteborg, Sweden 2015

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ABSTRACT

Timber has always been an important construction material in Sweden. Many countries around the world with the same connection to the material have incorporated this into the building of timber bridges, for instance Switzerland, Japan and the US. Numerous covered timber bridges can be found spread out through these countries, utilizing roofs as a constructive measure to protect the material. Sweden on the other hand is lacking these types of bridges and relies on small scaled constructive means such as protective steel details etc. The goal of this project was therefore to propose some innovative concepts for such roof covered timber bridges and show their potential by designing two examples.

The first aim of this thesis was to get a good sense of different ways of constructing covered timber bridges by studying built examples around the world. Inspired by the analysis of existing bridges a set of architectural design concepts were developed. These concepts were modelled and analysed until the four most promising ones were chosen for continued work.

Using the concepts, four actual bridge designs for a pedestrian and bicycle bridge were developed using the 3D-modelling program *Rhinoceros* and the plugin *Grasshopper*. The bridges spanning 30 meters were then analysed in the Grasshopper FEM-plugin *Karamba* for both ULS and SLS, to assess the capacities of the bridges according to *Eurocode*. The bridge designs were also turned into physical models to investigate their architectural qualities.

The two most successful bridges were then optimised on a more detailed level, looking at for instance connection types and railings. As these final modifications were finished the bridge designs were finally modelled into presentation renderings.

The project, and the two final bridge designs, showed that there is a high potential to develop interesting solutions for covered timber bridges. Not only is the durability issue of the timber controlled better but the bridge roofs can also be incorporated in the structural behaviour to make functionally better timber bridges. The project also proves that the addition of a roof does not have to negatively affect the appearance of a bridge but rather the opposite. Furthermore by optimizing the bridge elements the construction cost increase for a roof can be both limited as well as motivated by the improvements in performance and design.

Key words: Covered timber bridge, Timber engineering, Pedestrian bridge, Timber bridge, Bridge design, Bridge architecture, Grasshopper, Karamba.

Covered Timber Bridges

Inspiration, conceptual and final design

Master of Science *Thesis in the Master's Programme*
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Structural Engineering
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SAMMANFATTNING

Materialet trä har alltid varit en central del av svensk byggnadskonst. Många länder runt om i världen med samma koppling till materialet har inkorporerat detta i byggandet av träbroar, såsom Schweiz, Japan och USA. I dessa länder finns massor med täckta träbroar, där ett skyddande tak utnyttjas som konstruktivt träskydd. I Sverige är denna brotyp nästan helt obefintlig, istället förlitar man sig på småskaligt konstruktivt skydd via t ex skyddsplåtar och mindre ståldetaljer. Målet med detta examensarbete var därför att föreslå ett antal nytänkande koncept för taktäckta träbroar och visa på deras potential genom att designa två broförslag.

Det första syftet med denna avhandling var att få ett bra grepp om olika metoder för att skapa en täckt träbro, genom att studera olika byggda träbroar runt om i världen. Med denna studie i ryggsäcken utvecklades ett antal arkitektoniska koncept för potentiella nya broar. Dessa koncept modellerades och analyserades för att kunna välja de fyra mest lovande förslagen för vidare utveckling.

Från de fyra valda koncepten designades fyra stycken broförslag för en täckt gång-/cykelbro, med hjälp av 3D-modelleringsprogrammet *Rhinoceros* och insticks-modulen *Grasshopper*. Dessa broförslag, gjorda för ett 30-meter långt spann, analyserades sedan i FEM-insticksmodulen *Karamba* tillhörande Grasshopper. Såväl ULS som SLS kontrollerades för att avgöra broarnas kapacitet enligt *Eurocode*. Broförslagen modellerades även fysiskt för att undersöka arkitektoniska kvalitéer.

Därefter valdes de två bästa förslagen ut för vidare optimering analys på en mer detaljerad nivå, inklusive t ex detaljutformning och räckesdesign. Efter slutförd detaljoptimering modellerades slutförslagen upp och renderades för slutpresentation.

Projektet och de två slutgiltiga broförslagen visar på stor potential för utveckling av täckta träbroar. Inte minst hanteras hållbarhetsaspekten för trä på ett effektivt sätt men brotaket kan konstrueras så att det utnyttjas för att strukturellt förbättra brons bärkapacitet. Avhandlingen visar också hur träbroar kan påverkas positivt, ur ett estetiskt perspektiv, av att inkludera ett tak. Detta sammanfattat innebär att kostnadstillägget av ett tak kan motiveras både genom förbättrad funktion och design.

Nyckelord: Täckt träbro, Träkonstruktioner, Gångbro, GC-bro, Träbro, Brodesign, Broarkitektur, Grasshopper, Karamba

Contents

Preface

In this project a comprehensive covered timber bridge design process was performed incorporating my experience and that of my supervisors within the fields of Structural Engineering and Architecture. The process included an independent literature study to prepare for the design, followed by the iterative process of creating optimised covered timber brides.

The Thesis process spanned from August 2014 to February 2015. The work was performed at the Division of Structural Engineering, Steel and Timber Structures at Chalmers University of Technology. The project was carried out with Professor Robert Kliger as the main supervisor and examiner. Accompanying supervisors in the project were Kristoffer Ekholm, researcher and timber bridge specialist, and professor at *Architecture and Engineering* Karl-Gunnar Olsson.

I would like to thank both my examiner and supervisors for the effort and time put into aiding my work during this process. I also wish to thank my opponents Oskar Mangold and Peter Selberg for their helpful comments and support.

Göteborg February 2015

Jacob Flårback

Notations

Roman upper case letters

A	Section area
A_{net}	Steel section net area
C_e	Snow load exposure coefficient
C_t	Snow load thermal coefficient
E	Elastic modulus
$E_{g,0.05}$	Glulam elastic modulus, 0.05 percentile
$E_{g,mean}$	Glulam elastic modulus, mean value
E_Q	Equivalent spiral strand rope modulus of elasticity
F_k	Cable breaking load table value
$F_{railing\ etc}$	Assumed additional force from added components, such as railings
F_{Rd}	Dimensioning tension force capacity
F_{uk}	Characteristic tension force capacity
G	Shear modulus
$G_{g,0.05}$	Glulam shear modulus, 0.05 percentile
$G_{g,mean}$	Glulam shear modulus, mean value
$G_{k,j}$	Permanent loads, characteristic values
I_y	Area moment of inertia around y-axis
I_z	Area moment of inertia around z-axis
K	Failure force factor
K_v	Radius of gyration
L_0	Geometry length
$N_{pl,Rd}$	Dimensioning normal force capacity regarding plastic section
$N_{u,Rd}$	Dimensioning normal force capacity regarding local failures
P	Point load
P_k	Pre-tensioning loads
$Q_{k,1}$	Primary variable load, characteristic values
$Q_{k,i}$	Accompanying variable loads, characteristic values
R_r	Strand tensile strength

Roman lower case letters

c_e	Wind load exposure factor
c_{pe}	Wind load shape factor
d	Diameter
d_{in}	Inner side length
d_{out}	Outer side length
f	Cable fill factor
f_d	Design strength
f_k	Characteristic strength
$f_{c,0,g,k}$	Characteristic glulam compression strength parallel to grains
$f_{c,90,g,k}$	Characteristic glulam compression strength perpendicular to grains
$f_{m,g,k}$	Characteristic glulam bending strength
$f_{v,g,k}$	Characteristic glulam shear strength
$f_{t,0,g,k}$	Characteristic glulam tension strength parallel to grains
$f_{t,90,g,k}$	Characteristic glulam tension strength perpendicular to grains
k_e	End connection loss factor
k_h	Timber size factor
k_{mod}	Timber load duration factor
q_k	Characteristic imposed load
s	Characteristic snow load
s_k	Table value for characteristic snow load
v_b	Wind speed
w_e	Characteristic wind load
z_e	Height over ground

Greek letters

γ_{cable}	Cable equivalent density
γ_G	Permanent load partial factor
γ_M	Material partial factor timber
γ_{M0}	General material partial factor steel
γ_{M2}	Tension failure partial factor steel
γ_P	Pre-stressing load partial factor
γ_Q	Variable load partial factor
γ_R	Material partial factor steel cables
γ_{steel}	Steel density
μ_1	Snow load form factor
ρ	Air density
$\rho_{g,mean}$	Glulam average density
ν	Poisson's ratio
$\psi_{0,i}$	Variable loads psi-factor

1 Introduction

Throughout history timber has always been one of the most essential construction materials, in particular for Scandinavia. It is a versatile construction material and is widely utilized both for its aesthetic qualities as well as constructive properties.

In the current pursuit for environmentally friendly solutions the use of timber as a building material is a superb choice. Not only is the material renewable but also produced in large quantities in Sweden, in fact, the timber industry is among one of the largest ones in the country. This along with the material being significantly cheaper than for instance steel or concrete shows some of the potential with timber in constructions. Naturally the material also has some disadvantages compared to its alternatives. Important ones are significantly lower strength and stiffness than steel or concrete in compression. In addition, unless treated or protected the material suffers from deterioration when exposed to the weather, in particular too much water.

Up until the last few years the Swedish Transport Administration *Trafikverket* had a set of rules that restricted the use of timber for bridges. Many of these requirements concerned safety related issues, where the potential durability problems and decreasing strength of timber over time were looked upon conservatively. Sweden also has restrictions on the allowed treatments of timber, adding demands on detailing for timber bridges for exposed timber components.

Today the rules no longer hinder the construction of these timber bridges, instead the problem is that there is too little experience and a general misconception about using timber for instance in bridge design. Since few timber bridges are built, modern and efficient assessment and construction methods are limited and not further developed as much as they could have been. This leads to contractors often choosing the more well-known solutions of steel or concrete, to avoid the uncertainty of timber inexperience, which in many cases are not actual problems but simply lack of updated knowledge on the subject. In order to change this trend there needs to be some type of initiation, where the eyes of the building industry are opened to the potential of timber, because with experience comes improved techniques and lower costs for future projects.

One of the issues with timber that contributes to the fear mentioned above is the durability question, where the lack of experience in timber bridge design and maintenance leads to a fear among builders to choose timber over other materials (Fjellström P-A, 2007). One solution that is used in some countries is the addition of a roof. By adding a large climate shell to the bridge the durability issues can be reduced or even avoided. Additional advantages such as snow removal on the bridge add to the list of benefits from a covered bridge, yet there is not a single public covered timber bridge in all of Sweden.

1.1 Aim

The main purpose of this thesis is to investigate if a covered timber pedestrian and bicycle bridge can be designed with a strong aesthetic profile while still providing the desired performance and a reasonable budget, thus being a viable alternative to steel or concrete. The aim is for the following questions to be answered:

1.1.1 Phase 1 – Inspiration

- *What types of timber bridges (mainly covered) have been built before? With what aesthetic profile?*
- *What different construction principals have been used for timber bridges in the past? What are their benefits and disadvantages?*
- *What design problems need solving for a covered timber bridge compared to an open one? What are some possible solutions?*
- *In what ways can the roof and walls contribute to the bridge functionality other than for climate protection?*
- *Why is there not a single public covered timber bridge in Sweden?*

1.1.2 Phase 2 – Conceptual design

- *How can the roof and walls of the bridge be used as structural members?*
- *In what ways can the design be expressed through the structural components?*

1.1.3 Phase 3 – Final design

- *What is required in terms of detailing and dimensions for the bridge?*
- *What is needed from the cover for the bridge to protect the structural system from weather effects?*
- *How can the final solutions be optimised regarding the economical and required production time and effort aspects?*
- ***Can an aesthetically pleasing and fully functional covered timber pedestrian and bicycle bridge be built, having reasonable requirements from the construction workers while being sufficiently optimised to be economically feasible?***

1.2 Limitations

The project design phases are limited to covered timber bridges. The design of a vehicular traffic bridge is vastly more complex than for pedestrian and bicycle bridges and for the purpose of simplifying this task only the latter will be included. This is also motivated by the major calculation simplification given by adding a roof to a pedestrian and bicycle bridge, where the usual loads from snow removal vehicles can be completely disregarded.

The goal is to investigate several different solutions, something that would be very time consuming if done thoroughly, therefore this thesis will focus on the design and global calculations. This means analysing the structural behaviour of the whole structure and its timber elements, but only looking at the connection details in terms of design suggestions. The calculations needed for these will not be included.

The bridge geometry has a few given parameters to relate to in order to base the design on something. These points are realistic requirements that aim for the final bridges to be versatile:

- 30 meter bridge span
- Limited deck slope for handicap access
- 4 meter minimum deck width
- 2,5 meter minimum average roof height

1.3 Method

In order to get a better grip on the subject and to get inspiration from existing structures, firstly information has to be gathered (*phase 1*). The focus of this work will be on finding existing solutions and problems regarding timber bridge design, specifically covered timber bridges. By looking at methods used for design in previous projects a more efficient process can be assured for the design in this thesis. The main source of information in this phase will be literature studies.

The project will then enter the concept phase (*phase 2*). At this point a set of design concepts for potential bridges, preferably greatly varying from one another, will be developed. The most promising concepts will then be designed into 30 meter bridges and analysed in order to get a sense of which concept has the most potential for the final design suggestions. This will be done in the modelling program *Rhinoceros*, using the plugin *Grasshopper* and its plugin *Karamba*. Through calculations in these programs, along with a judgment of the architectural qualities two winning concepts will be determined.

Following this is the final stage of the project, further development and analysis of the most promising concepts (*phase 3*). The designs will be improved and optimised further, taking into account even more aspects in the design. Aiming to avoid extensive cumbersome work being spent on detailed calculations the focus of this phase will be on more general design calculations. Details and connections will not be checked by calculations but rather suggested based on desirable design and assumed structural behaviour. Since the final designs will be used in an attempt to sell in the concept of covered timber bridges, some form of pleasing graphical display of them will be created.

2 Historical Background

The art of timber bridge construction dates back far in history. Along with stone it was the main constructive bridge material until the rise of industrialisation and the introduction of iron, steel and reinforced concrete. It has long been known that timber exposed to the effects of weather, sunlight and moisture, will deteriorate if the conditions are unfavourable. Before modern treatment methods were developed the safest way to protect the structural timber was to simply shield it, thus creating a covered bridge. The oldest example of this to be found in Europe is *Kapellbrücke* in Switzerland (see Figure 2.1), which dates back to year 1333 (Baus U, Schlaich M, 2008). In other parts of the world this technique has been used even further back, records exist of covered timber bridges in China dating back over 1000 years (Nianzu G, 2009).

Figure 2.1 Kapellbrücke, Luzern, Switzerland [1]

While the phenomenon of covered timber bridges exist in many parts of the world it is only common in a few countries, an interesting way to look at the history of covered timber bridges is thus to investigate this tradition in the areas where it differs the most. With this goal in mind one can find two major different approaches to the design of timber bridges, the functional approach in North America and the aesthetical approach in Southeast Asia.

2.1 North American Covered Bridges

The covered timber bridge tradition of America is as old as the country itself. It all started with engineer Timothy Palmer. He is credited for the design of a 167 meter long, three span bridge in the proximity of Philadelphia built in year 1805, the first American covered timber bridge (Allen R.S, 2004). The palmer truss bridges consisted of an arched truss strengthened by the supports (see Figure 2.2). The structural system was then covered with a façade and a roof for protection, while it did provide some additional stiffness, the cover was intended for protection and not as a load bearing member.

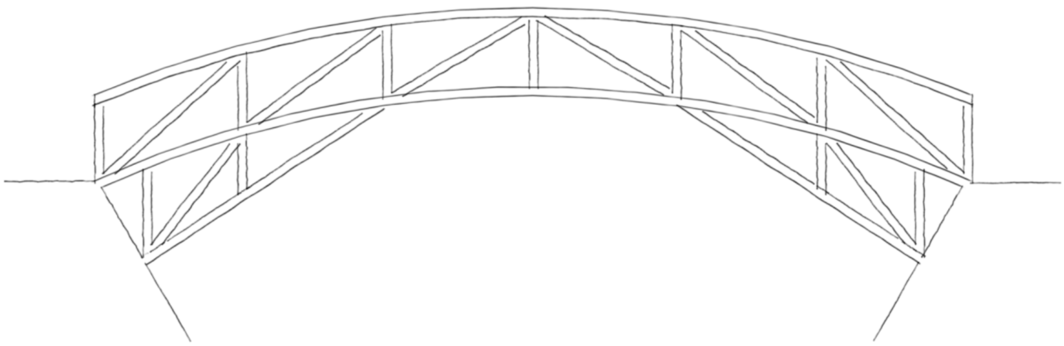


Figure 2.2 Palmer truss system

As the number of covered bridges in the US increased, another name emerged in the field, Theodore Burr. The Burr truss bridges were a combination of an arched beam and a regular truss frame (see Figure 2.3) (Allen R.S, 2004). The *Burr truss system* had the advantage of a framing for the walls being included in the load bearing system, thus the roof could be created by simply adding tie beams and rafters.

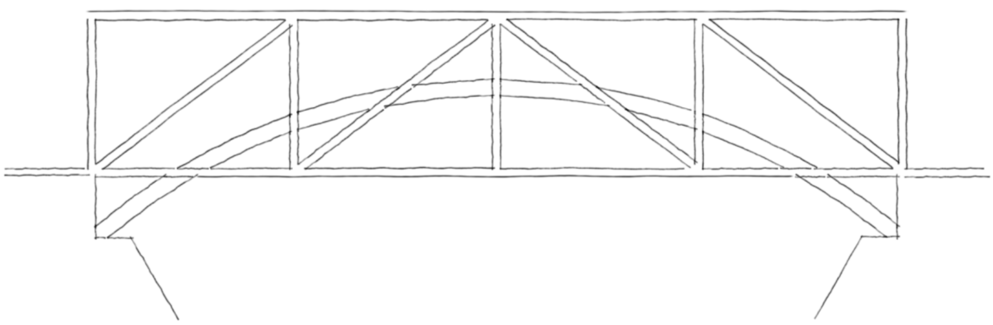


Figure 2.3 Burr truss system

The trusses above still relayed on arched beams in order to function, but around the late 1830's the development of more intricate truss systems enabled the arched beams to be removed (Allen R.S, 2004). The first system to incorporate this was the *Long truss system* (see Figure 2.4). This system replaced the arched beam in a Burr bridge with extra diagonals in each box of the truss.

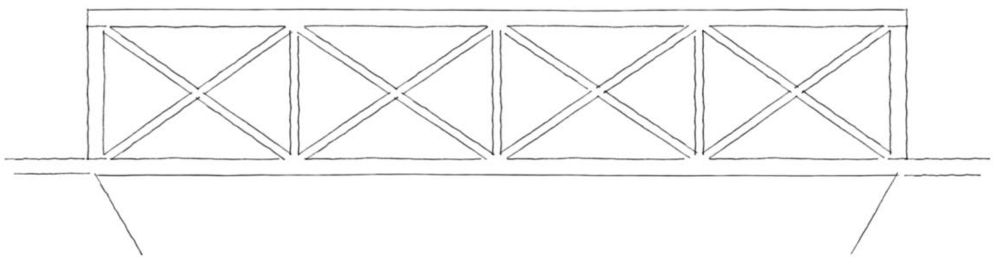


Figure 2.4 Long truss system

The advantage of having a complete wall frame with no arched beams in the load bearing system led to a set of other more intricate truss systems emerging. The two most prominent of these were the Town lattice truss system (see Figure 2.5) and the Howe truss system (see Figure 2.6) (Allen R.S, 1962).

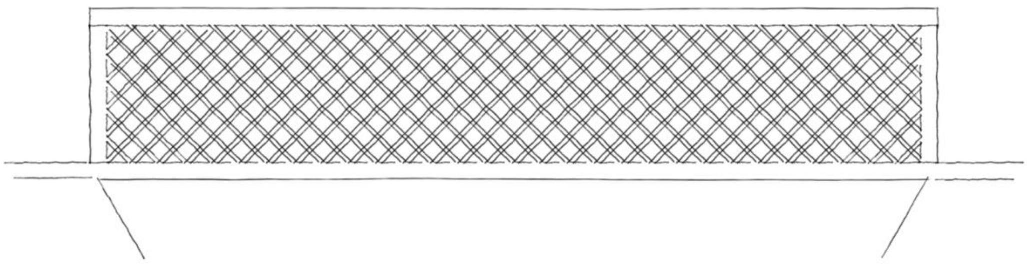


Figure 2.5 Town lattice truss system

The *Town lattice truss system* consists of a dense pattern of diagonals with no vertical posts. By creating meshed walls that acted as stiff plates this system was able to span up to 60 meters (Allen R.S, 2004).

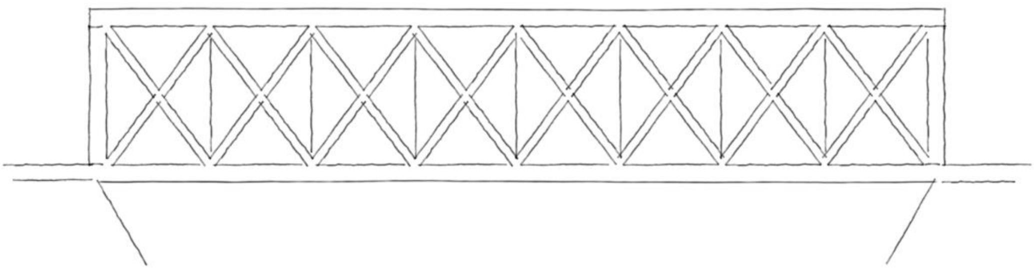


Figure 2.6 Howe truss system

The *Howe Truss System* is similar to a long truss but with the vertical posts being replaced with steel rods. Using the two materials in combination, this system was part of the starting transition into the steel era. Eventually timber in bridges would be almost completely gone as the era of steel and reinforced concrete began (Allen R.S, 2004).

The different bridge trusses above form the core of the covered bridge tradition in North America, but of course other truss types were also used. One of the more elegant ones was the *Bowstring Truss System* (see Figure 2.7). This system reinstated the arched beam in combination with an inverted Howe system, with timber vertical posts and steel diagonal rod crosses (Ritter M. A, 1990).

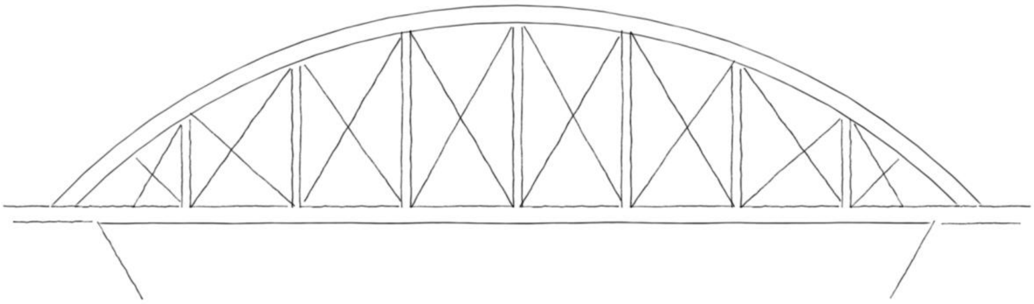


Figure 2.7 Bowstring truss system

2.1.1 Hartland and Cogan House Covered Bridges

The North American covered bridges all share the same aesthetical profile, the "Barn over river" design. The structural systems mentioned above were given a timber façade and roof, potentially with some openings to light up the interior. This was the case regardless of the bridge length or surroundings, leading to structures such as the longest covered bridge, the *Hartland Covered Bridge* (see Figure 2.8).

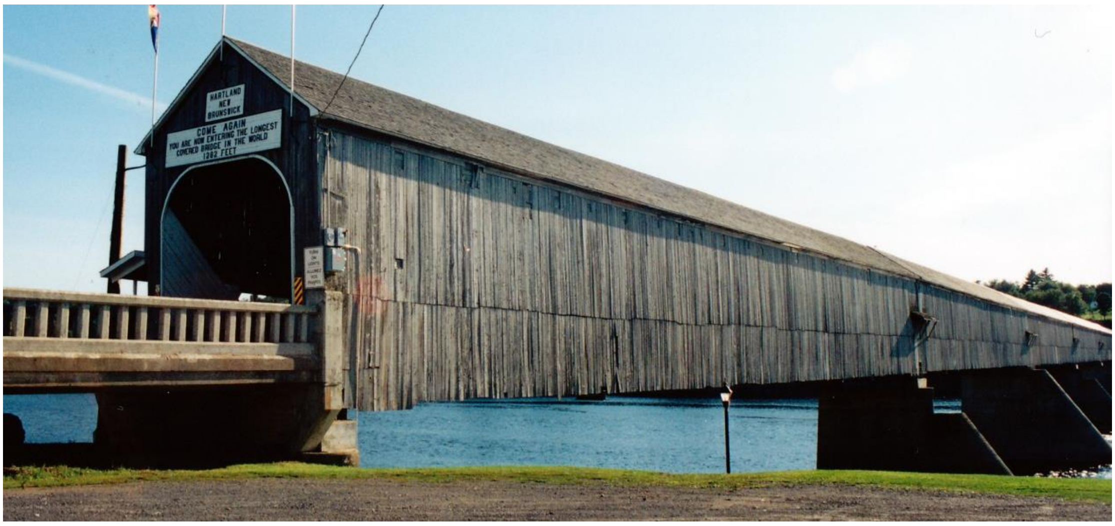


Figure 2.8 Hartland Covered Bridge, New Brunswick, Canada [2]

The Hartland Covered Bridge is a Howe Truss bridge with a total length of 390 meters, divided into seven spans (Town of Hartland, 2014). From an architectural point of view the building shows very limited creativity, the concept can be described as a simulation of an outhouse or a barn. The interior with the exposed structural system gives a little more to the visitor, but still only utilizes fractions of the potential with a covered bridge (see Figure 2.9)

Figure 2.9 Inside the Hartland Covered Bridge, New Brunswick, Canada [3]

As Figure 2.9 shows the play with light is limited to a set of tiny windows in a row close to the ceiling, in what otherwise becomes a long dark hallway. This aesthetic profile can be found throughout hundreds of covered timber bridges in North America, all with the same concept: A structural system that is easy to erect, and a design that imitates a stretched out barn. The same principle applies for the *Cogan House Covered Bridge* (see Figure 2.10)

Figure 2.10 Cogan House Covered Bridge, Lycoming County, USA [4]

The Cogan House Covered Bridge in Pennsylvania was built in 1877 (Cogan House Township, 2014). Just as with many other North American covered bridges the documented motivation for the covering describes weather protection and additional stiffness to the structure as the reason (Pennsylvania Covered Bridges, 2014). The only mentioned references to aesthetics are to the barn-like nature of the bridge. Rumors say that the local farmers wanted the bridge to look like the barns at the farms in order for the cattle to be less discouraged to pass (Pennsylvania Covered Bridges, 2014).

The interior of the bridge reveals a typical Burr truss system (see Figure 2.11). By opening up the façade the last meter under the roof this bridge has a much less claustrophobic inside than that of the Hartland covered bridge, while still protecting the structural system from the weather.

The structural behaviors of Burr trusses have in modern times been analyzed to determine the actual effects of the combination of arch and truss. By investigating how these bridges behave for arches that are unattached to the truss compared to when the arch and truss are in full interaction the arch was determined to be the main component carrying uniform loads. Seeing as a bridge is built for not only the uniform self-weight but heavy point loads from vehicles the deformations of an arch would be excessive and thus the truss works to reduce these deformations (Machtemes A, 2011). Of course these behaviors interact with one another but a majority of the loads have been shown to be handled in the setup: uniform load – arch, point load – truss.

Figure 2.11 Inside of Cogan House Covered Bridge, Lycoming County, USA [5]

A fun factoid on the aesthetics of the classic American covered bridges is that the *Wikipedia* site for the Cogan House covered bridge features a facts table with one post saying: "Design effort - Low". In a sense this captures the essence of the covered bridge tradition in North America. The bridges function well and the cover gives them a good durability, but little effort has been put into a creative design.

2.2 Southeast Asian Covered Bridges

As previously mentioned the covered bridge tradition of Southeast Asia dates back over a millennium, back to the *Song Dynasty* of China (A.D. 960 – 1127) (Xinping Y, 2009). The first wooden arch bridge of this age is said to have been the *Hong Bridge* or *Rainbow Bridge*. It is unclear if the bridge was covered at its construction but the structural system used was the very same as many following covered bridges in China, a *Beam Weaving System* (Xinping Y, 2009). Because of this origin the timber beam weave bridges in the country are commonly referred to as *Rainbow Bridges*. This structural system is created by interlocking two beam systems with transversal beams (see Figure 2.12).

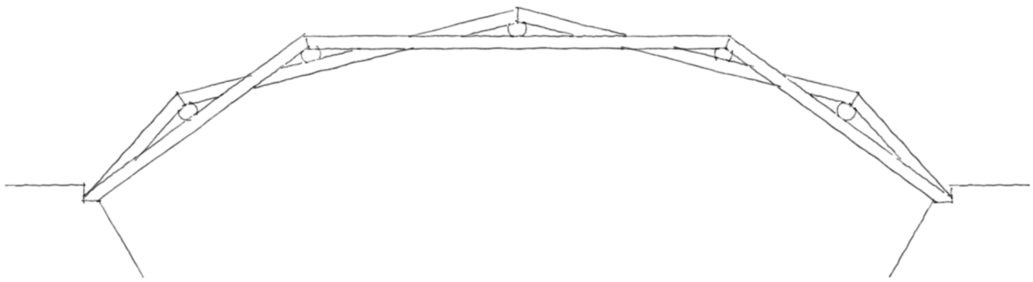


Figure 2.12 Timber Arch Weave (Rainbow) system

In a rainbow bridge the load is carried in a combination between arching action of the global structure and pure bending in the individual beams. Rainbow bridges without a cover often use the top of the arch of the structural system as the deck, however when a roof is planned it is common to lower the end supports so that the top of the arch aligns with the ground level. Through this so called *Timber Arch-Beam Weave System* (see Figure 2.13) numerous covered rainbow bridges with a horizontal deck have been made, making it the most common covered timber bridge system in China. The highest concentration of these can be found in Southeast China in the *Fujian Province* (Liu Y.).

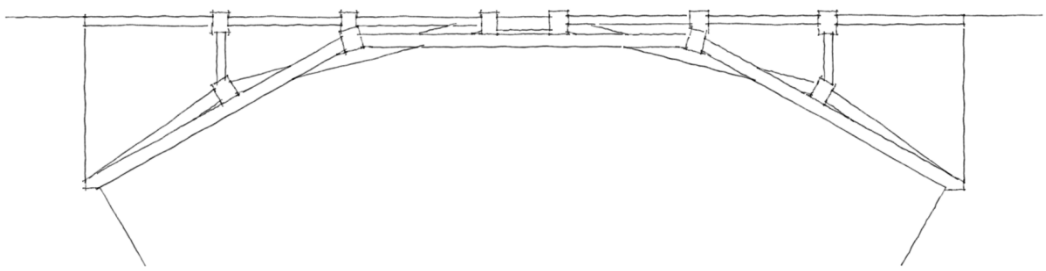


Figure 2.13 Timber Arch-Beam Weave system

As Figure 2.13 shows the Arch-Beam weave system also utilizes the transverse beams as the joints for the structure. This gives the bridge arch a smaller height and makes the top more flat, making the addition of a deck easier.

Apart from the weaves mentioned above, there is another common structural system for timber bridges in Southeast Asia, the *Cantilever Stacked Beam system* (see Figure 2.14). This type of structure relies on stacking and counterweights in order to span over openings, bridges with large roof structures can thus be beneficial for the load bearing system of this kind.

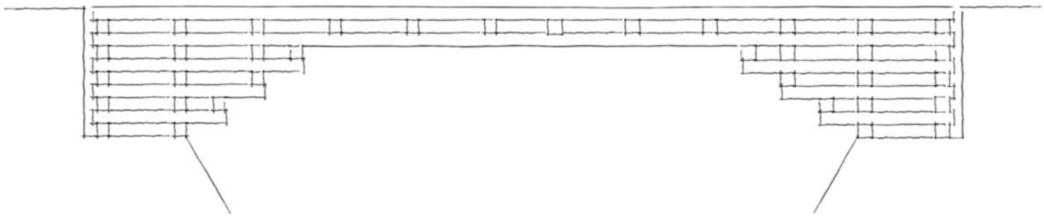


Figure 2.14 Cantilever stacked beam system

As the figures above show there are no structural elements over the deck level, the addition of the roof was therefore an extra load for the bridges (partly beneficial for the stacked beam system). Just as with traditional timber structures in general for Southeast Asia, the roof systems were made with intense detailing and craftsmanship into beautiful elements of the bridges. The demands on the craftsmanship were high not only for the roof but the three bridge systems as well, where the vast majority of all these bridges were built entirely out of timber joints completely free from metal details (Xinping Y, 2009). As a result many bridges in this area are very beautiful, for instance the *Saya Bridge* in Japan (see Figure 2.15).

Figure 2.15 Saya Bridge, Kotohira, Japan [6]

2.2.1 Xidong and Chengyang Bridge

To further understand the Southeast Asian tradition of covered timber bridges two of them have been looked at further, the timber weave bridge *Xidong Bridge* in Zhejiang (see Figure 2.16) and the stacked beam bridge *Chengyang Bridge* in Sanjiang (see Figure 2.18).

Figure 2.16 Xidong Bridge, Zhejiang, China [7]

The Xidong Bridge, also known as the East River Bridge, was first constructed in year 1570 but had to be refurbished in year 1754 (Chen B, Gao J, Yang Y, 2007). The bridge roof extends far past the span on each side to a total length of 42 meters, while the total span is 26 meters long. As Figure 2.16 hints the bridge is supported on a Woven Beam-Arch system, the picture shows how one layer of beams rest on the foundation. The deck over the bridge span is not entirely horizontal but flat enough not to rely on a stair system. The two entrances made from stone feature stairs to get up to the deck level and also work as support for the roof extensions. The interior of the bridge is kept bright via the openings in the façade (see Figure 2.17).

The roof structure, as Figure 2.17 shows, rests on columns on the bridge. Therefore the structural contribution from the roof is very limited, some lateral stiffness might come from it but the main purposes are cover and aesthetics. In addition to what is needed to cover the bridge the mid-span roof section has an elevated second roof level. This component will increase the load, its existence is therefore completely based on the appearance of the whole structure, a clear example of the aesthetical approach in Southeast Asia.

Figure 2.17 Xidong Bridge interior, Zhejiang, China [8]

The Chengyang Bridge in Sanjiang from year 1916 is one of the most impressive covered timber bridges in China (see Figure 2.18), it covers a length of 65 meters divided into four spans (China Network, 2002). It blends beautiful architecture and craftsmanship with an elegant structural system. The Stacked Beam system of this bridge is less material efficient than the Beam Weave of Xidong Bridge, however in terms of aesthetics it functions well in harmony with the rest of the structure. In order to optimise the structural system a set of pavilion modifications have been made to the bridge over the supports. These not only create a more dynamic space in the bridge but also work as counterweights for the cantilevering beams with joints over the supports.

Figure 2.18 Chengyang Bridge, Sanjiang, China [9]

The Stacked Beam system is for each layer connected at the ends with a transversal beam (see Figure 2.19). In this way the load is more efficiently distributed between all the beams of each layer. The figure also shows how the timber has been jointed without the use of steel. This is the case for the whole bridge structure where skilled craftsmanship led to a building like this without a single nut or bolt. The figure also shows how much of the timber used are full logs that have only been stripped from the bark. This is very common for most of the structural timber in Southeast Asia (as many of the figures here show) and is therefore a clear difference in the approach of North America where sawn timber was almost exclusively used. The whole logs will benefit in strength compared to the sawn timber where knots and other imperfections can emerge in the sawing process. However the sawn timber has the benefits of being more flexible in its geometry, in conclusion this is simply two different ways to use the material that gives two different appearances to the final product.

Figure 2.19 Chengyang Bridge stacked beam base, Sanjiang, China [10]

The interior of the bridge is very similar to that of the Xidong Bridge. The roof structure rests on a set of columns, with an elegant post-beam system that divides the loads in the roof centre line out to the row of columns (see Figure 2.20). The previously mentioned pavilion areas of the bridge have been supplied with benches for seating and contemplation when passing through the bridge. This type of setup is a simple way to enable visitors to further experience the passage over the river, by letting them stop at the pavilions to maybe look at the view or perhaps just to rest. Some travellers actually use the bridge benches as overnight lodging, showing how this bridge has managed to be utilized in many more ways than what its original purpose suggests (China Network, 2002).

The roof structure, just as with the Xidong Bridge, is not meant to be a stabilizing member. This bridge is an even stronger example of when the aesthetics are given an important role in the chosen design, not only did it help in the making of a beautiful structure but also managed to turn the bridge into an icon for the area.

Figure 2.20 Chengyang Bridge roof inside, Sanjiang, China [11]

The design of the bridge is said to symbolize a Chinese water dragon, resting over the river as a guardian of the area. The local word for this mythological creature is *Panlong*, the bridge is therefore often referred to as *Panlong Bridge* (Baidu, 2013).

Chengyang Bridge as an icon serves very well as a symbol for both the local region and Chinese architecture in general. Because of this the bridge has recently even been featured in video games, showing how widespread a structure can become when using the full potential of skilled craftsmanship and design (see Figure 2.21).

Figure 2.21 Chengyang Bridge in the video game World of Tanks [12]

2.3 Covered Timber Bridge building in Sweden

The covered timber bridge tradition in Sweden is virtually non-existent, only one bridge of this type exists in all of Sweden, *Vaholmsbron* (see Figure 2.22). This bridge is constructed with a simple king post truss, each span has two diagonals that meet at the centre, from this point there is a steel rod from which the deck is hanging.

Figure 2.22 Vaholmsbron, Vaholm, Sweden [13]

As Figure 2.22 shows the style of the bridge is very similar to that of the North American bridges, which is undoubtedly where the inspiration for it came from. The bridge is also similar to those bridges in the sense that the roof functions mainly as cover.

The lack of covered timber bridges in Sweden leaves little information to be gathered on the subject, therefore when looking at Sweden the scope will have to be widened to some other timber bridges. Very few timber bridges were built in the country for the most part of the 20th century, it is mainly after year 1990 that timber bridges have started to pop up more frequently in Sweden (Svenska Kommunförbundet, 1998). The timber bridge building in the country as of the last few years has been quite extensive, but too much respect towards the risks off the material still exists. Common problems that occur with timber bridges almost always relate to the durability of the material (Fjellström P-A, 2007), therefore it seems strange that covered bridges providing the timber with the ultimate protection – no exposure, still have not become a more common phenomenon.

2.4 Discussion - Form or Function?

This chapter has now looked at covered bridge building history in different cultures, this raises the question of what the benefits are with different levels of architectural design incorporated in bridge building. In every bridge design, the safety followed by functionality is of course of the highest priority, however based on the examples shown above there are many benefits to come from including creative design into a bridge project. Before completely rejecting the North American covered bridge style and praising the Southeast Asian one it is important to mention the cultural differences. Southeast Asia is famous for a unique and very impressive timber craftsmanship tradition, especially regarding roof structures. The beauty of the Xidong and Chengyang Bridges did therefore not require difficult new techniques in order to produce such a product. It is also important to remember that the North American bridges of course had strong elements of skilled craftsmanship in the constructing of joints and elements as well, only with less focus on the aesthetics. Comparing this to Sweden, where the experience in timber bridge building is not as high, the economical and time effects of a correspondingly intricate design would be signifficant. Nevertheless the positive effects of this tradition are an inspiration to further incorporate design into timber bridges. Through this the potential of a structure to become an icon could become the kick-off that is needed for Swedish covered timber bridges to become more common.

Ever since the 1980's there has been a noteworthy revival of the timber bridge building industry (Meierhofer U.A, 1996). As modern times more and more consider environmental aspects, the material timber and its superior sustainability features rises in usage. Another potential factor is the desire of countries to once again find its roots, even in the construction business. Take for instance Austria and Switzerland, two countries with a rich timber building tradition. A lot of interesting timber bridge projects have emerged here in the last decades, for instance *Pirkarchbrücke* in Austria (see Figure 2.23). Covered bridges of this kind show a clear influence of design combined with modern timber technology, a trend which oddly has not reached Sweden yet despite the rich timber tradition which exists here as well. Sweden has the timber technology, engineering capability and architectural creativity required for these projects. All that is left is experience and for that someone has to start!

Figure 2.23 Pirkach Bridge, Carinthia, Austria [14]

3 Bridge Types

Before digging into the task of designing a new covered timber bridge, a lot could be gained by looking at some other bridge types that have been built. Since the structural systems for the covered bridges looked at before have been limited to a few types, this chapter will not be narrowed down to covered bridges but rather look at different systems for timber bridges in general.

3.1 Slab and Beam Bridges

The first and most basic form of timber bridge is to let the material work in pure bending, this is probably how the bridge phenomenon all started in the early years of mankind, with trees fallen over rivers that were used to cross the water.

The timber slab bridge is a very basic system, a rigid plate of timber that is placed over a span. Since the load bearing system is a plate, it also works as a deck to walk on or put a top coating on leaving little additional required but the slab itself to form a functional bridge. A common modern way to make a rigid slab is through stress laminated timber, created by sideways stacking of timber beams that are then tensioned together laterally by steel rods to form a plate. This type of bridge is fast to construct and can be heavily prefabricated leading to many economic benefits. The main downside is the limited span lengths.

A timber beam bridge is also very simple in its structure and can often be built to a very low price and short construction time. These advantages to the bridge type are strong indicators towards why this bridge type was the first and most common bridge system for a long time. For instance the *Julius Caesar Bridge* of year 55 B.C. which was a 140 meter long beam bridge constructed in just ten days (Träinformation AB, 1996). However the disadvantages to the beam bridge are the same as for the slab bridge, short span lengths due to inefficient material usage.

Nevertheless these bridge types are very commonly used, much credited to their economic benefits. The issue of poor span lengths is usually solved with many supports, leading to bridges such as the *Beckholmsbron* in Stockholm (see Figure 3.1).

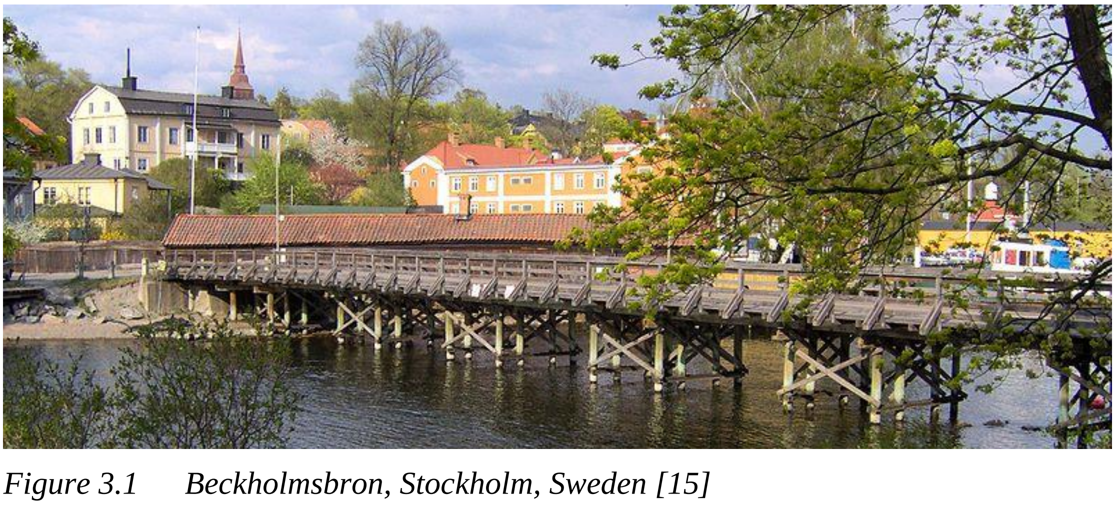


Figure 3.1 Beckholmsbron, Stockholm, Sweden [15]

3.2 Truss Bridges

One of the most common timber structure systems are the numerous variants of truss systems. In a truss system the material is used more efficiently than in the previously mentioned beam and slab systems. The truss utilizes the stability of triangles in order to form stable superstructures, in this way much greater stiffness and strength can be achieved with the same amount of material than for a beam.

Some truss types were mentioned earlier in the chapter about North American timber bridges, however there are countless other types and versions of trusses due to the broad nature of the concept. Two of the most essential features of a truss (apart from its general geometry) are the joint types and material alterations. The connections of a truss can either be simply supported (rotations allowed) or rigid (no rotations allowed). If the first of these is used for a member it is guaranteed to carry the loads in pure tension or compression and no bending, this is thus a way to ensure that the material works more efficiently. The potential of material alterations is commonly used for pure tension members in a truss. With steel being significantly stronger than timber the required dimensions of timber can be heavily reduced for a steel rod giving a more open feeling to the truss.

Since trusses generate a significant height it is common to use the trusses of a bridge as for instance railing on a pedestrian bridge, wall frame on a covered bridge (as in many North American bridges) or perhaps a horizontal bracing truss in the deck or roof structure. Utilizing this principal it is possible to have structural members that work as something more than load bearing leading to a more efficient design. This is for instance the case for the *Staffenbrücke* in Austria (see Figure 3.2) where the roof structure is part of the truss system.

Figure 3.2 Staffenbrücke, Kössen, Austria [16] photo by: R Exenberger

Benefits with a truss are hard to generalize due to the wide nature of the concept. However some benefits can be for instance low production costs if a truss is designed with many identical members, or perhaps simple transport to the site since the truss can be partly assembled after transport in components (compared to for instance a large timber arch beam).

3.3 Arch Bridges

The timber arch is another way of more efficiently utilizing the material. The arch system has for a long time been used for materials that carry load well in compression, which applies to most construction materials. In the arch the structure follows the force paths more closely making the downward forces of dead weight and loads travel continuously down to the supports in almost exclusively compression for the global structure.

A common arch system is to use large arched glulam beams that can take rather heavy loads compared to a flat beam. Another interesting method of producing an arch is what has been used for the *Kintai Bridge* in Japan (see Figure 3.3). Here an intricate truss system has been assembled into an arching global structure making the truss more efficient. Arches are also often used in combination with beams or trusses as cooperating measures of load bearing, as with for instance the Burr Truss system.

Figure 3.3 Kintai Bridge, Iwakuni, Japan [17]

The radius of the arch can vary greatly between different designs, often when the radius is very large giving a low arch the system is referred to as a pony arch. Arches throughout history have often been constructed so that the deck follows the arch. The benefits of these bridges are often related to the opening underneath, which then efficiently enables passage for instance for a boat. The sacrifice made is the sometimes problematic slopes of the deck that come with the arch, something that is often solved with stairs.

While the arch has many benefits, some disadvantages can also be named. A curved beam collects a lot of forces in the apex and is thus vulnerable to breaking in this location, this is sometimes handled by a two component arch with a joint at the apex.

3.3.1 Stress laminated timber arch bridges

Just as with the timber plate bridges discussed previously, timber arches can also be built using a stress lamination system with transversal steel rods. This system will give an arched solid slab of timber that theoretically could be used as a deck, but to avoid problems with the steep slope the stress laminated arch is often given a flat top deck, sometimes also of the stress laminated timber type.

3.4 Suspension Bridges

In timber bridge design the suspension bridge is not particularly common, the reason for this is the lower stiffness of timber bridges. A suspension bridge based on gravity can have a rather low stiffness and stability in general unless the suspended deck is very heavy. For a timber system that has both a low mass and stiffness compared to many other materials the combination with suspension can lead to extensive measures needed to avoid discomfort when walking over the bridge.

