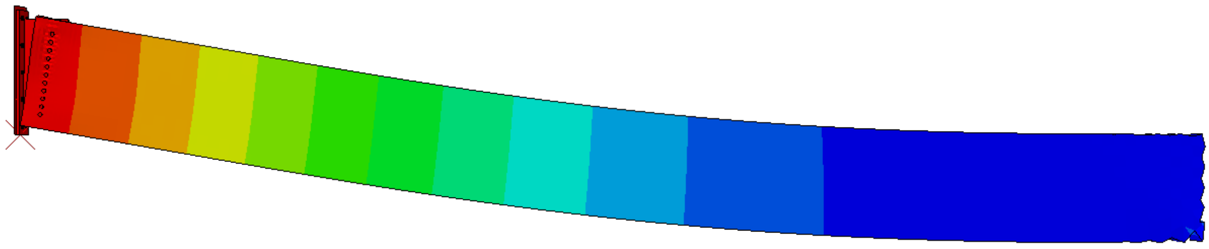

Effects of deformation differences on connections in hybrid structure

Master's thesis in Master Programme Structural Engineering and Building Thechnology

MALIN LARSSON AND ELIN WIDÉN

DEPARTMENT OF ARCHITECTURE AND CIVIL ENGINEERING

CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden 2025
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MASTER'S THESIS 2025

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Cover: Embedded knife plate connection subjected to a displaced support, modelled in ABAQUS.

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ABSTRACT

Hybrid structures that combine timber and concrete have become an increasingly attractive solution in modern construction, making use of the benefits of both materials. Timber offers a renewable, lightweight, and environmentally friendly alternative, while concrete provides stability and stiffness. However, the differences in long-term deformations between timber and concrete creates significant challenges, especially at the connections where these materials interact. If the timber elements deform more than the concrete ones in the structure, the connection that combine these, must accommodate the displacement mismatch. Such differences can lead to an increase in stress and forces, and potential long-term issues affecting the safety and performance of the structure.

The aim of this Master's thesis is therefore to investigate how these differential long-term deformations influence the structural performance of a timber-to-concrete connection in a hybrid structure. A specific focus is placed on an embedded steel knife plate connection, commonly used for its aesthetic appeal and fire-protective properties. Understanding how these deformations develop and interact at the connection will provide valuable insights for future designs.

To achieve this aim, a finite element model (FEM) was developed in ABAQUS. This model simulates the connection under imposed displacements that represent long-term deformation effects. The analysis includes the distribution of forces in bolts and screws, stress concentrations in the timber beam, and the overall moment and stiffness of the connection. Different configurations of the connection were explored to identify the most critical factors that contribute to structural performance.

The results of the study demonstrate that the difference in long-term deformations between timber and concrete significantly influences the load distribution within the connection. In particular, the analysis highlights the increased stresses and forces in key areas, such as the axial forces in the screws and the stress parallel to grain in the timber beam. The findings suggest that these effects should be carefully considered in connection design, especially when using embedded steel knife plate connections in hybrid structures. Furthermore, the results indicate the potential advantages of having a pinned connection and also allow steel parts to plasticise to decrease the moment development in the connection.

Key words: Hybrid structures, long-term deformation, timber-concrete connection, finite element analysis, embedded steel knife plate.

Effeckten från skillnaden i defromationer på förband i hybrida strukturer
Examensarbete inom masterprogrammet Konstruktionsteknik och byggnadsteknologi

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SAMMANFATTNING

Hybrida strukturer som kombinerar trä och betong har blivit ett allt mer attraktivt val i modern konstruktion, eftersom de utnyttjar styrkorna hos båda materialen. Trä är ett förnybart, lätt och miljövänligt, medan betong bidrar med stabilitet och styvhet. Samtidigt kan skillnader i långtidsdeformationer skapa vissa utmaningar, särskilt i förbanden där de två materialen möts. Om träelementen deformeras mer än betongelementen i byggnaden så måste förbandet kunna hantera denna skillnad i deformation. Det kan leda till ökade spänningar och krafter som i sin tur kan leda till långvariga problem som påverkar säkerhet och prestanda.

Syftet med detta examensarbete är därför att undersöka hur skillnader i långtidsdeformationer påverkar den strukturella prestandarden hos ett förband mellan trä och betong i en hybrid byggnad. Fokus ligger på ett förband med inslitsad plåt i träbalken, vilket är ett förband som ofta används för sitt estetiska utseende och goda brandsäkerhet. Genom att studera hur deformationerna utvecklas och påverkar detta förband är målet att bidra med insikter som kan vara värdefulla för framtida konstruktioner.

För att uppnå syftet så har en finit elementmodell skapats i ABAQUS. Denna modellen simulerar hur förbandet beter sig med ett förskjutet stöd i balkens andra ände, vilket representerar skillnaden i långtidsdeformationer. Analysen omfattar fördelning av krafter i bultar och skruvar, spänningar i träbalken samt hur moment och rotation utvecklas i förbandet. Olika konfigurationer av förbandet har undersökts för att hitta de kritiska delarna som påverkar kopplingens beteende.

Resultatet av studien visar på att skillnaden i långtidsdeformationen mellan trä och betong har en stor inverkan på lastfördelningen i förbandet. Studien visar framförallt en ökning av spänningar och krafter i balkens axiala riktning, vilket är kritiskt för de axiala krafter i skruvarna och spänningar parallellt med fibrerna i träbalken. Resultaten tyder på att dessa effekter bör beaktas noga, särskilt vid användning av inslitsade plåtsförband i hybrida byggnader där skillander i långtidsdeformationer kan förväntas. Från resultatet så framgår det även att det är till stor fördel att använda ett förband som är så ledat som möjligt och tillåter stålkomponenter att plastisera, för att minska momentutvecklingen i förbandet.

Nyckelord: Hybrida strukturer, långtidsdeformationer, förband mellan trä och betong, finit element analys, inslitsade plåtförband

Preface

This Master's thesis was carried out at the Department of Architecture and Civil Engineering, Division of Structural Engineering, Chalmers University of Technology, Gothenburg, Sweden, during the spring of 2025.

The work presented in this thesis has been both challenging and rewarding. It has given us the opportunity to explore the behaviour of hybrid structures and the effects of long-term deformations on timber-concrete connections, a topic we find both highly relevant and technically fascinating.

We would like to express our gratitude to our academic supervisor, **Zhengyao Li**, for her continuous support, valuable feedback, and encouragement throughout this project. We also want to thank our industrial supervisor at **COWI**, **Thomas Andersson and Adam Jonsson**, for their practical insights, guidance, and for generously sharing their expertise. A special thanks to our examiner, **Mohammad al-Emrani**, whose expertise and constructive feedback have helped us strengthen and refine this thesis. Our thanks also go to everyone at **COWI** who supported us along the way and contributed to a collaborative and stimulating working environment.

Gothenburg, June 2025
Malin Larsson and Elin Widén

Notations

Roman upper case letters

G	Permanent load (permanent last)
Q	Variable load (nyttig last)
L	Span length of beam (spännvidd)
E	Young's modulus
I	Second moment of area (tröghetsmoment)
M	Bending moment
V	Shear force

Roman lower case letters

ψ_0, ψ_1, ψ_2	Load coefficients for variable load
γ_G, γ_Q	Partial safety factors for loads
b	Width of the beam
h	Height of the beam
d	Diameter of dowels
k_{mod}	Modification factor for material properties
w_{inst}	Instant deformation
w_{shear}	Shear deformation
w_{fin}	Final deformation including long-term effects
k_{def}	Creep coefficient
ρ_k	Density of material
$f_{m,k}$	Bending strength
$f_{v,k}$	Shear strength
$f_{m,d}$	Design bending strength
$f_{v,d}$	Design shear strength
M_{Ed}	Design bending moment
M_{Rd}	Design bending resistance
V_{Ed}	Design shear force
V_{Rd}	Design shear resistance
σ_{load}	Stress from load in Abaqus simulation

Greek letters

ϵ_{cs}	Total shrinkage strain
ϵ_{ca}	Autogenous shrinkage strain
ϵ_{cd}	Drying shrinkage strain
β	Shear deformation coefficient
α	Angle in connection design

Acronyms

ABAQUS	Finite Element Analysis software used for simulation
BCs	Boundary Conditions
CLT	Cross Laminated Timber
EC	Eurocode (European standard for structural design)
FEM	Finite Element Method
FEA	Finite Element Analysis
GLT	Glued Laminated Timber
RH	Relative Humidity
ULS	Ultimate Limit State
SLS	Serviceability Limit State

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1

Introduction

This chapter provides background to the field of study to establish context, along with a more focused description of the specific problem being investigated. The aim and limitations of the study are introduced, as well as an overview of the intended methodology.

1.1 Background

Global warming is a growing planetary problem that demands sustainable solutions across industries. The building sector is a major contributor, responsible for approximately 30% of the total CO2 emissions and 40% of the total energy use in Europe (Swedish Wood, n.d.-a). Because of this, sustainability has become a big focus in the building industry. At the same time, urbanization is on the rise, with more people moving to cities as the population grows (Swedish Wood, n.d.-c). To accommodate this, higher buildings are needed to avoid having to reduce areas of greenery while meeting the needs of a growing population. To ensure sustainable construction development, it is necessary to use materials that are as environmentally friendly as possible.

Timber presents a more environmentally friendly alternative to traditional materials such as concrete and steel, thanks to its lower CO2 emissions (Swedish Wood, n.d.-b). The positive qualities of timber include its lightness in relation to its strength, as well as its ease of workability and the large supply of wood provided in Sweden. However, challenges arise when using timber as the primary construction material for taller buildings, especially with regards to wind forces and sound insulation. To address these issues, a combination of timber and concrete can be utilised in a structural system, forming what is known as a 'hybrid structure'.

Hybrid structures are often established to combine different qualities of different materials. These combinations can be used in different manners and scopes. In smaller scale applications, materials are often combined for specific installations or details, such as using steel connectors in joints or grouting techniques. In larger scopes, entire structural components made from various materials are integrated into the same system. These larger scale combinations of different structural materials are what this report will refer to as a 'hybrid structure'. In such systems that combine concrete and timber, concrete often provides the primary stability due to its heavier self-weight and stiffness. While timber can be used as the main building material in the remaining parts of the structure, once sufficient stiffness has already been established. This enables the possibility to build higher buildings using timber as a main structural material, but not the sole one.

1.2 Problem description

When developing taller structures using timber, several issues arise. As mentioned above, providing adequate stiffness is essential. This challenge is somewhat solved by implementing a concrete core. However, this approach introduces its own set of complications.

In previous master's theses, a timber building with a centric core was studied and it was concluded that the long-term effects differ a lot between the timber and concrete structural elements (Dahlqvist & Kollberg, 2023). The vertical timber elements have larger displacements than the concrete core mainly due to timber creep. The deformations on each floor contribute to increasing total displacements at higher levels. This happens because the deformation of a lower floor affects the position of the floors above, which then experience their own additional deformations. As a result, the total displacement at the upper levels become large compared to the adjacent concrete core, which has not deformed as much.

These differences can create inclinations from the core toward the timber structure, inducing strain on the concrete-to-timber connection and potentially compromising its stability over time. To resolve these problems, it is of importance to gain further knowledge of how the deformations occur in timber and concrete and investigate how the connection will be affected.

1.3 Aim and objectives

The aim of this thesis is to understand how long-term deformations in concrete and timber vary and how these variations impact the structural system in hybrid structures. A connection between a timber beam and a concrete core will be investigated to evaluate this. This connection is chosen due to its central role in hybrid systems, particularly in multi-story buildings where differential deformations can be more pronounced.

As a first step, different types of hybrid structures will be studied to identify key characteristics and challenges related to combining timber and concrete. Based on this, a suitable connection detail, between a horizontal timber beam and a vertical concrete core, will be selected and analysed in depth.

The focus will then shift to examining various configurations of this connection. Finite element modeling (FEM) will be used to simulate the behavior of these configurations under variously applied deformations. The thesis aims to identify critical factors affecting performance, and to provide insights that can support the development of future connection designs better suited to accommodate differential deformation effects in hybrid structures.

Figure 1.1 illustrates the specific issue being investigated. It shows one end (right) of a beam subjected to a deforming support. The area of interest is the circled region, where the focus is on observing how the connection responds to the downward movement at the opposite end.

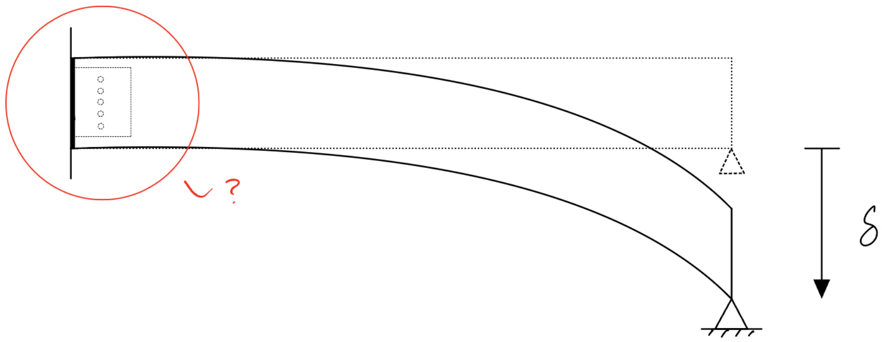


Figure 1.1: Illustration of issue aimed to be investigated

To help facilitate the process, some specific research questions have been developed:

- How do the differences in long-term deformations between timber and concrete affect the connection between them?
- What components of the connection are most affected by the difference in deformation?
- Is it possible to design a connection between a concrete core and a timber structure that accommodates these differences in an efficient way?

1.4 Scope and limitations

The focus of this master's thesis is on the structural behaviour of a timber-to-concrete connection in a hybrid structure, with the aim of evaluating how long term deformation differences affect such a joint. The connection is modelled in the context of a fictional multi-story office building located in Gothenburg.

To keep the study focused, a number of limitations have been set, both in terms of general scope and in the finite element modelling:

General scope limitations

- The analysis excludes environmental or external factors such as moisture, wind, or temperature effects. Only static simplified vertical loads and deformations are considered.
- The results are not intended for direct implementation, but rather to provide conceptual insights into the structural behaviour of hybrid connections under differential deformations. The connection is simplified and not fully developed for construction.

Finite element modelling limitation

- Only the timber and steel components of the connection are modelled. The concrete core is represented as a rigid boundary surface.
- Time-dependent behaviours such as creep and shrinkage are not simulated, instead, their effects are approximated using imposed displacements based on previous research.
- The timber is modelled using elastic material properties, while the steel components

are modelled as perfectly plastic. The contact interactions between different parts are also modelled linearly, and therefore friction is not included in the analysis.

- The model does not include a failure analysis or capacity check within the modelling software. Instead results will be evaluated manually.

1.5 Methodology

The methodology includes a number of different stages. The project starts with a literature study, followed by a detailed parametric numerical analysis on a connection using the finite element method, and is concluded with a discussion and conclusion of the analysis with regard to the stated objectives.

The literature study contain research on material properties of structural materials, existing hybrid buildings as well as various connection designs present in common structures and in hybrid structures. The numerical analysis is performed to determine the effects of long-term vertical displacement on a connection between the timber and concrete. The analysis is performed using the finite element software ABAQUS, with the intention to simulating realistic behaviours. In this stage knowledge obtained in the literature study guides the selection of an appropriate connection design and principles that will be further evaluated and analysed.

Before the parametric study begins, the finite element model is set up. This setup includes defining the initial conditions for the building scenario, preliminarily sizing components based on analytical calculations, and making model assumptions required to simulate the intended case. Once the model is accurately configured and verified against hand calculations, it is considered reliable and is used in the parametric study. The results from the study are evaluated to draw conclusions and support the discussion of the findings.

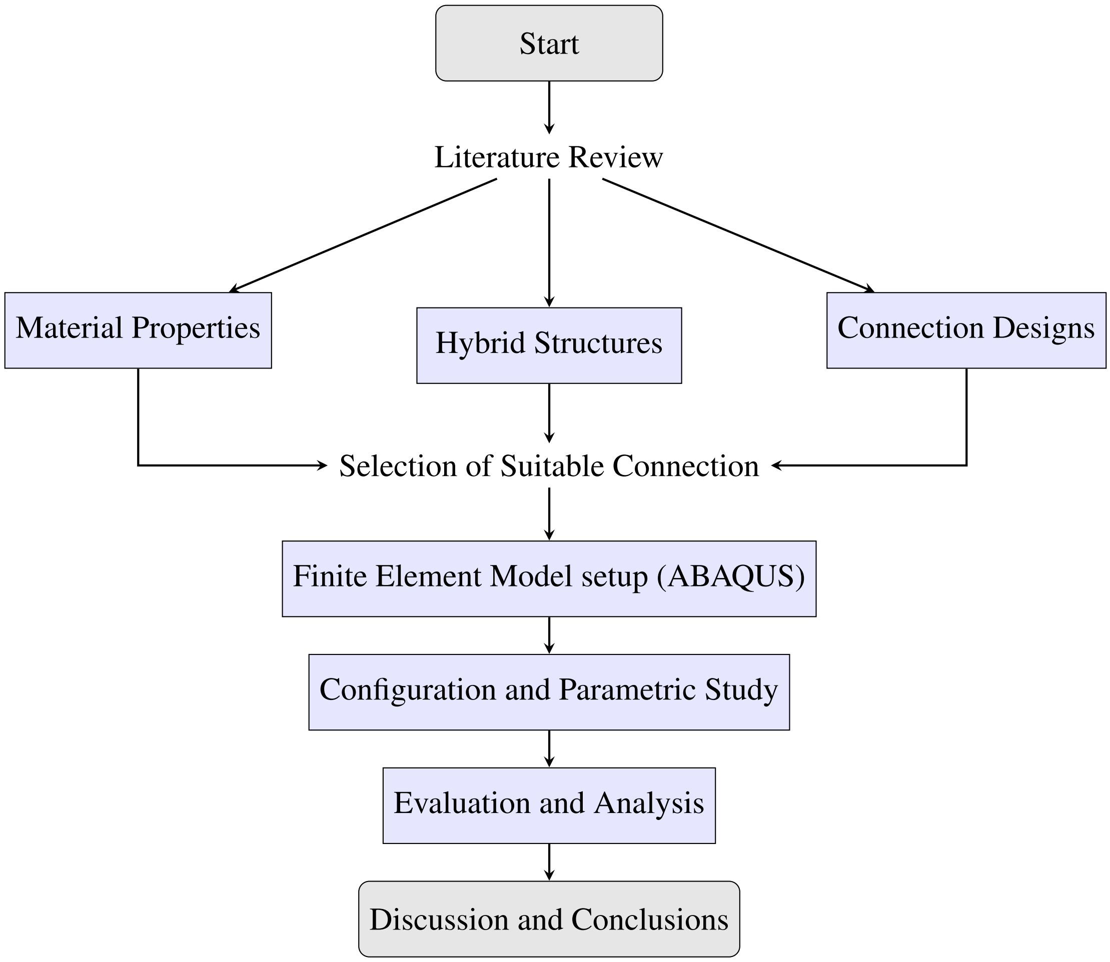


Figure 1.2: Project Methodology Flowchart

1.6 AI disclamer

In this Master's thesis, Artificial Intelligence (AI) has been used as a supportive tool to improve the overall quality of the report. Specifically, AI was used to correct grammar and spelling, assist in structuring the report, and provide guidance on various LaTeX commands used during the writing process. Additionally, AI tools were consulted when encountering errors in ABAQUS, helping to better understand the issues and suggest possible solutions.

2
Material

This chapter introduces the main materials used in the hybrid structure: timber and concrete. The focus is primarily on how long-term deformation affects each material. The chapter also addresses the challenges involved in combining these two materials in a single structural system.

2.1 Timber

The utilization of wood as a construction material has a long historical record, initially limited to smaller buildings with one or two floors (Swedich Wood, n.d.). However, as the only renewable building material, its use has increased in taller buildings for a more sustainable construction.

2.1.1 Strength of timber

Timber is an anisotropic material, meaning that timber has different material properties in different directions (Swedish wood, 2022a). These can be visualized in Figure 2.1. Although three directions are defined, in practice only two are usually mentioned: parallel and perpendicular to the grain. This is because the difference between tangentially and radially is usually small and is not taken into consideration. In both tension and compression, timber is much stronger parallel to the fibbers than perpendicular to the grain.

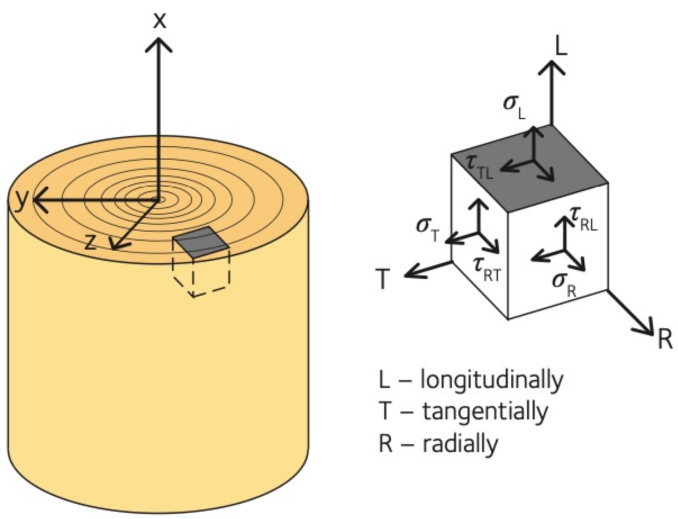


Figure 2.1: Different structural directions in timber (Swedish wood, 2022a)

Timber strength is influenced by factors such as moisture, time, and temperature, in addition to the direction of the fibers (Swedish wood, 2022a). The moisture content plays a significant role in the strength of timber, as higher moisture reduces strength, especially in tension. Increased moisture causes timber to swell, while lower moisture leads to shrinkage. These changes affect both the dimensions and the density of the wood. The moisture content is closely linked to the surrounding environment.

Time and temperature also affect timber strength (Swedish wood, 2022a). Longer loading times reduce strength, while temperature has a smaller impact under normal conditions. However, temperatures above 65°C to 95°C for extended periods can weaken the wood. Knots and other imperfections reduce the strength of timber, which is why knot-free timber is preferred for structural use, though producing it without knots is challenging. Engineered wood products offer improved strength as an alternative.

2.1.2 Engineered wood products

To improve the strength and versatility of timber, various types of Engineered Wood Products (EWPs) have been developed (Swedish wood, 2022a). These include both beams and panels made from timber, veneers, particles, or fibres, all bonded with adhesives. Common examples are glued laminated timber (glulam) and cross-laminated timber (CLT).

Glulam, one of the earliest and most widely used EWPs in Sweden, is made by gluing together layers of sawn timber, all aligned in the same direction (Swedish wood, 2022a). These beams can be produced straight or curved. While glulam and sawn timber of the same size offer similar strength, glulam has more consistent properties. This is because defects like knots are spread out during manufacturing, making glulam more predictable and available in a wider range of sizes than sawn timber.

2.1.3 Long-term deformation

A number of factors must be considered when assessing the effects of timber on deformation over time. The factor that has the largest effect on long-term displacement is the initial moisture content of the timber and its variations (Swedish wood, 2022a). Increased humidity leads to greater deformation. Eurocode 5 (2004) has formulated three service classes to account for these effects in calculations, categorising timber based on its average moisture content. Class 1 corresponds to the driest conditions, while Class 3 represents the highest moisture content.

In Eurocode 5 (2004) the long-term deformations are simplified and calculated with equation (2.1).

$$u_{\text{creep}} = k_{\text{def}} \cdot u_{\text{inst}} \tag{2.1}$$

The deformation factor k_{def} is determined based on a table in Eurocode 5 (2004) that specifies the material used and the applicable service class. If there is a higher moisture content, k_{def} will be higher and the creep deformation will increase. In order to establish the deformation factor, a series of experiments were conducted, and an approximate value was determined (Swedish wood, 2022a). This was done to simplify the calculations, which had previously been much more complex.

To calculate the final deformation in Eurocode 5 (2004), both the instantaneous deformation and the creep deformation are considered; see Equation (2.2).

$$u_{\text{fin}} = u_{\text{inst}} + u_{\text{creep}} = u_{\text{inst}} \cdot (1 + k_{\text{def}}) \tag{2.2}$$

The experiments to determine k_{def} were conducted under constant loading (Swedish wood, 2022a). It is acknowledged that the creep of timber is load-dependent; therefore, a variable ψ_2 is introduced to account for load variations. This factor is applied to the deformation caused by the variable load but not to the permanent load, as the latter remains constant and aligns with the conditions of the conducted experiments. The equation for variable loading is given in Equation (2.3), where ψ_2 equals 1 in the case of constant loading.

$$u_{\text{fin,Qi}} = u_{\text{inst,Qi}} + u_{\text{creep,Qi}} = u_{\text{inst,Qi}} \cdot (1 + \psi_{2i} \cdot k_{\text{def}}) \tag{2.3}$$

Composite wood products are affected by creep differentaly than sawn timber (Swedish wood, 2022a). This is because the adhesives used in products may have different creep properties that need to be taken into account. When a material with different creep properties is used, the previously mentioned method cannot be applied to calculate long-term deformation. To account for the creep behaviour of the timber, a reduced elasticity modulus is used, see Equation (2.4). This reduced elasticity modulus represents the final mean elasticity modulus and can then be used to calculate deformations. For timber products where the creep properties are different in the element, the previously mentioned method cannot be applied to calculate long-term deformation.

$$E_{\text{mean.fin}} = \frac{E_{\text{mean}}}{(1 + \psi_{2i} \cdot k_{\text{def}})} \tag{2.4}$$

2.2 Concrete

Concrete is one of the most used structural materials in the building industry, valued for its versatility, strength, and durability. However, challenges such as long-term effects and sustainability concerns must be considered in both design and construction.

