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Preface

This bachelor's thesis marks the end of a 3-year bachelor's program in construction engineering at the University of Stavanger.

We would like to express our gratitude to Professor Sudath C. Siriwardane and UiS for suggesting such an interesting topic, CLT-concrete composite, which not only captured our interest but also gave us a big learning curve throughout our research and studies. Thanks to both Sudath C. Siriwardane and Fredrik Bjørheim for their guidance and input throughout the entire semester.

We also want to thank Samdar Kakay and Andreas Skaare for providing us with the material and assistance we needed for the practical part, which is a large aspect of this thesis. Thanks to Gaute Hagtvedt, a master's student, for his help with the formwork and casting of concrete. We also want to thank Maria Bergfjord, a former master’s student who had similar problem statement and paved the way for our research. The calculations and insights presented were influential in carrying out the challenging aspects of our own study.

Abstract

This thesis investigates the structural response of timber-concrete composite (TCC) slabs, with a specific focus on the influence of shear fasteners. The primary objective is to investigate the load capacity of TCC with shear fasteners that are of different orientations and spacings.

The connection plays an important role in the degree of efficiency of the composite structure and the function of a corresponding floor structure. Shear connection refers to all the possible methods to connect timber and concrete and has a critical impact on the function of TCC structure. Shear fasteners are the most common connection system. It is a type of metal connector in the form of screws, bolts and nails.

The TCC investigated in this thesis is CLT-concrete slabs connected with CTC screws. CTC is a connector for timber-concrete floor which are self-drilling and easy to install. Several research articles have highlighted that the use of screws oriented at 45° angle results in higher stiffness and load capacity values. Similarly, the feedback from applying CTC screws at a 45° inclination supports this.

This thesis considered screws installed at a single 90° and 45° crossed orientation. The challenge is that the research on screws with 90° is limited. Therefore, two types of orientations, along with different spacings are investigated in this thesis. 15 specimens are used with 5 different groups, and each group has 3 identical specimens. 3 groups with 90° and 2 groups with 45°. One group from both 45° and 90° had the same number of screws to see the comparison between them. With the main goal being the reduction of time and effort in the installation of the system using 90°. Also, to see if the manufacturing process could be automated.

The theoretical calculation predictions are performed by using the γ-method and shear analogy method to find the maximum load applied and maximum deflection both for the short-term and long-term. All theoretical predictions were performed before the laboratory tests and the values from the maximum load applied were used to perform the four-point bending test. The long-term maximum load applied was the value used for the test setup.

The laboratory test was then compared to the theoretical predictions to discuss the results. In general, the specimens with screws installed at 45° inclination had a higher load capacity. They also had a capacity that was higher than the short-term predictions and the same as the

long-term predictions. 90° on the other hand underperformed and had results that could be compared to the short-term predictions. One of the groups for 90° screws showed results for the maximum load applied that were close to 45°. In the lateral displacement 90° showed that the slip between the concrete and timber was much greater.

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List of abbreviations

EWP	Engineered wood product
CLT	Cross laminated timber
GLT	Glued laminated timber
LVL	Laminated Veneer Lumber
TCC	Timber concrete composite
HBV	Holz-Beton-Verbund
SFRC	Steel fiber reinforced concrete
SCC	Self compacting concrete
ULS	Ultimate Limit State
SLS	Serviceability Limit State
CTC	Connector for timber concrete
LVTD	Linear Variable Differential Transformer

1. Introduction

1.1 Background

Cross laminated Timber (CLT) is an increasingly popular construction material with good strength properties. CLT is different from other timber products, because of the ability to perform structurally like concrete, but with lower self-weight [1, p.9]. The material is applicable for high rise buildings, office, and residential buildings. It is primarily used for walls and floor structures.

Mjøstårnet in Brumunddal, Norway is one of the tallest timber buildings in the world. In an article on Bloomberg, a respectable architect, Jørgen Tycho from OsloTre talks about how Norway utilizes this construction method:

"*There have never ever been as many trees in our country as right now. But we don't have the industry to process them, so what we do is cut down the trees, the raw materials, and ship them to Europe.*" These woods are manufactured into GLT and CLT and shipped back to Norway [4]. Today we have Norwegian companies specializing in the production and supply of engineered timber products in Norway. Splitkon has one of the biggest CLT factories in the world, and they provided the CLT panels used in this thesis.

In recent decades, there has been a growing emphasis on being more environmentally friendly. The use of reinforced concrete and steel in construction has a significant environmental impact, with high energy consumption and CO_2 emissions. As a result, there has been an increase in research around alternatives that can partially or fully replace concrete and steel.

Therefore, the combination of timber and concrete has become more attractive. The purpose of such a composite is to combine the properties of the materials to create an improved construction.

1.2 Problem statement

TCC floors and walls are gaining interest recently in building construction. Research in this area is very limited, and researchers often rely on studies from the same sources. In the current version of Eurocode 5 (Design of Timber Structures), Timber Concrete Composite

(TCC) system is not included. The theoretical predictions are made by using the γ-method, which is applicable for a 3 layered element, by reason of this the shear analogy method is added to make better predictions of the system.

This study focuses on comparing two different orientations of shear fasteners in the assembly of CLT-concrete slabs. The widely researched and tested orientation is screws installed at 45°, whereas the experimental aspect of this thesis is to compare 45° with the less explored 90°. Surprisingly, there is a shortage of available studies on the 90° even though installation is by far easier and could even be automated. With the lack of research and studies about this 90°, the theoretical predictions are even more uncertain, and the test had a purely experimental approach.

1.3 Objective

The main goal of this thesis is investigating the load capacity and analyzing the displacement behavior of timber-concrete composite. In addition to comparing the results with the theoretical predictions. Two ways of orienting the shear fasteners with different spacings are used.

Additionally, to the theoretical part, this thesis is based on a practical part where we learned how to prepare and conduct a test.

1.4 Outline of thesis

Chapter 2 is a literature review. The goal was to establish an understanding of timber and concrete as structural materials before diving into the main topic, TCC and their connections systems.

Chapter 3 focuses on the materials used in this thesis and the design considerations. It reviews the properties of timber and concrete components, as well as the design aspects of the screws.

Chapter 4 is about theoretical approaches that are used for the purpose of calculation and verification of the TCC system

Chapter 5 builds on Chapter 4 where all the theoretical approaches are being used to calculate the maximum load applied and verification of the system.

Chapters 6 and 7 describe and visualize the specimen preparation, equipment, and test setup.

Chapters 8 present the obtained results from the tests both graphically and with the help of tables.

Chapter 9 discusses the limitations and comparison of the results, as well as conducting comparisons with the theoretical predictions made in earlier chapters.

Chapter 10 is the conclusion chapter of this thesis.

2. Literature review

2.1 Structural timber materials

2.1.1 Timber

Timber as a building material is made of large pieces of wood and can be used as a structural component in buildings. This can be classified as construction timber (konstruksjonsvirke) or engineered wood products (EWP) depending on the use. Construction timber is typically used in smaller structures, while EWPs are used for more advanced and complex structures. For timber to be a good alternative for traditional building materials like concrete and steel, EWPs play an important role in expanding its range of applications [2]. EWPs are made from sawn timber boards, veneers, particles, or wood fibers bonded together using adhesives, heat, and pressure to create a high-strength material [3, p.47].

EWPs are grouped into two categories based on their materials [3, p.47]:

- Engineered wood products based on sawn timber boards:
    - Glue laminated timber (GLT)
    - Cross laminated timber (CLT)
- Engineered wood products based on veneers.
    - Laminated veneer lumber (LVL)
    - Plywood

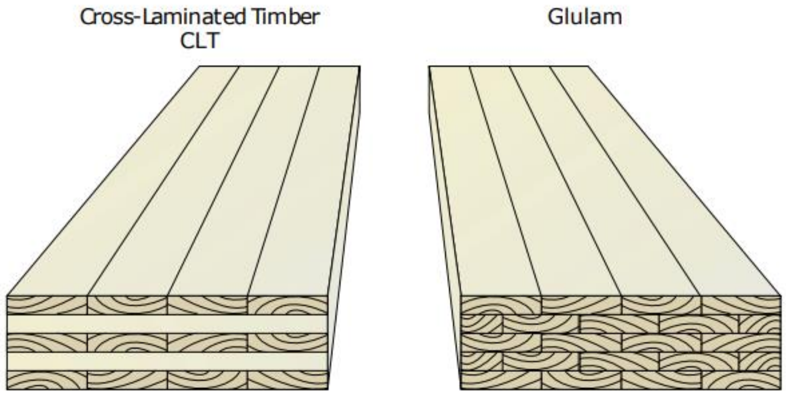

Figure 2.1 Most common EWPs [4, p.6]

It would be beneficial to point out GLT, LVL and plywood before diving into the details of CLT. This will provide context for understanding the differences and similarities between these products, and how it can be combined with concrete to form a Timber concrete composite (TCC).

GLT (limtre) is one of the oldest EWPs and consists of layers of finger-jointed sawn boards that are glued together. The most common GLT product in Norway is structural straight beams. LVL (parallellfiner) is a product built up of at least 5 veneer layers. It forms a larger structural panel where the orientation of fiber direction and layers are in the same direction. GLT and LVL have similar applications in terms of use in beams, columns, roofing, and other big structures. Both are dimensionally more stable than construction timber or solid timber, which means they are less prone to shrinkage and twisting, than natural timber like construction timber.

Plywood (kryssfiner) is one of the first engineered wood products to be produced. As with LVL, plywood is also made of veneers that are glued together to form a structural element. The veneers are arranged in a symmetrical pattern so that the fiber direction is the same for both outer layers. It looks very much like CLT, but layers are made of wood veneer that are glued together to create a flat panel. In this project, plywood is used as formwork to hold the concrete in place while it cures.

2.1.2 Cross laminated timber

CLT (Krysslaminert tre) is widely used in Europe and was first developed in Austria and Germany [2]. This panelized engineered wood product is created by stacking the sawn timber boards crosswise at 90-degree angles to each other and applying glue between each layer to bond them together. The layers are arranged longitudinally in the fiber direction. CLT panels typically consist of an odd number of layers, with a minimum of three and a maximum of nine layers. The reason for an odd number of layers is to ensure that the wood fibers run in different directions, which helps to increase strength and stability. The thickness of lamella, number of layers, and other dimensions are customized to each individual construction project. When supplying CLT, the width of the elements can vary between 60 mm and 300 mm, and even more for special needs. The thickness of the lamella can range from 10 mm to 50 mm. The length is up to 16 m, but is limited by transport capabilities [5].

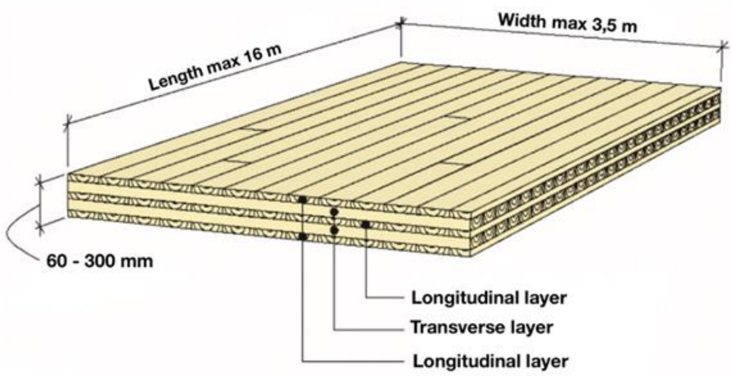

Figure 2.2 A typical CLT -layup [5]

- Advantages of CLT:
    - Renewable wood resources.
    - Good strength and stiffness properties.
    - Good dimensional stability.
    - Great flexibility in design and construction. Easy to combine with other materials.
    - Good load-bearing capacity in fire.
    - The prefabrication and simplicity of CLT make the building process save time and labor.
    - Low weight compared to concrete.
    - Good indoor environment.

[6] [1, p.8]

An example of the simplified construction process is shown in the Swedish CLT handbook, which describes how the integration of door and window lintels into CLT panels during the manufacturing process can eliminate the need for extra support when openings are cut into them [1, p.26]. This is because the CLT lintel can have high enough load capacity to support the weight of the structure above.

Shear fasteners are a fixing option when designing joints between CLT and other materials, but also in buildings. The most common type of fastener for CLT is self-drilling wood screws with diameters from 4 mm to 13 mm and lengths up to 1 m. It is important to consider the thickness and density of the panel, as well as the type and diameter of the fastener used. In general, larger diameter fasteners are required for thicker CLT panels, and longer fasteners are required for connections to other building structures. In the Swedish handbook for CLT self-drilling screws are used to keep building elements together [1]. Different types of shear fasteners for CLT are shown in figure 2.3.

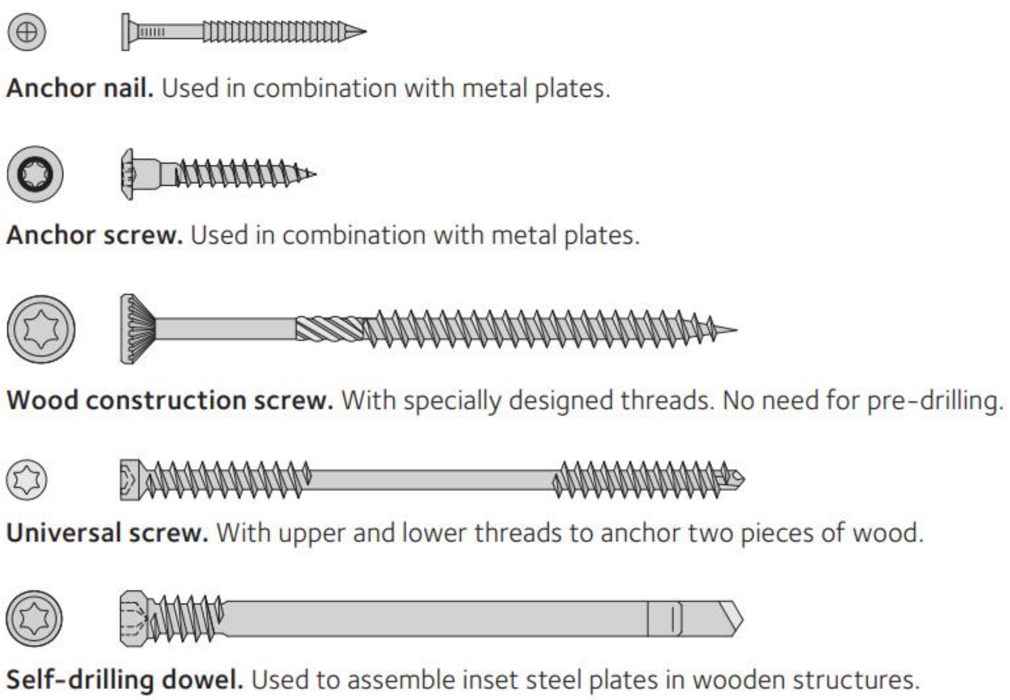

Figure 2.3 CLT shear fasteners [1, p.74]

One of the main challenges is that CLT is a relatively new product with limited information available. The lack of standardized requirements and approved calculation rules for the product can make it hard to ensure that it is being used effectively. For now, the challenges of standards and connections in CLT can often be addressed by the supplier of the CLT panels or the shear fasteners. CLT is mostly used as panels elements in walls, floors, roofs, and as beams. It can also be used in small structures and high-rise buildings. While there are some examples of tall CLT buildings, such as the 18-story Mjøstårnet in Norway, the use of CLT in high-rise buildings is still new and requires careful consideration of structural requirements.

2.2 Concrete

2.2.1 Concrete and its classifications:

Concrete is one of the most important inventions in the construction world. It’s a composite material, and consists of several components like coarse gravel, sand, cement, and water. Admixtures can also be added to ensure that the concrete has its desired property. In Norway concrete suppliers must ensure that the concrete they provide meets the required standard specified by the NS-E 206 [7].

- Concrete has different classification based on density [7]:
    - Lightweight concrete: Concrete with density less than 2000 kg/m^2
    - Normal weight concrete: Concrete with density greater than 2000 kg/m^2 but not exceeding 2600 kg/m^2.
    - Heavyweight concrete: Concrete with density larger than 2600 kg/m^2.

The exposure class of concrete refers to the level and type of environmental exposure that the concrete structure will be subjected to, and this should be determined in each individual case. The durability class, on the other hand defines the concrete's resistance over time to these impacts and sets requirements for concrete mix design, choice of materials, and type of cement.

Concrete also has different classifications based on strength classes. This represents the minimum compressive strength that a particular type of concrete must achieve after curing time. When performing concrete strength testing, cylinders and cubes are also often used to measure the compressive strength of concrete. Strength increases with the age of concrete. These samples are typically cured for 28 days and then tested in a compression testing machine. Strength classes, characteristic cylinder and cube strength for different types of concrete can be found in NS-E 206 [7].

2.2.2 Reinforced concrete:

The concept of reinforcing concrete with steel, also known as reinforced concrete, involves placing steel reinforcement rebars inside the concrete structure before it is poured. Once the concrete is cured, a strong bond between the two materials is created. Concrete is strong in compression but weak in tension, so the addition of steel reinforcement provides the concrete with the necessary strength to resist cracking and failure under tensile loads. The tensile strength of concrete is typically only about 10% of its compressive strength. which is why it is often considered negligible when doing design considerations [8, p.7].

Learning that the bond between the two materials allows the transfer of stresses from one material to the other can be helpful when discussing TCC in later chapters. Even though there might be some slip between the two materials, in the analysis and design of reinforced concrete, it's assumed that it`s full composite action between the steel and concrete. This means that the strain distribution is evenly distributed between the two materials [8, p.7].

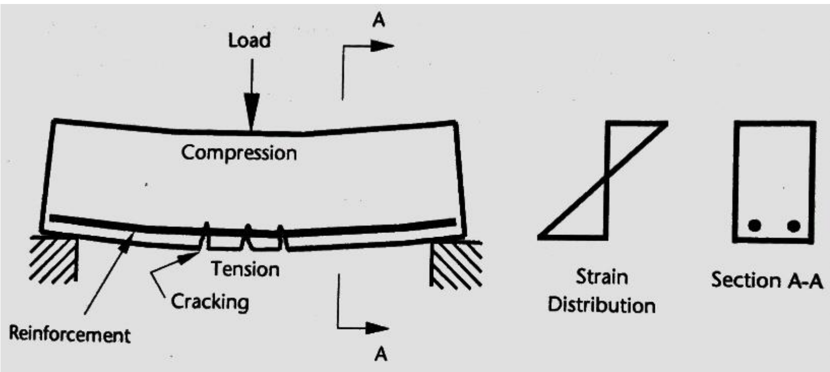

Figure 2.4 A composite action between concrete and steel [8]

2.2.3 Concrete in TCC:

Some research has been done on concrete types in TCC. When selecting concrete for this kind of project, it is typical to search for concrete with:

- Enough strength (normally small concrete compressive strength is needed in TCC)
- Low density (especially in cases where you want to strengthen existing timber structures).
- Good workability at the site. (Moldability and Compatibility).

Some research on self-compacting concrete (SCC) and fiber-reinforced concrete shows that these are the best possible solutions for TCC. The downside of this is the sensitivity to small changes in the mixed composition [9].

