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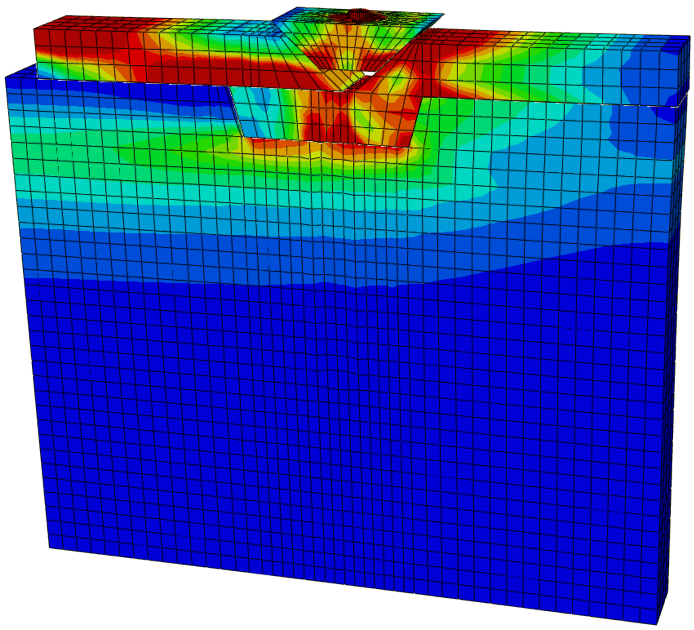

A new connection concept for use in prefabricated timber-concrete composite bridges

Investigation and numerical analysis of the connection focusing on its geometry, stiffness and behaviour

Master's thesis in Structural Engineering and Building Technology

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Master's thesis ACEX30-18-XX
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Cover:
Deformed connection at a prescribed displacement of 4 mm.
Colours showing von Mises stresses.

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Abstract

During the recent years, timber-concrete composite systems (TCC) have become more and more used as a new construction technique for buildings and bridges. Along Göta älv in Gothenburg, Sweden, a quay were constructed using this technique showing satisfying results over time. An idea arose concerning improvement of the construction, to use prefabricated concrete slabs for the bridge deck, instead of cast in-situ. Using prefabricated elements has the possibility to increase the interest in TCC bridges even more, due to faster erection time and possibilities for demounting. To use a prefabricated deck brings further challenges since more elements are to be connected with a sufficient stiffness for composite action of the bridge. The connection of these elements is the most critical part of the bridge and will be further investigated in this study.

Consequently, this study has the purpose of investigating and developing such a connection concept, as a pre-study of a larger research project concerning industrial manufacturing of TCC bridges. After performing a literature study where different previously made connections were investigated, a possible connection concept was elaborated and important parameters affecting the behaviour, was identified for further analysis. A desire was to find a connection that enabled a ductile behaviour, and at the same time provided a sufficient stiffness.

In order to investigate the connection, a model were created in the FE software Abaqus. Two analyses with different geometry were performed, showing the behaviour of the connection together with results of maximum stresses and the total stiffness achieved. For each analysis four iterations were made with the aim to improve the connection. The first of the two analyzes showed the best results, with the desired behaviour fulfilled, proving that the concept proposed were possible, with a stiffness of 100 kN/mm. Though, the stiffness value achieved were complicated to compare with the previous research in the literature study due to varied conditions. To confirm the sufficiency of the stiffness obtained, a global analysis needs to be made and is suggested for future research. Additionally, to validate the results of the connection laboratory shear tests needs to be performed.

To conclude, a connection between the elements in a TCC with prefabricated concrete deck had been proved possible. Yet, limitations of the study makes further research necessary before the connection can be implemented in a real bridge.

Keywords: timber-concrete composite bridges, TCC bridges, prefabricated, connections, FE analysis, Abaqus, stiffness

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PREFACE

Nomenclature

Greek letters

$\alpha_{\mathbf{e}}$	Expansion coefficient ($\frac{1}{K}$)
α	Angle (°)
δ	Slip (mm)
ρ	Density (kN/m^3)
σ	Second order stress tensor (MPa)
ε	Second order strain tensor (-)
ν	Poison's ratio
ν_{LR}	Poison's ratio, longitudinal-radial
ν_{LT}	Poison's ratio, longitudinal-tangential
ν_{RT}	Poison's ratio, radial-tangential
F	Reaction force (kN)
V	Shear force (kN)

Roman lower case letters

σ_{c90}	Compression stress ⊥ to the grain (MPa)
σ_c	Compression stress ‖ to the grain (MPa)
σ_{t90}	Tension stress ⊥ to the grain (MPa)
σ_t	Tension stress ‖ to the grain (MPa)
E_L	Youngs modulus, longitudinal (GPa)
E_R	Youngs modulus, radial (GPa)
E_T	Youngs modulus, tangential (GPa)
f_{c90d}	Dimensioning compression capacity ⊥ the grain (MPa)
f_{c90k}	Characteristic compression capacity ⊥ the grain (MPa)
f_{cd}	Dimensioning compression capacity ‖ the grain (MPa)
f_{ck}	Characteristic compression capacity ‖ the grain (MPa)
f_{t90d}	Dimensioning tension capacity ⊥ the grain (MPa)
f_{t90k}	Characteristic tension capacity ⊥ the grain (MPa)
f_{td}	Dimensioning tension capacity ‖ the grain (MPa)
f_{tk}	Characteristic tension capacity ‖ the grain (MPa)
G_{LR}	Shear module, longitudinal-radial (Pa)
G_{LT}	Shear module, longitudinal-tangential (Pa)
G_{RT}	Shear module, radial-tangential (Pa)
k	Stiffness (N/mm)
k_i	Slip modulus

Miscellaneous

‖	Parallel
⊥	Perpendicular

Roman capital letters

$\mathbf{E}$	Fourth order stiffness tensor (MPa)
A	Surface area (m^2)
E	Young's modulus (MPa)
I	Moment of inertia (m^4)
V	Volume (m^3)

1 Introduction

This study has the purpose to investigate and develop a connection between a prefabricated concrete deck and glue laminated timber beams. The project intends to be a forestudy of a larger project concerning a new concept of industrial manufacturing of timber-concrete composite (TCC) bridges. The following chapter presents the background, problem description, general aim, objectives, limitations and methods for the thesis.

1.1 Background

The idea of this project arose after an earlier construction of a quay located along Göta river in Gothenburg, Sweden. There was a request from the Port of Gothenburg to use other materials than steel in the construction of the quay, since there had been problems with corrosion found in many of the constructions along the river. This finally ended up in a timber bridge with a cast in-situ concrete deck on top, a so called timber-concrete composite (TCC) bridge, which have proven good results over time (Lundberg, 2018). The ambition of this study is to develop the concept further and to improve the results, mainly by changing the cast in-situ concrete deck to prefabricated elements. The elements will be reinforced with basalt fibre reinforced polymer bars (BFRP bars) giving the concrete elastic properties and the ability to avoid cracks, which ensures that the timber beams will be protected from moisture induced problems (Andersson & Brahme, 2017). With a prefabricated deck a lot of other benefits could be earned, as faster construction time (D. E. C. Yeoh, 2010), and possibilities to demount the bridge. However, the change results in challenges regarding the connections between the timber beams and the concrete decks, since more elements are involved. The connections needs to ensure transfer of shear forces between the two different materials, giving the structure its composite action.

Manojlović and Kočetov-Mišulić, 2016 stated, "One of the basic principles in modern and optimal structural design is to use an adequate construction material at the appropriate position in the structure". Based on this principle, TCC systems have during the recent years become more and more used as a new construction technique for buildings and bridges. The idea of TCC is that the concrete bears the compression forces in the structure, while tension and bending are resisted by the timber (D. Yeoh, Fragiacomo, & Carradine, 2013). This brings the superiority of the materials into full play (He et al., 2016). The concrete deck also provides protection for the timber beams, which reduces the risk of rotting and increases the service life of these elements up to twice that of a bridge only made of timber. (D. Yeoh et al., 2013)

If good solutions could be found regarding the connections between timber and concrete in composite bridges it can lead to, besides the short-term profits, a more sustainable development of bridge construction. As society today aims to be more sustainable in the long term, increased use of wood means the possibility of reducing our use of finite raw materials and reduce the carbon dioxide emissions from construction products. Building with timber has a strong tradition in Sweden and is now on the rise. Looking at its advantageous properties, together with the fact that Sweden has a growth of the forest larger than the harvest , it may seem to be the obvious to start build more in timber (SwedishWood, no date-b). However, some structures are not optimal built with timber alone. Instead, TCC systems is a compromising way of increasing the use of timber in construction (Velimirovic, Stojić, Djordjević, & Topličić-Ćurčić, 2017).

1.2 Problem description

The demand of the society regarding shorter in-situ time during erection is an increasing matter, especially in the cities. This as the building site is limited and other objects is often in production nearby, making the logistics more complicated. Traffic shutdowns are expensive, and to be competitive on the market, another solution is needed where the whole structure is prefabricated. That would shorten the construction time on site remarkably. Industrial manufacturing also enables higher quality, which makes it possible to keep the costs down. Thus, the challenge of connecting prefabricated elements and gaining a satisfying stiffness to reach continuity arises, which will be further investigated in this report.

An increasing requirement from the industry is at the same time to minimize the impact of the environment, and this therefore has to be taken into account more seriously in the production and at choice of building materials. Light prefabricated structures, as concrete with high capacity or timber, decrease the environmental impact both by lower material consumption and less green gas emissions due to transports.

In this report the possibility of using prefabricated concrete slabs and timber beams in a TCC bridge is investigated. If the materials could be used in composition for optimization, to exclaim the best properties of both materials, that will decrease the environmental impact and shorten the construction time.

1.3 General aim

The purpose of this study is to develop and investigate a connection concept between a prefabricated concrete deck and glue laminated timber beams. Numerical analyses and investigations will be performed to create a deeper understanding of the behaviour in such a connection, regarding its geometry, stiffness and behaviour. The development should aim at creating a connection that enables a satisfying stiffness together with a desired structural behaviour.

1.4 Objectives

This study is supposed to be a pre-study of a larger project that concerns a new concept of industrial manufacturing of composite concrete-timber bridges. To reach the aim; i.e. to develop a connection between composite elements, the following objectives were set:

- Investigate different solutions of connections from previous studies, regarding its stiffness and functionality.
- Present a possible concept for a connection with the intention to meet the requirements regarding behaviour and stiffness.
- Perform a numerical analysis of suggested connection in the FE software Abaqus, by investigating its behaviour, maximum stresses and stiffness.
- Iterate another numerical analysis, by evaluating results from the first numerical analysis.
- Evaluate results and come up with suggestions and improvements for further research and for shear tests in laboratory.

1.5 Limitations

To attain a desired depth in this project and keep focus onto the actual connection detail in FE modelling, a couple of limitations had to be made.

The long-term behaviour of a TCC composite system is complicated and depends on several aspects, among them is creep and shrinkage. Most of the shrinkage of the prefabricated deck has occurred before mounting at site and was neglected in analyses. Temperature changes has a relatively small impact when studying local problems as a connection and was not taken into account. In the calculations the connection was considered to be impenetrable and problems regarding moisture was not considered to be an issue, due to the use of basalt fibre reinforced polymer.

It is important to mention that friction was taken into account in the FE model of the connection. This affected the results since it increased the reaction forces. However, when a slip occurs between two materials, the friction force can, if large enough, destroy the weakest material locally on the surface. This fact was not considered in the analyses. If real tests would be carried out in the future showing this would be critical, methods for reinforcing the weak surfaces exist.

The study will focus on analysis of the connection at a local level. How the connection works in a bridge globally and how it affects the composite action of the structure will be left for future research.

Further, optimization of the connection is not an aim of the study, but instead to prove the new connection concept to be manageable or not, and thereafter perform an iteration. As the study aims to develop a connection in the FE software Abaqus, the results needs to be verified by real shear tests in laboratory.

1.6 Method

Initially a literature study was performed to increase general knowledge about timber-concrete composite bridges and investigate connections from previous research regarding its stiffness and functionality. A study of the composite action as a phenomena, leading to the local stiffness of a connection, were carried out as well. This to create a deeper understanding of the connections and its importance in a structure. Further, the reference project were studied, and based on that together with the outcome of the literature study, a proposed connection were presented.

The proposed connection was thereafter converted to a representative FE model in Abaqus for numerical analysis. Input values were set to be as corresponding as possible to the real connection. The first simulation was performed with the intention to sort out parameters affecting the results.

Further, the FE analyses was divided into two different analyses, consisting of models with different geometry. For each analysis four tests was performed, where previously found parameters was changed after each test to find the optimal parameter values presuming the chosen geometry.

After accomplished analyses, input values for the best tested connection was presented and discussed, proposing further investigations and suggestions of improvements.

2 Timber-concrete composite (TCC) bridges

This chapter presents a general introduction to timber-concrete composite (TCC) systems followed by a brief description of the different properties of timber versus concrete and how the materials can be used together to exploit their best qualities. The advantages of using prefabricated concrete instead of cast in-situ concrete in composite structures is also presented, and how the use of basalt as reinforcement can be advantageous in many ways. The phenomenon of composite action is described, followed by the importance of stiffness in a connection and different shear tests.

2.1 The history of composite bridges

The idea of composite bridges is not a new thing. During 1938 and 1942 the first full scale bending tests of composite systems were performed at the University of Illinois in USA, and even before that floors of TCC structures had been used in highway bridges, piers and buildings. The tests performed in Illinois investigated different shear connections. Most of them did not fulfill desired strength and stiffness, but some connections were found having high load carrying capacity, good integral beam action, small slip and deflections, and therefore TCC structures were found interesting. (Richart & Williams, 1943)

Other tests that was also carried out during this time have been known as the Oregon tests. These tests were requested from the Oregon State Highway to find a cost-effective short-span highway bridge. A study of the test were published in 1943 by McCullogh concluding that the ultimate strength of a composite beam is twice that of the same elements without connection, and the deflection less than 25 %. Based on the result, more than 180 composite bridges were constructed. (Lukaszewska, 2009 and references within).

In Europe there have historically been a strong tradition of building with timber since generous natural reserves could be found. Besides that, the use of timber in bridges became increasingly unusual in the 19th century and had to give ways for steel and concrete (Jutila & Salokangas, 2010). It was first after the two world wars the development of TCC started, as a result of shortage of steel for reinforcement in concrete (D. Yeoh, Fragiacomo, & Deam, 2011).

Originally the use of TCC had been a technique for renovation work and strengthening of old timber floors in historical buildings around Europe (Holschemacher, Klotz, & Weibe, 2002). One of the different methods that is still widely adopted is to connect a concrete slab to an existing timber floor (Riggio, Tomasi, and Piazza, 2014 and references within). This technique makes it possible for buildings designed at earlier times to meet new regulations, especially regarding sound insulation and fire resistance between floors (D. Yeoh et al., 2011). After reparation the floors receive a behaviour that is very similar to a composite beam. (Riggio et al., 2014).

In the 1980's timber construction got its real revival in Europe and, even though most of the development was made for buildings rather than bridges (Jutila & Salokangas, 2010), the interests of TCC systems started to rise. During the last 20 years a trend of building bridges in wood as well has started and is rapidly increasing (SwedishWood, no date-a). This mainly due to development of various timber components and new advanced timber products such as glulam, LVL, CLT, new types of connectors and connections and new methods for wood protection. (Crocetti, Sartori, & Tomasi, 2014).

2.2 Including timber in bridge construction

In a time when the world is facing a lot of environmental challenges, timber has been especially noted for being a versatile raw material and the only renewable construction material. The material is eco friendly in many aspects; the by-products from production could be used to generate energy, the production process creates minimum waste and the material stores carbon dioxide throughout its lifetime. When demolating the structure the material can be used as biofuel to replace fossil fuels. (SwedishWood, no date-b)

From a structural point of view, the major motivation of replacing concrete and steel with timber, is due to its high strength-to-weight-ratio. The fact that components could be prefabricated enables fast construction at site, where no or just a few hours closure of the traffic is needed. This makes a timber bridge a suitable option in many scenarios. The low weight of timber also benefits the groundwork and the transport of the components. (SwedishWood, no date-a)

Timber bridges are especially ideal for pedestrian and cycle traffic where the loads are small-scale, but the bridge could also be built to carry a full traffic load in line with the prevailing standard. Bridges made of timber tolerate aggressive environments and retains their properties over a long time if load bearing parts are properly protected from moisture, since it can affect the strength and the stiffness properties of the components (SwedishWood, no date-a). The weakest part of a timber bridge is usually found as the durability of the bridge deck (Jutila & Salokangas, 2010). Some disadvantages concerning timber as building material for bridges is poor shock resistance and risk of fire (Crocetti et al., 2014), which needs to be considered in the design.

2.3 The optimal concrete for TCC

Concrete have been used in different forms since as early as 500 BC, but the breakthrough of the material came during the 19th century when reinforcement of the concrete was invented (Hildago, no date). Today the material is used everywhere around the world; in buildings as well as in tunnels, bridges and roads. The most typical characteristic for hardened concrete is that the compressive strength is very high, but another great advantage of the material is that it can be customized for different uses of it. By changing the water amount and using different admixtures, specific wanted properties can be received. (Engström, Al-Emrani, Johansson, & Johansson, 2011)

In contrast to its high compressive strength, the tensile strength of concrete is very low. When subjected to tensile stresses it will crack, which is why structures made of concrete is normally provided with reinforcement. In most cases the reinforcement consists of steel bars. However, one disadvantage of using steel bars is that micro cracks in the concrete is developed, allowing moisture penetration which causes risk for corrosion of the bars. (Engström et al., 2011). On the market today there are a couple of alternative reinforcement materials to be found, that can provide resistance to corrosion. Some reinforcement consists of fibres that are mixed in the fresh concrete and gets randomly distributed, instead of placed in lines as steel bars, which enables a uniform behaviour of the concrete. The small bars also transfer the forces in a way that micro cracking is avoided, see Figure 2.1. One of these fibre reinforcements is Basalt fibre reinforced polymer bars (BFRP bars) that is presented more detailed in Subsection 2.3.1. One of the purposes with the concrete in the TCC system, as mentioned in Section 2.2, is to protect the underlying timber beam from moisture, which is another reason to strive for an impenetrable concrete.

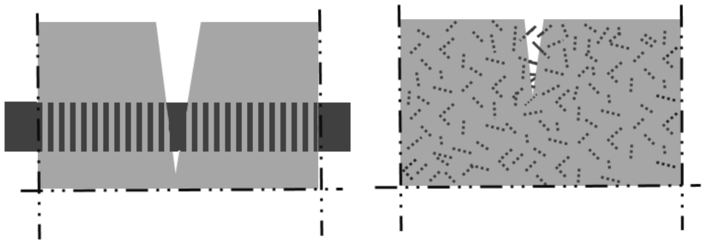


Figure 2.1: Different methods for reinforcing of concrete; with steel bar (left) and with fibres (right)

2.3.1 Basalt fibre reinforced polymer bars

A new technology that have been developed for fibre reinforcement of concrete structures is basalt fibre reinforced polymer composite bars (BFRP bars). The bars are an alternative to the more commonly used steel and fiberglass fibre reinforcement. As the name reveals, the fibres are made of basalt and is transformed to to a pre-reinforced concrete by applying the bars into the concrete (see Figure 2.2) The BFRP bars are capable of increasing the tensile strength of concrete and create a high average residual strength (post cracking residual strength). This enables slimmer structures and by that less carbon dioxide footprint. (Patnaik, Miller, Adhikari, & Cato Standal, 2017).

Basalt is naturally resistant to alkali, rust and acids, which eliminates risk of corrosion. Therefore BFRP bars is especially convenient for marine environments and chemical plants where corrosion is a common concern. When using BFRP bars a so called pre-fabricated concrete is obtained, which also is advantageous since the request for less demanding reinforcement solutions is increasing. (Patnaik et al., 2017)

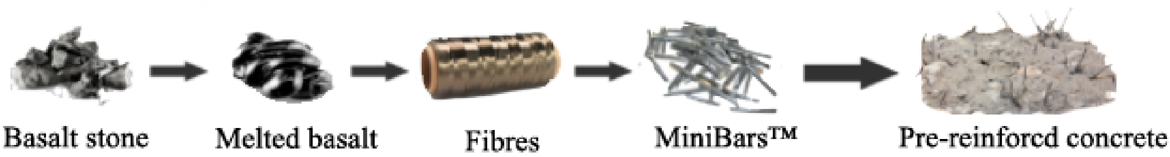


Figure 2.2: The five steps from basalt stone to structural pre-reinforced concrete (Patnaik, Miller, Adhikari, & Cato Standal, 2017).

2.3.2 Prefabricated versus cast in-situ concrete

Concrete bridges in Sweden are traditionally cast in-situ and prefabricated elements are quite rare. However, there are many examples of prefabricated bridges on the international field. (Harryson, 2008)

The bridges built cast in-situ have performed well regarding static and dynamic aspects, but some disadvantages concerning other aspects could be avoided if using prefabricated concrete deck instead. First, a lot of economical benefits could be obtained. As mentioned before, prefabricated components can be assembled rapidly on site minimizing traffic closure. The cast in-situ concrete also adds extra time required on site since it needs to be cured before next scheduled action can start. The curing takes 28 days and during this time the stiffness of the concrete will be low and the creep high. By using prefabricated elements instead this could be avoided. Prefabricated concrete is also more cost effective since cast in-situ results in a lots of expenditures, as fresh concrete, use of formwork and labor during casting. The expenditure on labour, including preparation and dismantling of formwork, reinforcing, casting and finishing of concrete, is estimated to as much as 40% of the budget (Löfgren, Gylltoft, & Kutti, 2001)

Prefabricated elements eliminates the risk of applying a wet material (fresh concrete) on the timber beams, which is preferably avoided (Crocetti et al., 2014). During the recent years there have been a focus on deconstructable composite structures, not only because they are more easily assembled, but also because the dismantling process facilitates the recycling and reuse in the end of the life cycle of the structure which reduces the ecological footprint of the structure (Khorsandnia, Valipour, & Bradford, 2018).