One way to solve this is the system used in the *Traversiner Steg* in Switzerland (see Figure 3.4). This bridge has a timber deck suspended by steel cables, but in order to stiffen the behaviour of the deck the whole cable system is pre-tensioned. Using strong foundations on each side the main cables here are tensioned until the point where the timber arches are in compression giving a much more stiff structure (Krippl V, Nigg O, 2010).

Figure 3.4 Traversiner Steg, Rongellen, Switzerland [18]

3.4.1 Cable Stayed Bridges

A common variation on the classic suspension bridge is when the cables holding up the deck are all connected to the supports instead of a primary cable, a so called *cable stayed bridge*. There are two main types of cable stayed bridges, the *fan design* in which all the cable meet in the top of the towers and the *harp design* where the cable supports on the towers are evenly distributed over the height. For instance the *Öresundsbron* between Sweden and Denmark is a harp style cable stayed bridge.

The longest one span timber bridge in Scandinavia is the cable stayed bridge *Älvsbackabron* located in Skellefteå (see Figure 3.5). It was built in 2011 and has a four meter wide deck that spans 130 metres (Martinsons, 2009).

Figure 3.5 Älvsbackabron, Skellefteå, Sweden [19]

3.4.2 Stress Ribbon Bridges

Another special type of suspension bridge is the *stress ribbon bridge*, in this type of structure the load bearing cable/ribbon system is incorporated in the deck. This enables bridges with a very long and slim appearance, however this also generates a lot of demands on the calculations and performance of the structure making it an uncommon bridge type.

One example of this type of bridge can be found in Germany, the *Holzbrücke bei Essing* (see Figure 3.6). As the figure shows the bridge get a very characteristic hanging chain curve aesthetic.

Figure 3.6 Holzbrücke bei Essing, Essing, Germany [20]

3.5 Discussion - Architecture of Bridges

One of the important issues of this project is to investigate the possibility to include architectural design in the design of bridges. The most common standard of the bridge industry of today is that the bridges being built are designed by the engineers, making a lot of the focus end up on the structural behaviour and less consideration made for the appearance of the final product. This is not always the case and for instance the Norwegian bridge *Leonardo Broen* in Ås (see Figure 3.7) was preceded by a thorough design process by the artist Vedbjørn Sand (Bjertnæs E. J, 2014).

Figure 3.7 Leonardo Broen, Ås, Norway [21]

There is always a major difficulty in analysing the appearance of a bridge due to the subjective nature of aesthetics. This also means that the discussion of how much architectural design one should include in the bridge design process becomes very difficult, since the evaluation of the final product will be hard to keep objective. It seems reasonable to assume that the general intention always is to create bridges that are pleasing to look at, but after considerations to safety, function and economy the final result might not always be as beautiful as intended. The first two of these (safety and function) are not necessarily working against a beautiful end product, in fact not seldom the opposite can be said where the beauty of a bridge might be in the load bearing system setup.

The main issue is the latter, the economy aspect. The extra work spent in the planning phase and the additional costs that can occur in materials and construction due to a more thorough design need to be funded somehow. The optimum situation would be if the extra design can be compensated by for instance lower maintenance costs or a similar cost reduction, this way there would likely be no objections to the additional design costs. Since this cannot be ensured in advance it will probably be hard to push this as a motivator to incorporate more architectural design in the process. Another approach is to focus on the potential of creating a landmark, a characteristic bridge of the site that will serve as an icon for the area. This of course also leaves no guarantees and the additional costs to achieve this goal might be way above the intended budget. In conclusion, there are no obvious ways to efficiently include architectural design in a bridge project that can make timber bridges compete more with other bridge types.

These speculations show that it is not a simple task to handle for the timber bridge industry, and as previously mentioned the current situation is very limited incorporation of architecture in bridge design. A reasonable guess is that there needs to be some type of initiation to spur the timber bridge industry further, perhaps through a very iconic timber bridge. Maybe this bridge will have to be significantly more expensive than a common standard solution, but will open the eyes of more people to the beauty of timber bridges. Because if the industry got a significant boost the construction costs would go down due to better experience thus enabling more architectural design to come in without unreasonable budgets.

Something more concrete that can be looked at in terms of timber bridge aesthetics are the potentials of the material, subjectively assumed as positive of course. Timber structures tend to be designed with a repetitive system characteristic, something following a pattern of some sort, for instance the repetitive nature of the underside of the *Kintai Bridge* (see Figure 3.8). This phenomenon of timber bridges has the potential of benefitting the aesthetics without adding but rather reducing the cost of the final product. By using repetitive systems the potential of prefabrication and even preassembly of modules can have a significant impact on the final price tag of a timber bridge.

Figure 3.8 Structure of Kintai Bridge, Iwakuni, Japan [22]

Another strong feature of timber is the appearance of timber as a material. Compared to for instance untreated steel or concrete, timber has a more decorative surface. By using untreated or somewhat transparently treated timber this aspect of the material can be uplifted, giving a more pleasing final surface to different parts of a timber bridge. Another related feature of timber as a material is its living nature, the material ages visibly over time in a way that can be highlighted. Though very subjective the very constant nature of concrete can be considered “boring”. Comparing this to timber that over the course of a few years can greatly change its colour it is possible to get a much more dynamic experience.

To summarize, it is hard to say something definite about the pros and cons of timber aesthetics, but the material has several features that can be used to benefit the final design without the sacrifice of additional costs.

4 Woodworking Techniques

A useful addition to the preparations for the conceptual design phase is to look at what can be done with timber in other fields than bridge design. This chapter will look at some different timber handling methods in terms of material adjustments and member connections. Since this is a very broad subject only some relevant points will be included.

4.1 Manufactured timber products

LVL (laminated veneer lumber) is a well-used manufactured timber product. The product consists of thin layers of wood veneers that are adhesively attached to one another, much like in a plywood sheet but with less crossovers thus having the majority of fibres in one direction. Since only thin pieces of timber are used the manufacturing can ensure little defects giving a finished product with a very high strength. LVL can thus be used in projects where high strength is required, or when the plate like nature of the product is desired. An example of LVL usage in projects is the *Metropol Parasol* in Spain (see Figure 4.1) where a square grid of LVL plates were cut and joined into a tree canopy structure.

Figure 4.1 Metropol Parasol, Seville, Spain [23]

Another common manufactured timber product is *CLT* (cross laminated timber). This product consists of several layers of sawn timber that have been crosswise stacked and glued together, in a similar manner to that of *glulam*. This process enables the resulting material to span in both directions. These properties of CLT make it very commonly used in high timber buildings, where it serves as the main shear walls for the structures.

4.2 Traditional Southeast Asian Inspired Timber Joints

The traditional timber joining techniques of Southeast Asia, as mentioned before, rely on no metal fasteners. The timber components are carved into various joint end shapes and cavities that are then merged into different types of connections depending on the desired behaviour. This methodology used to rely on skilled craftsmanship for hand making of all of the timber joints, a talent that was carried down through generations of craftsmen devoted to the art (Nianzu G, 2009).

This construction principal shows several benefits that when combined with modern technology has a strong potential for future use. The previously complex art of hand making the joints can today be replaced by efficient CNC-machines that can make accurate and fast cuts, generating construction sets to be sent to the construction site. Here the process is then significantly simplified and the main timber framing can be assembled fast and with relative ease in a puzzle like manner. Through this the majority of steel details needed for a structure can be reduced potentially lowering the total budget. This exact method is for instance used in Japan where the company Bakoko has built whole houses using the pre-cut CNC timber joint method (Bakoko, 2011). Since modern CNC technology enables virtually any type of timber shape the variety of different shapes for the joints are seemingly endless.

The economic benefits of this methodology are significant, and what extra time is spent in the design phase making the joints and in the cutting process can if planned out well be compensated for in the complete lack of steel details and the reduced construction time. In conclusion, this construction principal shows many promising features and this project could benefit greatly from exploring possible solutions including CNC precut timber joints.

5 Initial Concepts

The following chapter summarizes the conceptual phase as it progressed. Most of the work was done with sketches, discussions and analyses, the following pages will summarize the discussed ideas in different steps along the way.

The first part of the work was carried out with hand sketches. During the design process numerous skapes, sections and scribbles were made, most of which were not used for the actual concepts developed. In order to get a sense of the type of hand sketches produced Figure 5.1 shows a collection of a few of the sketches made.

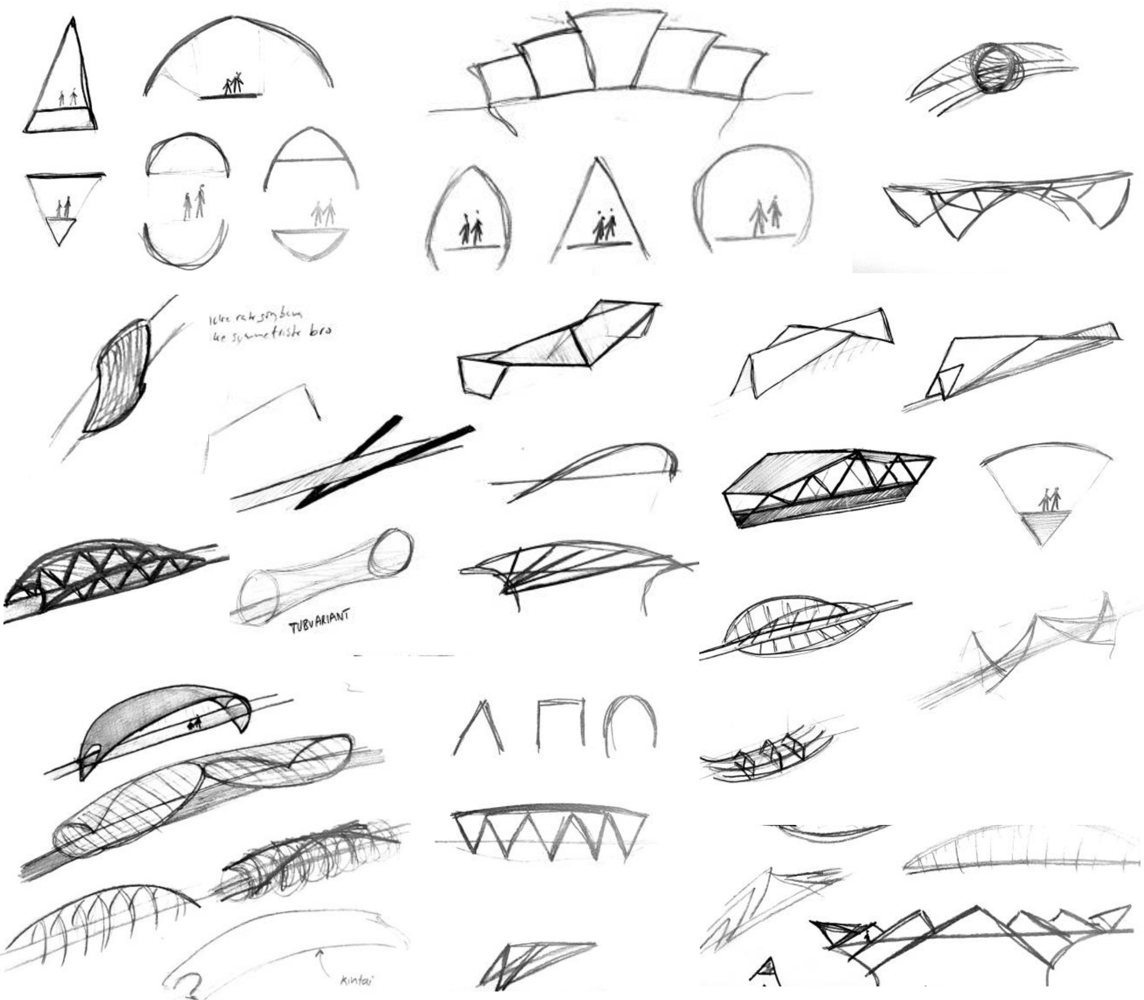


Figure 5.1 Collection of hand sketches from the conceptual design

With the help of sketches such as the ones above a refinement process started, where the ideas of from paper where turned into a bit more concrete concepts. At this stage the 3D modelling program *SketchUp* was used to test and visualize the concepts developed. The goal of this conceptual phase is to generate as many plausible concepts as possible, with a wider variety of ideas the chances of finding a great one is increased. Therefore the concepts that follow are meant to include many variations of the actual bridge in the sketch, and rather stand for the idea of one type of bridge.

The sketches are also greatly simplified to symbolize the concept rather than showing an actual functioning bridge within that concept.

In order to structure up this phase the concepts have been graded in a quick fashion from “conservative” to “radical”. This of course is hard to do to general concepts but the idea is that conservative concepts use more conventional and common structural principals whereas the radical ones are uncommon bridge types or complex structural systems.

5.1 The PRISM

The simplest concept, which also includes the vast majority of all built covered timber bridges is *the PRISM concept*. The principal of this concept is a bridge with a constant section, for instance a triangle or square, that contains a bottom and top horizontal plane thus creating a path and a cover and walls that work as high beams for the vertical loads (see Figure 5.2).

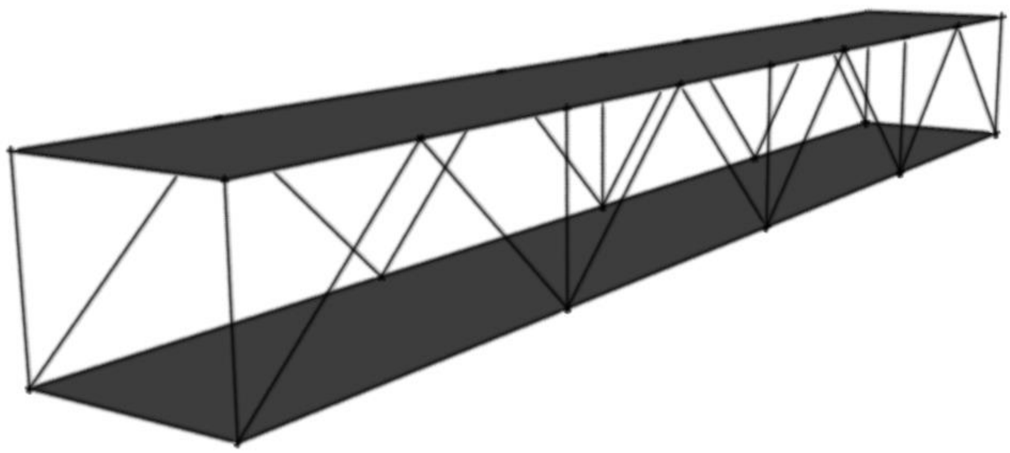


Figure 5.2 Sketch of the PRISM concept

Since these concepts aim to include many variations of a structure while maintaining the same architectural theme the prism concept can have any type of truss in the walls. With that in mind it is apparent from Figure 5.2 that this concept includes all of the North American covered timber bridges, and of course the Swedish one.

The structural principal of the PRISM concept is to handle the vertical loads via the trusses in the walls, thus using the height of the roof to get a much more stiff bridge. The horizontal loads can be handled by the slab and roof provided that they are constructed into stiff plates. Caution when checking the horizontal stability will have to be made for a triangular section where only the slab can act as a stiff plate (see Figure 5.3), however this section handles torsion more efficiently.

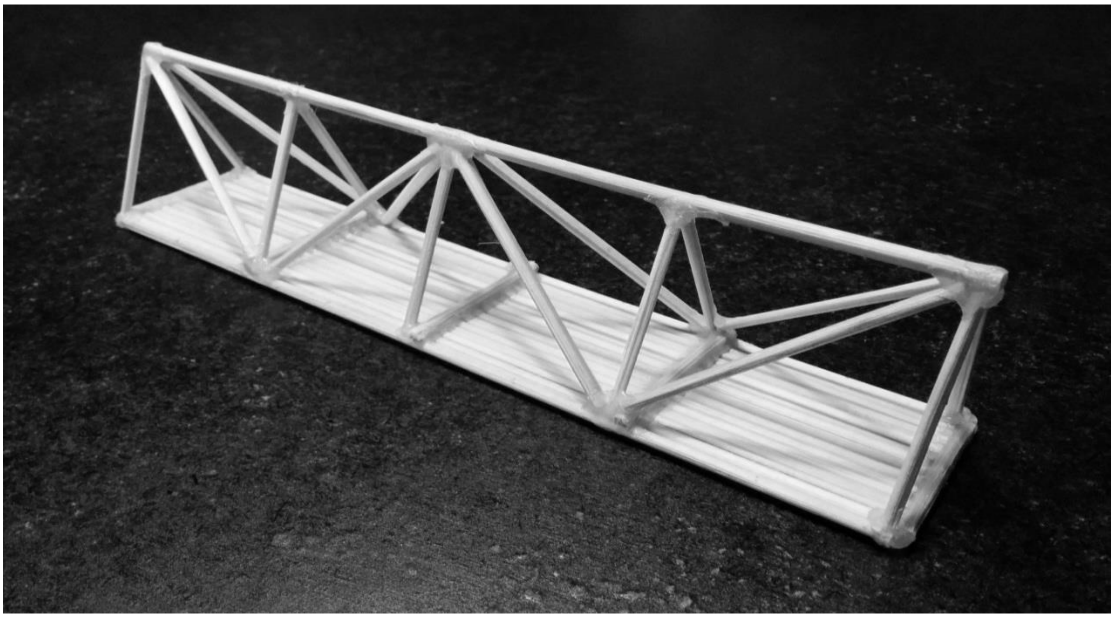


Figure 5.3 Photo of the PRISM concept

The potential designs that are included in this concept can vary greatly with a variety of different sections that would create very different final appearances of the finished bridge (see Figure 5.4).

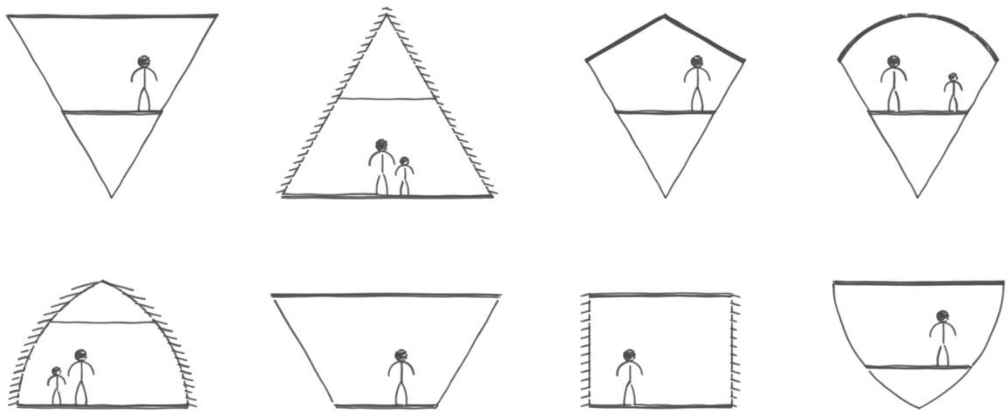


Figure 5.4 Potential sections for the PRISM concept

The sections in Figure 5.3 show that this concept might require some form of shielding lamellas to cover the bridge. This of course depends on if the roof is wide enough to protect even from rain on a windy day. Apart from different sections this concept also covers bridges that have an elevation at the centre, creating an arched walking path in the bridge. As mentioned before all these sections have a truss in the walls and the final major design component that varies is the type of truss used, leaving virtually countless designs included in this concept.

Summary the PRISM concept

- *Simple concept* – By using conventional solutions and a simple geometry this concept can save in both complexity of design and total budget.
- *Versatile concept* – As mentioned above there are very many variable factors within the concept opening up for many potential solutions.
- *Conservative concept* – The risk of using this conservative concept is to not achieve the desired eye opening effect of an interesting bridge.

5.2 The I-BEAM

The following concept, *the I-BEAM concept*, is based on a similar principal to the previous one but where the vertical loads are carried in one centre wall. This way the bridge is very open towards the outside while still having a roof cover (see Figure 5.5). The design will thus, as the name suggests, imitate the behaviour of an I-beam which is a proven working shape for a beam.

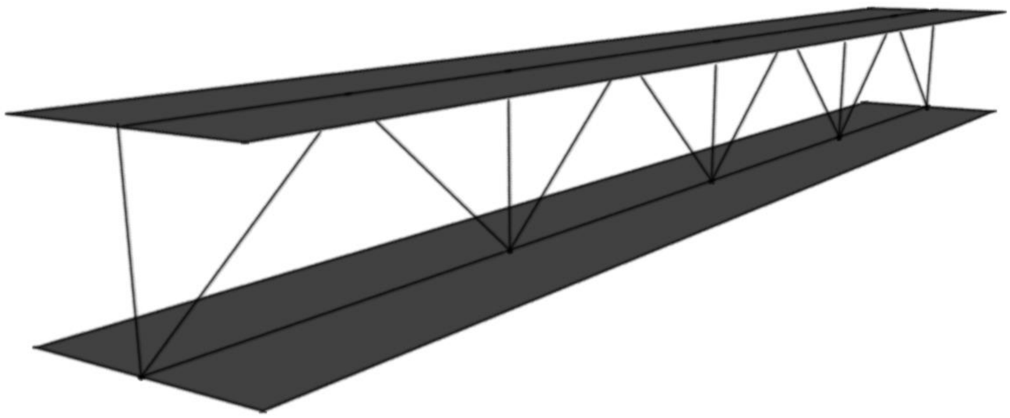


Figure 5.5 Sketch of the I-BEAM concept

One key thing to think of with this concept is that the two paths crossing the bridge have now been divided. By doing so with a centre truss the openings in the middle still enable passing people to turn around if they so desire. Another potential use of these openings can be to create pause areas with benches so that the open sides of the bridge are utilized to their full potential. If this type of bridge for instance spans a scenic river this would then let people stop and take in the nature of the site in a way that otherwise might be harder.

The structural behaviour of this concept is similar to the previous one regarding bending, vertical loads are handled by the wall and horizontal loads in a stiff floor and roof plate. With one wall instead of two the dimensions of this truss will be significantly increased. The torsional loads on the other hand will not be able to travel around in a shell as the previous concept (*Saint-Venant torsion*), instead this type of section will rely on shear in the top and bottom plates (*Vlasov torsion*) which is significantly less efficient. To enable these shear forces to carry the torsion some type of force transfer between the roof and slab is needed. Apart from this there is another critical behaviour, the torsion of the roof around an axis in the top of the centre wall. In the analogy with an I-beam this is not an issue since virtually all of the load comes in the vertical direction there, but for a bridge with wind loads of significant proportions this must be handled. A potential solution to these two torsional issues might be stiff steel profiles evenly spaced along the bridge (see Figure 5.6).

Figure 5.6 Photo of the I-BEAM concept

Just as with the PRISM concept many different sections can fit into this concept. The floor and centre wall have somewhat limited sectional modifications but can vary greatly in truss setup etc. The roof is not bound by the limitations of a walking path and can thus take on various curvatures, in both a lengthwise and crosswise section of the bridge. From a floor plan view not only the roof but the slab could also have a varying width, this could give extra space in the areas that potentially have both pausing areas and the regular path.

Summary the I-BEAM concept

- *Simple concept* – Just as with the previous concept the structural principles in general for this concept include conventional solutions that can act to reduce both the total work and cost needed for the bridge.
- *Open concept* – With almost nothing, except of course eventually a railing, this concept has nothing obstructing the incorporation of the bridge surroundings into the bridge space. In addition the open nature of the centre truss gives further potential to the bridge to get additional functions.
- *Torsion inefficient concept* – One of the major trade-offs of this concept is the significant decrease in torsional strength, which will require attention.

5.3 The SADDLE

The third concept also relates to the initial one but with one major difference, the bridge cross-section varies over its length. The most basic of these variations is a four sided section with one side of the roof higher, the roof then changes over the length of the bridge into a mirrored version on the other side creating a saddle shaped roof (see Figure 5.7).

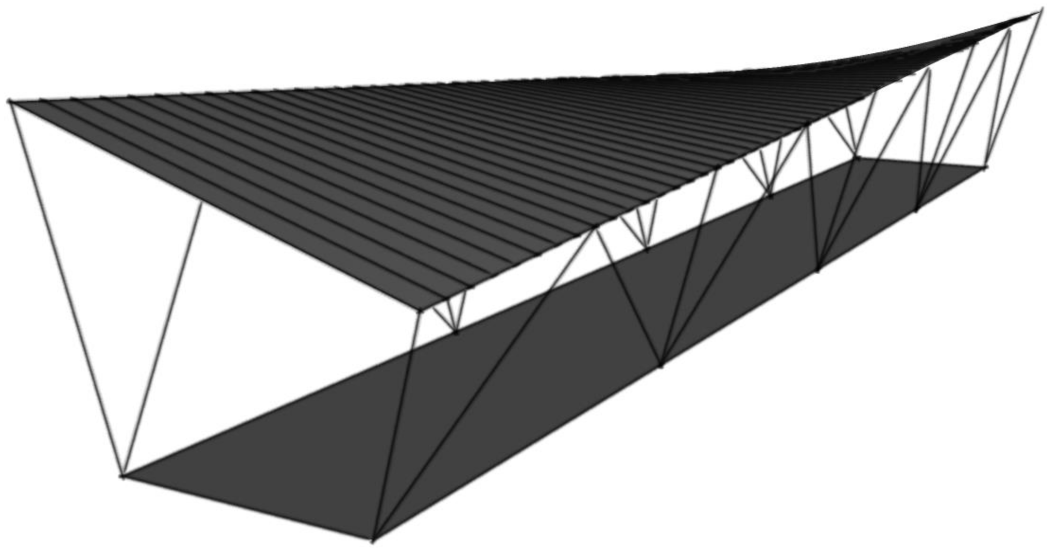


Figure 5.7 Sketch of the SADDLE concept

The structural behaviour of this concept is similar to that of the PRISM, this is of course depending on how much of an alteration there is in the roof section. Assuming a small change in section like in Figure 5.7 the behaviour will be almost the same, vertical loads in the wall trusses and horizontal loads in the slab and roof (that will have to be stiff). In a more dramatically altered roof, for instance where the edge sections are mirrored triangles, the behaviour will be more complex. Here it will be harder to simplify the load resistance in vertical and horizontal members, since the roof now is a diagonal working as a blend of wall and roof. For most cases of this concept the torsional resistance is handled in a Saint-Venant fashion.

In order to further investigate the space created by this concept and to get a sense of the stability of the double-curved roof, a model was created (see Figure 5.8).

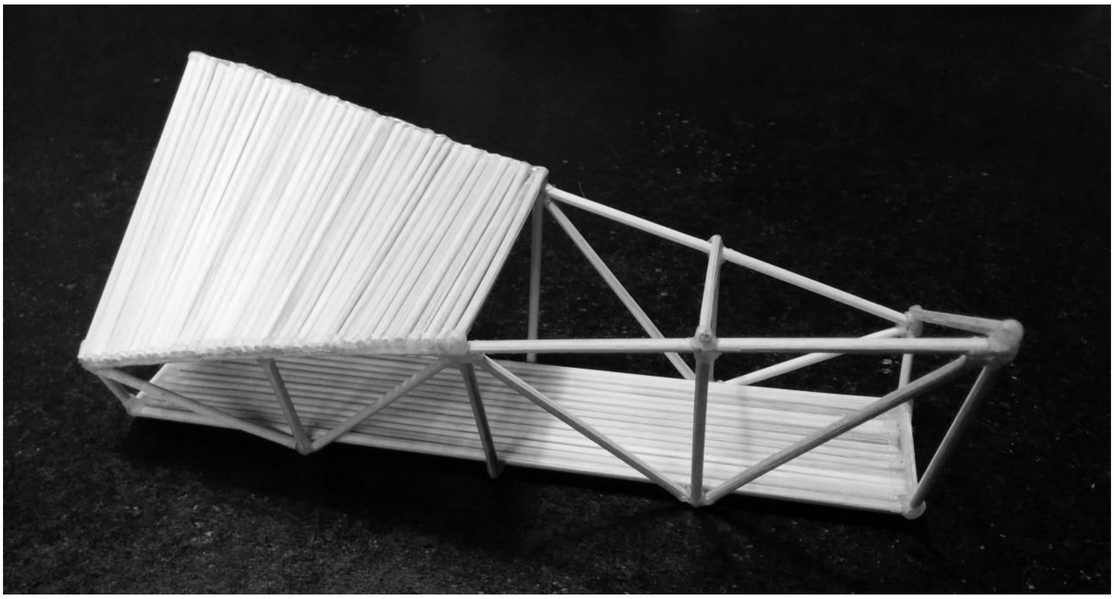


Figure 5.8 Photo of the SADDLE concept

In terms of the architectural design this concept stands as a more interesting version of the first concept. Drastic differences to the bridge space can be generated if the previously mentioned triangular end sections are used, this space will be more enclosed at some locations but even more open in others, compared to the PRISM concept. This can create a much more dynamic experience in the bridge which, if not taken too far, can be of advantage. The complexity and dynamic nature of this concept also has the advantage of being generated through relatively simple means, a cost and time saving feature that is not to be under-estimated.

Summary the SADDLE concept

- *Simple complexity concept* – This concept has the benefit of generating an interesting complexity in the design using very simple means, such as the double-curved roof consisting of only straight components.
- *Versatile concept* – As with the first concept there are very many variable factors within the concept and here especially the roof, leaving numerous possible designs to come from the concept.
- *Dynamic concept* – One of the benefits of the anti-symmetry in the concept is that the space inside the bridge can vary greatly over the length of the bridge thus creating a more interesting experience passing over the bridge.

5.4 The CROSS

The next concept goes further into complexity from simple means. By looking at what spatial qualities can be desirable for a bridge *the CROSS concept* was made. The basic principle is that the bridge space should open up at the entrances and towards the sides, but be otherwise protective. This can be achieved with a cross of two axes in the roof and ground supports at the four corners of the base (see Figure 5.9).

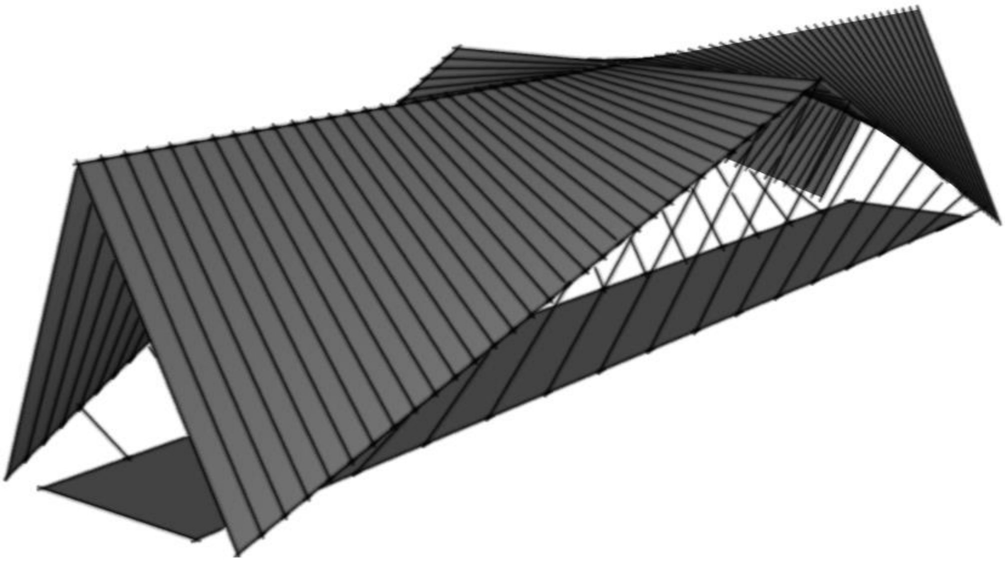


Figure 5.9 Sketch of the CROSS concept

The structural behaviour of this concept consists of a stiff roof structure from which the bridge deck is hanging. The roof skeleton is generated from four triangles that are all supported in the bridge corners, a cross in the roof then connects the top of the triangles stabilizing them. The roof can then be completed with a set of straight boards to distribute the loads between the different roof components down to the support and provide protection to the bridge. Evenly spaced steel wires can then be placed along the large triangles in order to support the bridge deck (see Figure 5.10), the angled geometry of these wires help prevent discomfort from dynamic behaviour of the deck.

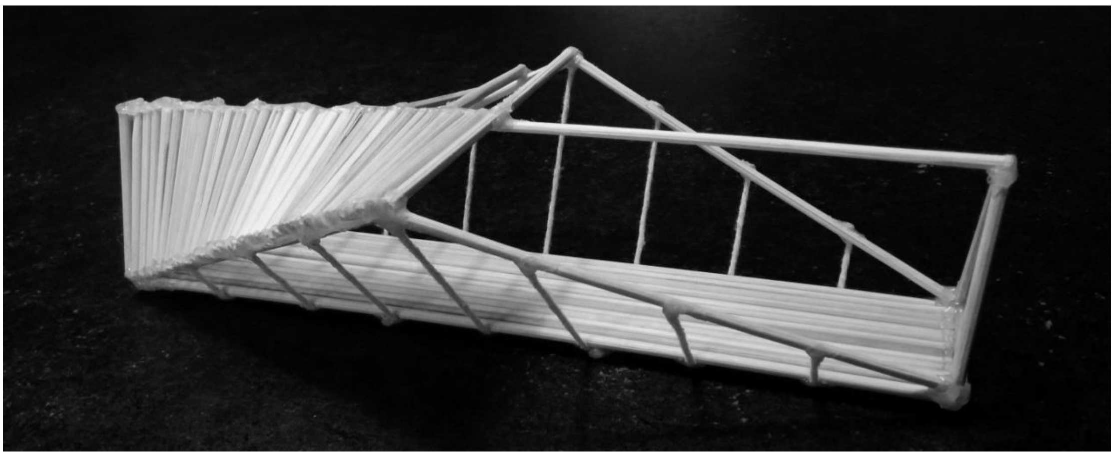


Figure 5.10 Photo of the CROSS concept

While this concept relies on the supports in four corners and the crossing axes in the roof, several variations in curvature and format of the roof components can be imagined. The main goal is to maintain the spatial qualities that the concept was based on, the clearly defined entrances and open sides.

Summary the CROSS concept

- *Simple complexity concept* – The relatively complex geometry of the bridge is achieved from a set off straight components, the concept of complexity from non-complex components gives an interesting touch to the bridge.
- *Spatial design concept* – Being generated on the basis of desired space the bridge has several architectural qualities that otherwise might be hard to generate if not included in a phase as early as this.
- *Inefficient structural concept* – One of the trade-offs for the design from spaces is in this case that several components carry loads rather inefficiently, using bending resistance instead of compression or tension.

5.5 The LEAF

The subjective nature of what one finds beautiful makes it hard to grade bridges as objectively pretty. Thus an alternate version of the CROSS concept aiming to be more elegant and seamless was created through *the LEAF concept*. The same open nature of the sides is here generated using two large arches that are then connected in to a leaf shaped roof. From the roof a suspended bridge deck is attached similarly to the last concept (see Figure 5.11). Where the last concept aimed to please those who find appeal in edgy and complex geometries this one instead aims towards a more soft and elegant bridge. Which approach is the most appreciated is hard to say, thus a comparison of the two will have to be based on other aspects.

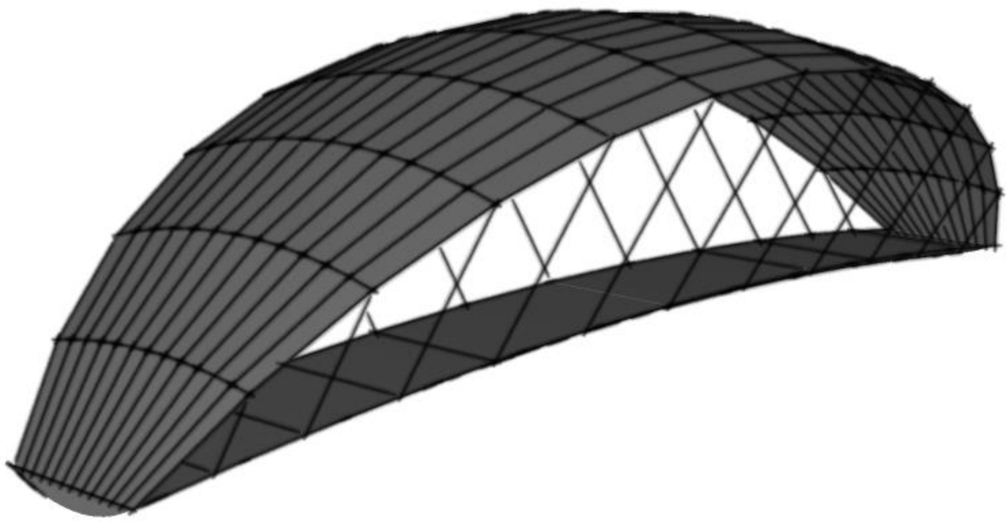


Figure 5.11 Sketch of the LEAF concept

The structural principal of this concept can be described with an analogy to a bow, without an arrow. The roof structure is assembled into a stiff top plate in compression, with an arched shape to efficiently carry the self-weight. The bridge deck is then a tensioned ribbon just as the string of a bow. In addition, though no longer in line with the bow analogy, a set of cables further stiffens the deck and roof structure along the bridge (see Figure 5.12).

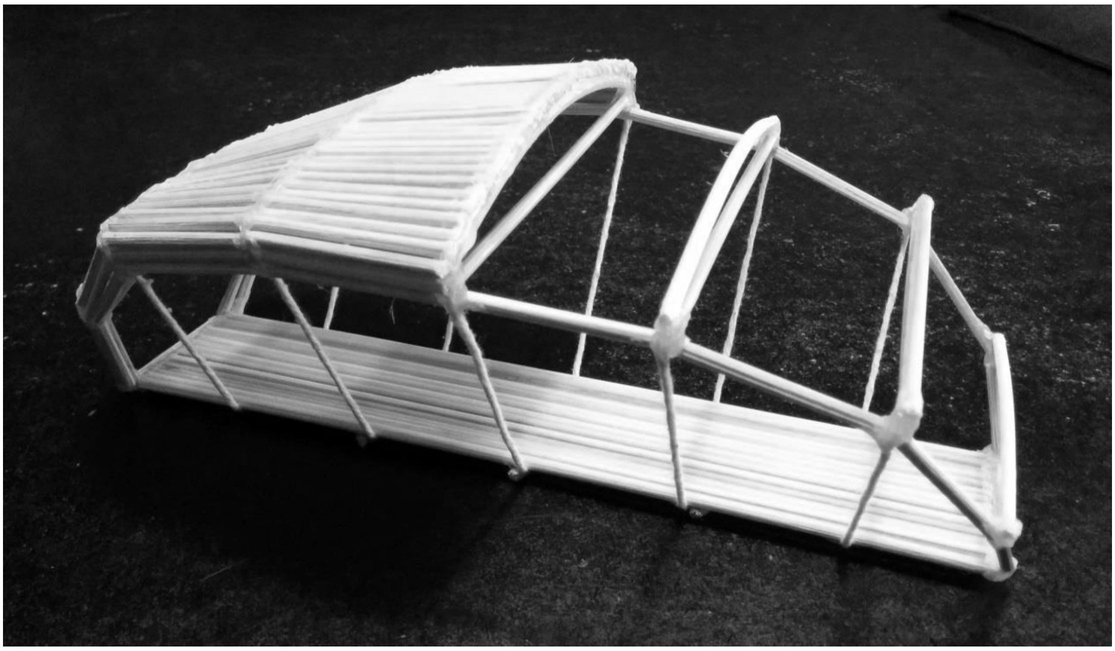


Figure 5.12 Photo of the LEAF concept

This concept also has a high focus on the architectural qualities, the bridge space and global shape are very elegant. Other positive features of the bridge are for instance the increase in roof overhang with increasing ceiling height, giving an efficient protection from the rain. The entrances, which have not yet been specified, can either come from openings in the roof by the supports, an elevation of the roof to enter under the supports or perhaps from a rotation of the whole roof so that it covers the deck in a slightly diagonal fashion.

Summary the LEAF concept

- *Elegant concept* – The curvature and clearly defined bridge components, such as the sweeping roof structure, gives a global appearance of the bridge that, in opposite to the last concept, has many architecturally clean qualities.
- *Spatial design concept* – Just as the last concept this one is also created based on desired architectural qualities, an advantage to many bridges today created from pure function or budget limits.
- *Extensive curvature concept* – One of the architectural positives of this concept is also a budged negative, the curvatures. Attention to optimization will have to be made here in order to avoid high costs for components.

5.6 The TUBE

The next concept comes from the desire to optimise the design to many identical components, thus improving on the economical and production aspects. The idea that played well with this desire was a circular cross section extruded into *the TUBE concept* (see Figure 5.13). The tube can then be discretized into a triangular, rhombus or hexagonal grid; a grid that if designed well can contain a large number of identical components.

Figure 5.13 Sketch of the TUBE concept

The structural behaviour is based on the grid turning into a stiff shell that forms the bridge tube. This tube shape, if anchored appropriately at the supports, will be very stiff in all loading directions due to its symmetrical nature. The bridge deck placed inside can then work as an additional stiffening agent in the bridge structure (see Figure 5.14). In the analysis phase, the type of grid will be of great importance, many grid types can give a stiff shell but the design of connections will have a large impact on the behaviour and final budget of the bridge.

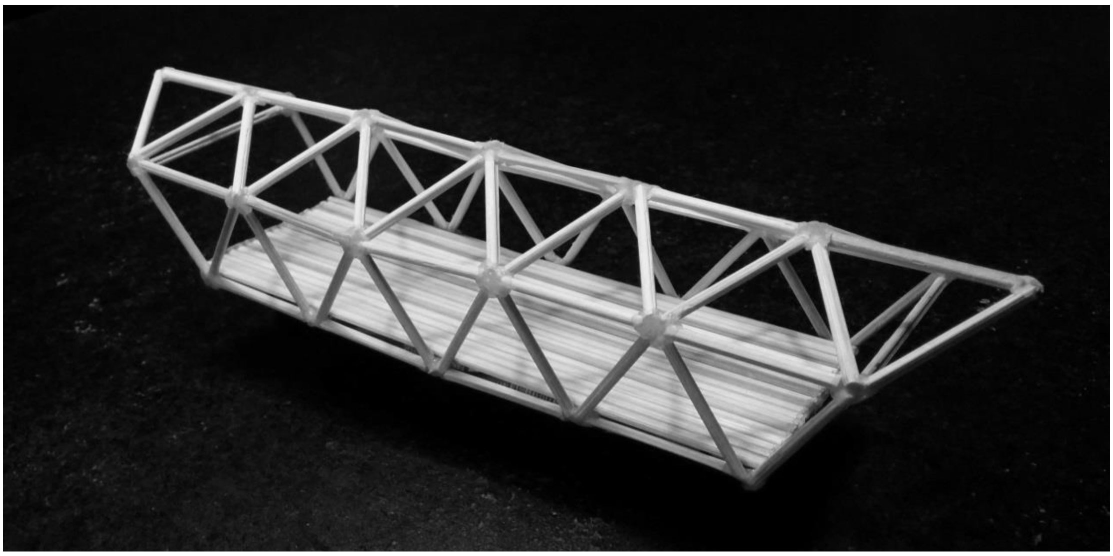


Figure 5.14 Photo of the TUBE concept

As can be seen in the two previous figures there are several variations imaginable for the global tube shape. The grid can consist of either a few very large members as in the previous figure, or be generated from many smaller ones as symbolized by the smooth tube in Figure 5.12. In addition the figures show the possibility of a curved tube, this form will have a more efficient global compression shape through the length of the bridge but more unique members. The straight version on the other hand can be optimised into every single grid element being identical and with the same type of connection for every intersection.

Summary the TUBE concept

- *Efficient design concept* – The tube shell geometry provides great stiffness in several directions directly from its shape, this means an efficient bridge that gets its stiffness without additional members to handle the bridge stability.
- *Optimization potent concept* – The geometry of the concept provides a large opportunity to optimise the construction process as well as the structural behaviour of the bridge.
- *Important detailing concept* – With a structural grid as in this concept the connections between elements become very important and crucial to a functioning and reasonably prized bridge, thorough design is needed.

5.7 The STACK

The first of the two most radical concepts is *the STACK concept*. This concept is based on plates instead of beams, in an actual structure this could be achieved for instance using CLT panels. The principal of the concept is to stack a large set of cut panels in the lengthwise direction of the bridge and then post-tension them together using steel cables. The potential geometries that can be generated with this method are virtually endless, ranging from simple box sections (see Figure 5.15) to complete freeform shapes. Since each plate can be cut individually a complex geometry can be created with ease and using modern parametric modelling this could efficiently be done for any shape to be turned into a set of stacked plates.

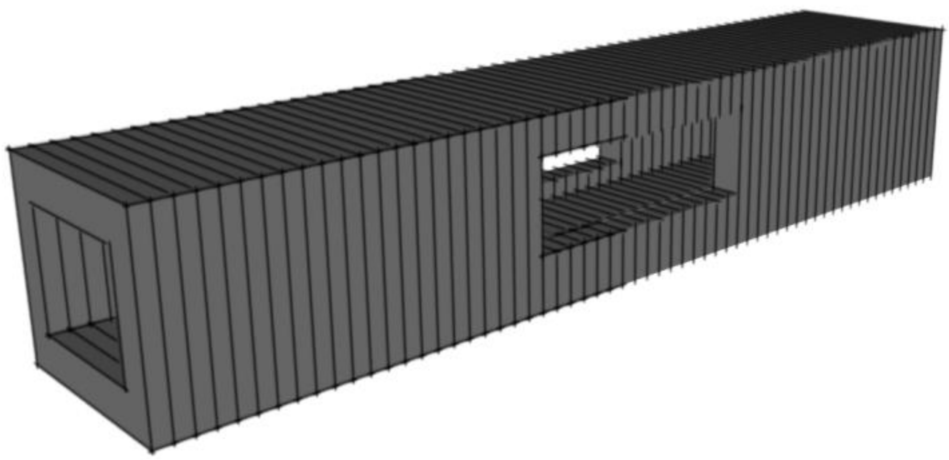


Figure 5.15 Sketch of the STACK concept

This concept turns the whole bridge into a heavy beam in bending. The tensile forces in the bottom of the bridge will be transferred by the post-tensioning cables. Some type of efficient load distribution plates or support anchorage will be needed at the ends here to avoid crushing of the timber. The compression in the top will be distributed in the stacked timber panels which, to compensate for being loaded in the weak direction of timber, have a large area of compression to share the loads. If the tensioning and attachment of the panels are done correctly the bridge can contain openings in the centre region where the forces acting on it will be almost exclusively top compression and bottom tension and only minor shear forces. The solid nature of this concept will lead to a very heavy timber bridge which will benefit the dynamic behaviour, this of course at the cost of material efficiency.

For the analysis of the forces in this concept it might be required to do a FEM-model that can handle the potentially complex geometry of the bridge. This might also simplify the work with controlling the shear and torsion of the bridge, which will be based on the transmittance of forces between the individual panels.

Summary the STACK concept

- *Simple freeform concept* – The setup of many individual panels enables them to be easily cut into any shape, enabling the bridge to take many complex forms without extensive extra work or cost.
- *Unusual concept* – This type of bridge is definitively uncommon which helps the possibility of creating a bridge that gains attention and thus hopefully helps trigger further timber bridge building in the country.
- *Heavy concept* – The massive weight of this concept, for a timber bridge, can benefit the otherwise problematic dynamic behaviour of a timber bridge.
- *Unconventional concept* – The benefits in attention mentioned above also go hand in hand with increased costs due to inexperience for this bridge type, extra time and money will be needed for such a first time structure.
- *Inefficient concept* – As mentioned before the loading of the timber acts perpendicular to the wood fibres which means roughly one tenth the strength compared to if the timber would have been loaded axially.
- *Exposed concept* – The timber end grains will be exposed extensively in this concept and requires extra protection to ensure durability of the material.