Steel Fiber Reinforced Concrete (SFRC) offers the possibility of not using steel bar reinforcement. This is important, particularly for strengthening existing timber beam ceilings normally located inside buildings. This concrete also makes it possible to create a smaller concrete height, which makes it possible to lower the dead load of the TCC [9]. Research shows that SFRC in TCC have great ductile structural behavior connected with dowel-type shear fasteners [9]. This means that it has more chances to deform plastically without fracturing. Figure 2.5 shows the comparison between specimens with SFRC with a fiber content of 50kg/m^3 and class C20 compared with plain concrete. SFRC resulted in a 27% increase in load capacity and a 60% increase in shear fasteners stiffness [9].

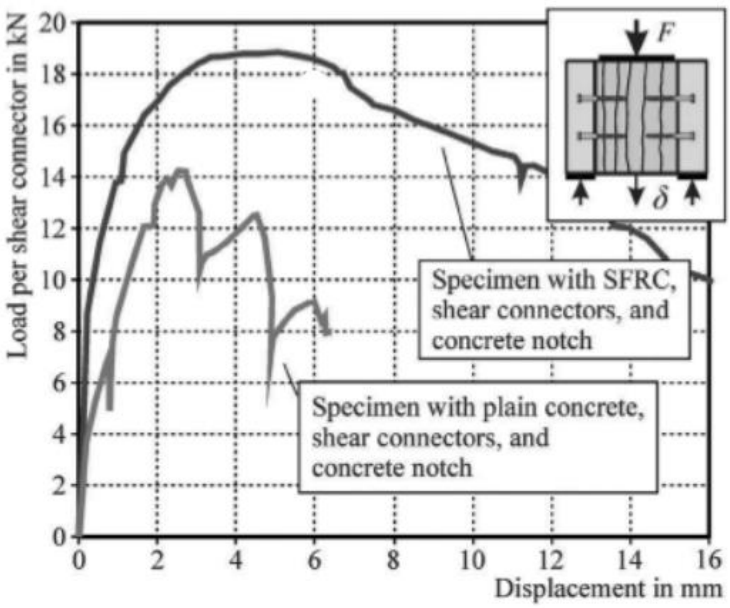

Figure 2.5 Improved structural behavior of shear fasteners with SFRC [9]

2.3 Timber Concrete composite

2.3.1 A composite system

The possibility of combining different structural materials has become increasingly important in the construction world. The idea of a composite system is to use one material to strengthen the weaknesses of another material to create composite elements or structures. The material for composites is selected for key reasons like strength and stiffness. When choosing composite elements, one can also consider several other factors, such as mass, acoustics, and fire resistance. From a business standpoint, composite systems can be more economical than non-composite ones. The most common composite structures consist of concrete and steel. For instance, a composite floor made of concrete and steel decking can be cheaper than a solid concrete floor.

As popular as concrete and steel are in the construction world, they require a lot of energy, which significantly affects global CO_2 emissions. To address these challenges, there are efforts underway to reduce the carbon footprint of construction materials. This includes developing new materials that require less energy to produce and finding ways to reduce emissions linked to the production of concrete and steel. One development is the growing use of design and construction practices that aim to minimize the environmental impact of buildings. Currently, TCC systems are used to reduce CO_2 emissions, and the application of TCC in buildings is increasing.

2.3.2 Introduction to timber-concrete composite

TCC is a construction material that consists of two layers: a structural layer made of engineered wood products and a layer made of concrete. The concrete layer improves the load capacity of the structure. This load capacity is also influenced by the shear connections used in the TCC system. The fasteners create a mechanical joint between the timber and concrete layers that allows the two sections to work efficiently together and take advantage of each other’s material strengths. The concrete element is the upper part and absorbs the compressive forces in the slab. Timber is the lower element and is in the tensile zone, where tensile stresses are expected. This way, the goal of an efficient load-bearing structural system is accomplished. A typical section of TCC system consists of timber, concrete, and shear connectors, as shown in figure 2.6.

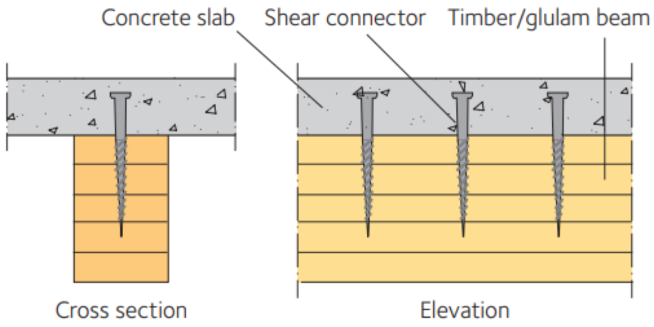

Figure 2.6 Typical TCC cross section [3, p.207]

In the beginning, the TCC system was mainly used for renovating old timber floors and other structures [10, p.8]. TCC can also be used on existing timber floors by adding the concrete part. This has become trendy in the European market, especially in Austria, Switzerland, and Italy. Examples of structural applications of TCC elements are floor structures, high-rise buildings, and bridges.

2.3.3 Advantages of TCC

Consequently, TCC slabs are made up of different materials, they have different properties that offer advantages over equivalent structures that are fully made of timber or concrete.

Concrete has a higher modulus of elasticity than timber. This means that it is stiffer and has higher resistance against bending, which leads to a smaller deflection. The rigidity of concrete can be beneficial in situations where a stable floor is required. As mentioned, concrete also has a higher compressive strength than timber, and optimal use of the properties of both materials leads to an overall higher load capacity when TCC is compared to an equivalent timber-only slab. A four-point bending test was conducted in a test [11], and the conclusion was that the bending capacity got 3-5 times bigger when CLT-composite floor was compared to CLT-floor. Larger spans of TCC structures can be built with the addition of concrete because of its higher bending stiffness [1, p.92].

The high density of concrete makes it a good material for blocking sound transmission. This can improve sound insulation. Limited tests are conducted to measure the natural frequencies, mode shapes, and damping ratios of the floors, which are important factors that affect a floor's dynamic behavior. The test [11] concludes that CLT-concrete composite slabs have better dynamic properties than CLT floors and make them more suitable for applications that require good vibration performance.

Introducing TCC floors is a practical solution to addressing challenges with timber light-weight floors, as it allows for an increase in floor mass, which can reduce vibrations and improve acoustic performance [3, p.207]. The use of timber reduces the weight of the structure, which results in less weight on the foundation. These elements, both timber and concrete, can be prefabricated, making transportation to the construction site easier. The conveniences, including a faster assembly of the elements, make the overall construction process faster and cheaper.

As mentioned, one of the key benefits of TCC constructions is to reduce the CO_2 emissions compared to concrete-only constructions. Timber is a renewable resource that stores carbon during its lifetime, while concrete emits CO_2. By including timber in the building structure, the overall carbon footprint of the structure can be reduced. Timber also has a higher thermal insulation capacity than concrete, which means that buildings made with TCC require less energy for heating and cooling. Timber is aesthetically pleasing to humans, and lots of structures have more aesthetic requirements than before. The benefits of TCC compared to timber-only and concrete-only slabs, are shown below:

- TCC compared to timber-only slabs:
    - Increase in bending stiffness
    - Increase in load carrying capacity
    - Longer span is possible
    - Improved sound insulation
    - Improved dynamic properties
    - Improved fire safety

- TCC compared to concrete-only slabs:
    - Reduced CO_2 emissions
    - Lower self-weight
    - Conveniences in the construction process
    - Better aesthetics view

[12, p.17] [13, p.9]

2.3.4 Behaviors of TCC:

The shear connectors rigidity and the ability to transfer shear force the two materials and affect the amount of slip between the timber and concrete layers. Composite action refers to the structural behavior of two materials when they work together to resist applied loads and stresses to some degree. A high degree of composite action increases the load capacity and stiffness of the structure. Different cases of composite action are shown in figure 2.7.

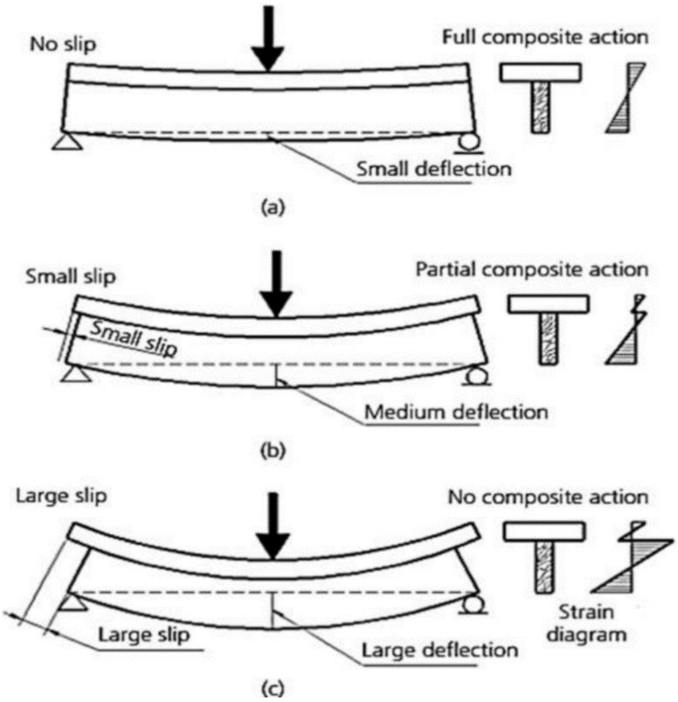

Figure 2.7 Timber concrete composite actions [10, p.38]

Full composite action, where the connection is as rigid as possible, will limit the slip between the layers. The timber is placed under tension, while the concrete is placed under compression to the greatest extent possible. This allows the stress to be distributed as evenly as possible between the two materials. Full composite action is desired in the design of TCC systems to maximize their structural performance [14, p.26]. In a non-composite action, the maximum slip between timber and concrete will occur. The two layers will act individually, with no connection between them. There is no shear force transferred. The deflection will be significant due to the slip, and the timber may experience failure.

The most realistic scenario for a TCC element is partial composite action. In this scenario, there is some connection between the layers, but some slippage may occur. This lies somewhere between the two extreme scenarios.

One way to evaluate the efficiency of the shear fastener is by comparing the theoretical prediction of the deflection with the actual deflection of the composite. A researcher, Gutkowski [10, p.38] provided an efficiency formula that ranges from 0% to 100%, representing the extremes of no composite action (where the two materials act separately) and full composite action (where they behave more like a single unit). The efficiency formula for TCC proposed is as follows:

$$Efficiency = \frac{D_N - D_I}{D_N - D_C}$$

Where:

D_N is the theoretical deflection, with no composite action

D_C is the theoretical full composite deflection, with full composite action

D_I is the actual deflection

Another way to measure efficiency is provided by comparing the theoretical prediction of bending stiffness with the actual bending stiffness. A formula for the efficiency of the interlayer connection is as follows, where $0 < \gamma < 1$ [10, p.38]:

$$\gamma = \frac{EI_{real} - EI_0}{EI_\infty - EI_0}$$

Where:

EI_0 is the theoretical bending stiffness, with no composite action

EI_∞ is the theoretical bending stiffness, with full composite action

EI_{real} is the actual bending stiffness

2.4 TCC systems:

2.4.1. Structural systems:

Normally, TCC consists of a timber slab and a concrete slab. This is because the majority of TCC is used on floors, ceilings, and decks of bridges. The widths of both the concrete and timber parts are equal, and the neutral axis is often located in the concrete slab. There is another design of TCC where a timber beam acts as the web and a concrete slab act as the flange. In this case, the neutral axis of the TCC is usually in the timber part. The TCC system where the timber part is both slab-type and beam-type is shown in figure 2.8 [15].

This definition can also be referred to as a linear TCC system with timber beams such as GLT, LVL, or other solid timber suitable for beams. On the other hand, a planar TCC system can be used with timber slabs typically made of CLT, GLT, or other timber boards [13, p.10].

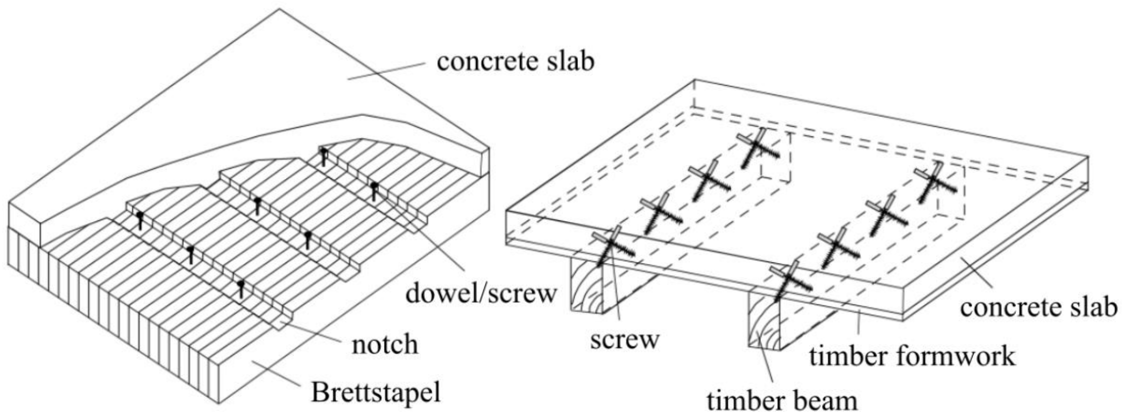

Figure 2.8 Slab-type and beam type of TCC [13, p.11]

2.4.2 Shear connectors:

As mentioned, shear connectors are an important component of the TCC floor system. The shear connectors play a fundamental role in transferring the shear forces between the concrete and CLT elements. Shear connectors in a composite system can't make a connection that is completely rigid, but the connection system must be designed so that it is as rigid as possible. This way, the detachment mechanism can be avoided. Detachment mechanisms can occur when the CLT panel and concrete slab are not properly connected, which can lead to separation between the two materials and reduced load-carrying capacity. Shear connectors prevent detachment mechanisms by ensuring a strong and rigid connection between the two materials [11].

The type of shear connector used can depend on several factors, such as the thickness of the concrete slab and the loading conditions of the structure. But also, the price and the complexity should be considered. The overall cost of the TCC structure can be affected by the type of shear connector used, and some shear connectors may be easier to install and more compatible with certain types of timber and concrete elements than others. The commonly used shear connectors are [9]:

- The commonly used shear connectors are:
    - Dowel type steel fasteners (e.g., screws, inclined screws, bolts, nails)
    - Notches
    - Combination of notches with steel fasteners.
    - Other connections (e.g., nail plates, glued connections)

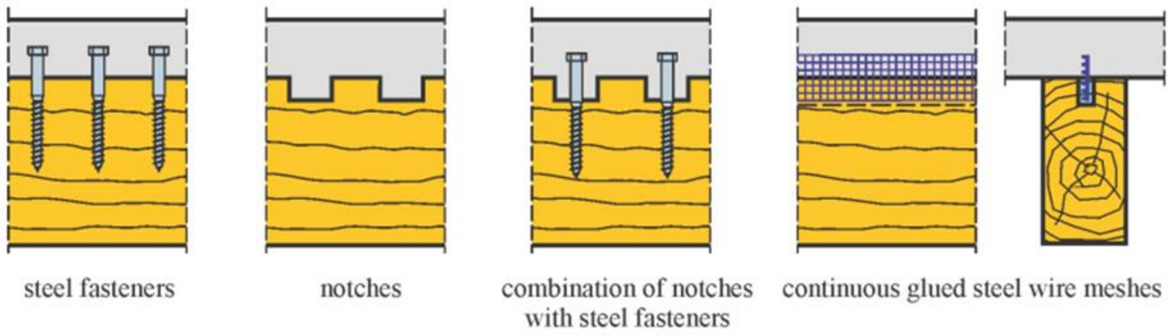

Figure 2.9 Different types of shear connectors for TCC [16]

2.4.3 Load-slip test:

The load-slip response describes the behavior of the shear connectors under load and how much they deform or slip as the load is applied. This depends on the type of connector used and its mechanical properties which will also influence the maximum slip that occurs in a TCC structure. This maximum slip between timber and concrete occurs when there is no connection between the two layers [17]. The slip modulus of a shear connector is a way to measure how well it keeps two parts from moving or slipping apart, like a timber and a concrete slab. When shear connectors have a higher slip modulus, it means that the shear connector is stiffer and can resist slipping better. This results in a lower slip value in the composite system, meaning that the amount of displacement between the timber and concrete components is reduced. A low slip modulus shows higher ultimate deformation. Table 2.1

shows the expected relation between the slip modulus and ultimate deformation for TCC connections. When choosing connections for TCC systems, it is coherent to select shear connectors with an ultimate deformation capacity higher than the maximum slip demand between the timber and concrete in the composite system. An overview of the load-slip behavior of different types of shear connectors is shown in figure 2.10 [17].

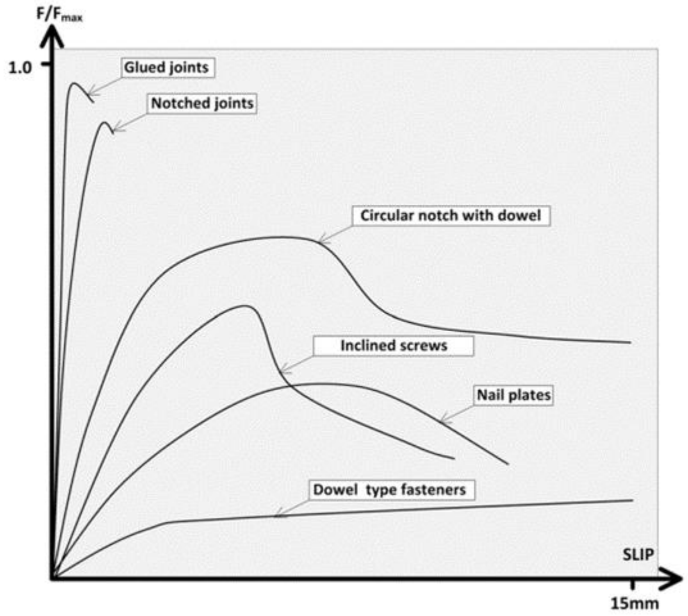

Figure 2.10 Load–slip diagrams of TCC connections [17]

Table 2.1 Slip modulus and ultimate deformation capacity for TCC connections [17]

Connection	K_{ser} (N/mm)	δ_{ult} (mm)
Nails	2 041	15
Screws	1 825	15
Dowels	7 600	15
Nail plates	48 800	10
Inclined screws	29 200	5
Steel mesh	415 460	4
Circular notch combined with dowels	79 500	15
Rectangular notches	132 500	0.5
Inclined glued-in rebar	103 000	2

2.4.4 Different connection system:

Screw connections

Screw connections are the most researched and used connectors for TCC. This is because screws are simple and easy to install. They can also be used for many different things, from small structures to big buildings. The load capacity of these fasteners can be improved by their spacing and increasing their diameter and length. These fasteners can also be used in combination with other types of connectors.