When using prefabricated concrete, the first shrinkage and hardening occurs inside the factory where the environment can be controlled. The temperature can be kept constant, which reduces the risk of cracking and also eliminates the risk of early freeze thaw phenomena. Restraint forces will be reduced, as most shrinkage occurs before connecting into a composite bridge. Before the elements leave the factory quality controls can be made, including check of concrete cover, surface and geometry. As a result there is a possibility to guarantee high quality before reaching the building site. Therefore concrete of higher quality can be used resulting in less material needed (Stoltz, 2001).

2.4 The combination of timber and concrete

As mentioned in Section 2.2 the weakest part of timber bridges is usually the durability of the deck. By replacing the deck with concrete slabs, this weakness could be avoided. The concrete will, besides protecting the timber beams from the weather, also give the structure good qualities in aspects where the timber lacks; it will increase the bearing capacity and stiffness, reduce the deformation and the vibrations, improve the fire resistance and sound insulation and upgrade the seismic performance by improved diaphragm action (Manojlović & Kočetov-Mišulić, 2016).

2.4.1 Composite action

To create an effective TCC bridge the degree of *composite action* needs to be as high as possible. The composite action depends on the horizontal displacement (slip) that can occur in the interface between the timber beam and the concrete slab when subjected to loading. The lower the slip, the higher efficiency of the composite structure. This with regard to both stiffness and strength. The ideal structure will have no slip at all, and thus full composite action, corresponding to a continuous stress-strain profile. This means identical bending strains at the interface between the materials, see Figure 2.3 a). A structure like that is optimal since most of the compression forces are taken by the concrete slab, while the timber beams is handling the tensile forces. The weakness of the concrete to handle tension forces is then avoided. Avoiding tensile stresses in the concrete reduces the risk for cracks and the need of reinforcement (D. Yeoh et al., 2013).

Practically, complete composite action in TCC structures is difficult to achieve since the connectors, shear connectors, between the materials tends to allow some level of slip. However, if the slip is relatively small, partial composite action can be achieved, see Figure 2.3 b). With a slip in the structure, the strain diagram is not longer fully continuous over the cross-section of the structure. This means tensile stresses will occur in the concrete as well, even though most of them still will be taken by the timber (D. Yeoh et al., 2013).

The lowest level is no composite action at all, where the materials are not connected to each other and thus acts independently. In this case the bending strain will be discontinuous over the cross section and have two individual neutral axises, see Figure 2.3 c). The concrete will loose its function and the use of it as a deck could be questionable since it is a heavy material and the timber beams have to carry all the dead load (Jutila & Salokangas, 2010).

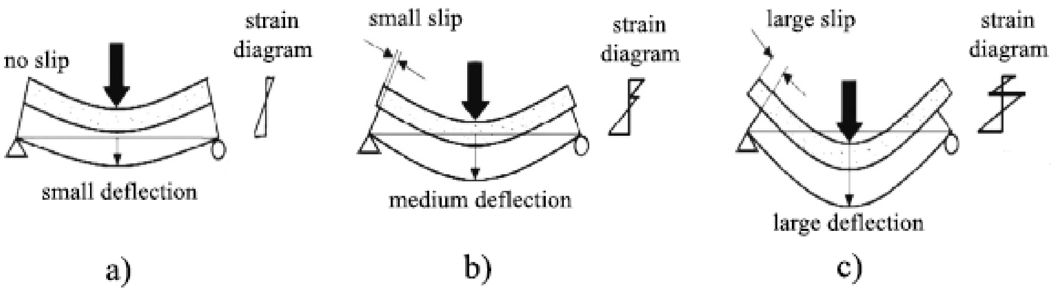


Figure 2.3: Three levels of composite action between a timber bream and a concrete slab; a) complete composite action, b) partial composite action and c) no composite action.
(D. Yeoh, Fragiacomo, & Carradine, 2013)

2.4.2 Connectors and stiffness

As mentioned above, a vital part of the TCC structure is the connectors between the timber and the concrete. They are the keys to achieve high composite action, since the natural bond between the surfaces of the materials is weak (Jutila & Salokangas, 2010). By using connectors that are strong and stiff enough to handle the shear forces in the structure, they can reduce the slip between the materials. (D. Yeoh et al., 2011)

The ideal shear connection system would be infinite stiff, but in reality they are deformable at some degree and often generate at least some slip between the interfaces. The structural behaviour of the bridge will due to this have a partial composite action, somewhere between complete composite action and non composite action (Crocetti et al., 2014). The stiffness of a connection, k, can be described as the slope of the curve in a force-displacement diagram and is calculated by Equation 2.1.

$$k = \frac{F}{\delta} \qquad [kN/mm] \tag{2.1}$$

where $F = Load$ and $\delta = displacement$

By using the curve at different values, different slopes could be obtained, leading to different values of stiffness. Stiffness at 40 % of the estimated maximum load is named $k_{0.4}$ and is used for design in serviceability limit state (SLS) as it corresponds to k_{ser}. $k_{0.6}$ answer to the stiffness at 60 % of the maximum shear force and is used in the ultimate limit state (ULS). Further, $k_{0.8}$ corresponds to 80% accordingly (Lukaszewska, 2009). If the curve is linear in the interval of above mentioned k-values, the stiffness will naturally be the same along the line, and the point of using them is of less importance.

When testing composite beams usually *beam bending tests* are performed to investigate the global behaviour of the beam. This gives an understanding of the level of composite action of a beam. If instead a more local behaviour is of interest, a *shear test* can be used to determine the stiffness of a specific connection. The results from the test gives results concerning the shear behaviour. The test set-up for a shear test can be either asymmetrical or symmetrical (see Figure 2.4 and 2.5). The reason for the asymmetry is the overturning moment arising from the eccentricity of the axial force, which creates a compression force at the interface between the concrete and the timber. This increases the friction which improves the mechanical properties and the test can handle higher forces. Therefore, using asymmetrical shear tests gives higher values of shear strength and slip modulus than if using symmetric shear test set-up. According to Lukaszewska, 2009 measured values by an asymmetric test overestimate true values of strength and slip modulus by approximately 10%. Symmetric shear tests is said to give more accurate results, but is in the same time more expensive and harder to construct (Lukaszewska, 2009).

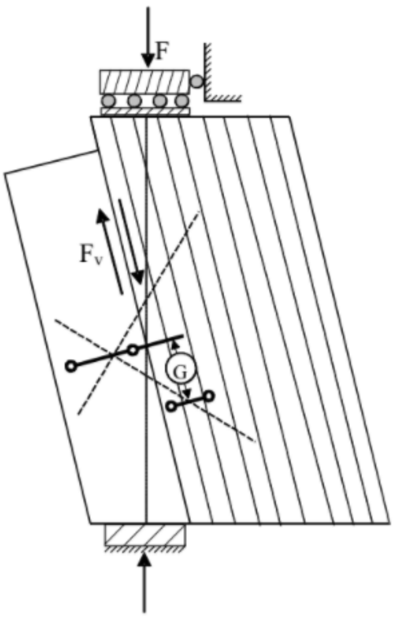


Figure 2.4: Symmetric shear test set-up (Jutila & Salokangas, 2010)

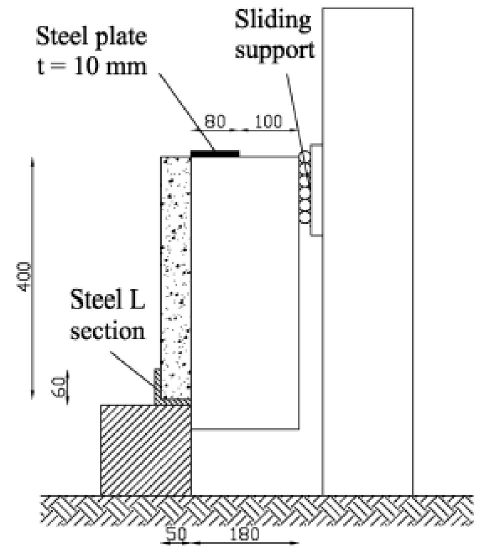


Figure 2.5: Asymmetric shear test set-up (Crocetti, Sartori, & Tomasi, 2014)

If a stiffness have been determined for a specific connection, it could be used in a simplified approach by making a global model with virtual springs having an equivalent stiffness, k, of that of the connection. This can be used to predict the load-displacement response and bearing capacity of a composite beam (Zhu et al., 2016), and as a tool when designing new composite structures. A simplified model with spring connectors are shown in Figure 2.6. A rigid shear connection provides a high bending stiffness of the beam and thus minimizes the deflections of the structure (Lukaszewska, 2009).

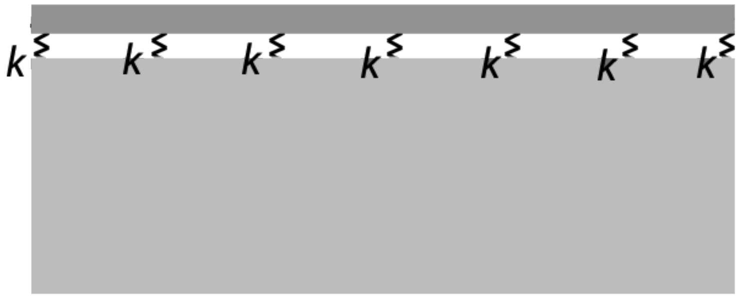


Figure 2.6: A beam modeled with the connectors as springs with an equivalent stiffness k

3 Composite connections

In this chapter previous made research regarding connections in composite bridges will be explained. Some with cast in-situ concrete deck and some with prefabricated deck, and with different application areas, such as bridges or floors.

3.1 Different connectors different behaviours

The connections presented could be divided into two groups; mechanical connections and notched connections (see Figure 3.1). The major difference between the two types is that the slip modulus for mechanical connections largely depends on the flexibility of the fastener and the timber around it, while notched connections depends on the stiffness of the wood in contact with the concrete inside the notch and the stiffness of the concrete in the notch (D. Yeoh, Fragiacomo, De Franceschi, & Heng Boon, 2010). Another alternative is to use glue in the connector, which could lead to a high composite action. However, it has several drawbacks regarding cost, environmental impact and time required for the glue to cure (Lukaszewska, 2009).

To develop a high partial composite action an effective connection is of highest priority, to accomplish the best combination of ductility, stiffness and strength. In Figure 5.9 the load-slip behaviour of two specimens with different connections are visualized. The notched connectors shows a stiff behaviour up to a sudden brittle failure, while the mechanical has a lower stiffness but a more ductile behaviour i.e. the slope is less steep. (Gutkowski, Brown, Shigidi, & Natterer, 2008)

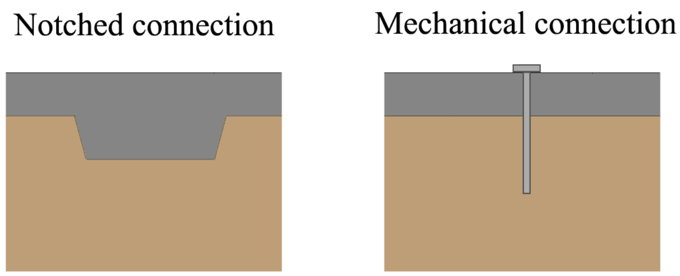


Figure 3.1: Two types of connectors; a notch connection to the left and a mechanical connector to the right.

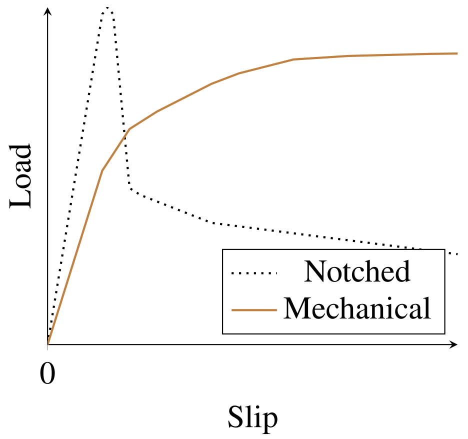


Figure 3.2: Typical load-slip curves for mechanical versus notched connections. (Gutkowski, Brown, Shigidi, & Natterer, 2008).

3.2 Connection by Gutkowski et al.

A connection combining the principle of mechanical and notched connections was investigated by Gutkowski et al., 2008 in the article "Laboratory tests of composite wood-concrete beams". The connection in the article was inspired by an earlier developed system for commercial and apartment construction by Natterer, Hamm and Favre in 1996. A detail of this shear key/anchor could be seen in Figure 3.3. The connection was used in larger beams that were tested laboratory. (Gutkowski et al., 2008 and references within)

The investigated connection was designed for a cast in-situ concrete slab where a plastic sleeve was placed within to make it possible for a dowel go through down to the underlying timber. In the timber there was a tapped pilot hole where the dowel was glued into. A cross section of the connection showing how the different parts are put together is shown in Figure 3.3 To avoid relaxation in the dowel due to shrinkage of the concrete, it could be post-tensioned with the nut. The idea is that after post-tension of the dowel, the void could be grouted to protect the connection from moisture and fire. (Gutkowski et al., 2008)

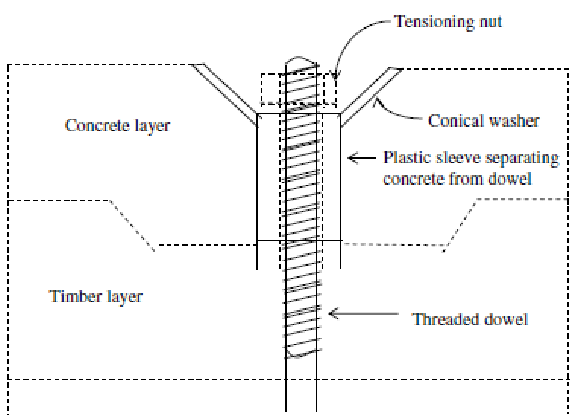


Figure 3.3: Notched shear key/anchor connection (Gutkowski, Brown, Shigidi, & Natterer, 2008).

Regarding the behaviour of the connection, the idea of Gutkowski et al., 2008 was that the shear key/anchor connection should be stiff by enabling shear force transmission by bearing stresses in the timber and horizontal shear in the concrete key, and thus achieve composite action in the beam. In Figure 3.4 a free body diagram of the connection is illustrated to show the reaction forces of the detail. The design ensures that the dowel is not subjected to shear, since the interlayer slip of the connection is low. The purpose of the dowel is instead to resist the small uplifting forces caused by the force acting on the inclination of the notch. To avoid high stress concentrations in this area that might cause splitting, an angle of 15° in the notch were chosen in the study. By this, high perpendicular-to-grain components of the bearing force was also avoided.

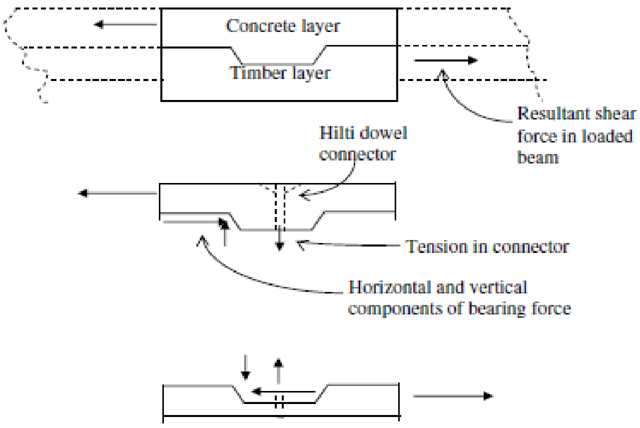


Figure 3.4: Free body diagram showing the force transfer of the connection (Gutkowski, Brown, Shigidi, & Natterer, 2008).

The dimensions of the notch were chosen based on prior slip tests also performed by Gutkowski et al., 2000, and are shown in Figure 3.5. Twenty specimens with this design were constructed and tested, in two groups with different dimensions. The tests were carried out with two point loads symmetrically loading the beam, having two notches on each side between the load and the support (see Figure 3.5). Between the two loads there were no notches since no shear should be present in this area, given that the loads are equal. (Gutkowski et al., 2008).

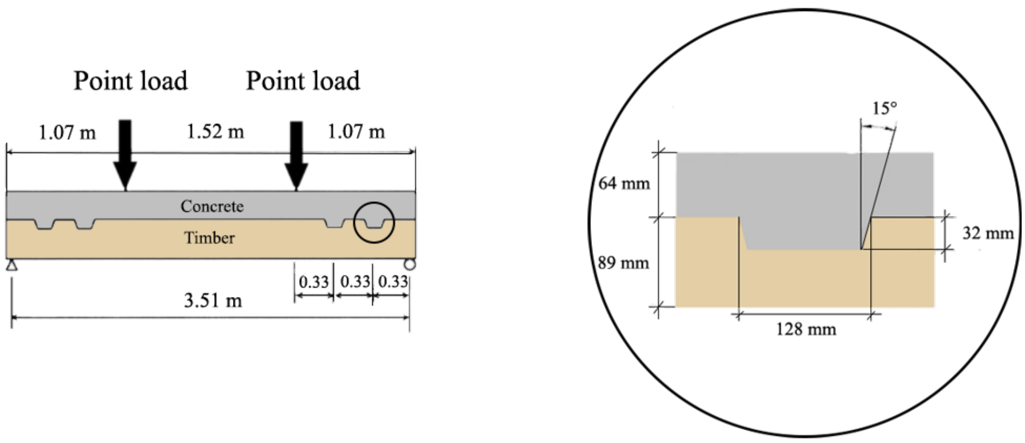


Figure 3.5: The slip test specimen of Gutkowski, Brown, Shigidi, and Natterer, 2008 with dimensions.

The results from the test showed a load-slip behaviour of the connections that up to failure was linear and handled by the notch. After failure a change in shear resistance of the steel anchor followed, and the remaining shear resistance corresponded to the residual resistance. This could be seen in Figure 3.6. A behaviour like that was also discovered by Kuhlman and Michelfelder for a similar connection but with a rectangular notch and screw instead of dowel. (Gutkowski et al., 2008)

For each specimen a plot was made for the average of the two point loads and the mid-span deflection. In Figure 3.6 representative results from the two different groups of beams are shown. The irregular response in the initiation of the loading was explained by problems with keeping the two loads equal, since they were controlled separately. However, after a load of 8.9 kN the beams showed a linear load-deflection behaviour up til initial sudden failure for both beam types. (Gutkowski et al., 2008). The composite efficiency of the beams reached were determined from the deflections measured at mid-span, and ranged from 54.9% up to 77% giving a mean of 67.2 %.

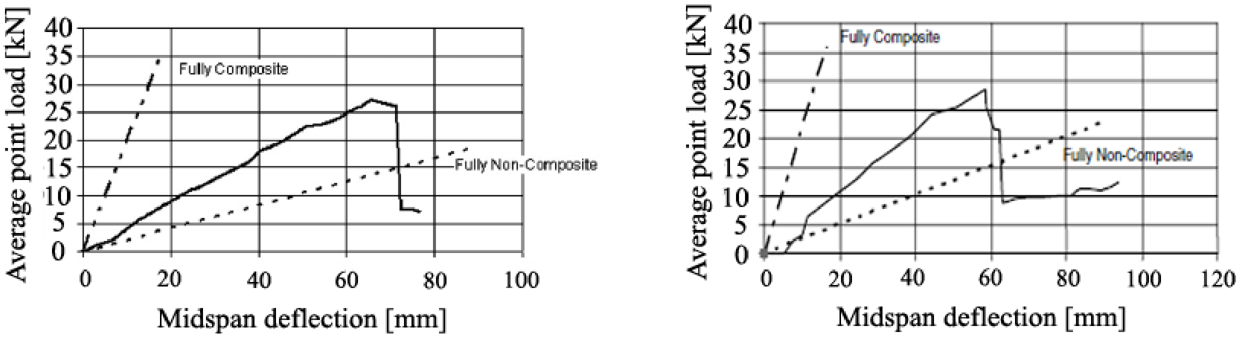
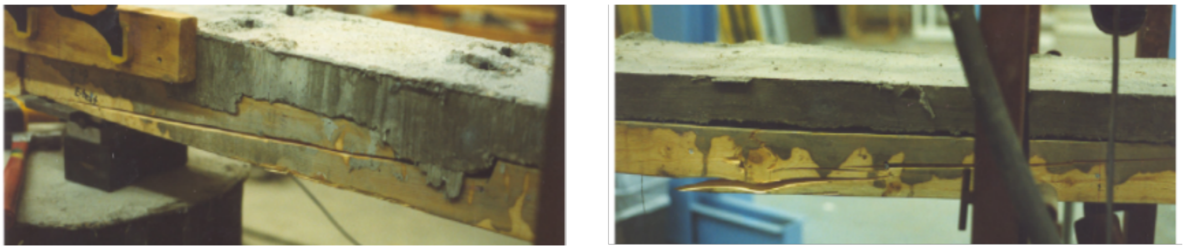


Figure 3.6: Average point load versus mid-span deflection for the tested beams (Gutkowski, Brown, Shigidi, & Natterer, 2008).