5.8 The ORIGAMI

The last concept is inspired by the way paper can be folded into stable shapes. By transferring this into CLT panels one can create a stable timber shell of panels that in this situation can serve as both the roof for the bridge as well as the main load bearing system. Many variations of potential folding patterns can be imagined and as long as they avoid horizontal crests that can collect water. The global shape of the roof will, in order to ensure stability for both vertical and horizontal loads, benefit from a valve shaped geometry (see Figure 5.16). The more vertical sides can then carry most of the vertical loads while the top horizontal part carries horizontal loads. The straight shape in the figure can be imagined instead as a more arching roof for a more compression based load handling, rather than a global shape in bending.

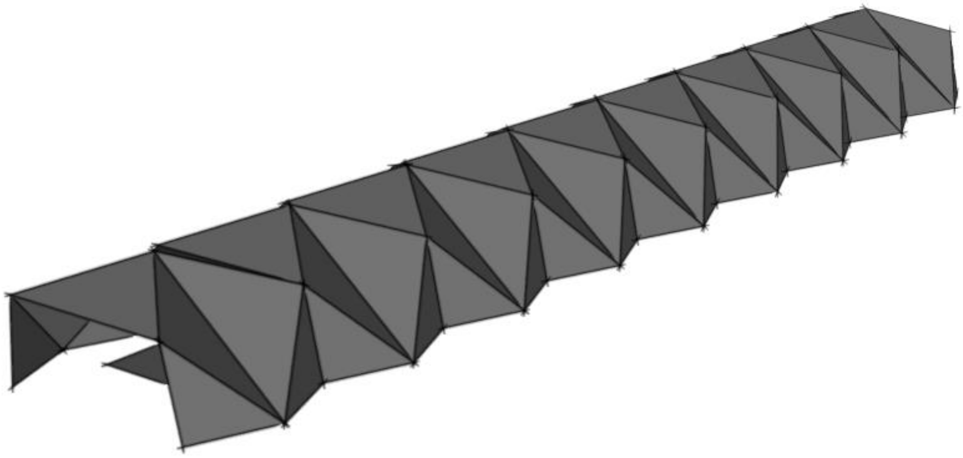


Figure 5.16 Sketch of the ORIGAMI concept

The bridge roof will in this concept work as a stiff structure from which the deck can be hung. Structurally the forces will be carried mainly in compression through the panels of the origami shaped roof, but to enable this some type of rigid long connections are needed. Perhaps long steel plates that are bent will be the simplest solution however this also risks a high cost. Another idea is to have longitudinal holes through the panels that enable tension cables to be run through the bridge at several points, tensioning the panels together into a compressed unit. The latter of course also comes with the risk of high costs.

Summary the ORIGAMI concept

- *Unusual concept* – Just as with the last concept this type of bridge has not been built before which might generate a lot of attention to a new structure of this type, perhaps helping to trigger the timber industry.
- *Heavy concept* – The use of solid CLT panels will give more mass to the bridge than most beam based designs, this extra weight can be used to make the bridge deform less under dynamic loads, where the added loads from people will be a smaller portion of the total loads.
- *Unconventional concept* – As a trade-off for the attention a new design gets the lack of experience for this bridge type can lead to an increase in costs and time for both design and construction.
- *Complex concept* – Since this type of structure transfers compression through a set of panels in a complex three-dimensional pattern the design calculations will be tricky to perform.

5.9 Concept Comparison

The process of elimination for the concepts at hand started with a realization analysis. While all of these concepts are assumed to be buildable, part of the goal was to maintain a reasonable budget. This does not necessarily mean that cheaper is better, but an increase in cost should bring a proportional improvement in the desired bridge features. With this in mind, the last two of the concepts were deemed too radical, thus unlikely to be able to be constructed in accordance to the previously mentioned budget criterion.

Secondly the elimination process focused on the desired criterion of an interesting bridge. The six remaining concepts all have qualities that can prove suiting for further analysis but two more have to go to avoid the work load to come becoming too extensive. The reasoning for the last elimination was based on the similarity between some concepts, the continuation of the project will cover more ground if the chosen concepts are different in behaviour. The similar concepts in question are The PRISM and The TUBE as well as The CROSS and The LEAF. In the interest of a wide variety in the continued design process the latter in the two pairs were chosen. It can be noted that there are no strong arguments against the last two rejected concept and that they definitively show promise. However with the remaining four a varied spectrum of shapes and structural behaviour is achieved which is why this set was decided on for further work.

To ensure sufficient time for the analysis the concepts were also ranked, so that if only time existed for three of them to be worked with, it would be the best of the four. The following ranking was decided based on the criteria of potential for an efficient design and interesting bridge geometry:

1. Concept 6 – **The TUBE**
2. Concept 5 – **The LEAF**
3. Concept 3 – **The SADDLE**
4. Concept 2 – **The I-BEAM**

6 Analysis Method – Karamba, Physical Models

Since the chosen concepts include numerous variations in the bridge designs, the chosen method of continued work was the *Rhinoceros 5* plugin *Grasshopper*. This plugin utilizes a parametric modelling system similar to programming code, this way a set of initial input variables can be the basis for the design and alterations in their values will generate variations in the bridge form. This means that when the parametric code is done the bridge geometry variables to be used for analysis can be changed and instantly provide an updated model for input into an analysis program.

For the actual analysis of the generated line and point geometry the Grasshopper plugin *Karamba 3d* was chosen. Since this FEM-analysis program is incorporated into Grasshopper the transition from model to analysis will be completely effortless and the problem of export/import through different file types is avoided. In order to explain Karamba the following sub-chapters will go through two calculation examples showing the methodology of Karamba as well as verifying the results provided by the program. Both of these examples are taken from the Structural Mechanics book: *Strukturmekanik* (Dahlblom O, Olsson K-G, 2010).

The architectural analysis will be partly based on the 3D modelling and the work with setting op the structures, however the main spatial analysis will come through the construction of a set of physical models based on the element geometries in the Karamba analysis. By adding a roof to the models, the spaces and the light in them can be analysed and desired adjustments be made.

6.1 KARAMBA Calculation Example 1 – Beam System

The first calculation to be performed is for the beam system from task 7-7 in the Dahlblom and Olsson book (see Figure 6.1).

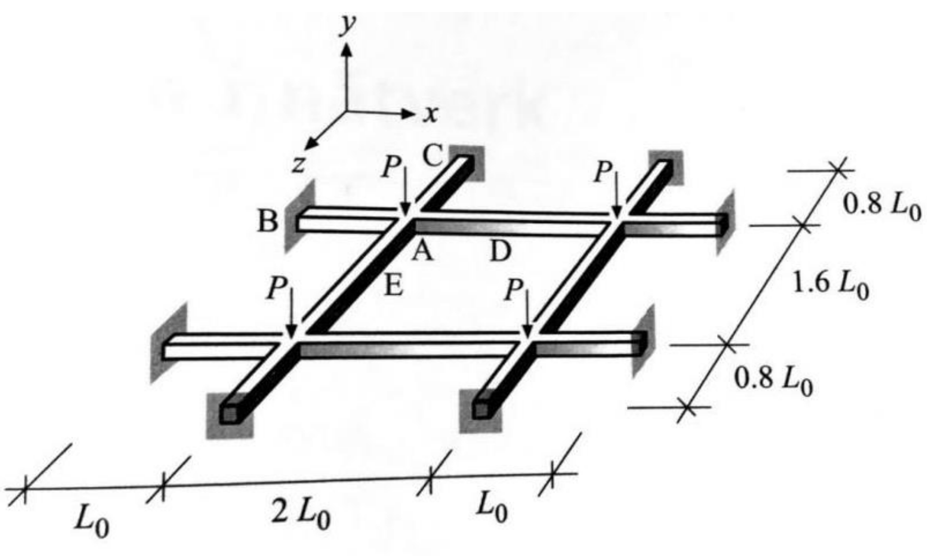


Figure 6.1 Karamba calculation example 1

The sectional data for the given beams along with the variables shown in the figure are given in the book as:

Table 6.1 Calculation example 1 input data

Geometry length	$\boldsymbol{L_0}$	$\mathbf{1,0}\ \boldsymbol{m}$
Point loads	$\boldsymbol{P}$	$\mathbf{20}\ \boldsymbol{kN}$
Section area	$\boldsymbol{A}$	$\mathbf{1,5 \cdot 10^{-3}}\ \boldsymbol{m^2}$
Section area moment of inertia	$\boldsymbol{I_{\bar{y}} = I_{\bar{z}}}$	$\mathbf{2,0 \cdot 10^{-6}}\ \boldsymbol{m^4}$
Section radius of gyration	$\boldsymbol{K_v}$	$\mathbf{3,0 \cdot 10^{-6}}\ \boldsymbol{m^4}$
Material elastic modulus	$\boldsymbol{E}$	$\mathbf{210}\ \boldsymbol{GPa}$
Material shear modulus	$\boldsymbol{G}$	$\mathbf{80}\ \boldsymbol{GPa}$

The symmetry of the beam system (as the node naming in the figure hints) enables for a simplification of the model into four elements. These elements along with the nodes they connect to are put into Rhinoceros according to the dimensions shown in Figure 6.2.

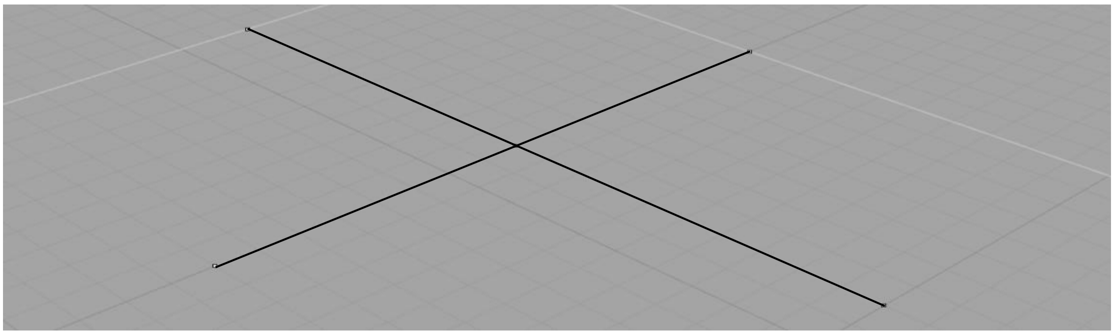


Figure 6.2 Input geometry for Karamba

For Karamba to perform the analysis the program will assemble a FEM-model based on the inputs shown in Figure 6.3. The *Assembly module* requires: Elements, Supports, Loads, Cross-Sections and Materials.

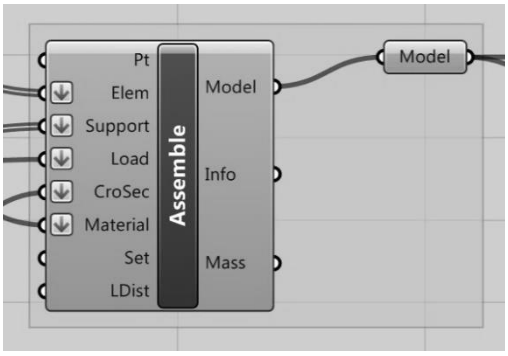


Figure 6.3 Karamba assembly module

The first of these, elements, are created from lines in the *Beam Elements module* (see Figure 6.4). Apart from the input line geometries the name of the elements corresponding to these lines is specified, here chosen as “Example Beam”.

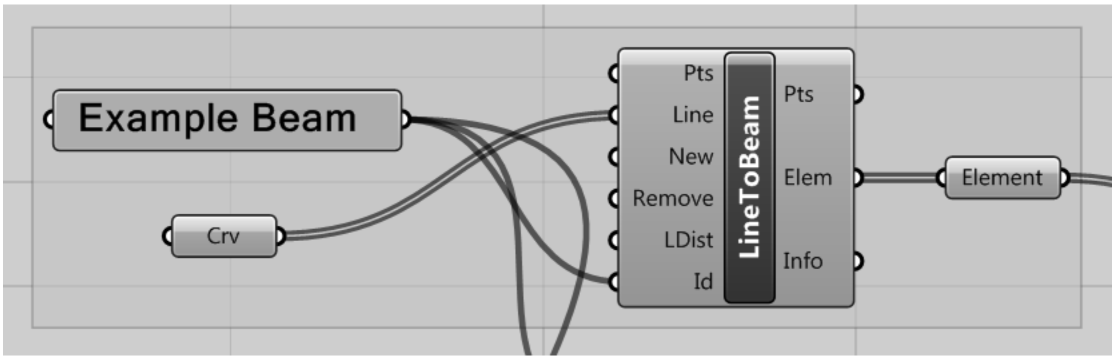


Figure 6.4 Karamba beam module

Next the support conditions are specified, in this model this means three different support types: One for the two fully fixed ends of the beams and one each for the x and y-axis symmetry boundaries. As Figure 6.5 shows these supports are defined by the input points and then locked for different degrees of freedom.

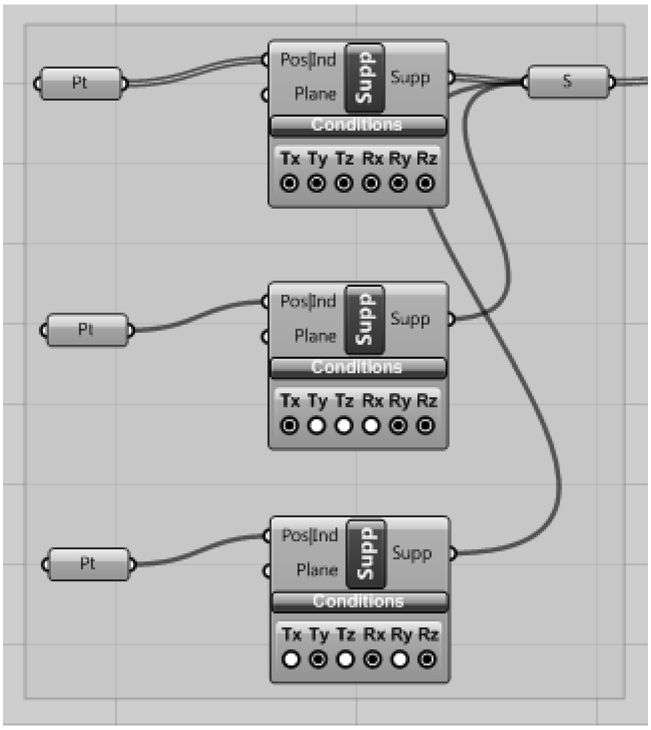


Figure 6.5 Karamba support modules

Karamba features many load types for a model, in this task the *Point Load module* is needed. The inputs consist of a vector defining the load and a point defining its location. The magnitude of this load is specified according to the given input giving the setup shown in Figure 6.6.

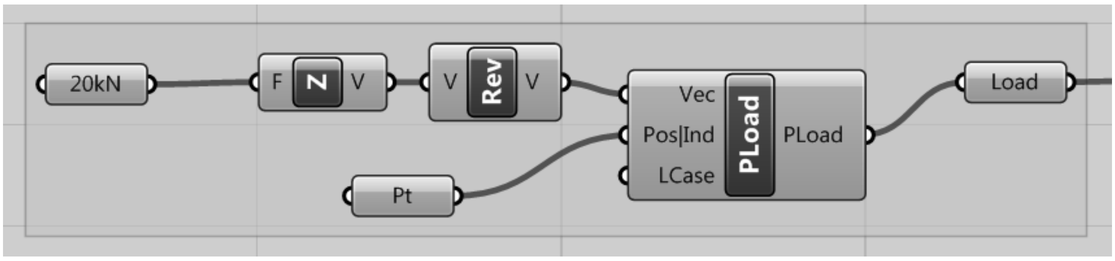


Figure 6.6 Karamba point load module

The next input is the cross-section data, which in this case will require a backwards engineered section based on the given data. Karamba creates the section data based on geometry only and the dimensions for which the given inputs are as stated above need to be determined. First a section shape is chosen, in this case a hollow square box section is convenient due to its simple calculation of sectional data. By setting up an equation system for the section area and area moment of inertia the following is generated:

$$\begin{cases} {d_{out}}^2 - {d_{in}}^2 = 1{,}5\ \cdot 10^{-3}\, m^2 \\ \dfrac{{d_{out}}^4 - {d_{in}}^4}{12} = 2{,}0\ \cdot 10^{-6}\, m^4 \end{cases}$$

$$for\ d_{out} = outer\ side\ length\quad d_{in} = inner\ side\ length$$

The solution this yields is:

$$\begin{cases} d_{out} = \sqrt{7/2}/20 = \mathbf{9{,}354\ cm} \\ d_{in} = \sqrt{29/10}/20 = \mathbf{8{,}515\ cm} \end{cases}$$

The remaining section constant, the radius of gyration, is more complex to calculate and will thus be disregarded. A rough estimation based on solid rectangular sections according to Chapter 7 in *Strukturmekanik* (Dahlblom O, Olsson K-G, 2010) can be calculated according to Equation 6-1.

$$0{,}846 \cdot \left(I_{\bar{y}} + I_{\bar{z}}\right) = 3{,}384 \cdot 10^{-6}\, m^4 \qquad (6\text{-}1)$$

This is relatively similar to the given value of $K_v = 3{,}0 \cdot 10^{-6}\, m^4$ and the above calculated dimensions are assumed to be accurate enough to test the validity of the program, keeping in mind the deviating radius of gyration for conclusions on the final result.

Along with the desired cross section name, chosen as “Example Section”, and the names of the elements for which this section is assigned the dimensions calculated are put into the Karamba *Box section-module* as shown in Figure 6.7.

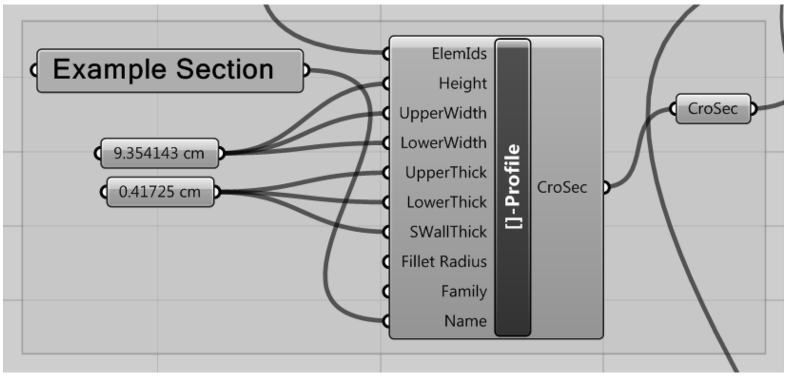


Figure 6.7 Karamba box profile module

Finally the materials are specified, the Karamba module for material properties has several inputs depending on the analysis to be made but for this task the elastic and shear moduli are the relevant ones. Just as with the cross-section module the inputs also consist of a name, here chosen as “Example Material”, and the name of the elements for which this material is assigned. This setup is shown in Figure 6.8.

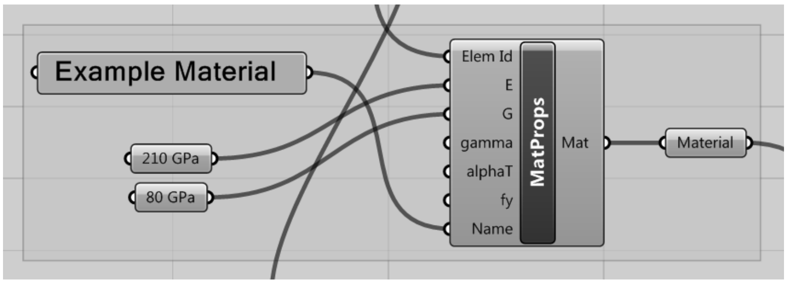


Figure 6.8 Karamba material module

The model is now assembled and different analyses can be performed, first the setup can be controlled by displaying the model, shown in Figure 6.9.

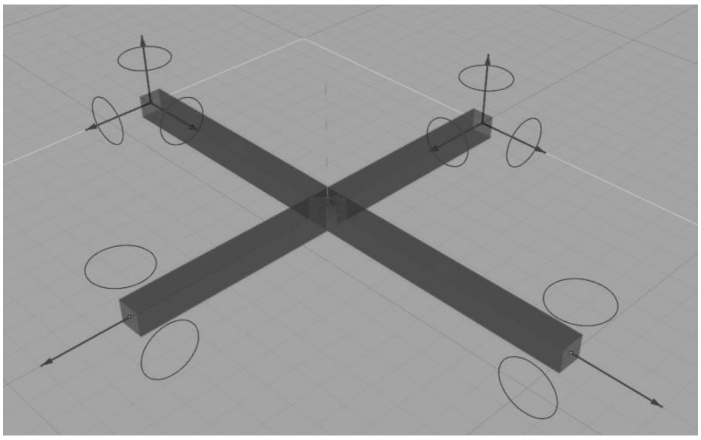


Figure 6.9 Karamba model display

The figure looks correct and the supports displayed correspond to the desired support conditions. Another possible check is to display tags for the elements regarding their numbering or assigned properties. In Figure 6.10 this display is shown for the node numbering, element numbering, element names, element materials and element sections, all of them correctly assigned as desired.

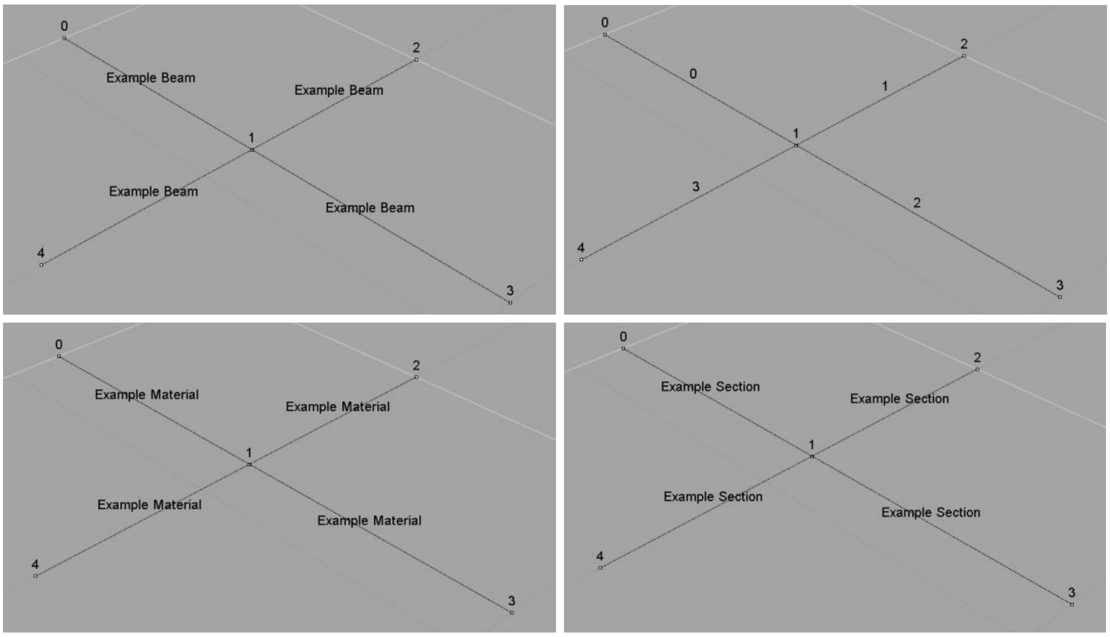


Figure 6.10 Karamba model display with various tags

The Karamba model is then attached to a static analysis module to perform the calculations and the desired results can subsequently be extracted. The first of these is for the deformations, Figure 6.11 shows an extract from the *Nodal Displacements module*:

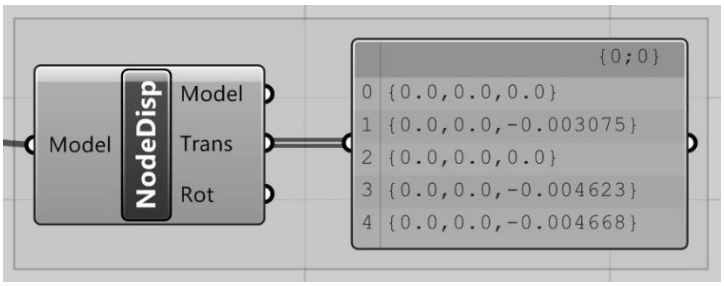


Figure 6.11 Karamba nodal displacements output

The displacement sought after in the task was the maximum vertical displacement, the maximum calculated above along with the corresponding value from the answers (Dahlblom and Olsson, 2010) are shown in Table 6.2

The second sought after result is the sectional forces in the supports. Similarly there is another Karamba module that extracts the reactions at the supports, this output can be seen in Figure 6.12.

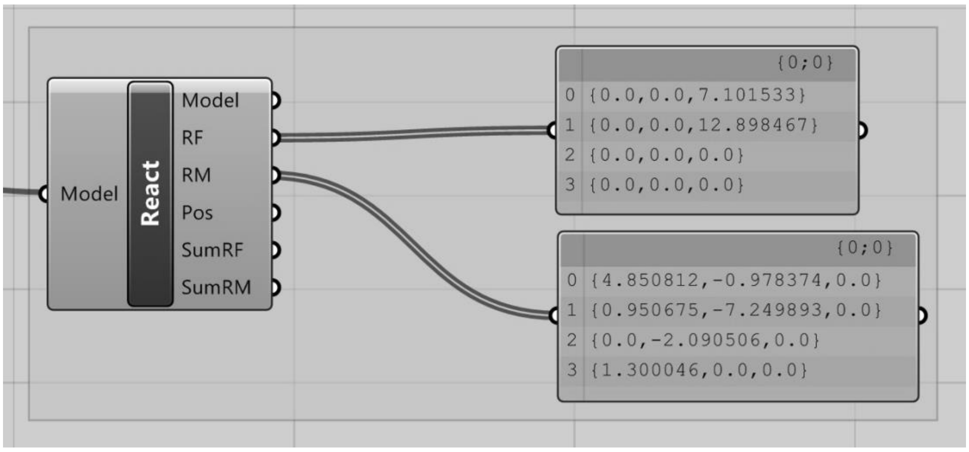


Figure 6.12 Karamba support reactions output

Checking these values compared to the known answers the following comparison can be set up from the two sources, see Table 6.2:

Table 6.2 Calculation example 1 result comparison

	Dahlblom and Olsson	***Karamba***	***Deviation***
Maximum displacement	***4,541 mm***	***4.668 mm***	***2,80%***
Shear force at 0	***7,014 kN***	***7,102 kN***	***1,25%***
Shear force at 1	***12,986 kN***	***12,898 kN***	***-0,67%***
Bending moment at 0	***4,799 kNm***	***4,851 kNm***	***1,01%***
Bending moment at 1	***7,308 kNm***	***7,250 kNm***	***-0,79%***
Torsion moment at 0	***0,955 kNm***	***0,978 kNm***	***2,41%***
Torsion moment at 1	***0,923 kNm***	***0,951 kNm***	***3,03%***

The conclusions to be made from this calculation are that the generated results are very similar to those of the known answers. The deviations are relatively small and might be contributed to the slightly different sectional constants. In addition, it can be noted that the deviations are mostly conservative by comparison (larger deformations and forces) and the only exceptions to this are for the two values that deviate the least. Based on these results, the program certainly could be used for this project where the loss in accuracy is heavily compensated for in the simplification in the calculation process. However, one more example for hinged elements will make sure that the program also handles trusses as desired.

6.2 KARAMBA Calculation example 2 – Truss System

In order to assure that Karamba works for joints that are not rigid, which is very relevant for a timber project, a hinged system task has also been performed. The chosen task is also from the Dahlblom and Olsson book, task 7-3, which can be seen in Figure 6.13.

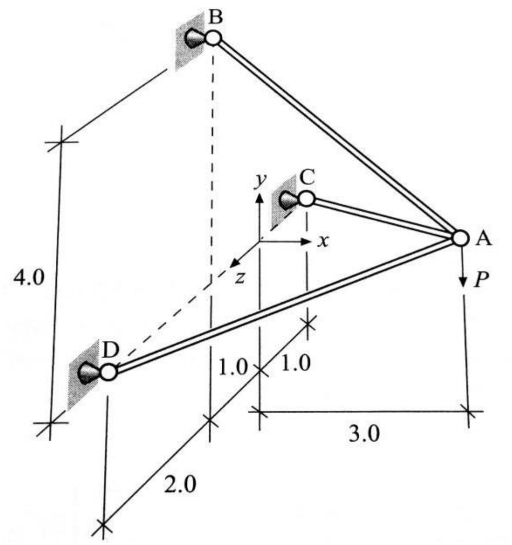


Figure 6.13 Karamba calculation example 2

Apart from the input geometry in the figure, the data in Table 6.3 shows the given task setup. The greatly reduced number of inputs hint to the more simple nature of a truss analysis compared to when bending is considered.

Table 6.3 Calculation example 2 input data

Point load	P	$50\ kN$
Section area	A	$4,0\ \cdot 10^{-4}\ m^2$
Material elastic modulus	E	$200\ GPa$

There are extensive similarities in the modelling process in this task compared to the previous one, therefore only the different modules are shown here. The rest of the process followed the same procedure as before. The main difference is in the joint definition, which in the previous task was not addressed. This leads to Karamba using the default joints consisting of rigid connections. In the case of hinges, there are two ways of creating moment free connections. The first of these is the simpler of the two, but it is limited to completely hinged connections. While this would work for this task, it is more appropriate to use the other since it will be relevant for the bridge design that follows.

The Karamba module for beam joints is best used by first assigning all the supports that are affected as rigid. Then, as Figure 6.14 shows, the starts and ends of the relevant elements are defined for connections in their local axes. To avoid an unstable model where the torsional rotation of the truss elements can get out of hand this rotation is not unlocked. This will be controlled not to induce forces later and is merely a modelling trick for calculation stability.

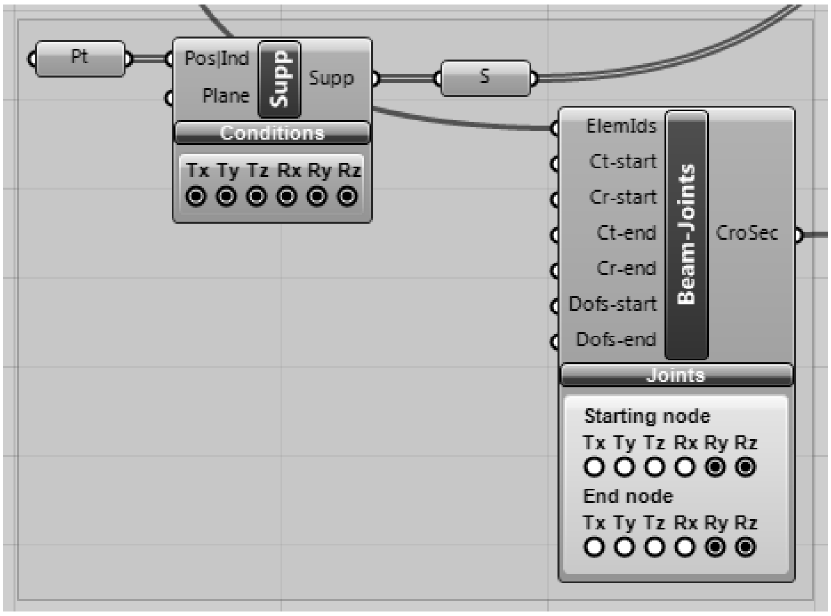


Figure 6.14 Karamba joint module

With both bending moments at each node released, the model can be completed. The corresponding model display at this stage to the one shown in the previous task can be seen in Figure 6.15. Note that the model shows rigid supports and then added local rotations in the nodes (white circles).

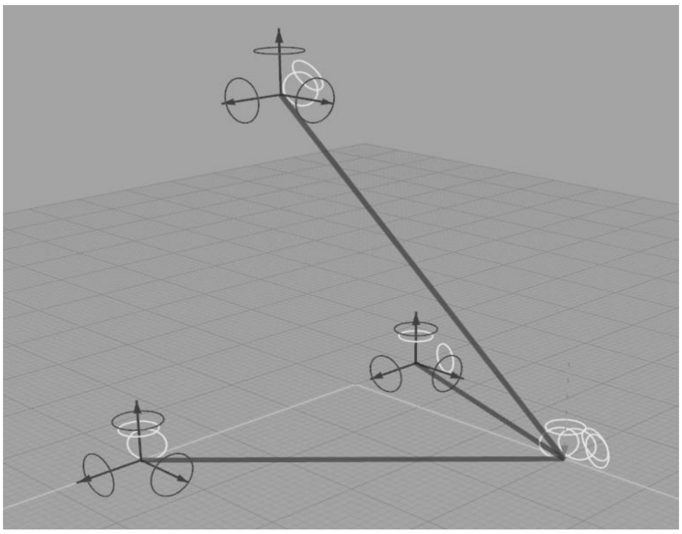


Figure 6.15 Karamba model display

Just as before this model can now be analysed for the desired results, the first of these is for the nodal displacement of the free node, shown in Figure 6.16.

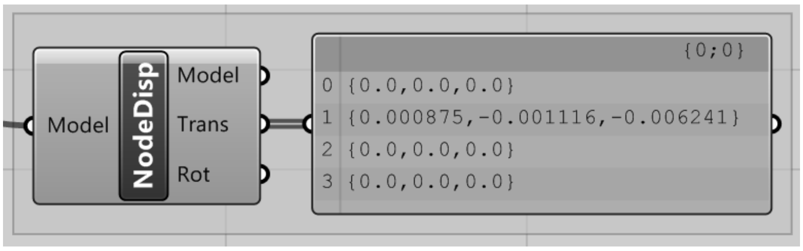


Figure 6.16   Karamba displacements output

Apart from the displacements the normal forces were sought, from the same module the section moments can be extracted. This is also done to ensure that the nodes indeed have no moments, as can be seen in Figure 6.17.

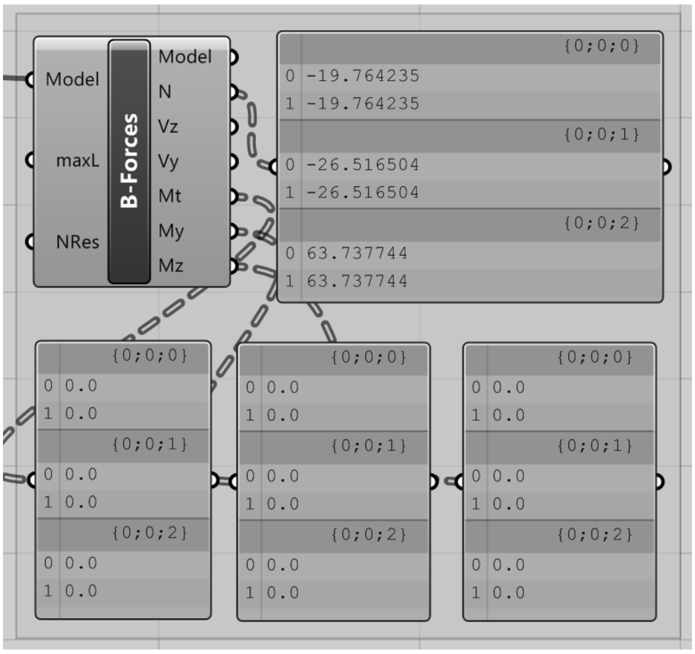


Figure 6.17   Karamba section forces output

The results from Karamba, applying the controlled hinge behaviour, can now be compared to the known answers (Dahlblom and Olsson, 2010). This is summarized in Table 6.4.

Table 6.4 Calculation example 2 result comparison

	Dahlblom and Olsson	Karamba	Deviation
Displacement x	**1.115 mm**	**1,116 mm**	**0,1%**
Displacement y	**6,233 mm**	**6,241 mm**	**0,1%**
Displacement z	**0,874 mm**	**0,875 mm**	**0,1%**
Normal force element 0	**-19,76 kN**	**-19,76 kN**	**0%**
Normal force element 1	**-26,52 kN**	**-26,52 kN**	**0%**
Normal force element 2	**63,74 kN**	**63,74 kN**	**0%**

As shown in the table, the model was very functional for trusses as well. The minor deviation in displacements probably comes from the rounding error made when the circular bar elements were created to match the given area in the task.

In conclusion Karamba seems to be working extremely well and especially for this stage in the design it will be very useful as the analysis tool.

6.3 Analysis Visualisation

Before moving on, the planned visualizations of results from Karamba need to be addressed. The program has a built in module that can display deformations and utilizations of the finite elements in the model, these plots however have limited applications since the program scales the results in a way that the figures only show the relative values in each model.

The first of these built in visualisation modules is the *Deformation module* shown in Figure 6.18, displayed for calculation example 2. This module allows for locating of the largest deformations in the model, however when comparing to other models the scaling function makes the comparison difficult. For example an identical setup with only doubled load will look almost identical despite the large difference in deformations.

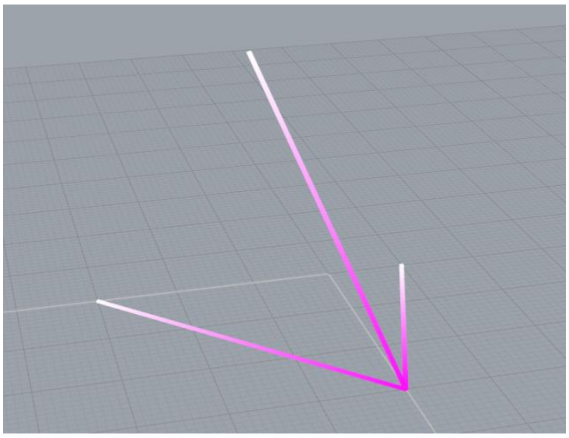


Figure 6.18 Karamba displacement visualisation module display

The second built in visualisation module is the utilization one shown in Figure 6.19, here for calculation example 1 since the module is more clear for elements in bending. This module plots the utilization of each finite element in the model on a scale from maximum tension (blue) to maximum compression (red). The problem with this module is that the utilization does not scale to account for neither over utilization nor disproportional stresses. The first of these means that the most blue areas can be utilization of many times the tensile capacity. This does not enable a conclusion to be made on if the system fails due to over utilization of the material. The second means that the intuitive interpretation that all red zones are compressed and all blue zones are in tension is incorrect. In fact in an extreme case with no compression the entire scale of colours will be different levels of tension.

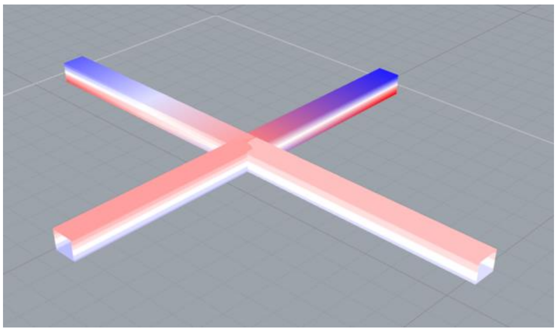


Figure 6.19 Karamba utilization visualisation module display

To counter these problems two new visualization modules were made that provide more absolute results for comparison and quick understanding of a structure. The first of these is a display of tension and compression. This can come in handy in an optimization situation where for instance the purely tensioned members can be identified and designed without regard to buckling.

This new module was made using the modules explained before that extract sectional forces in each element. This info was then used to plot the elements in one colour for the whole element, deep blue for all members in compression and bright red for all members in tension. In addition a yellow colour was added to all members without normal force, thus being without load or in pure bending. This new plot is shown for calculation example 2 in Figure 6.20, with the results looking good.

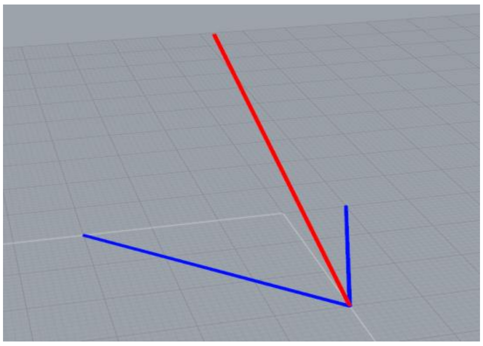


Figure 6.20 Karamba tension/compression display

The second new module serves to improve the *Utilization module* for a more clear display. The principal of colouring each element in one colour is used here as well, thus symbolizing the most utilized part of each element. The chosen colours are based on the colour map shown in Figure 6.21. The utilization plot as described above spans between zero and 100 per cent usage ranging from a light blue to red. For elements with more than full usage the colour becomes deep purple thus symbolizing a material utilization over 100%.

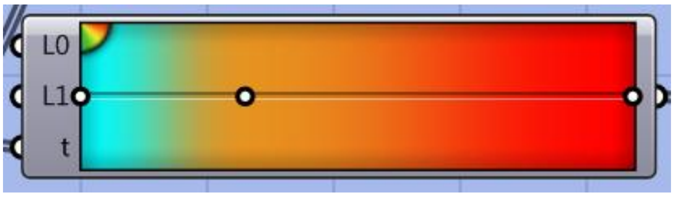


Figure 6.21 Karamba absolute utilization colour map

The Karamba module that calculates utilization can also check whole elements. This is done in at least three internal places (centre and both ends) where the capacity is checked for both failures in exceeding the strength as well as buckling. These utilizations can then be plotted with the above described colours, as shown for calculation example 2 in Figure 6.22.

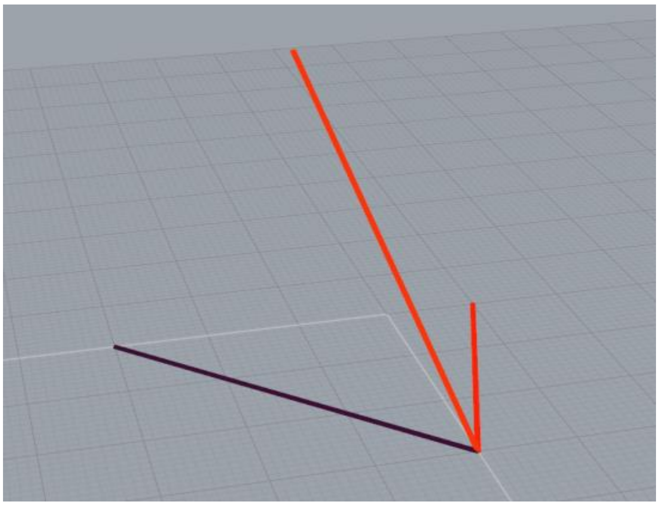


Figure 6.22 Karamba absolute utilization display

As showed in Figure 6.22, the truss has a very high usage and one element is even beyond its capacity. Looking back at the results from this example it can be noted that the tension member in the top is more loaded than the compression ones in the bottom, since the bottom ones still have a higher usage in the plot it can be concluded that the model does account for buckling.

6.4 Material Input

Before moving into the analysis some preparations need to be made regarding the material inputs. First of all the materials to be used need to be chosen, the rough nature of this structural analysis goes well with a simplified approach of choosing one glulam material, one steel rod material and one tension steel cable material.

6.4.1 Glulam Members

The timber used in the analysis is **GL30c,** the most common glulam class in Sweden, which has the characteristic properties according to *Eurocode EN 14080:2013,* shown in Table 6.5

Table 6.5 GL30c properties

	Capacity analysis	*Deformation/Dynamic analysis*
Bending parallel $\boldsymbol{f_{m,g,k}}$	**30 MPa**	**30 MPa**
Tension parallel $\boldsymbol{f_{t,0,g,k}}$	**19,5 MPa**	**19,5 MPa**
Tension perpendicular $\boldsymbol{f_{t,90,g,k}}$	**0,5 MPa**	**0,5 MPa**
Compression parallel $\boldsymbol{f_{c,0,g,k}}$	**24,5 MPa**	**24,5 MPa**
Compression perpendicular $\boldsymbol{f_{c,90,g,k}}$	**2,5 MPa**	**2,5 MPa**
Shear strength $\boldsymbol{f_{v,g,k}}$	**3,5 MPa**	**3,5 MPa**
Rolling shear strength $\boldsymbol{f_{r,g,k}}$	**1,2 MPa**	**1,2 MPa**
Elastic modulus $\boldsymbol{E_{0,g,05}/\ E_{0,g,mean}}$	**10800 MPa**	**13000 MPa**
Shear modulus $\boldsymbol{G_{0,g,05}/\ G_{0,g,mean}}$	**540 MPa**	**650 MPa**
Density $\boldsymbol{\rho_{g,mean}}$	**390 kg/m^3**	**430 kg/m^3**

These material properties need to be adjusted to the design values in accordance with *Eurocode SS-EN 1995-1-1, chapter 2.4.*. This adjustment is based on load duration, service class, timber material and material dimensions as shown in Equation 6-2.

$$f_d = \frac{k_h \cdot k_{mod} \cdot f_k}{\gamma_M} \qquad (6\text{-}2)$$

The characteristic values to be put into this equation can be found in Table 6.5. The next step is to find the other factors. The first of these is the partial factor γ_M which according to *Eurocode SS-EN 1995-1-1, table 2.3* is:

$$\boldsymbol{\gamma_{M,capacity} = 1,25}$$

$$\boldsymbol{\gamma_{M,deformation} = 1,0}$$

Secondly the k_{mod} factor is determined, for this both service class and load duration is needed. According to *Eurocode SS-EN 1995-1-1, table 2.2* the load duration classes for the relevant loads in this project are *Permanent* (P) for self-weight, *Medium-term* (M) for snow load and imposed loads and *Short-term* (S) for wind loads. These classes will reduce the characteristic strength more for the longer durations, a conservative approach would therefore be to choose the longest one. However Eurocode also states that for a collection of loads the shortest duration should be chosen, as a compromise the medium term will be used for this analysis (conservative approach). The service classes depend on the environment for which the timber will be exposed. Here the covered bridge approach will provide some extra capacity to the timber where *Eurocode SS-EN 1995-1-1, chapter 2.3.1.3* specifies service class 2 instead of service class 3 used for uncovered bridges. The following k_{mod} factor can now be used based on *Eurocode SS-EN 1995-1-1, table 3.1*

$$\boldsymbol{k_{mod} = 0,8}$$

Finally the k_h factor is addressed, giving a slight strength increase for small members in bending and tensile strength. Since the dimensions of the members have not yet been specified a conservative approach would be to disregard this factor. However since the last factor was conservatively approached and chances are many components in the bridges will be smaller than the limit of 600 mm in height, a lesser increase can be used. Equation 6-3 from *Eurocode SS-EN 1995-1-1, chapter 3.3* determines this factor as:

$$k_h = min\begin{cases}\left(\dfrac{600}{h}\right)^{0,1} \\ 1,1\end{cases} \tag{6-3}$$

Assuming a member height of 400 mm this comes to $\mathbf{k_h = 1,04}$ which will be used for the increase in strength. This assumption will somewhat cancel out with the previous one for load duration and the effects of this decision will thus hopefully generate a model that does not deviate from the real behaviour of timber.

As displayed in Figure 6.8 the Material module in Karamba requires an input of elastic modulus, shear modulus, density and strength. Since the first two differ significantly for capacity and deformation checks two sets of timber materials will be specified, one capacity glulam and one deformations glulam.