2.2.1 Structural qualities and environmental challenges

Concrete's strength and durability make it ideal for large-scale structures, such as bridges and tall buildings, as it can withstand heavy loads and harsh environments (Engström, 2014a). It performs best under compression, but its tensile strength is weaker. To address this, concrete is often combined with steel reinforcement, which helps handle the tensile forces.

Despite its advantages, concrete has significant environmental drawbacks, primarily due to the high carbon dioxide (CO_2) emissions associated with cement production, which accounts for around 8 % of global emissions (Cheng et al., 2023). This high level of emissions is due to both the energy used in the production process and the chemical reactions that occur when limestone and other materials are heated. As a result, there is a strong demand for material efficiency in concrete design.

2.2.2 Long-term effects

Over time, concrete undergoes gradual deformations due to sustained loads and environmental influences. These long-term effects impact the structural performance and must be considered in design. The primary causes of these deformations are shrinkage and creep (Harapin et al., 2024). Both are moisture- and time dependent, however, creep is also load dependent while shrinkage is not.

2.2.2.1 Concrete shrinkage

Shrinkage is dependent on moisture changes in the concrete mix and is time-dependent. Moisture changes occur during the hardening phase, when the cement undergoes hydration, and later as the concrete dries (Engström, 2014b). The shrinkage that occurs during hardening is known as autogenous shrinkage and primarily develops within the concrete element during the early days after casting. This process is most pronounced in concrete with a low water/cement ratio, particularly in high-strength concrete.

Drying shrinkage depends on moisture changes between the concrete and the surrounding environment, and it is most pronounced in areas exposed to drying (Harapin et al., 2024). This can vary between different regions and climates as well as on different parts of a structure, such as sun-exposed versus shaded areas. More importantly, shrinkage primarily occurs on the outer surface of the element rather than uniformly throughout the material. In general, the water/cement ratio and the relative humidity (RH) are two key factors in predicting drying shrinkage (Engström, 2014b). A specimen with a high water/cement ratio, meaning a high water content, placed in a dry environment will experience significant shrinkage. This means that the shrinkage will not develop homogeneously through a specimen. According to Eurocode 2 (2004), the total shrinkage is then the sum of the autogenous shrinkage and the drying shrinkage, see equation 2.5.

$$\varepsilon_{cs} = \varepsilon_{ca} + \varepsilon_{cd} \tag{2.5}$$

where:
ε_{cs} = total shrinkage strain
ε_{ca} = autogenous shrinkage strain
ε_{cd} = drying shrinkage strain

2.2.2.2 Concrete creep

Creep is moisture-, time- and load dependent and it will not occur if no load is applied. When a load is applied, immediate deformation/strain is induced and it is referred to as immediate elastic deformation (Harapin et al., 2024). When this load then act for a long period of time the strain can increase, which is the effect of creep. Creep occurs because, after the elastic deformation, the material redistributes stresses by adjusting its shape and volume. This adaptation leads to increased deformations over time but will reach a final deformation after long time.

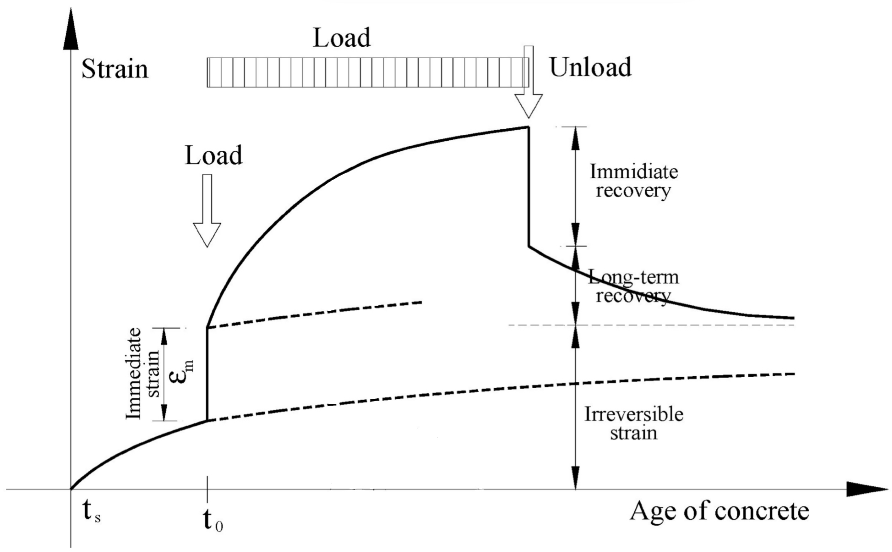


Figure 2.2: Time dependency of strain (Harapin et al., 2024)

The time dependence of the total strain is illustrated in Figure 2.2. The figure aims to show how the strain varies over time when a load is applied and removed. Shrinkage begins immediately, since no load is needed for it to occur. When the load is applied it causes the immediate elastic strain followed by creep. When the load is then removed the element will have some instant recovery. After that it will will continue to recover gradually but it will never reach the value it had before the load was first applied.

Similar to the effect of shrinkage, relative humidity plays a significant role in creep as well (Engström, 2014b). A lower relative humidity in the surrounding accelerates creep due to increased moisture loss from the concrete. It releases moisture to the air in an attempt to reach equilibrium. This dependency on humidity is important in long-term structural performance, as both increases and decreases in RH can either diminish or accelerate the effects of creep during its lifetime.

2.3 Challenges when combining materials

In Hybrid structures where different materials are combined, several challenges arise which will be explained below. Special considerations may be necessary when introducing a secondary material that behaves differently from the primary structural material. Long-term deformations, fire and moisture protection are three such challenges.

Swedish wood (2022a) states that variations in deformations between materials is something that can cause problems when combining materials in a structure. If, for example, one side of a structure deforms more than another, unexpected inclinations can occur. This occurrence was also observed in Dahlqvist and Kollberg (2023)'s Master thesis where long-term deformations in a fictive tall timber building with a concrete core was investigated. They also discussed the potential issue of an inclination in the beam that will connect to both the concrete core and the timber column. This could potentially affect the connection as well.

Different materials respond differently to fire and must therefore be protected and utilized in different ways (Eurocode 1, 2002). When being subjected to fire, concrete and steel retain their cross section while their structural strength diminishes under high temperatures. For timber it is the other way around. Timbers cross section reduces more rapidly when being subjected to fire, yet its core strength remains intact for a longer period of time. In terms of structural strength degradation, steel is the most vulnerable of the three materials (Eurocode 3, 2005). It not only loses its structural integrity but does so rapidly, making failure difficult to predict. This must be taken into account when incorporating steel into a timber structure. For example, steel components can be concealed to protect them from direct exposure to fire, either by additional fire protection measures or by embedding them effectively within timber elements.

Another important factor to consider when combining materials is their response to moisture (Svenskt trä, 2018). As mentioned earlier in this chapter, both concrete and timber are highly affected by moisture. Additionally, the moisture content in concrete is significantly higher than in timber, which can lead to moisture transfer from the concrete to the timber elements. To prevent this, moisture barriers must be incorporated, and steel components can serve this purpose. Therefore, integrating steel between concrete and timber members can be beneficial with regard to moisture protection.

3
Connections

To join different members in a system and transfer loads, connections are required. A structure will require a number of different connections in different parts. This chapter studies various types of connections, with a primary focus on timber-to-concrete joints between the concrete core and timber beam. However, general timber connections will also be covered. The main focus are on beam-columns and beam-wall connections.

3.1 Timber connections

When designing a timber structure, the connections are among the most critical components, and can often be the governing factor in design (Svenskt trä, 2018). Many factors need to be considered, and it is crucial that engineers conduct a thorough analysis to prevent failures. In the process of designing a connection, it is desirable to make it ductile. This ensures that visible warning signs, such as cracks, appear in time to detect and address potential issues before failure. In general, timber is ductile in compression but can be very brittle in tension.

All connections serve the purpose of transferring loads from one part to another. Therefore, it is important to analyse how these loads are transferred and whether moments need to be considered. Load transfers can range from simple supports, which only carry vertical forces, to fully fixed connections, which can transfer both forces and moments in all directions (Svenskt trä, 2018). In most cases, a connection falls somewhere between these two extremes.

As mentioned in Section 2.1, timber is highly affected by moisture, causing it to swell and shrink depending on changes in the surrounding moisture content (Svenskt trä, 2018). This must also be considered when designing connections, to allow for the timber to change in size. Moisture content often changes during the construction phase, but variations can also occur at other stages of the structure's lifespan. Moisture-related movement is complex and case-dependent, it must therefore be studied in detail. Furthermore, wood is an anisotropic material, which affects the moisture-related movement, as the rate of shrinkage and swelling varies in different directions. The most significant moisture-related movement occurs perpendicular to the grain, which is also the weakest direction in terms of strength. Therefore, it is crucial to design connections with the grain direction in mind and, if possible, avoid connections subjected to loads perpendicular to the grain.

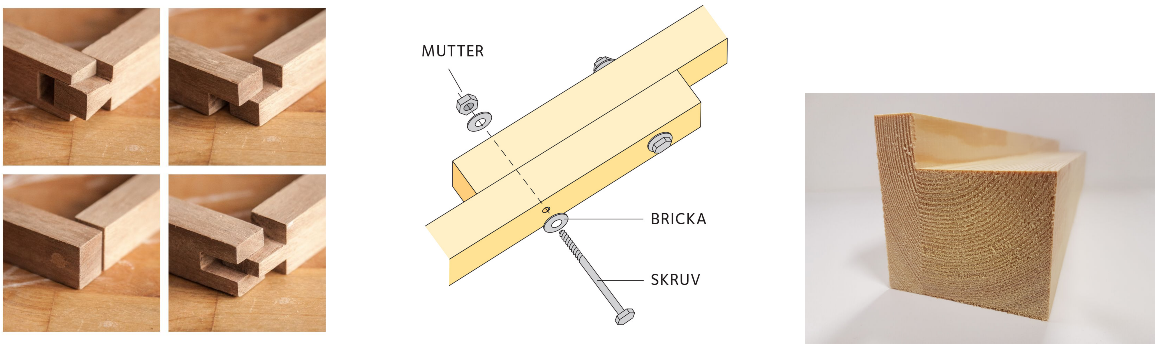
In glulam structures, steel connectors such as plates and dowels are commonly used (Svenskt trä, 2018). When a connection includes steel components, corrosion and fire safety must be considered, as steel performs worse in these aspects compared to timber. Additionally, the use of steel connectors often requires a reduction in the timber's cross-section, which must be accounted for to ensure the structural strength of the timber.

3.1.1 Type of joints

There are three main types of connections used to join timber parts: traditional timber joints, dowelled joints, and glued joints (Swedish wood, 2022a). Traditional timber joints involve the use of timber dowels, often made from higher-strength timber or cut outs from the timber, see Figure 3.1a. These are common in traditional timber buildings where steel dowels are not available. However, due to their cost, time consumption, and relatively low strength, this type of connection is no longer widely used in modern construction.

Dowelled joints are the most commonly used type (Swedish wood, 2022a). Various forms of dowels exist, including nails, screws, and bolts, one exampel can be seen in Figure 3.1b. Dowels can also be combined with steel plates to enhance the connection's strength. A key advantage of this type of connection is that it can be designed to be ductile.

Glued joints involve joining timber parts with adhesive (Swedish wood, 2022a). This is most commonly used when connecting structural timber to produce engineered wood products as can be seen in Figure 3.1. When using glue, it is important to control the surrounding conditions, as glue is sensitive to moisture and temperature.



a) b) c)

Figure 3.1: Different types of joints were a) is a traditional timber joint (Openculture, n.d.), b) is a dowelled joint (Snickarskolan, n.d.) and c) is a glued joint (Rundvirke komponent, n.d.)

3.2 Timber-concrete composite connections

Timber-Concrete Composite (TCC) elements are hybrid structural components that combine the benefits of both timber and concrete to enhance structural performance. TCC elements are commonly used in floor and deck structures, where they allow efficient load distribution and minimized deflections compared to traditional solutions only using timber (Johari et al., 2023). Although the focus of this study is on direct connections between timber beams and concrete cores, understanding TCC elements can be relevant. Their connection principles and load transfer mechanisms may provide understanding for how established techniques can be adapted to connections between timber beams and concrete cores as well.

Fully rigid connections create full interaction between two materials, enabling a nearly homogeneous elastic load transfer (Johari et al., 2023). These connections are extremely stiff and are often achieved using notched connections, where parts of the timber are carved out to interlock with the concrete, much like puzzle pieces, se Figure 3.2. Adding adhesives to the connection increases the stiffness further. While these connections offer the highest stiffness, they typically cause brittle failure, meaning they can fail suddenly without warning.

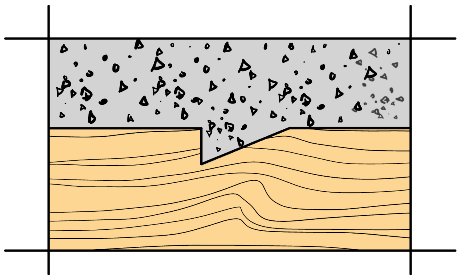
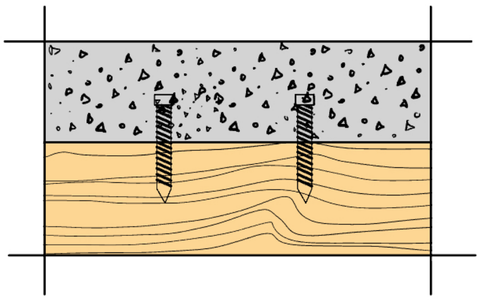
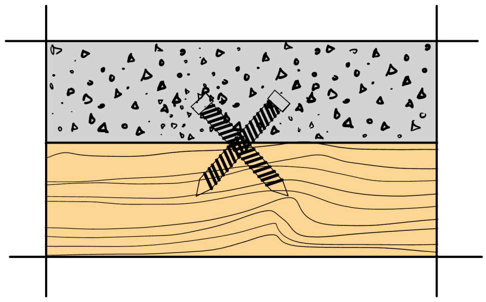


Figure 3.2: Notched connection (remade by author, adapted from Johari et al. (2023))

There are other ways to design the joints which will make it less rigid than notched joints, allowing for more ductility (Ling et al., 2022). These semi-rigid connections often incorporate steel connectors between timber and concrete. Steel connectors are widely used because they are relatively easy to produce and assemble while providing effective load transfer. Due to the natural ductility of steel, these connections can accommodate deformations more effectively than fully rigid alternatives. Typical steel fasteners include dowel-type fasteners, screws, and steel plates, see Figure 3.3. An efficient way to manage the shear stresses in TCC structures is to incline the screws. Two screws in opposite inclinations restrict the shear in each direction (Johari et al., 2023), see figure 3.3b.



(a) Dowels



(b) Inclined screws

Figure 3.3: Dowel conections (a) and connection with inclined screws (b) (remade by author, adapted from Johari et al. (2023))

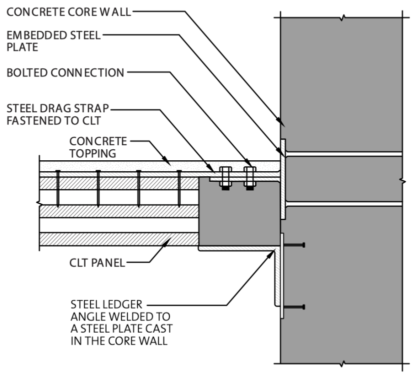
To assess the stiffness and rigidity of specific fasteners and connections, the slip modulus is a relevant parameter (Northcutt et al., 2012). It defines the relationship between shear force and displacements within the connection. A connection with high stiffness and rigidity typically has a high slip modulus, while a flexible connection that allows for a lot of movement has a low slip modulus. Eurocode 5 (2004) assign different slip modules for different types of fasteners. It also states that, when designing timber-to-concrete connections, the slip modulus should be doubled to account for the higher stiffness of concrete compared to timber.

3.3 Structural connections between timber and concrete

There are many different challenges when combining materials in a hybrid building, and one of them is the difference in long-term deformation between concrete and timber. Because of this, the connection that links the two materials must be able to accommodate this effect. This section presents some possible connection solutions between a timber beam or CLT panel and a concrete core.

3.3.1 Connection in Brock Commons Tallwood House

Brock Commons Tallwood House is an 18-storey hybrid structure located in Canada (Connolly et al., 2018). The building features two concrete cores, surrounded by timber columns and CLT panels. In Figure 3.4a, a drawing of a connection between the CLT-panel and the concrete core is presented. This connection is of significance as it is required to resist the vertical shear forces as well as account for the deformation differences between timber and concrete. To this aim, a connection was designed to transfer both vertical and lateral loads from the CLT-panel to the concrete core. The CLT-panel is supported on a steel ledger that is bolted to a steel plate casted to the concrete core, see Figure 3.4b. To transfer the lateral loads there is steel drag strap that is connected with screws on top of the CLT-panel. The steel drag strap is then bolted to an embedded steel plate in the concrete core, see Figure 3.4c. The strap's dimensions vary depending on the desired floor and strap orientation. The thickness of the strap is determined by the floor level, while its length is contingent on its orientation.



(a)

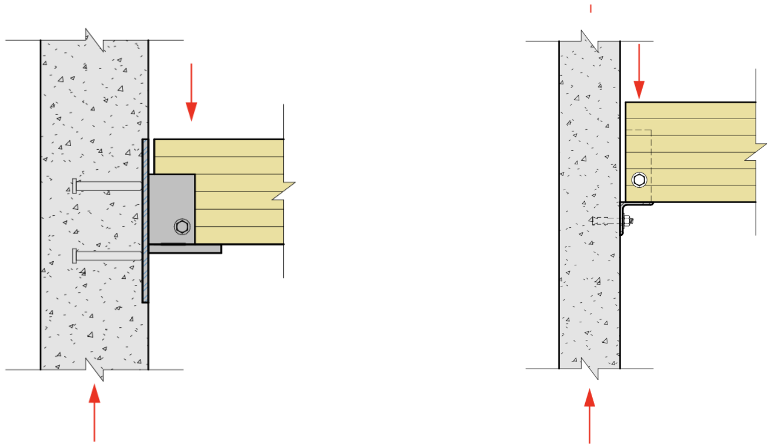
(b)

(c)

Figure 3.4: Connection between CLT-panel and concrete core, a) drawing of the connection (Naturaly:wood, 2016), b) CLT-panel supported on a steel ledger (Connolly et al., 2018). c) steel drag strap bolted to embedded steel plate(Wood works, n.d.)

3.3.2 Beam hanger

This connection has the capacity to transfer vertical load from the beam to the concrete core (WoodWorks, 2021). The timber beam is to be supported by a beam hanger, with dowel-type fasteners being utilised to secure the beam to the hanger, see Figure 3.5a. The beam hanger is then welded to a steel plate, which is cast into the concrete core secured by dowels. Alternatively, a knife plate with a foot can be used instead of a beam hanger, as illustrated in Figure 3.5b.



(a) (b)

Figure 3.5: Drawing of two connections where the beam is perpendicular to wall connected to face of wall, a) beam hanger b) knife plate. Taken from WoodWorks (2021)

The capacity of the connection is dependent of the capacity of the timber beam perpendicular-to-grain (WoodWorks, 2021). However, it is also important to consider the tensile and shear capacity of the dowels in the concrete. In the event of the beam not being placed centric within the beam hanger, an eccentricity will be introduced, resulting in a moment that must be taken into consideration. It is also essential to consider shrinkage and swelling of timber, which is influenced by the surrounding environment, particularly the timbers moisture content. In the absence of adequate bracing at the connection's uppermost point, it becomes important to verify if there can be lateral-torsional buckling occurring.

3.3.3 Beam hanger with top bearing

This connection bears a strong resemblance to the previously described connection (namely Beam hanger), with the key distinction in the attachment of the steel plate to the concrete core (WoodWorks, 2021). The steel plate is equipped with a top plate that is positioned within a pocket in the concrete that is later filled with grout, see Figure 3.6. The steel plate is also connected to the concrete core using screws. When designing the top plate (i.e. the wall anchor), it is important to consider the length of the anchor. This is to ensure that there will not be edge failure in the concrete. In order to establish this connection, it is also imperative to verify the presence of eccentricity in the connection. Due to the anchorage between the concrete core and the steel plate, this connection will resist higher loads than the previous one.

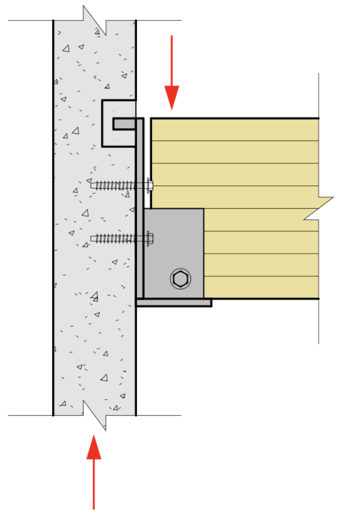


Figure 3.6: Beam perpendicular to wall connected to face of wall with top bearing. Taken from WoodWorks (2021)

3.3.4 Wall notch

The wall notch connection will transfer vertical load from a timber beam to a concrete wall. The timber beam is bolted to a knife or side plate. An illustration of the connection with a knife plate can be seen in Figure 3.8 . The knife plate is then welded to a plate with threaded studs. Both the plate and the beam are then placed within a pocket in the concrete wall, with grout filling the space between them (WoodWorks, 2021). This is to allow for any differences in thermal expansion between timber and concrete that may be present during the construction phase. The threaded studs are also embedded vertically in the concrete wall. The capacity for this connection is regulated by the strength of the timber beam perpendicular-to-grain. In the process of designing this connection, it is important to protect the timber from moisture where contact is present between the timber and the concrete. This connection has the capacity to withstand a high load.

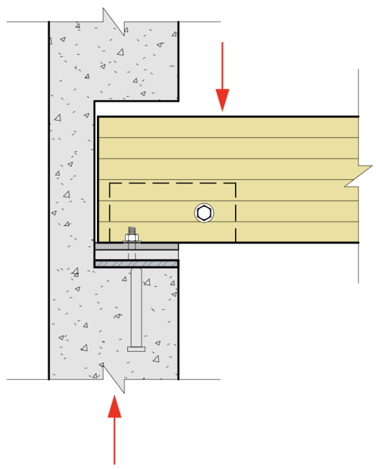


Figure 3.7: Connection with a beam in a wall notch. Taken from WoodWorks (2021)

3.3.5 Embedded knife plate

In this connection, the timber beam is bolted to a steel plate embedded in the beam (WoodWorks, 2021). This knife steel plate is then welded to a steel plate attached to the concrete core, thereby forming a connection that transfers vertical load from the beam to the concrete core. The capacity of this connection depends on the shear strength of the fasteners and the compressive strength of the timber where the fasteners are in contact.

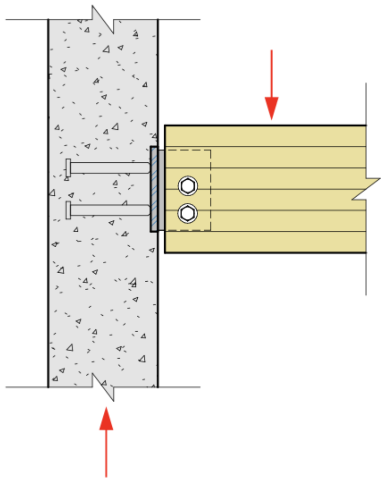


Figure 3.8: Connection with a steel knife plate in the timber beam. Taken from WoodWorks (2021)

A connection with an embedded steel plate in the timber beam is good with regards to fire (Swedish wood, 2022a). This is because the timber will act as a protective layer that protect the steel, to prevent the steel from overheating and loosing its strength. If the steel plate are placed on the outside there is a need for additional fire-resistant protection to protect the connection. Another positive aspect with this type of connection is that it is aesthetically pleasing, with the steel plate embedded and not visible from the exterior.

3.4 Design of timber-steel connection

To design connections in timber structures, Johansen's theory is used, in which different failure modes of the connection are evaluated (Swedish wood, 2022a). The theory considers three main types of failure. The first failure mode occurs in the timber member and is caused by compression failure due to the pressure from the dowel, this can be seen in Figure 3.9 a, c, f and j. The second mode occurs when the dowel yields, forming a plastic hinge in the material with the higher strength, in this case, the steel knife plate, see Figure 3.9 b, d, g and h. The third and final failure mode involves the formation of three plastic hinges at the dowel: one in the steel knife plate, and one on each side of the knife plate in the timber member, see Figure 3.9 e, h and m. In terms of ductility, the third failure mode is the most desirable.

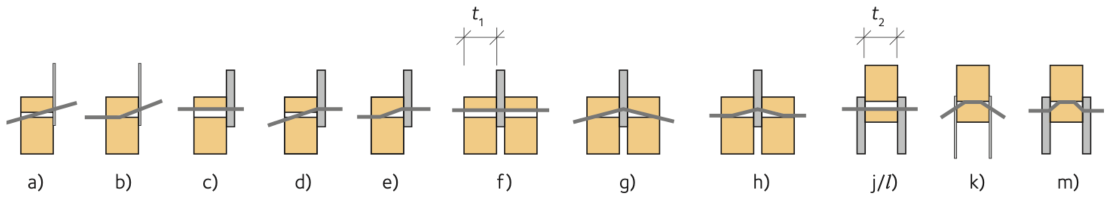


Figure 3.9: Failure modes fore timber-steel connection (Swedish wood, 2022b)

Each failure mode has a corresponding design equation depending on the configuration of the timber connection (Eurocode 5, 2004). These three failure mode are denoted as f, g and h for a embedded steel knife plate with dowels.