One of the conclusions drawn by Gutkowski et al., 2008 concerning the experiments was that the notch had not been designed in the most effective way. This was based on an earlier conclusion made by Weiligman et al., whom stated that if the notch is stiff the adjacent wood will fail in horizontal shear (see Figure 3.7 a)). If not, i.e. if the construction of the notch is imperfect, the wood will instead fail by a flexural tension failure in the mid span (see Figure 3.7 b)). The failures occurred in the tested beams were almost exclusively characterized by the last mentioned type of failure, meaning that the stiffness of the notch was not fully utilized. This could explain the determined mean composite efficiency of 67.2%. Theoretically, a notched detail should be able to achieve a efficiency of 95% according to Dias, 2005. However, the degree of composite action reached values almost 3 times that of another system tested using ordinary nails. (Gutkowski et al., 2008)



a) b)

Figure 3.7: Two of the failures obtained in the timber of the tested beams; a) flexural tension failure in midspan and b) horizontal shear failure. (Gutkowski, Brown, Shigidi, & Natterer, 2008).

3.3 Connections by Crocetti et al.

Crocetti et al., 2014 presents two different connections which is tested in shear tests. The first is called the Type T connection, mainly consisting of self-tapping screws driven into cast in concrete steel tubes and timber. The second connection is named the Type W connection and consists of wooden shear anchor-keys, i.e. wood blocks, into where self-tapping double threaded screws are driven. The connections is further described below, followed by the results of the shear tests performed on the different connections.

3.3.1 Type T connection

The first tested connection by Crocetti et al., 2014 consisted of screws that was driven into special steel tubes. The steel tubes, shown in Figure 3.8, was placed into formwork, before casting of the prefabricated concrete slab.

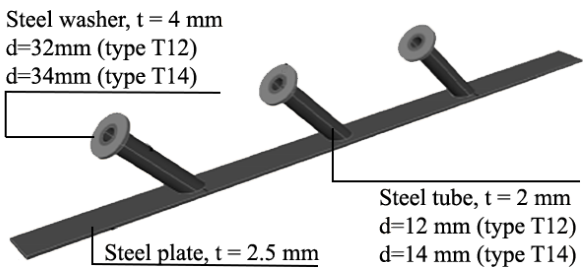


Figure 3.8: Geometry of connection Type T (Crocetti, Sartori, & Tomasi, 2014)

To increase the stiffness of the connection, they were placed with an inclination of 45°, which can be seen in Figure 3.9. After the concrete was cast and hardened (28 days), the steel tubes was stuck to the concrete, and then constituted the prefabricated slab. The prefabricated slab was thereafter placed onto the timber elements and the self-tapping pretensioned screws were driven into the steel tubes consecutively and tightened to the timber. Shear forces (V) were established at the interface between the materials, handled by shear action (F_v) and tension action (F_{ax}) (see Figure 3.9). Shear in the concrete part is resisted by contact pressure between the screw and the tube and tension is resisted by axial pressure of the head of the screw. In the timber elements on the other hand, shear is withstood by the wood embedment capacity while tension is resisted by withdrawal capacity. According to Crocetti et al., 2014, also compression stresses due to the inclination will be generated at the interface between the materials, evolving friction that adds to the stiffness and strength of the composite action.

Crocetti et al., 2014 presents two types of configurations of the Type T connection. The first formation, consisting of a tube with diameter 14 mm, used a fast setting cement in the tube with purpose to fill the gap between screw and tube. In the second formation, consisting of a tube with diameter 12 mm, no fast-setting cement was used. Those formations is called Type T12 and T14 respectively.

1. Connection Type T12 with tube diameter of 12 mm
2. Connection Type T14 with tube diameter of 14 mm

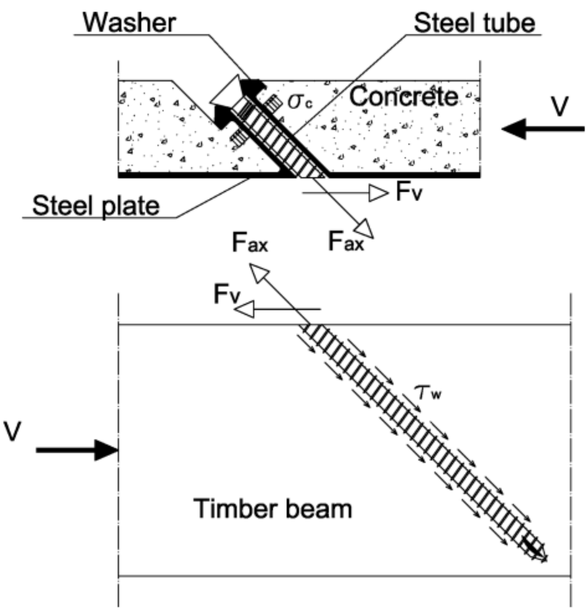


Figure 3.9: Load path of connection Type T (Crocetti, Sartori, & Tomasi, 2014)

3.3.2 Type W connection

The Type W connection consists of wooden shear anchor-keys, i.e. wood blocks, into where self-tapping double threaded screws are driven. As shown in Figure 3.10, the screws are applied in the longitudinal direction of the concrete element. Their existence has, according to Crocetti et al., 2014 two main functions; first, to guarantee proper anchorage of the shear anchor-key to the concrete slab and second, during loading reduce the risk for the anchor-key to split. The shear anchor-key were framed in the concrete before casting, and is after hardening part of the structure. At approximately 28 days after casting the prefabricated concrete element was put onto the timber structure. Thereafter, self-tapping screws with an inclination were applied through the shear-key and the timber element (see Figure 3.11). Inclinations of 45° and 30° was used, in specimens called W30 and W45. (Crocetti et al., 2014)

1. Connection Type W30 with screw inclination of 30°
2. Connection Type W45 with screw inclination of 45°

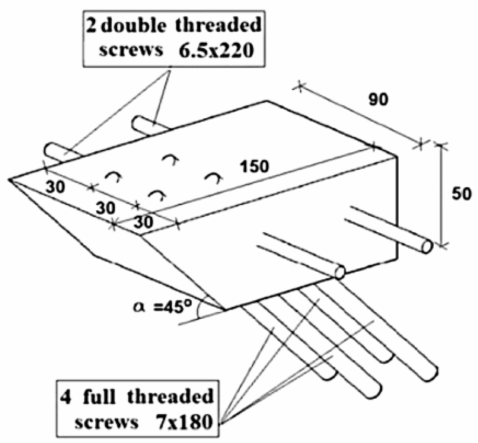


Figure 3.10: Geometry of connection Type W (Crocetti, Sartori, & Tomasi, 2014)

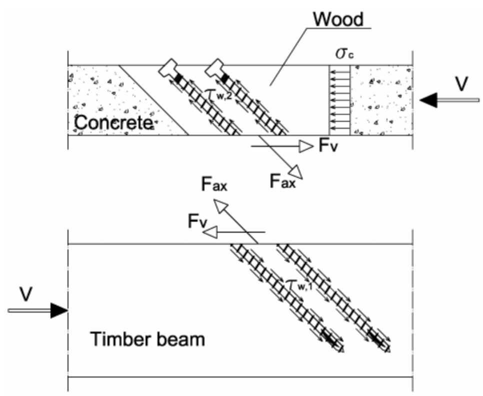


Figure 3.11: Load path of connection Type W (Crocetti, Sartori, & Tomasi, 2014)

The static principle of the Type W connection acts in a similar manner as for the Type T connection in Section 3.3.1, with shear force (V) transferred from concrete to timber element by shear action (F_v) and tension action (F_{ax}) (see Figure 3.11). A dissimilarity compared to Type T is that in this connection, shear is resisted by the wood embedment capacity and tension by withdrawal capacity, both in the wooden shear anchor-key and in the timber member. Also in this connection compression stresses develops between the members (Crocetti et al., 2014).

3.3.3 Evaluation of connections by Crocetti et al.

In order to study the load-slip behaviour of the connections explained above, shear tests were carried out on test specimens. Asymmetrical specimens were used in direct shear tests to determine the load-slip relations for the connectors. The set-up of the shear test is displayed in Figure 3.12. The load was applied at a 10 mm thick steel plate to the timber beam. In order to distribute the contact stresses evenly, a thin fibreboard was positioned between the L-support, which the concrete part is resting on, and the concrete surface. To minimize the vertical friction force, the upper horizontal support was chosen to a low friction sliding support. The relative displacement between the beam and the slab was measured at mid height of the specimen on both sides of the timber member. A clarification of which specimens that was tested in the shear test is presented in Table 3.1. Glulam GL30c was used in all specimens, while Class C24 was used for the shear anchor-keys for Type W45 and Type W30. The steel tubes for Type T12 and Type T14 connections was made of mild steel with yield strength of 355 MPa.

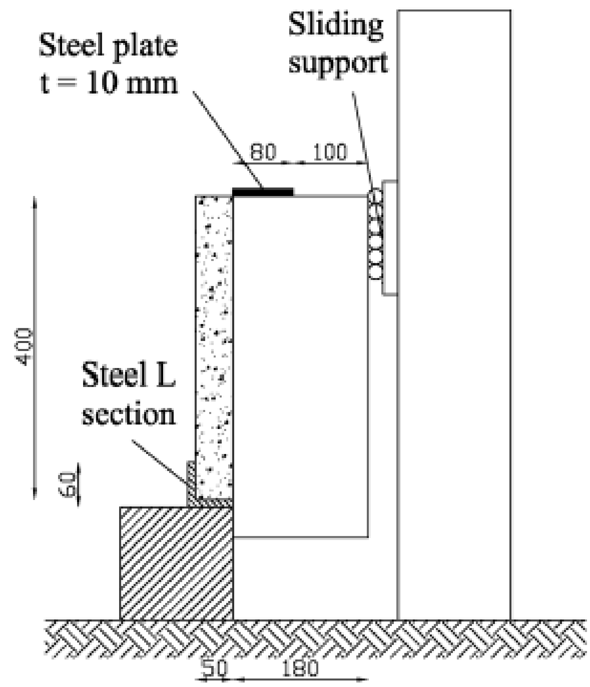


Figure 3.12: Set-up of shear test

Table 3.1: Specimens tested in shear test

T12:	Steel tubes diameter 12 mm, with fast setting cement
T14:	Steel tubes with diameter 14 mm, without fast setting cement
W30:	Wooden anchor keys, screws inclined with 30°
W45:	Wooden anchor keys, screws inclined with 45°

The stiffness and the failure load were extracted from the test for each connector, and resulted in values presented in Table 3.2. The values for Type W connectors are for the strength and stiffness of the entire system (i.e., anchor-key plus four screws), while results for the Type T connectors are for the strength and stiffness of only a single screw.

Table 3.2: Shear test results

Specimen	K_s [a] (kN/mm)	$K_{0.6}$ [a] (kN/mm)	F_{max} (kN)
W30_1	15	14	21
W30_2	13	10	19
Mean W30	14	12	20
W45_1	25	24	43
W45_2	29	27	38
Mean W45	27	25	41
T12_1	36	29	42
T12_2	36	29	36
Mean T12	36	29	39
T14_1	41	38	43
T14_2	34	30	44
Mean T14	37	34	44

[a] K_s is the stiffness defined according to EN 26891 (CEN 1991), while $K_{0.6}$ is the secant stiffness at a force equal to $0.6 \times F_{max}$.

When studying the results it is shown that T14 stands for the highest stiffness, while W30 has the lowest. The results also says W45 is 92% more resistant and 106% stiffer than W30. Therefore, according to Crocetti et al., 2014 the W45 connection seems to be more suitable for application in composite structures. The T14 connection is 10% stiffer and 13% more resistant than a T12 connection. The T14 connector is however much less manageable, and Crocetti et al., 2014 therefore recommends T12 for use in prefabricated composite structures.

3.4 Connections by Lukaszewska

A variety of connections for TCC floors are presented and investigated in a doctoral thesis named "Development of prefabricated timber-concrete composite floors" by Lukaszewska, 2009. The connections are divided into glued connections and mechanical connections. Experimental tests were carried out on these connections, including shear tests, bending tests, dynamic tests and long-term tests. After the experimental tests were performed, mechanical connections appeared to have the best properties to be used in a prefabricated connection for floors (Lukaszewska, 2009). This section will describe and evaluate the results of shear tests on the novel types of mechanical connections presented in above mentioned thesis, called SST+S, ST+S+N and SP+N (see Figure 3.13). The geometries of these connections is shown in Figure 3.14.

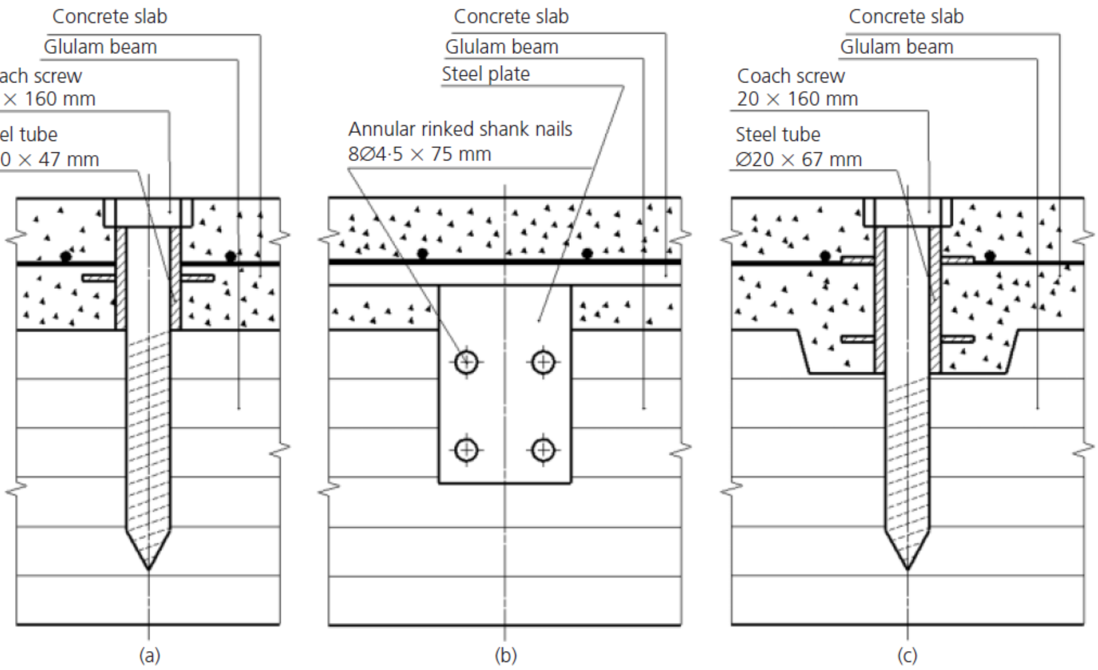


Figure 3.13: Connectors a) SST+S b) SP+N c) ST+S+N (Fragiacomo & Lukaszewska, 2011)

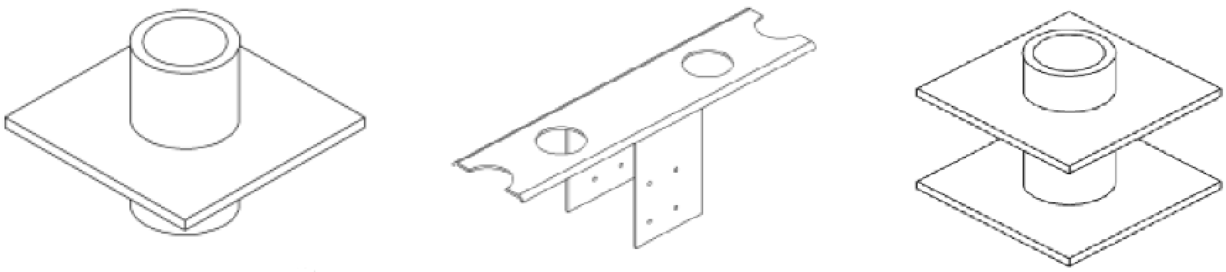


Figure 3.14: Connectors a) SST+S b) SP+N c) ST+S+N (Fragiacomo & Lukaszewska, 2011)

3.4.1 SST+S connection

The SST+S connection consists of a 47 mm long steel tube (see Figure 3.14 a)), welded to a rectangular flange embedded in the concrete. Using a hexagon head coach screw to be driven through the steel tube and down in the timber, connected the two materials and created the composite action, as shown in Figure 3.13 a). Before implementing the screw, pre-drilling was needed. The screw was then pre-tensioned (Lukaszewska, 2009).

3.4.2 SP+N connection

The SP+N connection constitutes of a U-shaped steel plate welded to a long punched steel plate, as shown in 3.13 b). The steel plate was embedded in the concrete and nailed into the timber. To avoid difficulties when placing each U-shaped steel plate in the form, the steel plate runs along the whole length of the beam, see Figure 3.14 b). (Lukaszewska, 2009).

3.4.3 ST+S+N connection

The ST+S+N connection (cross-section in Figure 3.13 c)) is composed by a 67 mm long steel tube with two rectangular welded flanges, which was placed into a notch that was previously cut out in the timber, see Figure 3.14 c). The notch had an inclination of 15° which can be seen in Figure 3.15. On top of the glulam element, a concrete slab was placed. The concrete slab had a hole in the middle, which was positioned just above the notch and the steel tube. The hole was then filled with concrete mix, and a pre-tensioned hexagon head coach screw was driven into the tube and the glulam. (Lukaszewska, 2009).

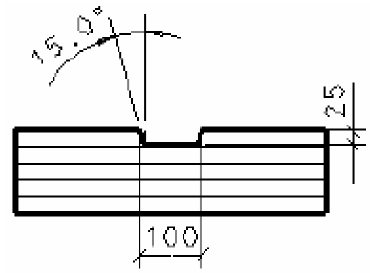


Figure 3.15: Inclined notch in glulam beam, for connection ST+S+N (Lukaszewska, 2009)

3.4.4 Evaluation of connections by Lukaszewska

Different tests of the connections were performed, where focus in this report will be on the shear tests. According to Lukaszewska, 2009, the aim of the shear tests was to investigate stiffness (slip modulus), shear strength and ductility of the connections. The test set-up is shown in Figure 3.16 and is operating similarly as in Section 3.3. The shear tests were carried out according to EN 26891. Each test was run by using displacement control until either failure load or a 15 mm slip was reached. Asymmetrical specimens were used in direct shear tests to determine the load-slip relations for the connectors. (Lukaszewska, 2009).

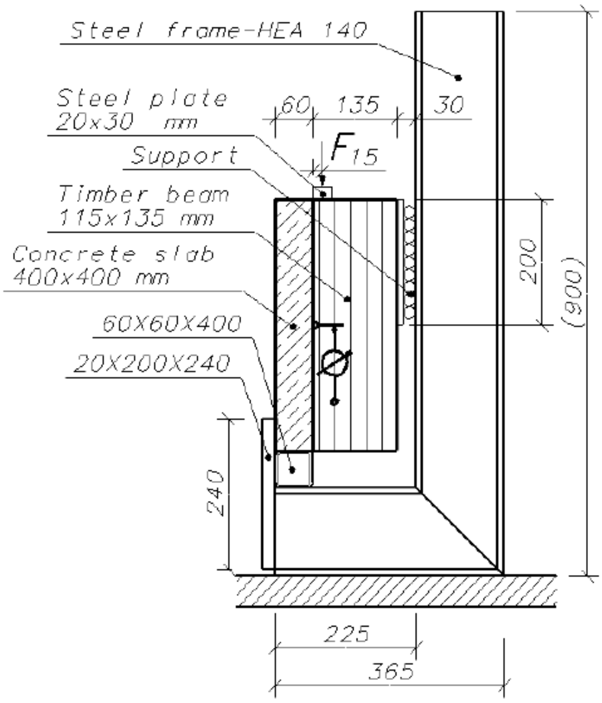


Figure 3.16: Set-up of shear test (Lukaszewska, 2009)

The shear force-relative slip curves for the specimens were evaluated in terms of stiffness, which is supposed to illustrate the performance of the connections. As previously explained, stiffness corresponds to the slope in a force-slip diagram, and by using the curve at different values, different values of stiffness will be obtained. Lukaszewska, 2009 is using $k_{0.4}$, $k_{0.6}$ and $k_{0.8}$ to describe the stiffness of the tests. The results of the shear tests is presented in Figure 3.17, where load-slip curves is plotted for each of the three connections. Lukaszewska, 2009 also presents the results in Table 3.18 in shape of different stiffness values.