Notched

In a notched TCC system, a notch is cut out of the timber member, and the empty place is filled with concrete later. The most common shapes are rectangles and triangles. To minimize the deflection forces, transfer tensile forces and improve ductility, screws are often used with this type of connection [1, p.94] [18]. Notched TCC system and combination with additional screws is the second most researched group after screws or dowels [13, p.28] [12, p.36].

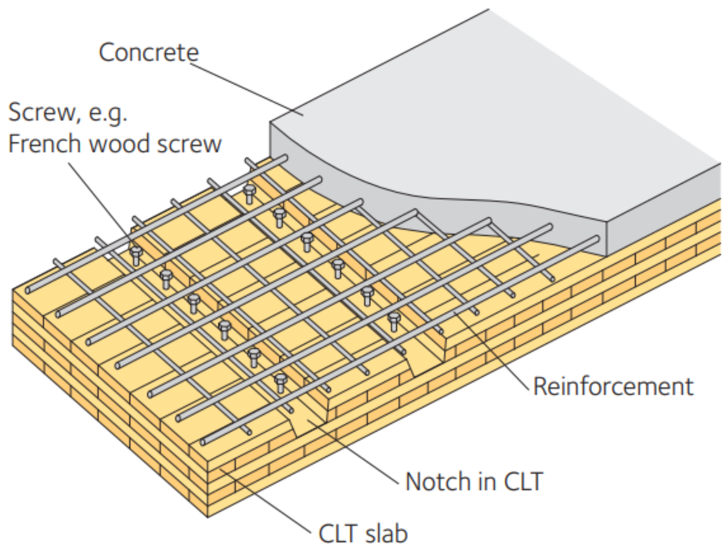

Figure 2.11 TCC system with notched connection and screws [1, p.94]

Micro-notches have been researched a lot in Switzerland, where smaller notches are created using computer milling machines. Normally, the connection between timber and concrete requires deep notches and additional connectors. This means a lot of milling and a lot more screws. With this new system, screws are not needed at all, and the milling process is easy. On the construction site, the installation is less complicated, and concrete can be poured on top. In comparison to typical notches and screw systems, micro-notches have also reached the requirements of good stiffness and strength values [13] [19]. The research with micro-notches showed that micro-notches as a connection system for TCC have a high stiffness, with slip modulus for most configurations being over 10 kN/mm^2 [13, p.74].

In a perforated steel plate system

Perforated plates are inserted in the timber slab, which creates the mechanical connection required between the two materials. The perforations allow the concrete to flow through and surround the plate which is where the stiffness comes in.

An example of this type of connection system is Holz-Beton-Verbund (HBV). Glued-in steel plate is inserted in the halves of both the timber and concrete members. German professor Leander Bathon is credited with introducing a type of glued steel plate connection with both high stiffness and good ductility [12, p.69] [20].

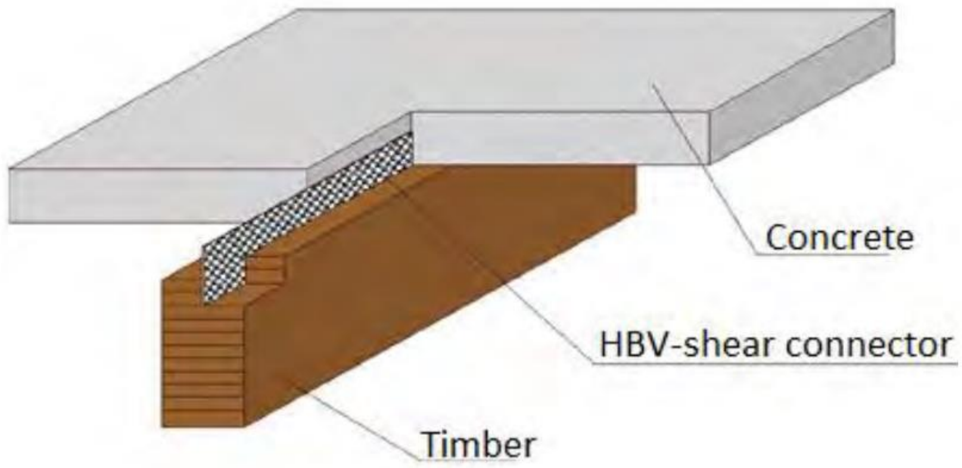

Figure 2.12 Example of HBV connection system [20]

3. Specimen design and properties

3.1 Overview of materials

CLT

The received CLT slabs (15 samples) can be applied for standard construction applications, and they have similar durability to regular construction timber. The slabs are intended to be used indoors, they don't require any additional protection or treatment beyond what is generally employed for standard construction timber [5].

CTC screw

500 CTC screws were received from Rothoblaas. The company was founded in 1992 and has since grown to become worldwide in the development and production of high-quality and innovative fastening solutions for timber construction, including structural connectors and screws [21].

CTC is a screw connection for timber-to-concrete floors and is useful for CLT and other timber-based panels. It is a specialized screw designed to fasten timber and concrete materials together in construction projects. Factors such as load capacity, compatibility with both materials, fastener spacing, and angles are all considered when selecting this type of connector. The choice of shear fasteners was based on availability and the recommendation of a former master's student who had undergone similar thesis work [21].

Concrete

Sola Betong supplied the concrete for this project. Of particular interest for this thesis are Sola Betong's efforts to increase their environmental sustainability. The company has established a "Concrete Hotel", which recycles excess concrete and uses it as a fully workable product the following day, promoting increased recycling and less waste. Sola Betong offers its own low-carbon concrete mixes that reduce CO_2 emissions compared to industry norms. These practices contribute to limiting CO_2 emissions associated with concrete production [22].

Because these are smaller CLT panels, self-compacting concrete (SCC) was an excellent choice for this test because of its ability to easily flow through the formwork and fill in without any extra vibration or a lot of compactions. The placement is fast and requires less labor, therefore it also makes sense that this type of concrete is more expensive because of increased usage of admixtures and particles. The smaller dimensions of the panels also allowed us to choose B35, a medium-strong concrete.

3.2 CLT slabs

15 CLT slabs were received from Splitkon, cut to a length of 1600 mm and a width of 600 mm. The panels were prefabricated prior to delivery from the supplier. The thickness of the layers varied with the total height being h_{CLT}=120 mm. The supplier has provided material properties for the slabs, such as characteristic strength values, stiffness, and density values [5]. The standard structure of the 5-layer CLT slabs is shown in table 3.1, where layer 1 starts from the bottom layer when performing the test.

Table 3.1 Structure of the 5-layer CLT slab [5]

Layer	Direction	Thickness (mm)	Strength class
1	Long, x-axis	19	T22
2	Trans, y-axis	21	T15
3	Long, x-axis	40	T15
4	Trans, y-axis	21	T15
5	Long, x-axis	19	T22

Table 3.2 Materials properties of CLT in general [5]

Parameter	Notation	Value
Length	L	1600 mm
Width	b	600 mm
Height	h_{CLT}	120 mm
Cross sectional Area	A_{CLT}	72000 mm^2
Moment of Inertia	I_{CLT}	86400000 mm^4
Partial factor	γ_M	1.15 [1, p.35]
Modification factor	k_{mod}	0.8
Deformation factor	K_{def}	0.85

Table 3 Material properties for CLT, strength class T15 [5]

Parameter	Notation	Value
Mean value of modulus of elasticity, along the grain	$E_{0,mean}$	11500 MPa
Mean value of modulus of elasticity,90° to the grain	$E_{90,mean}$	230 MPa
Mean value of the shear modulus, along the grain	$G_{0,mean}$	720 MPa
Mean value of the shear modulus, 90° to the grain	$G_{90,mean}$	72 MPa
Bending strength	$f_{m,k}$	22 MPa
Tensile strength along the grain	$f_{t,0,k}$	15 MPa
Compressive strength along the grain	$f_{c,0,k}$	21 MPa
Shear strength	$f_{v,k}$	4.0 MPa
Density	ρ	430 kg/m3

Table 3.4 Material properties for CLT, strength class T22 [5]

Parameter	**Notation**	**Value**
Mean value of modulus of elasticity, along the grain	$E_{0,mean}$	13000 MPa
Mean value of modulus of elasticity,90° to the grain	$E_{90,mean}$	430 MPa
Mean value of the shear modulus, along the grain	$G_{0,mean}$	810 MPa
Mean value of the shear modulus, 90° to the grain	$G_{90,mean}$	81 MPa
Bending strength	$f_{m,k}$	30,5 MPa
Tensile strength along the grain	$f_{t,0,k}$	22 MPa
Compressive strength along the grain	$f_{c,0,k}$	26 MPa
Shear strength	$f_{v,k}$	4 MPa
Density	ρ	470 kg/m3

3.3 CTC screws

3.3.1 CTC screw properties

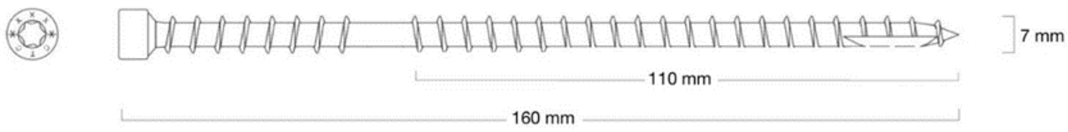

Figure 3.1 CTC screw [21]

Table 3.5 CTC screw properties [21]

Parameter	Notation	Value
Diameter	d	7 mm
Length	l	160 mm
Effective length	l_{eff}	110 mm
Characteristic tensile strength	$F_{tens,k}$	20 kN

3.3.2 Slip modulus

To ensure effective use of the screws one must consider stiffness of the screw. It is necessary to calculate the slip modulus K_{ser} and K_u of the screws. K_u is used to evaluate the slip resistance of the screws under SLS loads, while K_u ULS loads.

45° orientation

According to manufacturer's catalogue [21, p.227], K_{ser} for 45° orientation can be calculated using the equation:

$$K_{ser} = n*70*l_{eff}$$

$$K_{ser} = 3*70*110 = 23100\ MPa$$

where n is the the number of rows of screws and l_{eff} is the effective length of the screws in the connection. K_u is taken as two thirds of the slip modulus in the SLS Eurocode 5, [23, clause 2.2.2] and some assistance from [3, p.114].

$$K_u = \frac{2}{3} * K_{ser}$$

$$K_u = \frac{2}{3} * 23100 = 15400\ MPa$$

90° orientation

According to the ETA (European Technical Assessment) given by the manufacturer the slip modulus for 90° degree CTC-screws with a diameter of 7 mm is [24, p.9]:

$$K_{ser} = 1800\ MPa$$

Value for K_u is calculated according to Eurocode 5 [23]:

$$K_u = \frac{2}{3} * K_{ser} = 1200\ MPa$$

3.3.3 Spacing and orientation

To calculate the minimum distances for the shear connectors, the formula provided by the supplier Rothoblaas is employed. This formula considers various factors such as the diameter of the screws and the thickness of the timber and concrete layers. It assumes that the thickness of the timber beam is larger than 100 mm [21, p.227].

The maximum and effective spacing is taken from Eurocode 5 [23, clause 9.13] . This formula is available if $S_{max} \leq 4S_{min}$.

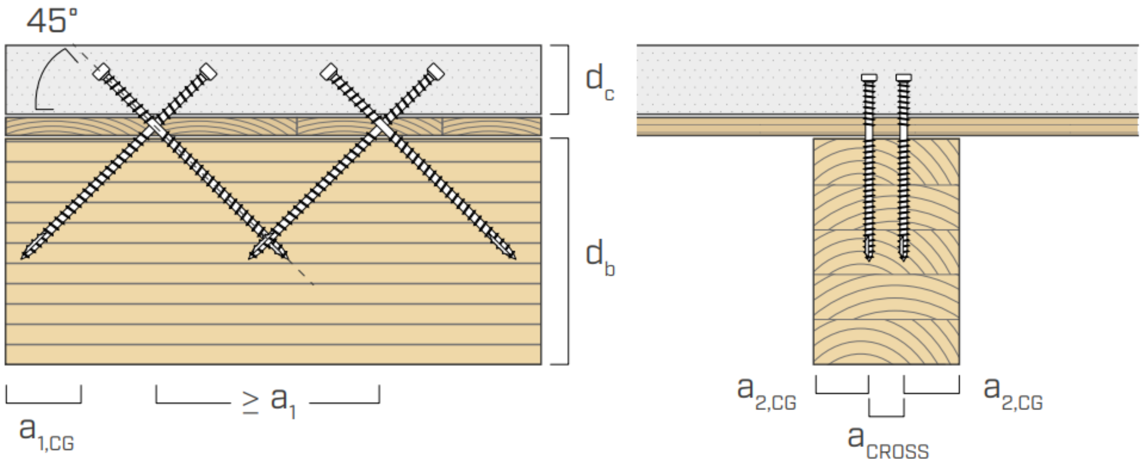

Figure 3.2 Minimum distances for 45° orientation [21, p.227]

Table 3.6 Minimum spacing, end and edge distances for 7 mm CTC screw [24]

Distance	Minimum spacing
Spacing parallel to grain a_1=S	130*sin (α)
Spacing perpendicular to grain a_2	35
End distance $a_{1,GG}$	85
Edge distance $a_{2,GG}$	32
Spacing between the two crossed a_{cross}	11

Spacing for 45° orientation

$$S_{min} = 130 * \sin (45)$$

$$S_{min} = 91.9\ mm$$

$$S_{max} \leq 4S_{min}$$

$$S_{max} \leq 367.7\ mm$$

$$S_{eff} = 0.75 * S_{min} + 0.25 * S_{max}$$

$$S_{eff} = 160.9\ mm$$

Spacing for 90° orientation

$$S_{min} = 130 * \sin (90)$$

$$S_{min} = 130\ mm$$

$$S_{max} \leq 4S_{min}$$

$$S_{max} \leq 520\ mm$$

$$S_{eff} = 0.75 * S_{min} + 0.25 * S_{max}$$

$$S_{eff} = 227.5\ mm$$

Choosing the spacing

Generally, the screw spacing should be between S_{min} and S_{max} to get the best results. It's also recommended to use spacing close to S_{eff}. Before determining the slip modulus values for 90° screws, we needed to determine the appropriate screw and spacing. Initially, we were unaware that the minimum spacing required was 130 mm. The reason behind using 125 mm spacing for the 90° orientation was that it provided the same number of screws as the 45° orientation with 250 mm spacing. We wanted to determine if this could generate similar results. Additionally, the 90° orientation was much easier to install and manufacture, resulting in a more cost-effective and potentially automated process.

3.4 Specimen arrangement and groups

The table 3.7 shows information about the different specimen groups. It is Important to note that all specimens under the same letter are identical in terms of screw spacing, screw orientation and quantity. The quantity of screws in each group was calculated manually based on the known spacings.

Table 3.7 Spacing and orientation for each specimens group

Specimens	Spacing	orientation	Number of screws
A1,A2,A3	200 mm	45° double crossed	126
B1,B2,B3	250 mm	45° double crossed	108
C1,C2,C3	200 mm	90° single	63
D1,D2,D3	250 mm	90° single	54
E1,E2,E3	125 mm	90° single	108

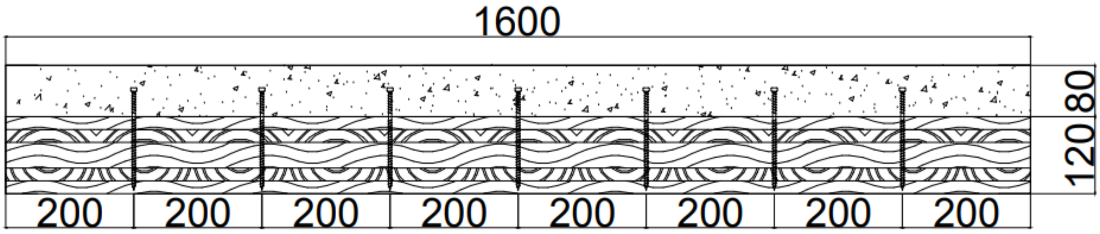

Figure 3.3 Long-side of cross section, CLT-concrete slab group C

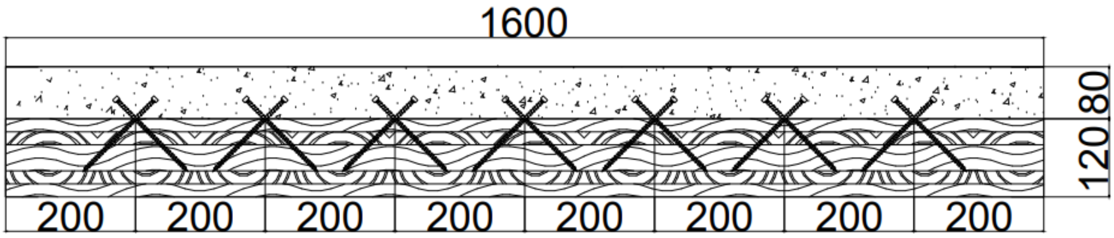

Figure 3.4 Long-side of cross section, CLT-concrete slab group A

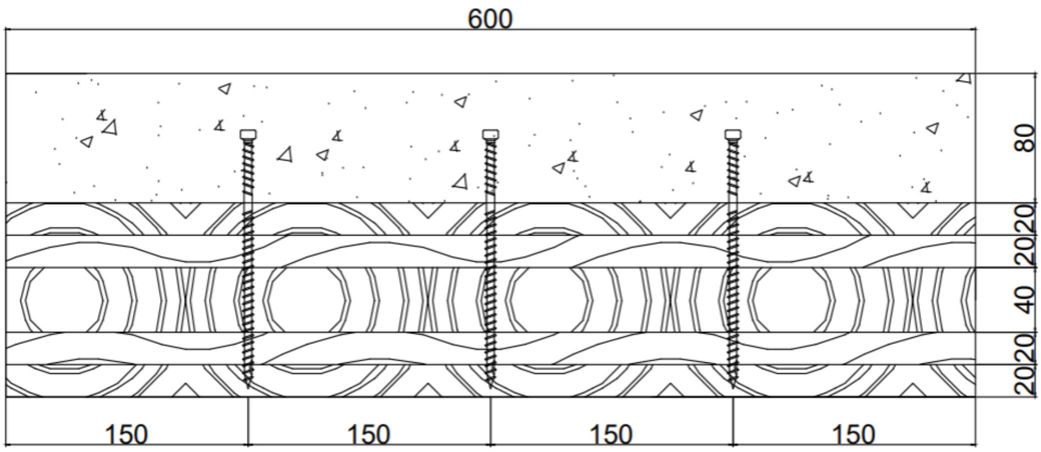

Figure 3.5 Short-side of cross section, CLT-concrete slab group C, D and E

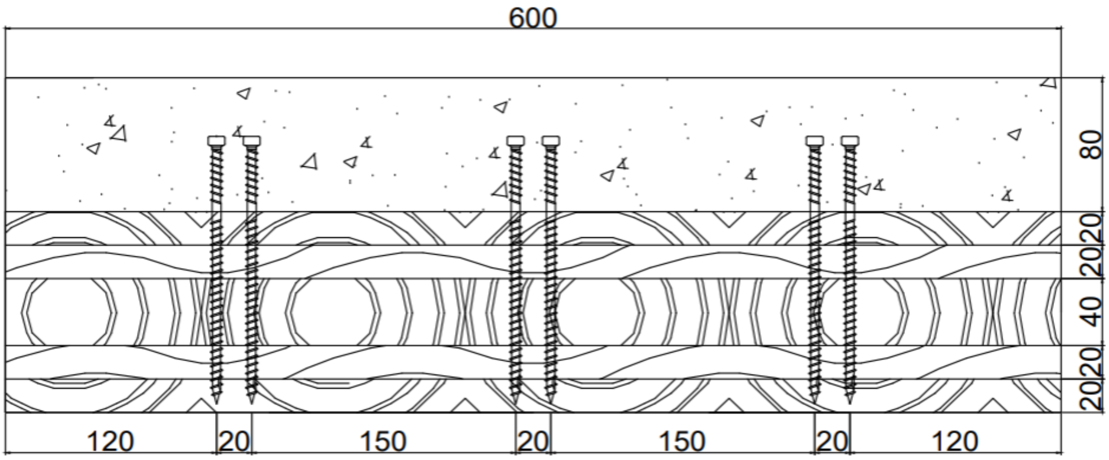

Figure 3.6 Short-side of cross section, CLT-concrete slab group A and B

3.5 Concrete height and properties

The required thickness of the concrete layer will depend on factors such as the screw properties, the properties of the CLT panels, and the loads applied to the TCC slab. Due to the lack of standards for determining the height of the concrete layer in TCC, various sources were relied on before deciding the concrete height. The appropriate height for the concrete layer in the project was determined to be $h_c = 80\ mm$. Some considerations were done based on several references including the same considerations in Marias master thesis [25]:

- Rothoblaas [21, p.227]:

$$50\ mm \leq h_c \leq 0.7 * h_{CLT}$$

$$50\ mm \leq h_c \leq 84\ mm$$

- CLT handbook [1, p.94]:

$$h_c = 0.4 * \frac{h_{CLT}}{0.6}$$

$$h_c = 0.4 * \frac{120}{0.6} = 80\ mm$$

- Wurth [26, p.15]:

$$50\ mm \leq h_c \leq 0.7 * h_{CLT}$$

$$50\ mm \leq h_c \leq 84\ mm$$

Table 3.8 Materials properties of concrete B35 [27, table 3.1]

Parameter	**Notation**	**Value**
Length	L	1600 mm
Height	h_c	80 mm
Width	b	600 mm
Cross sectional area	A_c	48000 mm^2

Moment of inertia	I_c	25600000 mm^4
Characteristic compressive strength of concrete	f_{ck}	35 MPa
Characteristic compressive cube strength of concrete	$f_{ck,cube}$	45 MPa
Modulus of elasticity	E_{cm}	34000 MPa
Characteristic tensile strength of concrete	$f_{ctk,\ 0,05}$	2.2 MPa
Partial factor	γ_c	1.5
Creep coefficient	φ	2.5
Density	ρ_c	25 kN/m^3

4. Analysis of TCC elements

4.1 The γ-method

In the present time, the most widely accepted analytical approach to timber-concrete composites is the γ-method. In this method the effective bending stiffness of a composite section depends on the degree of composite action. For simplification reasons, the shear connectors are assumed to be uniformly distributed along the span. To adopt the method and study behavior of the tested floor, effective fictious (equivalent) spacing was considered. The method uses the γ-factor ranging from 0 to 1. Whereas the degree of composite action is regarded, where 0 gives no composite action and 1 gives full composite action [28, p.132-133]. This concept is explained in more depth in chapter 2.3.4.

The composite must satisfy both ULS and SLS for short and long-term loads. The ULS is assessed by evaluating the maximum stresses in the component's materials (timber, concrete, and connectors). It us elastic analysis while the SLS is checked by evaluating the maximum deflection. Eurocode 5-Part 1-1, Annex B provides a simplified method for calculating these parameters of mechanically jointed beams with flexible elastic connections, under the following assumptions [10, p.41-42]:

- The beam is simply supported by a span *l*:
    - For continuous beams: *l* equal to 0.8 of the relevant spans
    - For the cantilever length: *l* equal to 2 times the relevant spans

- The individual part (of wood, wood-based panels) is either full length or made with glued joints.
- The individual parts are connected to each other by mechanical fasteners with a slip modulus k.
- The spacing S between the fasteners is constant or varies uniformly according to the shear force, between S_{min} and S_{max} $S_{max} \leq 4S_{min}$.
- The load acts in the z-direction giving a moment M=M(x) that varies sinusoidally or parabolically, and a shear force V=V(x) [23, clause B.1.2].

In the Eurocode 5 [23, Annex B], figure 4.1 shows which cross sections this method is applicable to and how the symbols are defined:

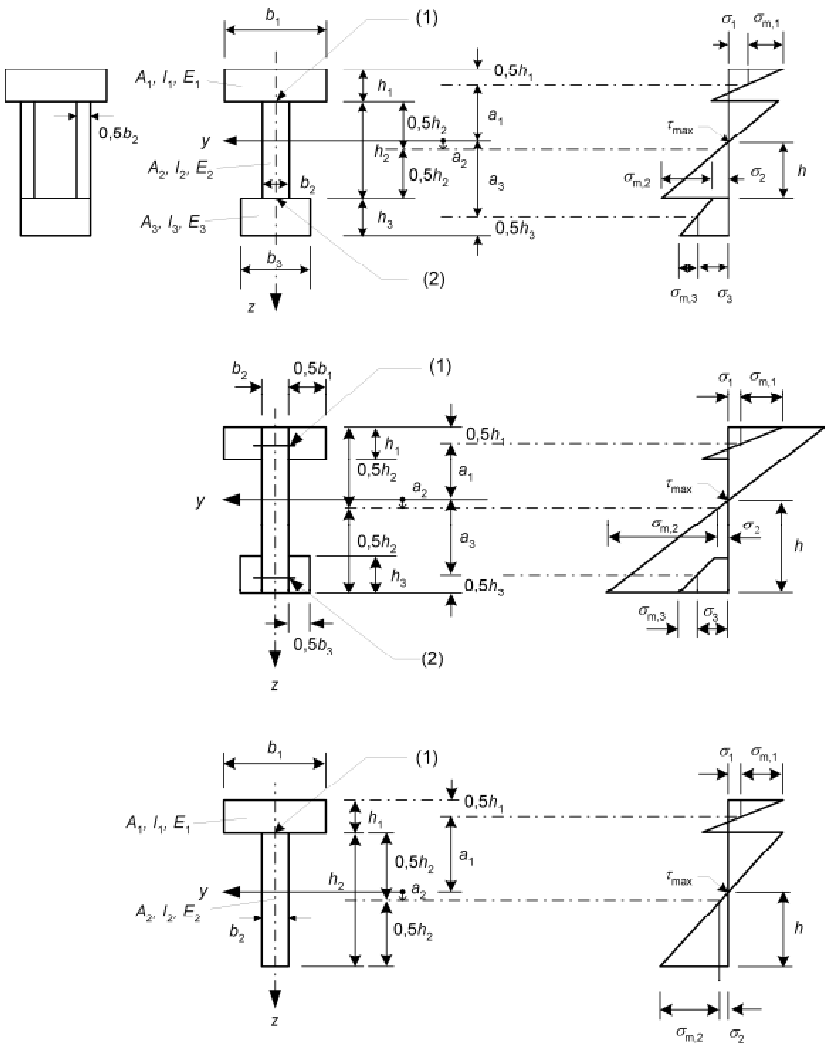

Figure 4.1 C/S of a composite with partial composite action [23, Figure B.1, in Annex B]

According to the y-method, the effective bending stiffness (EI)eff of simply supported timber-concrete composite is given as followed [23, clause B.2]:

$$(EI)_{eff=}\sum_{i=1}^{3}(E_i l_i+\gamma_i E_i A_i a_i^2)$$

Where E_i refers to the modulus of elasticity of a layer "i" and the other values are given by:

$$A_i = b_i h_i$$

$$I_i = \frac{b_i h_i^3}{12}$$

$$\gamma_2 = 1.0$$

$$\gamma_i = [1.0+\frac{\pi^2 E_i A_i S_i}{K_i l^2}]^{-1} \quad \text{for } i = 1 \text{ and } i = 3$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1+h_2) - \gamma_3 E_3 A_3 (h_2+h_3)}{2\sum_{i=1}^{3}\gamma_i E_i A_i}$$

Where the I_i is the moment of Inertia of a layer "i", A_i is the cross-sectional area of layer "i". a_i being the distance from the neutral axis to the center of the layer "i". S_i is the spacing between the shear fasteners. K_i is the stiffness of one single shear fasteners and depends on the limit state:

- $K_i = K_{ser,i}$ for serviceability limit state (SLS)
- $K_i = K_{u,i}$ for ultimate limit state (ULS)

Normal stresses

The normal stresses are given by:

$$\sigma_i = \frac{\gamma_i E_i A_i M}{(EI)_{eff}}$$

$$\sigma_{m,i} = \frac{0.5 E_i h_i M}{(EI)_{eff}}$$

Maximum shear stress

The maximum shear stress occurs when the normal stresses are zero. As an example, the web in member (2) in figure 4.1 are given by:

$$\tau_{2,max} = \frac{(\gamma_3 E_3 A_3 a_3 + 0.5 E_2 b_2 h^2}{b_2 (EI)_{eff}} * V$$

Load on shear connectors

Load on a single shear fastener is given by:

$$F_i = \frac{\gamma_i E_i A_i a_i S_i}{(EI)_{eff}} * V$$

Where i=1 and i=3 respectively. Si = Si(x) is the spacing of the fasteners that is described in [23, clause B.1.3(1)].

In the γ-method the spacing of the shear fasteners is considered of equal length. If the shear fasteners are of varying length along the longitudinal direction, the effective spacing should by calculated according to [23, clause 9.1.3(1)-9.1.3(3)]:

$$S_{eff} = 0.75 * S_{min} + 0.25 * S_{max}$$

By implementing the above equations for a T-section beam, it is possible provide a new equation for the timber-concrete composite [10, p.42]:

$$(EI)_{eff} = E_1I_1 + \gamma E_1A_1a_1^2 + E_2I_2 + E_2A_2a_2^2$$

Where γ-factor and distance to the neutral axis a_i is given by:

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1 S}{kL^2}}$$

$$a_1 = \frac{h_c + h_t}{2} - a_2$$

$$a_2 = \frac{\gamma E_1 A_1 (h_1 + h_2)}{2\gamma E_1 A_1 + E_2 A_2}$$

Where the values for i=1 is for the concrete element and the values for i = 2 are for the timber element.

4.2 Shear analogy method

Various types of analytical models for the evaluation of the basic mechanical properties of a CLT composite have been developed and proposed. CLT is a relatively flexible and lightweight building material suitable for slabs that resist out-of-plane loading. Because of this, the design is more driven by serviceability criteria (vibration, deflection, and creep) than by strength criteria (bending and shear force) [4, Ch.3, p.10].

The shear analogy method is found with the help of a plane frame analysis program, it considers the different moduli of elasticity and shear moduli of single layers for nearly any system configuration, meaning any number of layers or span-to-depth ratios. It is also not neglecting shear deformation [4, Ch.3, p.10].

In the shear analogy method, the characteristics of a multi-layer cross-section are separated into two virtual beams A and B. Beam A is given the sum of the inherent flexural and shear stiffness of the individual plies along their own centers. Beam B is given “Steiner” points, or an increased moment of inertia, because of the distance from the neutral axis of the flexural and shear stiffness of the panel. These two beams are coupled with infinitely rigid web members so that an equal deflection between the beams can be obtained [4, Ch.3, p.11]. The overlaying of bending and shear stiffness(stresses) of both beams leads us to figure 4.2 [4, Ch.3, p.11]:

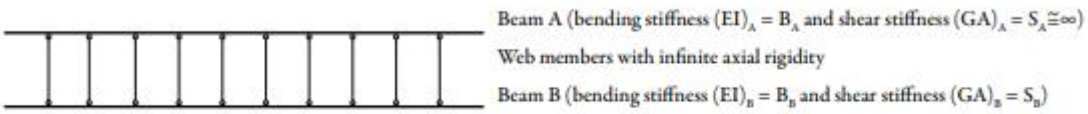

Figure 4.2 Beam modelling using shear analogy method [4, Ch.3, p.11]

Beam A is assigned a bending stiffness equal to the sum of the inherent bending stiffness of all the individual layers given as [4, Ch.3, p.11]:

$$B_A = \sum_{i=1}^{n} E_i I_i = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12}$$

Where:

$B_A=(EI)_A$

b_i=Width of each individual layer

h_i=Thickness of each individual layer

The bending stress and shear stresses of each individual layer of beam A is given as equation (1) and (2) respectively [4, Ch.3, p.12]:

$$\sigma_{A,i} = \pm \frac{M_{A,i}}{I_i} * \frac{h_i}{2}$$

$$\tau_{A,i} = \frac{E_i I_i}{B_A} * 1.5 * \frac{V_A}{bh_i}$$

Where:

M_A = Bending forces on beam A

V_A = Shear forces on beam A

The bending and shear forces on beam A using the shear analogy method are shown in figure 4.3 [4, Ch.3, p.12]:

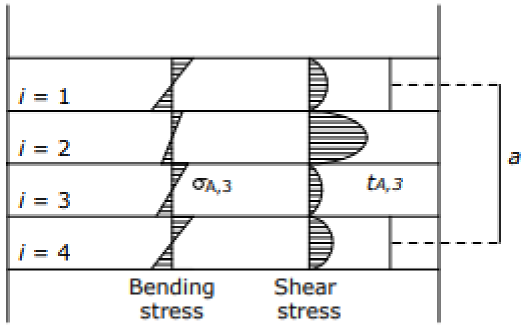

Figure 4.3 Bending and shear stresses in beam A [4, Ch.3, p.12]

The bending stiffness of Beam B is calculated by using the parallel axis theorem (given as the sum of the Steiner points of all individual layers. Here z_i is the distance between the center point of each layer to the neutral axis and B_B is $(EI)_B$ [4, Ch.3, p.12]:

$$B_B = \sum_{i=1}^{n} E_i A_i z_i^2$$

The bending and shear stresses for each individual layer of beam B is given as equation (3) and (4) respectively [4, Ch.3, p.12]:

$$\sigma_{B,i} = \frac{E_i Z_i}{(EI)_B} M_B$$

$$\tau_{A,i} = \frac{V_B}{(EI)_A} * \sum_{j=i+1}^{n} E_j A_j z_j$$

Where:

M_B = Bending forces on beam B

V_B = Shear forces on beam B

The bending and shear forces on beam B using the shear analogy method are shown in figure 4.4 [4, Ch.3, p.12]:

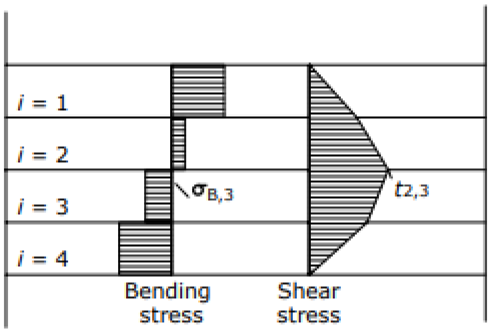

Figure 4.4 Bending and shear stresses on beam B [4, Ch.3, p.13]

The final stress distribution obtained from the superposition of the results from beam A and B is shown in figure 4.5 [4, Ch.3, p.12]:

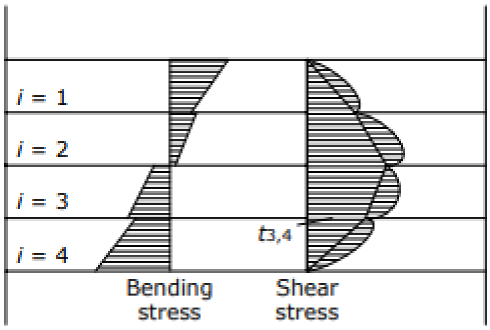

Figure 4.5 Bending and shear stresses on beam B [4, Ch.3, p.12]

From obtaining the shear and bending stresses of beams A and B the final effective bending stiffness is given by [4, Ch.3, p.13]:

$$(EI)_{eff}) = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

The effective shear stiffness is given as [4, Ch.3, p.14]:

$$GA_{eff} = \frac{a^2}{[\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i b_i}\right) + (\frac{h_n}{2G_n b})]}$$

In the equation above, it is important to use the correct material properties. E_0 (E parallel to the grain) shall be used for the longitudinal laminates, while E_{90} (E perpendicular to the grain) and $E_{90} = E_0/30$. For the longitudinal laminates, the shear modulus should be G, while for the perpendicular it should be G_R for rolling shear ($G_R = G/10$)

The shear deflection is of significant influence in CLT; hence this is included in the calculation. By adjusting the effective bending stiffness to an apparent bending stiffness, the earlier effective bending stiffness is reduced [4, Ch.3, p.14]:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Where EI_{eff} and GA_{eff} is calculated previously. L is the length of the span and KS is a constant based upon the influence of shear deformation. The constant is solved for different loading scenarios and expressed in table 4.1:

Table 4.1 Ks values for various loading conditions [4, Ch.3, p.5]

Loading	End Fixity	K_s
Uniformly distributed	Pinned	11.5
	Fixed	57.6
Concentrated at midspan	Pinned	14.4
	Fixed	57.6
Concentrated at quarter points	Pinned	10.5
Constant moment	Pinned	11.8
Uniformly distributed	Cantilevered	4.8
Concentrated at free-end	Cantilevered	3.6

4.3 Verification of cross section

The design and calculations of the structure should be in accordance with "Eurocode 0, Basis of Structural Design" [29]. The structure shall have the capacity to withstand the loads that it will most likely be influenced by during its design life. The system needs to be acceptable in terms of resistance, serviceability, and durability. In accordance with this, the structure must withstand both the ultimate limit state (ULS) and the serviceability limit state (SLS). Both short-term and long-term effects of the composite slab [29].

The structure consists of both timber and concrete. Verifications are made by two different parts since there is no official standard for TCC design and their material properties are different. The verification of the concrete structure is in accordance with Eurocode 2 [27]. The timber structure is verified in accordance with Eurocode 5 [23].

The design of TCC involves analysis of ULS and SLS. ULS implies verification of the normal stresses acting on the structure. SLS implies verification of cross section against vertical displacement.