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k}\, t_1\, d & \text{(f)} \\ f_{h,1,k}\, t_1\, d \left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,1,k}\, d\, t_1^2}} - 1 \right] + \frac{F_{ax,Rk}}{4} & \text{(g)} \\ 2.3\sqrt{\frac{M_{y,Rk}\, f_{h,1,k}}{d}} + \frac{F_{ax,Rk}}{4} & \text{(h)} \end{cases} \tag{3.1}$$

4

Initial design and numerical modelling

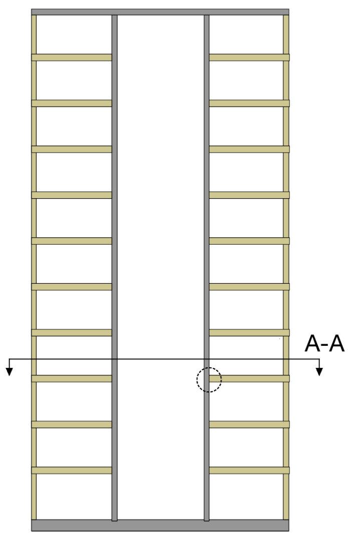
To establish a foundation for the study, this chapter begins by outlining the initial design assumptions for the fictitious building where the connection will be investigated. It includes a preliminary dimensioning of key structural components and connections, which serves as a baseline for the parametric analysis. The chapter also introduces the numerical modelling process, covering material properties, boundary conditions, and the method used to extract data from the finite element simulations.

4.1 Structural system and load conditions

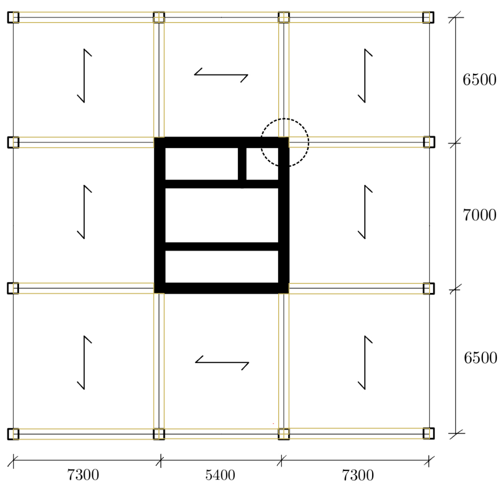
An overview of the structural layout is presented here, including the main load-bearing elements and applied loads.

4.1.1 Fictitious building

In order to analyse the effects of long-term deformation on a connection, a fictitious building is examined. The building model utilised in this study is identical to the one analysed by Dahlqvist and Kollberg (2023) in their Master's thesis. The building is located in Gothenburg, in an area close to the river. The building is designed for office use.



(a) Building side view



(b) A-A: Structural floor plan

Figure 4.1: a) Side view of structural system b) Structural floor plan with circled position for connection

The building has a square footprint, and its layout is shown in Figure 4.1. It features a beam-column system with a centrally located reinforced concrete core, which provides stability against horizontal forces like wind loads. The glulam columns are positioned along the facade and extend continuously over two floors, aligning directly with each other. Within the concrete core, staircases and elevators are placed.

The timber beam to be analysed is supported by a glulam column on one side and the concrete core on the other. The connection between the beam and the column is assumed to be simply supported, and will not be investigated further. For the floor construction, CLT slabs are used which rest on the beams for support. The slabs will transfer load in one direction, and it is estimated that this load will be divided 50/50 between the beams, see Figure 4.1 for an illustration of the floor's load bearing directions. To enhance structural stability, the roof is designed with a concrete structure, adding additional weight to the building.

4.1.2 Loads

The classification of loads is typically divided into two categories: permanent and variable. Permanent loads are defined as those that remain constant over time, including self-weight and fixed installations. In contrast, variable loads are those which change over time. Examples of variable loads include imposed loads from people and snow loads.

The loads of interest are presented in Table 4.1. The self-weight of materials is calculated from geometry and density, while the variable loads have been taken from Eurocode 1 (2002). The calculation of the self-weight can be found in Appendix A. The imposed load is also accompanied by a load coefficient, a factor which can be observed in Table 4.2.

Table 4.1: Permanent and variable load of interest, that affects the beam

Permanent load		
Self weight beam	G_{beam}	0.595 kN/m
Self weight slab	G_{slab}	5.57 kN/m
Installations	G_{inst}	6.75 kN/m
Variable load		
Impost load, Office	Q_{office}	20.3 kN/m

Table 4.2: Load coefficient regarding office imposed load

ψ_0	ψ_1	ψ_2
0.7	0.5	0.3

When calculating the total load, different combinations are used. There are two different states that are looked at, Serviceability Limit State (SLS) and Ultimate Limit State (ULS). When a structure is in SLS, the applied load corresponds to the load present when the structure is in service. This load combination is typically used for calculating deflection. The Equation to calculate the load in SLS is shown in Equation 4.1.

$$Q_{SLS} = G + \sum \psi_i \cdot Q_i \tag{4.1}$$

A structure is said to be in ULS when it has reached its load capacity. The load combination is utilised to calculate the maximum bending moment and the maximum shear force. The Equation to calculate the load in ULS is shown in Equation 4.2, and the partial safety factors, γ, are presented in Table 4.3. The resulting load combinations are presented in Table 4.4.

$$Q_{ULS} = \gamma_G \cdot G + \gamma_Q \cdot Q_1 + \sum \gamma_Q \cdot \psi_i \cdot Q_i \tag{4.2}$$

Table 4.3: Partial safety factor for permanent and variable load in ULS

	Partial safety factor
γ_G	1.35
γ_Q	1.5

Table 4.4: Load combinations for ULS and SLS

	Total load combinations
Q_{ULS}	47.8 kN/m
Q_{SLS}	33.2 kN/m

4.1.3 Long-term deformations

As concluded by Dahlqvist and Kollberg (2023), an investigation into long-term deformations in three different buildings revealed clear differences between timber columns and concrete walls. The long-term effects included for that investigation was moisture content, linear elastic behaviours, shrinkage and creep for timber and concrete. The buildings vary in height, and the timber columns differ in size. The maximum observed difference in long-term deformation is presented in Table 4.5, and this difference is used as a reference point for evaluating the effects on the connection. This study will simulate the difference in long-term deformation as an added displacement of the support located on the other end of the beam, opposite the connection. The maximum displacement that will be applied to the opposite support in this study is 35 mm, since that was the maximum difference attained in previous investigation.

Table 4.5: Maximum long-term deformation and the differences in deformation for different building heights (Dahlqvist & Kollberg, 2023)

Building	**10-storys**	**20-storys**	**30-storys**
Max long-term deformation for concrete wall	14 mm	32 mm	53 mm
Max long-term deformation for timber column	38 mm	63 mm	88 mm
Max difference in deformation	23 mm	31 mm	35 mm

4.2 Preliminary sizing

The preliminary sizing serves as an initial step to establish a reference model for further analysis by defining reasonable dimensions for the glulam beam and the steel knife plate connection. This initial design will later be adjusted as part of a parametric study to evaluate different configurations of the connection.

4.2.1 Glulam beam

A preliminary design of the glulam beam mentioned in Section 4.1.1 has been carried out to ensure that the dimensions are reasonable, based on the ULS and SLS load cases described in Section 4.1.2. For the dimensioning, the beam is assumed to be simply supported, as this represents the worst-case scenario. The chosen dimensions are presented in Table 4.6. The verification includes checks for bending moment, deflection, and shear capacity. More detailed calculations can be found in Appendix A.

Table 4.6: Dimensions GL30c beam

	Size [mm]
Height	675
Width	230
Length	7300

4.2.2 Embedded steel knife plate connection

A connection that is further investigated is the embedded steel knife plate discussed in section 3.3.5. This connection was chosen for the study because it is commonly used in practice. One of the main reasons is that the steel components are protected by the surrounding timber, which offers advantages in terms of fire safety. Additionally, the connection is often preferred by architects in buildings where the structural system is exposed, due to its more aesthetically pleasing appearance.

The embedded knife plate connection is designed with regards to Johansen's theory and Equations 3.1. The results of the design and the dimensions of the different parts are depicted in Figure 4.2 a) and detailed calculations are provided in Appendix B.

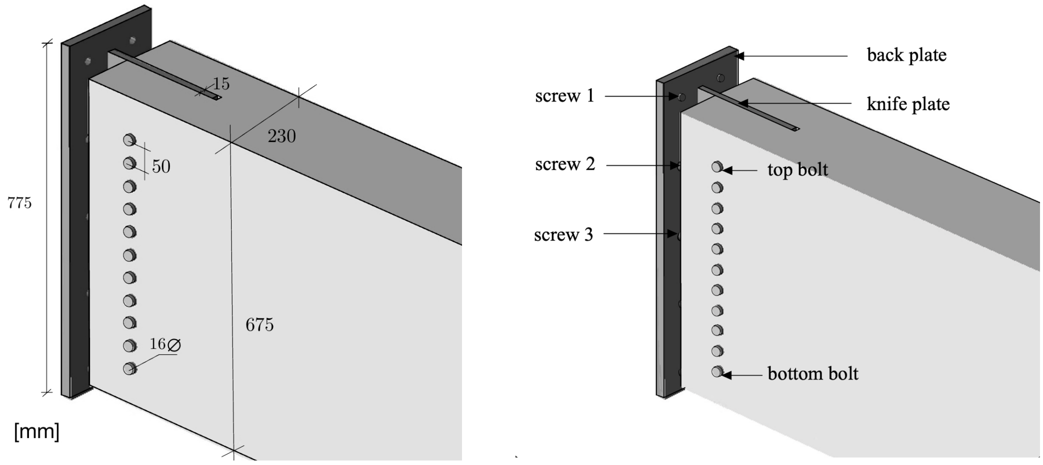


Figure 4.2: Connection layout, a) with dimensions and b) with predefined terms

To improve readability and ensure clarity when referring to the different components configured and evaluated, predefined terms are used throughout the text. These are illustrated in Figure 4.2 b). Additionally, a general definition is that 'bolts' always refer to the components connecting the beam to the steel knife plate, while 'screws' refer to the components connecting the back plate to the concrete wall behind.

4.3 Finite element model setup

This section describes the implementation of the finite element model in ABAQUS, including the definition of materials, boundary conditions, and interactions. It also covers how data is extracted from the model and the steps taken to verify its accuracy.

4.3.1 Material properties and interactions

Given that timber is an anisotropic material, the glulam beam is defined in ABAQUS using an engineering constants material model, allowing different stiffness values to be assigned in each principal direction. This material model is also used in Zhang et al. (2023). The timber material was assumed to behave linearly elastically, which is appropriate for analysis in the serviceability limit state where stresses and deformations are expected to remain within the elastic range. No plasticity or failure mechanisms were included in the model. The material properties used for the timber beam in the analysis are presented in Table 4.7. The table presents the material properties assigned to different directional parameters in ABAQUS. The first three values represent the elastic modulus of a GL30c beam in the parallel, perpendicular, and tangential directions. For further explanation see section 2.1.1. The remaining parameters are the corresponding Poisson's ratios and shear moduli in each of these directions.

Table 4.7: Elastic material properties for GL30c

E1	E2	E3	Nu12	Nu13	Nu23	G12	G13	G23
13000 MPa	300 MPa	300 MPa	0.3	0.3	0.3	650 MPa	650 MPa	50 MPa

The knife and back plate is made of structural steel with a strength of S355. Unlike the timber, the steel is considered isotropic, and its behaviour is modelled using an elasto-plastic material model. Plasticity is included for the knife and the back plate in order to better capture local yielding and stress redistribution that may occur under higher load levels. The elastic properties of the steel used in ABAQUS for the knife and back plate are presented in Table 4.8, while the plastic properties are given in Table 4.9.

The bolts and screws are also modelled with an elasto-plastic material model. Plastic behaviour was included for the fasteners to allow for stress redistribution and to more accurately represent their mechanical response under load. The elastic properties for the fastners are the same as for the knife and back plate and are also presented in Table 4.8, but the fasteners are made of higher-strength steel and therefore have different plastic material properties, which are presented in Table 4.10.

Table 4.8: Elastic material properties used in ABAQUS, used in all steel parts

Young's modulus	Possion's Ratio
210 000 MPa	0.3

Table 4.9: Plastic material properties used in ABAQUS, for the knife and back plate

Yield stress	Plastic strain
355 MPa	0
530 MPa	0.0176

Table 4.10: Plastic material properties used in ABAQUS, for the bolts and screws

Yield stress	Plastic strain
640 MPa	0
800 MPa	0.12

The different parts of the connection are assembled, and interactions between them are defined. All contacting surfaces are assigned a normal behaviour with hard contact to prevent the parts from overlapping. Friction between parts is neglected due to the assumption of linear behaviour in the analysis. The parts are also allowed to separate after contact.

4.3.2 Boundary conditions and loading

In ABAQUS, different analysis steps are defined to allow boundary conditions (BCs) and loads to be applied at different stages and times. The initial step is predefined by ABAQUS and serves as the starting point of the simulation, where the boundary conditions are applied. Next, a loading step is defined. In this step, the external load is applied incrementally, using a step size of 0.2. However, this increment can be adjusted if the model fails to converge at a particular step. In the second and final loading step, the displaced support is introduced, using the same increment size as in the loading step.

Since the concrete core is not included in the ABAQUS model, the BCs need to simulate the interaction between the plate and the fictitious concrete. The actual connection is made using screws from the plate into the concrete. These screws are modelled until the end of the steel plate. The BCs are then placed on the end of the screws and these nodes are fixed in all degrees of freedom, assuming the screws are fully restrained by the concrete. The BCs will be placed at the initial step and can be seen in Figure 4.3.

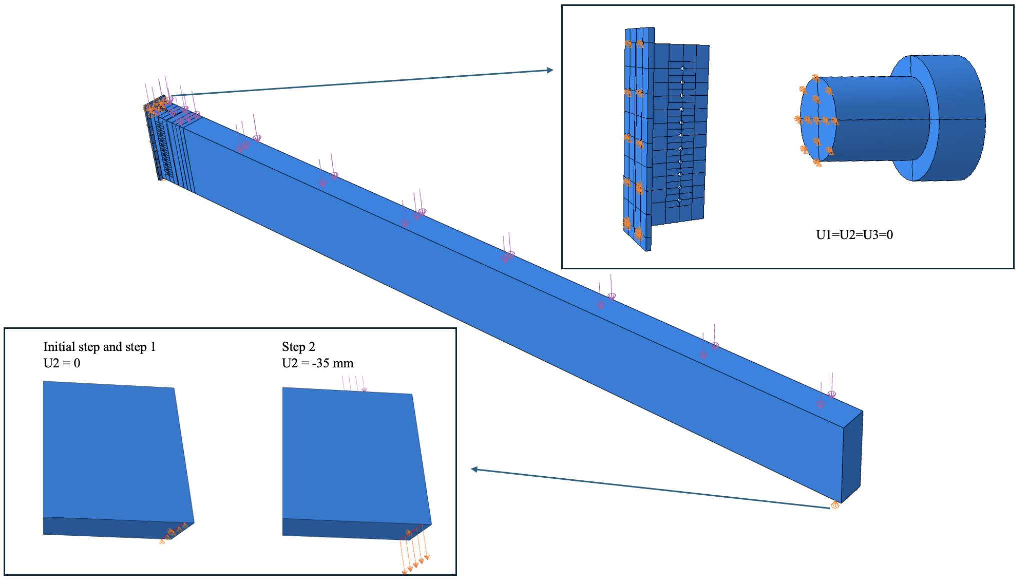


Figure 4.3: BCs and the load that are applied in ABAQUS

To further simulate the support provided by the concrete, a three-dimensional discrete rigid surface is implemented behind the plate. This surface is assembled parallel to the ends of the screws and is assigned a reference point, which is also locked in all degrees of freedom. In ABAQUS, a discrete rigid surface behaves as an undeformable plate that can interact with deformable elements through contact. It can push against other parts but would not let them pass through. This setup ensures that the steel plate cannot move into the concrete, resembling the physical back support of the concrete core.

On the other edge of the beam, the support opposite the connection, BCs are defined to simulate a support with stepwise deformation. In the initial step and the first loading step is the support modeled as pinned by restraining displacement in the U2 (vertical) direction along a line at the edge of the beam, as shown in the bottom left part of Figure 4.3. Additionally, one point is fully constrained in all degrees of freedom to ensure stability.

In the second loading step, the boundary condition aiming to represent the displaced support is introduced. The displacements are applied with increments up to a total vertical displacement of 35 mm. The displacement is applied along the same line as previously was constrained in U2.

The load is applied during the first loading step as a pressure on the top surface of the beam, indicated by arrows in Figure 4.3. The value of the applied load corresponds to the value calculated in Section 4.1.2.

4.3.3 Geometry and mesh

All parts are modelled as solid elements with the assigned element type C3D8R, which is an 8-node linear brick, reduced integration, and hourglass control, this element type is also used in Tako et al. (2023). To determine the element size of the mesh a convergence study, regarding the stresses and displacement in different parts is conducted. The detailed convergence study can be found in Appendix C. To achieve a valid model that accurately represents real-life behaviour while maintaining reasonable computational time, a finer mesh is applied in regions where stress concentrations occur and where the results are of greater interest. This is done to achieve accuracy while still maintaining efficiency throughout the process. A smaller mesh size is used throughout the beam region surrounding the connection. The bolts and steel plate also have refined mesh. All applied mesh configuration can be seen in Figure 4.4.

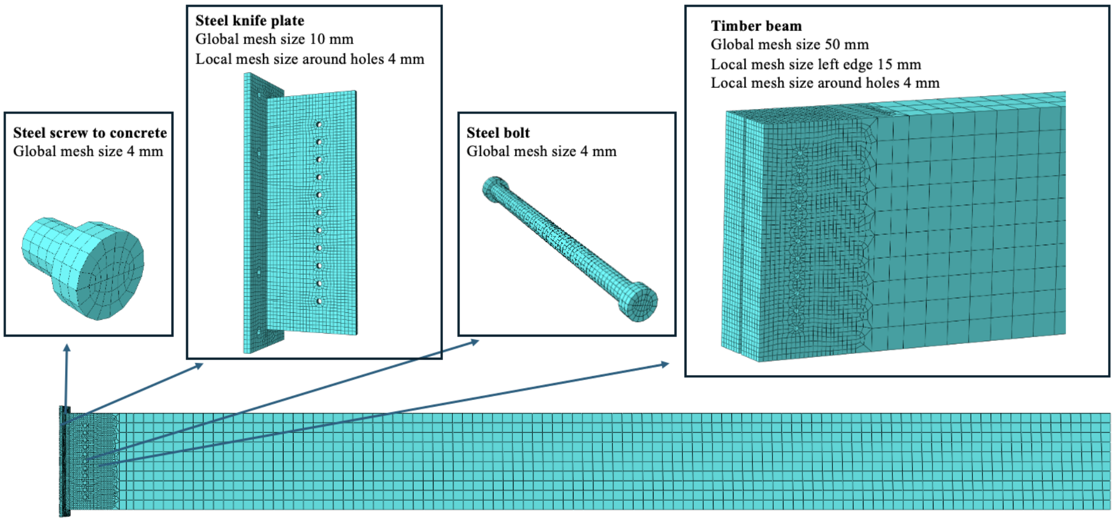


Figure 4.4: The different parts modelled in ABAQUS along with their corresponding mesh sizes

4.3.4 Extraction of data

To analyse the connection, various sets of data are collected. The primary focus is on how the displaced support effects moments, forces, and stresses in the modelled connection. For the glulam beam, stresses in different directions are analysed to assess to which extent the beam exceeds the critical values defined for this beam in Eurocode 5 (2004).

For the bolts, the distribution of forces between them is evaluated to observe how it changes when the displaced support is introduced. These forces are extracted in ABAQUS using the tool free body cut, where shear forces are extracted from the bolts in the section where the bolts interacts with the edges of the steel knife plate. The axial forces in the screws are also obtained using a free body cut at the end of the screws where they connect to the concrete core. The stresses in the bolts and the screws are also evaluated to determine when and if they reach their plastic capacity defined in section 4.3.1.

Regarding the steel knife plate, the regions expected to yield plastically are identified in order to study how and where the plate deforms. Additionally, the global moment of the connection is examined to assess the stiffness of the connection this moment is extracted at a section at the far end of the steel plate. The rotation of the connection is calculated by measuring the change in deformation between one point on the back plate and one point on the timber beam, just after the cut-out for the knife plate. The change in angle between these two points is then used to determine the rotation.

4.3.5 Verification of model

Verifications of the model are carried out to ensure that the results are reliable. Initially, simply supported timber and steel beams are modelled and compared with hand calculations to verify that the material properties are accurate. Then, the timber beam is modelled with an entirely fixed end on one side and a displaced support on the other end. This is done to gain an understanding of how to model the displaced support and to observe the resulting increase in moment at the fixed end, due to the added displacement.

To verify the reference model, reaction forces at the supports are extracted and compared with the applied load to ensure vertical equilibrium. Additionally, the deflection of the modelled beam is evaluated, with results falling between the deflection values for a beam with one end fixed and one simply supported, and a beam simply supported at both ends, see table 4.11 .

Table 4.11: Comparison of beam deflection for different support conditions

Support condition	Deflection [mm]
Simply supported	18.0
Fully fixed	12.1
Modelled connection	15.6

The analytical hand calculations follow standard methods, which assume a single elastic modulus for the timber beam. However, as previously discussed, timber is an orthotropic material with different strengths and stiffness in different directions. To ensure a fair comparison, the FE model used for verification was simplified by assigning the same single elastic modulus as in the hand calculations. Once the results were confirmed, the model was updated and the timber beam was assigned three distinct elastic moduli to better capture its true behaviour. The results of verifying the beam with one elastic modulus to hand calculations can be observed in Table 4.12

Table 4.12: Comparison between FEM results and hand calculations using single elastic modulus

	FEM	Hand calculation	Difference
Deflection [mm]	18.6	18.0	0.6

4.4 Method parametric study

To evaluate how variations in the embedded steel knife plate connection affect its performance under displaced support conditions, a parametric study was carried out. The study begins by analysing a reference model, which was established and verified in the previous section. This model serves as a baseline against which selected components are modified and evaluated. In each case, only one parameter is changed at a time, while the remaining components are reset to their reference configuration. This systematic approach ensures that the effect of each individual component can be isolated and assessed.

For each modification, the same set of results is evaluated in order to identify trends and draw conclusions. The parameters studied include rotational stiffness (based on moment–rotation behaviour at the connection), stresses in the timber beam both parallel and perpendicular to the grain, plastic behaviour in the steel components, and the axial and shear forces in the screws and bolt. The components modified in the study are bolt diameter, bolt configuration, knife plate thickness, and back plate thickness. All tested configurations are presented in Table 4.13, where the grey-highlighted values represent the reference model as defined in Section 4.2.2. The reference values remain unchanged when any of the other parameters are modified.

Table 4.13: Overview of parametric cases and varied connection parameters, where the grey parameters are used as the reference model

Bolt diameter [mm]	Bolt configuration	Knife plate thickness [mm]	Back plate thickness [mm]
12	1x11	15	5
16	2x11	20	8
20	2x9	25	10
24	2x6		15
	2x4		20

5

Parametric study

In this chapter, the results from the parametric study are presented. The evaluation begins with the reference model, followed by individual investigations where one component is modified at a time in each section. For each case, the same set of parameters is assessed, including the effect of the displaced support.

5.1 Reference model

To initiate the parametric study, the reference model defined in Section 4.2.2 is established and analysed. This model serves as a baseline for evaluating the impact of the variations introduced in following sections. The following section presents and analyses the behaviour of the connection under the applied displacements. These findings provide a foundation for comparison when assessing the influence of parameter changes in upcoming chapter.

5.1.1 Moment and stiffness

The moment is extracted at the knife plate close to the back plate and represents the moment in each connection configuration. To evaluate this moment and define the corresponding stiffness, a moment-rotation curve is used, see Figure 5.1. A fully fixed connection allows no rotation as the moment increases, resulting in a steep, almost vertical line. A pinned connection does not transfer any moment and would therefore result in a horizontal line in the moment–rotation diagram. The dot on the graph indicates the time when the displacement of the opposite support is introduced. What this graph shows on its own is that the response of the reference model is elastic, as indicated by the straight line, curved slopes would suggest plastic behaviour.