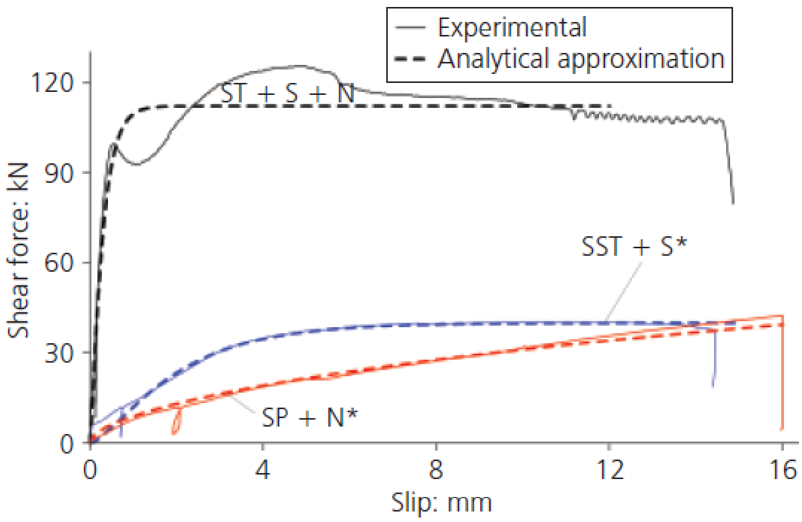


Figure 3.17: Shear test results (Fragiacomo & Lukaszewska, 2011)

Type of connection		Slip moduli			Shear strength, F_{max}: kN	Max. slip, s: mm
		$k_{0\cdot4}$: kN/mm	$k_{0\cdot6}$: kN/mm	$k_{0\cdot8}$: kN/mm		
SST + S* (4 samples)	Range	5·9–12·8	7·0–10·2	6·6–9·3	36·5–41·3	15·0
	Average	**8·5**	**8·3**	7·4	38·2	
	σ	3·0	1·4	1·3	3·0	
	5th percentile	—	—	—	**35·1**	
SP + N* (2 samples)	Range	5·0–5·7	3·1–3·5	2·4–3·0	37·0–43·0	16·0
	Average	**5·3**	**3·3**	2·7	40·0	
	σ	0·5	0·3	0·4	4·3	
	5th percentile	—	—	—	**37·3**	
ST + S + N (3 samples)	Range	221·3–245·6	231·6–248·2	139·6–217·9	99·6–126·7	12·0
	Average	**235·7**	**234·4**	178·0	110·6	
	σ	*12·8*	*8·7*	*39·2*	*14·2*	
	5th percentile	—	—	—	**100·2**	

In bold: values of slip moduli and shear strength used in design of timber–concrete composite beams

Figure 3.18: Table of results from shear test (Lukaszewska, 2009)

The highest maximum shear force reached 111 kN was obtained from the ST+S+N connector, as can be seen in Figure 3.17. At that point a slip of 12 mm had occurred. A stiffness value calculated at 40% of the estimated maximum load, i.e. $k_{0.4}$ turned out to be 236 kN/mm. The failure mechanism proved to be a composition of pure shear failure at the vertical cross-section of the notch and tension in the upper part of the notch.

The SP+N connector type on the other hand had the lowest stiffness $k_{0.4}$= 5.3 kN/mm and a maximum ultimate load of 40 kN at a slip of 16 mm. Shear failure was observed in the concrete slab while the nails remained undamaged. The SST+S connector gave a similar stiffness value, $k_{0.4}$=5.9 kN/mm and a maximum shear force of 34 kN with a slip of 15 mm. A crack along the depth of the timber occurred, while no cracks appeared in the concrete slab.

3.5 Connection by Auclair et al.

In the article "A new composite connector for timber-concrete composite structures" Auclair, Sorelli, and Salenikovich, 2016 presents a composite connector consisting of a composite cylinder made of ultra-high performance fibre-reinforced concrete (UHPFRC) shell with a steel cylindrical core. In the study, Auclair et al., 2016 made a prototype of the connection and performed a shear test, to determine the behaviour of it. The shear test was a symmetric test.

The manufacturing of the prototypes was initiated by adding concrete mix (concrete with UHPFRC or regular concrete mortar) into the connector moulding form with the steel core in place. The head of the mould, and thus the connectors, is square-shaped. After removal of the mould the connectors were cured for 7 days. A pre-drilled hole was made in the glulam timber beam, into where the cured connector was placed. Regular concrete mix was thereafter used to cast the remaining parts of the composite beam by pouring it into a formwork connected to the glulam beam. To protect the timber from getting wet, a plastic film was placed between the materials. The geometry of the connector is shown in Figure 3.19. (Auclair et al., 2016)

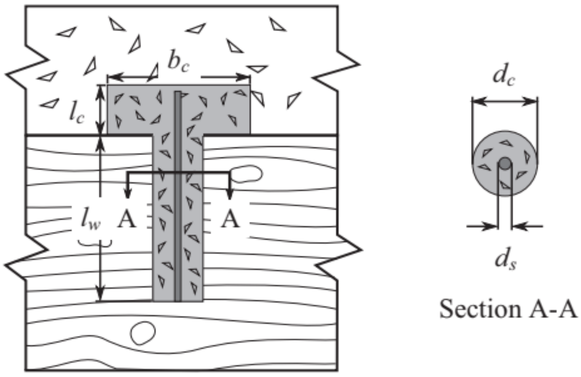


Figure 3.19: Geometry of the composite connector (Auclair, Sorelli, & Salenikovich, 2016)

The set-up for the shear test is presented in Figure 3.20. For comparison, the connector shells were made in two different sizes, where some were made with UHPFRC and the rest by regular concrete mortar. Two types of steel were used for the connector core; threaded rods (TR) and rebars (RB). Configurations of the tested connections are shown in Table 3.3

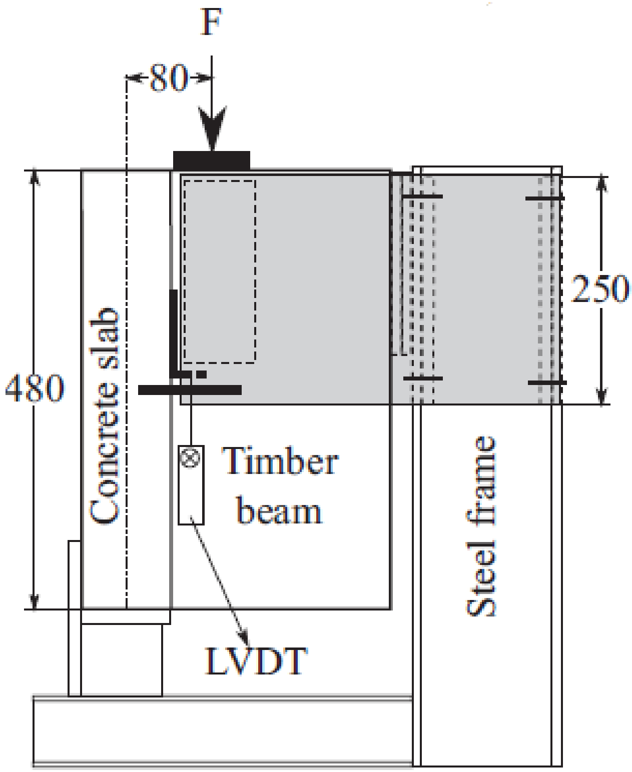


Figure 3.20: Test set up for shear test (Auclair, Sorelli, & Salenikovich, 2016)

Table 3.3: Configurations of tested connectors (Auclair, Sorelli, & Salenikovich, 2016)

Test	Concrete shell	Steel core	d_c [mm]	d_s [mm]	l_w [mm]
#01–02	UHPFRC	-	25.4	0	95
#03–04	UHPFRC	TR*	25.4	5 (M6)	95
#05–06	Mortar	TR	25.4	5 (M6)	95
#07–08	UHPFRC	TR	25.4	10.2 (M12)	95
#09–10	UHPFRC	RB*	25.4	10 (10 M)	95
#11–12	Mortar	TR	25.4	10.2 (M12)	95
#13–14	UHPFRC	TR	34.9	5 (M5)	135
#15	UHPFRC	TR	34.9	10.2 (M12)	135
#16	UHPFRC	RB	34.9	10 (10 M)	135

* TR: Threaded Rod, RB: Reinforcement Bar.

After shear testing, results in terms of maximum load (F_{max}), slip modulus (k_i), ultimate slip (δ_u) and failure modes were achieved. Auclair et al., 2016 measures maximum load within a slip of 3 mm. The linear portion of the load-slip curve is represented by the slip modulus, while the ultimate slip corresponds to the significant load drop indicating the failure of the connector. (Auclair et al., 2016).

The test results, as shown in Table 3.4, proves that an increase of the diameter of the concrete shell (d_c) increases the connection stiffness, i.e. the slip modulus (k_i). If instead enhancing the diameter of the steel core (d_s), the connection resistance, shown by maximum load (F_{max}), will increase. Using threaded rods improved the connection stiffness with 75% compared to use a rebar with the same diameter. The same table also presents the detected failure modes by numbers as presented below:

1. Shear failure of the connector at the interface
2. Pull-out of the steel core from the connector head
3. Pull-out of the steel core from the connector shank

The failure mode *Shear failure of the connector at the interface* mostly depends on the properties of the prefabricated connector. Those properties are easier to control and therefore less variable compared to the properties of the connected members. This failure mode is therefore the failure mode to prefer and was observed in most of the configurations in the test (in 10 of 15). Failure mode *Pull-out of the steel core from the connector head* was detected in connections with regular mortar, together with cracking in the concrete slab and is therefore not to recommend. Failure mode *Pull-out of the steel core from the connector shank* was found in the connectors where a rebar was used, together with crushing of the timber by the withdrawn steel rob. This failure mode also proved to be unwanted. (Auclair et al., 2016).

Table 3.4: Shear test results (Auclair, Sorelli, & Salenikovich, 2016)

Test	F_{max} [kN]	k_i [kN/mm]	δ_u [mm]	Failure mode	Test	F_{max} [kN]	k_i [kN/mm]	δ_u [mm]	Failure mode
#01	8.44	22.4	3.0	(1)	#09	27.85	14.4	>15	(3)
#02	12.42	13.5	5.0	(1)	#10	29.00	21.6	>15	(3)
#03	16.93	28.6	7.4	(1)	#11	20.38	19.9	12.9	(2)
#04	17.59	11.8	7.7	(1)	#12	19.73	8.8	12.6	(2)
#05	4.15	18.1	9.7	(2)	#13	28.06	48.0	1.1	(1)
#06	–	–	–	(–)	#14	27.82	36.3	1.8	(1)
#07	35.95	37.0	12.1	(1)	#15	51.24	52.8	12.0	(1)
#08	31.36	26.2	11.2	(1)	#16	50.11	29.9	>15	(1)

The connection analysis, which will be compared with the experimental shear tests, is performed by using the Winkler model. In the model the connector is represented by a beam placed on an elastic foundation, which makes it possible to predict the shear behaviour, i.e. shear force versus slip. The Winkler model is further described in the article "A new composite connector for timber-concrete composite structures" by Auclair et al., 2016. When comparing the calculated results from the Winkler model with the experimental shear test results, there was a poor agreement for many of the connector configurations. The wood foundation proportions was therefore calibrated by an inverse analysis of the model, creating calibrated curves. The calibrated curves and the curves achieved by the experimental shear test results is plotted and shown in Figure 6.1. (Auclair et al., 2016).

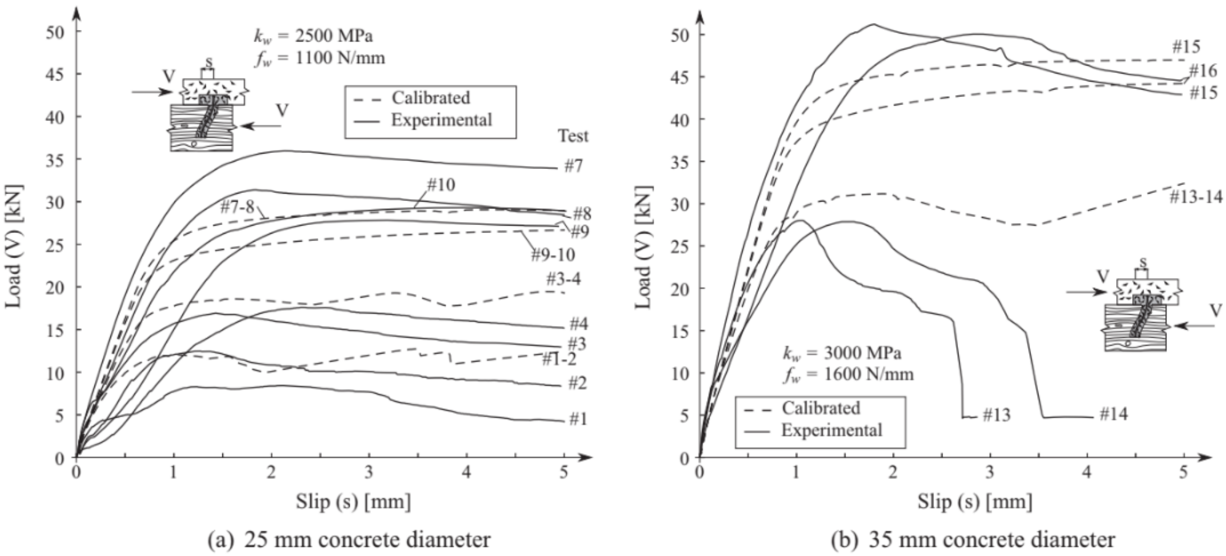


Figure 3.21: Calibrated vs experimental load-slip curves (Auclair, Sorelli, & Salenikovich, 2016)

By studying the results, one can conclude that desired stiffness, strength and ductility of a TCC structure can be reached by choosing the connector and the spacing between connectors. The connection stiffness can be controlled by the diameter of the concrete shell, while the connection resistance is governed by the size of the steel core. By performing a bending test with the connection in a beam, a load-deflection curve plotted in Figure 3.22 can be obtained. The figure remarks how close or not close some of the connection configurations is to a perfect composite action, also if respective connector has a brittle or ductile behaviour. #13 has a very brittle behaviour, while #03 has a ductile behaviour but also a reduction of the maximum load of 30%. (Auclair et al., 2016).

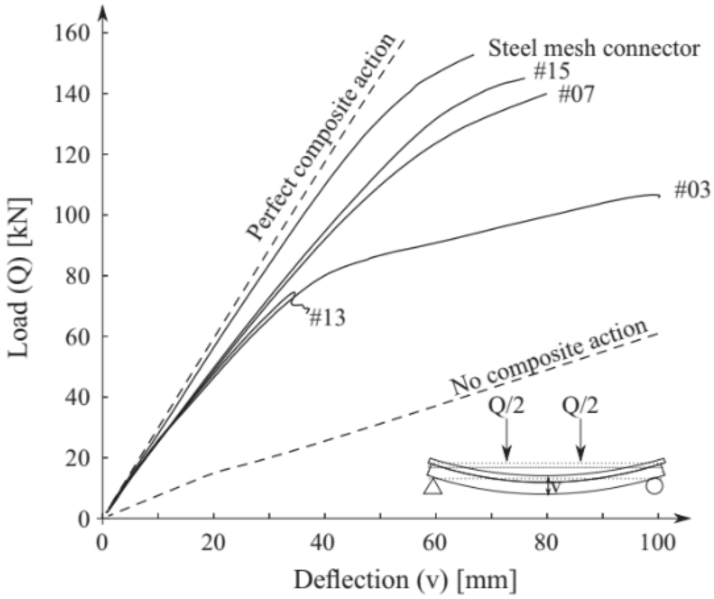


Figure 3.22: Calculated load-deflection curves of TCC beams with various connections (Auclair, Sorelli, & Salenikovich, 2016)

3.6 Reference project: "The Quay"

In 2017 a project started concerning a quay located along Göta river in Gothenburg. There had been a request from the client, the Port of Gothenburg, to use other materials than steel in the structure, since corrosion had been a recurring problem in structures along the river. Too meet the requirements of the clients, a new type of quay were constructed using glulam beams with a cast in-situ concrete slab on top. As basalt fibres were used as reinforcement (see Subsection 2.3.1), the risk of corrosion was reduced even further.

The quay is benamed a quay but treated as a pedestrian bridge during the project. 100 meter of quay were constructed with free spans of 12.5 m. The glulam beams were prefabricated with its composite connections. The connections consisted of rebars in steel inserted in the beams with an inclination, but following the beams horizontal over the top (see Figure 3.23). As can be seen in Figure 3.24 the distance between the connections were inconstant over the length of a span. The distance was based on calculations made for the shear forces in the structure, which are zero in the middle of the span and increases to maximum at the supports. When constructing the quay the concrete were cast in a form placed around the glulam beams, that was kept in the construction. The form can be seen in Figure 3.25 which shows a picture from the casting of the quay.

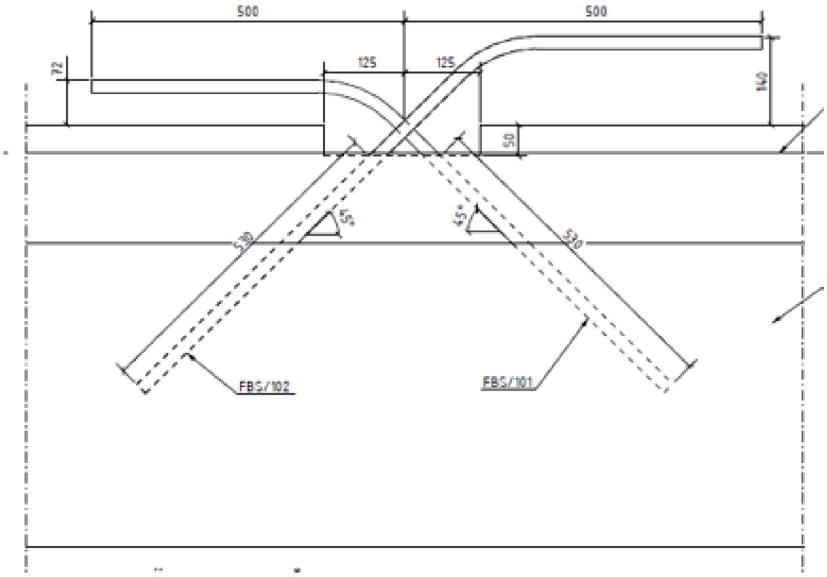


Figure 3.23: Detail of the connections used in The Quay. (ÅF, 2015)

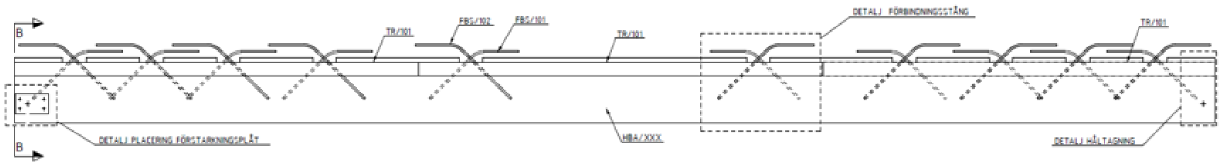


Figure 3.24: Detail of one span of The Quay. (ÅF, 2015)

Figure 3.25: Casting of the concrete slab during the construction of the quay. Some of the rebars that are soon to be cast into the concrete are visible. (ÅF, 2015)

3.6.1 Laboratory test

In order to verify the shear capacity between the glulam beams and the cast concrete slab in the quay, a shear test was performed by SP Technical Research Institute of Sweden (SP, 2015). Two specimens were tested, consisting of the connection used in the quay and 3 glued in rebars with a diameter of 16 mm. The concrete part had a cross sectional area of 200 x 400 mm and the timber 215 x 675 mm. For the tests a symmetrical test set-up were used, described in Figure 2.4 on page 9. The loading was increasing constantly up to failure after approximately 10-15 minutes. The displacement which the tests could handle was extracted continuously during load increase and related values was presented in a load-displacement graph shown in Table 3.26. A stiffness of approximately 300 kN/mm could be read out of the graph. The ultimate loads, equaling the shear force the test could handle, is presented in Table 3.5.

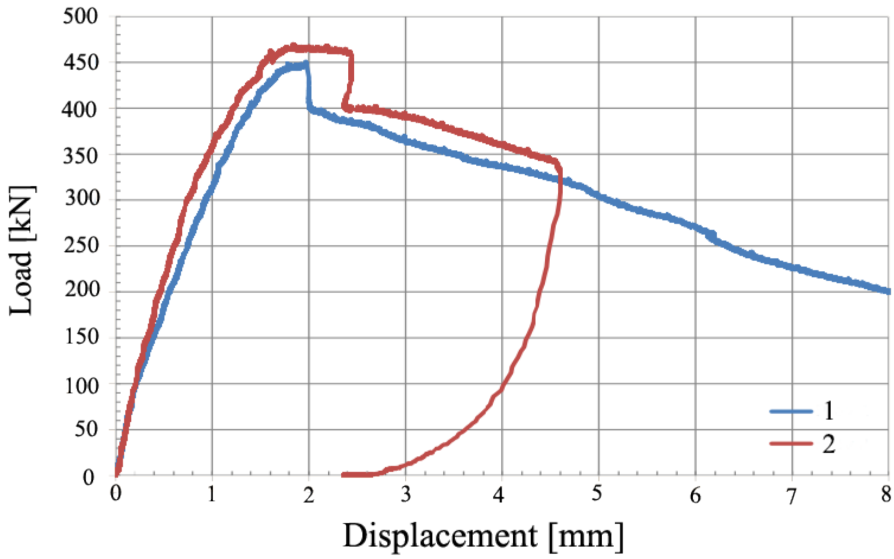


Figure 3.26: The load slip behaviour of the tested specimen (SP, 2015)

Table 3.5: Ultimate loads determined from the tests

Results

Test	Ultimate load [kN]
1	451
2	470
Mean value	**460**

4 A proposed connection

This chapter starts with an evaluation of Chapter 3, regarding what details of prior research that is reasonable to use in further development and in the connection later proposed in this thesis. Together with conclusions drawn, the work towards a proposed connection presupposing a suggestion developed by Lundberg, 2018 is shown in this chapter.

4.1 Evaluation of previous research

As concluded in Chapter 3, using a notch in the connection can increase the stiffness and strength of a structure compared to use just a mechanical connection. The reason is the ability of the notch to handle shear stresses in a more efficient way. A mechanical fastener, as a bolt or screw, is much better utilized when loaded in axial direction than in shear. The mechanical fasteners are desirable in bridge design since they ensures a ductile failure, which the notch can not provide.

Crocetti et al., 2014 used above mentioned facts in their study by using inclined screws in the structure. By doing so, those will handle the shear forces in the structure and resist the slip by axial loading (see Figure 3.3.1 and 3.3.2). This increases the strength compared to earlier solutions, but however, the optimal connection seemed to be a combination of the two previously mentioned options; mechanical fastener and notch. Solutions like this were found in the work of Gutkowski et al., 2008 (see Section 3.2) and Lukaszewska, 2009 (see Section 3.4). In the study of Lukaszewska, 2009 a number of different connections was investigated, where the so called ST+S+N connection was found as the best, showing a very high stiffness of 236 kN/mm taken at 40% of the estimated maximum load. ST+S+N is based on the principle of combining notches and mechanical fasteners. It is stated that the high stiffness is due to the notch in the timber beam. Previously investigated connections with notches have an inclination of the notch, to avoid stress concentrations. A recurrent inclination in previous works is 15°.