One problem now is the isotropic nature of the material that is assumed with only one strength input. Since timber is orthotropic with greatly varying characteristics in different directions this will have to be considered in the analysis. The assumption of an isotropic material is not problematic for slender members such as beams but for boards this will be inaccurate. The chosen solution for this is to assume the timber strength parameters parallel to the fibres and replace boards with crossed beams in the model. The second issue here is the difference in strength for timber depending on if bending, tension or compression is applied. The lower of these is for tension which will yield a conservative approach, but with the large difference between the three this might be over-conservative and lead to dimensions larger than needed. The chosen compromise here is to use the average strength between pure tension and pure compression and allow for the usage of the factor k_h in the calculations. The following design timber strengths will thus be used:

$$\boldsymbol{f_{d,capacity}} = \frac{k_h \cdot k_{mod} \cdot \left(\frac{f_{t,0,g,k} + f_{c,0,g,k}}{2}\right)}{\gamma_{M,capacity}} = \mathbf{14,64\ \boldsymbol{MPa}} \qquad (6\text{-}4)$$

$$\boldsymbol{f_{d,deformation}} = \frac{k_h \cdot k_{mod} \cdot \left(\frac{f_{t,0,g,k} + f_{c,0,g,k}}{2}\right)}{\gamma_{M,deformation}} = \mathbf{18,30\ \boldsymbol{MPa}} \qquad (6\text{-}5)$$

For the timber material inputs in Karamba this means the values given in Table 6.6 will be used:

Table 6.6 GL30c material data for Karamba

	*Capacity analysis**	*Deformation/Dynamic analysis*
Material Strength	**14,64 MPa**	**18,30 MPa**
Elastic modulus	**10800 MPa**	**13000 MPa**
Shear modulus	**540 MPa**	**650 MPa**
Density	**3830 N/m^3**	**4223 N/m^3**

** Based on characteristic values*

6.4.2 Steel Rod Members

The chosen steel type for tensile truss components in the analysis is the highest of the low grade type, namely **S460**. This material has the following characteristics, shown in Table 6.7, based on *Eurocode SS-EN 1993-1-1*, *table 3.1* and *chapter 3.2.6*, assuming an element thickness of over 40mm for conservative reasons:

Table 6.7 S460 steel rod properties

Steel yield strength $\boldsymbol{f_y}$	**430 MPa**
Steel ultimate strength $\boldsymbol{f_u}$	**530 MPa**
Elastic modulus $\boldsymbol{E}$	**210 GPa**
Shear modulus $\boldsymbol{G}$	**81 MPa**

In order to use these values the type of forces acting on the steel are needed, here the assumption of tension members is made thus enabling the use of the tensile members conditions. The following two equations show the tension force capacity for steel members according to *Eurocode SS-EN 1993-1-1*, *chapter 6.2.3*:

$$\boldsymbol{N_{pl,Rd}} = \frac{A \cdot f_y}{\gamma_{M0}} \tag{6-6}$$

$$\boldsymbol{N_{u,Rd}} = \frac{0{,}9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} \tag{6-7}$$

For Karamba to be able to use the material inputs, the definition of capacity needs to be a stress limit, thus the relevant information is given by dividing the tensile force by the area. The partial coefficients in the formulae are given in *Eurocode SS-EN 1993-1-1*, *chapter 6.1* as:

$$\boldsymbol{\gamma_{M0} = 1{,}0}$$
$$\boldsymbol{\gamma_{M2} = 1{,}25}$$

By inserting these factors in Equations 6-6 and 6-7 it proves that Equation 6-7 will be the critical one yielding a maximum stress of:

$$\boldsymbol{f_{capacity}} = \frac{N_{u,Rd}}{A_{net}} = \frac{0{,}9 \cdot f_u}{\gamma_{M2}} = \boldsymbol{381{,}6\ MPa} \tag{6-8}$$

Apart from the now obtained data the Karamba input also needs the density. From *Eurocode SS-EN 1991-1-1, appendix A4* this is defined as:

$$\boldsymbol{\gamma_{steel} = 77,0\ to\ 78,5\ kN/m^3}$$

An intermediate of the above stated interval is assumed giving the input information stated in Table 6.8 for the Karamba analysis:

Table 6.8 S460 steel rod material data for Karamba

Steel strength	**382 MPa**
Elastic modulus	**210 GPa**
Shear modulus	**81 GPa**
Density	**78 kN/m^3**

6.4.3 Steel Cable Members

The last of the materials to be used is the high capacity tensile steel cables. In difference to the steel rods above Eurocode has a separate control process for tension cables. The following three formulae are used to determine the Force capacity of a cable according to *Eurocode SS-EN 1993-1-11, chapter 6.2*:

$$F_{Rd} = min\left\{\frac{F_{uk}}{1,5 \cdot \gamma_R}, \frac{F_k}{\gamma_R}\right\} \tag{6-9}$$

$$F_{uk} = F_{min} \cdot k_e \tag{6-10}$$

$$F_{min} = \frac{K \cdot d^2 \cdot R_r}{1000}\ [kN] \tag{6-11}$$

In the last of these the strand strength R_r is included, for a solid steel cable this would have been the strength value to use. However, for the task at hand, a more appropriate input would be the equivalent strength for a cable based on its force capability divided by the nominal diameter. The value of tested tension force strength F_k in the top formula is a manufacturer table value, in order to know if this value is more critical than the equivalent strength based on the steel class the formulae above have to be combined. The combination will provide the following equation:

$$F_{Rd} = min\left\{\frac{K \cdot d^2 \cdot R_r \cdot k_e}{1500 \cdot \gamma_R}, \frac{F_k}{\gamma_R}\right\} \tag{6-12}$$

Using Equation 6-12, the two cable forces can now be compared and the lower one turned into an equivalent cable strength. For the comparison to be carried out, the included constants have to be determined. The first of these, the failure force factor K, varies between cable types. The type used for this project is thus here assumed as a **spiral strand rope**. Within the specified type of cable there are different values for K depending on the size of the cable and thus the number of strand layers used according to *Eurocode SS-EN 1993-1-11, appendix C2*. These values vary between 0,525 and 0,51 with the latter being used for larger cables. Since a lower value will be conservative, the slightly shifted average of 0,515 is chosen.

The next factor to be used is the end termination loss factor k_e, which also will lower the capacity with reducing size. The different end types given in *Eurocode SS-EN 1993-1-11, table 6.3* give values ranging from 1,0 down to 0,8. As a compromise the average of 0,9 is assumed for this task.

The final factor γ_R depends on if measures are taken towards reducing the bending forces at the attachments. If this is the case *Eurocode SS-EN 1993-1-11, table 6.2* specifies the value as 0,9 giving a slight capacity increase. Conservatively the value used here will thus be set as 1,0 assuming no measures of bending reduction have been made.

With all the factors determined, all that remains is to extract table values for the cable type to be used. An average of two cable dimensions will help make the assumptions made here more accurate. Table 6.9 contains this data from one of the manufacturers of steel cables:

Table 6.9 Spiral strand ropes characteristics

	Cable size 1	Cable size 2
Cable nominal diameter $\boldsymbol{d}$	**22 mm**	**40 mm**
Strand tensile strength $\boldsymbol{R_r}$	**1770 MPa**	**1770 MPa**
Cable breaking load $\boldsymbol{F_k}$	**259 kN**	**863 kN**

Putting these values into Equation 6-12, both of the cable types yield a lower capacity for the cable breaking loads. These are therefore used to reverse engineer an equivalent cable strength. This comes to 681 MPa and 687 MPa, the value assumed in the analysis will thus be something intermediate, namely **685 MPa**.

The next characteristic of the cables needed is the elastic modulus. This is given in *Eurocode SS-EN 1993-1-11, table 4.1* where the equivalent modulus for variable loads for spiral strand ropes is:

$$\boldsymbol{E_Q = 150 \ \pm 10 \ GPa}$$

From the elastic modulus the corresponding shear modulus can be estimated. For an isotropic material the relationship between elastic modulus and shear modulus given in *Eurocode SS-EN 1993-1-1, chapter 3.2.6* is:

$$G = \frac{E}{2(1+\nu)} \tag{6-13}$$

For steel, the value of poisons ratio is 0,3 yielding a shear modulus of roughly 58 GPa. In the case of a cable, the material however is not isotropic and this value will therefore not be accurate. Knowing the exact value of the shear stiffness of a steel cable requires some work and since this factor is not critically important to the model it is therefore assumed very low. A composite of strands has a significantly lower shear modulus than a solid of said material, due to the sliding effect of strands. For this calculation the shear modulus above is therefore greatly reduced down to about one tenth of the corresponding isotropic modulus.

Finally the density is needed, which due to the space in between the strands will be less than for the steel rod. In *Eurocode SS-EN 1993-1-11, chapter 2.3* this value is calculated using a fill factor (*f*) defining the ratio between steel area and bulk, given for various spiral strand ropes. Just as with the K-factor before these vary depending on number of layers, their average is 0,75 which will be used to calculate the reduced density of a cable. In addition, the density of the strand steel is defined here as 83 kN/m^3, this leads to the following calculation of density:

$$\gamma_{cable} = \gamma_{strand} \cdot f = 83 \cdot 0{,}75 = 62{,}25\ kN/m^3 \tag{6-14}$$

To account for the possibility of a plastic coating to protect the cable this value is rounded up to 65 before being put into Karamba.

This all together yields the Karamba inputs shown in Table 6.10.

Table 6.10 Steel spiral strand rope material data for Karamba

Steel strength	**685 MPa**
Elastic modulus	**150 GPa**
Shear modulus	**5 GPa**
Density	**65 kN/m^3**

6.5 Load Input

Apart from the material properties defined in the previous section, Karamba also needs some ground work for the loads before performing the analyses. The following section treats the Eurocode loading specifications for a timber bridge design.

6.5.1 Load Combinations

The loads that are put into Karamba should be combined in accordance with Eurocode, these combinations are different for capacity and deformation analyses. For capacity checks the relevant combination is Eq. 6-15, according to *Eurocode SS-EN 1990, chapter 6.4.3.2*.

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}+\gamma_p P_k+\gamma_{Q,1}Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \tag{6-15}$$

The partial coefficients γ here depend on if the load is permanent or variable and favourable or unfavourable. Most of the loads that are included in this project are working together in an unfavourable manner, the only exception is the uplift due to wind. Favourable forces that also are variable are disregarded since the most critical situation is given when their positive effect is absent. The partial factor for pre-stressing is in *Eurocode SS-EN 1993-1-11, 5.3* specified to be the same as for permanent loads. The following partial factors, as shown in Table 6.11, should therefore be used according to *Eurocode SS-EN 1990, appendix A2.4(B):*

Table 6.11 Partial factors for capacity analysis

	Unfavourable	*Favourable*
Permanent/pre-stressing loads $\boldsymbol{\gamma_G/\gamma_p}$	**1,35**	**1,0**
Variable loads $\boldsymbol{\gamma_Q}$	**1,5**	**0**

The second load combination set that will be treated is for the deformations. Eurocode separates between three different serviceability state combinations: Characteristic, Frequent and Quasi-permanent combination. For this analysis where deformations and natural frequencies will be looked at the most relevant of these is the characteristic one. The combination given for this is Eq. 6-16, according to *Eurocode SS-EN 1990, chapter 6.5.3*.

$$\sum_{j\geq 1}G_{k,j}+P_k+Q_{k,1}+\sum_{i>1}\psi_{0,i}Q_{k,i} \tag{6-16}$$

For both capacity and deformations all the needed information except one factor has been determined, the ψ-factor. From *Eurocode SS-EN 1990, appendix A.2.2* this factor as defined for bridges is shown in Table 6.12:

Table 6.12 Psi-factors for different load types in bridge design

Traffic loads	**0,4**
Wind loads	**0,3**
Snow loads	**0,8**

The values above should be used with their corresponding load type when acting as a secondary variable load, both for capacity and deformation analysis. While this provides many combinations of main/secondary loads, only one is likely to be the critical one.

6.5.2 Self-Weight

The effects of self-weights in Karamba is handled automatically provided the input densities are correct. The partial factor increasing the permanent loads in the previous section can be handled by multiplying the gravity vector by 1,35 thus increasing its effect on the structure.

What needs to be looked at regarding self-weights is the weight of permanent elements that are not included in the elements model. These loads will be relevant as an addition to the current loads for capacity analysis and as an increase in mass for the dynamic analysis. The way Karamba can handle additional mass is by specifying point-masses to be added numerically and they will be included in the dynamic behaviour. This will not affect the static analysis and they will therefore also have to be added here as well in the same way as imposed or snow loads.

The two loads that need to be added are the deck and roof systems. The conservative way of assuming these is to over-estimate the load for the static analysis while doing the opposite for the added dynamic mass. This will here be done by assuming a plausible setup and deviation interval, then the max and min of this interval will be used respectively in the above mentioned way.

Firstly looking at the deck, the assumption will be based on the load of a sample square meter of deck area. The components that will be used are a secondary beam system over the ones included in the model, a deck layer of planks to be walked on and additional components such as metal fasteners and the railing. An assumed calculation of this mass is shown in Eq 6-17

$$\gamma_{timber}(1m \cdot 1m \cdot 28mm + 45mm \cdot 90mm * 2) + F_{railing\ etc} \approx 0{,}2kN/m^2 \quad (6\text{-}17)$$

The calculation in Equation 6-17 assumed a solid layer of timber planks with a thickness of 28mm and two secondary beams thus giving the planks support every 500 mm. The density for timber was set to the same as for the structural glulam beams and the added mass for railings etc was assumed as about 50 N/m^2 when dispersed over the whole deck. The assumed deviation interval for this load is assumed as plus or minus 50%.

For the roof, a similar system will be used, of course depending on the not yet decided roof cover. If a timber, tar paper or metal roof system is used there will be little difference in the loads, the same mass will therefore be assumed. The extra self-weights are summarized in Table 6.13, before they have been modified with load combination factors.

Table 6.13 Partial factors for capacity analysis

	Static analysis	*Dynamic analysis*
Deck addition	**0,25 kN/m^2**	**15 kg/m^2**
Roof addition	**0,25 kN/m^2**	**15 kg/m^2**

6.5.3 Imposed Loads

The imposed loads on the bridge deck from pedestrian and bicycle traffic are according to *Eurocode SS-EN 1991-1-2, chapter 5.3.2.1* defined as 5 kN/m^2 for areas more prone to dense crowding. For the deck in general, the following formula is used, see Eq. 6-18

$$q_k = 2{,}0 + \frac{120}{L+30} \qquad 2{,}5\ kN/m^2 \le q_k \le 5{,}0 kN/m^2 \tag{6-18}$$

Where the length of the bridge (L) is put in to retrieve the value **4,0 kN/m^2**. This value will be used throughout the bridge except if there is a designated pause area which then will be given the higher imposed load.

Apart from the uniformly distributed load, two types of concentrated loads are described in Eurocode, the first is a characteristic focused load from a service vehicle sometimes simplified by a force of 10 kN over a square of 0,1 by 0,1 metres. The second of these is a horizontal load that corresponds to part of the total uniform load and acting in the direction of the bridge main axis at the deck level. The service load force is only considered for local deformations and thus not relevant for the global analysis here. The horizontal force is not particularly relevant for the static analysis since the dynamic behaviour is more critical for the horizontal behaviour. Therefore both of these loads will not be included in this analysis.

6.5.4 Snow Loads

The Eurocode specifications for determining snow loads can be summarized by Eq. 6-19, taken from *Eurocode SS-EN 1991-1-3, chapter 5.1*:

$$s = \mu_1 C_e C_t s_k \tag{6-19}$$

In the expression above, the coefficient of exposure C_e and thermal coefficient C_t are both usually set to one, this will be done for this project as well. The remaining two factors depend on roof shape and geographic location. From *Eurocode SS-EN 1991-1-3:2003(SV) NB* the characteristic snow loads s_k are given for all municipalities in Sweden. Since this project includes no site specification in the given conditions a reasonable assumption is to choose an average for the country, based on this the value for s_k to be used is **2,5 kN/m^2**.

The final factor, form factor μ_1, depends on the roof shape which varies between the concepts. In order to save some time all of the bridges are assumed to have flat roofs of an angle less than 30 degrees. Based on this the factor will have the value 0,8. This assumption will be conservative since several of the bridges have angles exceeding 30 degrees and thus less snow accumulating there. To avoid excessive conservatism in the analysis the bridge roof areas with angles greater than 50 degrees will be assumed to have no snow load.

In conclusion the uniform snow load acting on the bridge roofs is set to:

$$\boldsymbol{s = 0{,}8 \cdot 1{,}0 \cdot 1{,}0 \cdot 2{,}5 = 2{,}0\ kN/m^2} \tag{6-20}$$

6.5.5 Wind Loads

Wind loading in general is very complex when performed accurately, the Eurocode simplifications of this phenomenon makes the process much less problematic but for this project some more simplifications will save a lot of time. The general exterior wind load formula as defined in *Eurocode SS-EN 1991-1-4, chapter 5.2* is stated in Eq. 6-21.

$$w_e = q_p(z_e) \cdot c_{pe} \tag{6-21}$$

In Eq. 6-21 the first factor is the characteristic wind pressure at the given height and the second factor is a shape-coefficient based on the geometry of the loaded surface. The first simplification that will be made here is for the wind pressure, which varies over the height of the bridges. By calculating the wind pressure at a representative height for all the bridges the calculations can all use the same value. This assumed height will have to be an intermediate between ground level and the bridge peaks. Since the height of the bridges has not yet been defined an assumption is needed.

Based on the concept figures it is likely that the bridges will be at least twice the height at the openings over a sizeable portion of the span. If the openings are assumed to be three meters high this means six meters in height over parts of the bridge. Some concepts might even exceed this height but since the assumption should not be excessively conservative the height of 6 meters will be used. *Eurocode SS-EN 1991-1-4, chapters 4.8 + 4.10* give the characteristic wind pressure as in Eq. 6-22.

$$q_p(z_e) = c_e(z) \cdot \frac{1}{2} \cdot \rho \cdot v_b{}^2 \tag{6-22}$$

The first step in calculating the wind pressure when the height is known is by defining the terrain type, needed for the exposure factor $c_e(z)$. A reasonable conservative assumption here is class 2, defined as low vegetation with sparse objects in the surroundings. The value of the exposure factor for the combination of 6 meters in height and terrain type 2 as defined by *Eurocode SS-EN 1991-1-4, figure 4.2* is 2,0. The other two components in the formula are the air density and wind speed, the Eurocode recommended value of air density is 1,25 kg/m^3. The wind speed however depends on geographic location, based on the *Eurocode Swedish National Annex* an average for Sweden is 25 m/s. All of this put in to Equation 6-22 yields:

$$\boldsymbol{q_p(z_e) = 0,78\ kN/m^2}$$

The next step of the wind load calculations is to look at the shape factor. Eurocode defines a large set of shape factors based on the surface type and location on the surface. If done thoroughly the bridges with curvature to the roof and sides would not be able to use the predefined shape factors, therefore the assumption of only vertical and horizontal surfaces generates an extensive simplification. By conservatively choosing what zones of the bridges are included in vertical and horizontal surfaces the applied wind loads can be set to roughly simulate the actual effects of the wind.

Since some of the bridges might have walls, the first shape factors to look at are for vertical walls. For walls where the surface hit by the wind is wider than its height *Eurocode SS-EN 1991-1-4, chapter 7.2.2* defines the wind loads with one uniform factor for each side of the structure, depending on its height to depth ratio. If the bridges are assumed to be roughly the same width and height for a section these factors become 0,8 for the windward side and -0,7 for the leeward side, the negative factor standing for suction.

For the roofs the simplified approach that will be used is to assume screen roof action, where the wind can move both over and under the roof. This will be the case for most of the bridges but for the TUBE concept it is likely to be a deviation from the actual case, this concept however has the most aerodynamic shape for wind loads from the side and the horizontal forces from the wind will be of greater significance. Therefore the bridges are assumed as a simplification to have flat screen roofs.

The shape factors for screen roofs as defined in *Eurocode SS-EN 1991-1-4, chapter 7.3* are dependent on the level of obstruction under the roof. In a crowded case the wind has less space to pass through and the speed will thus increase creating more force on the roof. Since a dense crowding is very unlike to happen, the lower values for no obstructions under the bridge are more appropriate. These factors are shown in Table 6.14.

Table 6.14 Shape factors for flat screen roofs

	Main roof zone	*Edge zones windward/lee*	*Edge zones entrances*
Factors for no obstructions	**-0,6**	**-1,4**	**-1,3**
Factors for obstructed path	**-1,5**	**-2,2**	**-1,8**
Factors for wind down force	**0,5**	**1,1**	**1,8**

The size of the edge zones are one tenth of the respective depth, ergo if a rectangular bridge roof is investigated, the edge depths are 10% of the bridge length for the entrance edges on each side and vice versa. Subsequently, the total area of the main zone stands for 64% of the whole area. This known area ratio means that an equivalent shape factor for the whole roof can be calculated. Inserting the values from the Table 6.14 and their corresponding 18% yields, the factors -0,87 for no obstructions and 0,84 for the wind down force. Looking at these factors conservatively and weighing in the risk of obstruction the assumed shape factors used for this calculation are -1 and 1 respectively. This put into Equation 6-21 along with the wall shape factors yields the loads shown in Table 6.15:

Table 6.15 Calculated snow loads

Wind load on wall (windward side)	**0,62 kN/m^2**
Wind load on wall (leeward side)	**0,55 kN/m^2**
Wind load on roof (uplift)	**0,78 kN/m^2**
Wind load on roof (downwind)	**0,78 kN/m^2**

6.5.6 Pre-stressing Loads

The final loads to include are for the pre-stressed steel members. In Karamba there is a pre-stressing module which essentially stretches an element to a certain strain before inserting it to the model. This way the effects of post-tensioning for steel cables can be simulated by knowing the desired strain in the members. To save time the complexity of post-tensioning steel cables and its Eurocode definitions will not be treated further here, only the effects it give on a structure will be considered for the process.

6.6 Assumptions Used In Karamba

For the promising concepts the analysis is kept at a simplified level. The first way in which this is implemented is by only considering the vertical downward forces for the bridges. This means not looking at the wind horizontal loads or the wind uplift. All the downward vertical loads however will be incorporated, the wind down force and additional self-weight included.

The second simplification is for the Karamba modules of trusses. The simple hinge module explained earlier will be used at this stage, this means either completely hinged elements or completely rigid elements were used. For the second analysis of the two most promising concepts partly fixed/hinged ends were included.

The third simplification is for the load combinations, again with a simplified version here and a thorough one for the final two concepts. The most likely critical load case is for the imposed loads being the main variable load. This is implemented by assuming the psi-factors for both the snow and wind loads added. Since the imposed load is far greater than the other loads and has a low psi-factor it is very likely that this load case is the decisive one for design.

For the dynamic behaviour there will be no added simplifications, the simple nature of the Karamba dynamic analysis will be considered sufficient at this stage. By extension it can be argued that the previous simplification of either completely hinged or rigid connections is a dynamic assumption, this due to the close relation between the dynamic behaviour and connection types. This stage however, will not compensate for this connection assumption for the dynamic analysis. The limits to check are natural frequencies less than 2,5 Hz for horizontal modes and less than 5 Hz for vertical modes, according to *Eurocode SS-EN 1990- appendix A1* and *A2.4.3.2*.

The potential input dimensions for the bridge elements in this stage will be a set list of even numbers. The idea here is not to optimise for production at once but rather to look at the required dimensions and their effect on the bridge response.

7 Promising Concepts

In order to get to the final two concepts a comprehensive investigation of the four promising concepts was needed. The concepts were turned into actual bridge systems instead of design ideas, these systems were analysed, altered and reanalysed in an iterative process until a set of plausible bridge designs were generated. The analysis process, of Karamba and physical models, as described in Chapter 6 served as the main source of data for the design iteration process. This was however complimented by sketching, discussions and precedent inspirations for a comprehensive process.

7.1 The I-BEAM Analysis

The design process of this concept, as with the others, included many changes to the model geometry. This optimization process along with an analysis of the final model is summarized in this chapter. For the extent of this section, the I-BEAM Concept will be referred to as IBC.

7.1.1 Final Form

Starting at the end of the process in order to get a good grip on the actual bridge geometry discussed here the following figures, Figure 7.1 and 7.2, show two rendered perspective views of the IBC final form.

Figure 7.1 Rendered perspective of the IBC final form

Figure 7.2 Rendered perspective of the IBC final form

As the figures show the bridge consists of a centre truss that aims to carry the majority of the vertical loads. The bridge deck is supported on the bottom of this truss and serves as a horizontal stabilizer for this part of the truss. The top of the truss is stabilized by the roof which is supported partly on the truss and partly on the ground by the entrances. This means that the roof helps transfer loads from the truss via the ground supports and it gets stabilized by itself using the triangular ends on both sides.

In Karamba the bridge geometry is represented by points, lines and surfaces. A display of the bridge in this fashion from the Grasshopper model can be seen in Figure 7.3.

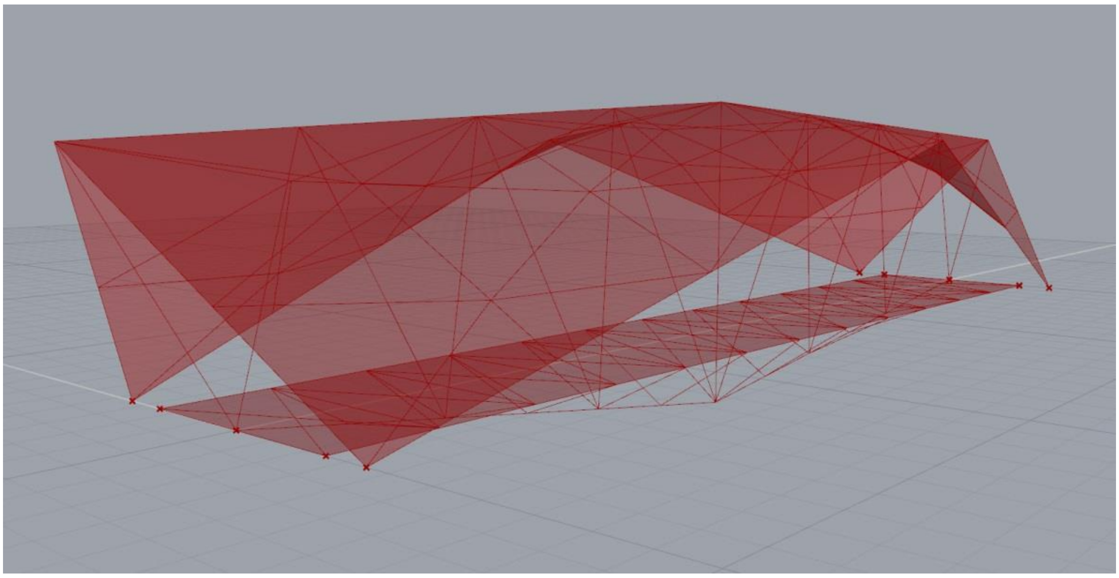


Figure 7.3 Perspective view of IBC final form

For a more extensive view of the bridge geometry, Figure 7.4 shows a side view of the IBC. From this view the change in height along the bridge is clearly visible, as well as the downward extension in the middle.

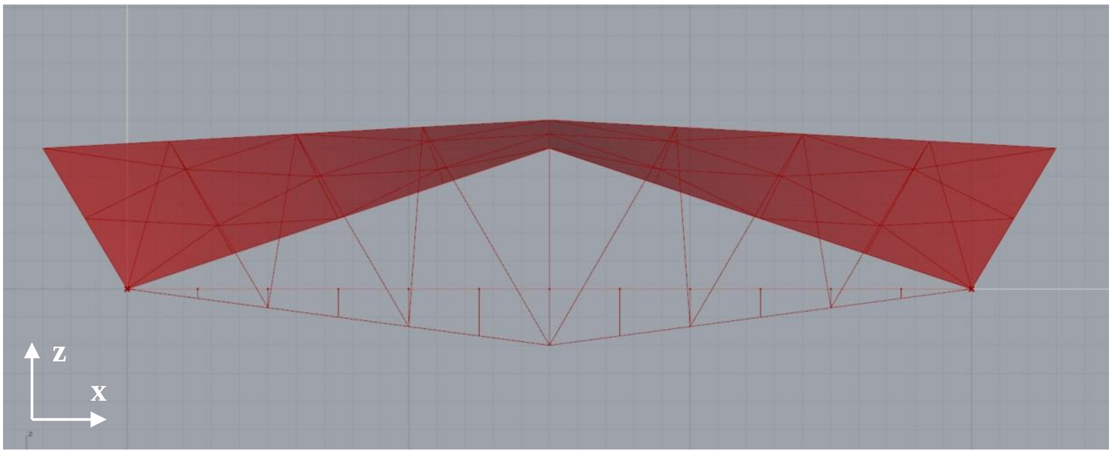


Figure 7.4 Side view of IBC final form

In Figure 7.5 the entrance and its triangular portal can be seen, in addition the diagonal supports for the deck are visible.

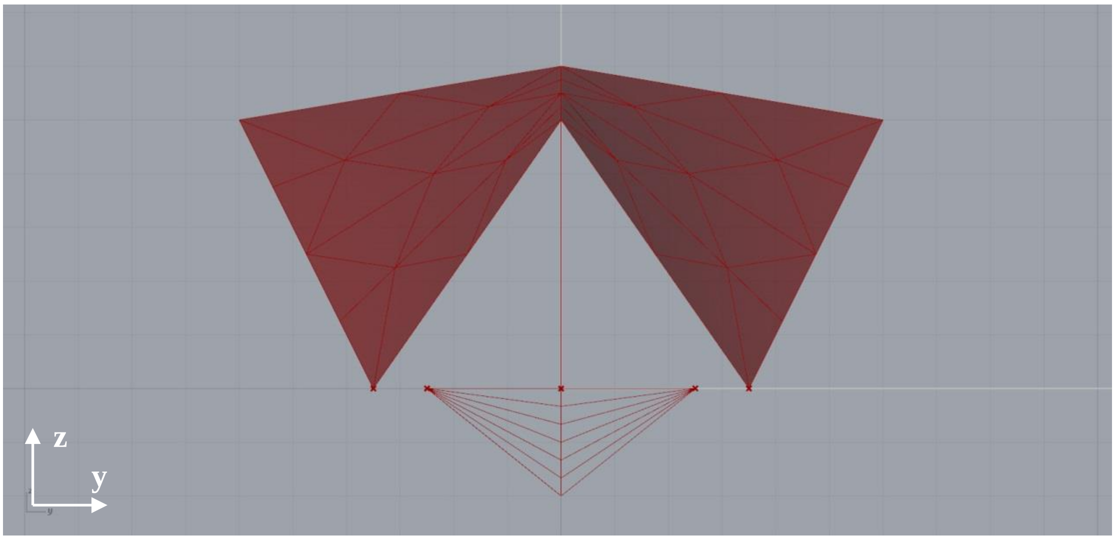


Figure 7.5 Entrance view of IBC final form

Finally looking at the bridge from the top, as shown in Figure 7.6, the footprint of the bridge can be seen. Here the roof overhangs for both the entrances and the middle of the span are shown. This figure also reveals the beam structure of the roof, with a regular triangle pattern across the span.

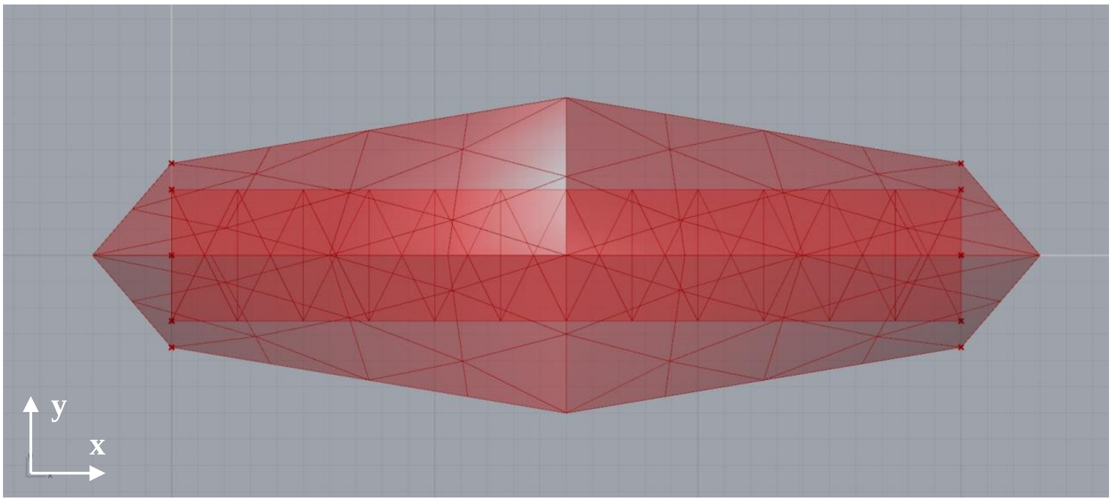


Figure 7.6 Top view of IBC final form

In all of the figures the support points for the model are also displayed. Placed in a line on each side they form a simply-supported bridge. The supports are all moment free points and they are all locked in z-translation. Apart from this, the two centre ones are locked in the y-direction and one of them also for the x-direction, thus avoiding a rigid body movement. The joints in the bridge are all assumed as rigid except for the bottom steel rods which are moment free.

7.1.2 Design Process

The process of getting from the IBC concept idea as described in Chapter 5 to the final bridge design as shown here before contained many alterations of varying magnitude. In an attempt to explain the process some of the components are described here.

The first and most significant modification came early when the overall design of the concept was merged with the CROSS concept from before. This concept has a strong axiality with a roof ridge along the centre, open sides and triangular edge sections, features that go very well with the content of IBC. The ridge in the roof forms a natural meeting with the centre truss, the open sides maintain this key feature of IBC and the edge sections of triangular shape are rotation stable which helps counter one of the main weaknesses of the concept.

At this point the IBC was modelled up in Grasshopper in a way that generated a bridge that was a pure combination of the two merged concepts. The geometry at this point had a flat base deck that would rely heavily on cantilever action to carry the deck. Therefore a lowering of the centre was added to allow for a more efficient deck support, this also worked well with the already edgy design theme of the bridge. With the lowered centre an addition of an elevation at the roof centre was also added. Through this the centre truss became at its highest where the loads are the largest, in addition, this worked well with the design just as with the lowering mentioned earlier.

Before various dimensions could be tested the truss setup for the centre truss and the roof had to be created. The main truss had some key locations to relate to, it seemed natural to create the first truss members as a connection between the entrance triangle tops and the centre of the deck ends. Another natural member was the middle one which connects the points in the top and bottom. After this an intermediate zigzag was generated that completed the centre truss design.

The roof truss had several designs versions before the final one was found. The idea for it was to include a crossed arch in the roof that efficiently helped with the loads while simultaneously having a repetitive regular system, that help create a beautiful aesthetic for the bridge. This all came together in a roof truss that was based on dividing each roof quarter in four sections longitudinally and then zigzag connecting these sections using four sections in the transversal direction as well. In this way a diagonal through each quarter was formed, and as the whole roof was completed, these diagonals formed a cross.

At this point Karamba enters the picture, by changing around with dimensions and geometry constants better and better combinations between aesthetics and structural behaviour were found. From an architectural viewpoint uniformity in the member dimensions is favourable, this also works well with reducing the work of trying various dimensions since many members will have identical sizes. In Karamba, this means creating sets of elements that share cross-sectional dimensions, see Figure 7.7.

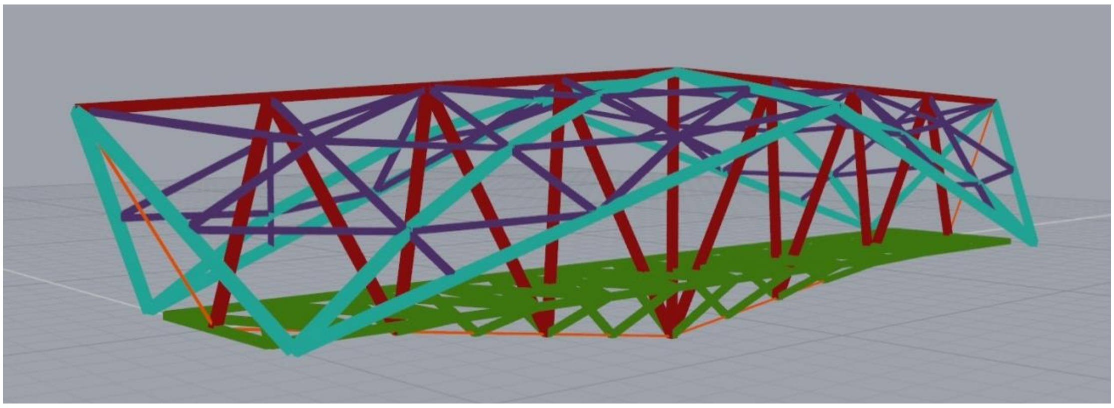


Figure 7.7 Perspective view of the IBC element groups

The final dimensions used in the IBC bridge design are shown in Table 7.1

Table 7.1 Dimensions of the IBC element groups, as shown in Figure 7.7

Set of elements	*Dimensions*
MAIN TRUSS	$\mathbf{250}\boldsymbol{x}\mathbf{250}\ \boldsymbol{mm}$
DECK MEMBERS	$\mathbf{200}\boldsymbol{x}\mathbf{200}\ \boldsymbol{mm}$
ROOF FRAME	$\mathbf{250}\boldsymbol{x}\mathbf{250}\ \boldsymbol{mm}$
ROOF TRUSS	$\mathbf{160}\boldsymbol{x}\mathbf{160}\ \boldsymbol{mm}$
STEEL RODS	$\boldsymbol{\emptyset}\mathbf{75}\ \boldsymbol{mm}$

The element division used above is of course a very simplified one and it is likely that some over dimensioned members will occur due to the desired uniformity. In a more thorough design only the architecturally important members would be allowed serious over dimensions for the appearance whereas non-visible members would be more structurally optimised.

Another method of optimization is the use of steel rods for the exclusively tensioned members. Tensile anchorage in timber can be problematic and since some members exclusively experience tension they make perfect candidates for steel rods.

In order to assess which members have this feature the compression/tension module described earlier was used. A set of figures for different load cases, varying from only self-weight to maximum load, were plotted and merged to show which members alternate in forces. Figures 7.8 and 7.9 show this merged image for the IBC.

In the Figures 7.8 and 7.9 **BLUE** means compression for all load cases, **RED** means always tension and **BLACK** is varying between compression and tension for various load cases.

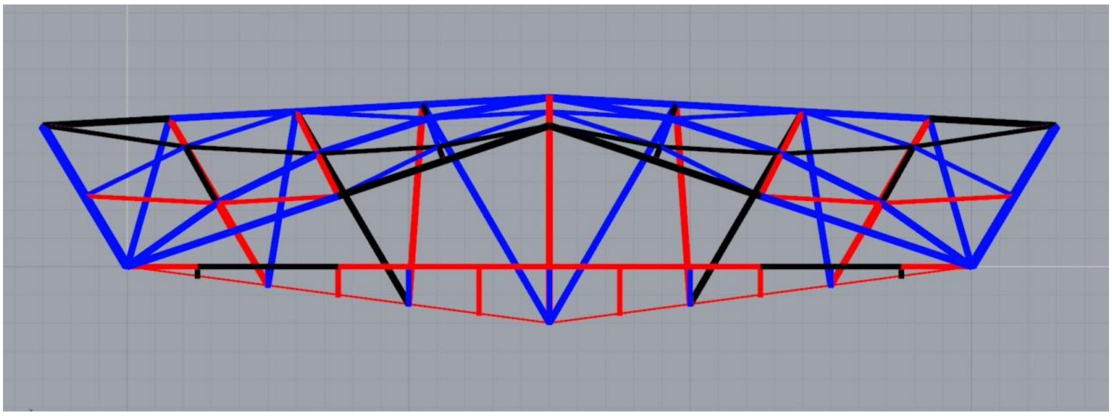


Figure 7.8 Side view of IBC alternating members

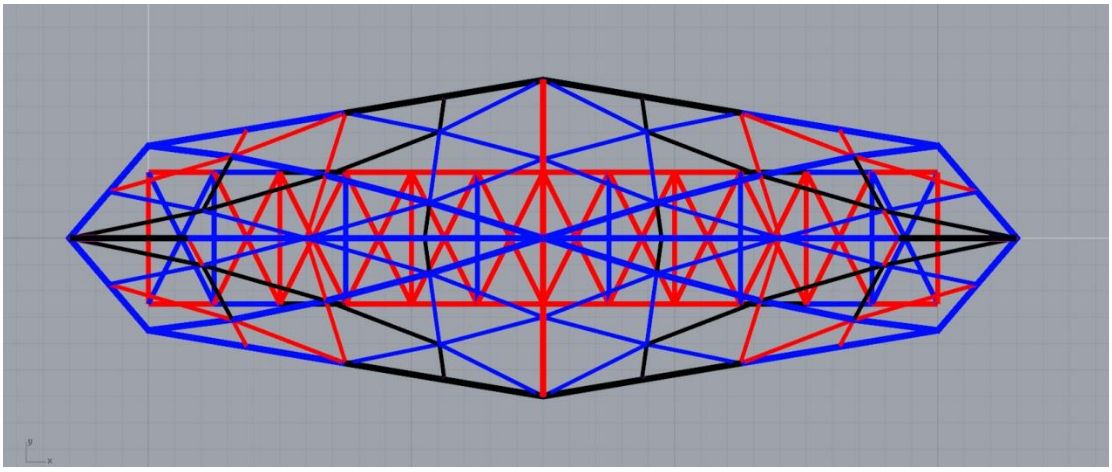


Figure 7.9 Top view of IBC alternating members

As the figures above show some members vary between tension and compression depending on the possible loads, here represented as black members. The figures also confirm that the steel rods in the bottom of the truss truly are always in tension. Another observation to be made here is that the crossed arch also works as intended with compression all the way down to the supports for all of the load cases.

Apart from all the mentioned process steps here many smaller modifications, some successful and some not, were tested. For instance a physical model was built to investigate the space and its light for one of the final bridge designs, this did not lead to many modifications but the analysis made can be seen on the following pages.

7.1.3 Architecture

The IBC from an architectural standpoint has a number of key features that make up the experience of visiting the bridge. The first of these, which saturated the IBC in the concept stage, is the open sides. In difference to all other concepts investigated only this one has completely open sides. In a more detailed design some type of railing would of course be required, but the rest of the sides have no visual obstruction at all. Architecturally this means that the bridge space can incorporate the surroundings in a very pure way, allowing the visitors to take in potential nature or urban environments as a strong part of the experience. For a fair analysis of the design the downside of this feature also require attention, fear of heights among visitors or an unfavourable surrounding will most likely take away pleasure from a visit.

The overall design theme of the IBC is a rather edgy appearance, mostly generated by triangular shapes and few smooth features. Since this theme saturates the bridge several components act to counter the otherwise potentially chaotic design. The first of these is the continuous deck which also passes over the centre, in this way visitors walk on a simple uniform plane where the truss beams in the middle just disappear down under the deck without disturbing its geometry.

Another feature that also aims to simplify the design is the regular roof truss. The numerous components in this truss can easily contribute to the bridge complexity but by having a very repetitive and symmetric system this addition of intricacy is minimized.

In order to get a better sense of the bridge design and space a plywood model was made of the bridge. Figure 7.10 shows an inside perspective simulating the experience of being inside the bridge.

Figure 7.10 Inside perspective photo of the IBC

The inside perspective photo shows that the bridge will have a clear variation in the daylight entering the space, along the span. Both entrances give a more dimmed light at first but as one approaches the centre the bridge is hit by full unobstructed daylight, giving a dynamic experience in the bridge.

The inside photo also shows that with a light inside of the roof cover it is possible to emphasise the repetitive roof truss and make it into a decoration for the bridge interior. The photo also shows that the curvature of the roof definitely comes across from the inside.

The same model was also photographed at a distance to give a sense of the exterior of the bridge, this can be seen in Figure 7.11, note that this does not represent the ultimate design of the IBC but rather one of the final versions, hence the timber base of the truss.

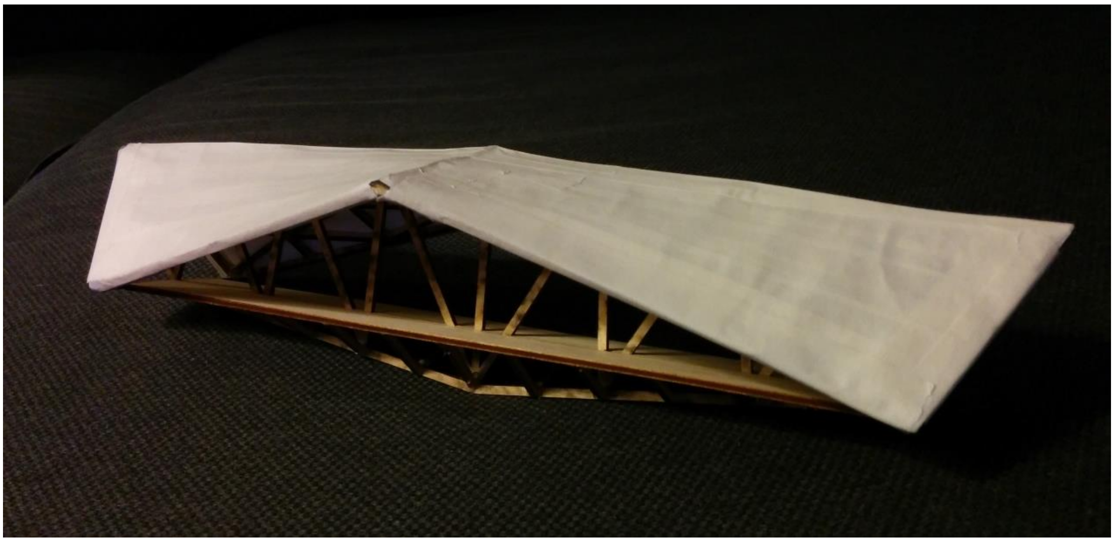


Figure 7.11 Outside perspective photo of the IBC

In the picture above the edgy shape of the bridge is clear, as well as the dynamic light mentioned above. The picture also shows that the IBC has a quite large roof which from an exterior perspective makes up a large portion of the visible bridge. While this can provide a sense of security upon entering it also runs the risk of a heavy appearance.

In summation the IBC can be said to definitively have an uncommon design for a covered timber bridge, and since attention was one of the goals for the designs in this project this definitively is favourable. The open nature of the bridge works well with a beautiful surrounding and the roof of the bridge has potential of relating to the classic duo-pitch roofs which are very common in Sweden.