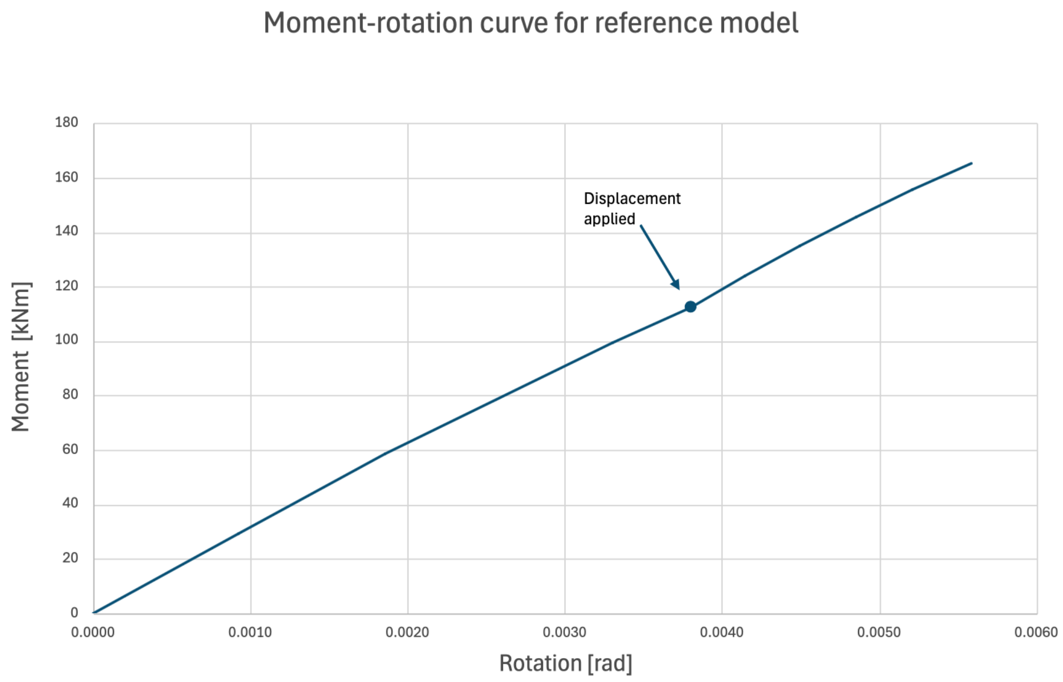


Figure 5.1: Moment-rotation curve for reference model

Table 5.1 presents the degree of rigidity (R) of the connection by comparing it to a fully fixed beam, in order to evaluate where the connection lies between a pinned and a fixed condition. The degree of rigidity ranges between 0 and 1, where 0 represents a fully pinned connection and 1 represents a fully fixed connection. The rotational stiffness (k), given in MNm/rad, describes the moment required to achieve rotation. A higher value of k indicates a stiffer connection. For both the degree of rigidity and the rotational stiffness, the values are taken from the models while they are still in the linear range. The degree of rigidity is calculated from the moment in the elastic range and then divided by the moment of a fixed beam subjected to the same load. The rotational stiffness is determined from the slope of the moment-rotation curve.

In this case, the connection shows a rigidity degree of 0.59, which means that this theoretical 'pinned' connection actually behaves almost halfway between a pinned and a fixed connection and should therefore rather be looked at as semi-rigid.

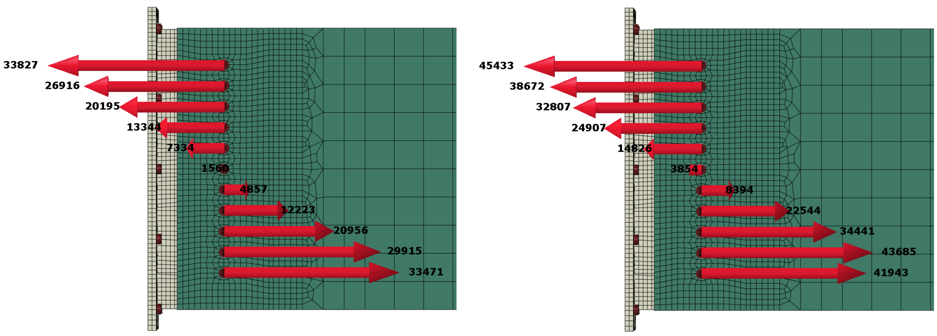
Table 5.1: Rotational stiffness and degree of rigidity for the reference model

	Reference model
Rotation stiffness k [MNm/rad]	31.4
Degree of rigidity R [-]	0.59
Degree of rigidity with displacement $R_{\text{displaced}}$ [-]	0.83

When looking at the moment in the connection with the full displacement applied, a substantially higher moment is observed. This value can be seen as a new degree of rigidity when the displacement is also active, referred to as $R_{\text{displaced}}$ in Table 5.1. However, this value is not extracted for all following configurations, since the concept of degree of rigidity is only valid when the system remains linear. Still, it gives a good indication of how much the displacement of the support increases the moment and thus adds more strain on the connection. In this specific model, the connection behaves as if it is 59 % fixed under load only, and as if it is 83 % fixed when the effect of displacement is added.

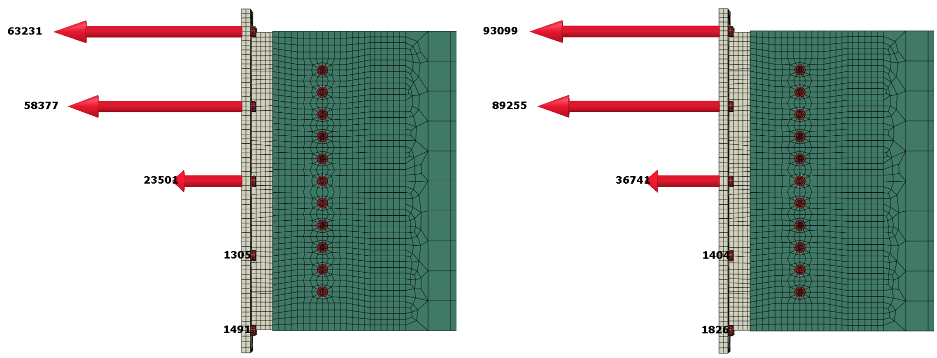
5.1.2 Forces in bolts and screws

The reaction forces in the bolts connecting the steel plate to the timber beam remains relatively symmetric during the step when the load is applied. As the displacement increases, all bolt forces rise, with the top and bottom bolts showing the most significant change and magnitude, see figure 5.2. However, during the application of the displacement of the support, the difference between bolts starts to level out, indicating a potential redistribution of forces, possibly due to plastic behaviour in the most loaded bolts. At the final state, when the displacement is 35 mm, the bolt above the bottom bolt have reached and surpassed the bottom bolt.



Support not displaced | Support displaced 35 mm

Figure 5.2: Forces in bolts [N], for reference model



Support not displaced | Support displaced 35 mm

Figure 5.3: Axial forces in screws [N], for reference model

The screws connecting to the concrete experience significant tensile forces, particularly in the upper three rows of screws, while the lower two rows mainly resists vertical loads, see

Figure 5.3. The middle row of screws, located at the vertical center of the plate, is also subjected to tension, suggesting that the tension zone of the steel backplate is larger than the compression zone. The screws seems to redistribute forces, since the difference between the upper two screws gets smaller from the addition of the displacement.

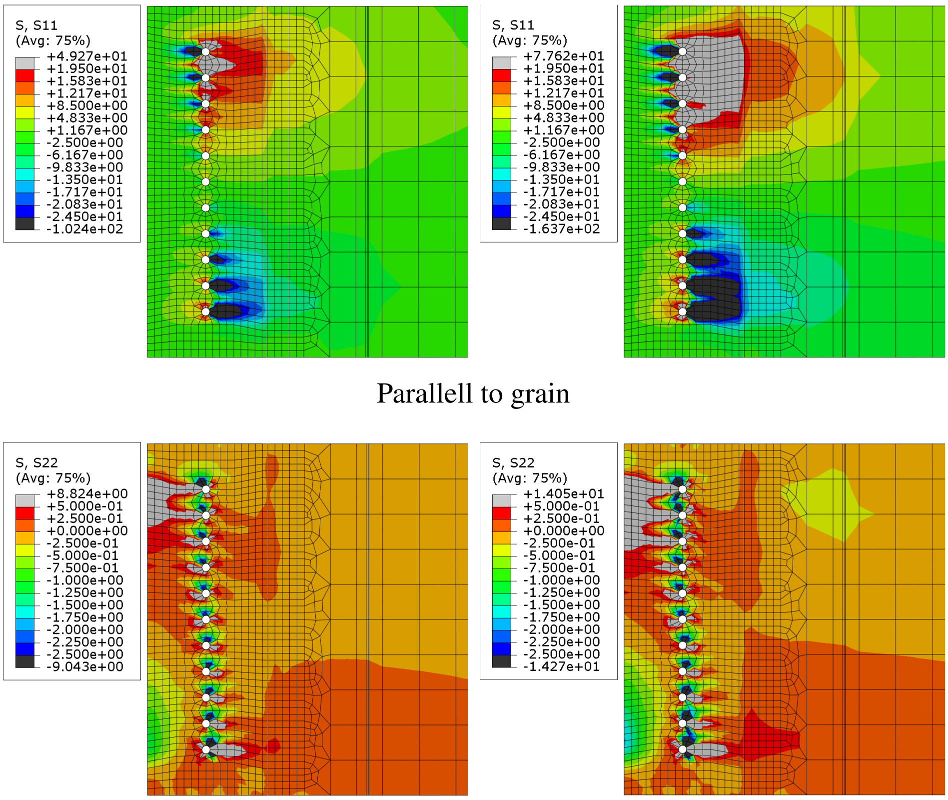
Table 5.2 aims to demonstrate when the different components in the connection begin to plasticise. Time is indicated by showing the percentage of applied load and displacement of the support. In this case, the table shows that all components start to plasticise during the load application. However, these timings are extracted based on the first signs of plastic stresses in the components, which may be very small and localised. Therefore, they do not necessarily have a direct impact on the overall ductility of the connection at that stage.

Table 5.2: Loading step at which the different steel components begin to plastically deform

Plate Thickness	**15 mm**	
	Applied load	Displacement
Knife plate	90 %	-
Bolt	50 %	-
Screw	100 %	-

5.1.3 Stress in timber beam

After 35 mm of applied displacement on the support, the areas where stresses become critical in the glulam beam expand significantly. This is visible in Figure 5.4, where the grey and black areas show where the stresses exceed the strength of the material. The grey areas represent the zones where the tension capacity is exceeded and the black areas are the zones where the pressure capacity is exceeded. The increased displacement leads to higher compression and tension in the beam element, posing a high risk of failure. Even before any displacement is applied, localised stresses in the beam already exceed the strength of GL30c. Although these stresses are limited to small areas, the location and extent of the critical zones are still important to note.



Perpendicular to grain

Left column: Support not displaced **Right column:** Support displaced 35 mm

Figure 5.4: Stresses in timber beam for reference model

When comparing the stresses in the beam parallel and perpendicular to the grain, it is clear that the applied displacement has a much greater impact on the former. This is expected, as the displacement induces moments that generate higher stresses in the horizontal direction of the beam, along the fibres. As a result, the critical areas parallel to the grain are significantly larger, increasing the risk of failure. The increase in critical area perpendicular to the grain is less pronounced, although it still exists and grows when displacement is introduced. Generally, when dimensioning timber beams for this type of load case, the strength in the perpendicular direction is considered the most critical factor due to the material's more brittle behaviour. However, these results suggest that when differential displacement is introduced, stresses parallel to the grain may be equally important, or even more critical, to consider.

5.1.4 Main insights and strategy moving forward

After evaluating the results from the reference model, the biggest takeaways are the moments, both at the initial and final step, as well as the stresses in the timber beam. It is clear that the connection is not pinned and thereby attracts moments, particularly when the imposed displacements are applied. This relative stiffness attracts stresses to the beam that drastically exceed its capacity. Moving forward, the main focus will be to investigate measures that affect the stiffness of the connection and the effect that has on the timber beam. To do this, the idea is to change the parameters that hypothetically can affect the rigidity and the ductility of the connection. As noted in Section 3.2, steel components are the most ductile components of an assembly. In our case, we have the bolts connecting the timber to the knife plate, the knife plate itself, as well as the back plate of steel parallel to the concrete wall.

5.2 Size of bolts

Different bolt sizes are evaluated in this section. To allow for a fair comparison between the different models, only the bolt size is varied. As a result, the model does not follow all the guidelines in Svenskt trä (2018) that were followed when designing the reference model. The bolt diameters considered are 12 mm, 16 mm (reference model), 20 mm, and 24 mm.

5.2.1 Moment and stiffness

The moment at the connection initially increases with the applied load and continues to increase as the displacement of the support is applied. Figure 5.5 presents the moment for various bolt diameters as a function of rotation, resulting from both the applied load and the displacement of the support. The diagram demonstrates that the moment increases with greater rotation. It also shows that bolts with larger diameters generate a higher moment at lower rotation levels, resulting in a steeper inclination of the curve. This is because increasing the bolt diameter attracts more load and better resist rotation, resulting in a stiffer connection. In comparison, the model with 12 mm bolts shows a decreasing gradient once the support starts to displace. This may suggest that the steel in the bolts has reached its plastic limit, leading to a reduction in stiffness.

Notably, for configurations with larger bolt diameters, the inclination of the curve becomes steeper after the dot (when the displacement is introduced). The explanation for this is that when the displaced support is added, the displacement reduces the rotation caused by the applied load. Together with the fact that the moment still increases, this makes the connection appear stiffer when the displaced support is added.

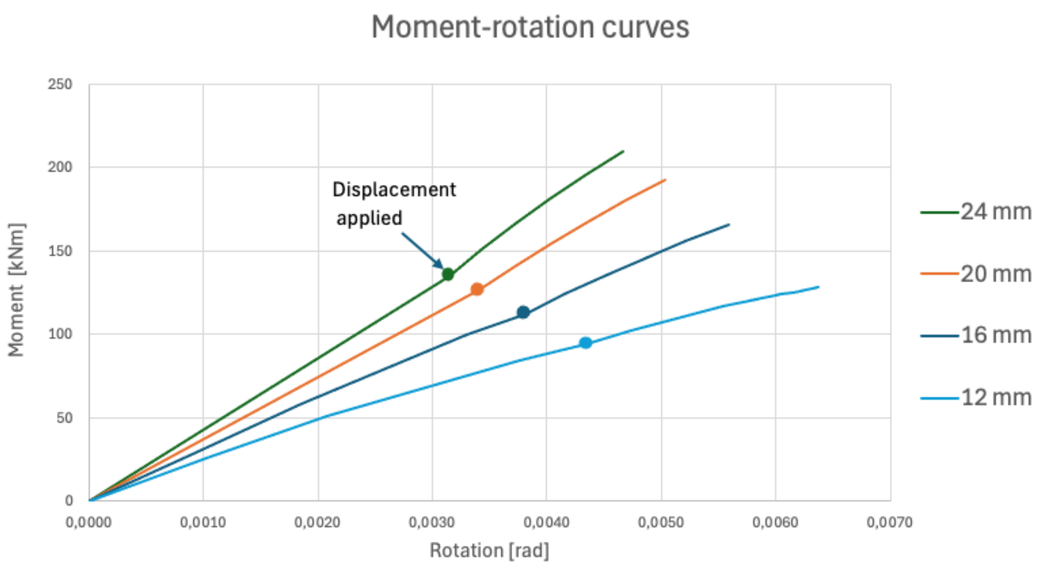


Figure 5.5: Moment-rotation curve for different bolt diameters. The load is applied up to the marked point; beyond this point, the displaced support is introduced

The stiffness of the connection is reflected in the slope of the curves in Figure 5.5, within the elastic range of the connection. The corresponding rotational stiffness and the degree of rigidity are presented in Table 5.3. The table shows that connections with larger bolt diameters demonstrate greater rigidity. The 20 mm and 24 mm bolt diameter maintain a consistently steeper curve compared to both the 16 mm and the 12 mm diameter, which points to a higher degree of structural stiffness. In contrast, the 12 mm bolts result in a noticeably flatter curve, indicating lower rigidity and allowing greater rotation under the same moment. Figure 5.5 also shows that for larger bolt diameters, the curve continues to rise with increasing load, indicating that the connection maintains its stiffness throughout the support displacement, without signs of global plastic deformation. This behaviour is not observed for the 12 mm bolts, where the curve starts to level out and even decrease, indicating that the connection is entering a plastic state.

Table 5.3: Rotational stiffness and degree of rigidity for different bolt diameters, where a fully fixed connection is 1 and a simply supported connection is 0

Bolt diameter	**12 mm**	**16 mm**	**20 mm**	**24 mm**
Rotation stiffness k [MNm/rad]	25.26	31.51	37.09	42.96
Degree of rigidity R [-]	0.51	0.56	0.63	0.68

5.2.2 Forces and plastic deformation in steel parts

The different steel components in each connection begin to plastically deform at different loading stages, as shown in Table 5.4. The results in the table indicates that the bolts and the steel plate typically start yielding around the same time, often before the screws to the concrete. As the diameter of the bolt increases, both the bolts and the knife plate can carry more load before plastic deformation occurs, due to their larger cross-sectional area. The screws, however, are less affected by the bolt size, even though the overall stiffness of the connection increases, as seen in Table 5.3.

Table 5.4: Loading step at which the different steel components begin to plastically deform

Bolt Diameter	**12 mm**		**16 mm**		**20 mm**		**24 mm**	
	Applied load	Disp	Applied load	Disp	Applied load	Disp	Applied load	Disp
Knife plate	50 %	-	90 %	-	80 %	-	80 %	-
Bolt	50 %	-	50 %	-	60 %	-	80 %	-
Screw	100 %	-	100 %	20 %	100 %	-	100 %	-

The plastic strain in the knife plate for the different bolt sizes is presented in Figure 5.6. The areas with plastic strain indicate where the material has reached its yielding capacity and plastic deformation has begun. When comparing the different bolt sizes, the overall pattern of deformation remains similar, with plastic strain mainly localized around the top and bottom holes. However, for the connection with 12 mm bolts, plastic deformation occurs around both the top two and bottom two holes, suggesting that the load from the second row of bolts has increased due to redistribution. In the connection with 24 mm bolts, a small amount of plastic strain can also be observed in the upper corner where the knife plate meets the back plate. This indicates a stiffer connection, which is confirmed from Table 5.3.

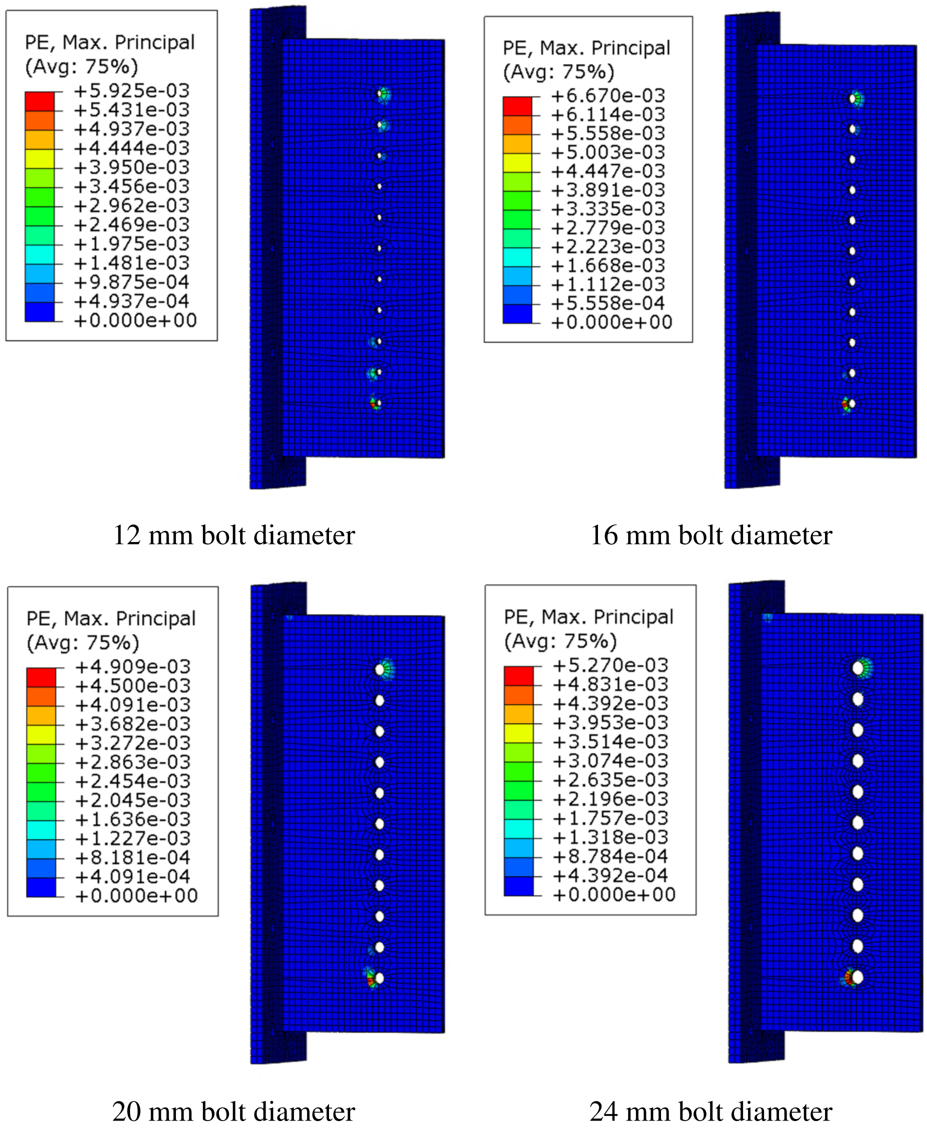


Figure 5.6: Plastic strain in a steel knife plate for different bolt diameters when the support is displaced by 35 mm. The blue areas do not experience any plastic strain, while the other coloured areas do

Figure 5.7 presents the horizontal forces in the top and bottom bolts for various bolt diameters. The diagram shows both the forces without displacement and the increase in forces that comes from the displaced support. From the digram it can be seen that the axial force in the top and bottom bolts increases when the support is displaced. The diagram also shows that the force in the bolts generally increases with bolt diameter. When comparing the change in force without displaced and withe displaced support, the connection with 12 mm bolts stands out. While the differences in force for the larger bolts are relatively similar, the 12 mm bolts show a noticeably smaller change. This may indicate that the top and bottom bolts with a 12 mm diameter have begun to plasticize, causing the load to redistribute to the other bolts in the connection.

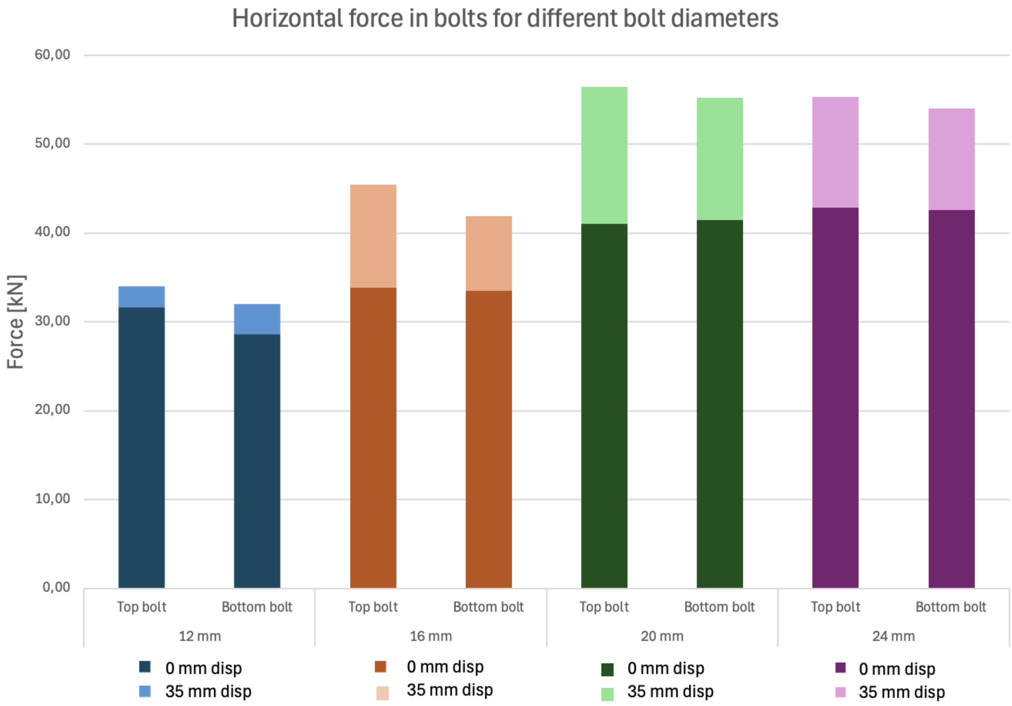


Figure 5.7: Horizontal forces in the top and bottom bolts for different bolt diameters, without displaced support and withe displaced support

Only the top three screws carry substantial axial load, so these are the focus of the analysis. Figure 5.8 presents the axial forces in these screws for various bolt diameters, both with and without a 35 mm support displacement. From the diagram, it is clear that the force in the screws increases with larger bolt diameters. This correlates to the increase in rigidity and moment that the larger bolts demonstrate, and the higher moment leads to an increase in the axial forces in the screws. For all bolt sizes, Screw 2 shows the greatest increase in force when the support is displaced. This may be due to the first screw beginning to plasticize, which would lead to a redistribution of load to the surrounding screws.