4.4.1 Ultimate Limit State

Normal stresses of the concrete section

Both the top and bottom stresses of the cross section are verified according to Eurocode 2[27]:

At the top of concrete section, the verification is as followed:

$$\frac{\sigma_{c,t}}{f_{cd}} \leq 1.0$$

Where:

$$f_{cd} = \frac{f_{ck}}{\gamma_c}$$

At the bottom of the concrete section the verification is as followed:

$$\frac{\sigma_{c,b}}{f_{ctd}} \leq 1.0$$

Where:

$$f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_c}$$

These parameters mean as followed:

$\sigma_{c,t}$: Normal stress of the top part of the concrete section due to compression

$\sigma_{c,t}$: Normal stress of the bottom part of the concrete section due to tension

f_{cd} : Design value of concrete compressive strength

f_{ctd} : Design value of axial tensile strength of concrete

γ_c : Partial factor of concrete

Normal stresses of the timber cross section

The timber cross section is subjected to combined bending and axial tension and are verified as followed [23]:

$$\frac{\sigma_2}{f_{t,0.2}} + \frac{\sigma_{m,2}}{f_{m,d}} \leq 1.0$$

Where:

$$f_{m,d} = \frac{k_{mod} f_{m,k}}{\gamma_M}$$

$$f_{t,0,d} = \frac{k_{mod} f_{t,0,k}}{\gamma_M}$$

These parameters mean as followed:

σ_2 : Normal stress due to compression

$\sigma_{m,2}$: Normal stress due to tension

$f_{m,d}$: Design value of compressive strength of timber

$f_{t,0,d}$: Design value of axial tensile strength of timber

k_{mod} : Modification factor for duration of load and moisture content

γ_M : Partial factor of material properties

Shear stress of the timber section

The verification of the shear fasteners, connected between the timber and concrete:

$$\frac{F_1}{F_{R,d}} \leq 1.0$$

Where the parameters mean as followed:

F_1 : The acting load per fastener

$F_{R,d}$: Design load-carrying capacity per shear plane per fastener

4.4.2 Serviceability Limit State

The SLS is concerned with the structure's functions and comfort of people also the appearance of the structure in consideration with the design in [29].

The composite system is verified both for the concrete and timber section as followed:

$$\frac{\omega}{\frac{L}{250}} \leq 1.0$$

When designing the composite all relevant variables must be included. The design should predict the structural behavior of the system and ensure that it won't fail. The connectors are a critical part of this and should be investigated closely. The load capacity of the carrying shear fasteners is verified when the forces and moments between the members are determined.

All properties on the structure, the material properties from concrete, timber and shear fasteners are affected by the forced on the system. This affects both the ULS and SLS verifications.

The TCC must be verified both for ULS and SLS in the short-term and long-term. From formulas in [23, clause 2.3.2.2] a general effect for stress and displacement, designed E^{Fu} and E^{Fs} caused by the ULS and SLS can be expresses as. Thes values depend on the load applied on the beam, and on the Young`s and slip modulus of the materials in the composite [10, p.43-44]:

For ULS:

$$E^{Fu} = E^{Fu}(E_{cm}(t_0), E_{0,mean}, k_u)$$

For SLS:

$$E^{Fs} = E^{Fs}(E_{cm}(t_0), E_{0,mean}, k_{ser})$$

Where K_u and K_{ser} depends on the limit state.

For ULS one load combination is considered. As for SLS three different load combinations are considered. One is for characteristic load, one for frequent load and the last one is for quasi permanent load the equations are taken from [10, p.44].

For ULS

$$F_{d,u} = \sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,1} \psi_{0,i}\, Q_{k,i}$$

For SLS

Characteristic load:

$$F_{d,r} = \sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i}\, Q_{k,i}$$

Frequent load:

$$F_{d,f} = \sum_{j\geq 1} G_{k,j} + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i}\, Q_{k,i}$$

Quasi-permanent load:

$$F_{d,p} = \sum_{j\geq 1} G_{k,j} + \sum_{i>1} \psi_{2,i}\, Q_{k,i}$$

Where:

G: Permanent load

Q: Variable load

γ: Partial factor (safety and serviceability)

ψ: Variable factors

4.4 Short-term verification

A simplified approach has been given in Eurocode 5 [23, clauses 2 and 3] for SLS and ULS verification of TCC. The procedures assume linear elastic behavior of all components (timber, concrete and connectors) for instantaneous loading [28, p.22]. At the initial state, all loads are applied instantaneously with no creep effect as per according to Eurocode 5 and Eurocode 2. The slip modulus for the connectors is also calculated from this.

The flexibility of the connection system is taken into account using the suggested formulas in Annex B of Eurocode 5 for timber-timer composites [23]. Because of the non-linear behavior of the connection, it must be accounted for by adapting different values for the elastic stiffness for ULS and SLS verification [28, p.22].

For SLS, K_{ser} is used for verification calculations as the slip modulus, with a secant value at 40% of the collapse shear load. As for the ULS, K_u is interpreted at approximately 60% of the shear collapse load. The slip modulus in Eurocode 5 K_u can be given as 2/3 of K_{ser}.

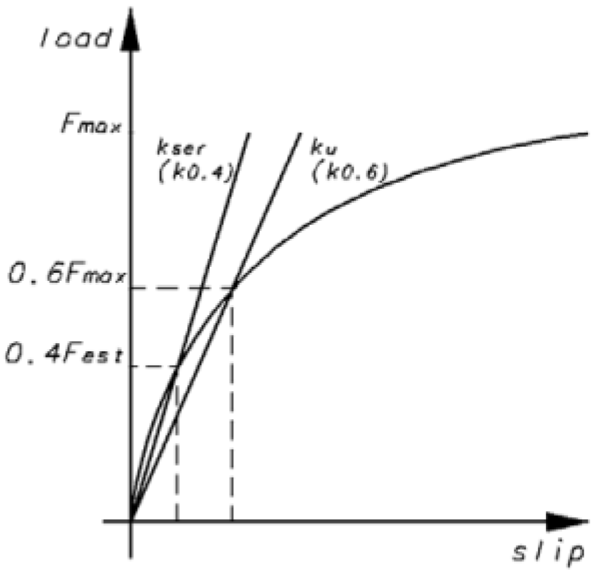

Figure 4.6 Load-slip for both K_{ser} and K_u [10]

From the European code the recommendation for short term limit state verification can be expressed as followed [30, p.10]:

Short term verification:
The short term effect in terms of stress "σ_{inst}" can be expressed as a function in the following form:

$$\sigma_{inst} = \sigma^{F_{d,u}}(E_{cm}, E_{0,mean}, k_u)$$

Where $F_{d,u}$ designates the ultimate limit state load combination.

The short term effect in terms of vertical displacement "u_{inst}" can be expressed

$$u_{inst} = u^{F_{d,r}}(E_{cm}, E_{0,mean}, k_u)$$

Where $F_{d,r}$ designates the rare load combination

4.5 Long term verification

Creep and shrinkage

A composite system must fulfill the requirements for the whole lifespan of the building. Because of this one must consider the long-term effect, as the short-term is not sufficient. A structure may be influenced by internal forces and deformation over time. The most important effects are creep and shrinkage. When it comes to the creep of the material it is because of the load on the composite that deforms it over time. The deformation will increase over time and is called creep deformation. Shrinkage or swelling of the material can be caused by several things. One is that if the composite is hardening by absorbing or emitting moisture, the volume of the cross-section changes. When it comes to hardening, the composite changes the elements in the system and its volume, which reduces the volume, and the cross section shrinks. The composite would increase its volume after added water. Again, if moisture were emitted out of the system, the cross section would shrink [12, p.106]:

The effects of creep deformation in a composite system [12, p.106]**:**

Deformation: When one of components in the composite deforms due to creep, the deformation increases for the whole composite. When the composite system has a longer span, around 5 meters, deformation is a decisive verification that must be met.

Internal stresses and forces: The creep strain can be interpreted as a reductio of stiffness. Stiffness is essential for the distribution of loads in a statically undetermined system and creep strain can do just that. The larger the difference between the creep coefficients of the components, the larger load distribution difference becomes. When one component has stronger creep than the other, it results with the stronger one reducing its load. Because of the equilibrium of the forces, the less creeping component will add load. In addition, the normal forces that get affected by the creep strain. The less creeping component will increase its bending moment and the normal forces will decrease. This results in an increase in stresses on the less creeping component.

Effect of shrinkage in a composite system [12, p.107]:

Deformation: When the concrete shortens and gets blocked by the connectors, an internal bending moment arises, and the deflection increases. The deflection due to shrinkage of concrete can be compared to the deformation due to a dead load and should not be neglected.

Internal forces: If the concrete shrinks, the normal forces will decrease, and the bending moment will increase. Then the stresses on the timber component increase because of the increased bending moment.

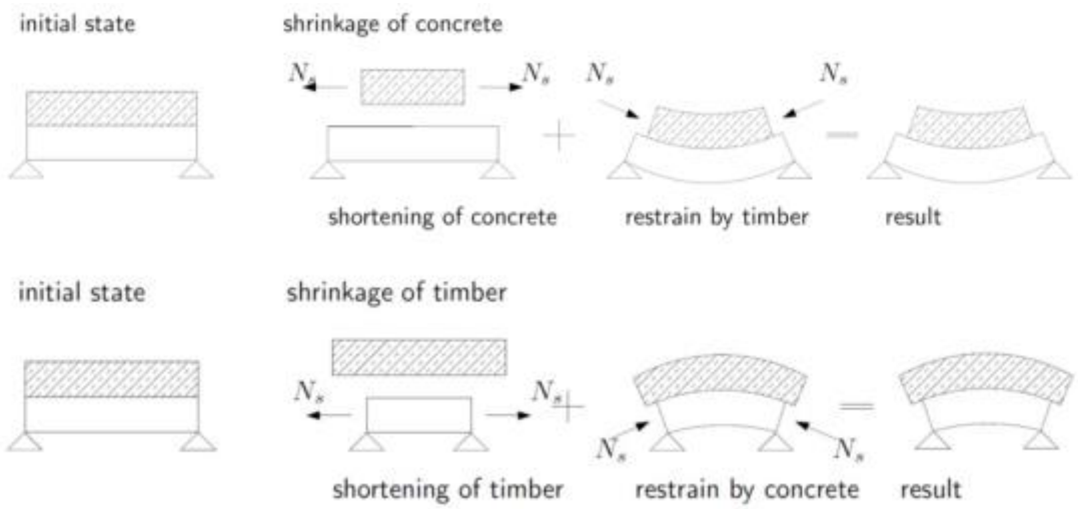

Figure 4.7 Shrinkage outcomes [12, p.75]

The verification of the composite system is utterly problematic, one must take into consideration the creep and shrinkage of the concrete. Also, the effects of the timber on the connections and the thermal strain of concrete and timber. Numerical programs and analytical formulas have been proposed, but no consensus research has reached an accurate method for the prediction of the long-term effect of a TCC [10, p.45].

A simplified approach by Ceccotti (2002) that does not account for shrinkage or thermal strains based on the Effective Modulus Method has been suggested. The creep and mechano-sorption of concrete, timber and connection are all accounted for. Here the elastic and slip modulus are expressed with the reduction [10, p.45.46]:

For concrete:

$$E_{c,fin} = \frac{E_{cm}(t_0)}{1 + \phi(t, t_0)}$$

For CLT:

$$E_{t,fin} = \frac{E_{0,mean}}{1 + k_{def,t}}$$

For shear fastener:

$$E_{t,fin} = \frac{E_{0,mean}}{1 + k_{def,t}}$$

Where:

$E_{cm}(t_0)$: The mean value of Young`s modulus for compression of concrete at the time of loading t_0

$\phi(t, t_0)$: The creep coefficient for concrete at a time of loading t_0

$E_{0,mean}$: The mean value of Young`s modulus for tension of timber in the grain direction

$k_{def,t}$: Creep coefficient for timber and fastener at a time t

K: Slip modulus corresponding to the secant value of 60% or 40% of the shear connector load carrying capacity, depending on the limit state.

From the European code the recommendation for long term limit state verification can be expressed as followed [30, p.11]:

Long term verification:
The long term effect in terms of stress "σ_{fin}" can be expressed as

$$\sigma_{fin} = \sigma^{F_{d,p}}(E_{c,fin}, E_{t,fin}, k_{ser,fin}) + \sigma^{F_{d,u}-F_{d,p}}(E_{cm}(t), E_{0,mean}, k_u)$$

Where $F_{d,p}$ designates quasi-permanent load combination and $F_{d,u}$ designates the ultimate limit state load combination .

The long term effect in terms of vertical displacement "u_{fin}" can be expressed as

$$u_{fin} = u^{F_{d,p}}(E_{c,fin}, E_{t,fin}, k_{ser,fin}) + u^{F_{d,r}-F_{d,p}}(E_{cm}(t), E_{0,mean}, k_{ser})$$

Where $F_{d,r}$ designates the rare load combination

5. Load calculations

The purpose of doing load calculations when designing TCC slabs is to ensure that the applied loads during testing are within the fitting range and representative of real-world conditions.

The four-point bending test will be simply supported on both sides and is described in Chapter 8.2. The only loads taken into consideration under the test will be the applied load and the dead load. Safety factors and variable loading are not considered in laboratory testing. The calculation of the characteristic value for dead load is as follows:

$$g_{0,k} = b * h_c * \gamma_c + b * h_t * \gamma_M$$

In this chapter, the maximum applied load for laboratory testing is calculated. Theoretical predictions for the TCC are performed in the following sub-chapters. All load capacity predictions can be found in Appendix A.

5.1 Shear analogy method for CLT elements

From the theory in chapter 4 the shear analogy method includes the shear deformation in the transverse layers for and element with three or more layers. This is done by calculating the effective bending stiffness of the CLT element. The middle layers 2,3,4 have material properties T15, and the outer layers have properties of T22. In the following equation "i" describes the number of layers.

The effective bending stiffness of the CLT element $(EI)_{eff}$ is determined by:

$$(EI)_{eff}) = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

For layer 1, 2, 4 and 5 the area is the same as the thickness of all these layers are the same:

$$A_1 = b * h_1 = 1200\ mm^2$$

$$A_1 = A_2 = A_4 = A_5$$

For layer 3 the area is:

$$A_3 = b * h_3 = 24000\ mm^2$$

The moment of inertia for layers 1, 2, 4 and 5 is also the same:

$$I_1 = \frac{(b * h^3)}{12} = 400000\ mm^4$$

$$I_1 = I_2 = I_4 = I_5$$

For layer 3 the moment of inertia is:

$$I_3 = \frac{(b * h^3)}{12} = 3200000\ mm^4$$

The following equations are for the distance z_i from each layer to the neutral axis in mm:

$$z_1 = \frac{h_1}{2} + h_2 + \frac{h_3}{2} = 50\ \ mm$$

$$z_2 = \frac{h_2}{2} + \frac{h_3}{2} = 30\ mm$$

$$z_3 = 0$$

$$z_4 = \frac{h_3}{2} + \frac{h_4}{2} = 30\ mm$$

$$z_5 = \frac{h_5}{2} + h_4 + \frac{h_3}{2} = 50\ mm$$

Table 5.1 shows the necessary calculations to find the effective bending stiffness. For the modulus of elasticity there is a difference in the grain direction. Longitudinal layers 1, 3 and 5 will use the main values for the modulus of elasticity parallel to the grain, while layers 2 and 4 will use the values perpendicular to the grain as mentioned in Chapter 3.

Table 5.1 Effective bending stiffness calculation

Layer	**$E_{i,mean}$ [MPa]**	**E_i*I_i [MPa]**	**E_i *A_i *z_i^2 [MPa]**
i=1	1300	5200000000	390000000000
i=2	230	92000000	2484000000
i=3	11500	36800000000	0
i=4	230	92000000	2484000000
i=5	1300	5200000000	390000000000
Sum		47384000000	784968000000

From table 5-1 the effective bending stiffness is:

$$EI_{eff} = 8.32352 * 10^{11}$$

The effective bending stiffness lacks consideration of shear deformation in the transverse layers. Therefore, a new formula is derived for an adjusted effective bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Where the effective shear stiffness is as followed:

$$GA_{eff} = \frac{a^2}{[\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i b_i}\right) + (\frac{h_n}{2G_n b})]}$$

Where "a" is the distance between the geometrical center of the two outer layers:

$$a = \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2} = 100\ mm$$

Table 5.2 Effective shear stiffness calculation

Layer	**G_i [MPa]**	**H_i/G_i**
i=1	810	0.02469136
i=2	72	0.27777778
i=3	720	0.05555556
i=4	72	0.27777778
i=5	810	0.02469136

Effective shear stiffness:

$$GA_{eff} = 9.436893204 * 10^6$$

The effective bending stiffness lacks consideration of shear deformation in the transverse layers. Therefore, a new formula is derived for an adjusted effective bending stiffness.

$$EI_{app} = 5.737154677 * 10^{11}$$

The modulus of elasticity for the CLT element:

$$E_{CLT} = \frac{EI_{app}}{\frac{bh_{CLT}^3}{12}} = 6640.225321\ MPa$$

5.2 Load capacity calculations using short-term verification for type A

After finding the modulus of elasticity, it's time to find the maximum load applied and verify it in accordance with standards.

Modulus of elasticity for the concrete element is:

$$E_c = 3400\ MPa$$

Modulus of elasticity for the CLT element is:

$$E_{CLT} = 6640.2\ MPa$$

For CTC screw the calculation for the slip modulus and spacing have been described in chapter 3:

$$K_u = \frac{2}{3} * 23100 = 15400\ \frac{N}{mm}$$

Where minimum and maximum spacing have been found and from that the following spacing is chosen:

S = 200mm

From the total of calculations, the effective bending stiffness can now be found by the γ - method [23]. The system will be considered of two elements, one being the CLT-element and the other one being the concrete element. From the calculation the γ -factor is showing the composite action of the whole system. Where 0 being no composition, and 1 being fully composite.

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1 S}{kL^2}} = 0.01064160141$$

$$\gamma_2 = 1$$

The distance from the neutral axis to the center of the i-layer is determined by:

$$a_2 = \frac{\gamma E_1 A_1 (h_1 + h_2)}{2\gamma E_1 A_1 + E_2 A_2} = 3.50222874\, mm$$

$$a_1 = \frac{(h_1 + h_2)}{2} - a_2 = 96.49477713\, mm$$

From this the effective bending stiffness can be determined in accordance with [23].

$$(EI)_{eff,tot} = E_1 I_1 + \gamma E_1 A_1 a_1^2 + E_2 I_2 + E_2 A_2 a_2^2 = 1.611698850 * 10^{12}\ Nmm^2$$

From these calculations the maximum applied load can be found. This is done by determining the moments on the top and bottom of the CLT-element and the concrete element. This is done by using the formulas for normal stresses for the γ-method.

Normal stresses top part of the concrete:

$$\sigma_{c,t} = -\sigma - \sigma_{m,1} = \frac{f_{ck}}{\gamma_c}$$

Moment top part of the concrete:

$$M_1 = \frac{f_{ck}}{\gamma_c(\frac{\gamma_1 E_1 a_1}{EI_{eff,tot}} + \frac{0.5E_1 h_1}{EI_{eff,tot}})} = 26.9596032\ kNm$$

Normal stresses bottom part of the concrete:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1} = \frac{f_{ctk,0.05}}{\gamma_c}$$

The moment for the bottom part of the concrete:

$$M_2 = \frac{f_{ctk,0.05}}{\gamma_c(-\frac{\gamma_1 E_1 a_1}{EI_{eff,tot}} + \frac{0.5E_1 h_1}{EI_{eff,tot}})} = 17.83901997\ kNm$$

Normal stresses top part of the CLT-element:

$$\sigma_{t,t} = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} \leq 1.0$$

Where the values for $f_{t,0,d}$ and $f_{m,d}$ are found by:

$$f_{t,0,d} = \frac{k_{mod} f_{t,0,k,t22}}{\gamma_M}$$

$$f_{m,d} = \frac{k_{mod} f_{m,k,t22}}{\gamma_M}$$

From this the moment at the top of the CLT-element can be found:

$$M_3 = \frac{\frac{k_{mod}}{\gamma_M}}{(\frac{\gamma_2 E_2 a_2}{EI_{eff,tot} f_{t,0,k,t22}} + \frac{0.5 E_2 h_2}{EI_{eff,tot} f_{m,k,t22}})} = 79.39979454\ kNm$$

Normal stresses bottom part of the CLT-element:

$$M_4 = \frac{\frac{k_{mod}}{\gamma_M}}{(-\frac{\gamma_2 E_2 a_2}{EI_{eff,tot} f_{t,0,k,t22}} + \frac{0.5 E_2 h_2}{EI_{eff,tot} f_{m,k,t22}})} = 93.39475180\ kNm$$

The bending moment for the bottom part of the concrete is neglected, the value is considerably small. Now the maximum moment (design moment) of the system can be determined:

$$M_{Ed} = \min[M_1, M_3, M_4] = 26.9596032\ kNm$$

The maximum applied load from the four-point bending test can be determined:

$$P_{Ed} = 175.4620583\ kN$$

After determining the maximum applied load, it is necessary to verify both the CLT-element and concrete on the top and bottom to ensure that they can withstand the maximum applied load.