The Quay in Subsection 3.6 is especially of interest since it is part of the background to this project. However, these two projects are different in many ways. The major difference is the fact that the concrete slab was cast on site for The Quay. This enabled an easy production, and also the possibility to use optimized distance between the connections. In The Quay, the distance between the connections could be increased closer to the mid span as the shear force is decreasing. To use variable distances can be complicated at production stage, but since the connections were manufactured at the factory and transported to site as prefabricated elements the concrete could be cast easily on site. This without considering the different distances between the connections.

Just as in the connection by Crocetti et al., 2014, an inclination of the reinforcement steel bars in the structure of The Quay was used. As mentioned, those will handle the shear forces in the structure and resist the slip. One part of the force is affecting the screw axially, which is the most efficient for the screw. Though, the other effective component does not act axially, which is not optimal. Again, the optimal connection seemed to be a combination of a mechanical fastener and notch, to load the mechanical fasteners mainly by axial forces.

The stiffness reached from the tests performed of The Quay is not found fully comparable either, due to the different concrete slabs. The connection of The Quay proved to enable a very high stiffness, that was not found in the other investigated connections. Anyway, the structure is not possible when a fully prefabricated structure is desired. However, what was found interesting with the The Quay, is the satisfactory values of moisture in the structure. It was proved by Andersson and Brahme, 2017 that moisture was not an issue, due to the use of BFRP bars in the concrete.

The major difference in the previous research studied and the case of this study is the fact that the slab will be prefabricated, and used in bridge design. This will result in larger loads on the structure and larger spans than TCC structures used for flooring. No previous research was found describing how to connect different prefabricated concrete elements together with a connection between timber and concrete, which is a demand for the connection developed in this work. In the next section a conceptual design of a connection for this use will be presented.

4.2 Design of connection

The primary aim with the connection is to possess a high stiffness, and thus provide a desired composite action for the bridge. Further, the connection should enable an uncomplicated construction, and a possibility for dismantling. It is vital that the failure mode, when studying the connection locally, is in the mechanical connector. When failure occurs in a mechanical connector a formation of a plastic hinge occurs, which corresponds to a ductile failure. A ductile failure is desired, since it is much safer than a brittle failure. Though, when observing the bridge globally the failure mode should be bending failure in the timber beams, and not a failure in the connection. A bending failure in the timber beams creates a ductile behaviour. If the connection globally would reach failure first, all parts will fall apart in a hazardous manner. However, in practice there is always a risk for accidental loads, fires or other unpredictable cases, which could lead to a failure in the connection first. That explains the vitality in securing the failure mode of the connection itself to a plastic hinge in the connector.

An initial concept of a connector was developed by Lundberg, 2018, shown in Figure 4.1. The idea with the concept is that prefabricated concrete slabs, are placed perpendicular to glulam beams in notches. The desired behaviour was not verified at the time, as no tests or analyses was made of the initial concept. Further development and analysis of the connection was left for this study.

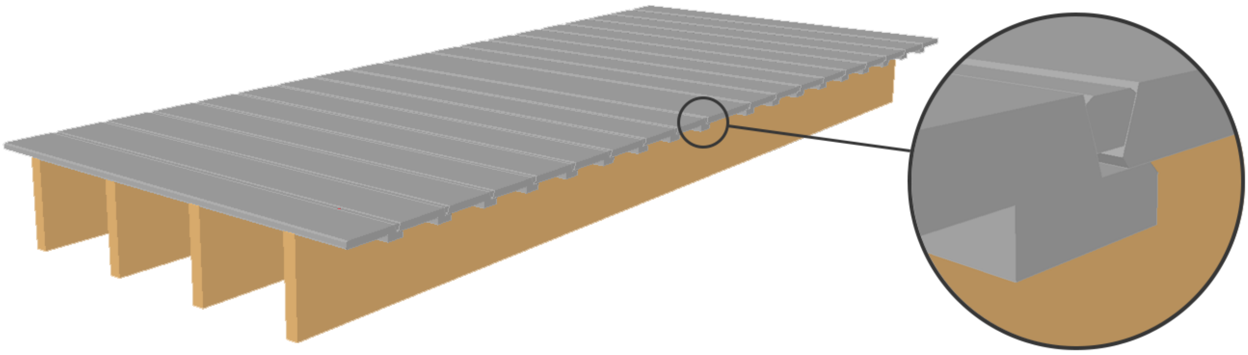


Figure 4.1: A model of the bridge with detail of the connection shown specifically.

Based on the concept by Lundberg, 2018 in combination with minor changes after evaluating previous research in Section 4.1, a concept was developed. A section of the concept is shown in Figure 4.2, consisting of a mechanical fastener and a notch in combination. The bottom member in the figure represents the glulam with an inclined notch, preliminary inclined with 15°. The top members constitutes the concrete and the intersection between two prefabricated concrete elements is shown in the figures. When they are positioned, a concrete wedge is placed in between to keep them in place. The wedge has an inclination as well, which will vary between analyses, as it is vital for achieving desired behaviour. The right concrete member and the concrete wedge is pre-drilled so that a screw can be driven down through the glulam. The diameter of the pre-drilled holes in the concrete will be larger than that of the screw, allowing small movements to avoid screw and concrete to affect each other significantly. Accordingly, the screw is fully fixed only to the glulam member. A thin cover is placed above the connection as protection. Globally, the two end elements on each side of the bridge is fixed at the edges to resist the upcoming forces when placing the concrete wedges. Since the high performance concrete in the reference project presented in Section 3.6 contained basalt reinforcement bars with satisfying results, the same will be used in presented concept.

Since part of the aim with the analyses was to find dimensions for the connections, only a few were set in the start. These were the thickness of the concrete and the height of the beam, because these parts are mostly dependent on the global behaviour of the bridge. To get reasonable dimensions a simplified model of the bridge was simulated in the FE software Robot. Besides the self weight, the traffic loads acting on the bridge were taken into consideration in accordance with SS-EN 1991-2: 2003. Load Model 1 (LM1) was simulated by a moving load in the software to find extreme values. The simulation determined the thickness for the concrete slab and the height of the glulam beams used in further analyses.

The distances between the connections in the concept presented above is important during production, in contrast with The Quay. This due to the fact that the distance between the connections also decides the width of the concrete slabs. Using different widths of the slabs is considered problematic from a production point of view when using only prefabricated elements, and simplicity in the production is considered more important than optimization. Therefore, a constant distance between the connections will be used in the connection concept of this study.

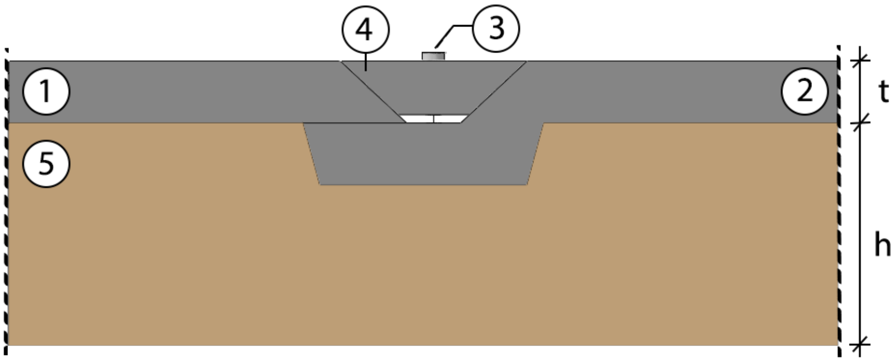


Figure 4.2: Detail of the connection and its elements.

Table 4.1: The parts in the connection

Part	Structural element	Material	Dimension [mm]
1	Prefabricated deck, right half	Concrete	t = 15
2	Prefabricated deck, left half	Concrete	t = 15
3	Screw	Steel	-
4	Wedge	Concrete	-
5	Beam	GL32c	h = 1575

When the bridge is loaded, shear forces will develop and a slip between the concrete slab and the glulam beams arises. The overall aim of the connection is to reach the desired behaviour; to make the stiffness as high as possible. A prerequisite to reach a valid stiffness, is to have a connection which parts is loaded sufficiently and also has a ductile behaviour. The ductile behaviour is reached by ensuring yielding of the screw before failure of concrete or timber. This behaviour, where the wedge is moving upwards and causes yielding in the screw before the timber fails, is shown in Figures 4.3 - 4.5. Figure 4.3 shows how a movement of the left short side of the timber gives rise to shear forces in between the materials, as the left short side of the concrete is fixed. Simultaneously, small movements of the concrete elements occurs inwards against the wedge (see Figure 4.4), forcing the wedge to move upwards. As illustrated in Figure 4.5, the screw is tensioned as a consequence of the wedge moving upwards. An axial stress is prevailing in the screw, and when reaching yield stress, starting to create the desired plastic hinge, leading to a safe ductile behaviour.

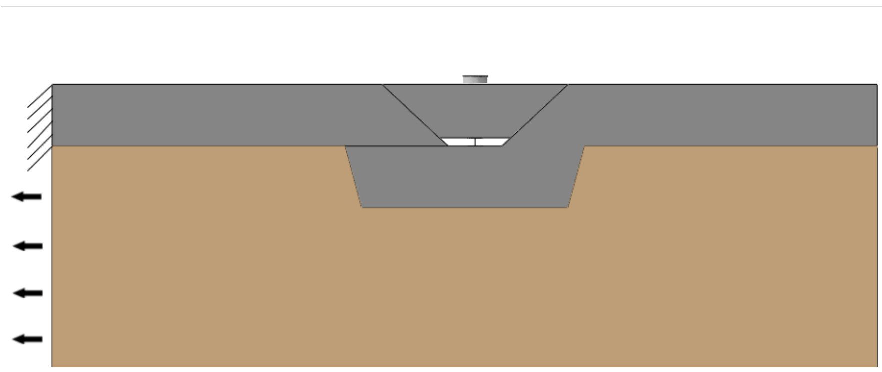


Figure 4.3: Displacement of left short side of the timber beam starting

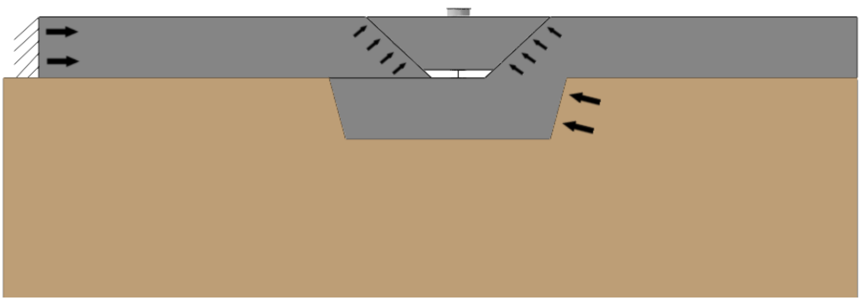


Figure 4.4: The largest reaction forces illustrated and how they want to push up the wedge

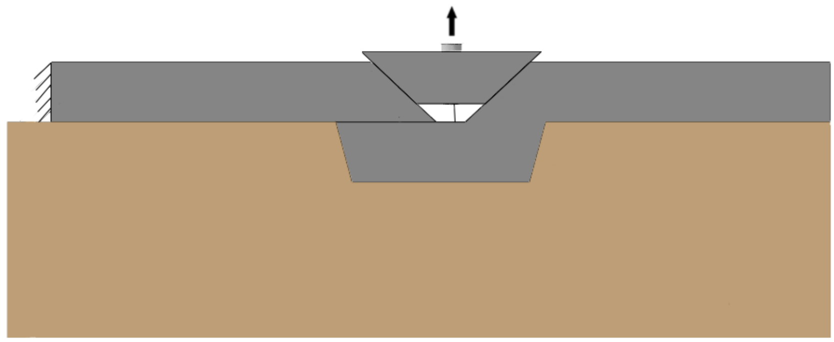


Figure 4.5: The wedge moving upwards causing an axial force in the screw

4.3 Outputs for analysis

Based on the proposed initial connection concept developed in this chapter, together with the desired behaviour described in Figures 4.3 - 4.5, relevant outputs from the analysis were selected to bring forward to the FE analyses. As one of the most important factors of the connection is the stiffness, analyzing the relationship between the load and the slip will be the main focus. In addition, the desired ductile behaviour will be analyzed. This behaviour can be accomplished if stresses arisen in the materials are kept below their capacities, when the steel in screw have reached its yield strength. Consequently, the stresses in critical areas of the timber and the stresses in the screw is of high importance for the approaching FE analyses presented in the following chapter.

5 FE analysis

To be able to analyze the connection, a FE model were created in the software Abaqus. The FE analysis is an iterative process with the aim to find satisfying dimensions that gives the connection a desired behaviour, further described in Section 5.1. In this chapter the process of developing the connection will be described i.e. the analysis process, followed by the details concerning the FE model.

5.1 Analysis procedure

The overall aim with the FE analysis was to reach the desired behaviour of the connection and to make the stiffness as high as possible. Stiffness, k, is calculated by Equation 2.1. A prerequisite to reach a valid stiffness, is to have a connection which parts is loaded sufficiently and also will have a ductile behaviour. The ductile behaviour is reached by ensuring yielding of screw before failure of concrete or timber. A flowchart presented in Figure 5.1 describes the procedure to reach the desired behaviour in the FE analysis. It is an iterative procedure which goes back and change variable parameters until a satisfying stiffness, and therefore behaviour, has been reached. This when all the critical steps are fulfilled; yielding in screw, stresses in other materials below their respective capacities and stiffness of the whole connection sufficient.

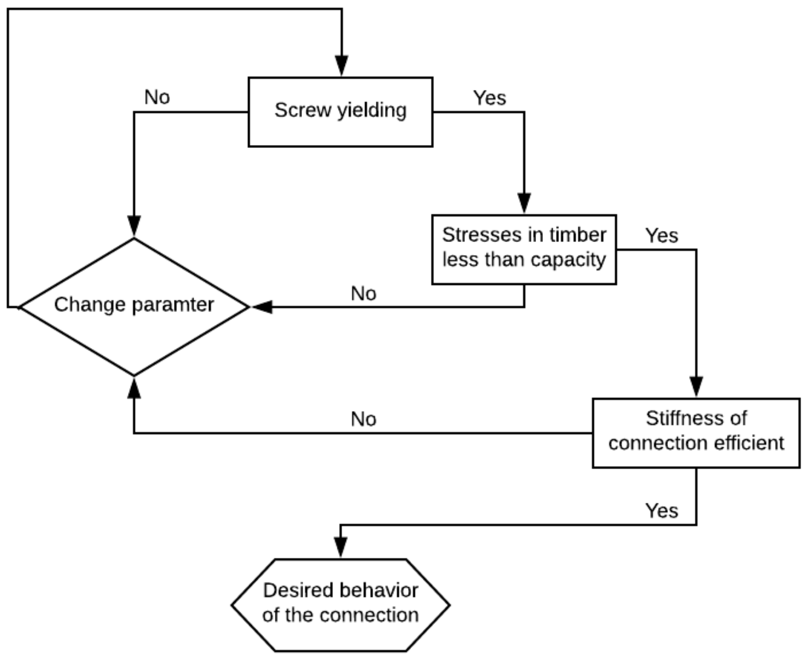


Figure 5.1: Flowchart describe the iterative process

The analysis was divided in two analyses, and from each analysis four tests has been performed. The first tests in each analysis had the aim to identify the most critical areas in the proposed connection. With this knowledge, the important parameters affecting these areas were identified and changed one by one to make the connection possible. Further, the parameters were deeper analyzed to increase the stiffness of the connection. Four iterations, tests, were performed for Analysis 1. Based on these results, Analysis 2 could start. The intention with Analysis 2 was to develop the connection further by changing the inclination of the wedge, which affects the behaviour and stresses in the connection significantly.

5.2 FE model of connection

The FE model was built of seven different parts assembled together, see Table 5.1. By studying Figure 4.1 on page 31 together with 5.8, an understanding can be achieved when it comes to the correlation between the bridge and the model. The model of the connection consists of two concrete elements, corresponding to the concrete deck, along with the concrete wedge and screw keeping them together. Placed underneath described elements is the glulam beam, from which width also decided the width of the concrete parts in the model. The dimensions are presented in Table 5.2, where some are varying through the different analyses.

The element type used were three-dimensional (3D) solid elements for all parts except the screw. Solid elements are three-dimensional finite elements that can model solid bodies and structures without any need for geometric simplification. Using solid elements is straight forward; boundary conditions, forces and displacements can be more realistically treated. Still, there are disadvantages with using solid elements, concerning effort and time spent on modelling and mesh preparation. Although, solid elements was chosen in the model, as the advantages was considered important.

Table 5.1: Table of the containing parts of the model

Part	Name	Element type	Material
1	Concrete left	Solid	Concrete
2	Concrete right	Solid	Concrete
3	Concrete wedge	Solid	Concrete
4	Screw wire	Beam	Steel, plastic
5	Screw head	Solid	Steel, elastic
6	Glulam beam	Solid	Glulaminated timber

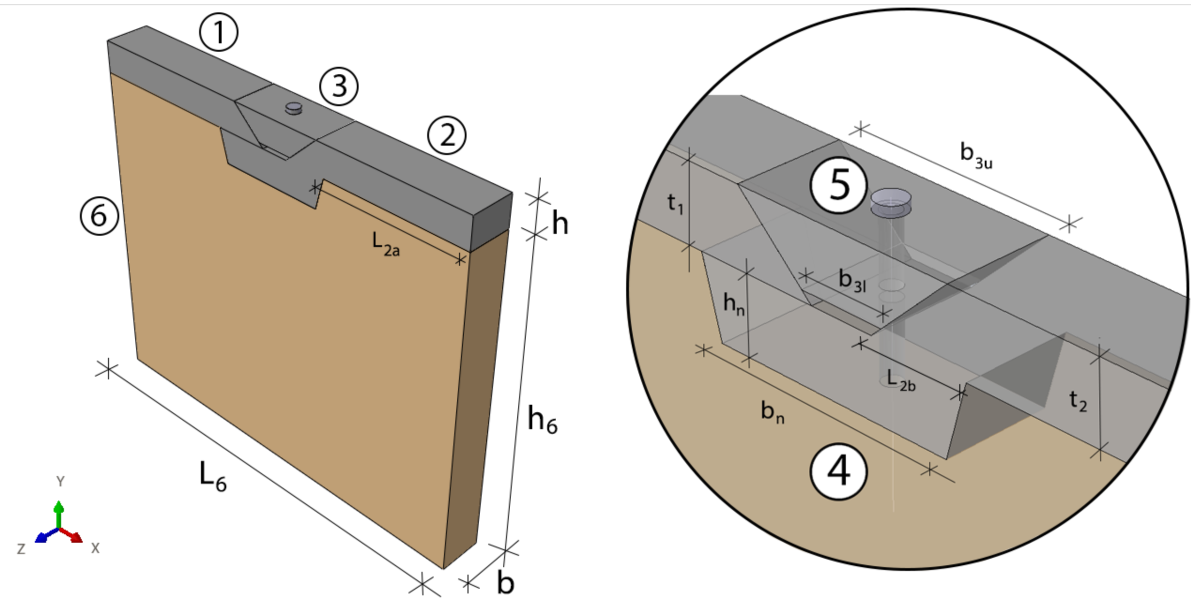


Figure 5.2: The FE model with dimensions

Table 5.2: The dimensions of the FE model

Dimensions

Element no	1	[mm]	2	[mm]	3	[mm]	4	[mm]	5	[mm]	6	[mm]
	t_1	150	t_2	150	b_{3u}	0.45/0.21*	L	1000	D	x	L_6	2000
	b	215	b	215	b_{3l}	0.17/0.14*					h_6	1575
	h		L_{2a}	710	alfa	30					b	215
			L_{2b}	200							b_n	500
											h_n	150
												15

*Dimensions changed between Analysis 1 and Analysis 2, the first value for Analysis 1.

A beam element was used for modelling of the screw since it was considered appropriate for achieving the desired results. A beam element is less advanced as solid elements, which is more time sufficient when running analyses in Abaqus. In Abaqus the beam element is one-dimensional, meaning stresses can only occur in the longitudinally direction of the beam. Other stresses is not of interest in the analysis of the screw, making this simplification relevant.

After placing the screw in the connection, the screw is given an extra torque moment to improve the stiffness of the connection. To simulate torque moment in the screw, a specific tool in Abaqus simulating thermal expansion were used, called predefined fields. The input is then a temperature. Using a beam element simplifies the use of predefined fields. Calculation wise the torque moment is equalized with pretensioning, where a certain stress is applied onto the screw. To find the temperature to use as input values for the predefined temperature field corresponding to a stress, following relationship between stress and temperature is used, see Equation 5.1.

$$\sigma = E\alpha_e \Delta T \tag{5.1}$$

where $\alpha_e = 12 \cdot 10^{-6}\frac{1}{K}$ and $E = 210GPa$

5.3 Mesh elements

The results obtained from the FE model are highly dependent on the mesh. To be able to generate a reliable model the meshing were created manually. The different parts of the model were partitioned to make sure the mesh could be finer in areas important for the results. These areas were the edges of the notch, both in the timber and the concrete parts, and in the screw. When manually creating the mesh, the aim was to form structured steam-lined hex-elements, or sweep hex-elements in the areas around the screw, see Figure 5.3.