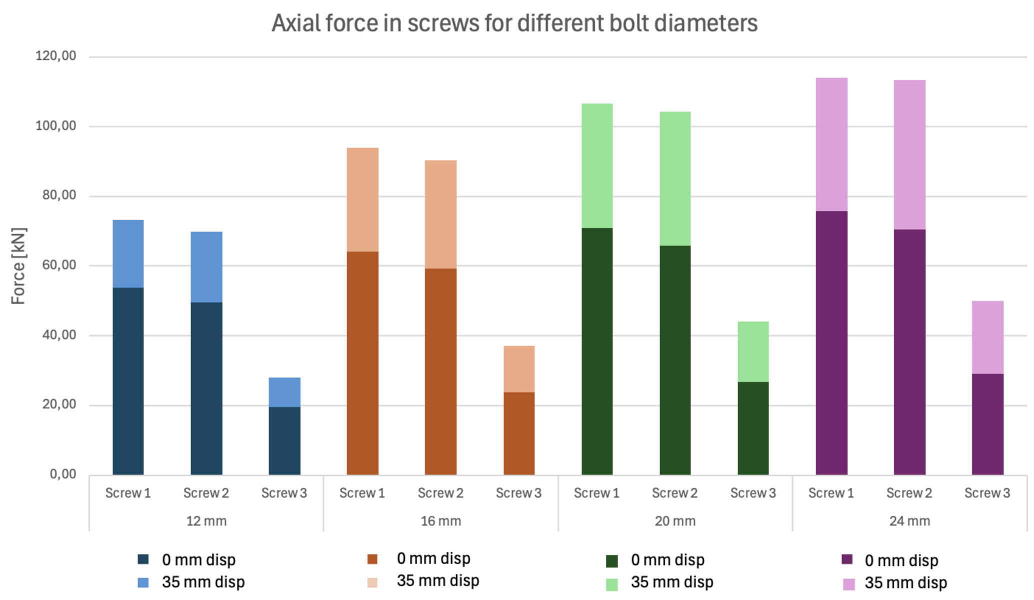
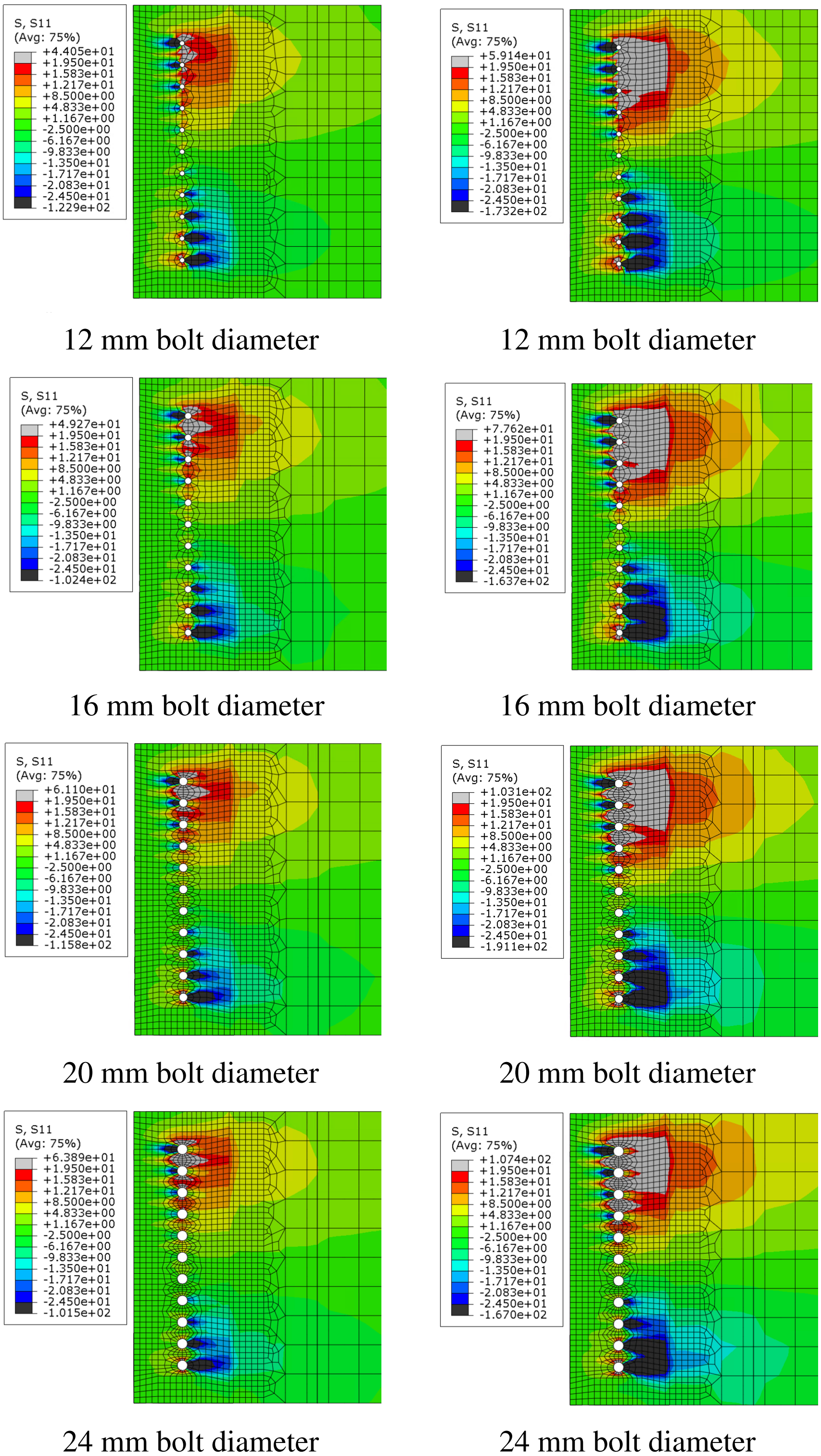


Figure 5.8: Axial forces in the top three screws for different bolt diameters, without displaced support and withe displaced support

5.2.3 Stress in timber beam

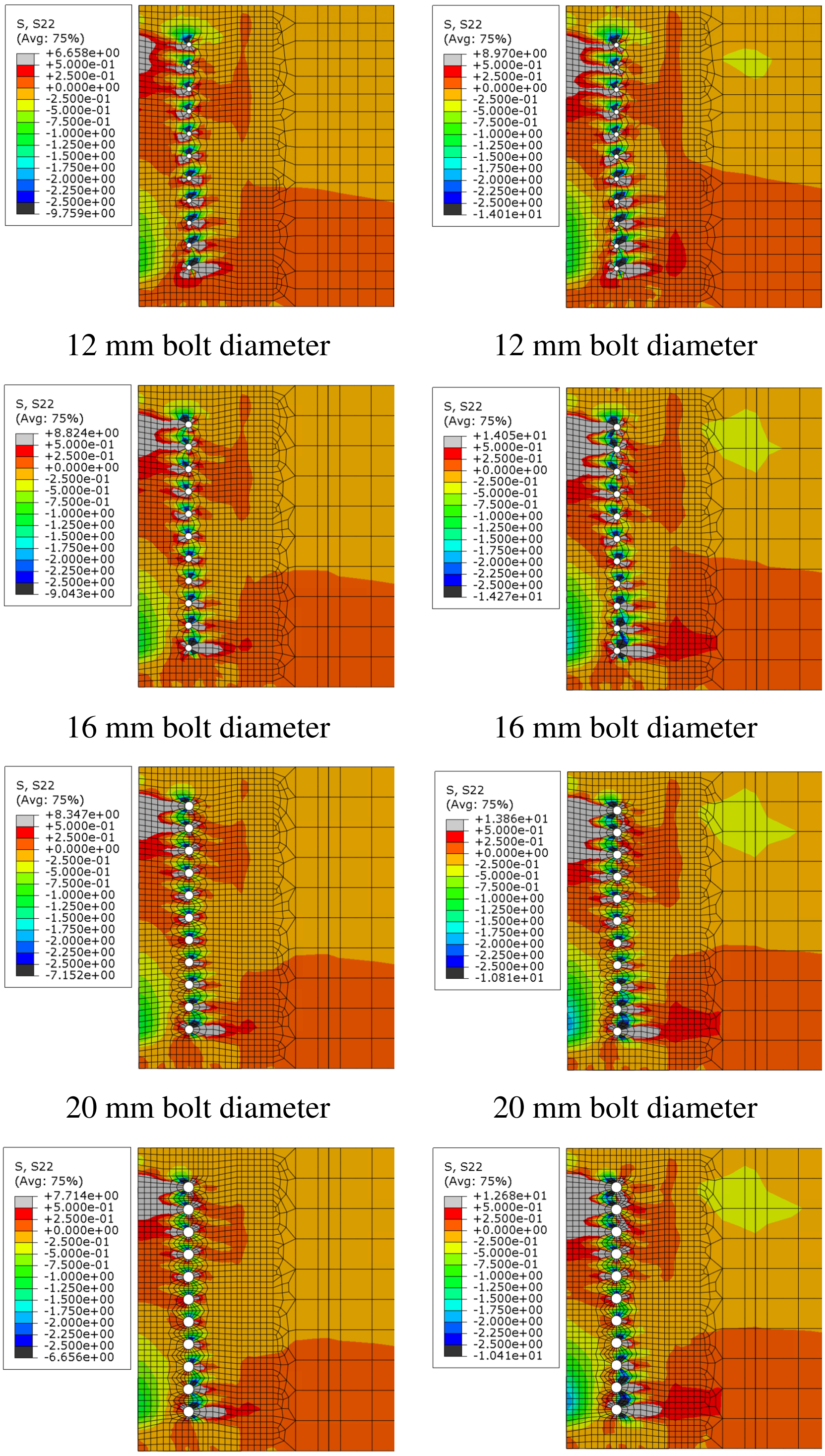
The stress in the timber beam increases both parallel and perpendicular to the grain as the support deforms. This leads to an expansion of the critical area, defined as the region where stress exceeds the strength of the timber. Figure 5.9 shows the stresses parallel to the grain for all bolt diameters, comparing the case with no support displacement to the case with a 35 mm support displacement. In the figure is the white area plastic deformation in compression and the black area is in tension.

The results show that the critical area grows noticeably when the support is displaced. While the stress distribution patterns are similar across the different bolt sizes, the extent of the critical area is slightly influenced by the bolt diameter. In particular, the configuration with 24 mm bolts appears to produce a somewhat smaller critical zone compared to the others, indicating a marginally better distribution of stress. It can also be observed that for all bolt sizes, the highest stress concentrations occur around the bolts themselves, especially at the upper and lower bolts. The overall shape of the critical area remains consistent, but its intensity and spread increase with the displacement of the support. These results suggest that while bolt diameter has some effect on stress distribution, the displacement of the support plays a more significant role in determining the size of the critical area.



Left column: Support not displaced **Right column:** Support displaced 35 mm

Figure 5.9: Stress in the middle of the timber beam parallel to the grain with critical areas in grey and black for different bolt diameters



12 mm bolt diameter | 12 mm bolt diameter

16 mm bolt diameter | 16 mm bolt diameter

20 mm bolt diameter | 20 mm bolt diameter

24 mm bolt diameter | 24 mm bolt diameter

Left column: Support not displaced **Right column:** Support displaced 35 mm

Figure 5.10: Stress in the middle of the timber beam perpendicular to the grain with critical areas in grey and black for different bolt diameters

The results for the stress perpendicular to the grain are shown in Figure 5.10. As with the parallel stress, an increase in stress is observed when the support is displaced by 35 mm compared to no support displacement. However, unlike the parallel stress results, there are clearer differences in the critical area between the different bolt diameters. Notably, the 12 mm bolt configuration consistently shows the smallest critical area, both when the support is not displaced and when it is displaced by 35 mm. The critical area increases with bolt diameter, with the 16 mm, 20 mm, and 24 mm bolts producing progressively larger zones of high stress. This trend suggests that smaller bolt diameters result in a more localized stress distribution.

One explanation for this behaviour is that the lower stiffness of connections with smaller bolt diameters allows for greater deformation, which may reduce stress concentrations perpendicular to the grain. This is because the moment is smaller, and as a result, the timber does not need to resist as much force. Despite these differences, the overall pattern of stress distribution remains similar across all configurations.

5.3 Configuration of bolts

In this section, different configurations with two columns of bolts are evaluated and compared to the reference model, which consists of a single column. Four alternative models are considered: one with 2×11 bolts, chosen to match the number of bolts per column in the reference model; another with 2×9 bolts, designed in accordance with the guidelines in Svenskt trä (2018); and two additional models with 2×6 and 2×4 bolts, included to investigate the behaviour of the connection when the capacity is reduced. These latter configurations, while below the recommended design capacity, are useful for gaining insight into how the connection performs under less favourable conditions and to better understand the failure.

5.3.1 Moment and stiffness

The moment increases initially with the applied load and continues to rise as the support deforms for all bolt configurations. This is illustrated in Figure 5.11, where the moment is plotted as a function of rotation. The graph shows that models with two columns of bolts develop a higher moment, indicating greater stiffness. Additionally, having more bolts in the columns further increases the moment. When comparing a configuration with one column of 11 bolts to a setup with two column of 9 bolts, both designed to have similar capacity according to Svenskt trä (2018), it becomes clear that the 2×9 bolt model attracts a greater moment. Similarly, when comparing the 1x11 bolt configuration to the 2x6 bolt configuration, which have nearly the same effective number of bolts, the two-column model again shows a higher moment capacity. The moment-rotation curves also indicate that all models, except one, remain within the elastic range, as the curves are still linear. The exception is the model with 2x4 bolts, where the curve begins to decline once the support starts to deform. This suggests that the connection has begun to plasticize, meaning the deformation is permanent and the connection will no longer be able to resist the moment in the same way.

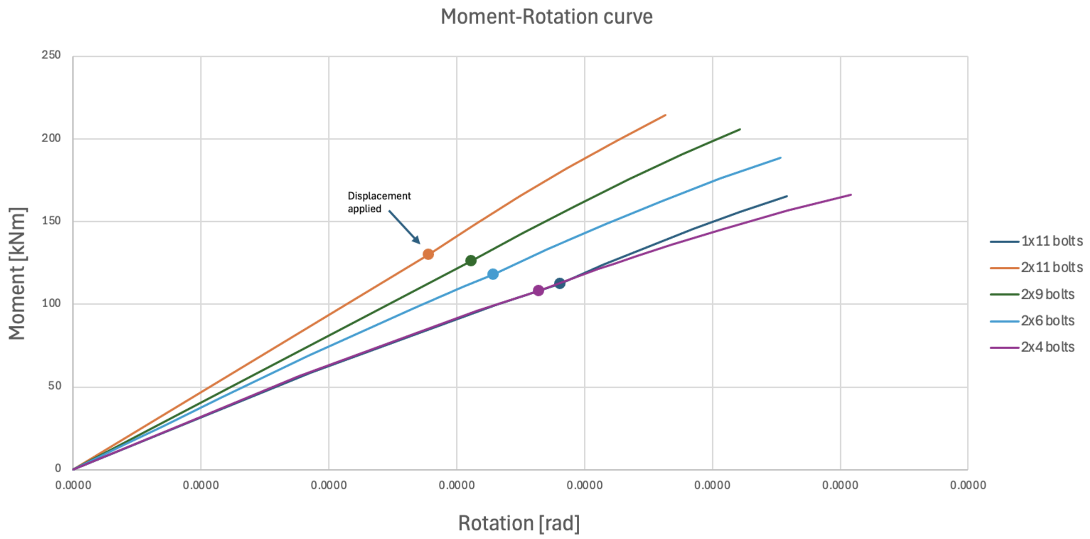


Figure 5.11: Moment-rotation curve for different bolt configuration. Until the dot is the load applied and after the dot will the support start to deform until 35 mm

Table 5.5 presents the rigidity values for the different bolt configurations, where a higher value indicates a stiffer connection. The results show that connections with two columns of bolts provide noticeably higher rigidity compared to those with a single ciolumn. Additionally, rigidity tends to increase with the total number of bolts, suggesting that both the layout and the quantity of bolts play a significant role in the overall stiffness of the connection.

It can also be observed that the connection with 1×11 bolts has a similar degree of rigidity as the one with 2×4 bolts. This indicates that the positioning of bolts, particularly those placed at the top and bottom, has a strong influence on the connection's stiffness. This is because the bolts are placed at the same position from the moment centrum, and also further away, and therefore getting most affected by the moment. In the 2×4 configuration, there are two bolts at both the top and bottom, compared to just one in the 1×11 configuration, which may explain the comparable stiffness despite the overall lower number of bolts

The configuration with 1×11 bolts shows a similar stiffness to the 2×4 configuration, despite having more bolts. This can be attributed to the placement of bolts in relation to the neutral axis. In the 2×4 configuration, are two bolts located at both the top and bottom of the connection, positions that experience the highest moment due to their distance from the neutral axis. These bolts contribute significantly to rotational stiffness. In contrast, the 1×11 configuration has only one bolt at the top and bottom, while most bolts are located closer to the centre, where their contribution to stiffness is smaller. This explains the comparable rigidity, despite the difference in effective number of bolt.

Table 5.5: Rotation stiffness and degree of rigidity for different bolt configurations

Bolt configuration	1x11 bolts	2x11 bolts	2x9 bolts	2x6 bolts	2x4 bolts
Rotation stiffness k [MNm/rad]	31.51	46.75	40.51	37.41	32.02
Degree of rigidity R [-]	0.56	0.65	0.63	0.59	0.57

5.3.2 Forces and plastic deformation in steel parts

The start of plastic deformation varies depending on the bolt configuration. Table 5.6 presents the loading stage or support displacement level at which plastic deformation begins in the different steel components of the connection, namely the bolts, the knife plate, and the screws anchored to the concrete. In general, the bolts are the first to yield across most configurations, followed by the knife plate and finally the screws.

Table 5.6: Loading step at which the different steel components begin to plastically deform

Bolt Configuration	1x11		2x11		2x9		2x6		2x4	
	Applied load	Disp	Applied load	Disp	Applied load	Disp	Applied load	Disp	Applied Load	Disp
Knife plate	90 %	-	100 %	-	100 %	40 %	100 %	20 %	90 %	-
Bolt	50 %	-	80 %	-	100 %	-	60 %	-	50 %	-
Screw	100 %	-	100 %	20 %	100 %	60 %	100 %	40 %	80 %	-

The results show that the 2×9 configuration can withstand the highest load and displacement before any plastic deformation occurs. In comparison, the 2×11 configuration, despite having more bolts, experiences plastic deformation earlier. This behaviour is likely due to the increased stiffness of the 2×11 configuration, which attracts larger moments and leads to higher stresses in the individual components. Configurations with fewer bolts or a single column, such as 1×11, also show earlier plastic deformation. This is likely because the load is distributed over fewer bolts, resulting in higher stresses per bolt. The 2×9 configuration therefore appears to provide the most balanced load distribution, able to delay plastic behaviour more effectively than more rigid configurations or with less bolted configurations.

Plastic deformation in the steel knife plate occurs primarily locally around the top and bottom bolt holes. However, in models with two columns of bolts, plastic deformation can also be observed in the upper corner where the knife plate is connected to the back plate, as shown in Figure 5.12. This shift in location is particularly noticeable in the stiffer configurations (2x11 and 2x9), where no visible plastic deformation appears around the top holes, but there is plastic deformation in the top corner. Having plastic deformation in the upper corner instead of around the bolts is beneficial. It reduces the risk of local failure around the bolt holes, where stress concentrations can otherwise lead to early damage. The corner region can typically accommodate more plastic strain due to its structural support and geometry, improving the overall ductility.

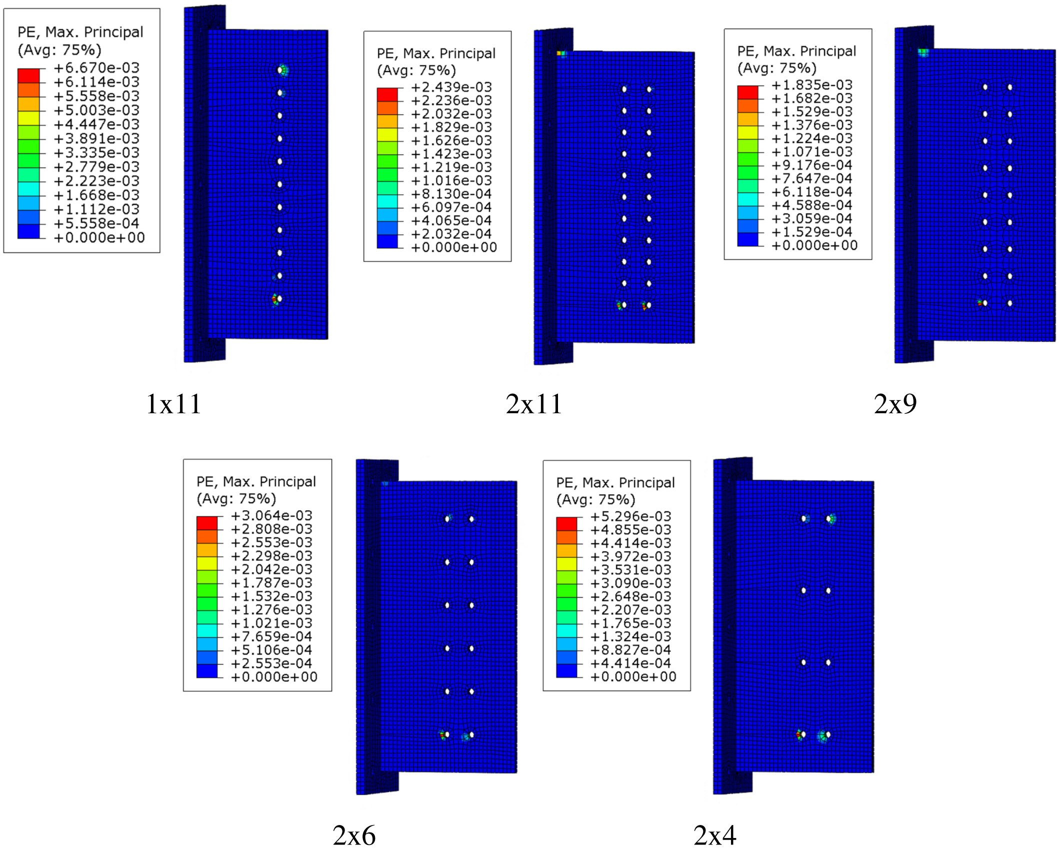


Figure 5.12: Plastic strain in the steel knife plate for different bolt configurations with a support deformation of 35 mm. The blue areas do not experience any plastic strain, while the other coloured areas do

The horizontal reaction forces in the bolts are highest in the top and bottom bolts, therefore only these are shown in Figure 5.13. The diagram presents these forces for various bolt configurations, both with and without a 35 mm support displacement. From the results, it can be observed that the total force in the top and bottom bolts generally increases when a second column of bolts is added. Additionally, the overall force in each individual bolt tends to decrease as the total number of bolts decreases, except for the 2×4 configuration, which shows the highest total force among all setups. In general, the results suggest that increased rigidity leads to higher forces in the bolts, with the 2×4 configuration being an exception to this trend. This is probably because, at this point, it is no longer the lower stiffness that contributes most to the forces. Instead, it is the fact that with fewer bolts, is the load on each individual bolt larger, since there are fewer fasteners available to carry the load. For the configurations with two columns of bolts, there is a noticeable shift in how the load is distributed, depending on whether the support is displaced. When the support is not displaced, the bottom bolts carry a greater portion of the load. However, when the support is displaced by 35 mm, this changes and the top bolts take on more load, suggesting a redistribution of force due to altered boundary conditions.

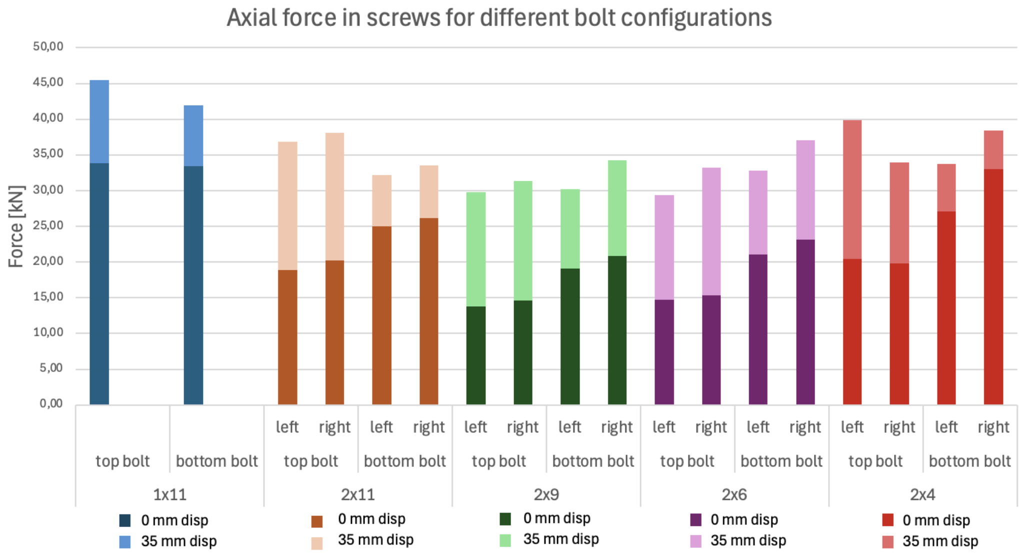


Figure 5.13: Horizontal force in the top and bottom bolts for different bolt configurations

The axial forces in the top three screws are presented in Figure 5.14. When comparing the different bolt configurations, it can be observed that the configurations with two columns of bolts initially generates lower axial forces in the screws than the single-column bolt configuration, when the support is not displaced. However, once the support is displaced by 35 mm, the forces in the screws from the two-column bolt configurations increase significantly, surpassing those from the single-column configuration. Additionally, as the number of bolts decreases, the forces in the screws also tend to decrease. This trend may be related to the increased rigidity of connections with more bolts, which in turn attracts a greater moment.