Stresses in the concrete element:

$$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_{Ed}}{EI_{eff,tot}} = 0.5840086937\ MPa$$

$$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_{Ed}}{EI_{eff,tot}} = 22.74932464\ MPa$$

Normal stresses top part of the concrete:

$$\sigma_{c,t} = -\sigma_1 - \sigma_{m,1} = -23.33333333\ MPa$$

Normal stresses bottom part of the concrete:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1} = 22.16531595\ MPa$$

Verification of the top part of the concrete:

$$Ver_{top,c} = \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}} \leq 1.0$$

$$Ver_{top,c} = -0.999999999$$

Verification of the bottom part of the concrete:

$$Ver_{bottom,c} = \frac{\sigma_{c,b}}{\frac{f_{ctk,0.05,c}}{\gamma_c}} \leq 1.0$$

$$Ver_{bottom,c} = 15.11271542 \geq 1\ NOT\ OK$$

The bottom part of the concrete section does not satisfy the verification that has been calculated. Because of this a modified calculation has been made for the effective compressive height of the concrete. This adjustment, to the effective bending stiffness of the bottom part of the concrete can be verified.

The γ-factor will remain the same and adjustments will be made for a_1 and a_2. This means that the distance from the Neutral Axis to the center of the i-layer will be changed.

Quadratic equation:

$$a_{1,eff} = a_1^2(4\gamma_1^2 E_1 b) + a_1[2E_2A_2(1+\gamma_1)] + E_2A_2(2h_1 + h_2) = 138.3428530\ mm$$

Effective compressed height of the concrete:

$$x = 2\gamma_1 a_{1,eff} = 2.944379\ mm$$

From this the new modified values can be found and helps with calculating a new effective bending stiffness:

$$a_{2,new} = h_1 - 0.5x + 0.5h_2 - a_{1,eff} = 0.1849575\ mm$$

$$A_{1,eff} = bx = 1766.6274\ mm^2$$

$$I_{1,eff} = bx^3 = 1276.29509\ mm^2$$

The new modified effective bending stiffness can be found by:

$$EI_{eff,tot} = E_1I_{1,eff} + \gamma_1E_1A_{1,eff}a_{1,eff}^2 + E_2I_2 + \gamma_2E_2A_2a_{2,new}^2$$

$$EI_{eff,tot} = 5.860085367 * 10^{11}\ Nmm^2$$

Now with the help of the new modified effective bending stiffness the CLT-element and concrete can be confirmed if they satisfy the conditions.

Normal stresses in the concrete element:

$$\sigma_1 = \frac{\gamma_1 E_1 a_{1,eff} M_{Ed}}{EI_{eff,tot}} = 2.302778606\ MPa$$

$$\sigma_{m,1} = \frac{0.5 E_1 x M_{Ed}}{EI_{eff,tot}} = 2.302778606\ MPa$$

Normal stresses top part of the concrete:

$$\sigma_{c,t} = -\sigma_1 - \sigma_{m,1}$$

$$\sigma_{c,t} = -4.605557212 MPa$$

Stresses on the bottom part of the concrete:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1}$$

$$\sigma_{c,b} = 0$$

Verification of the top part of the concrete:

$$Ver_{top,c} = \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}} < 1.0$$

$$Ver_{top,c} = -0.1973810234$$

Verification of the bottom part of the concrete:

$$Ver_{botton,c} = \frac{\sigma_{c,b}}{\frac{f_{ctk,0.05}}{\gamma_c}} < 1.0$$

$$Ver_{botton,c} = 0$$

Normal stresses in the CLT element:

$$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_{Ed}}{EI_{eff,tot}} = 0.05650206445\ MPa$$

$$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_{Ed}}{EI_{eff,tot}} = 18.32920464\ MPa$$

Normal stresses on top of the CLT element:

$$\sigma_{t,t} = -\sigma_2 - \sigma_{m,2} = 18.38570670\ MPa$$

Normal stresses on the bottom of the CLT element:

$$\sigma_{b,t} = -\sigma_2 + \sigma_{m,2} = 18.27270258\ MPa$$

Verification of the stresses in the CLT element:

$$Ver_{timber} = (\frac{\sigma_{t,t}}{\frac{k_{mod}f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{mod}f_{m,k,t22}}{\gamma_M}}) < 1.0$$

$$Ver_{timber} = -0.3401253489$$

Shear stress in the CLT element:

$$\tau_2 = \frac{0.5E_2b(0.5h_2 + a_2)^2}{bEI_{eff,tot}}P_{Ed} = \ 3.600874736\ MPa$$

Verification of the shear stress in the timber element:

$$Ver_{shear} = \frac{\tau_2}{\frac{k_{mod}f_{v,k,t22}}{\gamma_M}} < 1.0$$

$$Ver_{shear} = 1.294064358\ \geq 1\ NOT\ OK$$

The verification was not ok which might indicate failure due to shear stress in the timber element.

Load per shear fastener can:

$$F_1 = \frac{\gamma_1E_1A_{1,eff}a_{1,eff}}{EI_{eff,tot}}P_{Ed} = \ 5.295376788kN$$

Verification of the shear fastener:

$$Ver_{F_1} = \frac{F_1}{3\frac{k_{mod}f_{tens,k}}{\gamma_M}} < 1.0$$

$$Ver_{F_1} = 0.1268684022$$

5.3 Load capacity calculations using long-term verification of type A

The long-term calculation introduces a new modulus of elasticity for both CLT and concrete element, and new slip modulus of the shear fasteners. As discussed in chapter 4, the creep and shrinkage of concrete and timber.

The modulus of elasticity for the CLT element:

$$E_2 = \frac{E_{CLT}}{1 + k_{def}} = 3589.310984\ MPa$$

The modulus of elasticity for the concrete element:

$$E_1 = \frac{E_{cm,c}}{1 + \varphi_c} = 9714.285714\ MPa$$

Slip modulus for the shear fasteners:

$$K_{ser,g} = \frac{K_{ser}}{1 + k_{def}} = 12486.48649\ MPa$$

$$K_u = \frac{2}{3} K_{ser,g} = 8324.324327\ MPa$$

Again, the bottom part of the concrete does not satisfy the verification. As a result, the effective compressive height of the concrete is considered. This can be found in Appendix A.1. Table 5.3 show the new parameters:

Table 5.3 Adjusted design load and moment

Variable	Value
γ_1	0.0199434728
γ_2	1
$a_{1,eff}$	136.9326803 mm
a_2	0.3364065 mm
X	5.461826370 mm
A_{eff}	3277.095822
I_{eff}	8146.736585
$EI_{eff,tot}$	3.221294607 * 10^{11} Nmm^2
M_{Ed}	37.15746148 kNm
P_{Ed}	243.4477801 kN

From the obtained results, it is possible to verify the top and bottom parts of the CLT and concrete element.

Normal stresses in the concrete element:

$$\sigma_1 = \frac{\gamma_1 E_1 a_{1,eff} M_{Ed}}{EI_{eff,tot}} = 3.06009111\ MPa$$

$$\sigma_{m,1} = \frac{0.5 E_1 x M_{Ed}}{EI_{eff,tot}} = 3.060091112\ MPa$$

Normal stresses in the top part of the concrete element:

$$\sigma_{c,t} = -\sigma - \sigma_{m,1} = -6.120182222\ MPa$$

Normal in the bottom part of the concrete:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1} = 2 * 10^{-9}\ MPa$$

Verification for the top part of the concrete:

$$Ver_{top,c} = \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}} < 1.0$$

$$Ver_{top,c} = -0.5574132030$$

Verification for the bottom part of the concrete:

$$Ver_{bottom,c} = \frac{\sigma_{c,b}}{\frac{f_{ctk,0.05,c}}{\gamma_c}} < 1.0$$

$$Ver_{bottom,c} = -6.81818181810^{-10}$$

Stresses in the CLT element:

$$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_{Ed}}{EI_{eff,tot}} = 0.1392807373\ MPa$$

$$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_{Ed}}{EI_{eff,tot}} = 24.84150645\ MPa$$

Stresses on top of the CLT element:

$$\sigma_{t,t} = -\sigma_2 - \sigma_{m,2} = -24.98078719\ MPa$$

Stresses on the bottom of the CLT element:

$$\sigma_{b,t} = -\sigma_2 + \sigma_{m,2} = 24.70222571\ MPa$$

Verification of the stresses on the CLT element:

$$Ver_{timber} = (\frac{\sigma_{t,t}}{\frac{k_{mod} f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{mod} f_{m,k,t22}}{\gamma_M}}) < 1.0$$

$$Ver_{timber} = -0.468023100$$

Shear stress in the CLT element:

$$\tau_2 = \frac{0.5 E_2 b (0.5 h_2 + a_2)^2}{b E I_{eff,tot}} P_{Ed} = 4.937593668\ MPa$$

Verification of the shear stress in the CLT element:

$$Ver_{shear} = \frac{\tau_2}{\frac{k_{mod} f_{v,k,t22}}{\gamma_M}} < 1.0$$

$$Ver_{shear} = 1.774447724\ \geq 1\ NOT\ OK$$

Again, the shear stress verification is not satisfied.

Verification of the shear fasteners:

$$Ver_{F_1} = \frac{F_1}{3\frac{k_{mod}f_{tens,k}}{\gamma_M}} < 1.0$$

$$Ver_{F_1} = 0.3148254833$$

Load per shear fastener:

$$F_1 = \frac{\gamma_1 E_1 A_{1,eff} a_{1,eff}}{EI_{eff,tot}} P_{Ed} = 13.14054191\ kN$$

5.4 Maximum deflection short-term verification based on SLS for type A

Verification of the SLS has been described in Chapter 4. The maximum load calculations are the same as for ULS. In this chapter the results of the deflection came out. The biggest change is not using the K_u and having K_{ser} as the slip modulus without any changes which gives some differences in the maximum load applied. There will also be no consideration of the effect of the compressive height of the concrete, and the Gamma method will be the only one used. Meaning the quadratic method will not be applied.

The modulus of elasticity for the concrete and CLT element is the same as before the change is K_{ser} the slip modulus will not be changed according to Eurocode 5 [23, clause 2.2.2(2)] because it is only used for ULS:

$$K_{ser} = 3 * 70 * l_{eff} = 23100\ \frac{N}{mm}$$

Because of the change in the slip modulus due to SLS the parameters for the effective bending stiffness will change, table 5.4.

Table 5.4 Adjusted effective bending stiffness

Variable	Value
γ_1	0.01587791888
γ_2	1
a_2	94.85867032 mm
a_2	5.141329679 mm
$EI_{eff,tot}$	$1.689920498*10^{11}$ N/mm^2
P_{Ed}	182.0088465 kNm
$F_{d,sls}$	1.517734993 kN/m

Then the vertical deflection can be calculated using:

$$\omega = \frac{5\left(\frac{P_{Ed}}{L} + f_{d,SLS}\right) * L^4}{384EI_{eff,tot}} = 4.792226413\ mm$$

The limit for the short-term verification is:

$$\omega_{lim} = \frac{L}{250} = 6\ mm$$

Verification of the vertical deflection:

$$Ver_{deflection} = \frac{w}{w_{lim}} < 1.0$$

$$Ver_{deflection} = 0.7987044022\ mm$$

5.5 Maximum deflection long-term verification based on SLS for type A

Some modification for the long-term verification have been made, mostly because of the creep and shrinkage in the concrete. The modulus of elasticity will change. There will also be a new slip modulus for the shear fasteners.

New modulus of elasticity for CLT:

$$E_2 = \frac{E_{CLT}}{1 + k_{def}} = 3589.310984 \ MPa$$

New modulus of elasticity for concrete:

$$E_1 = \frac{E_{cm,c}}{1 + \varphi_c} = 9714.285714 \ MPa$$

New slip modulus for shear fasteners:

$$K_{ser,g} = \frac{K_{ser}}{1 + k_{def}} = \ 12486.48649 \ \frac{N}{mm}$$

New parameters for the effective bending stiffness and deflection have been calculated table 5-5.

Table 5.5 Adjusted effective bending stiffness

Variable	Value
γ_1	0.02961984787
γ_2	1
a_2	94.92681989 mm
a_2	5.073180113 mm
$EI_{eff,tot}$	$6.899085749*10^{11}$ Nmm^2
P_{Ed}	253.7811787 kN
$F_{d,sls}$	1.517734993 kN/m

Vertical deflection is calculated as:

$$\omega = \frac{5\left(\frac{P_{Ed}}{L} + f_{d,SLS}\right) * L^4}{384EI_{eff,tot}} = \ 16.31018894\ mm$$

The limit for long-term deflection:

$$\omega_{lim} = \frac{L}{150} = 10\ mm$$

The verification of the vertical deflection:

$$Ver_{deflection} = \frac{\omega}{\omega_{lim}} < 1.0$$

$$Ver_{deflection} = 1.600025884\ mm\ \geq 1\ NOT\ OK$$

6. CLT-concrete slab preparation

Figure 6.1 15 CLT slabs received

After the CLT panels were received, an inspection was conducted to observe if there was any damage that could affect the load capacity testing. The panels were checked for cracks, knots, voids, and other minor damage, and any issues identified were documented for the experiment. The thickness of the slabs was measured to ensure that they were within the specified tolerances and to verify that the dimensions of the slabs were consistent with the specifications provided. The panels were found to be slightly undersized, with a difference between the ordered width and the received width being 3-4 mm shorter. A small variance in the thickness of the layers was observed. Table 6.1 shows the dimension measured where L1 is the thickness of the bottom layer etc.

Table 6.1 More accurate dimensions of the panels

Specimen	Length	Width	L1	L2	L3	L4	L5	Spacing	Orientation
A1	160.00	59.60	1.90	2.00	4.00	2.10	2.00	200	45
A2	159.80	59.70	2.90	2.10	2.90	2.10	2.10	200	45
A3	160.00	59.60	2.00	2.10	4.10	2.10	1.90	200	45
B1	160.90	59.70	2.00	2.10	4.10	2.10	1.90	250	45
B3	160.00	59.80	2.00	2.00	4.10	2.10	1.90	250	45
B2	161.00	59.90	2.10	2.10	4.00	2.10	1.90	250	45
C1	162.00	59.60	1.90	2.10	4.10	2.10	1.90	200	90
C2	160.00	59.60	1.90	2.10	4.10	2.10	1.90	200	90
C3	160.10	59.60	1.90	2.10	4.10	2.10	1.90	200	90
D1	160.00	59.60	1.90	2.10	4.10	2.10	1.90	250	90
D2	160.00	59.60	2.10	2.00	4.10	2.10	1.90	250	90
D3	160.20	59.40	2.00	2.10	4.10	2.10	1.90	250	90
E1	160.00	59.60	2.10	2.10	4.10	2.10	1.90	125	90
E2	160.20	59.60	2.00	2.10	4.10	2.10	1.90	125	90
E3	160.00	59.60	2.10	2.10	4.10	2.00	1.90	125	90

Figure 6.2 Crack observed on A2.

To provide more accurate guidance for screw installation, AutoCAD software was used to draw precise points and dimensions for the screw locations on the CLT panels. These drawings were then used to create a cardboard template that matched the exact dimensions and screw hole locations on the panels. The cardboard template was then carefully positioned on the CLT panel and then the screw holes were marked onto the CLT panel by spraying paint over the template onto the panel. This provided a clear indication of where the screws should be installed to ensure that they were in the correct location.

By using AutoCAD to create precise drawings and then transferring those dimensions onto a cardboard template and using that template to guide the installation of the screws, the accuracy of the screw placement was improved.

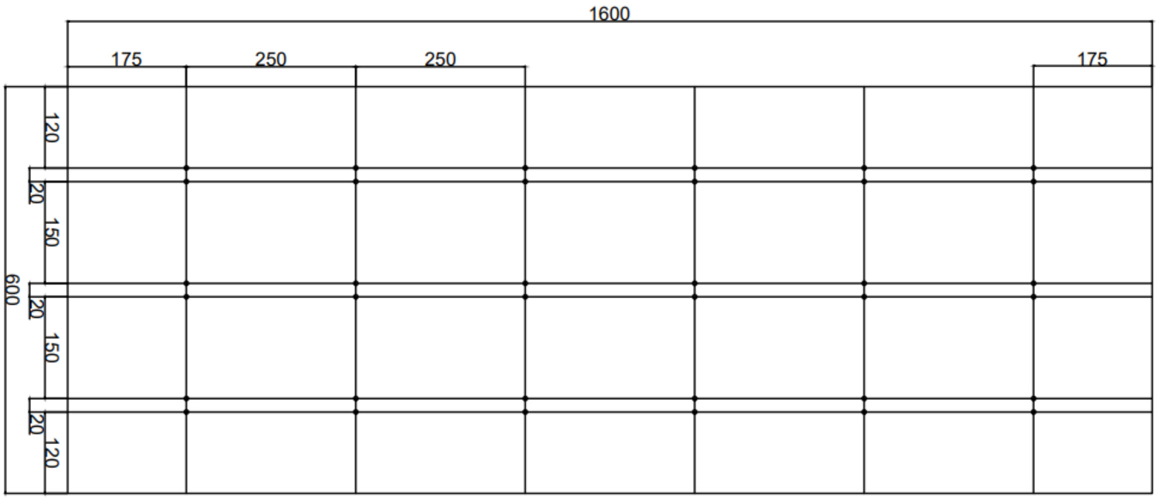

Figure 6.3 Autocad template

Figure 6.4 Preparing marks before installing screws

The screw installation involved drilling the screws in the market locations. For the 90° pattern, the screws were installed straight down on the panels. But for the 45° specimens, the installation was more complex as the screws needed to be installed at an angel and cross over each other. To ensure more accurate installation, a jig was used to guide the drill and ensure that the screws were installed at the correct angle.

Figure 6.5 The jig used to get the right angel

Figure 6.6 Fastening the CTC screws into the CLT slab (screws was installed further inn)

Figure 6.7 CTC installed in two different angles

After the end of screw installation, the next step was to move onto the formwork phase. Formwork is the temporary structure used to support and shape the newly poured concrete until it has cured. Ensuring that the concrete element is in the specific length, width and height used in the test. In general formwork provides a safe working environment for workers by creating a barrier between the workers and the concrete. It also prevents the concrete from spilling a lot into the surrounding area. The benefits of using plywood as formwork is that it’s easy to handle, light and strong enough to support our CLT-concrete slabs.