7.1.4 Physical Behaviour

Looking at the behaviour of the final IBC design can be done by first studying the tension/compression behaviour of the model. Figures 7.12, 7.13 and 7.14 show this display from three different angles, **BLUE is compression** and **RED is tension**:

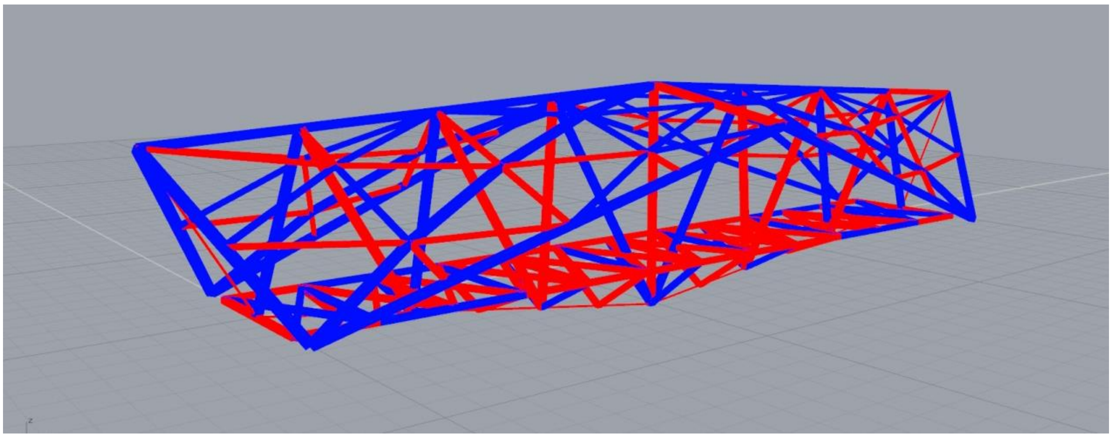


Figure 7.12 Perspective view of the IBC final form compression/tension behaviour

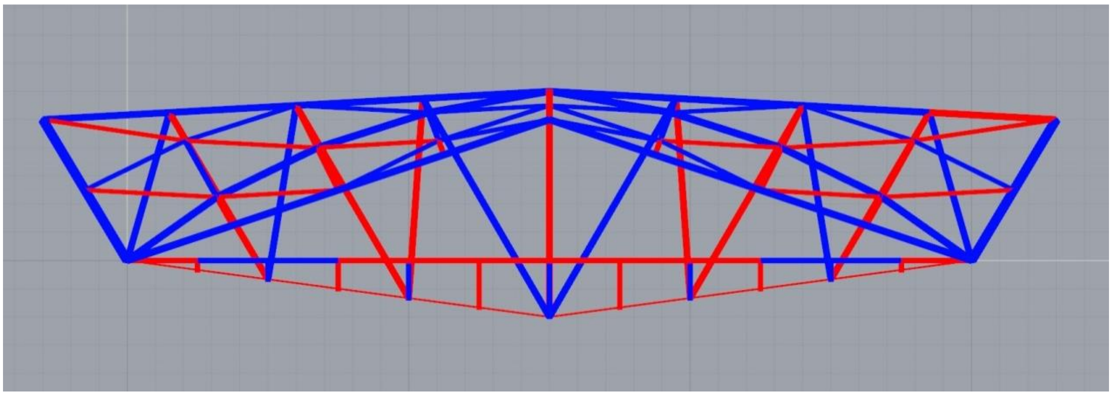


Figure 7.13 Side view of the IBC final form compression/tension behaviour

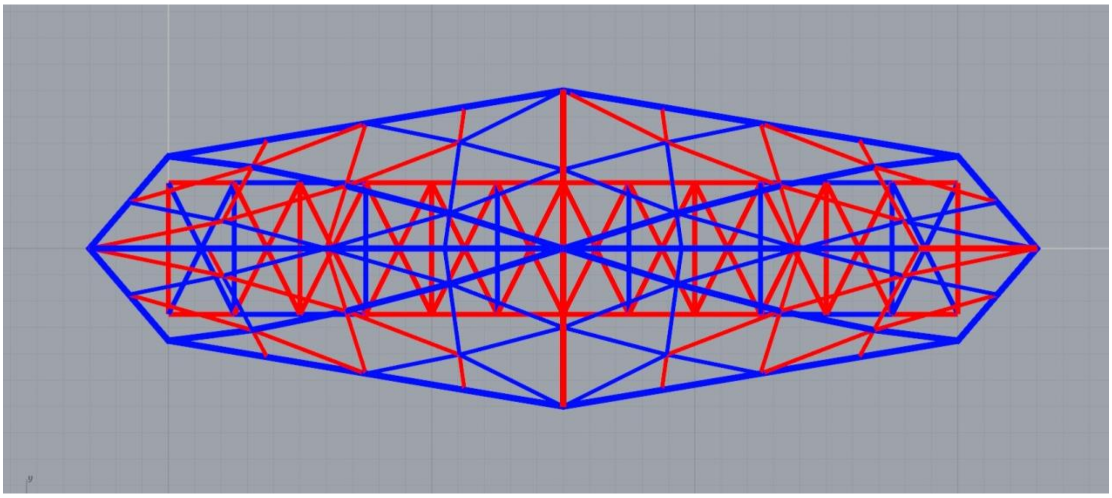


Figure 7.14 Top view of the IBC final form compression/tension behaviour

To accompany the compression/tension figures on the previous page in the final behaviour analysis, Figures 7.15, 7.16 and 7.17 show the element utilization in the same model. An approximate legend for the utilization colour map in the figures is:

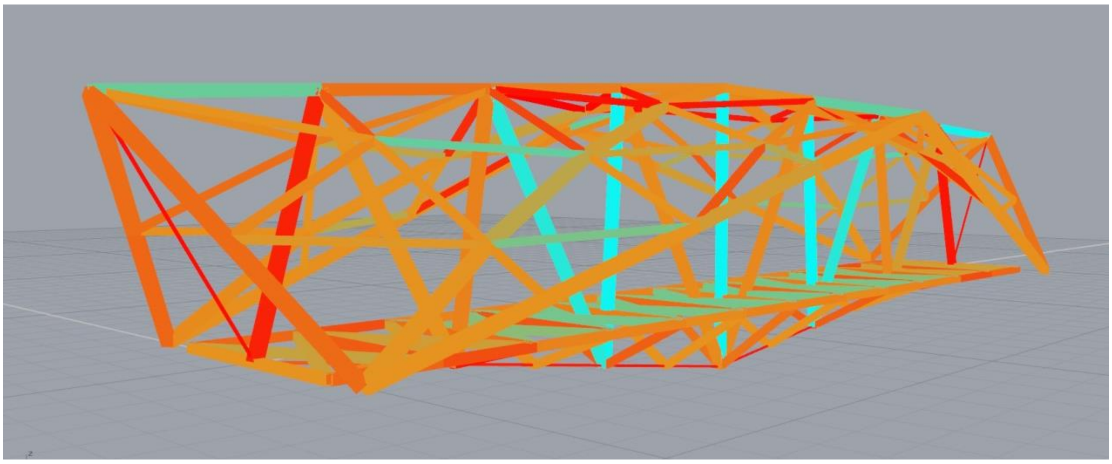


Figure 7.15 Perspective view of the IBC final form element utilization

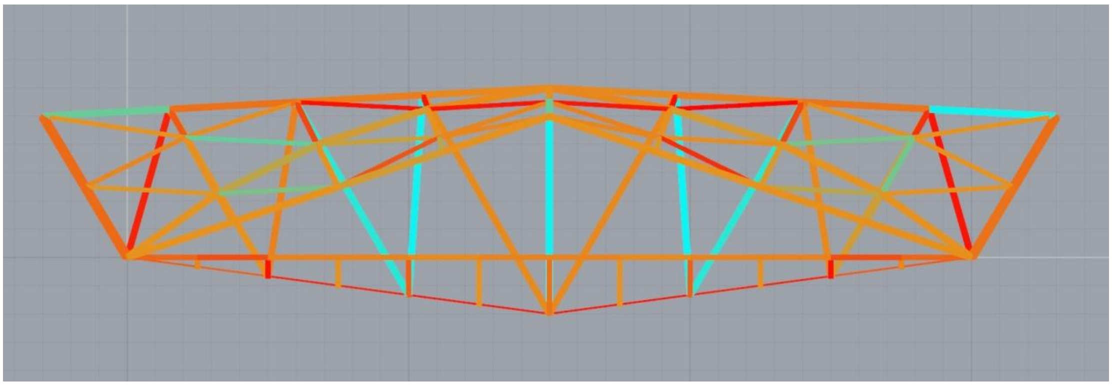


Figure 7.16 Side view of the IBC final form element utilization

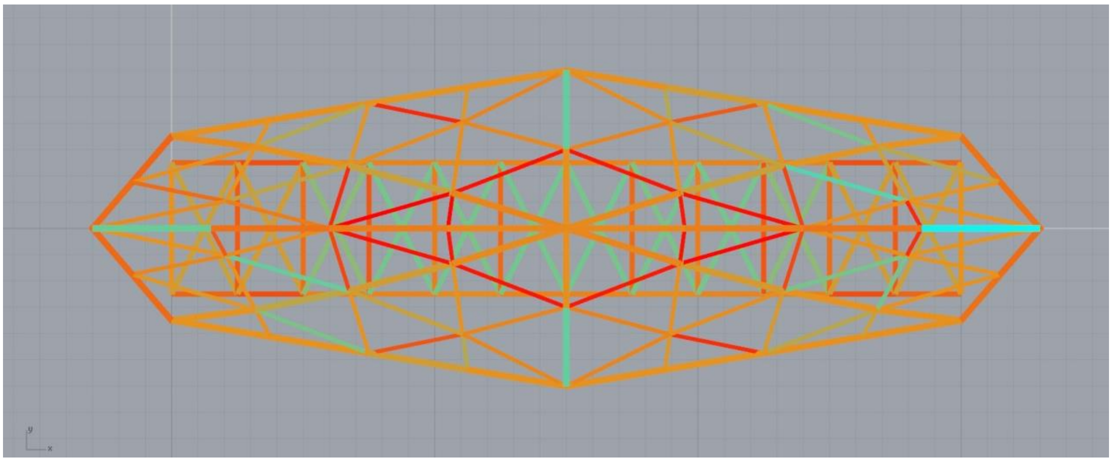


Figure 7.17 Top view of the IBC final form element utilization

Looking at the roof to begin with it is clear from the first set of figures that the main response of the roof is compression, as expected with it being part of the truss top. Another key observation for the figures is the two crossing arches that are part of the roof geometry, starting in each roof ground support going over to the diagonally opposite one on the other side. These arches are, as the figures show, clearly in compression as well. This means that there are two competing load bearing systems in the bridge carrying the vertical loads, the truss beam in bending and the compressed arch cross. In the current state they are both active which on the positive side can provide a good redundancy for the bridge, but also makes the bridge behaviour harder to read.

The bottom of the truss, consisting of the steel rods, is as the figures show both completely in tension and also efficiently utilized when fully loaded. The truss centre has several low utilization members, but from an architectural point of view a common dimension for all the vertical truss members is more elegant. Therefore the highly utilized end components determined the size for all these members.

Just as with the vertical centre truss members the roof will architecturally benefit from a uniform size in its truss. The highly compressed diamond shape clearly visible in Figure 7.17 thus governed the size of all the small roof truss members. Since the forces will follow stiffness the structurally efficient crossed arch was excluded from the otherwise uniform dimensions and given a large size to help with the bridge loads.

After looking at the ultimate state the serviceability limit state was checked, starting with the deformations which are displayed in Figure 7.18. From the figure it is easy to see that the most deformed zones are the roof ridges perpendicular to the bridge, the maximum vertical deformation here is 114 mm. These zones have the least efficient load paths and work mainly in cantilevering action. This however is not a critical problem since the deformations in this area only visually affect the visitors.

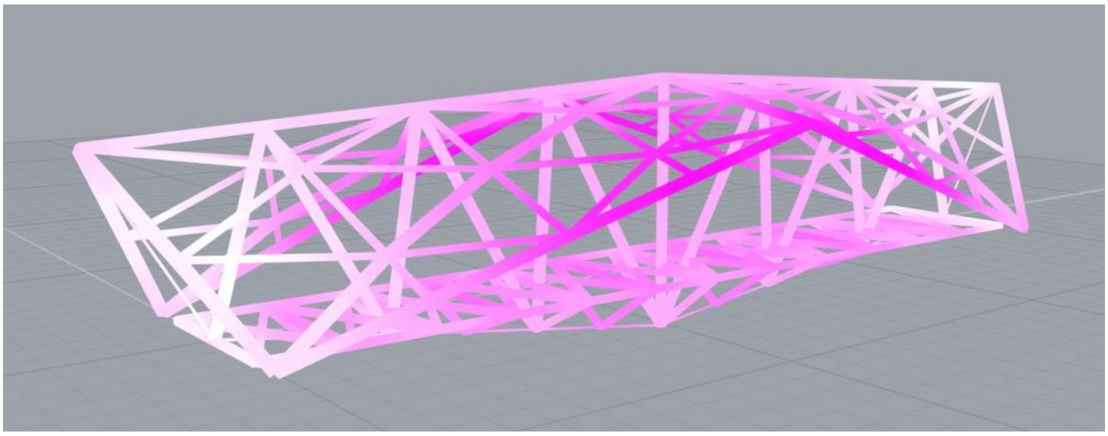


Figure 7.18 Perspective view of the IBC final form deformations

The more important deformations are for the deck and here the maximum characteristic deflection at mid span is 47 mm. This can be compared to the limit from *Eurocode SS-EN 1995-2 table 7.1* which says no more than between 2 and 2,5‰ of the whole span. Translated into an actual deflection limit for the 30 meter span in this task means between 60 and 75 mm, therefore the deflection is okay.

The final serviceability analysis regards the dynamic response. The final bridge model has only one critical Eigen mode under the Eurocode vertical modes limit. Figure 7.19 shows the shape of this mode for the IBC.

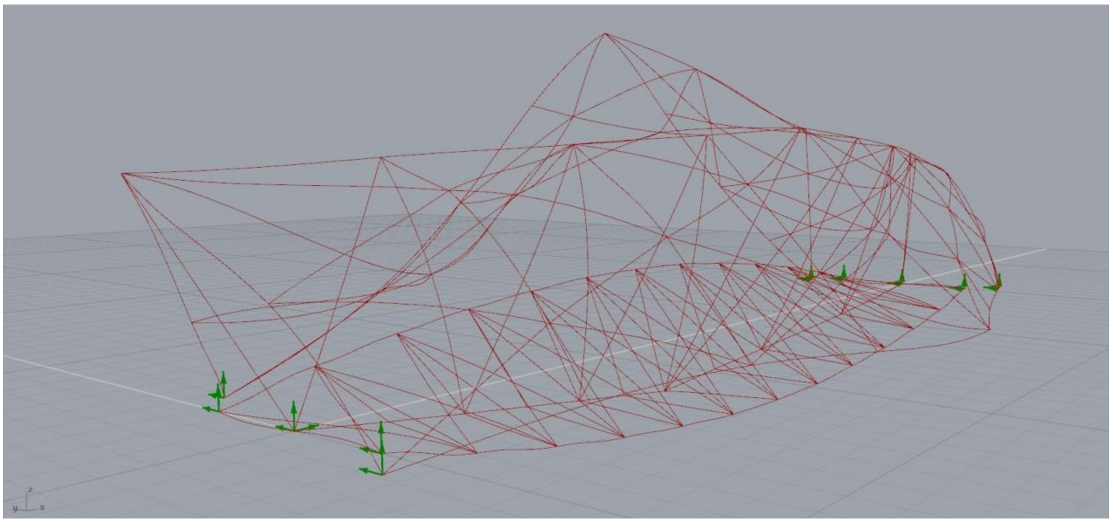


Figure 7.19 First eigenmode of the IBC

The figure shows that the shape of the mode is a twisting at mid span, this mode is at 3,6 Hz which is under the limit of 5 Hz. The case for which this mode is relevant is for a very asymmetric loading of the bridge with both heavy imposed loads and the wind pushing down the roof , all on one side. For an analysis of this depth no further actions will be taken towards thoroughly investigating this dynamic response, but this data shows that further control of the dynamic response is needed.

7.1.5 Concept Comments

When summarizing up this concept it is clear that some further work would be beneficial for this design. The dynamic response is not yet within the Eurocode specified limits, just as was suspected early on when the torsional weakness of the IBC was pointed out. Furthermore some elements have excessive dimensions due to the simplified grouping used in the design process. The structure is relatively well optimised to efficiently carrying the loads however some areas such as the roof ridge that suffered the largest deflections could benefit from further work.

Except for the downsides mentioned above the bridge has a very interesting design that fulfils the desired architectural and structural goals.

7.2 The SADDLE Analysis

The next concept analysed is the SADDLE concept, for the rest of this chapter referred to as SC. Some of the topics mentioned for the previous concept will here be treated less thorough.

7.2.1 Final Form

Just as with the previous concept an initial description of the final form of the SC is helpful to understand the rest of the section. The two pictures shown in Figures 7.20 and 7.21 are renderings of the final design for the SC:

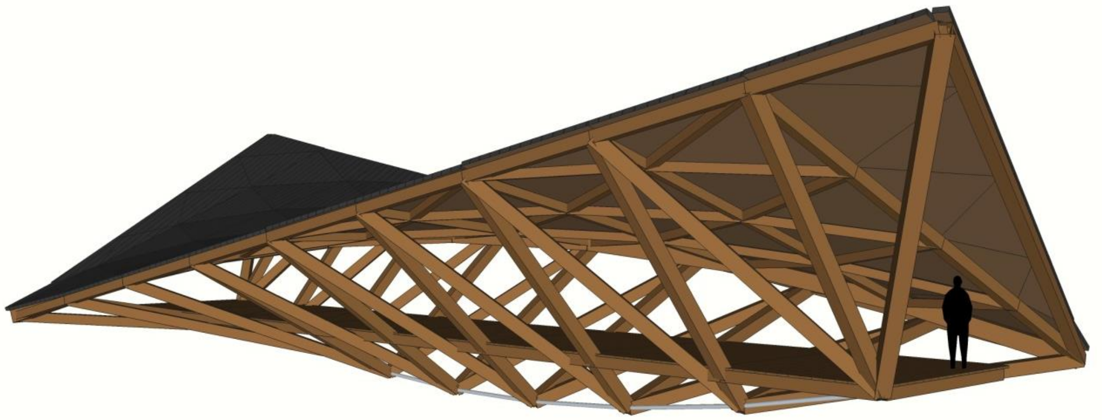


Figure 7.20 Rendered perspective of the SC final form

Figure 7.21 Rendered perspective of the SC final form

The final design is also described via the grasshopper model, Figure 7.22 page shows the bridge design and supports in a perspective view.

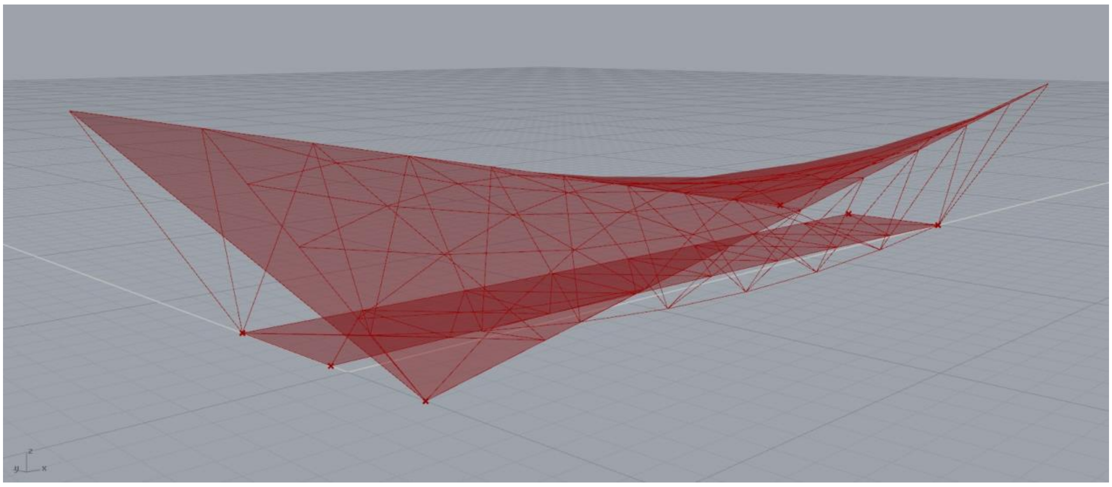


Figure 7.22 Perspective view of SC final form

As the figures show the SC is based on a sizeable roof structure that rests on two twisting trusses and two ground supports. The roof also includes a strong compressed arch inside the truss. This component, as the figures show, blends in well with the rest of the roof truss from several angles, but serves an important role carrying loads in the structure. The truss upon which the roof rests starts off by the roof supports with a zero height, in this area the arch takes most of the loads, but further along the bridge the height of the truss increases. The truss design is based on a set of triangular sections that rest on a tensioned steel base. They have diagonal truss members connecting them, which can be seen in Figure 7.23:

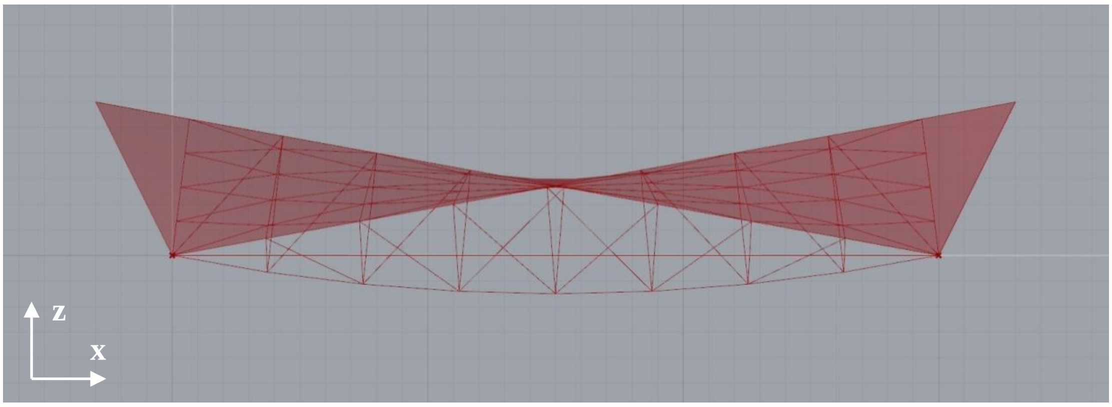


Figure 7.23 Side view of SC final form

The bridge supports are placed in a line on each side, displayed as points in the figure above. This simply supported setup is generated by fully locking the translations of the roof arch supports except the longitudinal translation in one end. The other four supports, connected to the deck, are only locked in the vertical displacement.

Just as with the IBC this concept also has mainly fixed joints in the bridge, the only exception being the steel rods in the bottom that have no locked rotations in the connections. This bottom steel arch can be seen more clearly in the view across the bridge in Figure 7.24.

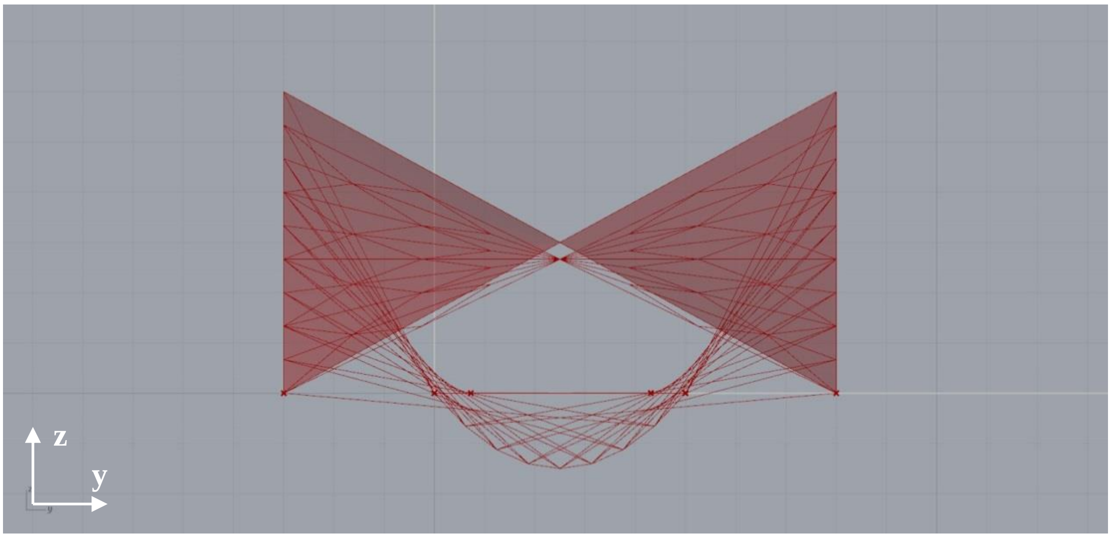


Figure 7.24 Entrance view of SC final form

The last figure, Figure 7.25, shows the top view of the bridge. Here the overhangs for all of the bridge sides can be seen. Another observation to be made from this figure is that the deck is not completely parallel to the roof. This slight rotation makes visitors enter the bridge in a bit more welcoming way as the roof is opened up more towards them.

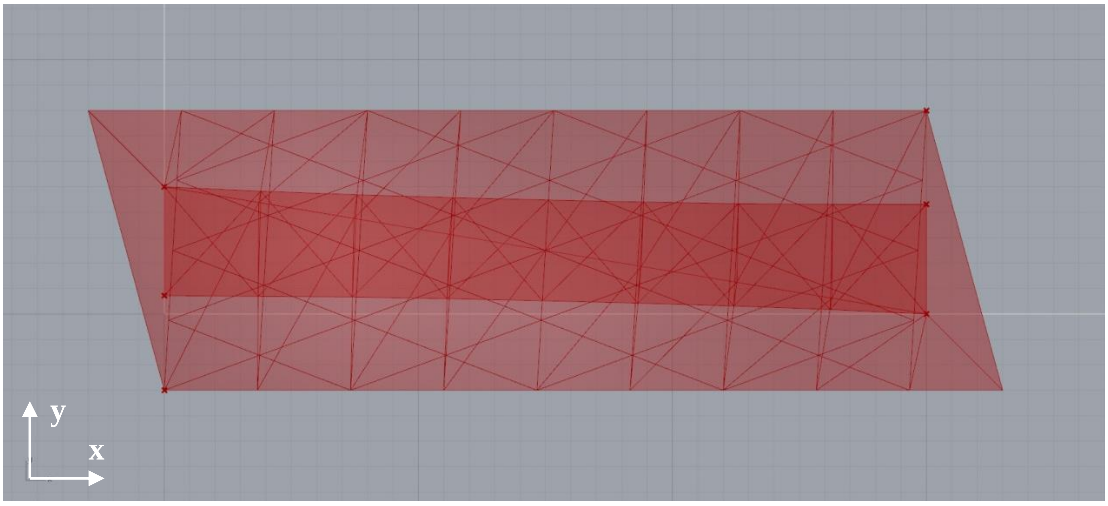


Figure 7.25 Top view of SC final form

7.2.2 Design Process

The early stages of the SC design process led to the triangular entrances. This had the major benefit of torsional stiffness at the ends, it provided another ground support for the roof and gave the whole bridge a more slim look. Another early stage decision was the compressed arch inside the curved roof, which after the above mentioned modification also had a ground support on each side.

At this point the Grasshopper modelling started, different overhangs on all sides along with various roof and side trusses were tested. These versions were put into Karamba and returned insufficient values of strength and stiffness for the desired dimensions.

The design of the roof truss, which has the same regular pattern as the IBC, gave a significant improvement to the appearance of the bridge inside. In terms of structural performance the key modification that led to a functional bridge was the addition of the tensioned base steel.

The functionality of the base had several positive factors, the side truss and roof now formed triangular sections instead of quadrilaterals, giving a better torsional stiffness. These triangular sections also form a global shell for the bridge, where the deck rests inside without having to carry large loads inefficiently as with the previous designs. The tensioned base also forms the opposite diagonal to the compressed arch, this means that the combined arc cross aids the truss in bending giving a more structurally efficient bridge. The final positive effect of the change was that the deck now was slightly rotated to fit into the new truss shapes. This rotation meant the deck faces away a little bit from the roof supporting side and moves closer to the open part, a clear improvement architecturally.

As the global shape found its final design the dimensions were optimised to work with the geometry. Figure 7.26, shows the element set division used for this bridge, the corresponding dimensions are displayed in Table 7.2:

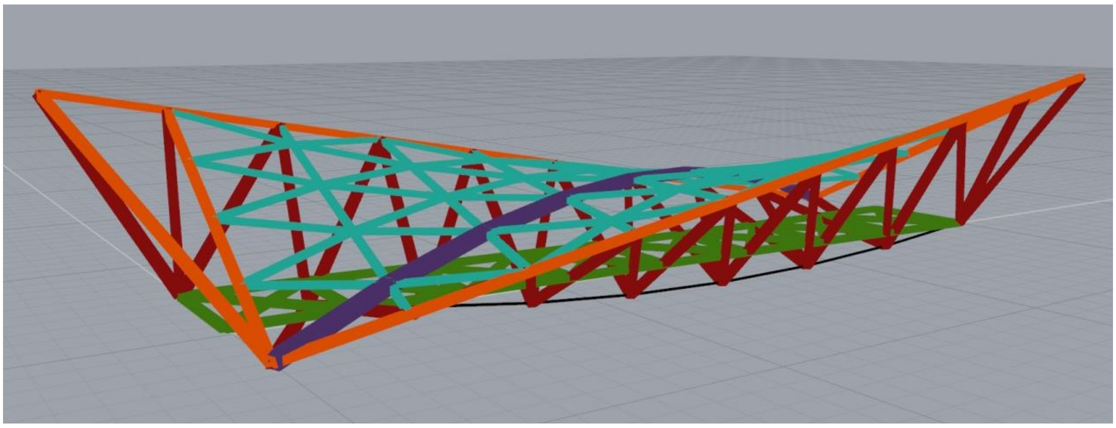


Figure 7.26 Perspective view of the SC element groups

Table 7.2 Dimensions of the SC element groups, as shown in Figure 7.26

Set of elements	*Dimensions*
MAIN TRUSSES	$\mathbf{300x300\ mm}$
DECK MEMBERS	$\mathbf{250x250\ mm}$
ROOF TRUSS	$\mathbf{200x200\ mm}$
ROOF ARCH	$\mathbf{400x400\ mm}$
ROOF FRAME	$\mathbf{250x250\ mm}$
STEEL RODS	$\mathbf{\emptyset 75\ mm}$

Some of the design methods described for the IBC, such as the tension/compression check for different load cases, were also used here. This is how the steel rods in the base were determined not to experience compression. More in depth description of this part of the process will be a repetition of the last concept and will thus not be further included here.

7.2.3 Architecture

The design of the SC undoubtedly manages the desired feature of an unconventional bridge design. The design concept is based on a set of clear repetitive flat systems that come together in a curved global shape. The model picture in Figure 7.27 is from an earlier stage of the bridge design but still gives a sense of the bridge exterior.

Figure 7.27 Exterior perspective photo of the SC

While the exterior of the bridge is both unconventional an interesting, the interior has an even more intriguing space. The following model photo in Figure 7.28, also from a non-final stage design, shows the space inside the bridge. The final design of the bridge has a significant change in the truss setups but the volume is virtually unchanged from the photo below.

Figure 7.28   Interior perspective photo of the SC

As the figure shows the room inside the bridge has a twisting nature, swept in a regular timber element pattern. The final pattern has a different setup, changed into a more repetitive and simple design. This makes the final design less chaotic than what Figure 7.28 might suggest.

The models also show a hint of how daylight would act on the bridges. In general the interior of the bridge is shaded due to the large roof but towards the entrances the light gets in gradually more and more. The less lit areas in the centre will still get a lot of secondary light from the open sides, but might require interior lighting on cloudy days.

As a summation of the design the unique and exciting spaces and global shape of the bridge stand out. The bridge concept is more an architectural play with spaces and shapes rather than moulding of light. With this in mind the generated spaces definitively do intrigue and the bridge would make for an excellent example of the potential with timber bridge building.

7.2.4 Physical Behaviour

Just as with the IBC a set of analysis result figures have been made for the SC. The first of which are for the compression and tension behaviour, shown in Figures 7.29, 7.30 and 7.31, **BLUE is compression** and **RED is tension**:

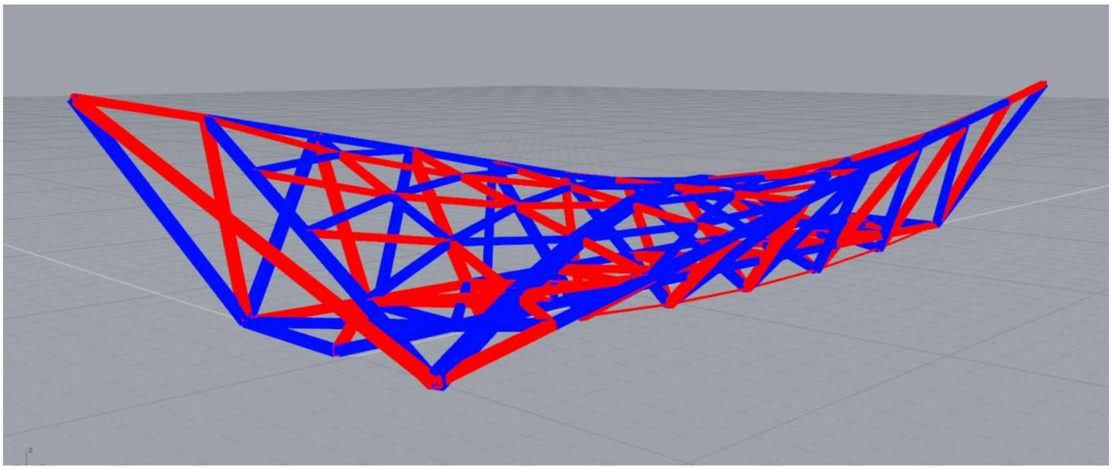


Figure 7.29 Perspective view of SC final form compression/tension behaviour

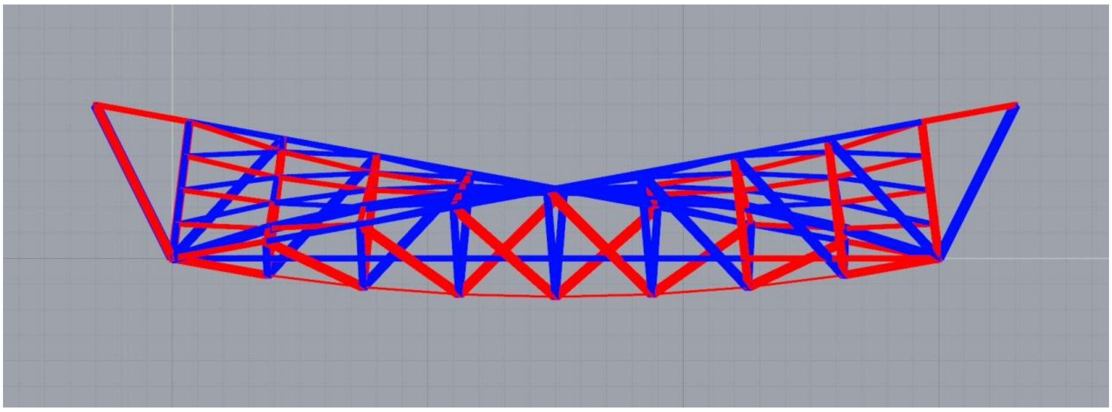


Figure 7.30 Side view of SC final form compression/tension behaviour

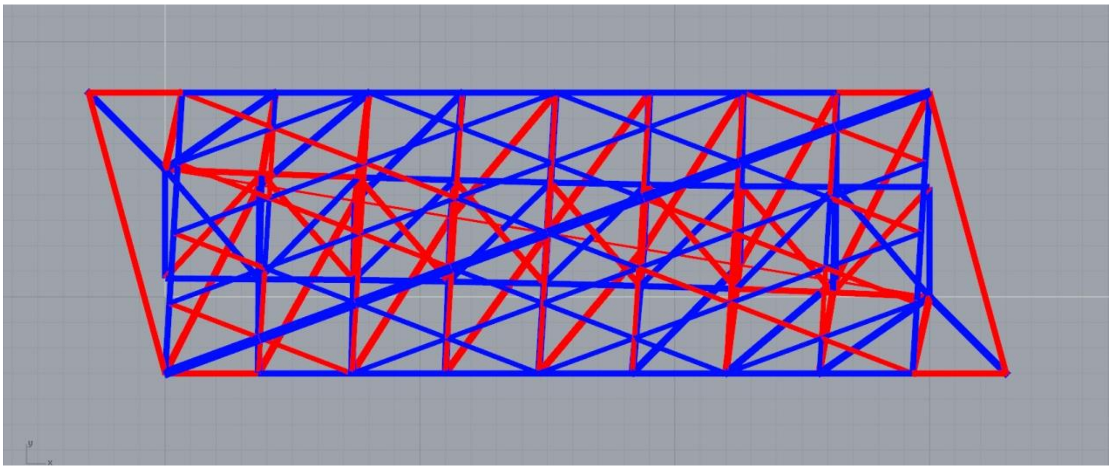


Figure 7.31 Top view of SC final form compression/tension behaviour

To accompany the compression/tension figures on the previous page, Figures 7.32, 7.33 and 7.34 show the element utilizations. An approximate legend for the utilization colour map in the figures is:

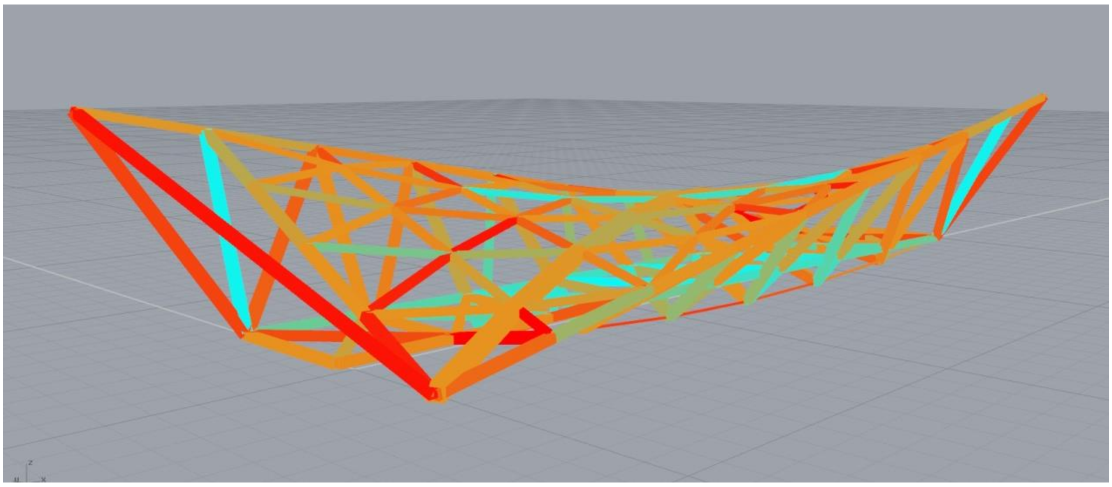


Figure 7.32 Perspective view of the SC final form element utilization

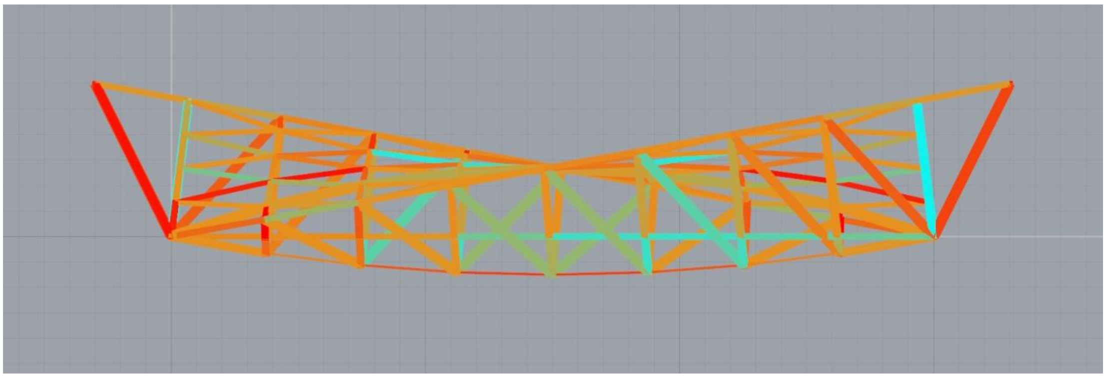


Figure 7.33 Side view of the SC final form element utilization

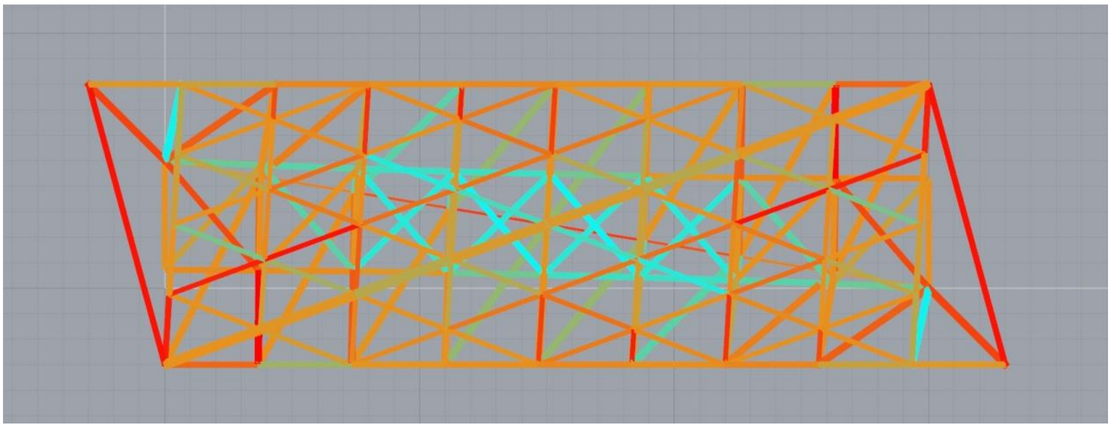


Figure 7.34 Top view of the SC final form element utilization

First analysing the compression/tension behaviour it is clear that the roof arch is fully in compression from support to support. The roof truss members have a seemingly irregular system for compression and tension, this is due to the two competing load bearing systems of the arch and the side trusses. The triangular section modules along the bridge are for the most part compressed at the sides and tensioned at the top, this system changes around the tops though. In these areas the truss is high and stiff enough to change the load paths into that of a standing truss. The deck also follows these somewhat unclear load paths where different parts of the bridge show different behaviours, depending on the dominating system in that zone.

The openings at the entrances have their own local system in the bridge with the roof tops being attached by the three members connected to them, one compressed in the bottom and two tensioned at the top.

Looking at the utilizations a set of highly utilized members near the bridge ends define the required dimensions for most of the element sets. It might seem poorly optimised to have a few members defining the dimensions of other members that do not require it, however this is not the whole truth. Just as with the last concept it is possible to get a uniform and controlled appearance by using identical dimensions for members of the same category. The other issue comes from the combined structural behaviour, by reducing the size of some members the changed stiffness yields different load paths that are less favourable for the desired setup. This is for instance why the compressed arch is only utilized at about 30%, its additional stiffness prevents the smaller roof truss from taking too much of the loads.

The relatively low utilization of the deck members is deliberate. Hidden under the deck cover these elements could be individually optimised without the risk of negatively affecting the appearance. Here they are left relatively over-sized in anticipation for a more thorough analysis including the side loads of the wind, where the deck would be significantly more loaded to cope with the lateral bending.

Considering the serviceability limit state the bridge yielded the plot shown in Figure 7.35 for the deformations:

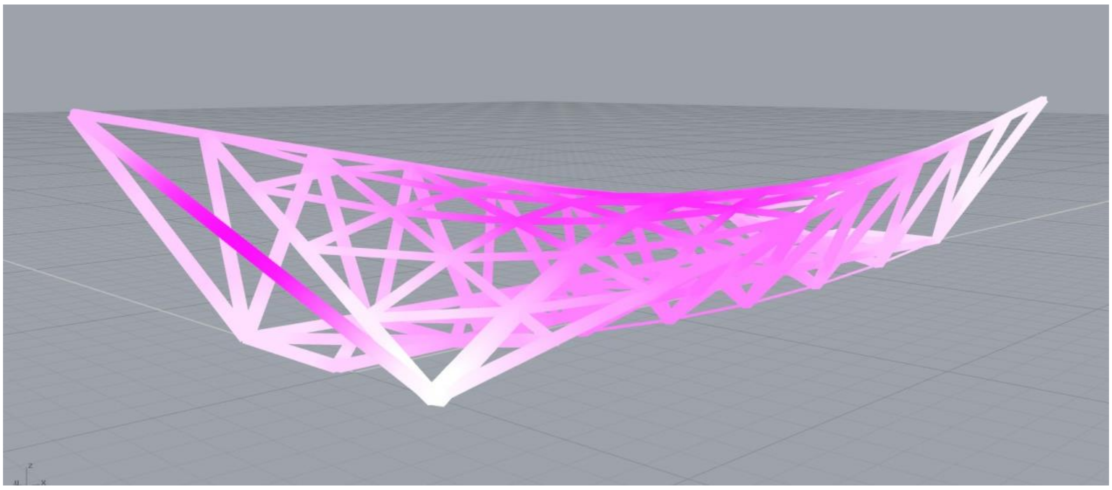


Figure 7.35 Perspective view of the SC final form deformations

In Figure 7.35, the maximum deformation is for the roof end beams, this however is a misleading value since the addition of a roof cover would stiffen the roof behaviour. The maximum deformation disregarding the end beams is the roof centre which is deformed by 108 mm, an acceptable deformation in the ultimate state since it only has an aesthetic effect. The most relevant deformation is the deck which is deflected 58 mm at the most. This value is lower than the limit of 75 mm mentioned for the last concept.

Finally, the dynamic response was checked and for the assumed geometry with its joint definitions the first eigenmode is found at 5,8 Hz. This is over the limit defined in Eurocode and does therefore not require further analysis. This however depends on many assumed rigid joints in timber which is hard to achieve, so in a continued design situation this would be redone with partially fixed rotation joints for many parts of the bridge.

7.2.5 Concept Comments

The SC in general shows the potential of designing a covered timber bridge that becomes an attractive object, both as a potential structure in an environment and as a space for visitors. Some compromises were made from what would have been a more efficient design, but this was kept at a level of which the final shape is still highly functional whilst generating a significant improvement to the architectural design.

7.3 The LEAF Analysis

Continuing on the systematic of the previous concepts the LEAF concept is described in this section, for the rest of the chapter the abbreviation LC will be used.

7.3.1 Final Form

To better understand the LC geometry, Figures 7.36 and 7.37 show two renderings of the final design of the concept.

Figure 7.36 Rendered perspective of the LC final form

Figure 7.37 Rendered perspective of the LC final form

A perspective image of the corresponding Grasshopper geometry for the final form is shown in Figure 7.38:

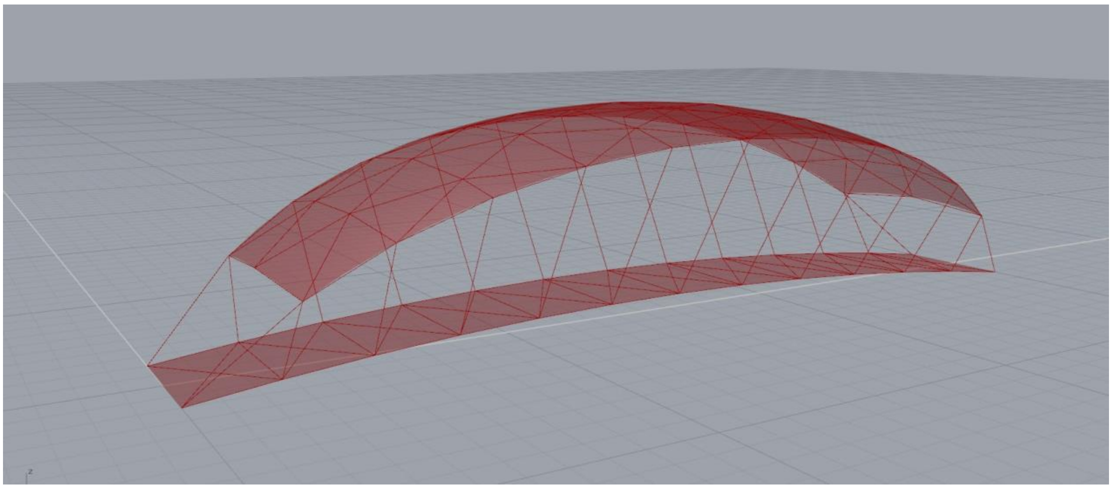


Figure 7.38 Perspective view of the LC final form

The general setup for the SC is two main arches that cover the whole span of the bridge, one on each side of the deck. The arches are connected by a set of curved beams that are connected by zigzag elements forming a continuous roof truss. Under the roof two main cables are tensioned countering the compression forces from the roof, these cables are connected to the roof by a set of secondary cables along the bridge. Inside the tensioned rectangles formed by the cables a set of compressed timber crosses are inserted, upon which the deck can be placed.