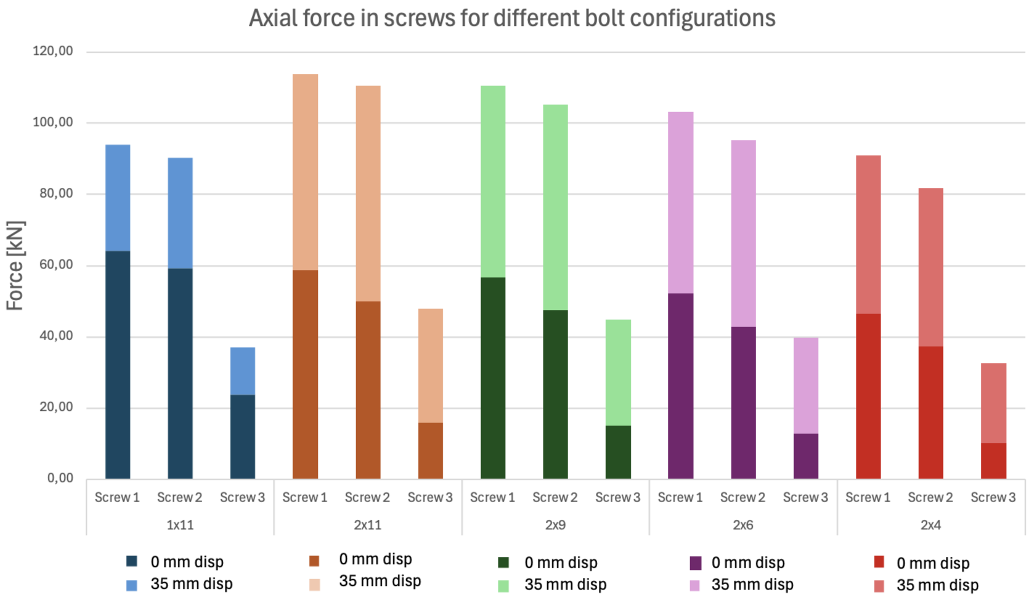
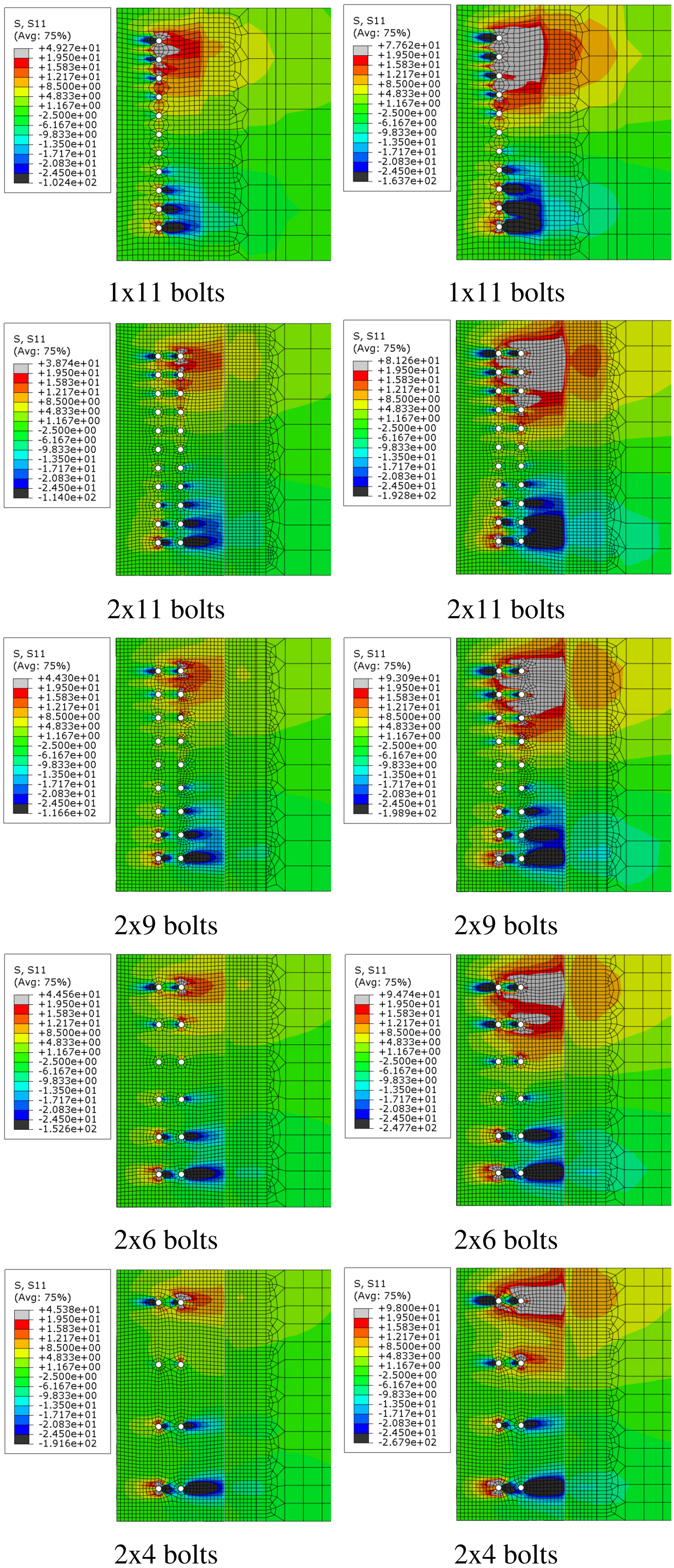


Figure 5.14: Axial forces in the top three screws for different bolt configurations

5.3.3 Stress in timber beam

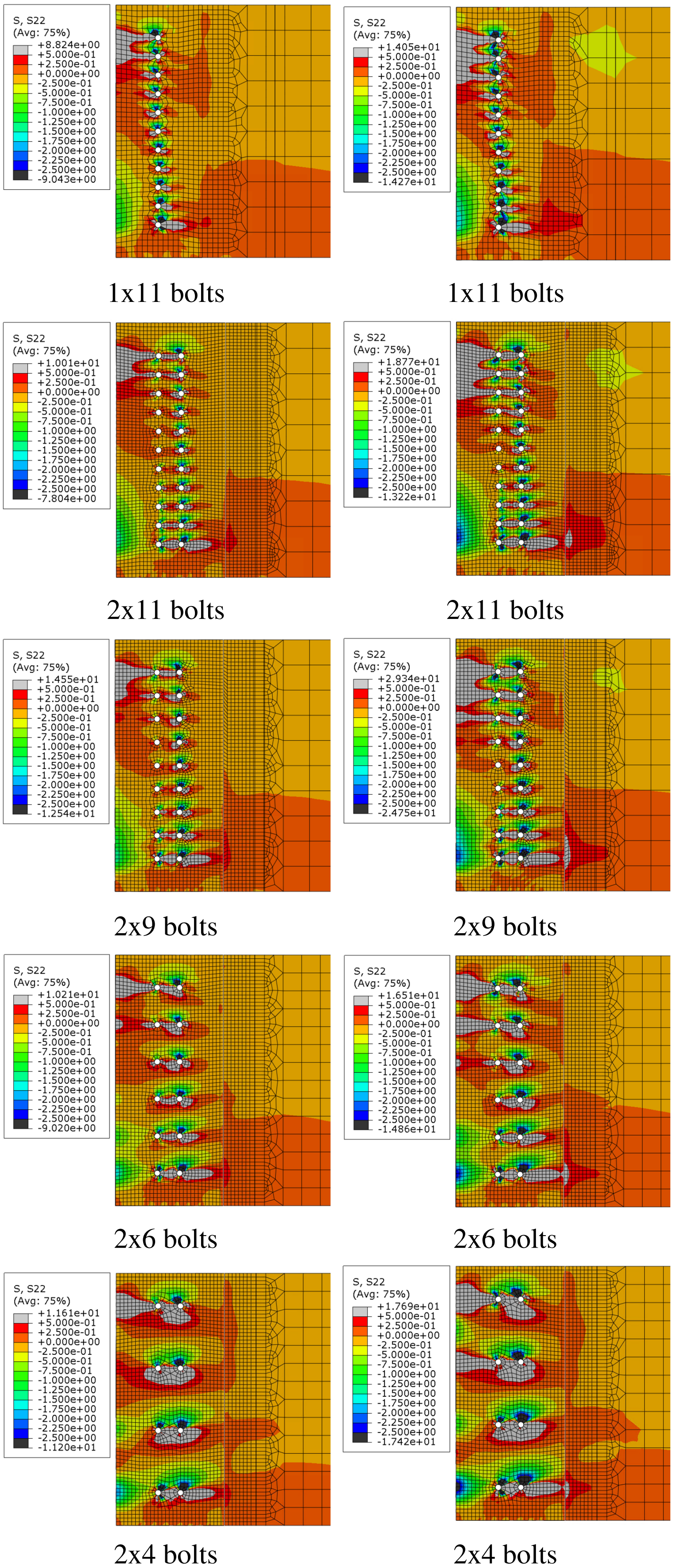
When analysing how the stress parallel to the grain in the timber beam varies between the different configurations, it is evident that using two columns of bolts reduces the critical area that plasticise, both with no displacement of the support and when the support is displaced by 35 mm. This is shown in Figure 5.15, which presents the stress parallel to grain in the middle of the beam by the connection. The figure also illustrates that the critical area increases as the number of bolts increases. However, the highest stress is larger for 2x4 bolt configuration, which indicts that the area becomes smaller but more concentrated. The smallest critical area is found in the connection with 2x4 bolts. However, even the connection with two columns of 11 bolts shows a smaller critical area compared to the configuration with one column of 11 bolts. It can also be observed that the critical area increases more at the top, where the timber is subjected to tension when the support is displaced. One explanation is that timber is weaker in tension than in compression, but it could also be due to the effect of the displaced support. An accumulation of plastic deformation can also be observed, particularly in the 1×11, 2×11, and 2×9 configurations. This concentration of deformation around closely spaced fasteners may increase the risk of group failure, where the surrounding timber fails collectively rather than around individual bolts.

When analysing the stress perpendicular to the grain in the middle of the timber beam, the smallest critical area is found in the connection with 1x11 bolts, which is the opposite trend compared to the parallel stress. Figure 5.16 presents the stress perpendicular to the grain, showing that the connection with a single column of 11 bolts has the lowest critical area. The critical area increases when using two columns of bolts and with fewer bolts in each column. This pattern is evident both when the support is not displaced and when it is displaced by 35 mm.



Left column: Support not displaced **Right column:** Support displaced 35 mm

Figure 5.15: Stress parallel to the grain in the middle of the beam



Left column: Support not displaced **Right column:** Support displaced 35 mm

Figure 5.16: Stress perpendicular to the grain in the middle of the beam

5.4 Steel knife plate

In this section of the parametric study the results from changes made to the thickness of the steel embedded knife plate is shown. The steel knife plate is the plate inside the timber beam, where the bolts penetrate the plate and the beam connecting the two parts.

5.4.1 Moment and stiffness

The extracted moment for the connections did not increase significantly due to the increased thickness of the knife plate, see Figure 5.17. All configurations displays with similar behaviour for their moment-rotations curves. This is also evident in table 5.7 where the rotational stiffness, k, and degree of rigidity, R, remains similar for all thicknesses.

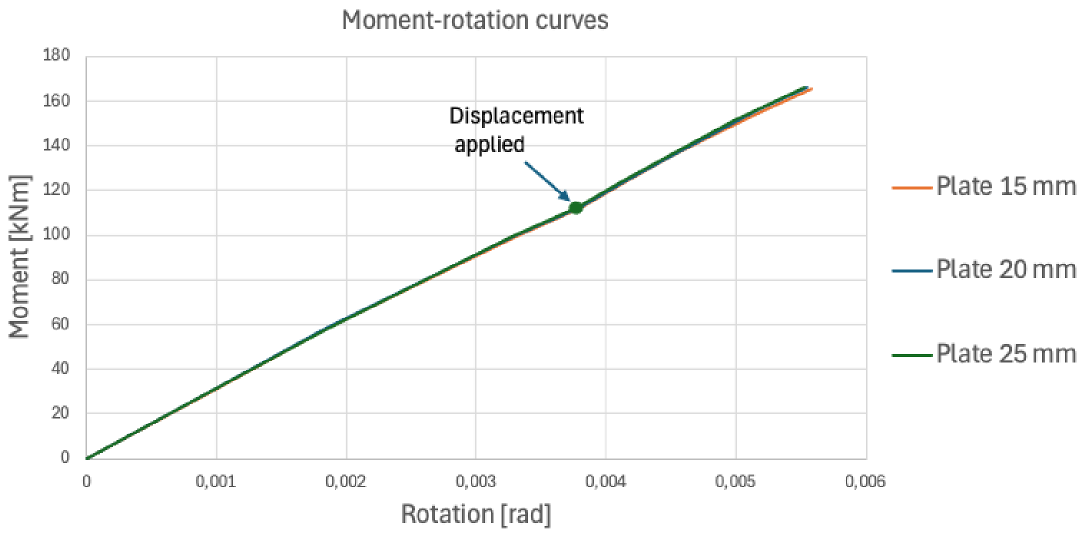


Figure 5.17: Moment-rotation curve for various knife plate thicknesses

Table 5.7: Rotational stiffness and degree of rigidity for various knife plate thicknesses

Plate thickness	**15 mm**	**20 mm**	**25 mm**
Rotation stiffness k [MNm/rad]	31.4	31.5	31.4
Degree of rigidity R [-]	0.59	0.59	0.58

5.4.2 Forces and plastic deformation in steel parts

The knife plate shows signs of plasticity in the reference model, already in the initial step, when the load is being applied, see table 5.8. These regions are, however, very small and local. The plastic regions are confined to the areas around the holes in the plate, where the bolts penetrate. Because of the bolts, these regions have very high stress concentrations. It is also clear that the bottom hole has the largest stress distribution.

This occurs because the concrete backing prevents the plate from moving back wards while still allowing it to separate. In the lower part of the plate, stresses can then be distributed over a larger surface, whereas in the upper part, they become concentrated around the screw holes. All configurations exhibit similar plastic behaviour across all components of the connection.

Table 5.8: Loading step at which the different steel components begin to plastically deform

Plate Thickness	15 mm		20 mm		25 mm	
	Applied load	Disp	Applied load	Disp	Applied load	Disp
Knife plate	90 %	-	90 %	-	90 %	-
Bolt	50 %	-	50 %	-	50 %	-
Screw	100 %	-	100 %	10 %	100 %	10 %

Increasing the steel plate thickness generally reduces the reaction forces in the bolts when the load is applied. This trend is evident for all bolt forces during the loading stage, except when comparing the top bolt with 15 mm and 20 mm plate thicknesses, see Figure 5.18. However, the increase in this case is minimal, only 0.3 kN, and is therefore considered negligible. Overall, the forces show a slight decrease as the plate thickness increases during the loading phase. This reduction is perhaps due to larger contact area between bolts and plate and thereby distribution of stresses. When the displacements is applied the bolts with a larger plate thickness seems to experience larger forces.

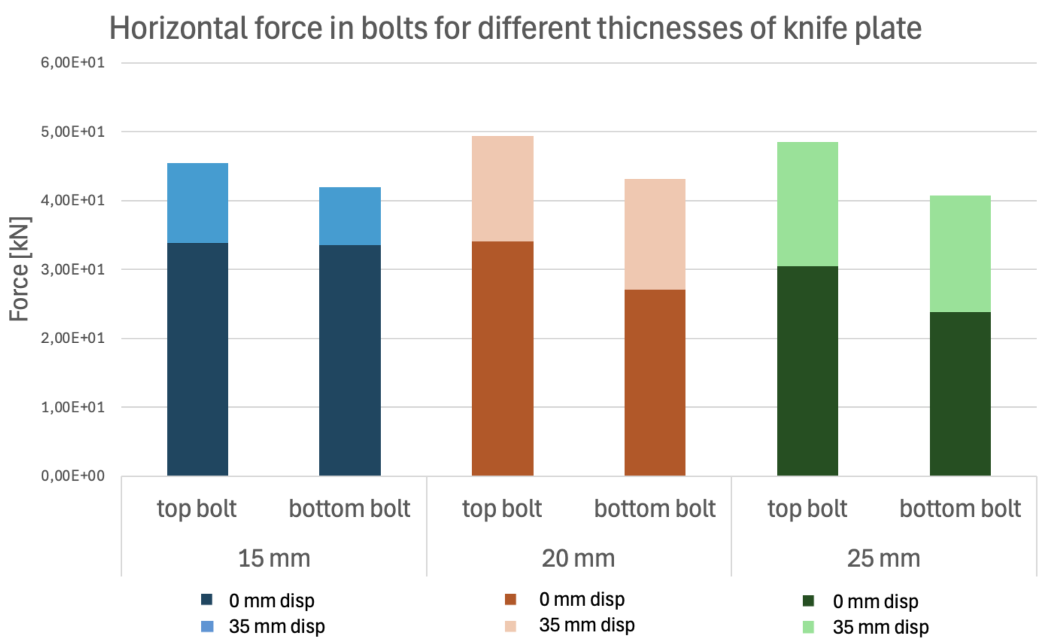


Figure 5.18: Horizontal forces in bolts with various knife plate thicknesses

When examining the forces in the screws, all configurations result in similar differences between the non displaced and fully displaced support, see forces in Figure 5.19. In this case, the screw forces do not display any consistent pattern in relation to the varying plate thickness. The differences between configurations are generally small and behave similarly, which makes it difficult to draw clear conclusions. Small variations may cancel each other out or shift in different directions without indicating a consistent trend. This suggests that the plate thickness has limited influence on the local response in the screws, at least within the tested range.

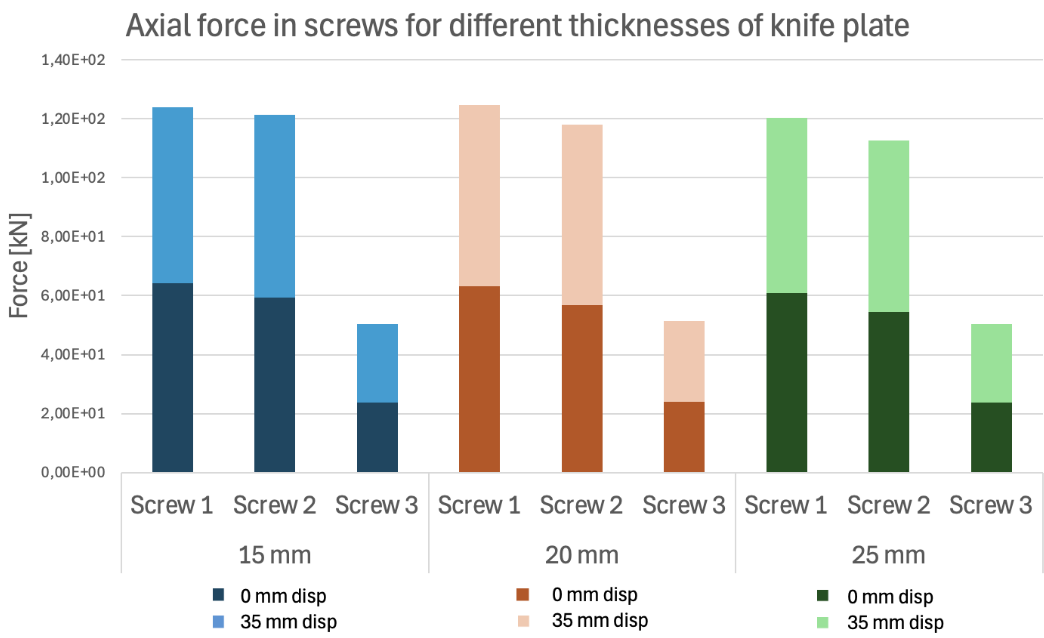
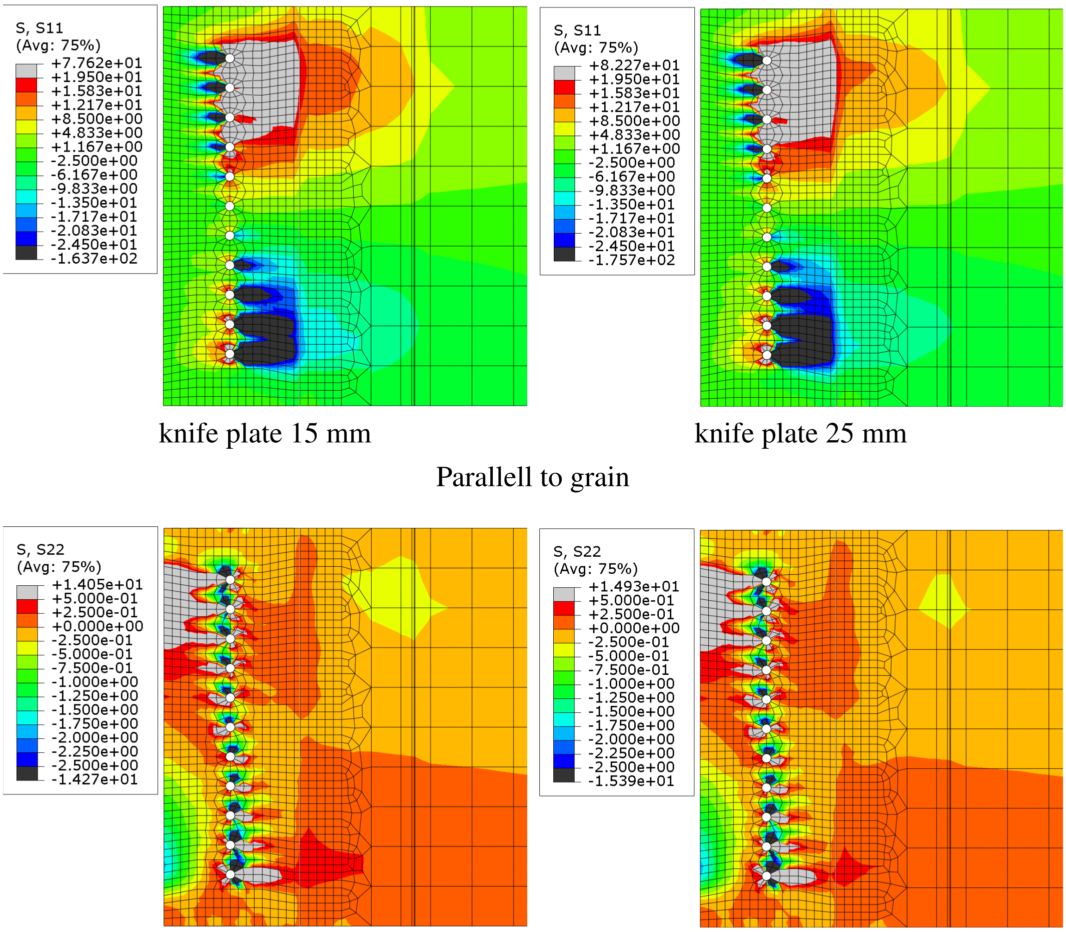


Figure 5.19: Axial forces in screws with various knife plate thicknesses

5.4.3 Stress in timber beam

The stresses in the timber beam remains similar for all thicknesses, see Figure 5.20. The increase due to applying the displacement still occur. Small increases can be observed in maximum stress parallel to grain, however very marginally. Stress increases in the beam due to increases of the plate are expected, since increasing the plate thickness also decreases the area of the timber and thereby is the area that takes the load smaller, resulting in higher stresses. This observation is in accordance with Johansen's theory since this is the type of modification that can shift failure from the plate to the timber.



knife plate 15 mm | knife plate 25 mm

Parallell to grain

15 mm | 25 mm

Perpendicular to grain

Figure 5.20: Stresses in timber beam with various knife plate thicknesses (15 25 mm), when support displaced 35 mm

5.5 Thickness of backplate

In this section, the thickness of the backplate is changed to see how it affects the behaviour of the connection. In the reference model the backplate is 20 mm thick and has not shown any signs of plastic deformation or reaching its capacity. To investigate whether the thickness plays a larger role, additional thicknesses of 15 mm, 10 mm, 8 mm, and 5 mm are tested.

5.5.1 Moment and stiffness

The study shows that the moment in the connection decreases with reduced backplate thickness. As seen in Figure 5.21, the reference model, which has the thickest backplate, also displays the highest moment relative to rotation. The changes between 15 mm to 10 mm and from 10 mm to 5 mm result in noticeable shifts in the moment-rotation curves. This indicates that the thickness of the backplate is a highly influential parameter for the global stiffness of the connection. The reference model and the modified 15 mm backplate show similar behaviour. Therefore, to simplify readability, the reference model will not be included furthermore when comparing the different modifications in this section.