Given the variance in dimensions of the CLT slabs received. Data in table 6.1 was used to cut 60 different types of plywood boards to fit all sides of the 15 specimens. Panel saw machines were used to cut the correct lengths and widths of the boards. The plywood boards were later placed on the sides of the CLT with timber screws as fasteners. This was constructed to create a level surface and to prevent the concrete from spilling over the edges of the panel.

Figure 6.8 Panel saw machine

Steel net was laid down on top of the CLT panels. The steel net was cut to the suitable length using a bolt cutter, plastic rebar spacers were used to keep the reinforcement in place. The purpose of using steel net in this situation is to reinforce the concrete and distribute the load more evenly across the surface of the CLT panels.

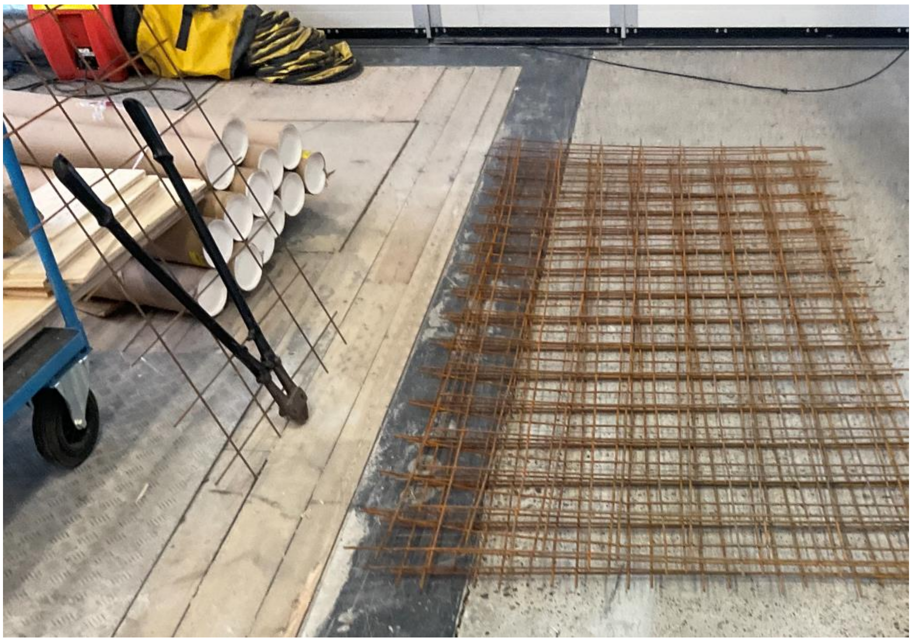

Figure 6.9 Steel mesh and bolt cutter

Figure 6.10 The specimens ready for casting of concrete

Before the concrete was delivered, the site is prepared. This include making sure the area where the concrete will be poured is ready and the steel net is located. The concrete supplier was contacted and came with 1500 L of concrete. The minimum required amount of concrete was calculated beforehand:

$$L * b * h_c * 1000 * number\ of\ specimens$$

$$1.6m * 0.6m * 0.08m * \frac{1000L}{m^3} * 15 = 1152\ L$$

Figure 6.11 Pouring concrete

Once the concrete car arrived it was ready to be poured into the specimens. Wood plank is for spreading of the concrete and leveling the surface of the slab. Plastic cover was initially placed over the concrete, but it was removed after 3 days due to the danger of dust inside of the laboratory. To ensure the proper curing and hydration of the concrete, it was regularly watered in the following days. Five days were waited before removing the formwork to ensure that the concrete has reached a necessary strength level and to prevent damage to the slab. Once the concrete was cured, the formwork was removed, and a smooth surface that is aesthetically pleasing is achieved.

Figure 6.12 After concrete is poured

Figure 6.13 Plastic cover

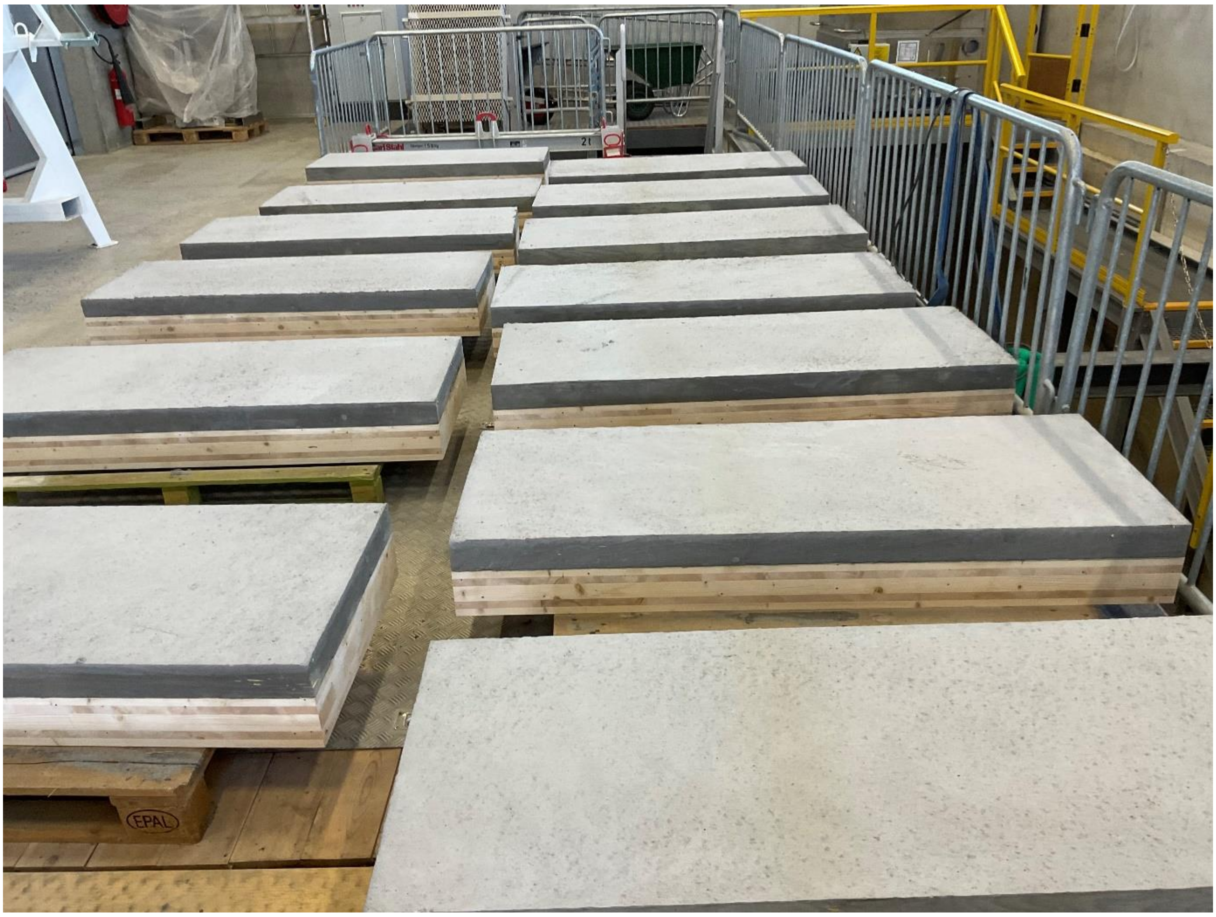

Figure 6.14 The CLT-concrete slabs 7 days after couring

7. Laboratory test preparation

7.1 Four-point bending test

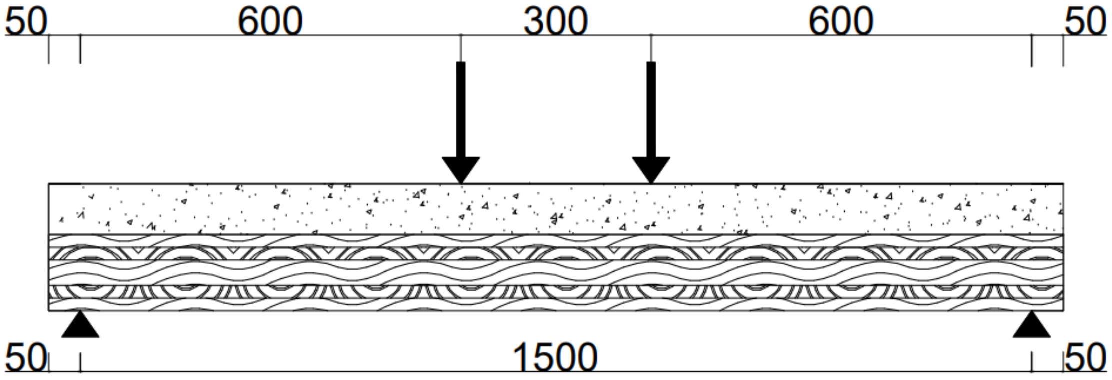

Figure 7.1 Four-point bending test cross section, Autocad template

The test machine and setup format for the test are all done in accordance with NS-ISO 6891:1991 [31]. The numerical values used in the setup of the four-point bending test are shown in table 7.1. The test is used to find the maximum load that the composite can withstand and to measure the displacement of both timber and concrete separately. The load procedure contains one cycle where pre-loading is applied and then continuously loaded until failure. The estimated failure load F_{est} is found in the ULS load calculations appendix A. where the estimated load is the same as the long-term maximum loading in the calculations. The load rate can be found assuming that reaching failure takes 10 minutes [31]:

$$Load\ rate = \frac{F_{est}}{10\ minutes}$$

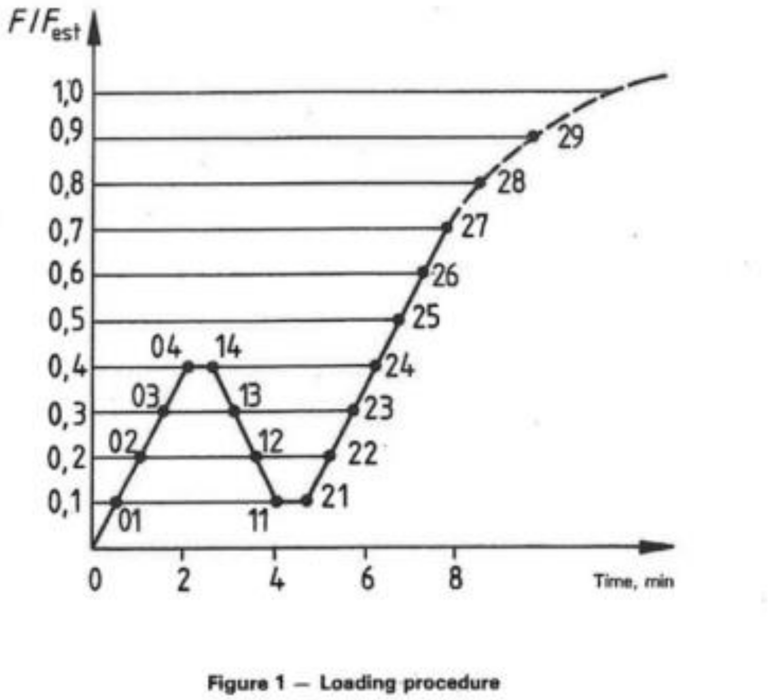

Figure 7.2 Loading procedure [31, clause 8.4]

Figure 7.2 describes the loading procedure in stages. First, the cycle speed is used to go from stage 0 until it reaches stage 4, which is the upper 40% of F_{est}. From stage 4 to stage 14 the load is applied continuously for 30 seconds. From stage 14 the load is unloaded to stage 11 where it reaches the lower step, which is 10% of F_{est}. Then the load is applied continuously for 30 seconds until it reaches stage 21. From stage 21 and forward, the load is applied at a constant load rate until ultimate failure. The input for each specimen group can be read in Table 7.1.

Table 7.1 Load procedure for each specimen group

Input	Slab A	Slab B	Slab C	Slab D	Slab E
Cycles	1	1	1	1	1
Cycle speed	0,405 kN/s	0,398 kN/s	0,368 kN/s	0,368 kN/s	0,37 kN/s
Upper step	97,2 kN	95,6 kN	88,4 kN	88,4 kN	89,2 kN
Overall dwell time upper cycle	30 s	30 s	30 s	30 s	30 s
Cycle speed	0,405 kN/s	0,398 kN/s	0,368 kN/s	0,368 kN/s	0,37 kN/s
Lower step	24,3 kN	23,9 kN	22,1 kN	22,1 kN	22,3 kN
Overall dwell time lower cycle	30 s	30 s	30 s	30 s	30 s

The tests were performed using a building material testing program called Toni Technik. The values specified in Table 7.1 were input into the program. Figure 7.3 shows a graphic picture of the cycle with load as the vertical axis and strain as the horizontal. In figure 7.4, one can see the measurements of specimen A1 and the spacing between supports and load applied.

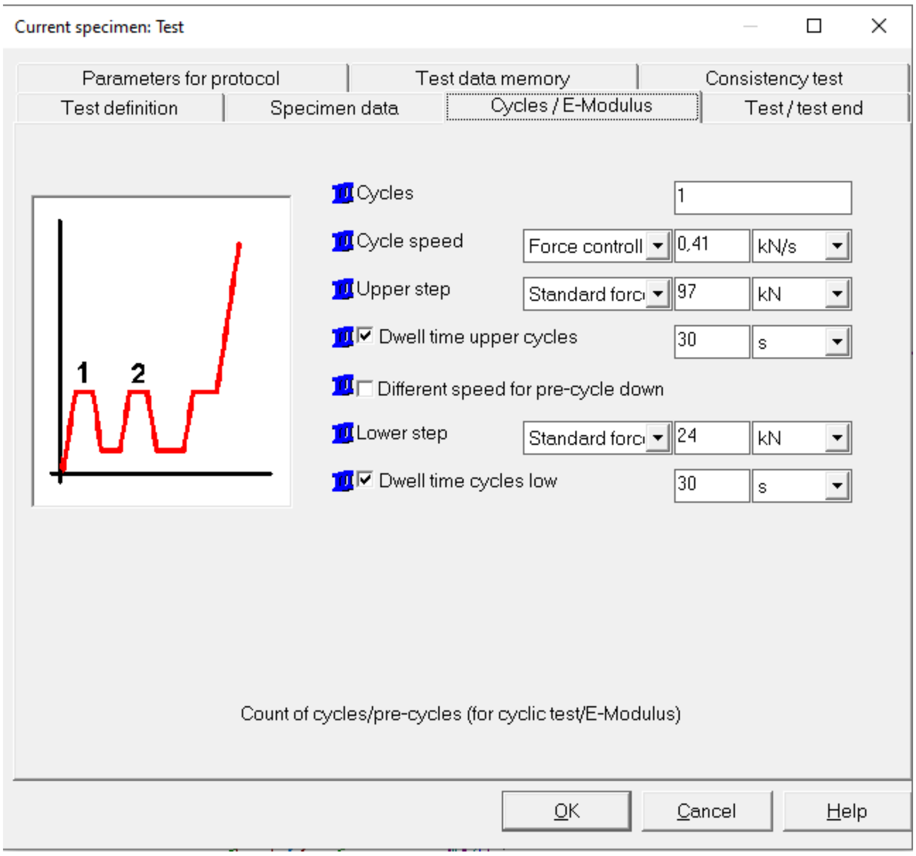

Figure 7.3 Cycle speed, upper step, lower step, and dwell time

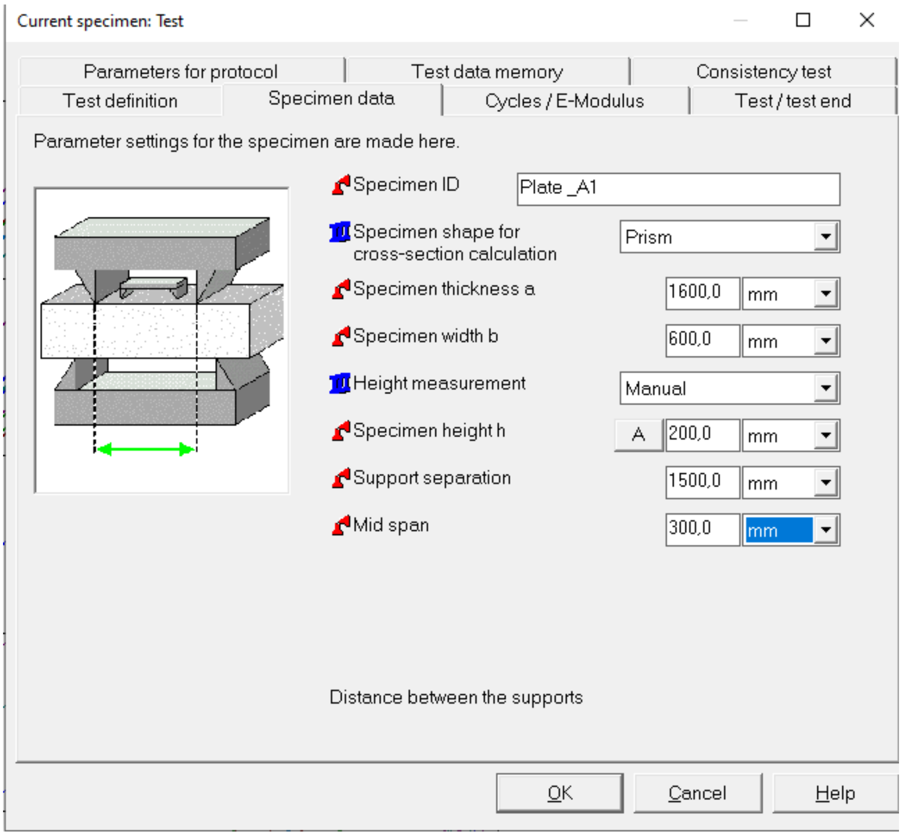

Figure 7.4 Dimensions of the specimen and distance between the supports

Table 7.2 Calculations for the loading procedure

Notation	Type A	Type B	Type C	Type D	Type E
Estimated failure load	243 kN	239 kN	221 kN	221 kN	223 kN
Load rate per minute	24,3 kN/m	23,9 kN/m	22,1 kN/m	22,1 kN/m	22,3 kN/m
Load rate per second	0,405 kN/s	0,398 kN/s	0,368 kN/s	0,368 kN/s	0,372 kN/s
40% of the estimated failure load	97,2 kN	95,6 kN	88,4 kN	88,4 kN	89,2 kN
10% of the estimated failure load	24,3 kN	23,9 kN	22,1 kN	22,1 kN	22,3 kN

The estimated failure load was the same as the long-term maximum load capacity, ULS (Appendix A.).