The ceiling height inside the bridge varies over the span, this variation and the overall curvature of the bridge can be seen in Figure 7.39. This figure also shows the secondary cables along the deck, and how they in their tensioned state create a slight elevation over the bridge span:

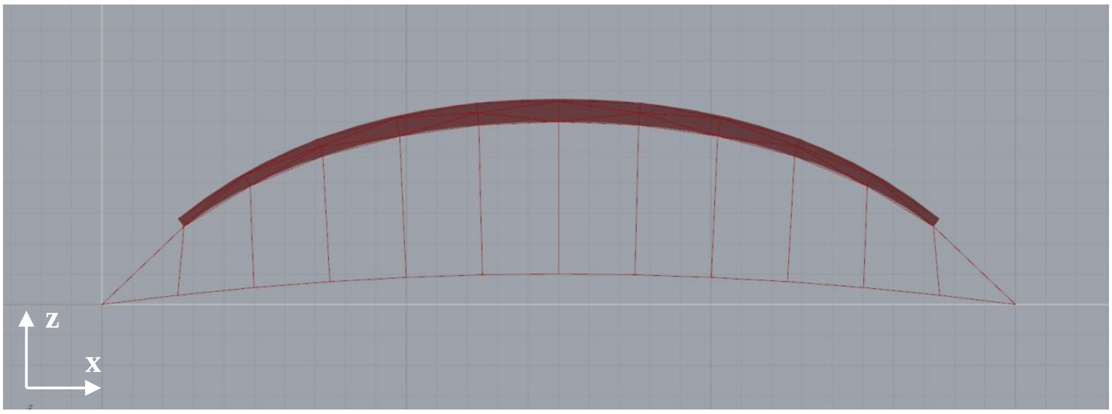


Figure 7.39 Side view of the LC final form

The entrances to the bridge are very simple and consist of the last roof section not having a roof cover, the curved beam in the end of the next section above therefore turns into the top of an opening into the bridge, showed in Figure 7.40:

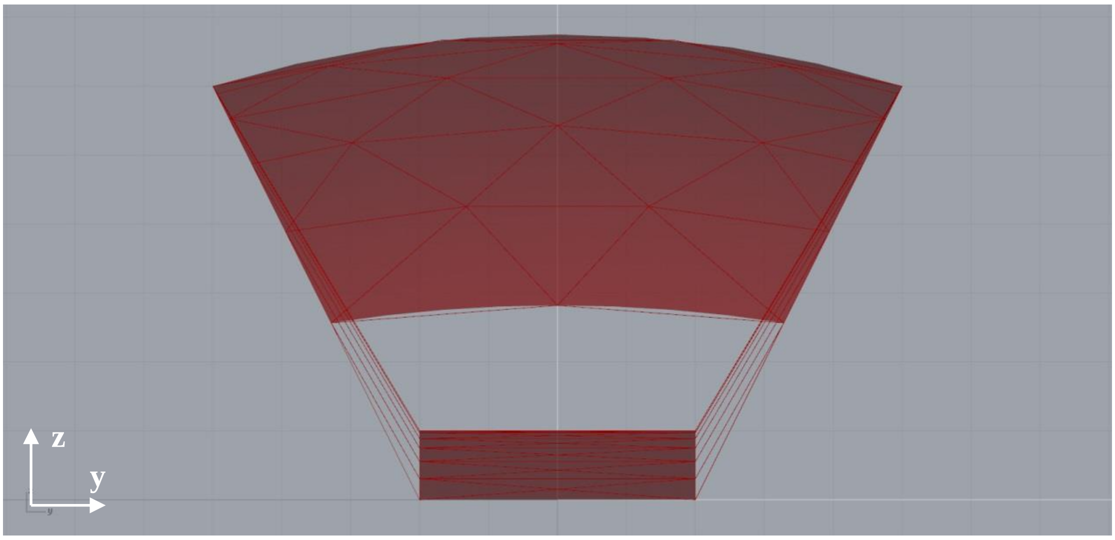


Figure 7.40   Entrance view of the LC final form

The last figure, Figure 7.41, shows the bridge top view which displays the side overhangs of the roof, its truss pattern and the length of the uncovered areas by the openings.

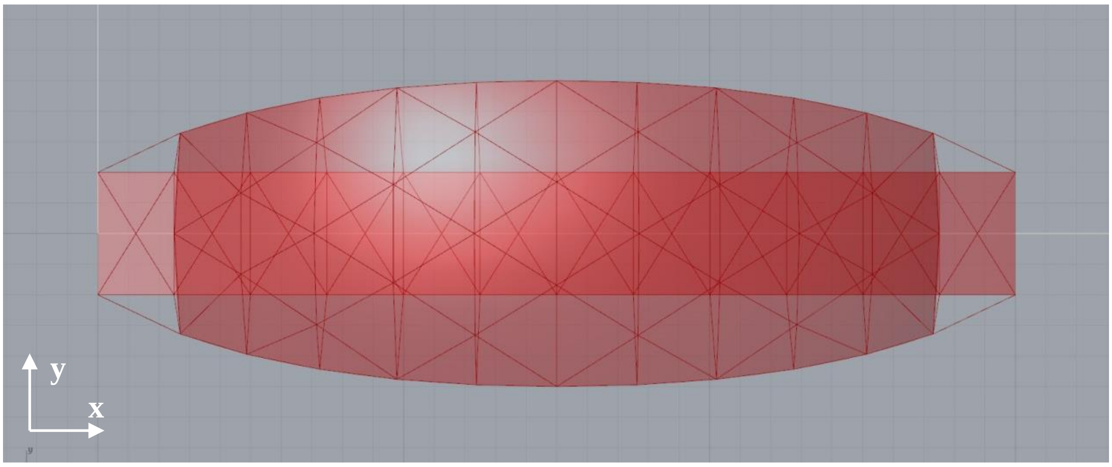


Figure 7.41   Top view of the LC final form

In the model, the supports used are at the four corners of the deck, all of them fixed from vertical translations, one on each side locked for transversal displacements and one support also locked in the longitudinal direction. The joints assumed are fixed for all timber members and moment free for all steel cables. The assumed pre-tensioning of the steel cables is set to the strain being 0.002, this is of course a very rough estimation of the needed values but was deemed sufficient for this phase.

7.3.2 Design Process

Out of all the concepts the LC was the closest to its final shape in the conceptual phase. The very clear and efficient nature of the compressed arch leaves little improvements to be made, although some optimizations took place.

The point loads from the secondary cables with even spacing in the arches are a deviation from the optimal nature of a compressed arch. If only loaded with self-weight the arch would be the structurally most optimal geometry, the point loads however change this shape and for a weightless member the optimum shape would be a discretized curve with a kink at each load. Since the timber arch is relatively lightweight the kinked arch is more efficient for this design, this also allows the arches to be assembled from straight members instead of requiring curved glulam. The joints used for the joining of the main arch members can consist of steel with a connection at the bottom to efficiently transfer the cable loads into the arches.

The roof truss quickly got a regular pattern through the transversal curved members that were connected. The main optimization made here was for the curvature. Significant cost reductions for curved beams can come from having the same curvature for many members. By generating a roof shape that has the same curvature for all transversal members and the varying factor being their length, this benefit is included in this design.

The secondary cables have two potential setups in the final design, either the straight cables shown in Figure 7.42 or the zigzag pattern shown in figures 7.43 and 7.44. The first of these has the benefit of cheaper costs and less material but the latter provides extra stability potentially needed for the dynamic behaviour. During the process no definitive answer to which was the best solution was determined, both solutions thus remain at this stage.

Both cable versions have the secondary cables continuing under the deck creating rectangles with tensioned borders together with the main cables. The final system of supporting the deck consists of inserting timber beams diagonally inside these rectangles, the cable tension will put the beams in compression forming a stable equilibrium for the cable forces. The deck is the put on top of the timber beams.

After the above mentioned optimizations were carried out the bridge now had the element setup determined by the sets shown in Figure 7.42.

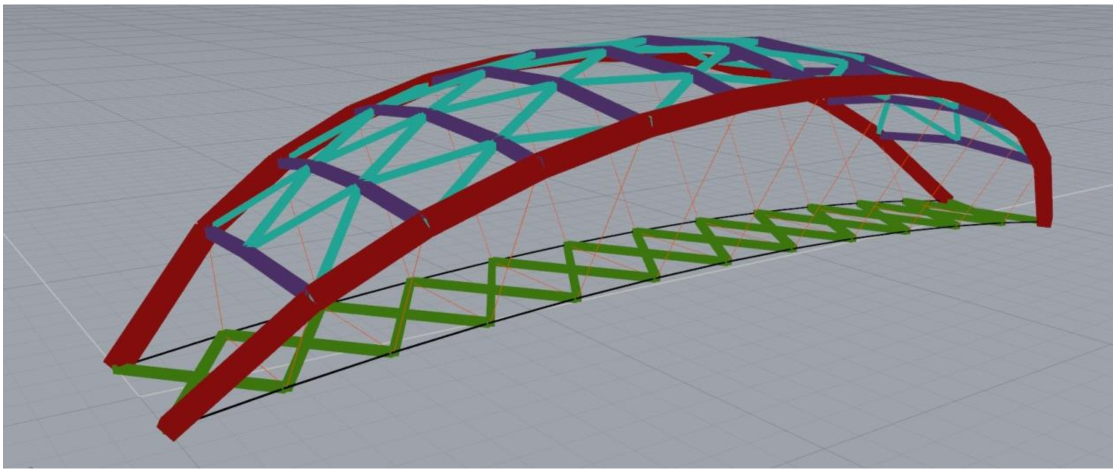


Figure 7.42 Perspective view of the LC element groups

The dimensions corresponding to the element sets shown in Figure 7.42 are displayed in Table 7.3.

Table 7.3 Dimensions of the LC element groups, as shown in Figure 7.42

Set of elements	*Dimensions*
MAIN ARCHES	$\mathbf{450x450\ mm}$
DECK CROSSES	$\mathbf{200x200\ mm}$
ROOF TRUSS	$\mathbf{160x160\ mm}$
ROOF CURVED BEAMS	$\mathbf{250x250\ mm}$
SECONDARY CABLES	$\mathbf{\emptyset 16\ mm}$
MAIN CABLES	$\mathbf{\emptyset 50\ mm}$

Since the dimensions used in this stage are a set of simplified square sections further optimization is needed to turn for instance the large main arches into a functional glulam dimension.

7.3.3 Architecture

During the design phase the LC was also made into a physical model. Figure 7.43 shows the exterior of the bridge at this stage. The picture also shows the experimentation work with roof covers, here testing a design where the roof cover continues out over the entrances.

Figure 7.43 Exterior perspective photo of the LC

Another photo showing an interior perspective from the bridge can be seen in Figure 7.44.

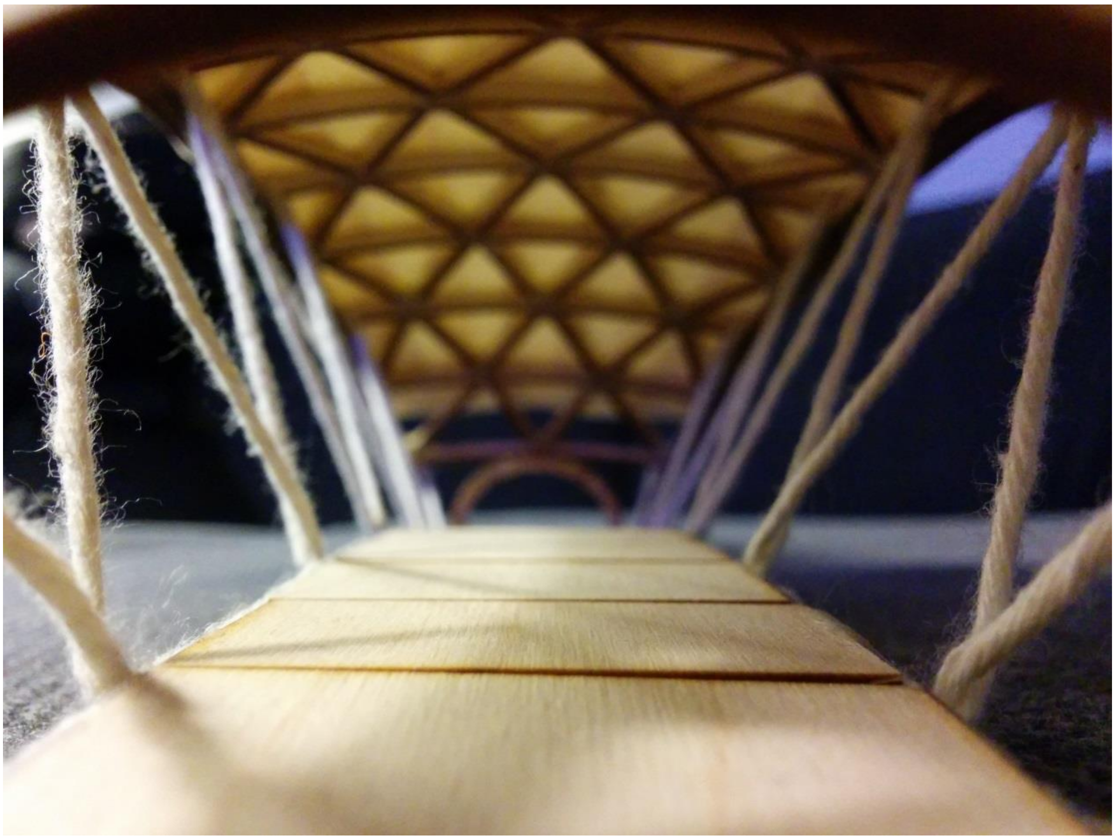


Figure 7.44 Interior perspective photo of the LC

From the picture above the desired regular roof truss pattern can be clearly seen. The simple and controlled nature of it does not steal the attention from the rest of the bridge while still being intriguing when looked at. The desire is to have the same effect with the cables. The modelling method utilizing cotton string yields a partially misleading picture of the final design. But disregarding the oversized nature and fuzzy texture of the strings the geometry setup for the cables also seem to achieve the desired simple form to some extent. It is very likely though that the alternative single cable solution would be even better in this aspect.

The light in the bridge is very generous, to the extent where being inside the bridge is likely to feel a lot like being outside. Both visually and daylight-wise the bridge is very open and just like with the IBC the potential of taking in the environment is taken to high levels in this design.

The design is very elegant and clean but some further work might be needed for the entrances. The solution shown in most figures of this chapter is the simplest one, not adding anything and thus exposing some parts by the entrances to the weather. In further development some more enclosed entrances should be tried, making the transition into the large bridge space pass through a more covered zone.

7.3.4 Physical Behaviour

The following figures show the analysis output from Karamba, starting with the compression and tension behaviour, shown in Figures 7.45, 7.46 and 7.47, **BLUE is compression** and **RED is tension**:

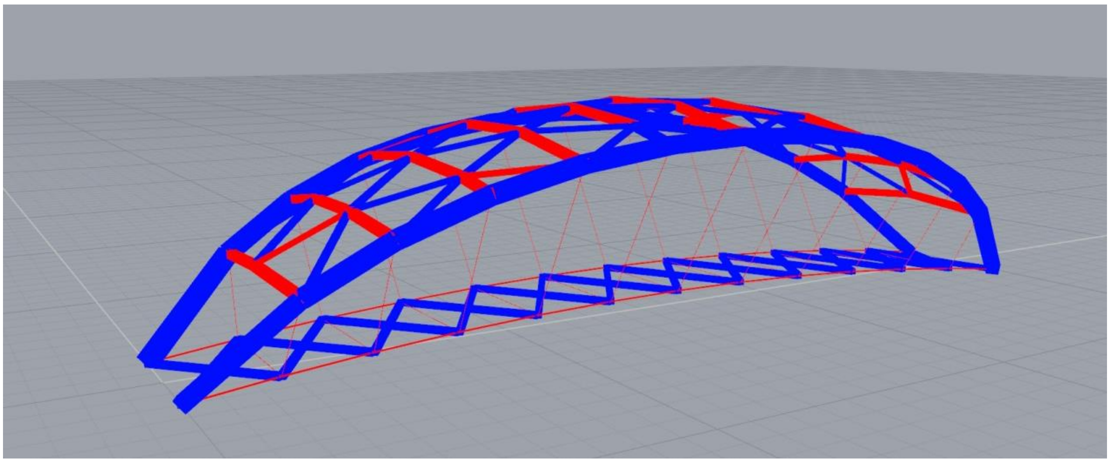


Figure 7.45 Perspective view of the LC final form compression/tension behaviour

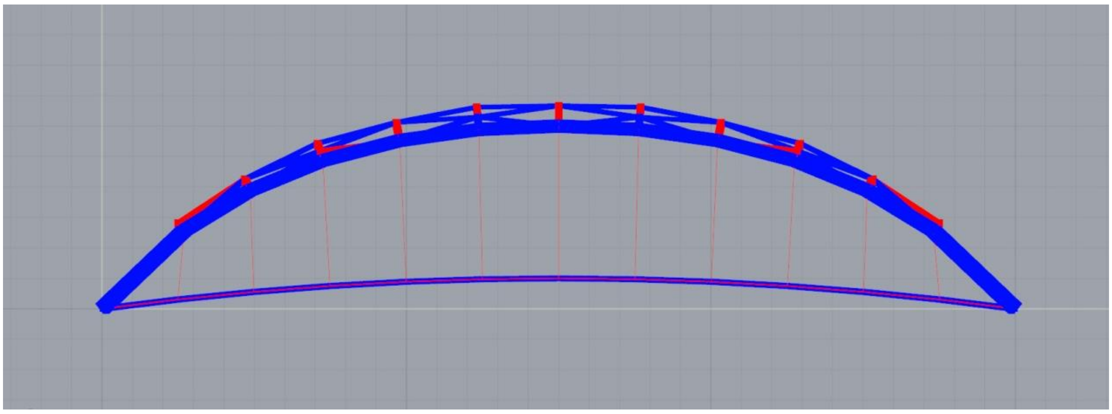


Figure 7.46 Side view of the LC final form compression/tension behaviour

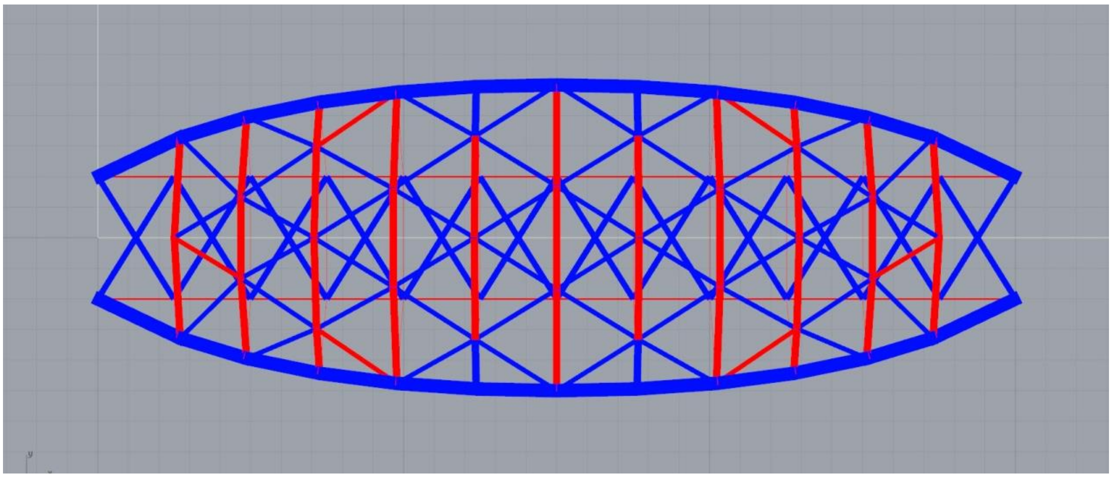


Figure 7.47 Top view of the LC final form compression/tension behaviour

After that Figures 7.48, 7.49 and 7.50 show the element utilizations. An approximate legend for the utilization colour map in the figures is:

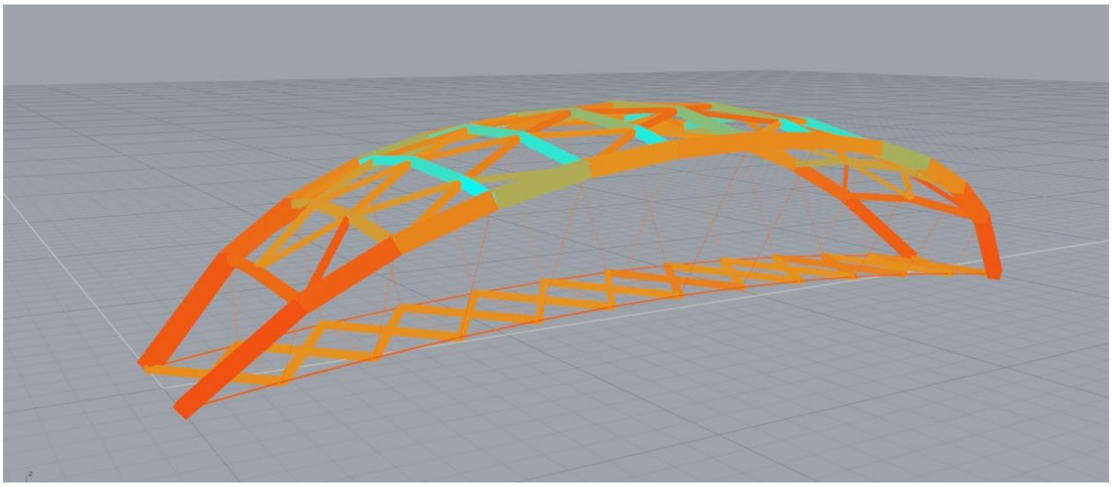


Figure 7.48 Perspective view of the LC final form element utilization

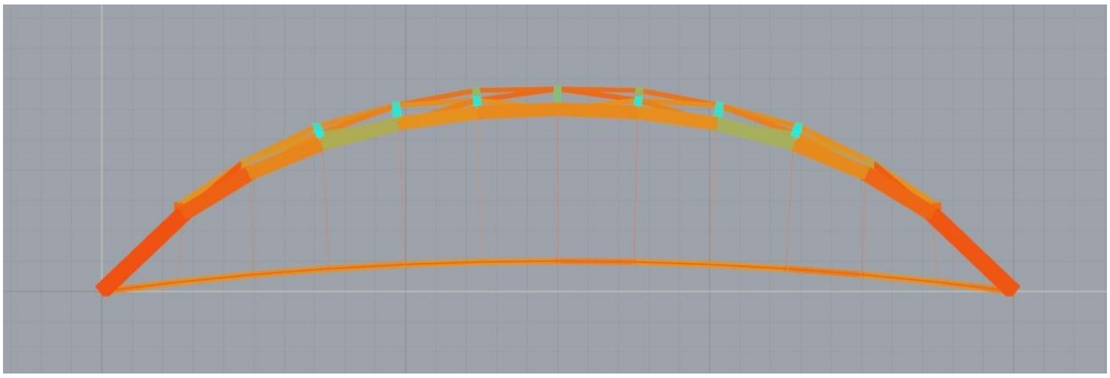


Figure 7.49 Side view of the LC final form element utilization

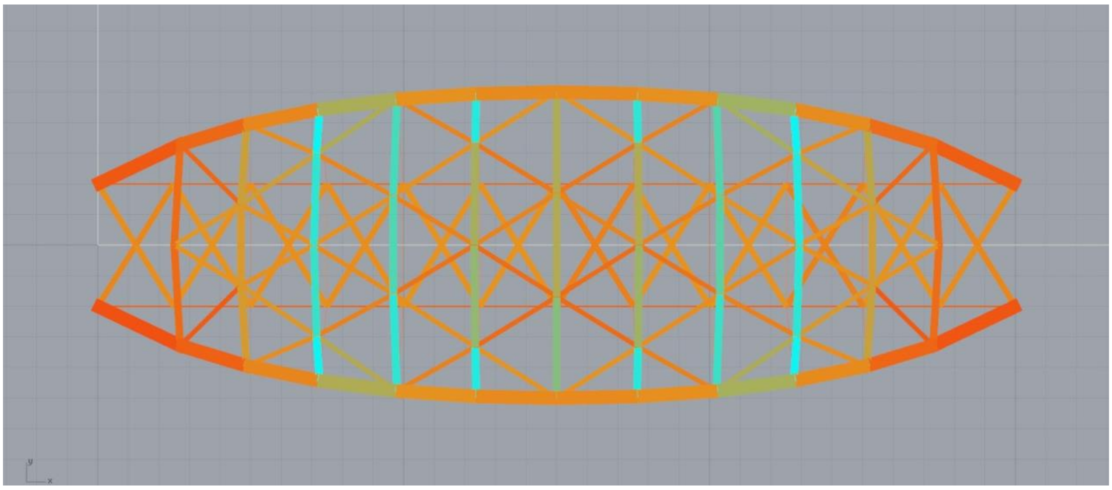


Figure 7.50 Top view of the LC final form element utilization

The first take on the analysis outputs is that the desired compression behaviour in the main arches is achieved. Furthermore the cables and deck also show the desired clear stress pattern.

The only part which requires further explanation is the roof truss. For almost the entire roof the transversal curved beams are in tension and the zigzag truss is compression. The forces will always chose the stiffer route, the horizontal tension in the main arches therefore choose the shorter route through the larger curved beams instead of going through the rest of the truss.

The bridge utilizations are just as for the other concepts governed by the more loaded members. The tensioned cables have the effect of more evenly distributing the loads, and with some exceptions for the curved beams most of the structure is being utilized in the ultimate state.

The side views in the figures also show the material efficiency of steel cables which almost become invisible due to their small required dimensions.

One of the possible analysis outputs that have not been included in this phase is the foundation reaction forces. Out of all the concepts the most critical one in terms of the support is the LC. All other concepts are mainly based on bending, this puts vertical forces on the supports but the rest is mainly handled in the bridge. In this case the strong tensile forces from the main cables have to be handled in the foundation. The compression from the arch will counteract this effect but not entirely.

When it comes to the serviceability limit state, Figure 7.51 shows the deflections of the LC.

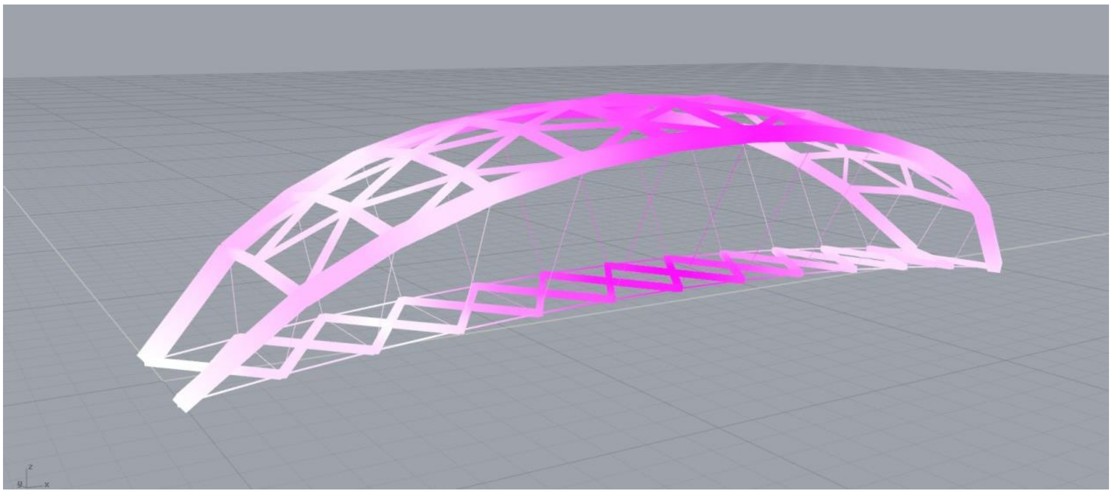


Figure 7.51   Perspective view of the LC final form deformations

The relevant deformations as shown in Figure 7.51 are the maximum roof deflection of 108 mm and deck deflection of 115 mm. These values are higher than the Eurocode limit but do not account for that 62 mm is the bridge deflection at the centre after the pre-tensioning of the cables. Once this is accounted for the deflections are within the desired limits.

The final output is for the dynamic behaviour, Figures 7.52 and 7.53 show the two relevant eigenmodes of the LC, both under the critical limit of 5 Hz

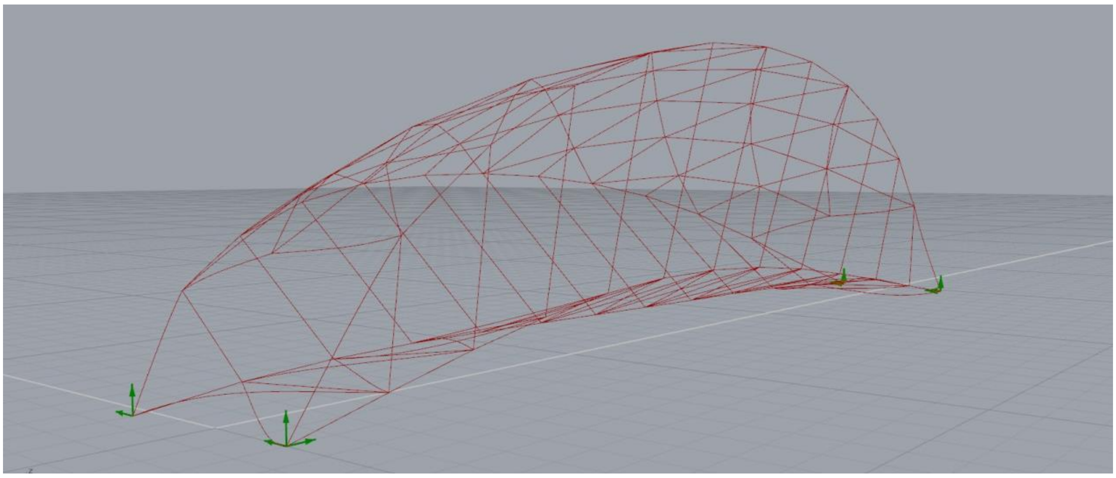


Figure 7.52 First eigenmode of the LC

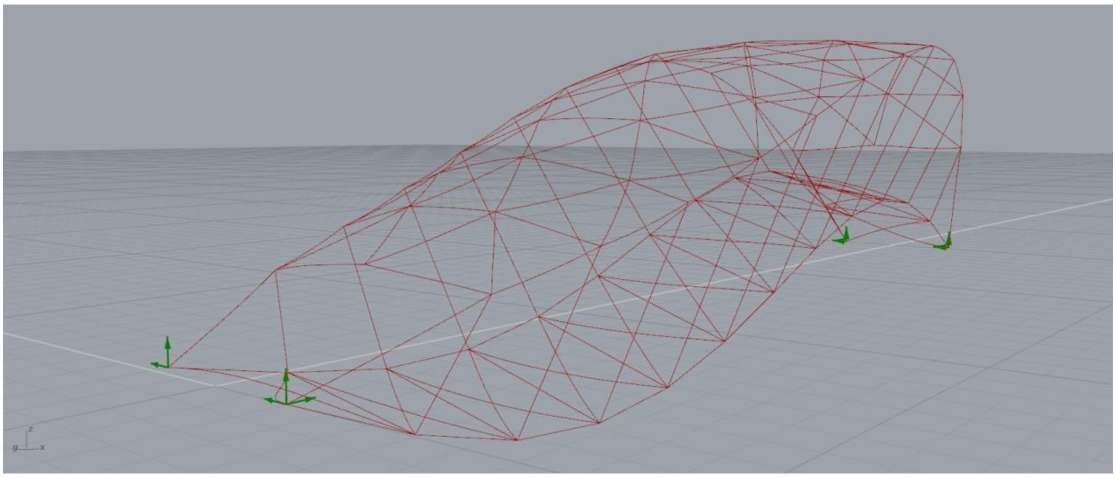


Figure 7.53 Second eigenmode of the LC

The conclusion to take from these eigenmodes is that in a thorough analysis the dynamic behaviour of the LC would require further analysis. The first eigenmode is for the rotation and one potential fix is to stiffen the entrance opening frames for torsion. The second mode comes from the straight cables which do not transmit shear from the roof to the deck, the second cable setup, with a zig zag pattern, will have a better performance here.

7.3.5 Concept Comments

Concluding remarks on this concept are that the material efficiency and optimised shape of the bridge makes it a strong candidate for the final design, so long as the dynamic behaviour is checked further. The aesthetics are very clean and apart from the not completely finished entrances, the bridge has a strong architectural profile in the smooth and efficient form.

7.4 The TUBE Analysis

The final concept analysed is the TUBE concept. For the rest of the chapter it will be referred to as the TC.

7.4.1 Final Form

Continuing on the same system as for the other concepts first the final form is looked at. The renderings in Figures 7.54 and 7.55 show the final design from two different angles.

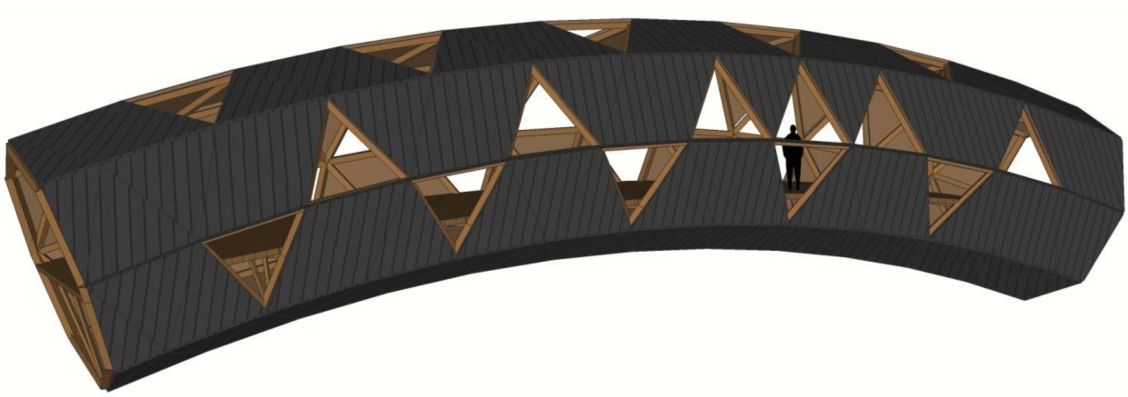


Figure 7.54 Rendered perspective of the TC final form

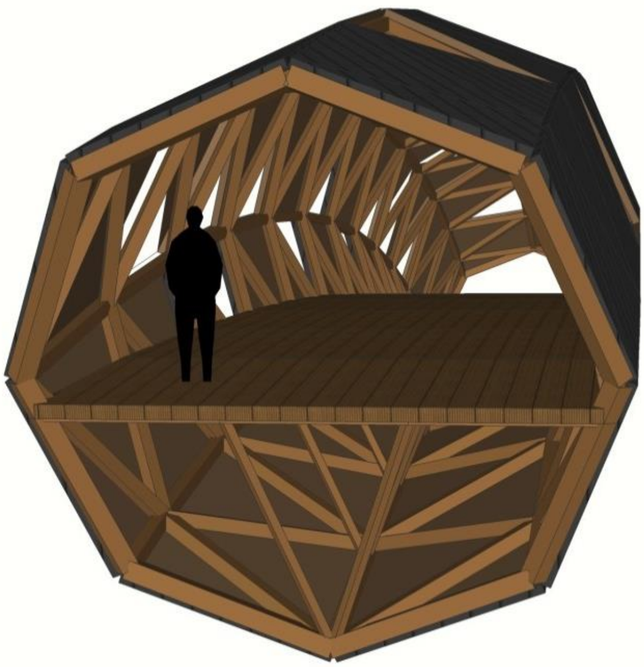


Figure 7.55 Rendered perspective of the TC final form

The Grasshopper point, line and surface geometry that corresponds to this design is shown in Figure 7.56.

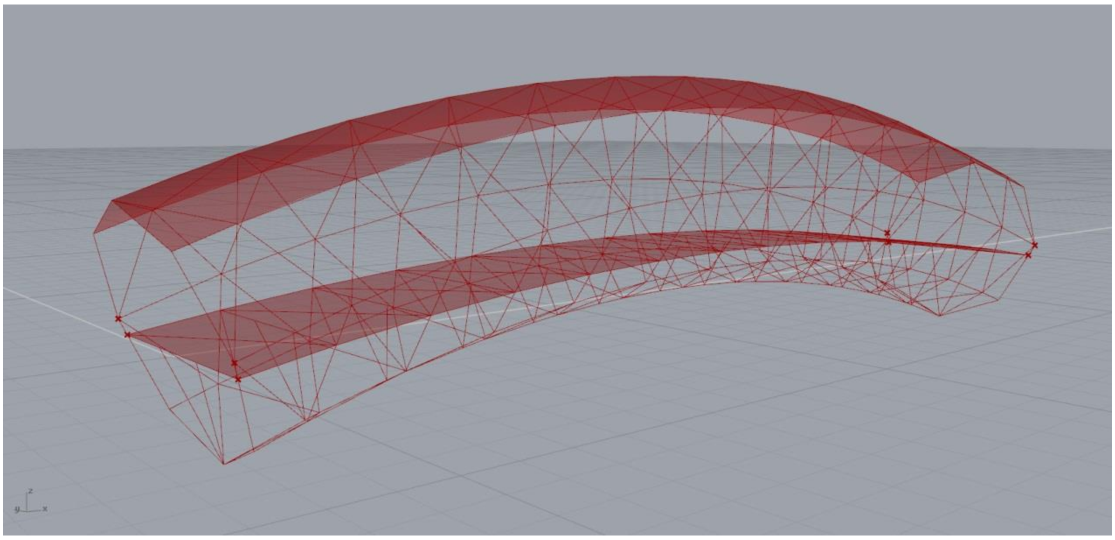


Figure 7.56 Perspective view of the TC final form

The bridge setup is very straight forward, an octagonal triangulated tube truss which spans the whole 30 meters. The curvature of the tube is stronger than the allowed limit for a walking path, the deck inside is thus less curved. The deck is attached to the tube truss along the perimeter but also aided by a set of supports going down to the tube bottom. In the figure the roof surface must not be confused for the planned roof cover design, this zones is instead representative for the area affected by snow loads.

To get a clear view of the curvature of both the tube and the deck inside it Figure 7.57 shows a side view of the geometry.

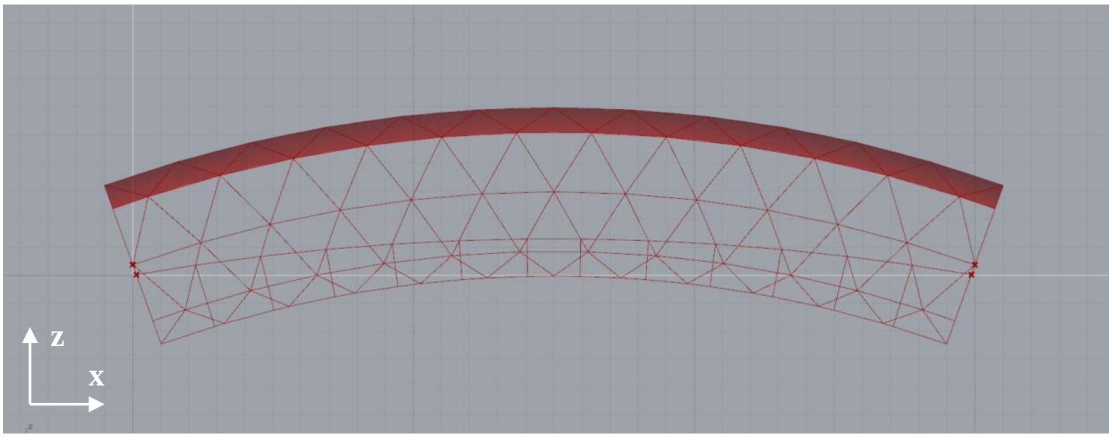


Figure 7.57 Side view of the TC final form

When entering the bridge the visible part of the tube is slightly more than the top half of the section. Inside this will vary with the centre having the highest ceiling height. The bridge and its octagonal section can be seen in Figure 7.58:

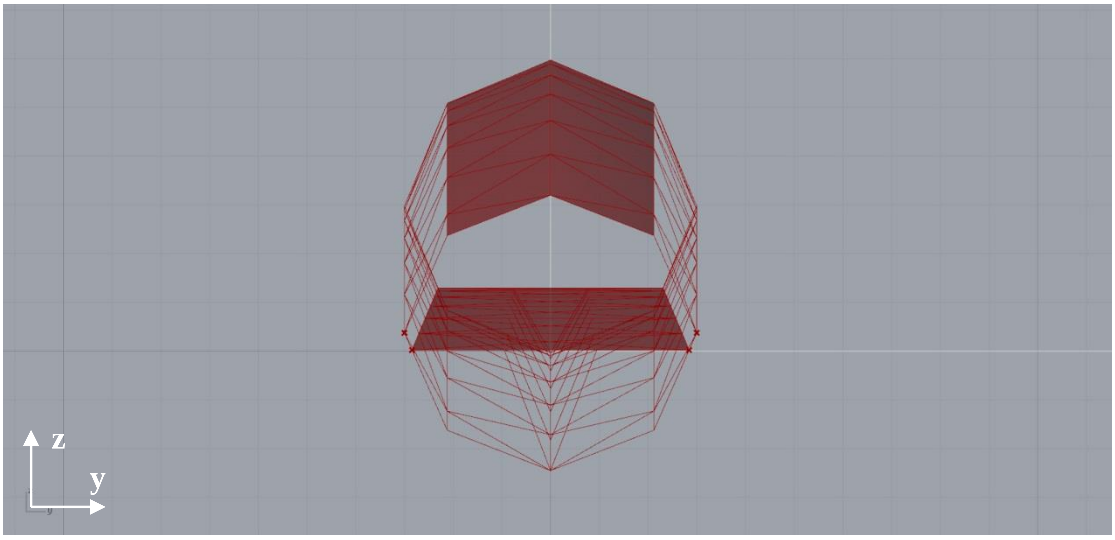


Figure 7.58 Entrance view of the TC final form

The final figure shown, Figure 7.59, displays the top view of the bridge. While there is no overhang needed as with the other bridges one feature visible here unique to the TC is the widening of the deck by the entrances.

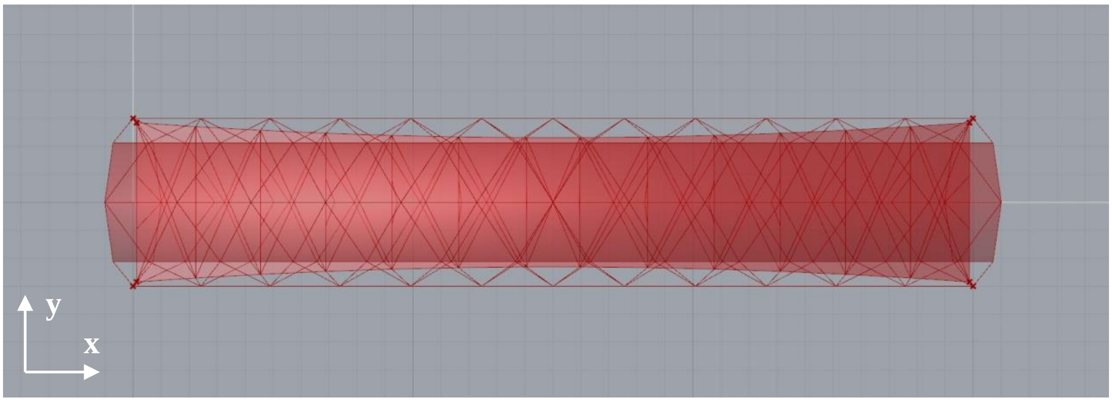


Figure 7.59 Top view of the TC final form

The bridge is just as the other concepts simply supported. The deck corners and two of the end section midpoints are fixed from vertical translation while the other two are further fixed to avoid a rigid body movement as well as the bridge being statically indeterminate. All the joints in the bridge are in this phase fixed, this is an over-simplification for more thorough analyses but will save some time in this case.

7.4.2 Design Process

Similarly to the LC this bridge had many components determined already by the end of the concept phase. The efficient shape of the bridge that already provided the intriguing and appealing features desired allowed for further optimization into the production of the bridge.

Originally the tube had a chain-curve shaped arc which is a very optimum shape. This had the major issue of yielding a bridge with over 400 unique elements. By changing the curvature of the tube into a circular arc section the bridge got a repetitive system that instead has a set of 9 unique members used numerous times.

For a longer period of the design phase the bottom beams of the tube had a larger dimension than the rest. Since the deck rests on these members it seemed natural to increase their strength. This however had the effect of large sections of the tube being very inefficiently utilized. From an architectural standpoint the bridge gains a lot by having identical dimensions for all the visible truss members, this will yield over capacity of some members in less loaded areas. By reducing down the base beams to the same size as the rest some of these low utilization areas were activated more thus improving the bridge material efficiency.

The same reasoning as used above, where the dimensions are motivated by the appearance and might as well be utilized to carry as much load as possible, was used for the supports. The tube entrances had an extended design for a while where the ends continued down to the ground to enable large supports all around the perimeter of the entrances. This was determined not to be an efficient solution since the material in the bridge can handle a classic simply supported setup. The large foundation was therefore scrapped and the bridge truss given an even better utilization, this simultaneously reduced the requirements for the foundation.

As the final shape of the bridge emerged the element set division in Grasshopper looked as displayed in Figure 7.60

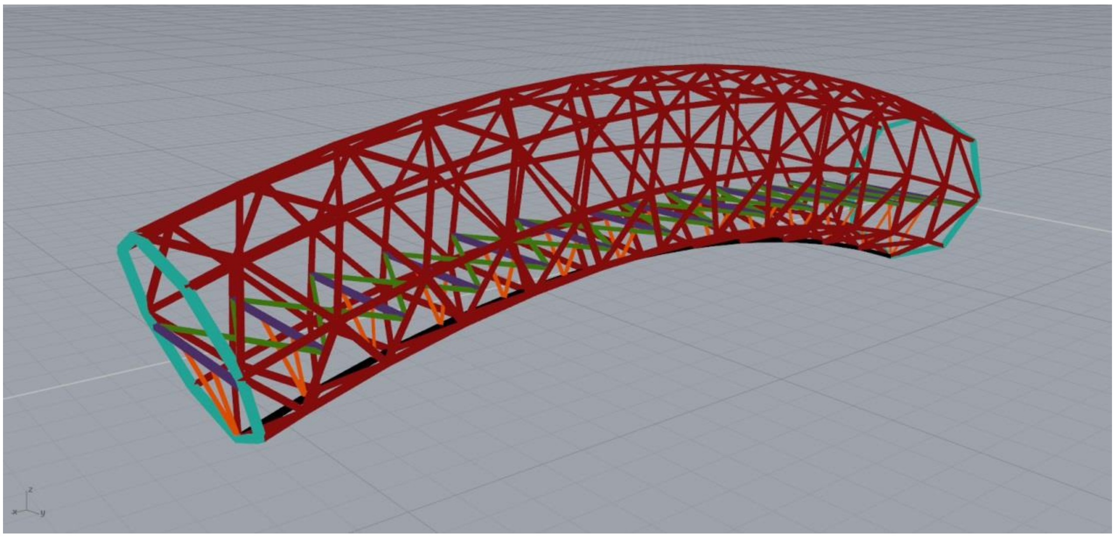


Figure 7.60 Perspective view of the TC element groups

The dimensions from Figure 7.60 are shown in Table 7.4, note that all members in the whole bridge except for the deck have the same section.