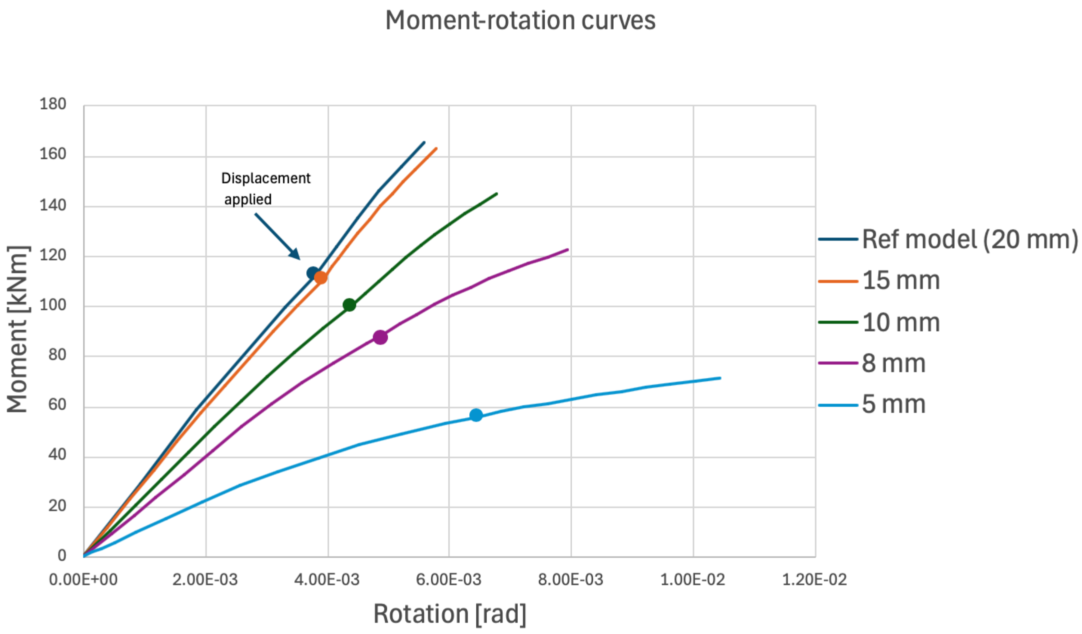


Figure 5.21: Moment-rotation curve for various backplate thicknesses

The curve for the 5 mm backplate stands out in several ways. Unlike the other configurations, the inclination of the graph decreases after the applied displacement is introduced (marked by the dot). However it still has a small upwards inclination, so some stiffness remains. This is an indication of plastic behaviour, where the connection continues to deform without attracting additional moment. Another observation is that the 5 mm configuration shows a change in stiffness even before the displacement is applied, around 35 kNm. This decrease occurs earlier than in the other models. The 8 mm plate also displays a similar behaviour, with a visible shift in curve inclination shortly before the imposed displacement is added. For the model with a thickness of 10 mm, a small shift in inclination can be detected after the displaced support has been added. However, for the model with a thickness of 15 mm, the behaviour remains linear, and no indication of global plastic deformation can be seen. The decreasing stiffness is also evident when examining the rotation stiffness and degree of rigidity. In Table 5.9 it is clear that both k and R decreases when the backplate thickness decreases.

Table 5.9: Rotational stiffness and degree of rigidity for various backplate thicknesses

Plate thickness	**15 mm**	**10 mm**	**8 mm**	**5 mm**
Rotation stiffness k [MNm/rad]	29.26	24.04	19.92	11.09
Degree of rigidity R [-]	0.57	0.52	0.47	0.33

5.5.2 Forces and plastic deformation in steel parts

The parametric study concerning the back plate thickness reveals a strong correlation between its plastic behaviour and the global response of the connection. For the thickest plate

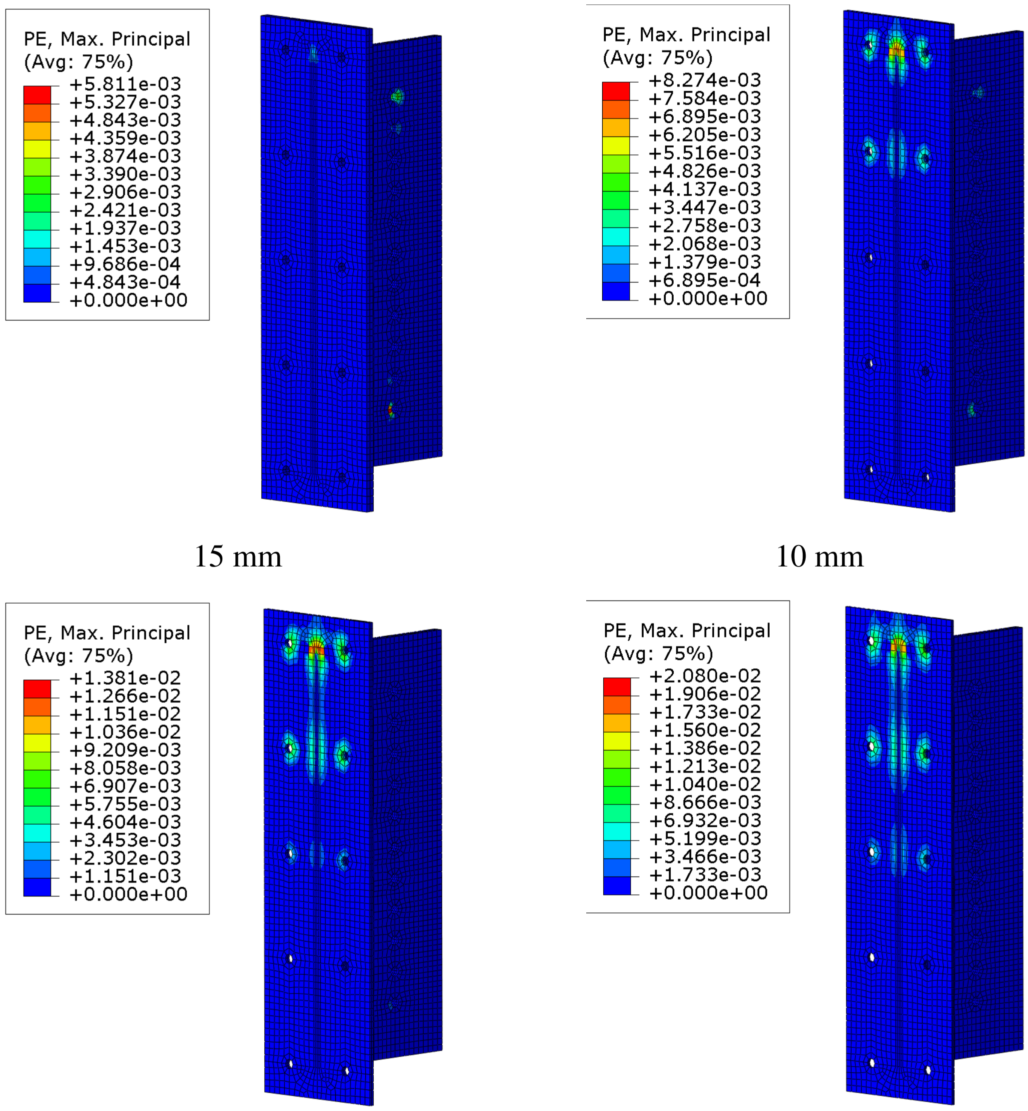
(15 mm), the back plate remains elastic throughout loading and only minor plastic deformation when the entire displacement is applied, indicating a stiff connection. In contrast, when the backplate thickness is reduced to 10 mm, the back plate begins to plasticise when 70 % of the load is applied. Table 5.10 displays the time when plastic behaviour begins for each component and configuration.

Table 5.10: Loading step at which the different steel components begin to plastically deform

Backplate Thickness	**15 mm**		**10 mm**		**8 mm**		**5 mm**	
	Applied load	Disp	Applied load	Disp	Applied load	Disp	Applied load	Disp
Backplate	100 %	70 %	70 %	-	55 %	-	40 %	-
Bolt	40 %	-	50 %	-	55 %	-	80 %	-
Screw	80 %	-	60 %	-	55 %	-	50 %	-

For the 8 mm backplate, plasticisation of the backplate initiates earlier. Notably, the timing of this plastic behaviour aligns with a distinct reduction in slope inclination in the moment-rotation curve in Figure 5.21. A similar pattern is seen for the 5 mm plate, where plasticity initiates even earlier and the timing of that also coincide with a loss of stiffness.

Across all variations, the results consistently show that the back plate dominates the early and global plastic response. This confirms that it is primarily the back plate's plastic deformation that governs the initiation of global ductile behaviour in the connection. It is also clear that the final plastic deformation increase in the backplate when the plate gets thinner which can be seen in Figure 5.22.



15 mm 10 mm

8 mm 5 mm

Support displaced 35 mm

Figure 5.22: Plastic distribution in backplate with different thicknesses (15, 10, 8 and 5 mm), when support displaced 35 mm. The blue areas do not experience any plastic strain, while the other coloured areas do

When studying the plastic behaviour of the bolts, it is clear that a thinner backplate leads to a later start of plastication and lower maximum stress in the bolts, for initiation of plastic behaviours see Table 5.10. A backplate thickness of 15 mm causes plastication of the bolts to start when about 40 % of the load is applied. The 5 mm thick backplate causes plastication to begin when 80 % is applied.

The plastication and stress trend for the screws is the opposite of that for the bolts. When the thickness of the backplate decreases, the screws start to plasticise earlier. They never reach their ultimate strain, but the maximum stresses are still very high. Due to the limitations of this study, of not modelling the concrete or considering the capacity of the steel to concrete joint, these higher stresses will not be further evaluated. However, the withdrawal capacity and anchorage in the concrete would be of high importance.

The forces in the bolts exhibit a consistent reduction when decreasing the thickness of the back plate, which corresponds to a lower stress demand on the connection, see Figure 5.23. A similar trend is observed for the screws, where the force magnitude decreases with reduced plate thickness, see Figure 5.24. However, the previous plasticisation evaluation indicated that the screws initiate yielding at earlier stages as the plate becomes thinner. The yielding is initiated earlier but the forces is largely distributed and final magnitude is not as large since the ductility from yielding attracts less stress.

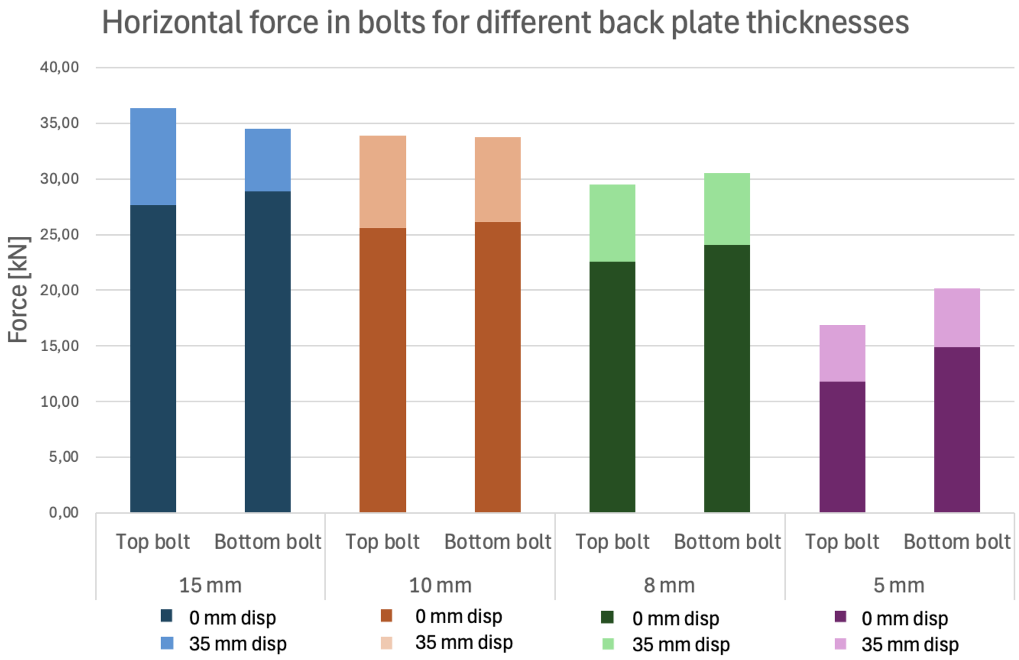


Figure 5.23: Horizontal forces in bolts, with various back plate thicknesses

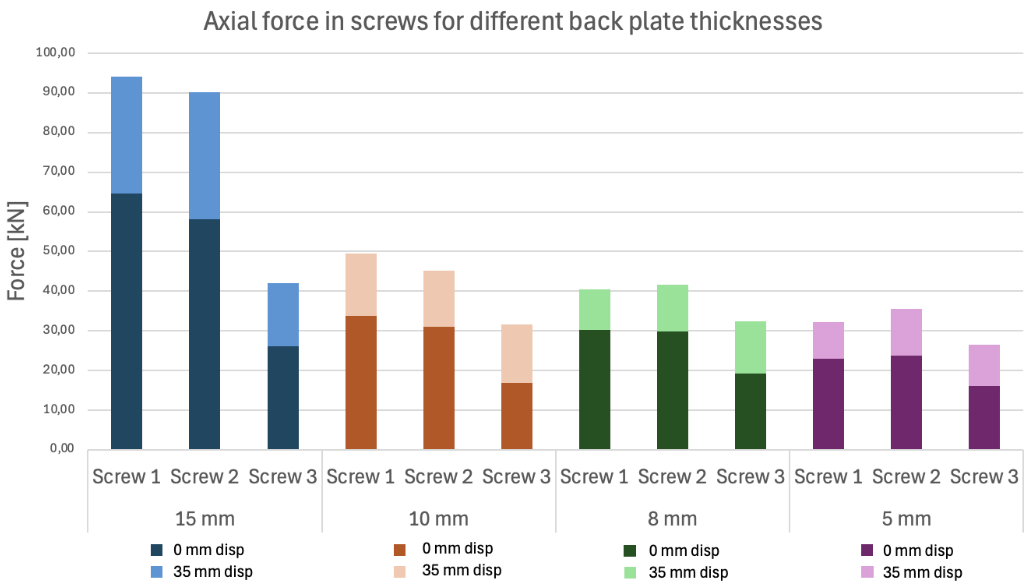
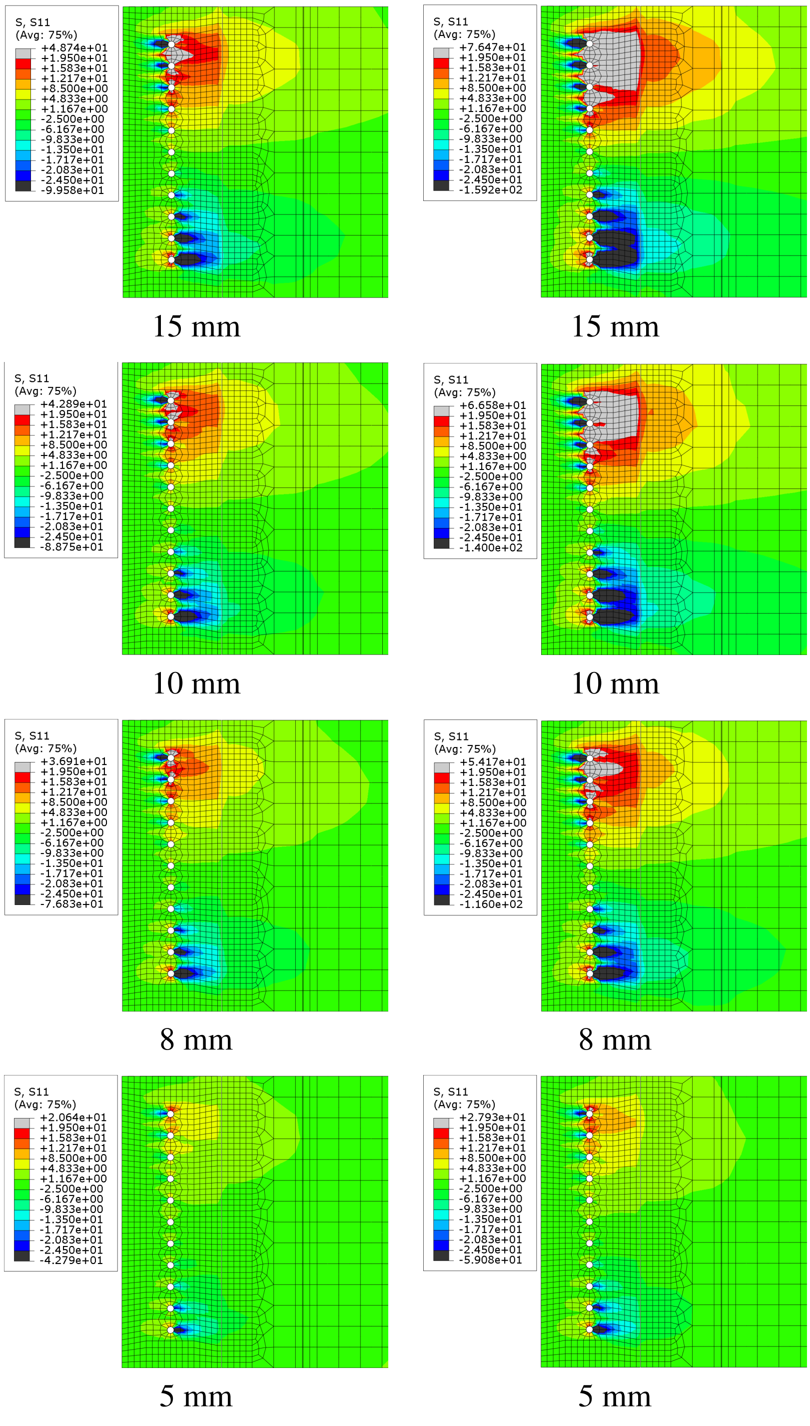


Figure 5.24: Axial forces in screws, with various back plate thicknesses.

5.5.3 Stress in timber beam

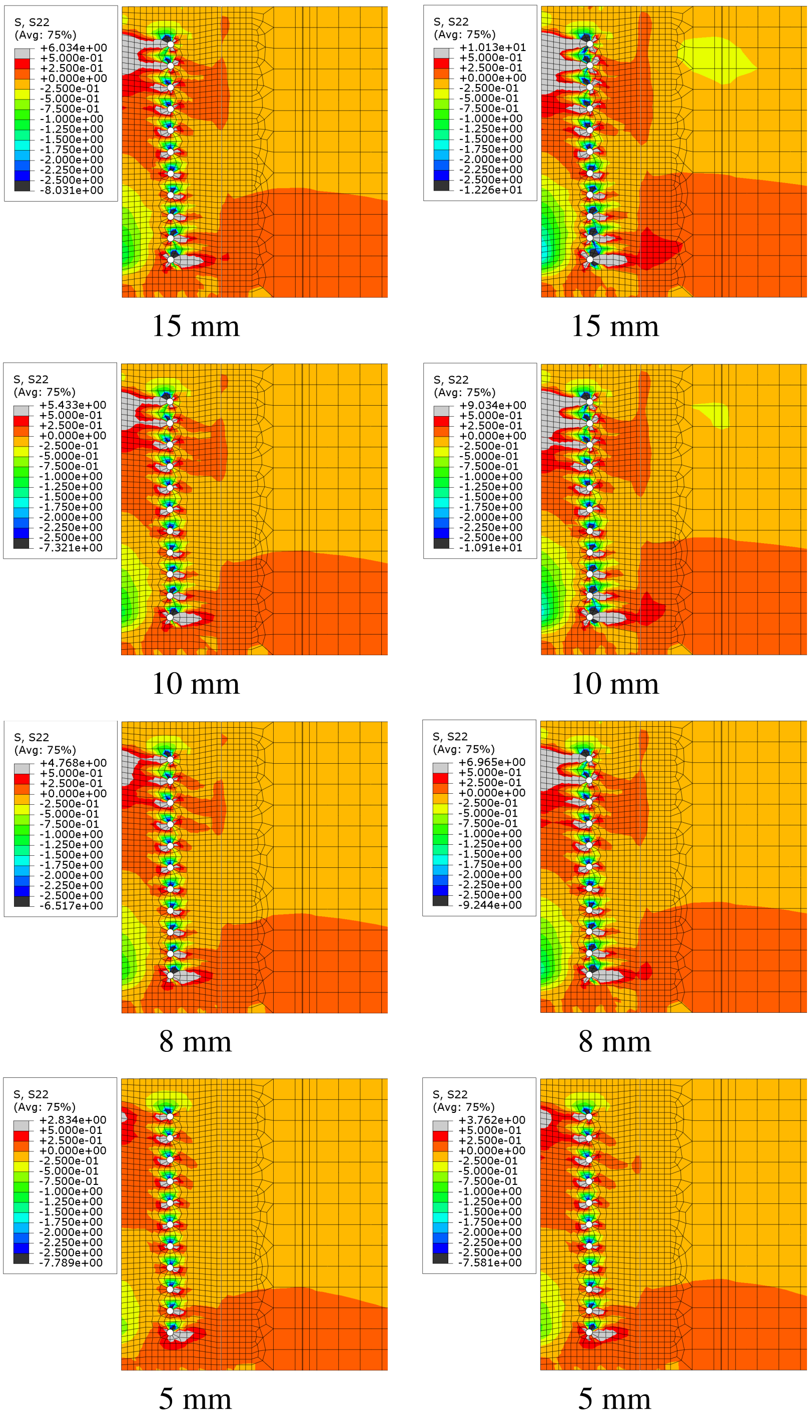
The stresses in the timber beam, especially those parallel to the grain, decrease with reduced backplate thickness. This is consistent with the observed drop in stiffness, as a more pinned connection attracts less stresses. Figure 5.25 illustrates how the critical stress areas shift and reduce, especially when comparing the final displacement stage across different thicknesses. A particularly large change is observed when decreasing the backplate from 10 mm to 8 mm. Despite the small difference in thickness, the reduction in stress is significant, perhaps suggesting a threshold effect where even minor modifications lead to a notable shifts in behaviour.



Left column: Support not displaced **Right column:** Support displaced 35 mm

Figure 5.25: Stresses in timber beam parallel to grain, with various back plate thicknesses

Stress regions perpendicular to the grain are also reduced across the thinner plates, see Figure 5.26. The most visually notable change occurs when going from 8 mm to 5 mm, however, the difference between 10 mm and 8 mm also shows clearly noticeable effects.



Left column: Support not displaced **Right column:** Support displaced 35 mm

Figure 5.26: Stresses in timber beam perpendicular to grain, with various back plate thicknesses

6

Discussion

This chapter discusses the trends and observations from the parametric study, with a focus on the effect of the displaced support and how different components are influenced. Possible strategies to accommodate these effects, as well as alternative solutions not explored in this study, are considered. The reliability of the method is also reflected on, and potential ways to apply the findings in practice are discussed.

6.1 Analysis of the Parametric Study

To gain a deeper understanding of the results from the parametric study, several topics are discussed in more detail. The focus lies on how the displaced support affects the connection across different configurations. Based on this, the most critical aspects are analysed and identified.

6.1.1 Effect of support displacement

Overall, the results clearly show that the displaced support has a significant impact on the connection. This is important to consider when designing connections in hybrid structures, where long-term deformations differ between concrete and timber. One clear consequence of the introduced displacement is an increased moment in the connection, which in turn affects the stress distribution in various components. In general, the stresses increase when the support is displaced. The largest increase in stress occur in the axial direction of the beam, as this is the primary effect of the added moment. This has the greatest impact on the stresses parallel to grain in the timber beam, and on the horizontal reaction forces in bolts and axial force in screws. The magnitude of this increase, in both moment and stress, varies depending on the different configurations studied in the parametric study.

From the moment-rotation curves the degree of rigidity, R, was calculated. For the reference model, the calculated R-value is 0.59. This indicates that the connection behaves semi-rigid and is capable of transferring moment. This differs from the common assumption in practice, where embedded steel knife plate connections are typically considered as pinned. However, the results of this study suggest that this assumption may not always be accurate, which proves to be particularity critical when long-term deformations differ. Assuming the connection to be pinned, when in reality it transfers moment, could potentially lead to an underestimation of stresses and internal forces in the surrounding structural elements. Recognizing the moment behaviour in the connection during design, could lead to more accurate and efficient solutions.

6.1.2 Influence of components on global stiffness

One interesting observation regarding the stiffness of the different bolt configurations is that the 1×11 setup shows an R-value that is very close to that of the 2×4 configuration. Notably, both configurations have bolts positioned at the same height at the top and bottom of the connection. This indicate that the bolts located at the top and bottom of the connection contribute more significantly to the overall stiffness than the total number of bolts.From the parametric study, it is evident that while the number of bolts has some influence on the connection's stiffness, their placement plays a more decisive role. To achieve a connection with lower rotational stiffness, bolts should be placed closer to the neutral axis. It could also be of interest to investigate how a configuration with fewer but larger-diameter bolts would perform under the influence of displaced supports. However, due to the limited scope and time available for this study, that aspect was not explored in further detail.

A second factor contributing to the observed rotational stiffness in the reference model is the back plate. It is likely that the plate used in the model is somewhat stiffer than what would typically be used in a real-world application, which may lead to an overestimation of the connection's rigidity. A thick back plate was chosen initially because, at that stage, the intention was to make it very stiff in order to represent the stiffness of the concrete. At the time, there was no awareness that it could have a major impact on the overall behaviour of the connection. This underlines the importance of carefully evaluating the stiffness of steel components when designing timber-to-steel connections. To reduce rotational stiffness in the connection, the back plate should be designed as thin as possible. However, it still needs to be sufficiently strong to safely transfer the shear force from the applied load without reaching failure.

In contrast, the knife plate shows a very limited effect on global behaviour. Changing the thickness of the knife plate does not significantly affect the stress in the timber or the forces in the screws and bolts. The rotational stiffness of the connection remains nearly unchanged, and there is no significant variation in the stress levels in the surrounding components. This indicates that the knife plate is not significantly affected by the displaced support and should therefore be designed according to Eurocode, based on the applied load.