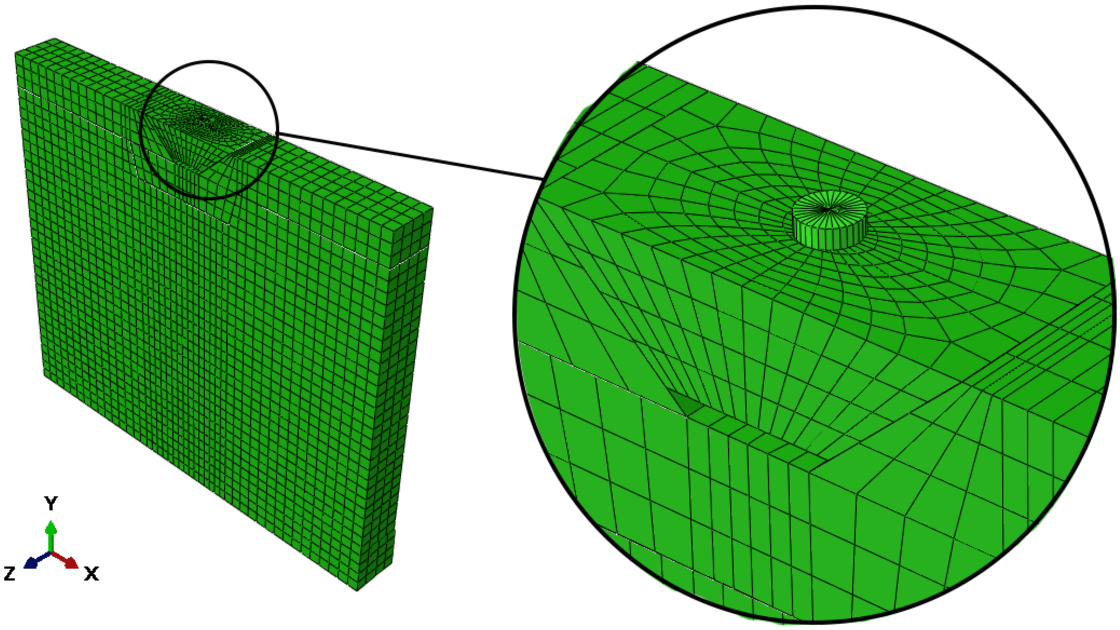


Figure 5.3: The mesh. Detail shown for the wedge and the screw specifically.

5.3.1 Convergence study

It is of importance in the analysis that the size of the mesh elements generates accurate results, and at the same time gives an effective and fast analysis of the model. A high-density mesh will produce results with high accuracy, but if too dense it will require a large amount of computer memory and long run times. To meet these requirements the elements should be as big as possible. In the process of finding the ultimate mesh size, a convergence study was made by extracting results in the model for different sizes.

For the convergence study it is relevant to extract some values that is of interest and compare them when using different mesh sizes. Maximum stress in screw and maximum stress perpendicular to grain in timber at the notch was chosen. The results of the reaction force was also of interest, but this part of the model were independent of the mesh size since the force were taken over the whole cut area (including all the elements in the area). The results obtained and how they converged are presented in Table 5.3. The final mesh size were set to 0.05 and was to be used for further analyses.

Table 5.3: Mesh convergence study

Mesh size	Timber stress [MPa]	Change [%]	Screw stress [MPa]	Change [%]
0.1	1.79	-	323	-
0.075	1.17	34.6	322	0.3
0.06	1.03	15.4	321	-
0.05	0.98	4.8	321	-

5.4 Boundary conditions

The boundary conditions of the model refers to how specific nodes are fixed or not in different directions, alternatively nodes being prescribed with certain displacements. The slip of the connection is simply simulated by applying a prescribed displacement at the short side of the timber part, in the same time as the left concrete part is fixed in all directions (see Figure 5.4 and 5.5).

Although the FE model is not fully a local part of the global bridge, but a model to simulate the slip, boundary conditions BC3 and BC4 have been added having the bridge in mind. These boundary conditions is added to improve the function of the model. The glulam beam is considered continuing in x-direction, why the right short side of the model is fixed in y-direction preventing this side to "fall down" (see Figure 5.6). The concrete slab is locked in z-direction, as the concrete slab is continuing in z-direction (see Figure 5.7).

The method used for modeling the slip is called deformation controlled load application and is useful since the force is unknown for every millimeter displacement. The load was introduced by prescribing a deformation using the boundary conditions. The movement results in resisting forces in the body, correlating to the forces needed to make the displacement. This way of simulating the slip could be compared to the asymmetrical test set-up shown in Figure 2.5 on page 9.

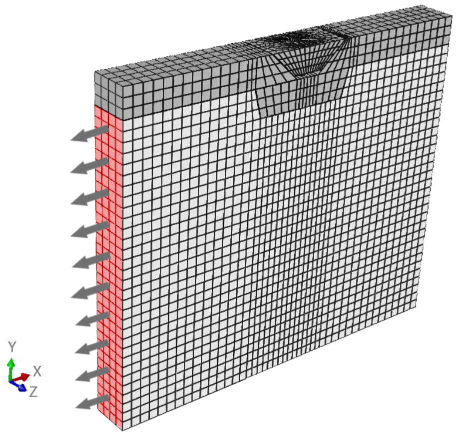


Figure 5.4: BC1, prescribed displacement

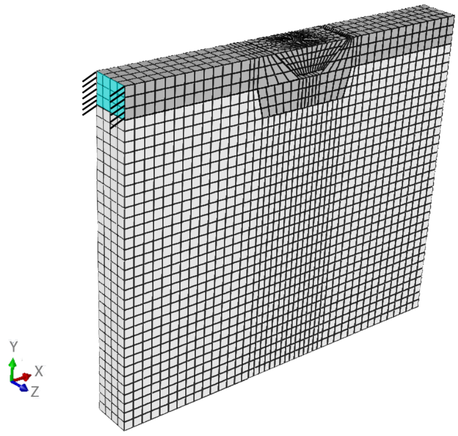


Figure 5.5: BC2, fixed support

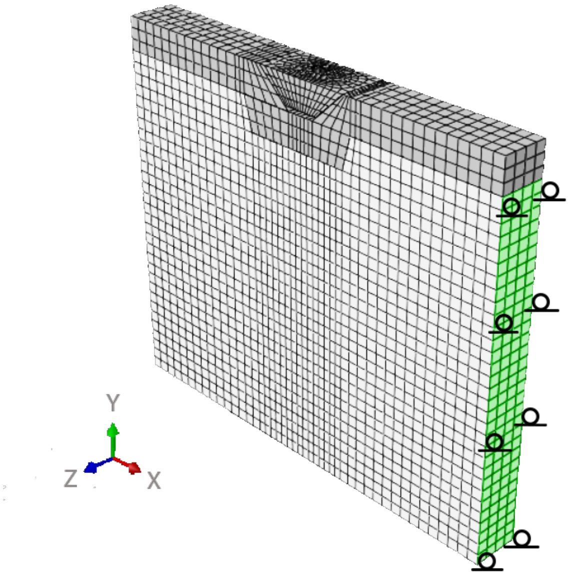


Figure 5.6: BC3, fixed in Y-direction

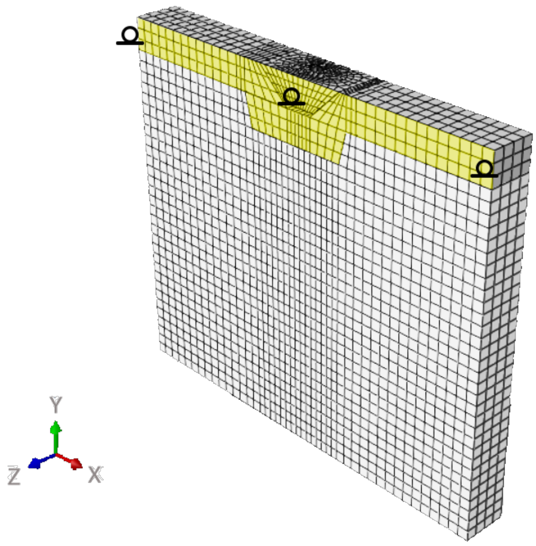


Figure 5.7: BC4, fixed in Z-direction

5.5 Material properties

The behaviour of a model is highly dependent on the material properties. To simplify the model this analysis was based on linear behaviour of all the materials except for the screw. The properties significant for the analysis is the density and the elastic properties, which could be seen more specific in Table 5.4 - 5.6. For the screw some additional properties were of importance. The *expansion coefficient* (used in Equation 5.1), which is needed since the pretensioning of the screw were simulated with thermal expansion. The *plastic properties* is also required since the aim with the connection was to cause yielding in the screw. Thus, the screw was modeled as a plastic material.

Table 5.4: Table of concrete properties

Concrete properties	
Parameter	Values
Density	2600 kg/m^3
Young's modulus	70 MPa
Poisons ratio	0.2

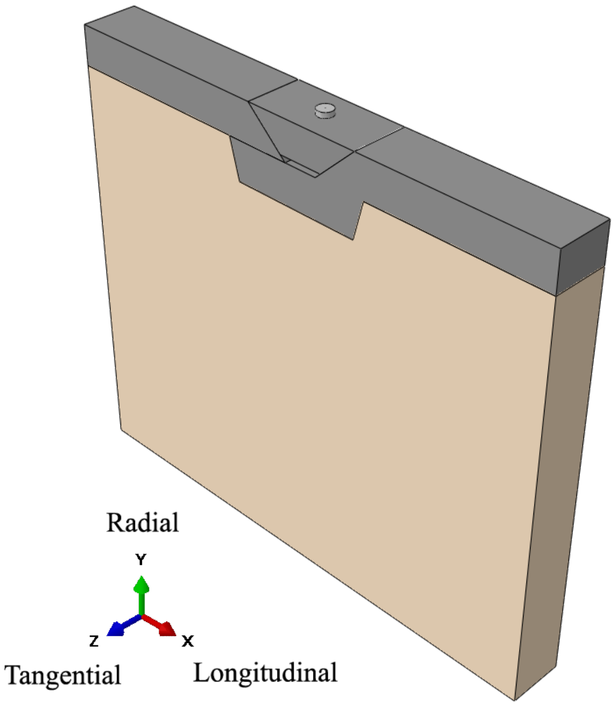


Figure 5.8: The fibre directions of the glulam beam shown.

Table 5.5: Table of timber properties

Timber properties GL32c	
Parameter	Values
Density	420 kg/m^3
E_L	13.7 GPa
E_T	4.2 GPa
E_R	8 GPa
ν_{LT}	0.03
ν_{RT}	0.35
ν_{LR}	0.04
G_{LT}	0.4 GPa
G_{LR}	0.78 GPa
G_{RT}	0.03 GPa

Table 5.6: Table of steel properties

Steel properties	
Parameter	Values
Density	7800 kg/m^3
Young's modulus	210 GPa
Poison's ratio	0.3
Expansion coeff.	$1.2 \cdot 10^{-05}$

A bi-linear stress-strain curve was used for input values, including yield stress, yield strain, ultimate stress and ultimate strain. The bi-linear model is very approximate, especially for the plastic part. This is considered enough, as it is just the level of when the screw starts to yield that is of interest and not the plastic behaviour itself. The different screw qualities used is presented with its values of stress and strain in Table 5.7, based on ISO 898-1, Mechanical properties of fasteners made of carbon steel and alloy steel Appendix A (ISO, no date). They are plotted in graphs in Figure 5.9.

Table 5.7: Stress-strain values for screw qualities 4.6, 5.8 and 8.8

Class	Yield stress *[MPa]*	Yield strain [-]	Ultimate stress [MPa]	Ultimate strain [-]
4.6	240	0.0011	400	0.22
5.8	420	0.0020	520	0.22
8.8	660	0.0031	830	0.12

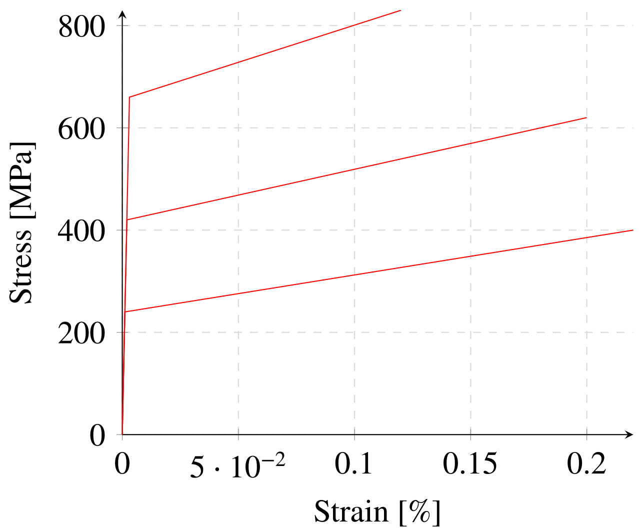


Figure 5.9: Stress strain relations for screw qualities 4.6, 5.8 and 8.8

5.6 Interaction

The interaction module in Abaqus let the user control how materials in contact interacts with each other. Interactions between timber and concrete is modelled with a friction coefficient of 0.62 in tangential direction, according to Ronca, Gelfi, and Giuriani, 1991, while the behaviour in direction of the normal is modelled by hard contact relationship. A hard contact relationship is used to transmit the contact pressure between the surfaces. Interactions between two concrete elements, is modelled equally with the difference that the friction coefficient is set to 0.4. The surfaces of different interaction is shown in Figure 5.10, where the left figure displays the area of interaction between timber-concrete and the right figure the area of interaction between concrete-concrete.

To simulate how the screw is connected to the glulam beam a tie constraint was used. The tie constraint gives the nodes in contact the same displacement and rotation. In Figure 5.11 the beam and the screw, highlighted in red, is shown. All the nodes in one partition of the beam is highlighted as well, where the screw is tied to those within a distance of 50 mm.

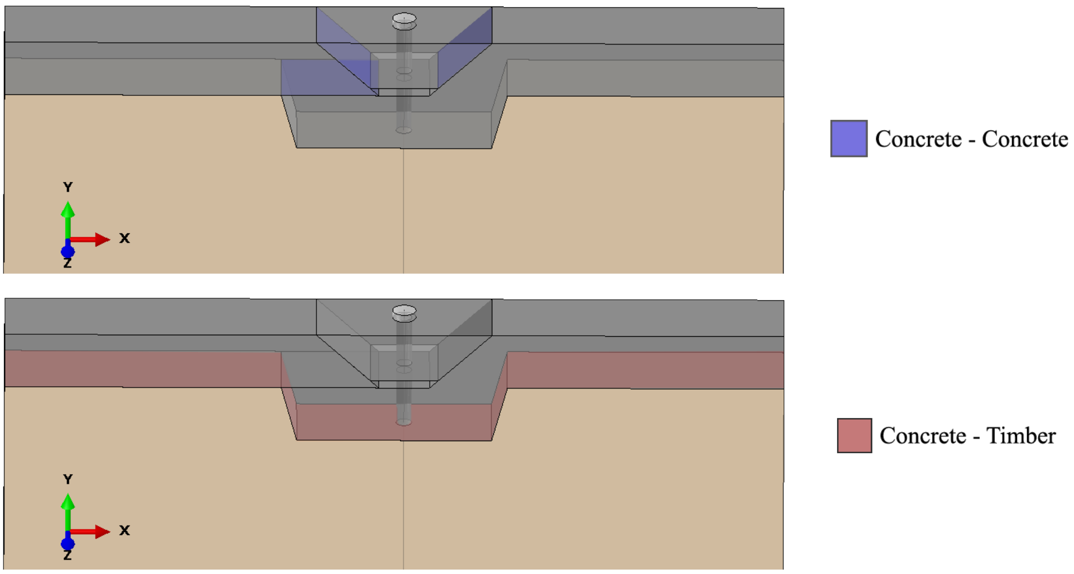


Figure 5.10: Interaction surfaces. Between timber-concrete to the left and concrete-concrete to the right.

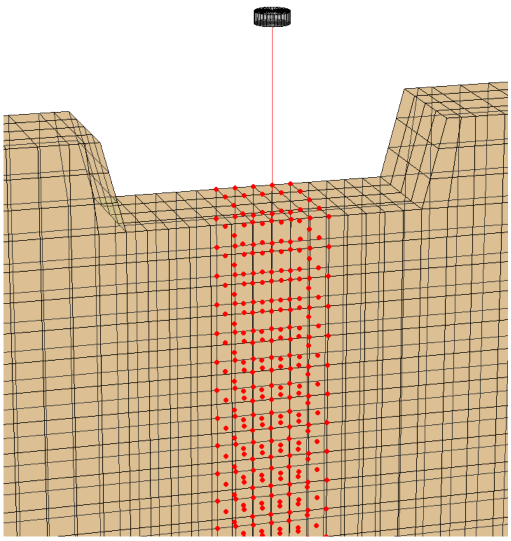


Figure 5.11: The screw connected with a constraint to nodes in the timber, within a distance of 50 mm.

5.7 Time steps

The step-module makes it possible to define which calculation to be carried out in which step. A predefined temperature field, PF1, is created in the initial step with a temperature set to 0, while no boundary conditions is created. PF1 is propagated through all steps, as another predefined temperature field, PF2, is created in Step 1 to simulate pretension in screw as described in Section **??**. In the same step a boundary condition is set for left short side of the timber, consisting of free movement in x-direction and locked in y and z. PF2 is propagated in Step 2, where also the prescribed displacement of 1 mm is set, to simulate slip in the connection. This division is made to simulate that pretensioning occurs before the connection is loaded. Thereafter the simulated slip is increased by 1 mm per step, up to 10 mm at the last step. The time steps for which yielding in the screw was expected, was divided into 10 increments each by using fixed increment size, this to increase the accuracy of the results. Remaining time steps was divided into increment sizes decided automatically by Abaqus.

Table 5.8: Clarification of time steps

Step	Thermal field	Displacement
Initial	PF1 created	-
Step 1	PF1 propagated, PF2 created	U1= free, U2= 0, U3= 0
Step 2	PF1 propagated, PF2 propagated	U1=prescribed displ. 1mm , U2=0 , U3 =0
Step 3 - Step 11	PF1 propagated, PF2 propagated	U1=prescribed displ. 2-10mm , U2=0 , U3 =0

6 Results

As mentioned in Chapter 5 the main purpose of the FE analysis was to reach a high stiffness of the connection, since this gives the bridge its total composite action. Stiffness can be described as the relationship between the force (F) needed to entail a specific slip (δ), see Equation 2.1 in Chapter 2.4.2. As explained, the stiffness corresponds to the slope in a force-displacement diagram. The relationship between F and δ were analyzed in Abaqus by creating a body force output for the left short side of the concrete, i.e the fixed side as shown in Figure 5.5. This gave the required force for a prescribed displacement representing the slip between the timber and the concrete. When determining the stiffness, results from the elastic part of the force-displacement curve was used.

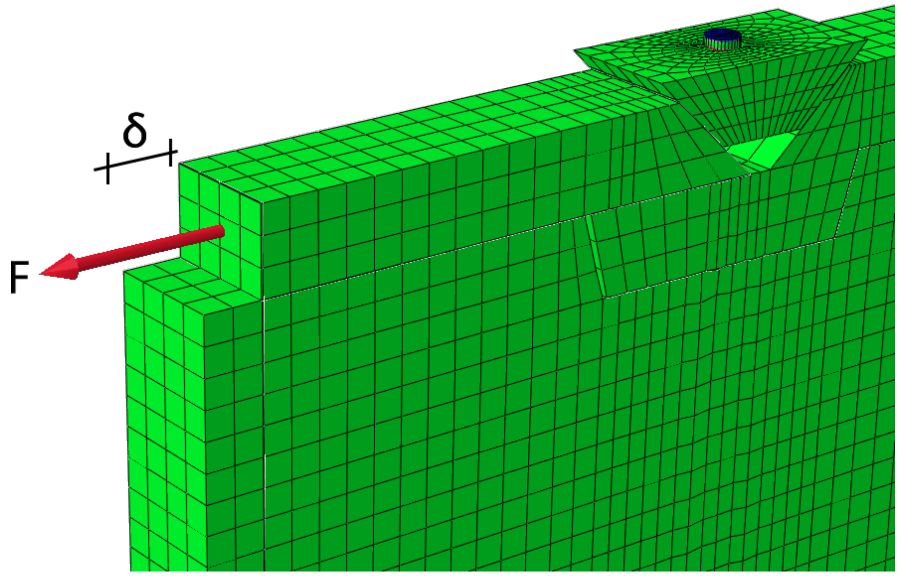


Figure 6.1: The Abaqus model showing the body force (F) and the displacement (δ) of the timber beam.

A prerequisite to reach a valid stiffness, is to have a connection which parts is loaded sufficiently and has a ductile behaviour. A challenge with timber is its anisotropic material properties, were it has the highest capacity parallel the fibres. The prescribed displacement on the beam will be in the direction of the grain, but since the connection has an inclined geometry stresses will spread in different directions. Consequently, the inclination of the notch will have a major impact on the capacity of the connection. For Analysis 1 a high inclination was used, which was decreased in Analysis 2. It was of importance in both the analyses to investigate in which direction the stresses in the timber was largest, and how they correlated to the capacity of the material, see Equation 6.1 - 6.4. The intention was to reach as high utilization of the material as possible, and this was calculated for the direction in which the stresses were most critical.

$$f_{c90d} = \frac{f_{c90k}}{\gamma_M} k_{mod} = \frac{3}{1.25} \cdot 0.6 = 1.44 MPa \tag{6.1}$$

$$f_{t90d} = \frac{f_{t90k}}{\gamma_M} k_{mod} = \frac{0.45}{1.25} \cdot 0.6 = 0.22 MPa \tag{6.2}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_M} k_{mod} = \frac{26.5}{1.25} \cdot 0.6 = 12.72MPa \tag{6.3}$$

$$f_{td} = \frac{f_{tk}}{\gamma_M} k_{mod} = \frac{19.5}{1.25} \cdot 0.6 = 9.36MPa \tag{6.4}$$

Besides the stresses in the timber beam, the stresses in the screw were of importance since the aim was to reach yielding in the screw before reaching the full capacity of the timber. To obtain relevant outputs, a prescribed displacement of 10 mm was needed to show both the elastic and plastic behaviour of the screw clearly. However, it is important to not put any value in the plastic part since the modelling of the plastic part is very approximate. The intention was just to clearly see where yielding occurs. In each analysis the first check was to see at what displacement, i.e. slip of the connection, the screw started to yield. Not until this was reached the connection had a satisfying behaviour. For the same displacement the timber stresses was checked whether they exceeded the capacities or not, from what the connection test was classed to be possible or not possible. Thereafter the stiffness could be calculated, to see the efficiency of the connection.