Table 7.4 Dimensions of the TC element groups, as shown in Figure 7.60

Set of elements	*Dimensions*
TUBE TRUSS	$\mathbf{160}\boldsymbol{x}\mathbf{160}\ \boldsymbol{mm}$
DECK CROSSES	$\mathbf{115}\boldsymbol{x}\mathbf{115}\ \boldsymbol{mm}$
TRUSS SIDES	$\mathbf{160}\boldsymbol{x}\mathbf{160}\ \boldsymbol{mm}$
DECK TRANSVERSALS	$\mathbf{160}\boldsymbol{x}\mathbf{160}\ \boldsymbol{mm}$
DECK SUPPORTS	$\mathbf{90}\boldsymbol{x}\mathbf{90}\boldsymbol{mm}$
TUBE BOTTOM BEAMS	$\mathbf{160}\boldsymbol{x}\mathbf{160}\boldsymbol{mm}$

Another noteworthy observation from Table 7.4 is that the bridge components cooperate well, this can be seen in the significantly smaller dimensions used for the TC compared to the other concepts.

7.4.3 Architecture

The TC just as the other concepts was also made into a physical model. For this concept this process was extra relevant since the enclosed nature of this concept runs the risk of a unpleasantly dark interior. The photo in Figure 7.62 shows an exterior view of the bridge, here with one of the roof covers/window setups tested.

Figure 7.62 Exterior perspective photo of the TC

More interesting for the analysis of the interior light is the photo in Figure 7.61, showing what entering the bridge during sunlight might look like.

Figure 7.61 Interior perspective photo of the TC

The pictures give a promising view of the potential of this concept. The inside does not seem too dark and the combination of being much surrounded but with many visual openings gives a very interesting experience inside the TC. In this version of the model the window placements are rather evenly distributed, this works well but an interesting test would be to focus the windows more to the centre to have somewhat of a focal point in the middle.

The regular pattern formed by the tube truss provides the TC interior with a well-balanced combination of interesting complexity and clean simplicity. This also stands in general for the bridge where the light curvature and round section shape is in combination with an intriguing window pattern and the previously mentioned truss.

The variation in ceiling height in the bridge is a product of the slope requirements, fortunately this works in favour for the design. The entrances with the lower ceiling become passages into the main bridge space which opens up at the centre. Furthermore this difference in deck and tube curvature yields a widening of the deck by the ends. This works, granted only slightly, as a funnel making the entrances feel more inviting and open to the visitors.

7.4.4 Physical Behaviour

The following figures summarize the analysis output from Karamba, starting with the compression and tension behaviour, shown in Figures 7.63, 7.64 and 7.65, **BLUE is compression** and **RED is tension**:

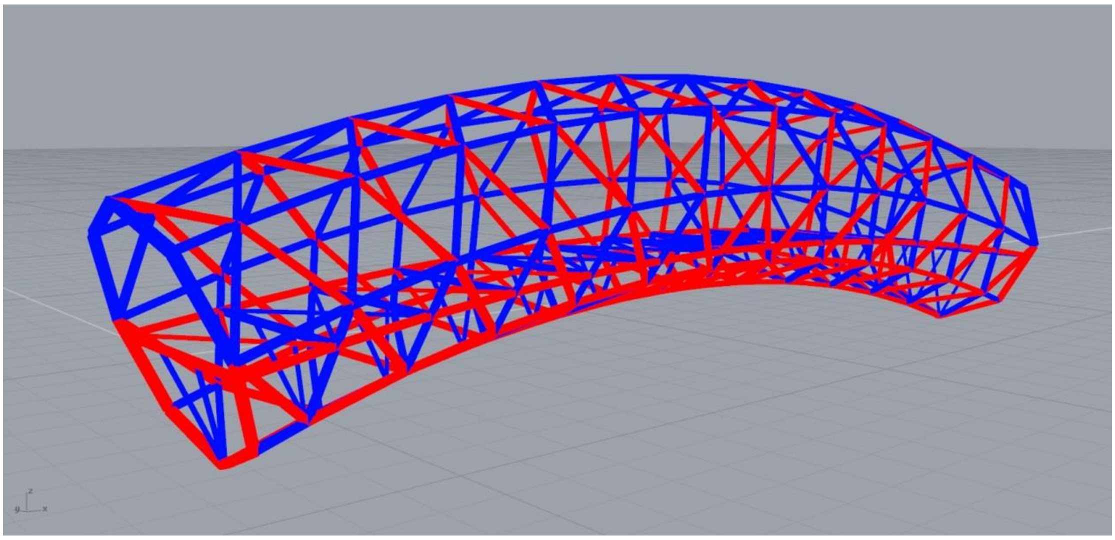


Figure 7.63 Perspective view of the TC final form compression/tension behaviour

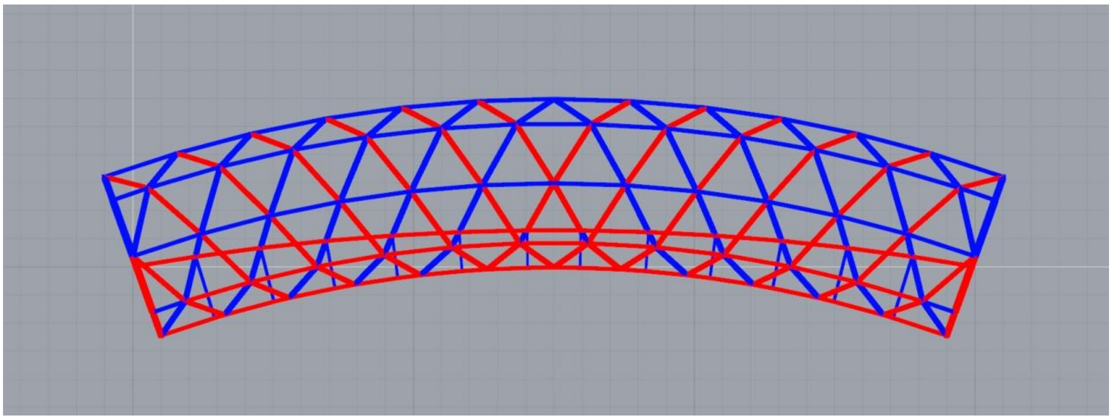


Figure 7.64 Side view of the TC final form compression/tension behaviour

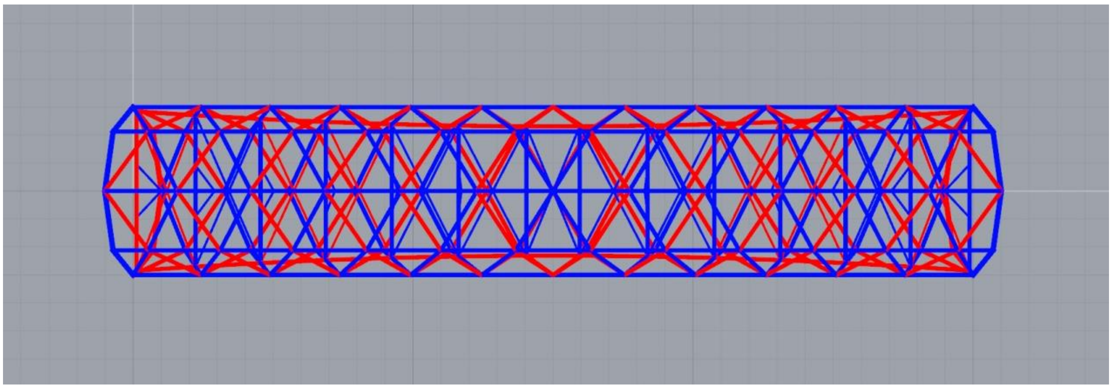


Figure 7.65 Top view of the TC final form compression/tension behaviour

Furthermore Figures 7.66, 7.67 and 7.68 show the element utilizations. An approximate legend for the utilization colour map in the figures is:

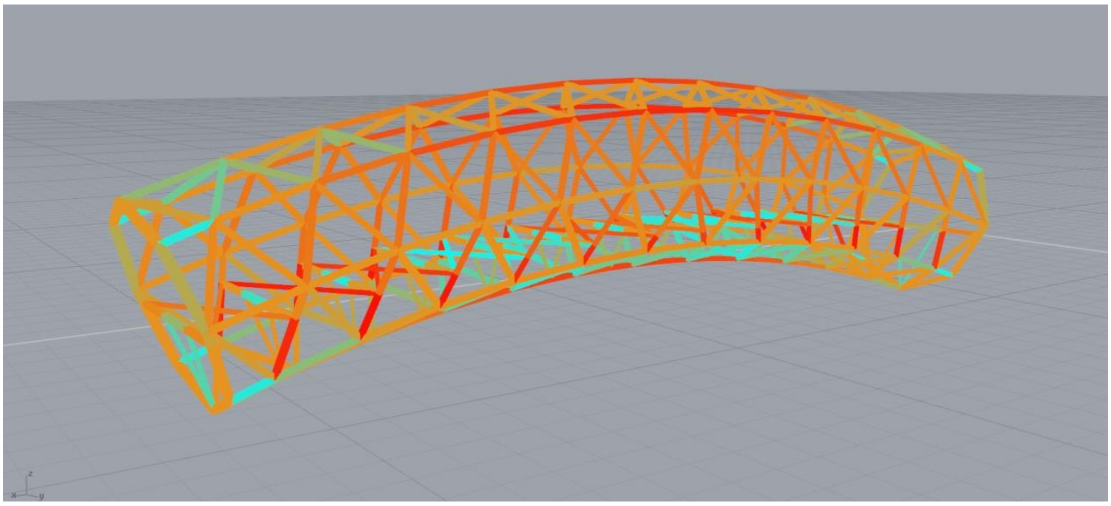


Figure 7.66 Perspective view of the TC final form element utilization

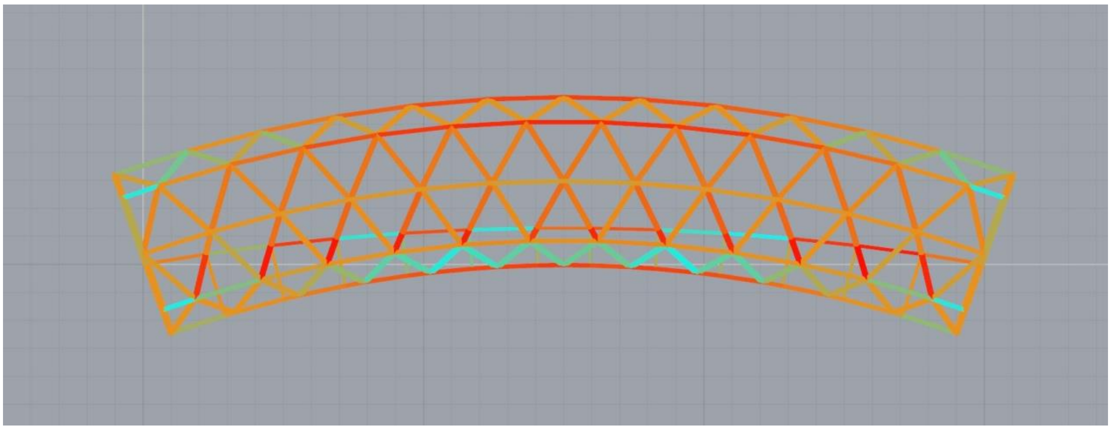


Figure 7.67 Side view of the TC final form element utilization

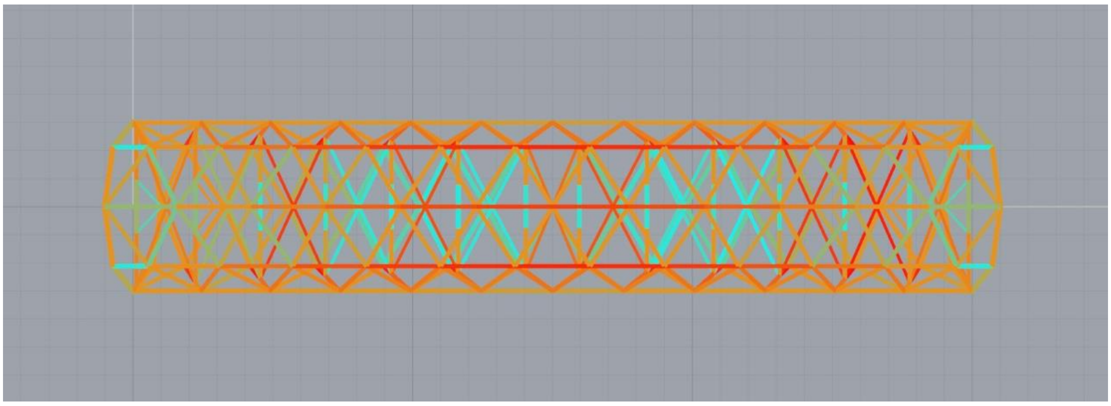


Figure 7.68 Top view of the TC final form element utilization

The large numbers of elements make the first set of figures for compression and tension quite hard to read, some conclusions can be made though. The dominating compression in the top half as well as tension for the bottom give away that the bridge tube works in bending. The arched shape is thus not essential for the load bearing but rather an architectural design choice.

Another interesting observation is the forces in the side view plot. This figure shows that the bridge compression in the top and tension in the bottom travels in arches in the TC. Theoretically this can be imagined as a set of compressed arches and hanging tension curves merged together into one.

The last observation is for the deck, since the deck is located on the lower half of the tube its side beams are all in tension. In addition, the supports under the deck all work purely in compression.

The utilization plots show the effects previously mentioned of trying to activate the tube truss, most parts of it are highly utilized in the ultimate state. The only low utilization is around the deck in the middle and by the entrances.

The TC yielded the deformations plot displayed in Figure 7.69.

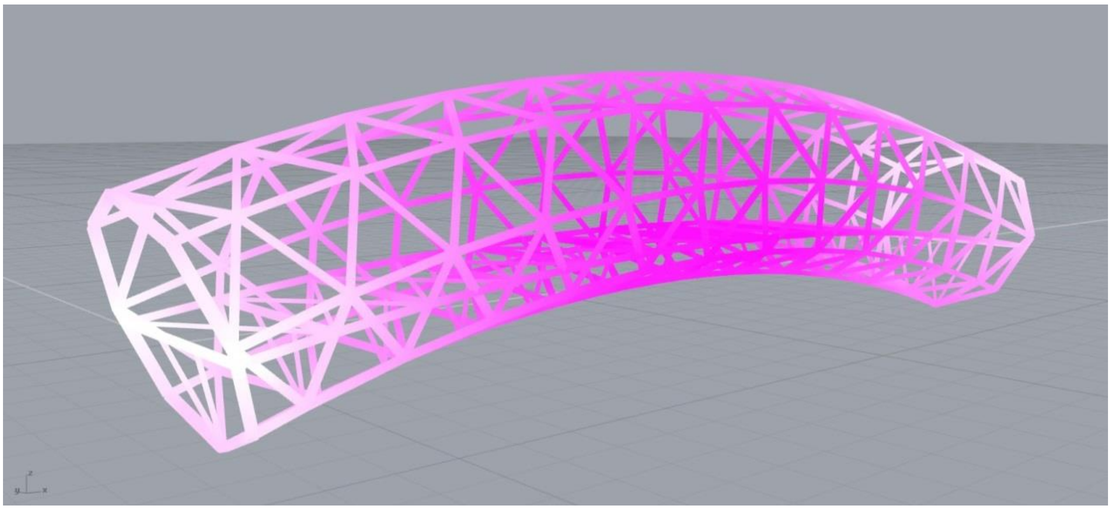


Figure 7.69 Perspective view of the TC final form deformations

The deflections in this figure are only 46 mm for the roof, which is lower than the other concepts due to the stiff tube. This in extension means that the TC has its largest deflection locally for the deck, namely 61 mm. Both of these values however are within the critical limits.

The dynamic response of the TC is excellent and the first eigenmode is found over 6 Hz and is therefore not an issue.

7.4.5 Concept Comments

In general this concept shows a lot of potential, virtually all the functionality and design demands are met. What still remains for this bridge design is to perform the analysis with more realistic joints, in addition the design work with the windows and of course a more thorough detailing are still left on the list of continued work.

7.5 Grasshopper Modelling Algorithm

The bridge optimization process eventually led to a set of parametric Grasshopper models, utilizing the built in modules in the program to form the geometries earlier in this chapter. Since a sizable part of the project was spent working on these models this process ought to be described in some way. For someone not familiar with the Grasshopper methodology it is hard to read a finished code, therefore the bridge codes will not be included in the report. The following section aims to explain the code setup in words instead. Since the different designs have different setups the principal for The SADDLE concept will be described here:

1. ***ORIGIN***, definition of a base point that the rest of the geometry can relate to.

2. ***BASE DIMENSIONS***, creation of a set of variable constants that define the base geometry of the bridge, width and length. For the final design here set as 4 and 30 meters.

3. ***VECTOR CREATION***, the base dimensions are turned into vectors to be used for translations of geometries. The forming of vectors like this in Grasshopper is common and will therefore not be treated as a separate point later in this list.

4. ***ORIGIN TRANSLATION***, the origin point is copied using the translation vectors into the diagonally opposing corner of the base rectangle.

5. ***MIDPOINT***, the new point from 4 and the origin are connected into a temporary line, the midpoint of this line is extracted.

6. ***MIDPOINT DE-ELEVATION***, a new constant is defined for the vertical distance to move the midpoint downwards, the constant is turned into a vector and the midpoint is moved.

7. ***BOTTOM CURVE***, the three now existing points are connected into a curved arc that can be used for the bottom of the bridge. For an arc with a low curvature such as this a chain-curve based arc will be very similar to a circular arc. The simpler circular arc is therefore used in this case, see Figure 7.70.

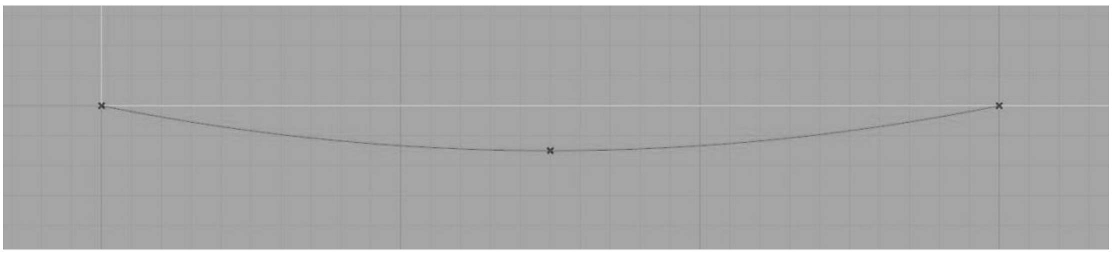


Figure 7.70 Side view of the bottom curve from 7

8. ***BRIDGE TRUSS NUMBER***, one of the key constants for the bridge is defined, namely the number of sections to divide the bridge into when forming the truss. This number is kept even in order to work with the roof truss which requires a section division at the centre of the bridge. For the final design this number is set to 8.

9. ***CURVE DIVISION***, the bottom curve defined in 7 is divided into the above defined number creating the base points for the main truss.

10. ***STEEL ROD CREATION***, the division points from the last action are connected with lines that now form a discretized version of the bottom curve, these lines will be used as the geometry for the steel rods, see Figure 7.71.

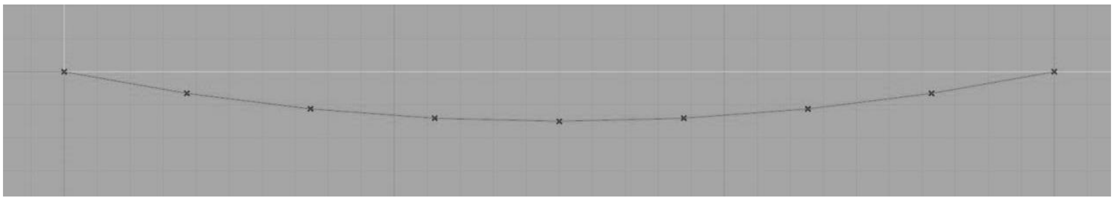


Figure 7.71 Side view of the connected points from 10

11. ***XZ MIRROR PLANE***, a xz-plane to be used for mirroring transformations is created with its origin in the midpoint from 5.

12. ***BASE RECTANGLE***, the two points used in 5 are mirrored using the plane created in 11 to form a base rectangle for the bridge, see Figure 7.72. Note that this is not the bridge deck which will be formed later.

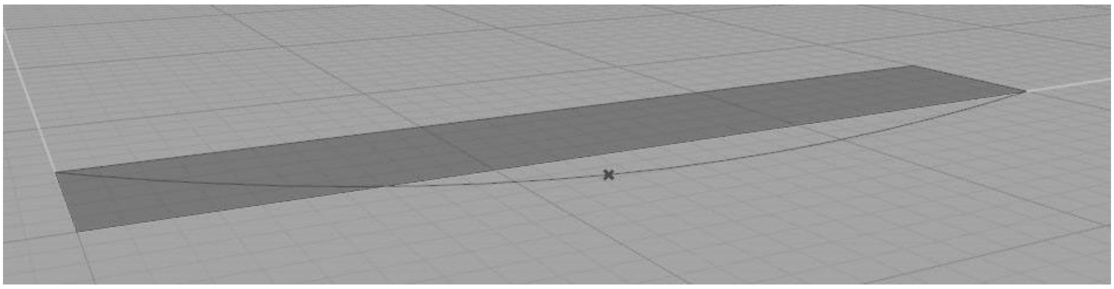


Figure 7.72 Perspective view of the base rectangle from 12

13. ***ROOF DIMENSIONS***, the translation distances for the roof corners relative to the base rectangle corners are defined and turned into vectors. For the final design the roof is shifted 3 meters horizontally out from the bridge base rectangle everywhere except at the two support points which are not translated in the longitudinal direction. The roof top points are in addition raised 6 meters to form the final design.

14. ***ROOF SIDES***, the base rectangle points are copied to the desired roof corners forming the anti-symmetric shape of the roof, they are then subsequently connected with lines.

15. ***SIDE DIVISION***, the two long sides of the roof are divided into one more section than the truss number in order to match the bottom division for the planned truss, see Figure 7.73. Using built in mathematic modules this number is linked to the truss number and has one added before being used.

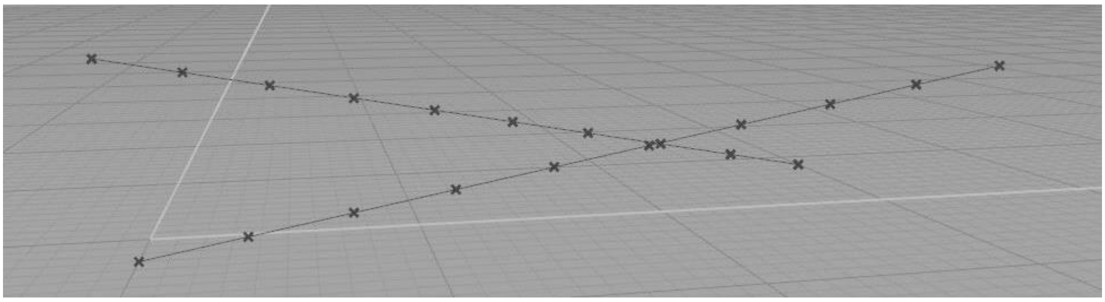


Figure 7.73 Perspective view of the divided roof sides from 15

16. ***LIST HANDLING***, the division points of the roof sides are copied into two identical sets for each side. Each side then has the first point removed for one set and the last point for the other. When both these lists for each side are connected to the bottom points a zigzag pattern will be formed.

17. ***MAIN TRUSS***, the newly formed sets are connected with the bottom points as described above, these lines will make up the main truss of the bridge, see Figure 7.74.

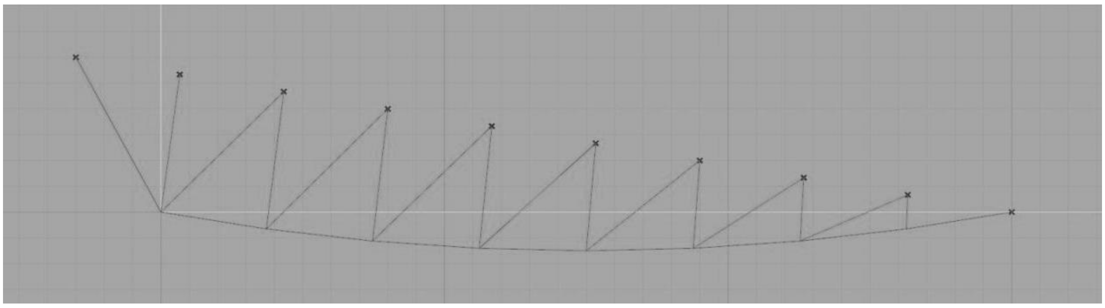


Figure 7.74 Side view of the main truss lines from 17

18. ***ROOF TRUSS TRANSVERSALS***, the roof side division points are connected across, excluding the two top points, forming an odd number of transversal lines.

19. ***ROOF TRANSVERSALS DIVISION***, the lines from the previous point are divided by the same amount of points as number of lines in 18. This way the symmetry of the divisions enable for a diagonal arch inside the roof.

20. ***LIST HANDLING***, the division points from 19 are systematically culled out in a pattern that allows for a regular truss in the roof. This means two different culling patterns: N-Y-N-Y-N-Y-N-Y-N and N-N-Y-N-Y-N-Y-N-N, where Y means remove the point and N means keep it. By alternating between these patterns for the nine transversals in the final design the truss points are created.

21. ***ROOF TRUSS***, by connecting the points from 20 in a manner similar to that of the main truss the final roof truss geometry can be obtained, see Figure 7.75. When put into Karamba later the long diagonal discretized arch formed in the roof can be treated separately to give it the larger main arch dimensions.

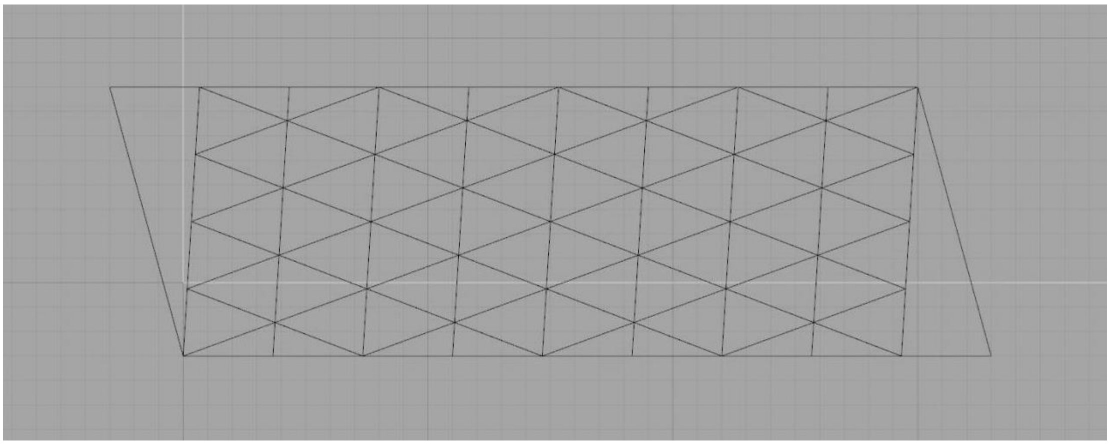


Figure 7.75 Top view of the roof truss lines from 21

22. ***DECK OUTLINE POINTS***, the base rectangle from before does not deviate much from the planned deck, but for most setups in the roof dimensions from 13 the main truss does not perfectly line up with the base rectangle sides. By creating a section plane at the base rectangle height and finding the main truss intersections, the outline points for the deck can be found, see Figure 7.76.

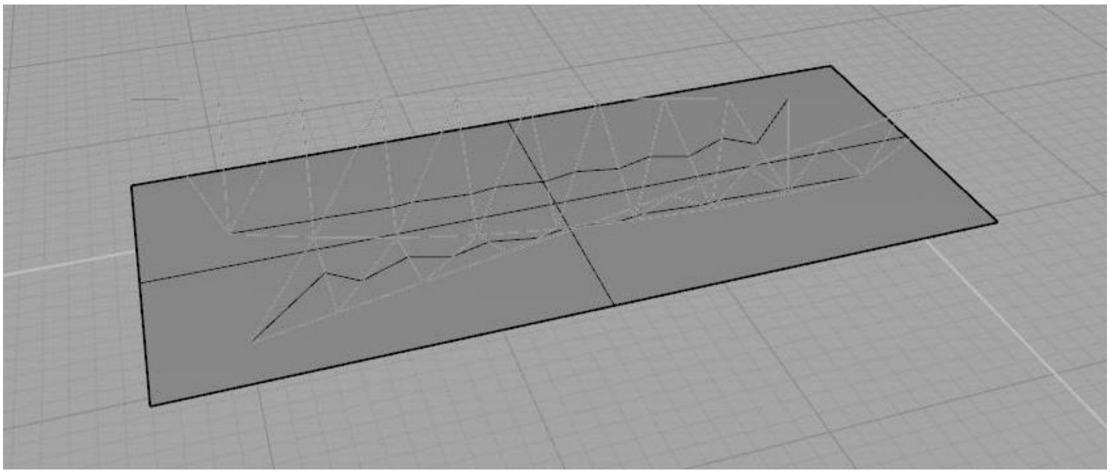


Figure 7.76 Perspective view of the main truss and deck intersection from 22

23. ***DECK SIDES***, the outline points towards the ends of the bridge form a slight zigzag- pattern which is unfavourable for the deck shape. By culling every other point and keeping the inner ones the deck side points form a line that can be supported on the main truss and still has a straight shape, see Figure 7.77. These points are now connected to form the deck sides.

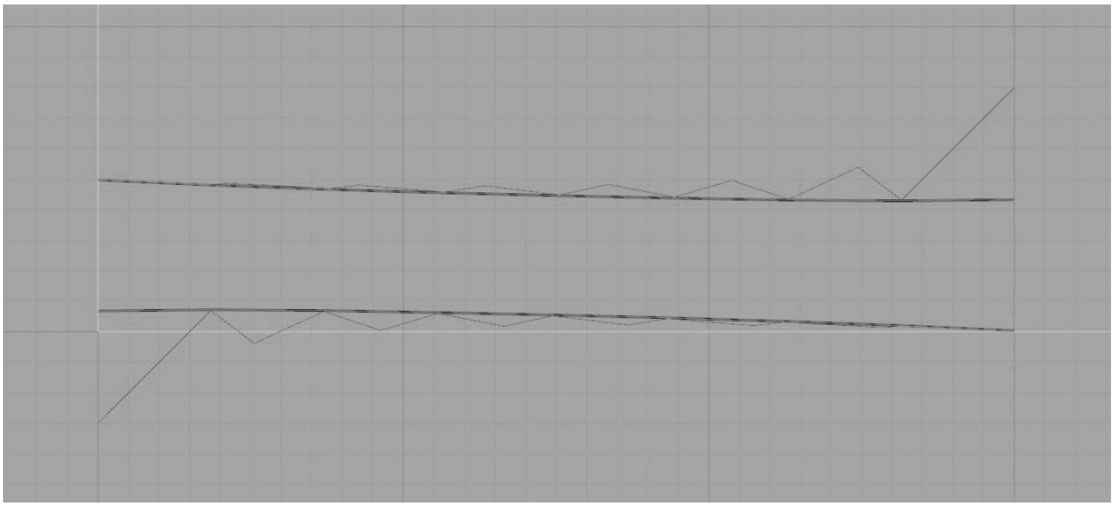


Figure 7.77   Top view of the deck sides from 23

24. ***DECK TRUSS***, by connecting the points from 23 diagonally with the next point on the other side and repeating for the other side, a deck truss is created, see Figure 7.78.

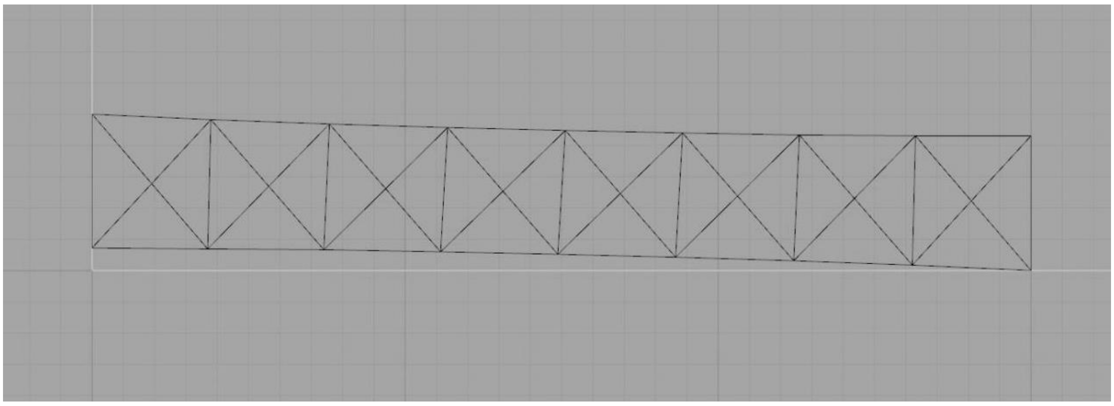


Figure 7.78   Top view of the deck truss formed in 24

25. ***SUPPORT POINTS***, the four corner points of the deck, two of which also serving as endpoints for the trusses, are extracted. In addition the two points of the roof that reach down to the base plane are taken, giving the six support points of the bridge, see Figure 7.79.

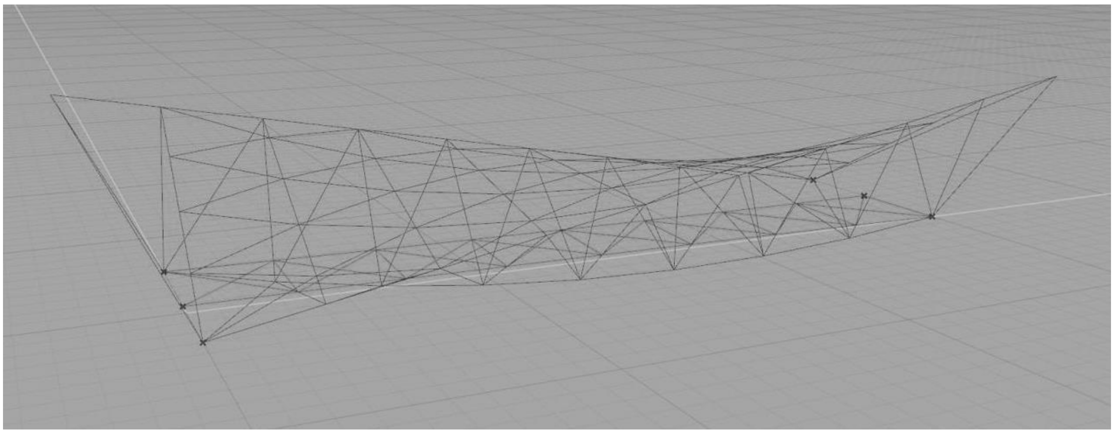


Figure 7.79 Perspective view of the bridge geometry after 25

26. ***MAIN TRUSS DIVISION***, before entering Karamba the main truss has to be divided at the intersections points left in 23. After doing this using a Grasshopper split-module the geometry is ready for Karamba.

The algorithm described here for the SADDLE concept has some similarities with the other three concepts, but the main content and specific list handling processes differ greatly. The actual steps of the other codes are not important for the final results, which are the focus in this project, therefore only this algorithm is showed in order to give a sense of the work that is put into Grasshopper modelling.

7.6 Concept Comparison

At this point all of the four concepts have been investigated enough to make an informed decision to be made on the potential for a final design process. Since the end goal of this project is to determine and optimise the structural system for a set of covered timber bridges that then can be thoroughly analysed in a more accurate program, the bridges could at this point be considered ready for the next stage. However to make sure the bridges perform well both architecturally and structurally some more work can always be done before going into a FEM-program for design of connections and members. For this final design stage of this project only two bridges are selected due to the time required for further analysis. The process of elimination in this phase is based on the performance, architectural qualities and potential of the bridges as determined by the analysis in this chapter.

The great variation between the concepts and their behaviour makes it difficult to compare them in a fair manner, especially in terms of the “soft” features such as spatial qualities. All of the bridges are in some sense viable candidates but in order to finish the final design of the chosen bridges the criterion of work still required became one of the main parameters of the elimination.

The two concepts that were eliminated are the I-BEAM and the SADDLE. In theory these two bridges also have enough work done to move on to the next phase, but since the other two show even more promise the IBC and the SC will not be further treated.

The two chosen designs, **the LEAF and the TUBE**, were picked partly because of their spatial qualities and structural behaviour but also since they are the closest to being finished designs.

8 Final Design

Entering the last design stage for the two chosen bridges the following chapter sums up the road to the finished designs of the bridges. As mentioned before this project does not include calculation based design for the connection details, suggested design systems are therefore included here to be used as the basis for continued work with these bridges.

8.1 Analysis Refinement

The following points summarize the modifications made to the Karamba models in order to get a more realistic analysis of the winning bridges.

The previously rigid connections for all timber members in the bridges are replaced with more realistic connections. Mainly this means that all truss members will be connected via slotted plates, having moment stiffness in two directions left but being assumed completely moment free around their local z axes. In order to insert this type of connections another important modification was needed, namely rotation of elements. In the previous analysis round the elements were oriented along the inserted lines but with their local z-axes as close to the global z-axis as possible, the standard orientation in Karamba. In for instance the TUBE concept this meant that many of the truss members were rotated incorrectly, though still representing the bridge structural behaviour in a relatively accurate way. Thus a thorough reorientation of all members and a subsequent connection modification was carried out for the bridges.

The next change to the analysis is the addition of wind loads other than the downward vertical ones. The LEAF concept is very resistant to effects from side wind so for this bridge only uplift was included but for the TUBE the effects of horizontal wind was added to the analysis.

In order to further optimise the production and construction costs of the bridges the dimension inputs were modified to the glulam standards of Sweden. This means that the beam heights will be a multiple of 45mm starting from 180 mm, and the beam widths will be a multiple of 25 mm starting from 90 mm. In addition, the width has an upper limit of 215 mm for single beams where if larger dimensions are needed two beam will have to be joined. Curved beams also have another standard, to allow for efficient bending of the lamellas, where the height instead is a multiple of 30 mm.

Another addition to the analysis was the foundation reactions. Since both bridges have been given attention to put less requirements on the supports, this section also includes the results on what the supports actually need.

8.2 The LEAF

Apart from the general modifications mentioned on the last page the LEAF concept had two significant changes to the design, the largest of these was the decision to go with the zig zag secondary cable setup. This change significantly improved the dynamic response without drastically changing the architectural design. Along with this change the deck had to be divided into one more section, this means two more members but also shorter spans for the deck system.

After those changes the final dimensions of the LEAF concept elements were as described in Figure 8.1 and Table 8.1:

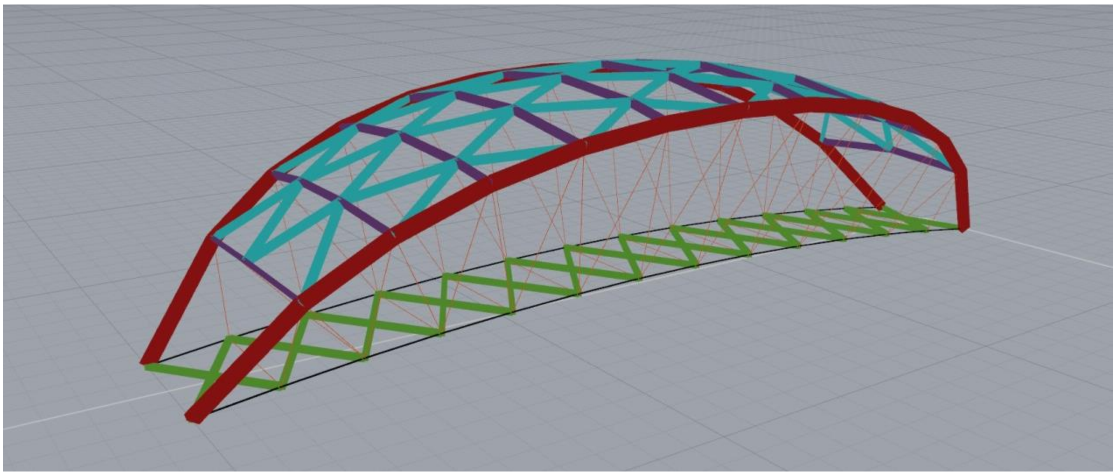


Figure 8.1 Perspective view of the LEAF concept final element dimensions

Table 8.1 Final dimensions of the LEAF concept, as shown in Figure 8.1

Set of elements	*Dimensions*
MAIN ARCHES	$215x405\ mm$
DECK CROSSES	$90x180\ mm$
ROOF TRUSS	$140x225\ mm$
ROOF CURVED BEAMS	$140x240\ mm$
SECONDARY CABLES	$\emptyset 19\ mm$
MAIN CABLES	$\emptyset 50\ mm$

In the final design, the main cables have a pre-tensioned strain of 2.5‰ and the secondary cables 1,5‰. These values will have to be checked more thorough in a more comprehensive analysis but can be a good estimation of what is needed.

8.2.1 Final Analysis

After changing the model the following analysis outputs were generated, starting with the ULS utilisation of elements shown in Figure 8.2.

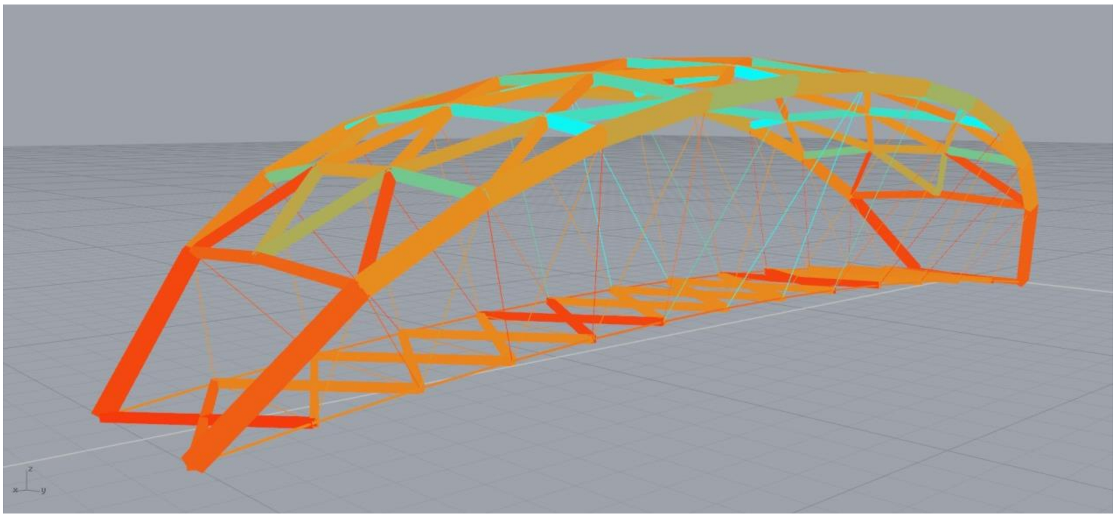


Figure 8.2 Perspective view of the final LEAF concept element utilisations

Figure 8.2 shows a slight difference in terms of the utilisation from Chapter 7.3 towards higher usage of the elements than before. This partly comes from the change in connections which allowed the elements to work more efficiently, but also from the updated dimensions table which allowed for the finding of more optimum dimensions.

The wind load uplift was also checked but yielded an almost undetectable effect on the bridge. Since this design features a strong tensioning downwards of the roof this bridge is very resistant to the effects of uplift. A check of local uplift for the roof fasteners is also needed in a full design but since the fasteners have not been specified here this is not included in this project.

The serviceability limits were also checked after the updates and Figure 8.3 shows this result for the deformations:

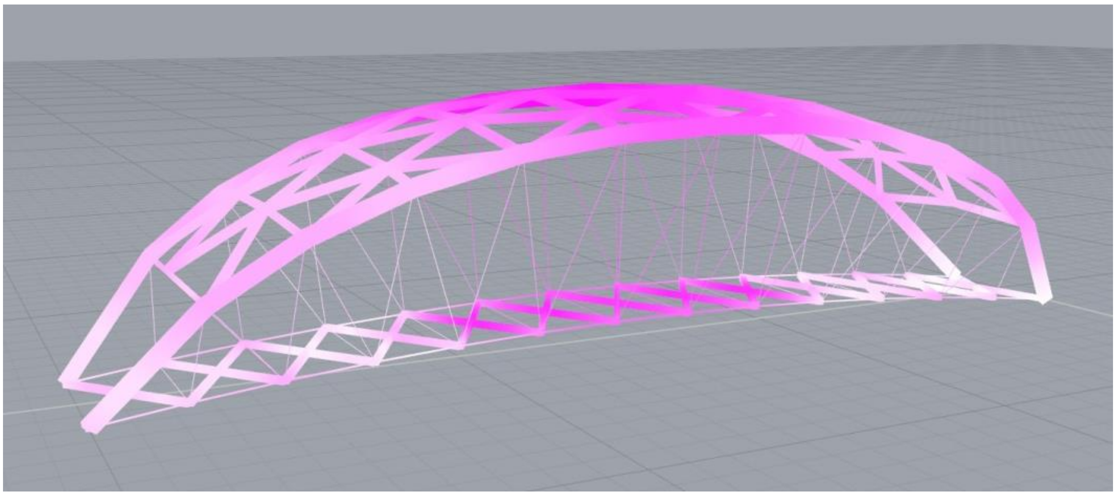


Figure 8.3 Perspective view of the final LEAF concept deformations

The deformations at the centre in this figure are 67 mm for the deck and 66 mm for the roof, both these values are within the Eurocode limit of 75 mm.

The dynamic behaviour was also checked, now with the improvement of changed secondary cables included. After this modification there were two relevant eigenmodes left, shown in Figures 8.4 and 8.5.