6.1.3 Influence on stress distribution

When examining how different configurations effects the rigidity of the connection, it is clear that increasing the bolt diameter, adding more bolts (especially in two columns), and increasing the thickness of the back plate all contribute to a higher degree of rigidity (R-value), resulting in a stiffer connection that attracts more moment. While these changes are generally seen as improvements, leading to a stronger connection, this is not always beneficial in terms of stress distribution. A comparison of how the stiffness influences stresses in the timber beam shows that both parallel and perpendicular stresses in the timber tend to decrease as stiffness decreases. This trend becomes particularly noticeable when the thickness of the back plate is reduced or when fewer bolts are used, especially in a two-column arrangement. This is likely because a stiffer connection contracts a higher moment which it needs to resist. In turn, this leads to increased stresses the connection and the timber beam.

However, this is not universally true across all configurations. For instance, the stress parallel to the grain decreases with an increased bolt diameter. This is likely because the increased structural strength of the connection from the bigger bolt diameter outweighs the increase in moment. Moreover, with larger bolt diameters, the connection can transfer the moment more efficiently, reducing stress concentrations. This highlights the importance of finding a balance when designing the bolts. Using larger or more bolts helps distribute the forces, but it also increases the rotational stiffness, which can lead to higher moments being attracted to the connection.

6.1.4 Screws and bolt forces

When analysing how the axial forces in the screws vary across different configurations, it is evident that an increase in connection stiffness generally leads to higher forces in the screws. when the stiffness of the connection is increased the axial reaction forces in the screws to concrete gets higher. This is primarily because a stiffer connection attracts a larger moment and thereby more axial forces, which in turn requires a greater force to be transfer through the screws to the concrete element. The screws thus experience higher shear and tensile forces as they play a more critical role in resisting the increased moment. However, an exception to this trend is observed when the thickness of the knife plate is increased. In this case, the forces in the screws tend to decrease slightly. A possible explanation for this is that the overall stiffness of the connection does not change significantly with the increased plate thickness, but the thicker plate itself becomes stronger and more effective in distributing the internal forces. As a result, the load is shared more efficiently between the steel components and the screws, leading to a reduction in forces in the screws.

When examinating the other steel components in the connection, no clear relationship between stiffness and bolt forces can be observed across the configurations. For bolt arrangements 1×11, 2×11, and 2×9, can it be seen that when the rotational stiffness is reduced, the forces in the bolts decrease. However, in the 2×6 and 2×4 configurations, an opposite trend appears, bolt forces increase despite lower global stiffness. This may be due to the reduced number of bolts in these configurations. With fewer bolts distributed over the same connection area, each bolt must carry a higher part of the force.

6.2 Design strategies for improved performance

Based on the results of the parametric study, some improvements can be made with regard to failure modes and ductility. These findings, along with other suggestions, will be discussed in this section.

6.2.1 Promoting ductile failure

From a theoretical perspective, the goal is to achieve a ductile connection that follows the third failure mode according to Johansen's theory (Swedish wood, 2022a). To ensure this, it is preferable that the steel components are the ones to yield before the timber reaches its ultimate capacity. This is because steel is a ductile material that can undergo significant plastic deformation, while timber is more brittle and prone to sudden failure. When reviewing the results of the parametric study, it is evident that a reduced thickness of the back plate has the most significant effect in lowering the stress in the timber beam. This suggests that decreasing the stiffness of certain steel components can help ensure that plastic deformation occurs in the steel before failure in the timber beam. Another parameter that contributes to lower stress in the timber is the use of multiple columns of bolts. This helps to distribute the load more evenly between the top and bottom edges of the connection. However, in configurations with two columns, it may be desirable to use fewer bolts overall to avoid excessive stiffness, which could again increase stress in the timber. Finding a balance between bolt quantity and layout is therefore critical in achieving the desired failure mode.

6.2.2 Alternative design considerations

In Section 3.3, various types of connections between a concrete wall and a timber beam are presented. The embedded steel knife plate connection was selected for this study due to its good performance in fire scenarios and its common use in building construction. It is also considered visually appealing, as the steel components are concealed within the timber, which is often preferred by architects for aesthetic reasons. However, the results show that this type of embedded steel knife plate connection leads to high stress concentrations in the timber when the support is displaced. Since this behaviour is strongly influenced by the rigidity of the connection, it may be worth considering alternative design solutions. One alternative could be a beam hanger connection, see Figure 3.6. In this type of connection, the vertical loads are not only transferred through bolts but are also supported by a steel plate positioned underneath the timber beam. This could help relieve some of the stress in the timber. Additionally, this type of connection is generally assumed to behave as a pinned connection, which offers advantages when designing hybrid structures where differences in long-term deformation can lead to increased moments and stresses in more rigid connections. However, as seen in the case of the embedded knife plate connection, this assumption needs to be carefully evaluated to confirm that the beam hanger connection truly behaves as a pinned connection.

The only connection within a hybrid structure discussed in the reviewed literature is found in the Brock Commons Tallwood House. In that case, a drag strap was placed on top of the CLT panel to manage the lateral forces caused by differential movement (Connolly et al., 2018). This study similarly shows that axial forces increase when the displaced support is introduced, highlighting the importance of addressing such forces in design. Although the connection in Brock Commons Tallwood House differs, being located between a concrete core and CLT-panels, the idea of implementing a drag strap may still be relevant for an embedded steel knife plate connection, see Figure 3.4. Such strap could potentially help reduce the axial forces in the screws and stresses in the bolts and timber beam, because it will redistribute the load over a larger area. However, adding a drag strap may also increase the overall stiffness of the connection, which in turn could attract more moment and potentially worsen stress concentrations. For this reason, further investigation is necessary to determine whether the inclusion of a drag strap in this type of connection would be beneficial overall.

Another potential way to redistribute the stresses at the top and bottom bolt holes in the timber, where stresses are highest and most likely to exceed the timber's strength, is by making the bolt holes oval in the steel plate. Changing the shape of the bolt holes specifically at the top and bottom positions would allow more rotation in the connection, which would reduce its overall stiffness. This modification could also reduce the stress concentrations at those critical points. This suggests that since the top and bottom bolts don't engage immediately but require some rotation before coming into contact with the knife plate, the initial load is instead distributed to the inner bolts. As a result, this may lead to a more even stress distribution across the entire connection. Although this approach was not investigated in the current study, it may be an interesting concept to explore in future research as a way to reduce stress levels in the timber and improve overall performance.

6.3 Method reflections and limitations

A reflection on the chosen methodology has been carried out and is presented in this section. This includes simplifications made, challenges encountered during the modelling process, and limitations in the numerical approach. It also mentions what aspects of the methodology that strengthen the reliability of the results.

6.3.1 Knowledge gaps and simplifications

The literature study covered a broad scope and gave a good foundation for the work that followed. Some useful insights emerged that helped guide and support the parametric study, for example material behaviour of timber and the role of steel components to enhance ductility. However, when it came to connections in hybrid structures, available research was limited. While variations in vertical deformation had been identified, there was no clear guidance on how such effects are typically handled in a real structure. Technical details on how similar connections have been designed or implemented in past projects were difficult to access. A few requests were to contractors, but this kind of information was not shared. Although such input could have strengthened both the logic and effectiveness of the process, its absence also became a motivation and helped shape the direction of the master's thesis.

Another aspect that could have strengthened the analysis was the way certain parts of the connection were modelled, particularly the concrete wall and its interaction with the steel screws connected to it. Simulating this would have created a more realistic representation. In reality, this support would likely be more flexible than the fully fixed boundary condition used. While these simplifications may lead to a slight overestimation of force uptake, capturing the full behaviour would have required a far more advanced approach. Since the aim of this thesis was to compare relative trends across configurations, these assumptions were considered reasonable within the scope of the work and are also assumed to be on the safe side.

6.3.2 Modelling limitations and reliability

The modelling approach included some simplifications, such as excluding failure modes and capacity checks. Timber was modelled with linear behaviour, while steel was assigned plastic behaviour, making it the only non-linear component. A more advanced model could have included additional non-linearities and captured the post-critical behaviour of timber, which is typically brittle rather than ductile. However, the aim of this study was to explore how long-term deformations affect the connection and its components. In this context, excluding failure allowed the analysis to continue beyond the point of local failure, enabling a clearer view of relative trends between configurations. The convergence study also revealed local singularities, particularly in stress concentrations, which made absolute values unreliable. Therefore, the analysis focused on stress distribution patterns and general plastic behaviour, rather than exact numerical outputs.

Despite these limitations, the results are still considered meaningful within the scope of the study. The modelling strategy enabled a consistent comparison of different configurations, and the structured parametric setup helped maintain clarity throughout the process. While the model does not aim to predict absolute capacities, it provided valuable insight into how different parameters influence the behaviour of the connection. Most simulations were successfully completed, and the results were interpreted in a critical and analytical way, which gives the study a solid basis for its conclusions.

6.4 Implementation of results

While the results are not intended to be used for design values, the patterns identified between different configurations could offer helpful guidance when choosing connection types in early design phases. For example, the observed sensitivity of stresses parallel to grain, due to differences in vertical deformation, could be a reminder to avoid overly stiff connections when attaching timber beams to concrete cores. And it might perhaps raise the thought that stresses perpendicular to grain are not the only thing worth paying attention to. It does not mean the findings should dictate design choices, but they might help start a conversation around what matters in these types of connections. Today, there are no clear regulations or guidelines for how to handle this type of deformation difference, but the patterns found here could help raise that question, especially since the differences observed were quite significant. While further work would definitely be needed to confirm the effects in more detail, this study could still serve as a useful starting point for that discussion.

One suggestion when designing connections in hybrid structures, where differences in deformation can occur, is to first follow Eurocode recommendations to base the design on the vertical load. After that, the connection's rotational stiffness should be determined, and the potential increase in moment because of this stiffness due to the long-term deformations should be taken into account in the final design.

7

Conclusion

The main objective of this Master's thesis was to investigate how differences in long-term deformations between timber and concrete affects the connection between them. The results show that when a stiff connection is used, the displacement can lead to increased moment and stress in the connection. This is especially noticeable in the axial direction.

A related objective was to examine which components within the connection are most affected by this deformation difference. The results clearly indicate that the components that resist forces in the axial direction of the beam are the most impacted, particularly if they lack sufficient stiffness in that direction. For example, the knife plate is very stiff in this direction due to its geometry, and is therefore less affected. In contrast, the bolts and screws experience significant increases in force. A similar trend is seen in the timber beam, where stresses parallel to the grain, i.e. in the axial direction, increase considerably. This makes it a critical factor to consider when designing these types of hybrid connections.

Although theoretically, the embedded steel knife plate connection is often assumed to behave as a pinned support, the results from this study show that it does not act as a purely pinned connection in practice. Instead, its high stiffness attracts load and stress to the connection, which increases the impact from long-term deformations. For this reason, the actual behaviour of the connection should be carefully considered during design, rather than relying on simplified assumptions.

Another objective was to see if it is possible to design a connection that accommodates these differences in an efficient way, and there is. There are a few ways to improve how a connection handles these effects. One is to promote ductility in the steel components so that yielding occurs in the steel parts rather than causing brittle failure in the timber. For instance, reducing the thickness of the back plate led to earlier yielding in this study. Another is to distribute forces more evenly across the timber by using two columns of bolts, which lowered local stress concentrations even when overall stiffness remained fairly constant. Results also showed that the top and bottom bolts have the largest impact on the generated stiffness, suggesting that placing them closer to the neutral axis of the beam can help minimise moment build-up.

Although the study focused on a specific type of embedded connection, the findings suggest that it is indeed possible to design more efficient connections that better accommodate the differences between timber and concrete. For example, with a thinner back plate. Exploring alternative connection strategies could help reduce stress concentrations and improve overall performance. Future research should explore these and other alternatives in greater detail. to ensure safe and resilient hybrid structures.

7.1 Further studies

One potential direction for further studies is to explore how the difference in long-term deformation influence the concrete-to-steel interface. This study showed that the axial forces in the screws connected to the concrete increased significantly due to the differences in long-term deformation. One aspect that remains unclear is how critical this effect is in terms of its withdrawal capacity. Could the increased axial loads compromise the withdrawal capacity of the anchorage in the concrete? Further investigation is needed to determine whether additional reinforcement or alternative anchorage solutions are required to ensure structural safety over time.

This study has shown that differences in long-term deformation have an impact on the connection, and several suggestions have been proposed to help minimize this issue. However, no definitive design recommendation could be made, as further research is needed, both on alternative configurations and different types of connections, to enable more conclusive results. Future studies should also include more detailed finite element modelling and be complemented with laboratory tests to improve the reliability of the findings.

This study focused on the effects of differential long-term deformation between the concrete core and the surrounding timber structure, particularly how the displacement influences the connection. However, an interesting direction for further research would be to investigate whether the issue could be addressed already at the design stage, by 'building it away', meaning structurally compensating for the expected deformation. For example, could the timber columns initially be built slightly taller to account for future displacements? Then, the relative displacement to the concrete core would be minimized over time. Another idea worth exploring is the use of adjustable or demountable connection systems that that can be realigned or adapted after construction, either by lowering the connection to the core, or raising the support connected to the timber column. These kinds of strategies might offer a practical way to reduce long-term effects without relying solely on ductility in the timber-to-concrete connection.

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Appendix A – dimensioning of beam

Preliminary sizing timber beam

```
clear all
clc
% Beam geometry
h = 0.675;          % [m]
b = 0.230;          % [m]
L = 7.3;            % [m]

I = b*h^3/12;       % Moment of inertia [m^4]

% Slab (L240-7s)
t_s = 0.240;        % [m]

L_s1 = 6.5;         % [m]
L_s2 = 7;           % [m] L_s2 = 20-2*6.5

% Material properties
% Glulam: GL30c
E_0mean = 1.3e10;   % Young's modulus parallel to the fibers[Pa]
E_90mean = 3e8;     % Young's modulus perpendicular to the fibers[Pa]
G_mean = 650e6;     % Shear modulus [Pa]
rho_k = 390;        % Density [kg/m^3]
f_mk = 30e6;        % Bending paralle to the fibers [Pa]
f_vk = 3.5e6;       % shear capacity [Pa]
k_mod = 0.8;        % Cervice class 1 and load duration class M
gamma_m = 1.25;
f_md = k_mod*f_mk/gamma_m;
f_vd= k_mod*f_vk/gamma_m;

% Slab L240-7s C24
rho_kC24 = 350;     % Density[kg/m^3]

g = 9.82;

% -----------------------------------------------------------------
% Load
% Permanent load
G_beam = b*h*rho_k*g;                       % Self weight beam [N/m]
G_slab = (L_s1/2 + L_s2/2)*t_s*rho_kC24*g;  % Self weight slab [N/m]
g_inst = 1000;                              % Self weight installations and inner walls [N/m^2]
G_inst = g_inst*(L_s1/2 + L_s2/2);          % [N/m]

% Variable load
q_office = 3000;                            % [N/m^2]
Q_office = q_office * (L_s1/2 + L_s2/2);    % [N/m]

% Load coefficients
psi_0 = 0.7;
psi_1 = 0.5;
psi_2 = 0.3;

% Partial safety factor
gamma_g_ULS = 1.35;
gamma_q_ULS = 1.5;
gamma_g_SLS = 1.0;
gamma_q_SLS = 1.0;

% SLS
Q_d_SLS = (G_beam + G_slab + G_inst) * gamma_g_SLS + Q_office*gamma_q_SLS; % [N/m]
Q_d_SLS_on_beam = (G_slab + G_inst) * gamma_g_SLS + Q_office*gamma_q_SLS;  % [N/m]
G_SLS = (G_beam + G_slab + G_inst) * gamma_g_SLS;       % [N/m]
Q_SLS = Q_office*gamma_q_SLS;                           % [N/m]
```

```matlab
% ULS
Q_d_ULS = (G_beam + G_slab + G_inst) * gamma_g_ULS + Q_office*gamma_q_ULS; % [N/m]

% Aplied pressure on the beam in Abaqus
sigma_load_abaqus = Q_d_SLS/b/10^6;        % [MPa]

% -----------------------------------------------------------------
% Deformations
k_def = 0.6;                                % Service class 1

% Instant deformation from the total applied load
w_inst = 5*Q_d_SLS*L^4/(384*E_0mean*I);    % Instant deformation (Elementarfall) [m]


% Effect from shear
beta = 1.2;
w_shear = beta*Q_d_SLS*L^2/(8*G_mean*b*h); % From Allmänna grunder Handboken Bygg page 363
w_tot = w_inst+w_shear;

% Instant defromation from permanent and variable load
w_inst_G = 5*G_SLS*L^4/(384*E_0mean*I); % Instant deformation from permanent load (Elementarfall) [m]
w_inst_Q = 5*Q_SLS*L^4/(384*E_0mean*I); % Instant deformation from variable load (Elementarfall) [m]

% Long-term deformation
w_fin_G = w_inst_G*(1+k_def);             % Long-term defromation for permanent load [m]
w_fin_Q = w_inst_Q*(1+psi_2*k_def);       % Long-term defromation for variable load [m]
w_fin = w_fin_G+w_fin_Q;

ratio_w = w_fin/(L/300)

if ratio_w < 1
    disp('Dimension ok, with regard to deformation')
else
    disp('Dimension NOT ok, with regard to deformation')
end

% ---------------------------------------------------------------
% Bending moment
W_y = b*h^2/6;                                % [m^3]
M_Rd = f_md * W_y;                            % [Nm]

M_Ed = Q_d_ULS * L^2 / 8;                     % simply supported [Nm]

ratio_M = M_Ed / M_Rd

if ratio_M < 1
    disp('Dimension ok, with regard to bending moment')
else
    disp('Dimension NOT ok, with regard to bending moment')
end


% ---------------------------------------------------------------
% Shear capacity
k_cr = min(3/(f_vk/10^6),1);
b_ef = k_cr*b;
V_Rd = 2 * f_vd * b_ef*h / 3;                 % [N]

V_Ed = Q_d_ULS*L/2;                           % [N]

ratio_V = V_Ed / V_Rd

if ratio_V < 1
    disp('Dimension ok, with regard to shear')
else
    disp('Dimension NOT ok, with regard to shear')
end
```

```
ratio_w =

    0.8835

Dimension ok, with regard to deformation

ratio_M =

    0.9496

Dimension ok, with regard to bending moment

ratio_V =

    0.8781

Dimension ok, with regard to shear
```

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Appendix B – Connection Design

Connection Design. Embedded knife plate.

```
clc
clear all
% dimensions
b=230e-3;                          % [m]  Dimension of beam attained from Preliminary sizing
h=675e-3;                          % [m]
d=16;                              % [mm] dowel diameters, iteratively changed


% Loads
Vrd=294;         % [kN] shear capacity of beam
Ved=149.85;      % [kN] Load applied on beam

% Material properties, GLC30, dowels service class 1
kmod=0.8;
gamma_m=1.25;
dens_glc30=390;                             % [kg/m^3]
fu=355;                                     % yeilding strength dowel
fuk=fu*0.95;                                % [Mpa] dowel strength

Myk=0.3*fuk*d^2.6;                          % [Nm]
fh0k=0.082*(1-0.01*d)*dens_glc30;           % [Mpa]
fh90k=fh0k/(1.35+0.015*d);

% required timber thickness
ts=15;                                      % thickness of steel embedded knife plate [mm]
t1_min=sqrt(2)*sqrt(Myk/(fh90k*d));
t1req_new=1.15*4*sqrt(Myk/(fh90k*d))-0.5*ts;    % limträhandboken
t1=(b*1000-ts)/2;                           % remaining timber thickness [mm]

if t1 > t1req_new
    disp('required timber thickness is sufficient')
else disp('required timber thickness is NOT sufficient')
end


% failure modes Johansen's theory (8-11 EC5)
f1=fh90k*t1*d;
f2=fh90k*t1*d*(sqrt(2+4*Myk/(fh90k*d*t1^2))-1);
f3=2.3*sqrt(Myk*fh90k*d);
Fail=[f1 f2 f3];
Fvrk=min(Fail);                         % [N] dimensioning failure load one dowel


% Required number om fasterners

% Distances dowels. (EC5 8.5)
alfa=0;
a1=(3+2*cos(alfa))*d;                   % distance between rows parallell to grain
a2=3*d;                                 % distance between rows perppendicular to grain
a4_t=max((2+2*sin(alfa)*d),3*d);
a4_c=3*d;

% antal dowels i rederna
antal_ca=Ved/(Fvrk/1e3);

n_perp_allowed=(h*1000-(a4_c+a4_t))/a2;     % how many fit
n_perp=11;                              % rows perp tp grain, choose iteratively
n_parr=1;                               % rows parallell to grain, choose iteratively if perp not sufficient

if n_parr>1
    n_parr_ef=min(n_parr,n_parr^0.9*(a1/(13*d))^(1/4));
else
    n_parr_ef=n_parr;
end

Ftot_vrk=n_perp*n_parr_ef*Fvrk/1000;            % [kN]

ratio=Ved/Ftot_vrk
```

```
if ratio < 1
    disp('connection ok with regard to shear load')
else disp('connection NOT ok with regard to shear load')
end
```

```
required timber thickness is sufficient

ratio =

    0.9873

connection ok with regard to shear load
```

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Appendix C – Mesh Analysis

To ensure appropriate mesh size in ABAQUS, a mesh sensitivity analysis was carried out for the different components. Figures C.1 and C.2 show the global mesh analysis of the timber beam and the steel connector, respectively. Based on these results, a mesh size of 50 mm was selected for the timber beam and 10 mm for the steel connector. These sizes were chosen as the analysis showed that the results had converged at these mesh densities.

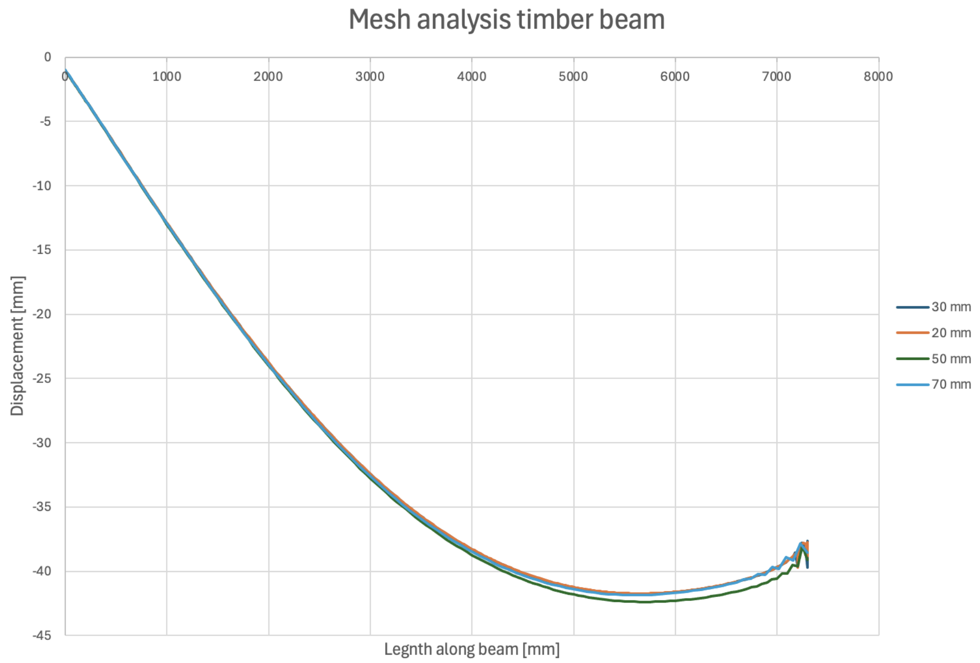

Figure C.1: Global mesh analysis of timber beam

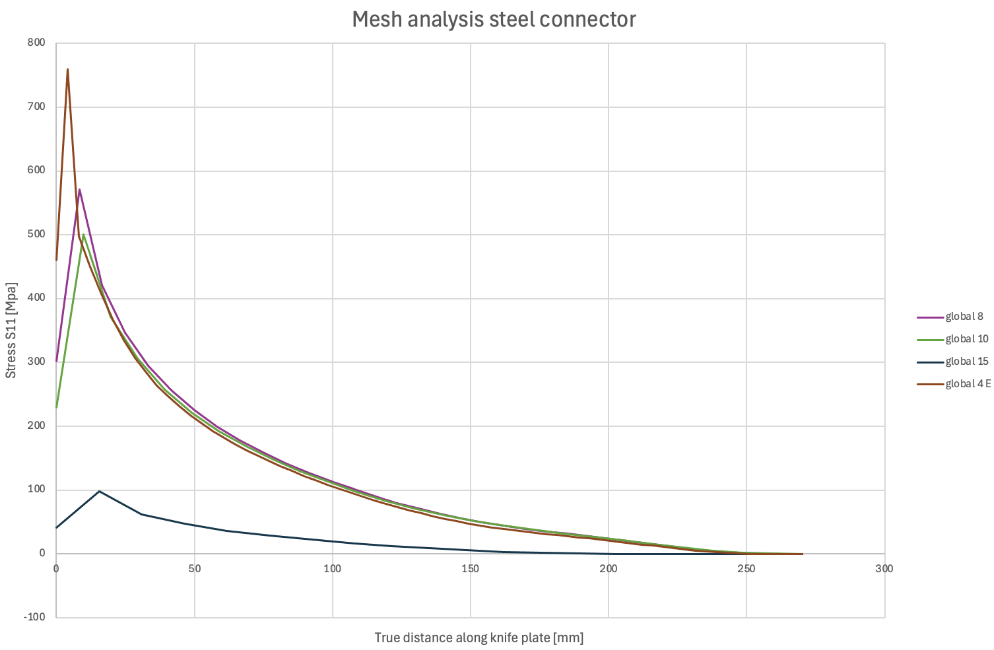

Figure C.2: Global mesh analysis of steel connector

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