Laboration of different input values parameters was made in the different tests, to find a good combination within each analysis. These parameters are presented in Table 6.1 below. The parameter Yield strength is decided by the screw quality, where the used ones is introduced in Table 5.7 in Section 5.5. Pretensioning is calculated by Equation 5.1 and originates as a torque moment, while the screw diameters used were based on standard dimensions.

Table 6.1: Parameters in tests

Screw diameter	Pretensioning	Yield strength

6.1 Analysis 1 - 30° inclination

After estimations based on the need for the screw to yield, before timber has reached its full capacity in compression perpendicular to grain, an inclination of 30° in the concrete wedge was chosen (see Figure 6.2). This dimension was specific for Analysis 1 and the differentiating factor in between the different analyses. As mentioned earlier the inclination has a large impact on the behaviour of the connection since it spreads the stresses in different directions. By using an inclination of 30° the expectation was to create a vertical component acting on the wedge, large enough to move the wedge upwards and fulfill the desired behaviour (see Figure 4.3-4.5). To be able to run a first simulation and get an idea of the behaviour of the connection, start values were chosen. From the start values, see Table 6.2, the analysis were then to be further developed.

Table 6.2: Start values Analysis 1

Parameters	
Screw diameter [mm]	20
Pretensioning [MPa]	168
Yield strength [MPa]	660

The first simulation, Test 1, showed satisfying results with a movement of the wedge upwards as a result of the slip, see Figure 6.2. As a consequence a big tension force was created in the screw, as desired. However, the movement at the same time increased stresses in the notch perpendicular to the grain. Since the capacity of timber is significantly lower perpendicular to grain, these areas were paid attention to and further investigations were performed. The critical areas are shown in Figure 6.3. For other directions and parts, the values of the stresses were well below the capacities for the different materials and considered not in need of further investigations.

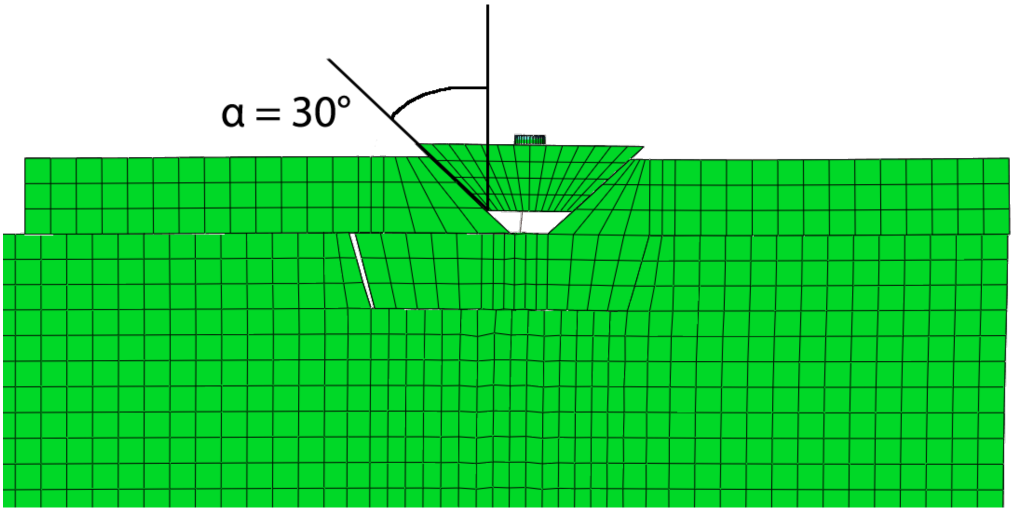


Figure 6.2: The 30° inclination of the wedge making it move upwards.

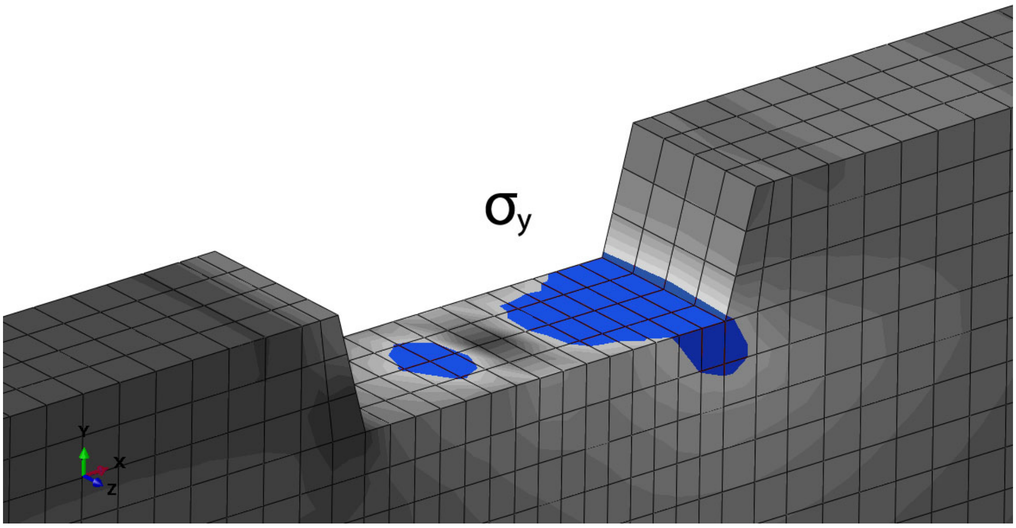


Figure 6.3: Critical areas of the timber beam shown in blue, compressive stresses acting perpendicular to grain.

Based on the results from the start values, the intention was to, in an iterative process, change the parameters in three more tests depending on the result from the previous test. This to develop a connection with as high stiffness as possible, see the procedure of changing parameter in Figure 5.1. The results from the four iterations is presented together in Table 6.3. For Test 1 the stresses in the critical part of the timber reached an utilization of 56 % at the time were the screw reached yielding. This means acceptable results locally in the different parts, but there is a possibility to utilize the timber more. Analyzing the stiffness the results was not as satisfactory, reaching a value of 52.6 kN/mm. A higher stiffness was desired, and since the utilization of the timber was relatively low there were more possible capacity in the connection.

Test 2 was performed trying to increase the stiffness, by changing the screw diameter from Test 1. The diameter was increased to 30 mm, with the intention to increase the stiffness and increase the utilization of the timber perpendicular to the grain. The result was satisfactory concerning the stiffness which increased from 52.6 kN/mm to 78 kN/mm, see Table 6.3. However, the increased stresses in the timber proved to exceed its capacity, making the connection with these parameters not acceptable since a brittle fracture would occur.

The process proceeded by performing Test 3. Parameters was chosen to decrease the stresses in the timber. The diameter from Test 2 was kept, instead the screw quality was changed to investigate its impact on the behaviour. Previously used screw quality 8.8 with yield at 660 MPa was changed to screw quality 5.8 with yield at 420 MPa. This resulted in satisfactory stresses in the timber, with an utilization factor of 88% in the most utilized direction of the timber. This is explained by an earlier yield in the screw. Regarding the stiffness for this iteration, it did as expected not differ from Test 3. For Test 4, the pretensioning of the screw was increased with the intention to reach a higher stiffness. It was understood that the change would increase the stresses in the timber as well. The pretensioning stress was increased to 378 MPa, which corresponds to 90% of the yield stress for screw quality 5.8. In summary, the test showed good results; a higher stiffness was reached while the timber had an utilization of 92%.

The results from the four iterations is declared in Table 6.3, were the changed parameters in each test are marked in red. Additionally, the force-displacement graphs of the tests is presented in Figure 6.4 to get an overview of how the stiffness increased from Test 1 - Test 4, i.e. how the slope of the curve increases. The table shows the elastic part of the load-slip diagram, as this is the part used to calculate the stiffness. A specific load-slip diagram presenting the full simplified bi-linear behaviour of both elastic and plastic parts, is plotted for Test 4 in Figure 6.5. The graph shows that an elastic behaviour is present up to a displacement of 4 mm, when yielding of the screw occurs and the plastic behaviour takes place.

Table 6.3: Results of tests in Analysis 1 (changed parameters marked in red)

		Test 1	**Test 2**	**Test 3**	**Test 4**
Input values	Screw diameter *[mm]*	20	30	30	30
	Pretensioning *[MPa]*	168	168	168	378
	Yield strength *[MPa]*	660	660	420	420
Results	Displacement at yield *[mm]*	5	8	5	4
	Force at displacement *[kN]*	263	625	397	396
	Stiffness *[kN/mm]*	52.6	79	79	99
	Stress in timber σ_{c90} *[MPa]*	0.783	1.94	1.24	1.3
	Utilization timber *[-]*	0.56	1.38	0.88	0.92

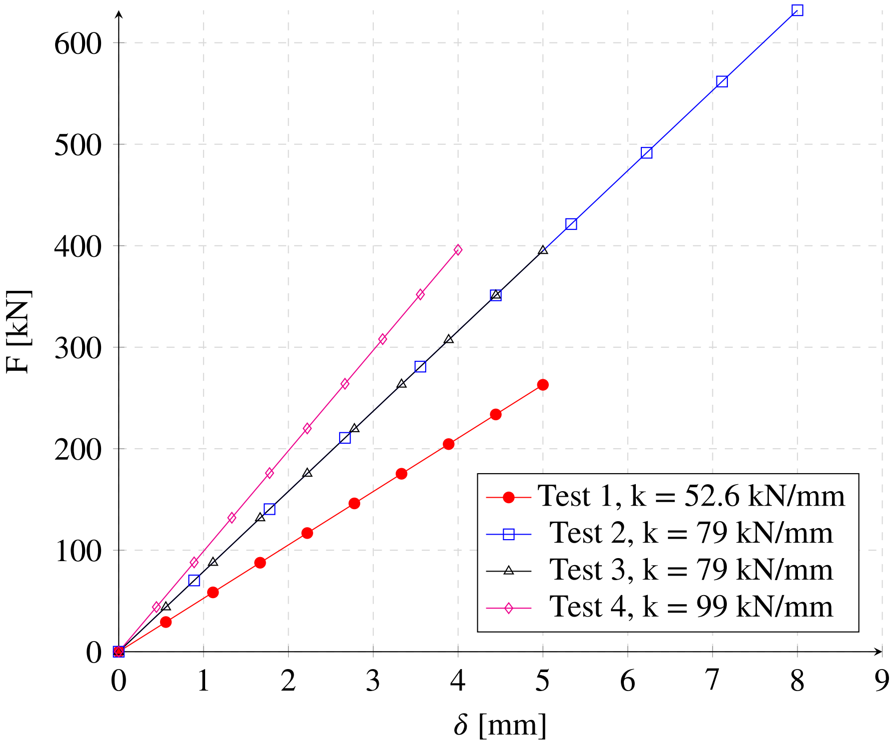


Figure 6.4: The load slip relationship for Test 1 - 4 (only elastic part used for calculation of stiffness shown)

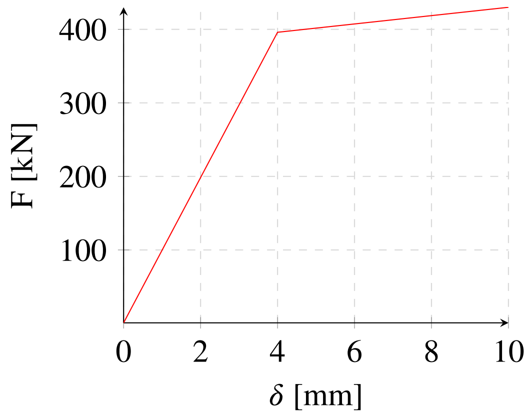


Figure 6.5: The load slip curve of the most efficient connection from Analysis 1.

In summary, the results of Analysis 1 showed a connection with a reasonable stiffness and adequate utilization of materials. The behaviour of the connection in Test 4 is shown in Figure 6.6-6.7, illustrating the FE model with its deformation, stresses and displacements. For a better understanding, a scale factor 25 is used for the figures, displaying deformations 25 times bigger than the corresponding result. In Figure 6.6 the colour of the model illustrates the stresses parallel to the connection, i.e in x-direction. The stresses is taken at the step corresponding to when the screw starts to yield. Stress values representing respective colour is explained in the box to the left. Displacements in x-direction is shown in Figure 6.7 where 4 mm displacement is shown for the timber, which is when the screw started to yield.

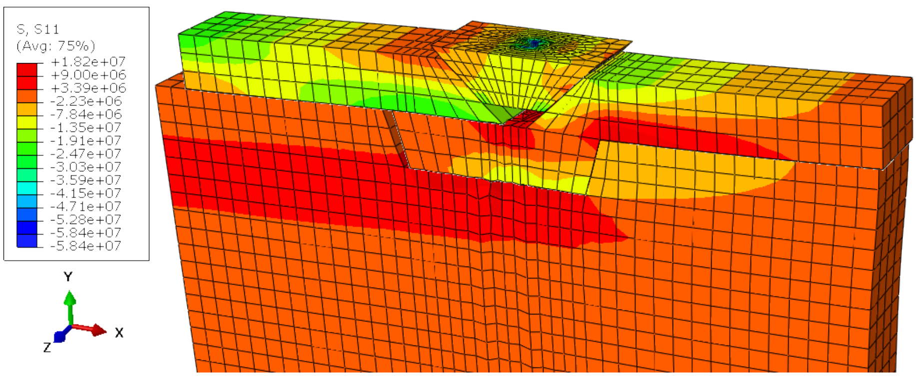


Figure 6.6: Deformed connection at a prescribed displacement of 4 mm.
Colours showing stresses in x-direction (S11)

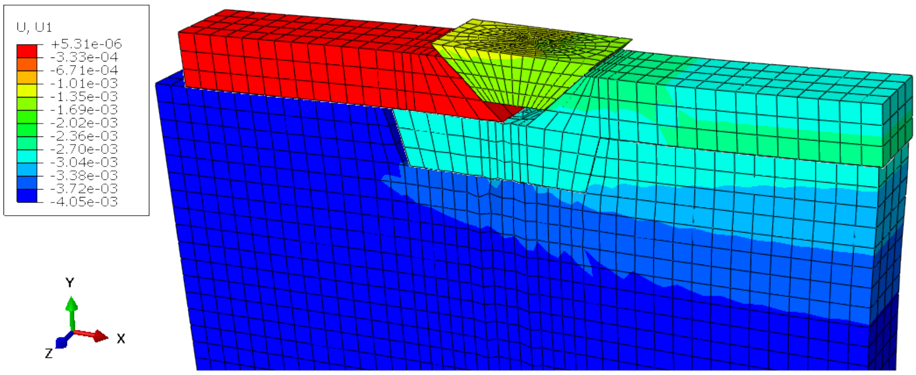


Figure 6.7: Deformed connection at a prescribed displacement of 4 mm. Colours showing deformation in x-direction (U1)

6.2 Analysis 2 - 15° inclination

The change for Analysis 2 concerns changing the inclination of the notch. By decreasing the inclination (see Figure 6.8), the stresses in the connection will be distributed differently compared to Analysis 1. The vertical component acting on the wedge will be decreased resulting in smaller stresses in the screw, and smaller stresses in the timber perpendicular to the grain. Instead the horizontal stresses will increase in the strong direction of the timber, which hypothetically will result in a higher stiffness of the connection. To investigate the behaviour, new tests were performed with a new inclination of 15°. The changing parameters was the same as in Analysis 1, i.e. the screw diameter, yield strength and pretensioning.

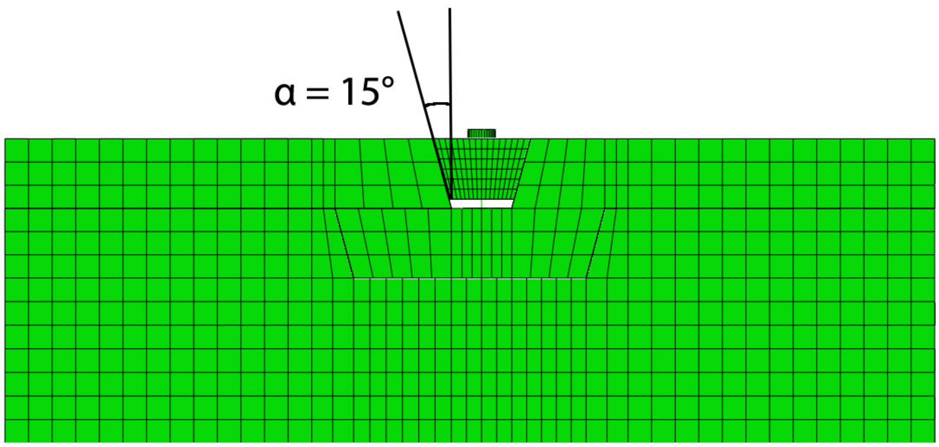


Figure 6.8: 15° inclination of the wedge.

The initial test in Analysis 2 was executed with input values corresponding to Test 4 in Analysis 1, presented in Table 6.4. The results from this first simulation had the intention to describe the behaviour of the connection, and which areas and capacities that was critical and in need of further investigation.

Table 6.4: Start values Analysis 2

Input values	
Screw diameter [mm]	30
Pretensioning [MPa]	378
Yield strength [MPa]	420

As expected, Test 1 showed that the stresses in the screw decreased compared to the results from Analysis 1, while stresses in x-direction in the timber increased. As a result, the stiffness increased due to larger resisting forces in the x-direction. Figure 6.9 illustrates the movement of the wedge which were pulled downward due to the pretensioning force. In the following steps, the forces from the slip never reach a value high enough to move the wedge up again. Consequently, the test resulted in no yield in the screw at all after reaching the maximum displacement of 10 mm. Before this was reached the timber had exceeded its capacity in all evaluated directions. The connection was far from acceptable.

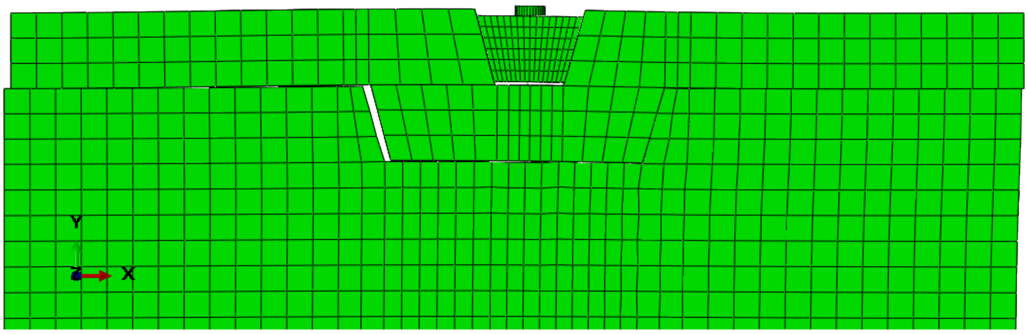


Figure 6.9: The wedge pulled down by the pretensioned force.

Unlikely from Analysis 1, the most critical capacities proved to be tension stresses perpendicular to the grain (σ_{t90}) and compression stresses parallel to the grain (σ_c). The location of the worst stresses of σ_{t90} is presented in Figure 6.10, where the capacity is very low and therefore also low stresses became critical. As the grain direction is in x-direction, the mentioned perpendicular stresses acts in y-direction. The most critical areas for compression stresses parallel to the grain (σ_c) is shown in Figure 6.11. Due to the smaller inclination of 15° in Analysis 2, more stresses were acting in x-direction, while less stresses were transferred upwards to the screw. Compression stresses perpendicular to the grain (σ_{c90}) seemed not to be critical in Analysis 2, but values was checked and presented to be on the safe side. However, the stresses in the timber were considered irrelevant in this first test since the first step in the analysis, yielding of screw, was not fulfilled and thus gave an unsatisfactory behaviour of the connection.

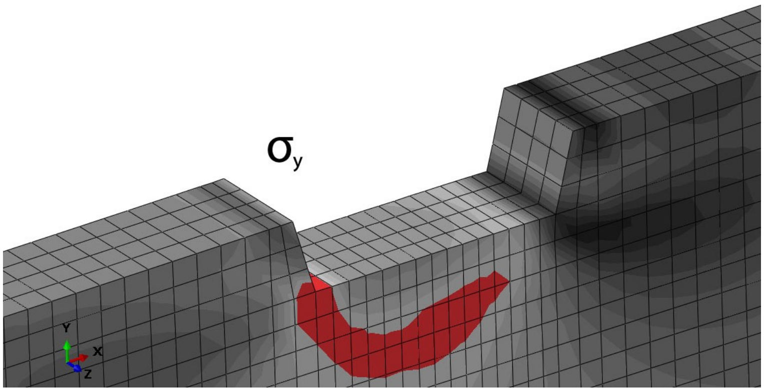


Figure 6.10: Critical area of the timber beam shown in red, tensile stresses acting perpendicular to grain.