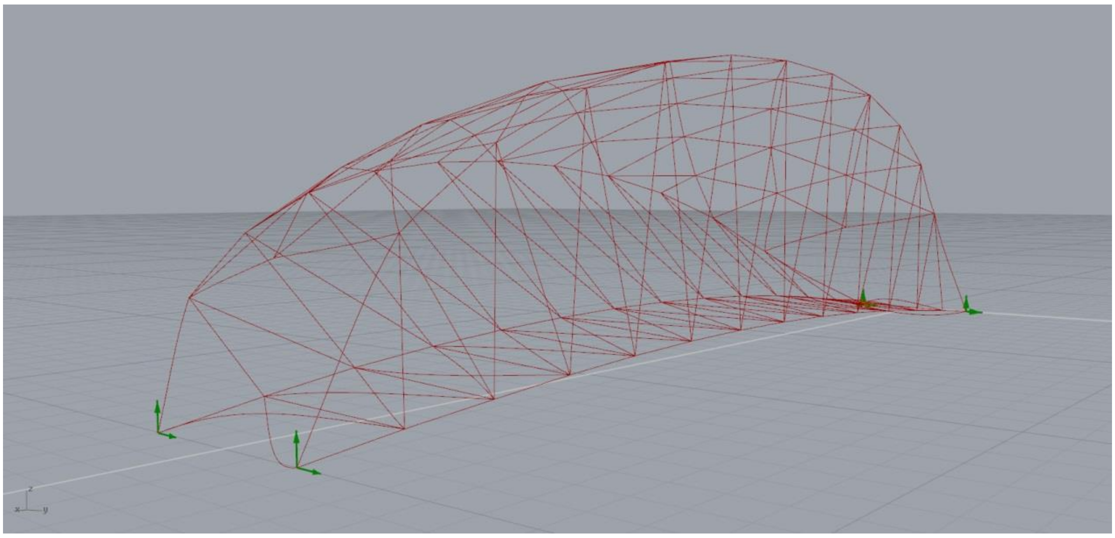


Figure 8.4 Perspective view of the first eigenmode of the LEAF concept

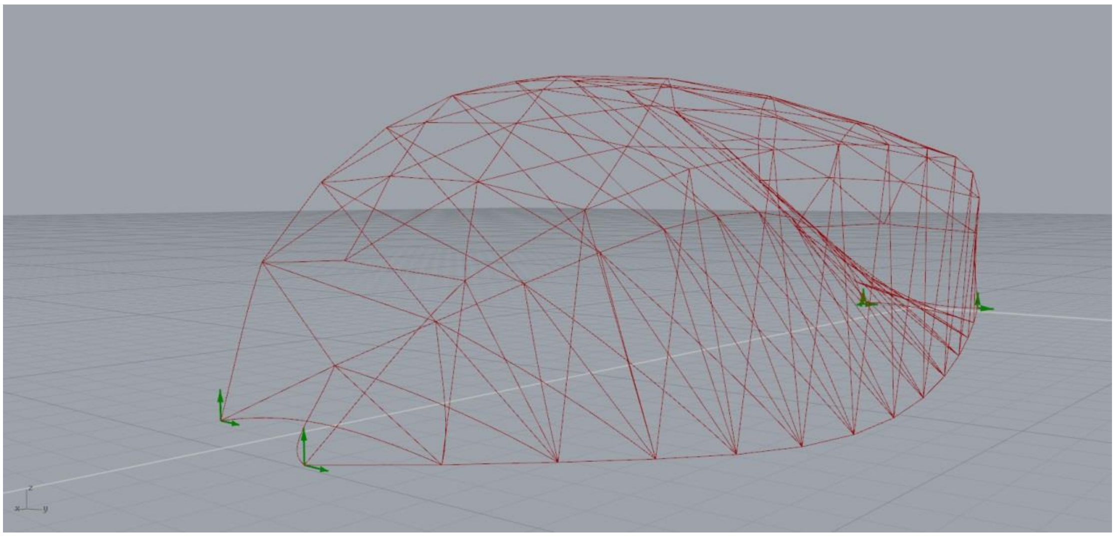


Figure 8.5 Perspective view of the second eigenmode of the LEAF concept

These two modes are located at 1,03 Hz and 4,84 Hz. Both these values require further analysis according to Eurocode, however they are both outside the most critical zone around 1,5 to 3,5 Hz for pedestrian bridges. Therefore it is reasonable to assume that the dynamic response is sufficiently handled in this stage not to be an unmanageable problem when looked at further.

8.2.2 Connection Details

The analyses show that the bridge now is fully functional, however this depends on the assumed connections. This section therefore contains suggestions on details that fulfil these assumptions and work with the aesthetic design.

The first important connection is between the main arch elements. The discretised arch made from straight elements will have to be merged in a location where in addition the secondary cables and curved roof beams also connect. One solution is shown in Figure 8.6.

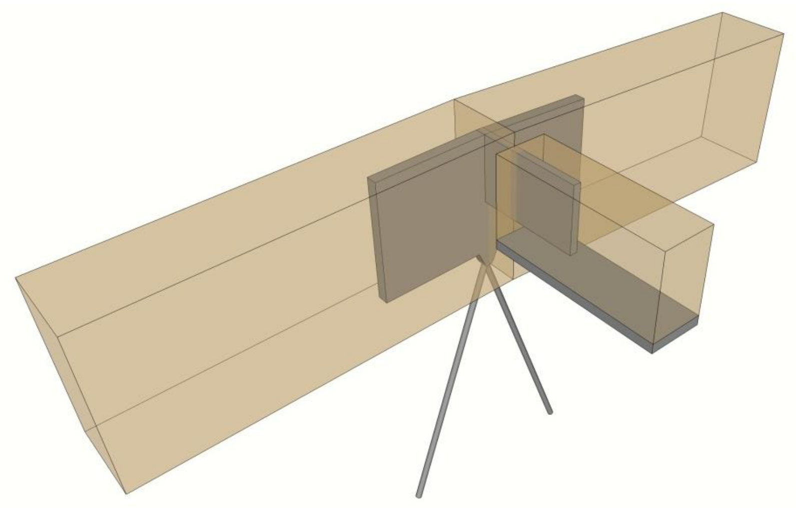


Figure 8.6 Perspective view of a potential main arch connection detail

The principal of this detail is a T-shaped steel plate that joins the arch seamlessly and creates a small gap of 20 mm to the curved beam. This gap creates a slight shadow in the final design that gives a seamless yet marked connection. The plate is extended down slightly to allow the cables to attach to the steel.

The next detail is the connections in the roof truss, shown in Figure 8.7

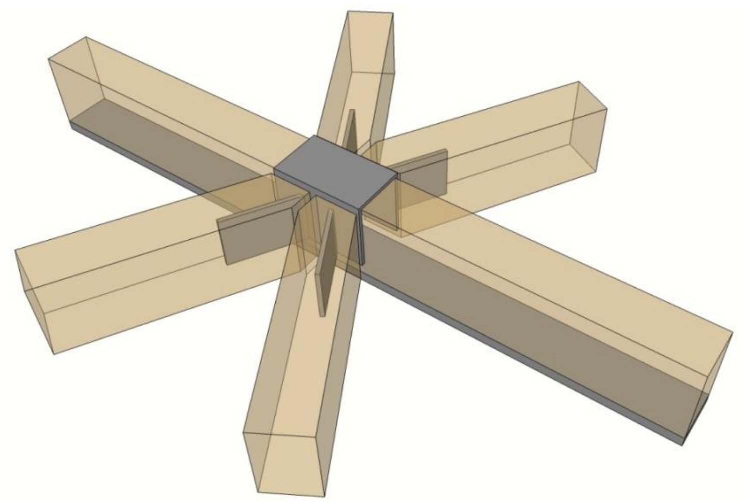


Figure 8.7 Perspective view of a potential roof truss connection detail

The connection showed in Figure 8.7 that the truss elements, just as assumed in the calculations, are connected with slotted-in steel plates. The plates are welded to a steel hat which is hanging on the curved beams, this way it can all be connected without large intrusions on the curved beams. Just as with the end connections of the curved beams the straight truss members have a 20 mm gap, giving a uniform appearance to the whole roof. Both of the figures on the last page also show a lighting strip attached to the bottom of the curved beams, this way the lighting will seem in harmony with the bridge design and not appear as an added feature in mismatch with the rest. This lighting design will also create an interesting light pattern during night time where a set of curved lights sweep over the bridge deck.

Another important detail is the connection between the cables at the deck level. Here the main cables have to attach to the secondary cables in three directions and simultaneously hold the deck beams. A suggested design which also incorporates the railing is shown in Figure 8.8.

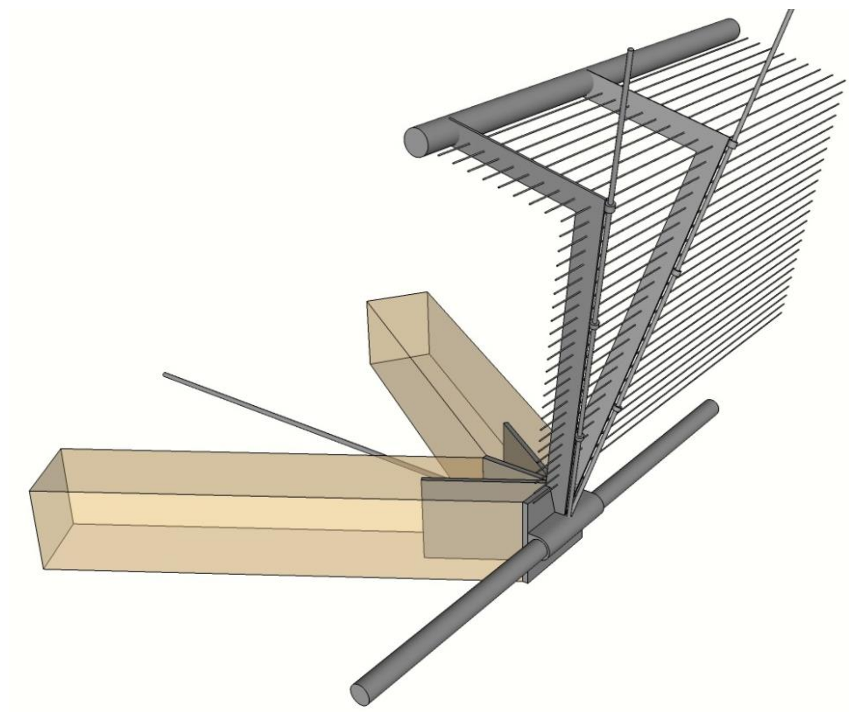


Figure 8.8 Perspective view of a potential cable connection detail

The suggested design above features a four plate welded connector that has attachments for the bottom secondary cables in between the slotted-in plates for the deck. On the outside of the plates the main cable can be connected in a module that also has attachments for the two secondary cables coming from the arch. The railing suggestion is based on trying to get a very slim system so that the light nature of the bridge is emphasised. Therefore two steel arms are welded to the plate connection in an reach up in line with the secondary cables. The arms then kink in perpendicular to this to support a handlebar at a height of 1,1 meters. In order to protect visitors from falling down a set of small steel wires are tensioned up through the railing arms to form a protective but see-through barrier.

The final steel detail is for the supports in the ground. Since the main cables of the bridge are tensioned this will put extra tensile capacity requirements on the foundation. However these demands can be reduced by making sure that the roof compression going down through the arches counteracts the cable tensile forces. This solution is the suggested setup shown in Figure 8.9.

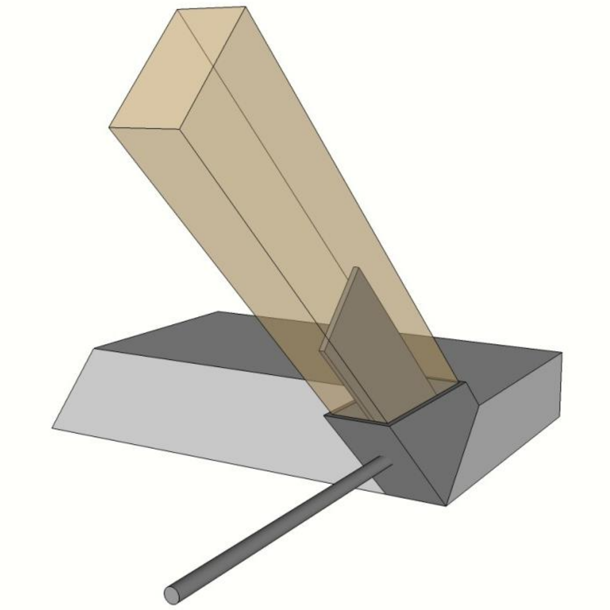


Figure 8.9 Perspective view of a potential support connection detail.

The suggested solution is mainly to ensure that the steel connection at the arch base, perhaps with a slotted-in plate as shown in Figure 8.9, also allows the main cables to attach at the centre. The steel module then still has to have a tension capable attachment to the foundation, but with less tensile force acting on it.

The final details are for the roof and deck setups. The roof only has requirements on cover and since the roof truss elements have been rotated to match the roof curvature, a set of LVL boards can easily be attached on top. These plates the only require a metal plate cover and a functional roof will be made. The deck has the crossed beams visible in the picture on the last page but to enable a layer of deck planks a secondary set of beams is needed. Figure 8.10 shows a suggested setup of this with one layer of 90x180 mm beams and then a perpendicular layer of 125x28 mm planks.

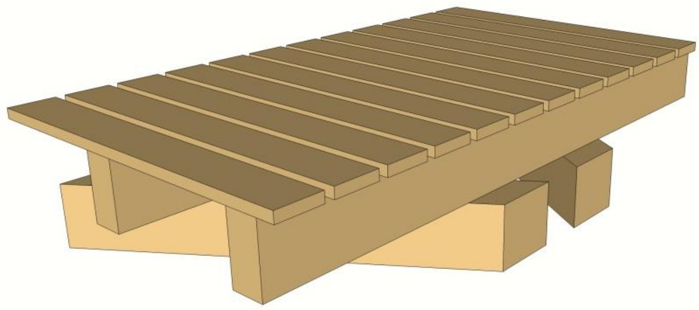


Figure 8.10 Perspective view of a suggested deck setup

8.3 The TUBE

Just as with the previous design the general modifications mentioned at the start of this chapter were made here as well, with some additions. The most apparent change was the removal of the deck cross beams. When checking the side wind load the bridge performed better with a rectangular deck beam setup, and since this also meant less elements it was natural to go with it.

The other modification made was the location of the tube truss elements. In the previous design version all truss members were based on the sections being placed with the centre in the geometry lines from Grasshopper. Upon further analysis it became clear that if the zig zag elements were offset outwards by about 35 mm the truss elements would line up much better with the outside roof and planned connection details. This has the downside of creating a slight eccentricity on the offset members compared to before, but after including this in the model it became clear that the strength reductions generated were very small.

The final element setup after all these modifications is shown in Figure 8.11 and Table 8.2

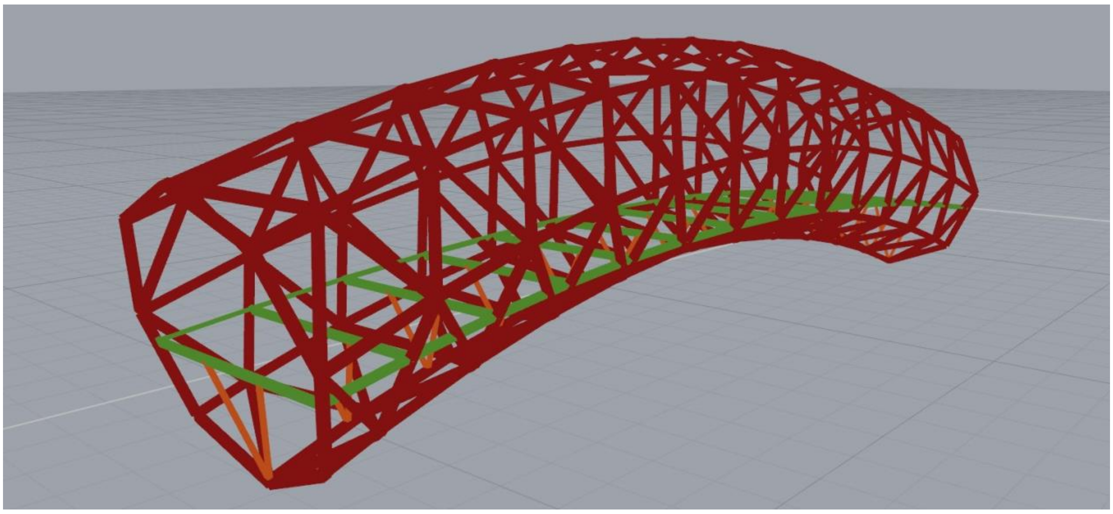


Figure 8.11 Perspective view of the TUBE concept final element dimensions

Table 8.2 Dimensions of the TC element groups, as shown in Figure 8.11

Set of elements	*Dimensions*
TUBE TRUSS	$165x180\ mm$
DECK BEAMS	$90x180\ mm$
DECK SUPPORTS	$90x90mm$

In the final design the support was also changed by removing the supported points in the deck. The bridge is now only supported on the two points in the middle of each tube end, thus activating the tube elements even more.

8.3.1 Final Analysis

With the final element setup specified the following utilization results, shown in Figure 8.12, were given:

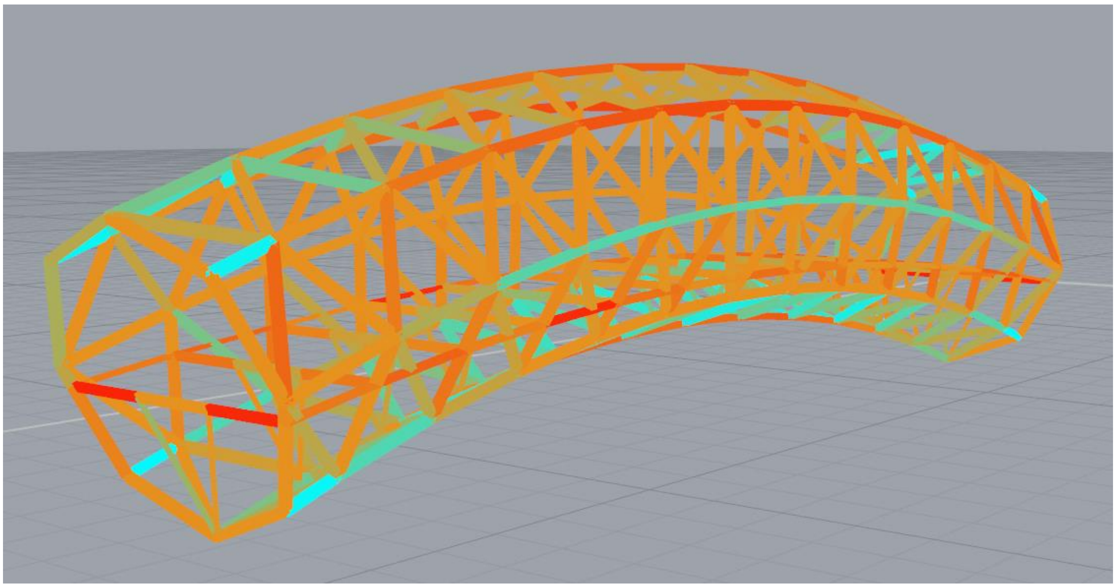


Figure 8.12 Perspective view of the final TUBE concept element utilisations

Compared to the previous analysis of the concept there is little change to the final utilizations. The strengthening due to better connections and standard adjusted dimensions are countered by the added wind loads and eccentricities as well as removed supports. This adds up to an efficiently utilized bridge in the ultimate state.

The serviceability limit check was also updated and the deformations in the final design are shown in Figure 8.13.

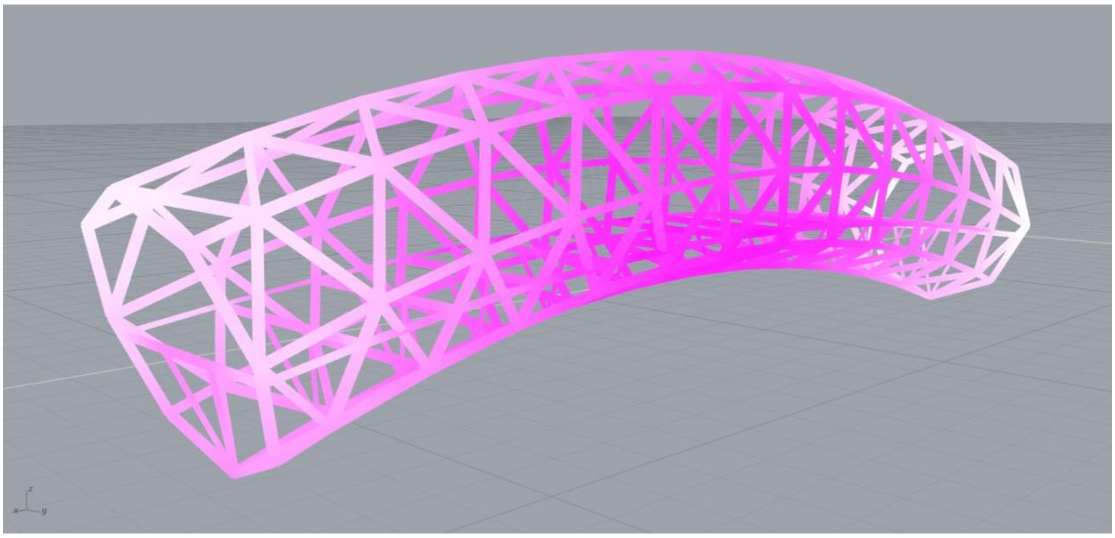


Figure 8.13 Perspective view of the final TUBE concept deformations

The results from the figure on the previous page are a deck maximum deflection of 61 mm and a roof maximum deflection of 48 mm, both of these within the limits of Eurocode.

The bridge dynamic response was also checked after the updates and just as before this design is very rigid. The first eigenmode was found at 6,24 Hz which is well over the Eurocode limit.

8.3.2 Connection Details

For this concept, just as with the last one, the assumed connections were designed to show suggestions on ways to achieve the desired behaviour.

One of the driving design features of the TUBE concept has been the repetitive systematic setup of the tube. Therefore the connections in the truss are a key component to follow this design in order for the bridge to feel uniform in its design scheme. The bridge also needs to include lighting and handlebars to be fully functional, and the goal was therefore to incorporate these in the truss connections. The result of this is the suggested steel connector shown in Figure 8.14.

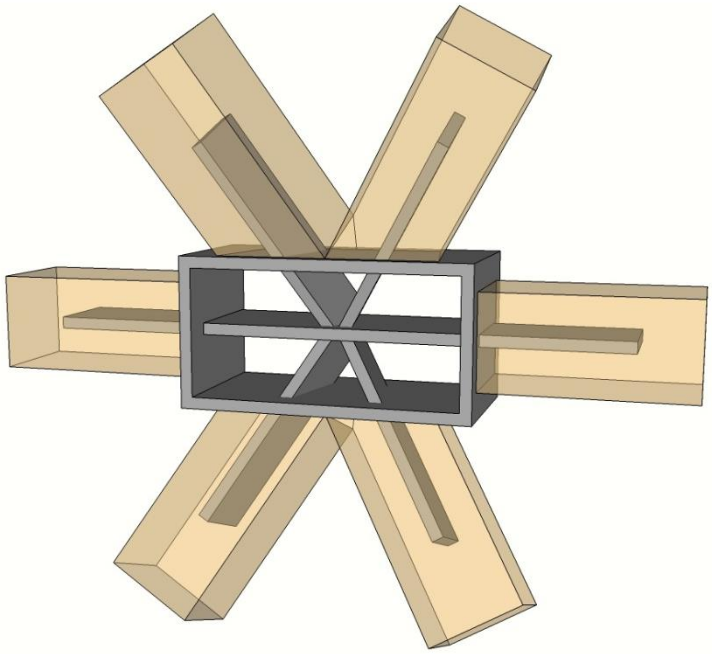


Figure 8.14 Perspective view of the suggested truss connection detail

In this detail, the truss is connected via slotted-in steel plated just as assumed in the analysis, but with the modification of a steel frame around the meeting. This frame makes the joint more stiff but also allows the connection to be hidden if covered by a lid. This is where the incorporation of the lighting and railing comes into the detail. By covering the frame in different ways different functions needed in the bridge can be included in the connection while still following the architectural design concept.

The following collection of pictures, shown in Figure 8.15, displays four different versions of this for four different areas of the bridge:

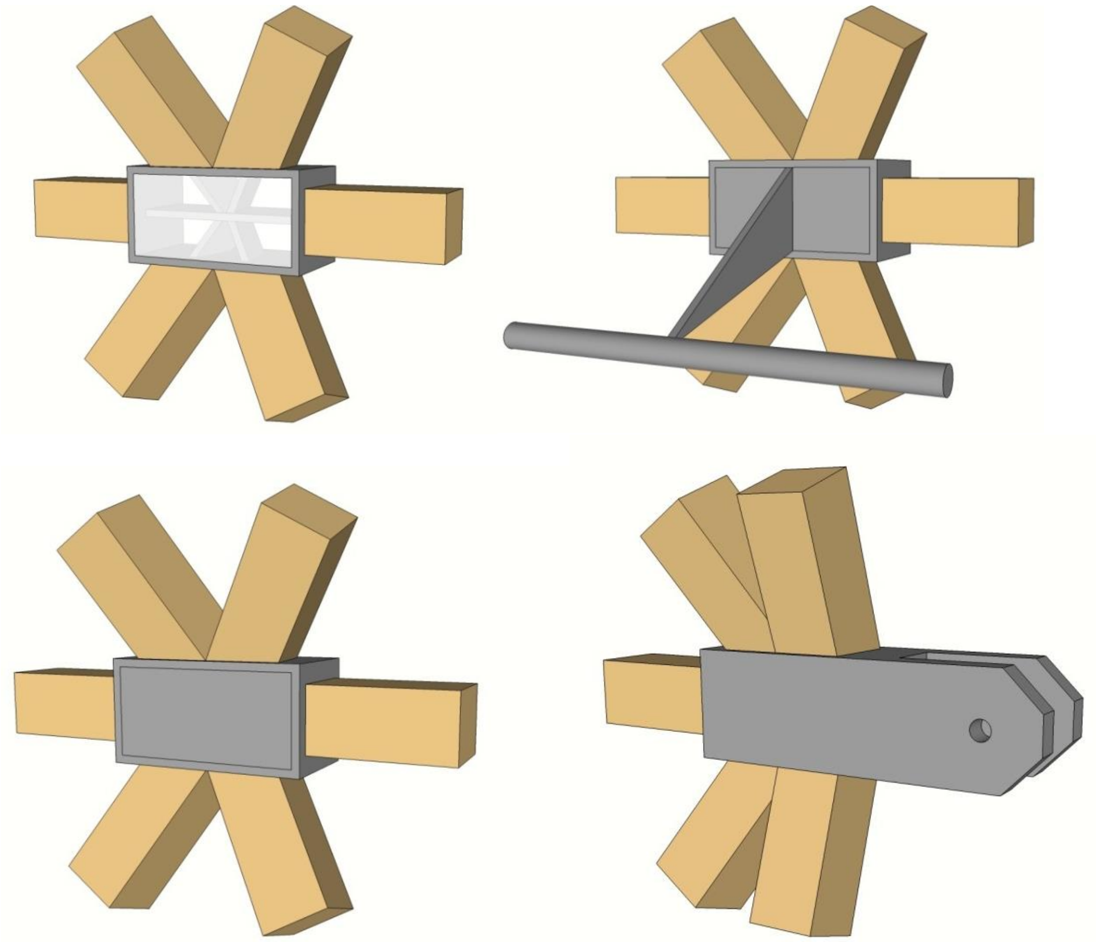


Figure 8.15 Perspective view of different truss connection detail versions

The different frame modifications include a lighting module, railing support, plain cover and foundation modification.

The lighting is based on the opening having lights placed inside and then a glass or plastic cover put on. This type of light will have the aesthetic benefit of being completely incorporated in the structural design. This module would be used in the three top rows of the tube, creating a repetitive light system along the bridge deck.

The railing module would be used in the two middle rows, where the handlebar can be supported on the connections with a welded plate. Since the curvature of the tube is greater than that of the deck the plates will be different along the bridge to ensure the handle bar is continuously located at 1 meter over the deck.

The three bottom rows would have the plain cover or no cover since they will be hidden. Finally, the four connections that also are supports would have a special stronger frame that extends out to moment free joints in the foundation. All in all, this yields a very uniform aesthetic to the bridge.

The remaining details are all hidden from sight in the bridge and are thus optimised for price and efficiency rather than including the aesthetic. The deck support beams for instance can be attached to the tube and deck beams with slotted-in plates as shown in Figure 8.16:

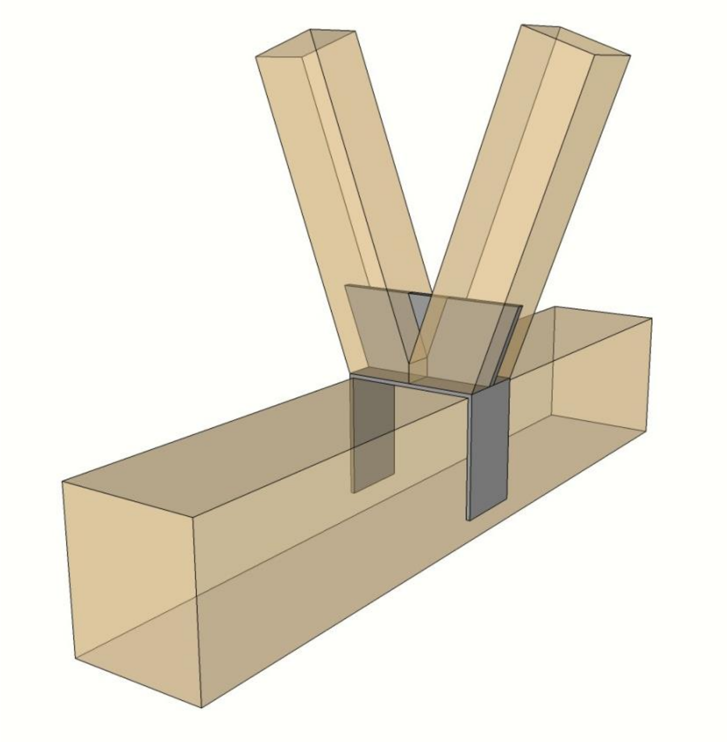


Figure 8.15 Perspective view of different truss connection detail versions

The connection showed in Figure 8.15 is easy to make and does not require preparations made to the tube truss member. If a similar detail is made for the upper connection of the support beams the desired behaviour is generated with only the small support members requiring cut slits to be made.

The roof and deck setups for the TUBE concept are suggested as the same as used for the previous bridge. The secondary beams of 90x180 mm can rest on the deck beams and the 125x28 mm planks can be put on top of them. The tube truss members have been rotated and offset as mentioned before, this is optimised for the roof cover as well. The tube has a planar outer perimeter formed by the truss that can be covered by LVL boards and the metal plating.

9 Final Bridges

With the two winning concepts optimised and having connection details, lights and railing designs suggested all that remains is a thorough look at the final result. The following chapter contains a set of rendered pictures of the two bridges showing what they would look like during day and night time.

9.1 The LEAF

Figure 8.16 Perspective day-time exterior rendered view of the LEAF bridge

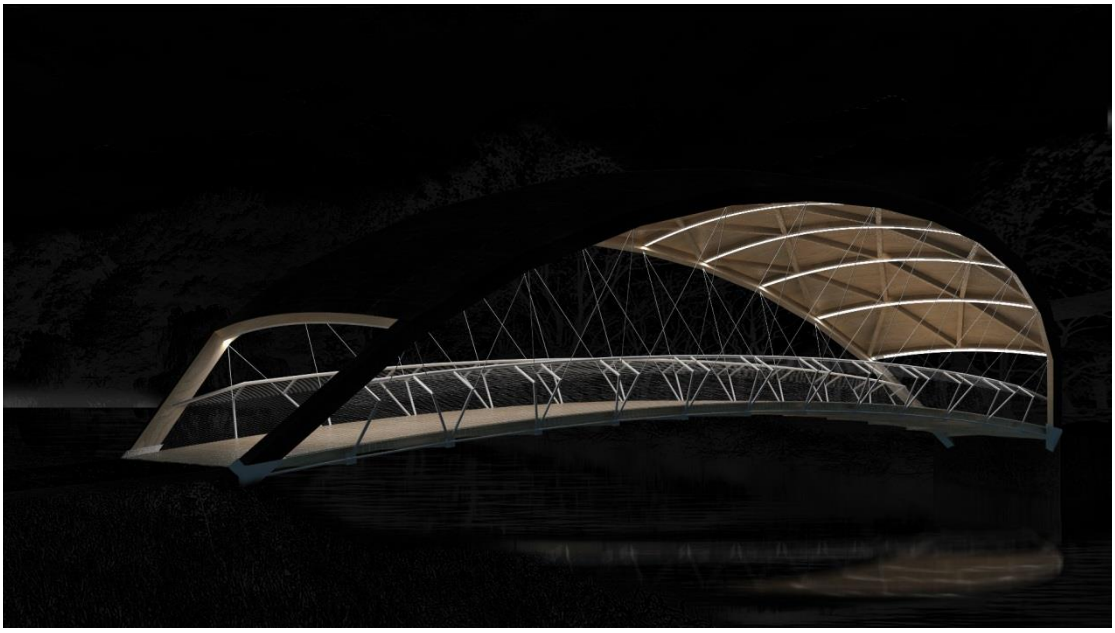


Figure 8.17 Perspective night-time exterior rendered view of the LEAF bridge

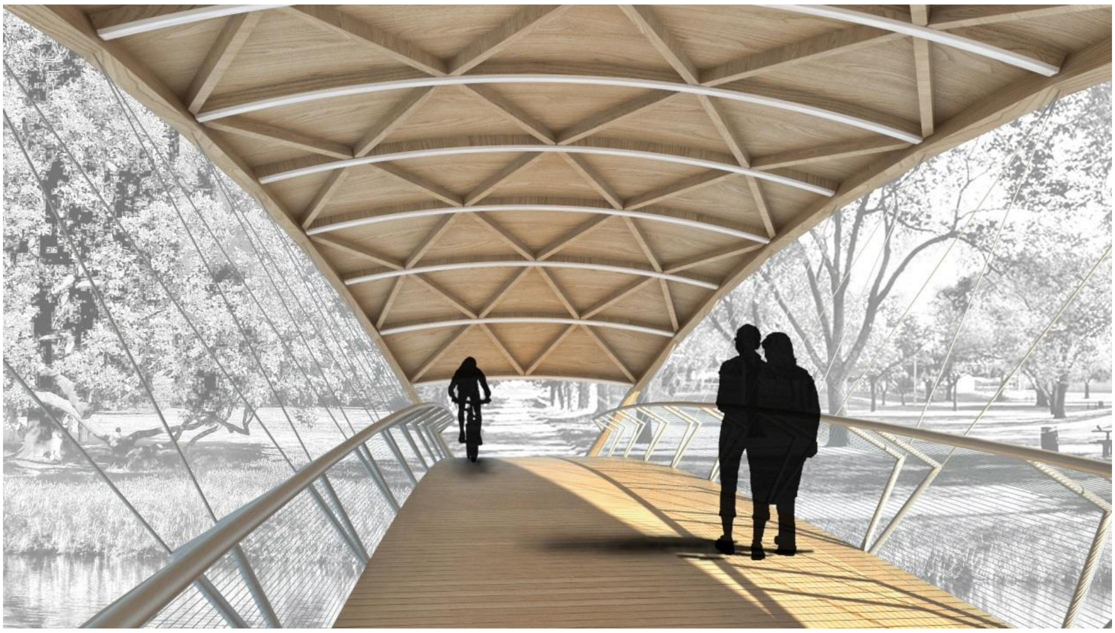


Figure 8.18 Perspective day-time interior rendered view of the LEAF bridge

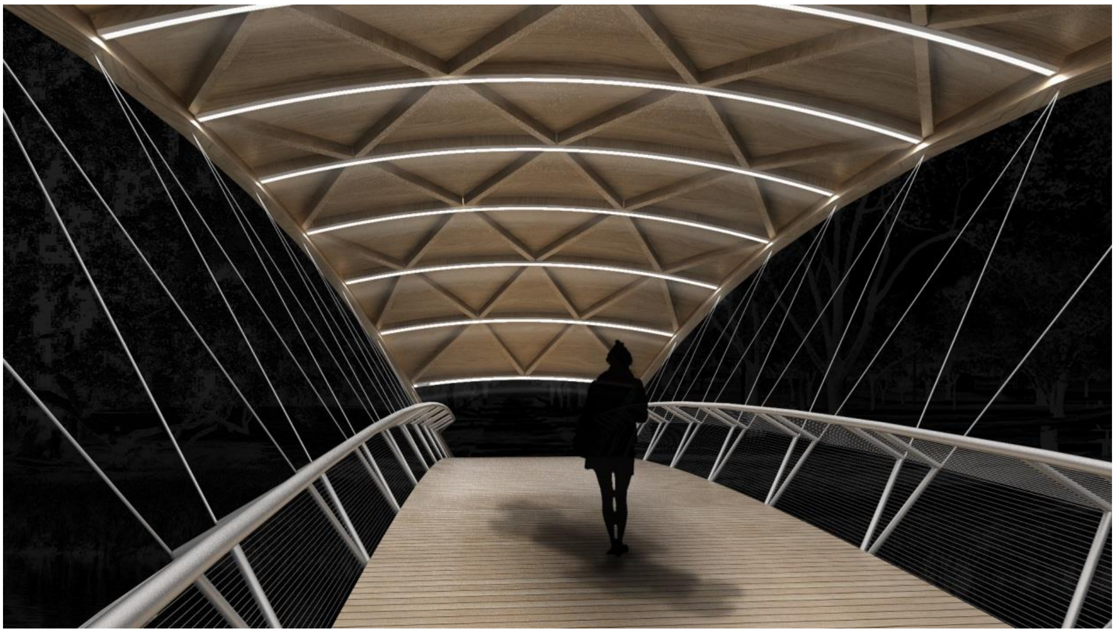


Figure 8.19 Perspective night-time interior rendered view of the LEAF bridge

As the four rendered images show the LEAF bridge open nature definitively comes across in the final design. This not only lets the surrounding area into the bridge space but gives a very light and material efficient feeling to the structure in general. The lighting strips in the ceiling blend in during the day-time giving the roof truss a clean and seamless feeling. In the night-time the light gives a uniform illumination to the bridge interior, which is contrasted by the dark exterior metal plate cover. All in all this comes together in a bridge that is both elegant and efficient, and with its unique design is has a lot of potential to work as a monument.

9.2 The TUBE

Figure 8.20 Perspective day-time exterior rendered view of the TUBE bridge

Figure 8.21 Perspective night-time exterior rendered view of the TUBE bridge

Figure 8.22 Perspective day-time exterior rendered view of the TUBE bridge

Figure 8.23 Perspective night-time exterior rendered view of the TUBE bridge

The TUBE final bridge, as the images show, has a clear contrast between the interior and exterior in as well complexity, colours and light. These strong differences in aesthetics give the visitors a dynamic experience when using the bridge and emphasises the design inside the TUBE. The pictures also show the way the dynamic light during daytime changes over to the uniform incorporated lighting during the night. The enclosed nature of the TUBE avoids the risk of visitors feeling insecure to enter at night through the strong illumination numerous windows along the span. In total this gives a bridge that is very unique and monumental, that allows to visually take in the surrounding while still isolating the interior from noises and weather and that has a strong interior design of incorporated lighting and railings.

10 Conclusion

10.1 Project Aim Questions

PHASE 1 - INSPIRATION

- ***What types of timber bridges (mainly covered) have been built before? With what aesthetic profile?***
  The literature study came across a variety of different covered bridges built around the world, showing a great variation in aesthetics all the way from the decorative Southeast Asian ones to the efficient approach of North America.

- ***What different construction principals have been used for timber bridges in the past? What are their benefits and disadvantages?***
  Many different bridge types were compared in the literature study, showing both benefits and disadvantages from using certain structural principals.

- ***What design problems need solving for a covered timber bridge compared to an open one? What are some possible solutions?***
  The comparison between covered and open timber bridges was looked at and while many of the issues for open bridges are the same as for closed ones, it is also possible to avoid some problems by adding the roof. In the analysis the durability issue was mentioned but not supported by calculations, this is the most obvious benefit of a roof cover. In addition the structurally incorporated roof improved the dynamic behaviour which otherwise often proves critical for timber bridges. Other benefits include not having to account for snow on the deck which requires snow removal vehicles to cross the bridge, this means both less calculations and demands for the bridge design.

- ***In what ways can the roof and walls contribute to the bridge functionality other than for climate protection?***
  The early investigations included some methods for incorporating the roof and wall structure to improve the structural performance of the bridges. In most of these bridges the roof was mainly carrying horizontal loads while the walls carried the main vertical loads.

- ***Why is there not a single public covered timber bridge in Sweden****?*
  The reason for the lack of covered timber bridges in Sweden is hard to narrow down to a few simple factors. But in general, it seems that the approach chosen in the country is rather small scaled constructive protection, through for instance panels and steel plates, rather than roofs. I think that this protection method above has been used for a long time leading to a both safe and optimised method of utilizing it. Incorporating a new approach such as the roof cover will cost some extra money at first and requires somebody to take that leap. Therefore I think the fear of an expensive and problematic first attempt the solution of a roof cover, which this Thesis suggests is a great solution, is avoided for a more conventional solution more commonly used in Sweden.

PHASE 2 – CONCEPTUAL DESIGN

- ***How can the roof and walls of the bridge be used as structural members?***
  Building on the information from the literature study many different solutions for using the roof and walls as structural members were tested. In difference to the built examples that were studied, several concepts used the roof as a vital structural component, instead of using it mainly for horizontal stability.

- ***In what ways can the design be expressed through the structural components?***
  In all of the posed concepts for covered bridges the structural components were used as an important factor for the aesthetic design. The Thesis shows that if desired, the structural system can be both included and essential for a pleasing final appearance.

PHASE 3 – FINAL DESIGN

- ***What is required in terms of detailing and dimensions for the bridge?***
  Commenting on the requirements for detailing and elements will be somewhat supported by this analysis but for a completely accurate answer a more thorough analysis is needed. The required element dimensions as described in Chapter 8, show that depending on the chosen structural system the element sizes vary greatly. The shell type structure of the TUBE yields small dimensions but many members while the strong arch has larger members but in fewer numbers. In addition the requirements on details as the report shows, depend much on the assumptions made in the calculations as well as desired aesthetics. With this in mind the thesis also shows that the details can be made into another beneficial aesthetic component if considered in the architectural design process.

- ***What is needed from the cover for the bridge to protect the structural system from weather effects?***
  What the roof cover needs to protect the structural system has not been investigated extensively but some guiding factors were looked at. The limit used in the thesis for rain angle based on Swedish standards was 30 degrees, apart from this the bridge roofs were designed mainly on the basis of architecture and structural behaviour. Looking at the two winning concepts it is clear that the LEAF concept will be more exposed, especially at the entrances, but for most types of weather the roof would still cover the bridge. The TUBE on the other hand is very protected and only strong winds towards the entrances or moisture stuck to footwear could potentially get in to the bridge components.

- ***How can the final solutions be optimised regarding the economical and required production time and effort aspects?***
  Both of the final designs went through many stages of economical and production optimizations. In general the assumptions were that many identical members or few members in total was beneficial financially. The first of these approaches was used for the TUBE which has very many identical members whereas the LEAF instead is very material efficient and uses little material. Both bridge designs were also modified to include only standard dimensions.

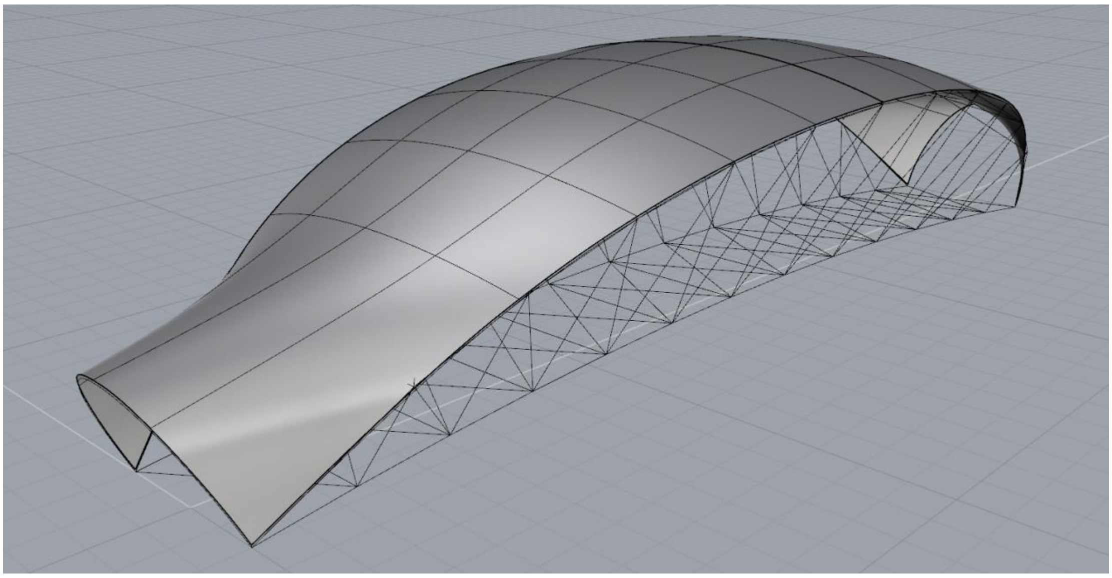
- ***Can an aesthetically pleasing and fully functional covered timber pedestrian and bicycle bridge be built, having reasonable requirements from the construction workers while being sufficiently optimised to be economically feasible?***
  Answering the main question of this Thesis based on the work done, much points towards yes. However for a fully supported answer to be made further analysis is needed. The goals of aesthetics, functionality and optimization have all been included well in the Thesis but the still missing information regard the economic aspects. While several steps towards economical optimization were made, such as the previous point, no actual sums can be said based on this work. Therefore for the driving question of this thesis there seems to be little arguments against covered timber bridges being a great solution.

10.2 Further Work

The structural design carried out in Karamba could, if checked prove to be completely accurate, but the main plan for this Thesis was to perform a simplified global analysis of different solutions. For a finished design to be ready for construction, a more thorough analysis is therefore needed.

An analysis that builds on this work will be able to benefit greatly from the already performed work. The structural models for the global geometry are finished and while the structural analyses will need to be performed again this Thesis produced a good estimation on the required dimensions. The two main analysis steps that have to be added to the work here are a thorough dynamic analysis and calculations on details.

For the connection detail calculations that are completely left out here, there is a lot of information suggested in the Thesis. The aesthetics of connections and details are a key component in architectural design and a common issue is when this was not included in the design phase but rather added later without regard to appearances. This Thesis therefore has a useful summary on the design of details that can help a final bridge aesthetic with a clear theme.



10.3 KARAMBA

Since this was a first use of the FEM-analysis plugin Karamba for me and it is a new program, some conclusions about it are relevant. The general impression I get of Karamba is that it is a fantastic program, especially for the type of work carried out in this project. When working in Grasshopper and Rhinoceros it is cumbersome to jump to other programs for analysis, especially in stages where the design is constantly changing and recalculations are needed. There are some plugins to Grasshopper that specialize in connecting to other programs but the by far simplest solution is to perform the analysis in the program.

On the main plus sides are the immense convenience of doing everything in one program. The process of going from line and point geometry in Grasshopper to a FEM-model in Karamba is very simple after some basic training in the program. Karamba also handles all of the relevant analyses for a general design such as performed here.

The main downsides to the program are what makes me feel Karamba is optimum for this type of analysis and not a final analysis. The results from Karamba from the static analysis of the timber members might very well be just as good as in other more complex programs, but it is hard to include steel connections and other details in a good way. The effects of the details can be incorporated, but for a full analysis they need to be checked as well, which will require another program which might as well have been used for the rest too then.

But in general my conclusion is that for conceptual design that include some structural analysis this is an immensely convenient tool to use.

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11.3 Images

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