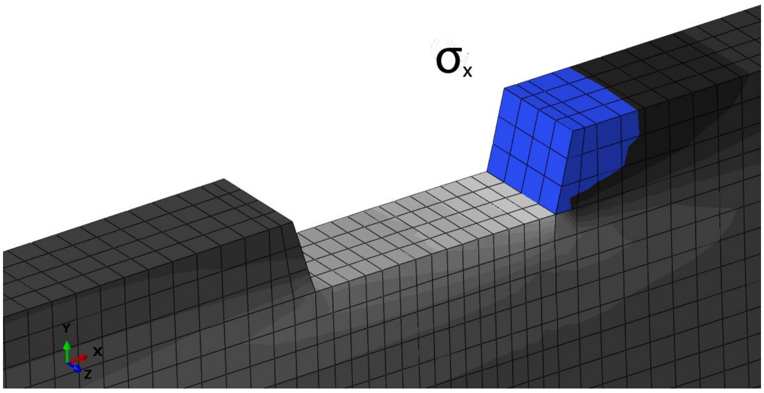


Figure 6.11: Critical area of the timber beam shown in blue, compression stresses acting parallel to the grain.

Test 2 was performed with the intention to fulfill the first criteria, reach yielding in the screw, and to decrease the forces acting on the timber. The changed parameter to fulfill this requests was the screw diameter that was decreased from 30 mm to 20 mm. The simulation showed yielding in screw starting at a displacement of 9 mm. Regarding the timber, the results however showed too large stresses in all directions. Tension perpendicular to the grain (σ_{t90}) was the most exposed, corresponding to an utilization factor of 740%, which was far away from satisfactory. Thus, Test 2 did not fulfill the requirements. A high stiffness could be determined, but not of value since the desired behaviour with yielding of screw before timber reaches its full capacity was not accomplished. Similar to Test 1 the wedge was not pushed upwards, but downwards instead, which further proved the connection not to be satisfactory. A notation is that the screw seemed to be too strong for the connection, leading to yield at a large displacement (9 mm) and large stresses increasing in the timber. The results could be seen in Table 6.5.

Based on the conclusions from Test 2, the screw quality was decreased in Test 3 to obtain an earlier yield in screw. Screw quality 4.6, with yielding at 240 MPa was tested. At the same time the pretension force was decreased accordingly. The simulation of the test resulted in a bit lower stiffness, and a yielding of the screw was reached at 5 mm displacement. However, this displacement created stresses in the timber that were significantly high. (σ_{t90}) corresponds to an utilization factor of 407%, and (σ_c) marginally under that. The simulation also showed that the behaviour of the connection were insignificantly changed in between the three first iterations. The movement of the wedge were still mostly dependent of the pretension force, and not affected by the displacement. Figure 6.12 shows the deformation of the connection with a scale factor and gives an understanding of how the stresses are transferred through the connection.

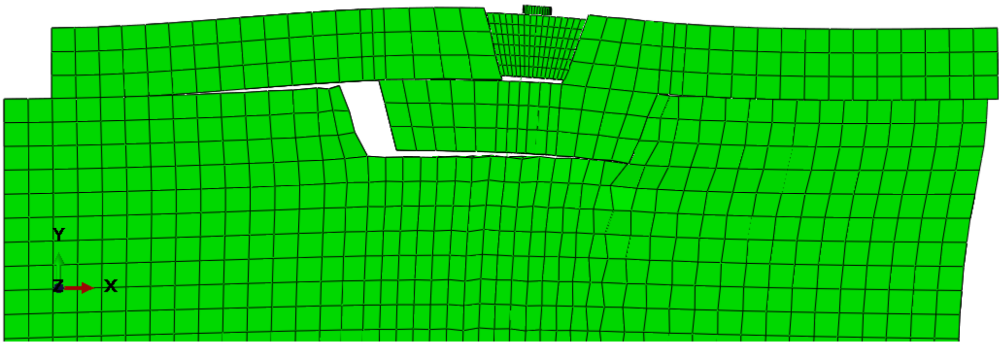


Figure 6.12: The deformation with a scale factor 10 of the connection analyzed in Test 3.

For the last test, Test 4, the intention was to see if the movement of the wedge could be altered by changing some of the parameters, and by that change the stresses in the timber. The screw diameter was further decreased to 16 mm and the torque moment was completely removed to contribute to an easier movement of the wedge. The simulation proved this to work, but the movement upward of the wedge was negligible small and did not give a desirable behaviour of the connection. In Table 6.5 the results from all four iterations are presented.

Table 6.5: Results of tests in Analysis 2 (changed parameters marked in red)

		Test 1	**Test 2**	**Test 3**	**Test 4**
Input values	Screw diameter *[mm]*	30	20	20	16
	Pretensioning *[MPa]*	378	378	168	0
	Yield strength *[MPa]*	420	420	240	240
Results	Displacement at yield *[mm]*	-	9	6	5
	Force at displacement *[kN]*	-	2026	1314	1043
	Stiffness *[kN/mm]*	-	225	219	208.6
	Stress in timber σ_{c90} *[MPa]*	-	1.24	0.73	0.67
	Stress in timber σ_{t90} *[MPa]*	-	1.6	0.88	0.71
	Stress in timber σ_{c} *[MPa]*	-	77	42.3	40
	Utilization timber* *[-]*	-	7.4	4.07	3.28

*The utilization ratio refers to the highest utilization ratio out of the capacities; which are σ_{c90}, σ_{t90} and σ_c.

As a result, Test 4 entailed in timber stresses still being considerable high, exceeding the capacity over 3 times. The overall behaviour of the model in Test 4 is illustrated in Figures 6.13 - 6.16, where stress values and deformation values is presented for respective colour in the box to the left of the figures. Figures 6.13 and 6.15 shows the stresses in x-direction develop at deformations shown in 6.14 respective 6.16, where the first is at 2.6 mm displacement and the second at 5.2 mm, which is where the screw starts to yield.

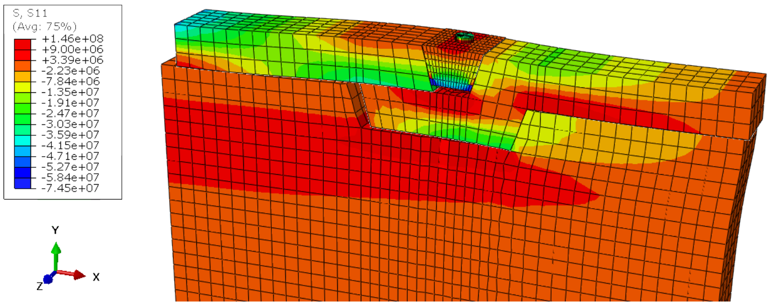


Figure 6.13: Deformed connection at a prescribed displacement of 2.6 mm. Colours showing stresses in x-direction (S11)

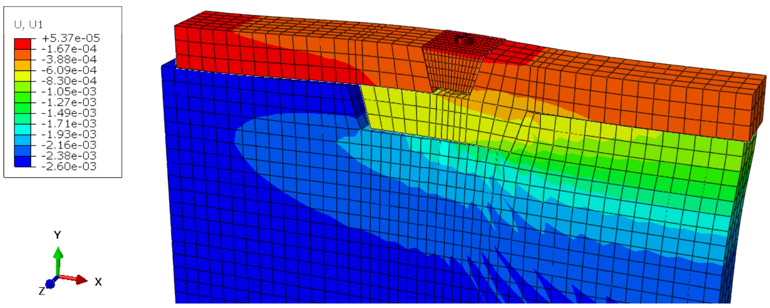


Figure 6.14: Deformed connection at a prescribed displacement of 2.6 mm. Colours showing displacements in x-direction (U1)

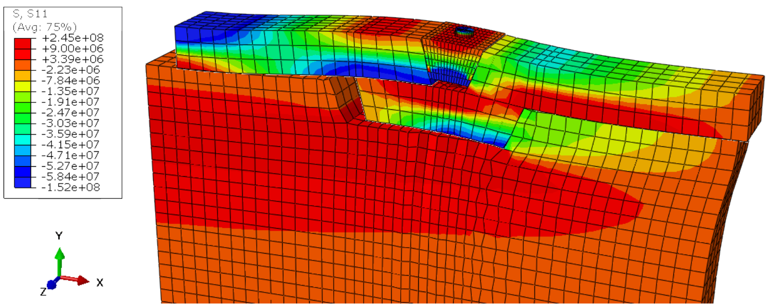


Figure 6.15: Deformed connection at a prescribed displacement of 5.2 mm. Colours showing stresses in x-direction (S11)

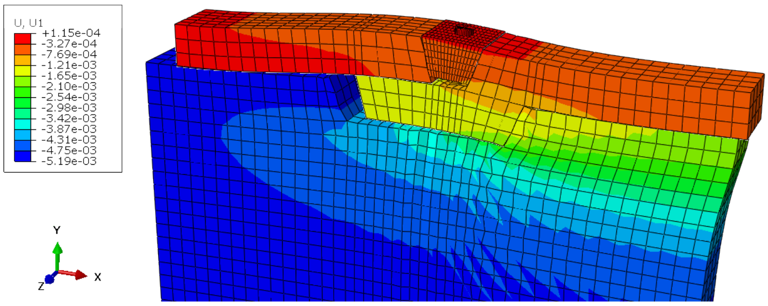


Figure 6.16: Deformed connection at a prescribed displacement of 5.2 mm. Colours showing displacements in x-direction (U1)

7 Discussion

In this chapter simplifications made during the project are discussed, followed by uncertainties in the FE model and a general discussion of the results. Finally, limitations of the work are highlighted together with proposals for further studies.

7.1 Simplifications

During the work with this project the FE modeling proved to be a very time consuming part. Some simplifications were made to decrease this time, which were considered not to affect the results significantly. However, they are important to mention. One of the major simplifications made was the modeling of the screw and how it is connected to the timber, which is not fully comparable to a real case. The screw was modeled as a beam and connected to nodes within a radius of 0.05 m in the timber beam. This means that the embedment capacity of the timber when the screw is loaded axially is not investigated in the result i.e. the connection between the screw and the surrounding timber is infinite. This was not considered affecting the results, since the importance in the analysis is to investigate how large the stresses are in the screw and the behaviour of the connection. The embedment capacity of the timber can be handled in a later stage when setting the final dimensions for the connection.

The stress-strain relations for the materials timber and concrete is highly simplified as the materials is modelled with a linear stress-strain relation. Though, this simplification does not affect the results significantly as the stresses in the materials are controlled to be under the limit for plastic behaviour. Meanwhile, the steel screw is modeled with a bi-linear stress-strain curve, which is also simplified. However, it is mostly the point of yielding which is of importance in this study, not the plastic behaviour itself.

The boundary conditions chosen for the FE model are also a simplification of reality. BC3 and BC4, as shown in Figures 5.6 and 5.7 in Chapter 5 is fully locked in one direction each, while the two other directions are fully free. The locked BCs are a result of the elements continuing in that direction. In reality, BCs are seldom corresponding to extreme values of fully fixed or fully free. Instead they are dependent on the response from the materials in contact, affected both by the displacement and the deformation of it. In x-direction the timber beam will follow the movement of the global slip, while in z-direction the concrete slab prevents the modeled part to bend out.

7.2 The FE model

There were a couple of uncertainties in the FE model which needs to be considered when evaluating the results. The FE model is modelled with friction coefficients for areas between concrete-concrete and concrete-timber, since the aim of this study was to create a model with results as comparable as possible to a real case. However, the effect of the friction were not handled locally at the surfaces of the materials. As the timber is anisotropic, some stresses could be increased by using friction factors, for example tension stresses parallel to the grains, σ_t. When the test results was studied, big stresses of σ_t just at the surface of the timber in the bottom of the notch was noticed. These stresses clearly arose due to the friction factor assigned in Abaqus, and need to be investigated further in the future.

Solutions for reinforcing the timber exists, making it acceptable to neglect those stresses in this stage.

Since the FE analysis was made to simulate a shear test of the connection, it is important to understand how the arrangement of the test set-up affected the results. Asymmetrical loading (see Figure 3.12) have been used in this model, which adds a moment to the model. This creates a compression force at the interface between the concrete and the timber, which in turn increases the friction and thus improves the mechanical properties. Due to this the stiffness from the test results shows a higher value than if a symmetric test set-up had been used. It is of importance when comparing results between different studies to take into account what kind of test set-up compared studies have been used.

The accuracy of the results received from the FE analysis is dependent on the number of increments chosen for the different time steps. Reasonable increment sizes were chosen, as a compromise between the level of accurate results and time efficiency. Smaller increments could have generated more accurate results. However, another reason for not using even smaller increments, is uncertainties of the model in general. Simplifications made will anyhow result in a certain lack of accuracy compared to real tests.

7.3 The results

The results from Analysis 1 is considered satisfying showing that the proposed connection is possible and behaves in a desired way. With the chosen dimensions, the simulation showed that the screw started to yield before other capacities were exceeded, giving the connection a ductile behaviour and fulfilling the first of two requirements. However, the second requirement of reaching a high stiffness, proved to be more difficult if comparing it with the reference project. The final stiffness of the four tests in Analysis 1 proved to be approximately 100 kN/mm. The stiffness reached in the reference project, The Quay, has a significantly higher stiffness of approximately 300 kN/mm, but is not fully comparable to the proposed connection in this work since the connections were cast into the concrete instead of assembling the elements on site. However, some interesting values for comparison can be found in the work of Lukaszewska, 2009. Two of her connections studied showed a very low stiffness of about 5.5 kN/mm, while the third tested connection resulted in a stiffness $k_{0.4}$ of 236 kN/mm. The difference is remarkably big between the SST+S connector and ST+S+N, according to Lukaszewska, 2009 mostly due to the notch. ST+S+N is also prefabricated, but an important matter is the difficulty in manufacturing of ST+S+N, making it much less manageable to use and even less possible to demount. As manageability and possibility to demount the connection is important matters for the connection developed in this work, it is reasonable to not achieve as high value as ST+S+N by Lukaszewska, 2009.

Comparing the received stiffness of this work with other connections, as the ones analyzed by Auclair et al., 2016 or Crocetti et al., 2014, the results can be considered more promising. In the mentioned research the achieved stiffness reached a maximum of 53 kN/mm respective 41 kN/mm. From that perspective, the result of a stiffness of 100 kN/mm achieved in this work can be considered satisfactory.

One of the major factors affecting the results and the plausibility of comparison is, besides the design of the structure for which the connection is used within, the method used for analysis. The investigated research show values based on performed real shear tests, while the results in this study are based only on FE analyses, which only gives theoretical results. Another important difference to reflect upon when comparing stiffness is what the designed connection are intended to be used for. The majority of the previously made research aims to be used in composite floors, which requires a lower stiffness than that for use in a bridge. This due to both larger spans and larger loads in a bridge.

To summarize, there are difficulties knowing what stiffness values that are sufficient for the connection proposed in this work, as previously made tests has very different presumptions and results. Values found are both markedly below, and above, the values achieved in this report as a consequence of differences in methods, scope of use etc. To get an idea of the sufficiency of the achieved stiffness, it needs to be implemented and investigated in a global model. The need of a global analysis is further discussed in Subsection 7.4.

The main purpose when starting Analysis 2 was to investigate if there was a possibility to increase the stiffness by changing the design of the wedge. The inclination was changed from 15° to 30° with the expectation to receive larger horizontal reaction forces increasing the stiffness. It turned out that when using this inclination the pretension force was unfavorable for the connection. The results shows that the movement of the wedge never became large enough to move the wedge upward. Regarding the stiffness the change was however efficient, since the stiffness reached is increased approximately 100% from Test 4 Analysis 1 to Test 4 Analysis 2. Nevertheless, all the tests in Analysis 2 has invalid results, as the stresses in timber show values high above the capacities for the displacement at which the screw starts to yield.

The results can be partly explained by comparing Figures 6.6-6.7 in Analysis 1 and Figures 6.13-6.16 in Analysis 2, showing stresses and deformations of the modelled connection. The figures of Analysis 2 shows a lift in y-direction of the left concrete element, which can not be seen in the figures of Analysis 1. The reason for the movement is the small inclination, proved to be too small by preventing the wedge to move upwards. Consequently, the compression forces between the left concrete element and the wedge are forcing the concrete element to bend. Increasing the prescribed displacement further, the concrete element moves upwards and creates a curved shape. Compared with Analysis 1, where a desired behaviour of the wedge is achieved, the concrete element can move further in x-direction as the wedge moves upwards. The movement of the wedge, leads to the desired plastic behaviour in the screw. In the same time though, this movement makes the connection more flexible, resulting in lower stresses between the materials corresponding to a lower received stiffness. The large stresses in Analysis 2 gives the connection a higher stiffness, which nevertheless can not be utilized due the failure in timber before the screw starts to yield. Based on these results, it is realized that a more comprehensive iteration process is needed to find the optimal inclination of the wedge and geometry of connecting elements. This is proposed in Subsection 7.4. An alternative, since both the analyses shows that the timber is the critical part of the connection, is to find methods of strengthening critical areas in the beam. This could improve the results from both the analyses, and perhaps make the the connection in Analysis 2 possible. However, without any major arrangements of strengthening contributions, an inclination of the wedge in between 30° and 15° is considered optimal.

One of the most important notations of this work, is the value in other parameters of the connection than those investigated in this study. The parameters changed was limited geometrically to the inclination of the wedge, and additionally by the grade of pretension, the yield strength and the diameter of the screw. The results achieved was the best combination of those parameters based on three iterations divided in two analyses. Changing other parameters or performing more iterations can result in a better connection. Especially the geometrical parameters have been reflected upon after carrying out the analyzes. For example, changing the depth of the notch, could hypothetically spread out the stresses acting on the critical timber areas, increasing the stresses in critical areas. Another thought based on the results of this project is if a bolt going through the whole timber beam can decrease the stresses in the timber, instead of using a screw embedded in the timber. An issue though is how to reach the bottom area during production or maintenance of the bridge.

To summarize, in both the analyzes the timber is shown to be the critical part of the connection. The two analyzes shows a different behaviour of the connection due the difference in the inclination of the wedge. Analysis 1 shows the most promising results by proving the proposed connection to be possible, with an acceptable stiffness. Analysis 2 shows higher values regarding the stiffness, but has unacceptable stresses in the timber. The limitations of the study have been reflected on which have given rise to proposals for future studies presented in the following section.

The results from this study does not provide a final connection with dimensions for direct implementation into construction of new bridges. However, in align with the aim, a concept have been developed and presented for how prefabricated concrete slabs could be used in TCC bridges. The study has been performed as a pre-study of a larger research project of industrial manufacturing of TCC bridges, with a greater purpose of enable shorter erection time of bridges and increase the use of timber in bridge construction. The contribution from this work towards the greater purpose, is that it has been proven that a connection for prefabricated concrete deck in TCC structures is possible. The stiffness reached was 100 kN/mm and suggestions for improvements have been stated.

7.4 Further research

Since this project itself does not provide a finish product to be used in the industry, further research is needed to reach a higher goal of using more timber in the construction of bridges. This study has investigated the behaviour and possibility for a connection between glulam beams and prefabricated concrete decks, and based on the findings from this work following futures research are proposed;

- **Perform a parametric analysis of the connection.**
  This study have been limited to do a few different tests of the connection for different input values. A parametric analysis could include more variable parameters and enable a more optimal solution. Parameters not accounted for in this study, which could be optimized is for example the depth of the notch and the length of the screw.

- **Perform real tests of the connection.**
  Real shear tests performed in laboratory is needed to verify the stiffness estimated from the FE analyses.

- **Investigate strengthening methods for the timber**
  The results from this work shows that the most critical part of the connection is the timber close to the notch. If methods for improving the strength in the notch was used, the connection proposed in this work could be functioning with a smaller inclination and therefore obtain a higher valid stiffness.

- **Investigate how the connection is affected by time.**
  The verification of a composite beam in the long term is problematic because of concrete creep and shrinkage, creep in timber and thermal strains of concrete and timber. Therefore durability of the connection under variations of temperature and humidity needs to be further examined, whether there is a risk for fatigue etc.

- **Apply the connection into a global model.**
  When found a definite stiffness of a connection, the next step will be to implement it into a global analysis where the stiffness is represented by springs with obtained stiffness value. Hence, it could be verified if the stiffness is enough or not and if the connection could be used for optimal composite efficiency of a structure.

8 Conclusion

To conclude this study, a new connection concept has been developed and analyzed by a FE analysis, which met the requirements regarding functionality and stiffness. Two analyses were performed with different geometry, were the highest stiffness were found in Analysis 1 reaching a value of 100 kN/mm. This includes accomplishment of a ductile behaviour, due to yielding in the steel screw before any failure occurs in concrete or timber. Analysis 2 were made as an iteration with the intention to increase the stiffness, but with less satisfying results due to the stresses in timber exceeding their capacity. The dimensioning part of the connection in both the analyses proved to be the timber in the notch, which were very sensitive.

One important reflection of this study is its limitations, as the results obtained are based on a limited number of parameters. If other aspects were taken into consideration, more optimized results could have been obtained. For example strengthening of the timber or parametric design could have enabled a valid connection with a higher stiffness.

In previous research plenty of connections were found, with a varying stiffness in a range of 5 and up to 300 kN/mm. These connections were not fully comparable due to different conditions and the wide range of stiffness values. To confirm the sufficiency of the stiffness value obtained in the connection concept of this study, the connection needs to be implemented in a global analysis of the bridge which is suggested for future research. Additionally, an important matter is the fact that the connection needs to be tested in laboratory shear tests. The validity of the connection could thereafter be proven.

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