Example of the calculations (group A)

Long- term estimated failure load: $F_{est,\,A} = 242$ kN

Load rate per minute: $F_{est,\,A}$ / 10 minutes = 24,2 kN/m

Load rate per second: $F_{est,\,A}$ / 600 s = 0,405 kN/s

40% of the estimated failure load: 0,4 * $F_{est,\,A}$ = 97,2 kN

10% of the estimated failure load: 0,1 * $F_{est,\,A}$ = 24,3 kN

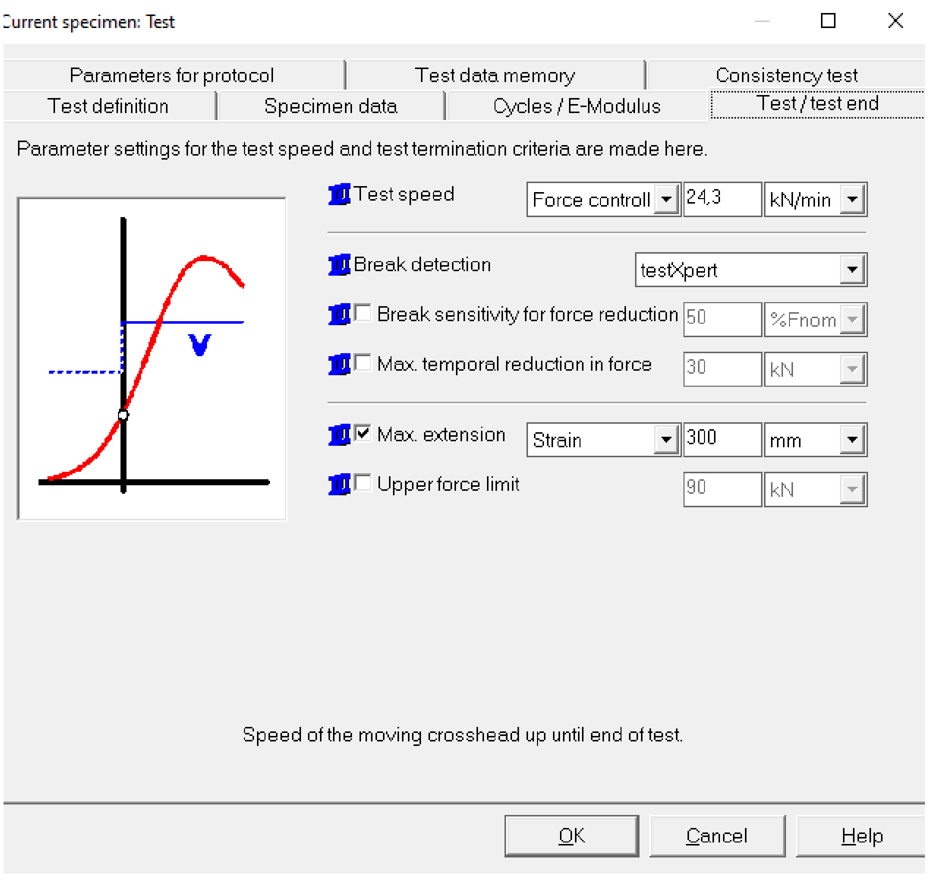

Figure 7.5 Test speed after the cycle in kN/m until failure

7.2 Test setup

To finalize the specimen before testing a grinding machine was used to grind the short edges of the CLT for proper fitting for a “L”- shaped steel profile. This was installed to prevent the crushing of timber under the tests. The "L"-shaped profiles were installed using regular timber screws.

Figure 7.6 Pre-grinding the edges

Figure 7.7 Picture of the installed steel profile

To transport the slabs to the testing machine, a forklift is used along with lifting straps. Firstly, the slab was lifted from the side and lifting straps were positioned under the slab. The straps are placed strategically to ensure optimal weight distribution and secure attachment. This provides additional support during the transportation process. With the slab securely fastened to the forklift using the lifting straps, the forklift then proceeds to transport the slab to the testing machine. The slab is carefully maneuvered to ensure it is accurately aligned with the machine's supports.

Measuring the displacement (LVDT) setup

Linear Variable Differential Transformers (LVDT) were used during the test with a total of four LVDTs. The LVDT is a sensor commonly used in four-point bending tests to accurately measure displacement or deformation. It is carefully set up and positioned to capture the movement of the specimen for precise and reliable measurements of its deflection throughout the testing process. Three LVDTs were placed in the transverse layers to measure the slip between the elements in the composite. The fourth LVDT was positioned beneath the slab to measure the displacement at the location where the load was applied. The arrangement of the LVDTs, along with the load cell used to measure strain, is presented in Table 7.3.

Table 7.3 Arrangement of the LVDT's

Number	Location
1	Load cell
2	Lateral displacement on timber, left
3	Lateral displacement on concrete, left
4	Lateral displacement on concrete, right
5	Vertical displacement under the load

Figure 7.8 LVDT number 2 and 3

Figure 7.9 LVDT number 4

Figure 7.10 LVDT number 5

In addition to the LVDTs, a marker was used to create lines at regular intervals of 200 mm, to see the displacement in the transverse direction. A ruler was then used to measure the displacement between the elements after the test.

Figure 7.11 Lateral displacement lines on the specimen

Just before the test started, rubber pads were placed on top of the concrete surface under the applied load. The purpose of using rubber pads was to spread the applied load across the specimen. To also prevent the concrete from being crushed when the load was exerted on the specimen.

Figure 7.12 The rubber pads used

7.3 Test summary

On March 30th, the curing process for the CLT-concrete composite began, and after 28 days, it was ready for testing. Due to other testing on the machine, the test had to be postponed. The first specimen was eventually tested on May 4th, 35 days after the curing period. The testing method used was a four-point bending test. During the testing process, the L-shaped steel profile was not placed under the CLT and over the supports, which resulted in a mistake. The second mistake was that there were two cycles instead of one cycle. In the end, we did not stop the test in time, and the LVDT that was placed under the load was almost damaged. This also gave a higher load capacity (specimen E3), and it was stopped before the composite broke down completely.

Later, the tests were carried out in the following week, starting on May 8th with one test. On Wednesday, May 10th, four tests were conducted, including the strength test of concrete cubes. The testing pace increased on May 11th with seven tests, concluding with a final test on May 12th. There was consistency in the test, which was very favorable. Sometimes the test was stopped a little too fast because of a conservative approach influenced by the first test (E3). This first test, where adjustments had to be made in terms of decision to stop the machine is shown in figure 7.13.

Figure 7.13 Failure of the first specimen tested (E3)

Table 7.4 Specimen testing date and Curing Period

Specimen	Date of testing	Curing Period
A1	12.05.2023	42 days
A2	11.05.2023	41 days
A3	11.05.2023	41 days
B1	11.05.2023	41 days
B2	11.05.2023	41 days
B3	11.05.2023	41 days
C1	11.05.2023	41 days
C2	11.05.2023	41 days
C3	10.05.2023	41 days
D1	10.05.2023	41 days
D2	10.05.2023	41 days
D3	10.05.2023	41 days
E1	09.05.2023	40 days
E2	08.05.2023	39 days
E3	04.05.2023	35 days

8. Laboratory test results

8.1 Compressive strength

The main reason for doing a compressive test is to measure the quality of the concrete used in this research work. Three cubes 100mmx100mmx100mm were tested with a Toni Technik machine. The cubes were placed in the middle of the Toni Technik machine, which is a type of compression testing machine used to measure the compressive strength of concrete. The machine applies a compressive load to the cube until it fails.

Compressive strength formula:

$$\sigma = \frac{Load}{Area} MPa$$

Minimum characteristic cube strength:

$$f_{c,cube} = 45\ MPa$$

Area of (100mmx100x100) cube:

$$A = 10\ 000\ mm^2$$

Table 8.1Test results for compressive test of concrete cubes

Cube	F [kN]	σ[MPa]
1	542.45 kN	54.25
2	536.77 kN	53.68
3	542.09	54.21
Average compressive strength		54.05

8.2 Four-point bending results

In this chapter, the results from the four-point bending tests will be shown. The tables include the drops that happened under the testing, the load at which they happened, and their corresponding vertical deflection. The maximum load applied, and the corresponding maximum deflection will also be included. This data is taken from Catman, a software used to measure displacement with the help of LVDTs.

The applied load varies between the two programs around 4-10 kN. This is because the Catman software is not totally compatible with Toni Technik. The sign * marks drops that happen before the cycle has occurred, meaning that they occur before 40% of the estimated load is reached.

Table 8.2 Test results for group A

Drops	Load/Deflection	A1	A2	A3
1st drop	Load [kN]	101.816	98,671	100.328
	Deflection [mm]	4.337	5.425	4.506
2nd drop	Load [kN]	106.533	104.367	224.29
	Deflection [mm]	4.922	5.978	15.588
Max	Load [kN]	234.151	244.56	229.032
	Deflection [mm]	19.366	19.023	16.717

Table 8.3 Test results for group B

Drops	Load/Deflection	B1	B2	B3
1st drop *	Load [kN]	88.467*	98.26	83.697*
	Deflection [mm]	3.571*	3.79	3.095*
2nd drop	Load [kN]	111.449	115.505	96.114
	Deflection [mm]	4.984	5.155	4.064
Max	Load [kN]	205.417	245.658	218.658
	Deflection [mm]	17.044	21.803	17.718

Table 8.4 Test results for group C

Drops	Load/Deflection	C1	C2	C3
1st drop	Load [kN]	73.844	80.093*	71.677*
	Deflection [mm]	3.355	3.509*	3.394*
2nd drop	Load [kN]	167.68	82.065*	90.419
	Deflection [mm]	18.08	5.479*	5.881
3rd drop	Load [kN]			164.494
	Deflection [mm]			16.087
Max	Load [kN]	171.688	175.125	166.785
	Deflection [mm]	20.688	16.709	17.876

Table 8.5 Test results for group D

Drops	Load/Deflection	D1	D2	D3
1st drop	Load [kN]	74.29*	83.659*	68.388*
	Deflection [mm]	3.122*	3.959*	3.121*
2nd drop	Load [kN]	87.924	85.413*	76.01*
	Deflection [mm]	6.1	4.885*	4.645*
3rd drop	Load [kN]	173.708		160.229
	Deflection [mm]	18.224		15.172
Max	Load [kN]	176.336	155.375	163.218
	Deflection [mm]	19.756	15.613	17.688

Table 8.6 Test results for group E

Drops	Load/Deflection	E1	E2	E3
1st drop	Load [kN]	74.574*	82.521*	102.891
	Deflection [mm]	3.578*	3.239*	3.281
2nd drop	Load [kN]	85.006*	84.146*	114.628
	Deflection [mm]	5.148*	3.9*	5.086
3rd drop	Load [kN]	190.541		
	Deflection [mm]	20.235		
Max	Load [kN]	194.233	190.905	230.891
	Deflection [mm]	22.709	20.561	19.491

8.3 Graphical representation of the four-point bending test

8.3.1 Load and vertical deflection under the applied load

From the four-point bending test, it was possible to implement the data from the Catman software and show the corresponding failure drops and maximum failure of the specimens as graphs. The figures below show each type and their corresponding graphs. In appendix C one can see the test results of each specimen and get a better picture of each.

Appendix C shows load and vertical deflection response as graphs obtained from Catman. As discussed before, the results of the applied load vary from the data obtained in Catman. The machine is not designed for the measurement of vertical deflection. Therefore, the x-axis is named strain. It measures the displacement of the element while pushing on it, but that does not give the correct values. In addition, the rubber pads give incorrect values as they are also included.

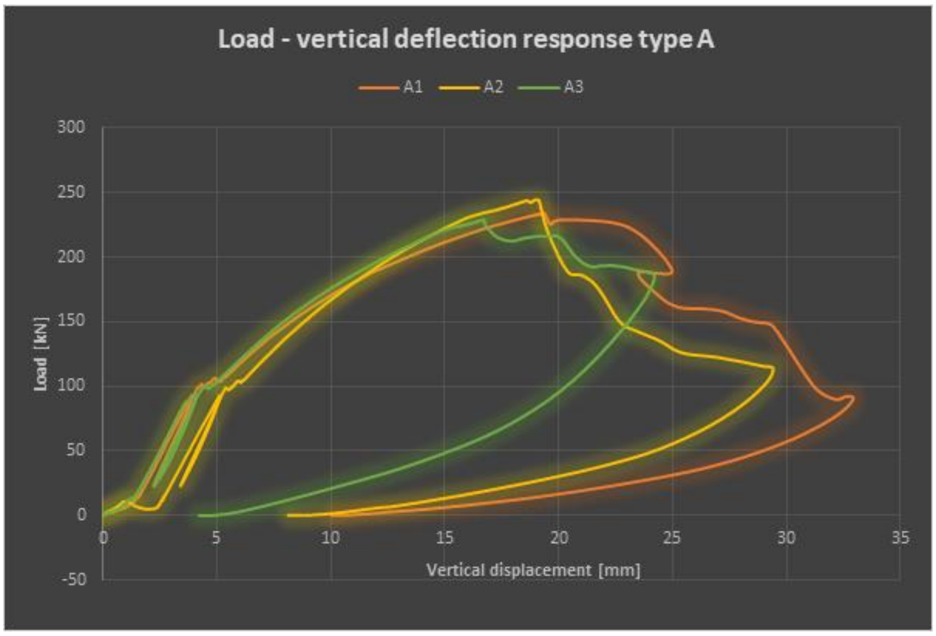

Figure 8.1 Load – vertical deflection response for type A slabs

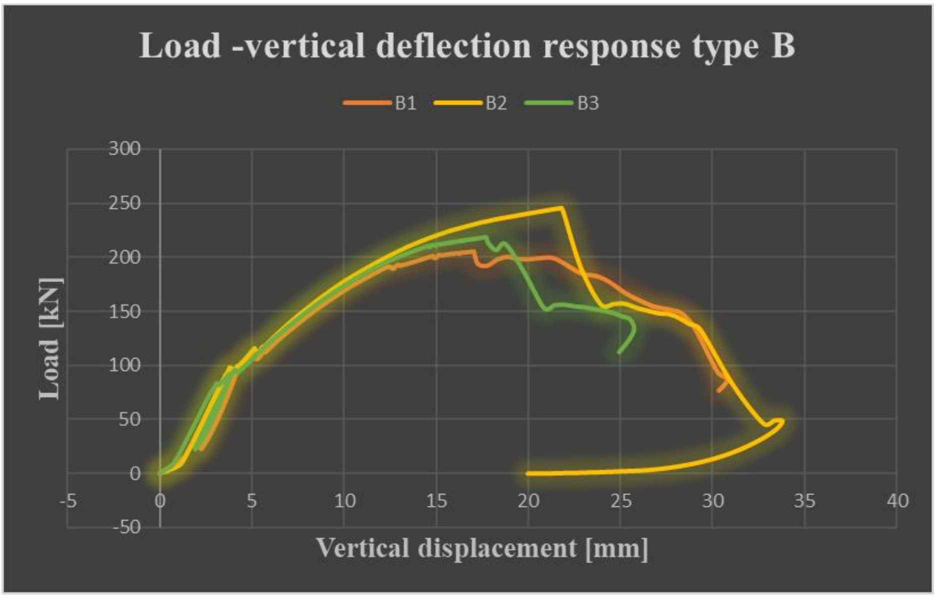

Figure 8.2 Load – vertical deflection response for type B slabs

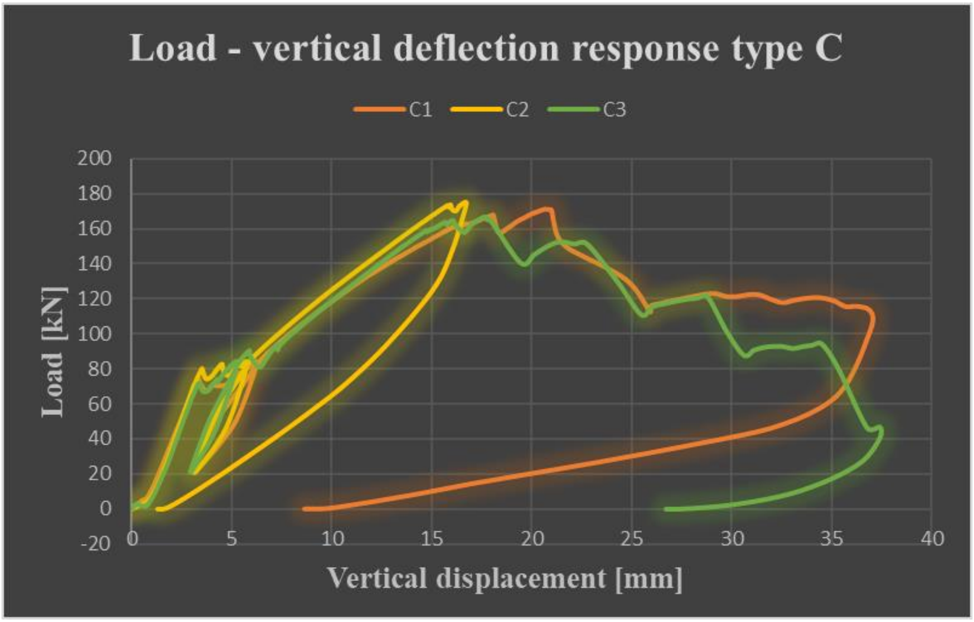

Figure 8.3 Load – vertical deflection response for type C slabs

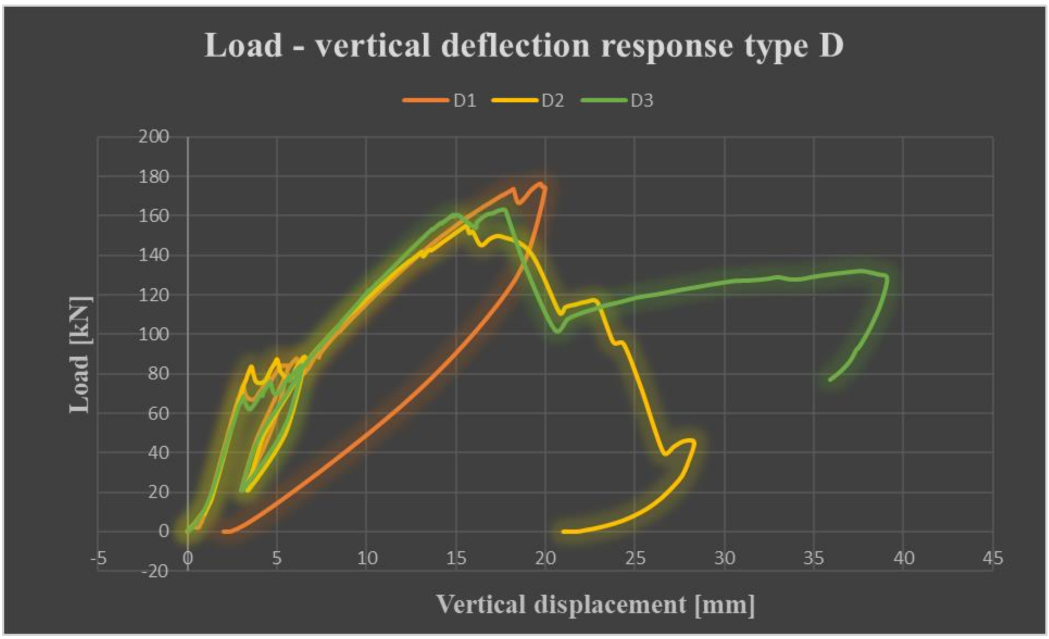

Figure 8.4 Load – vertical deflection response for type D slabs

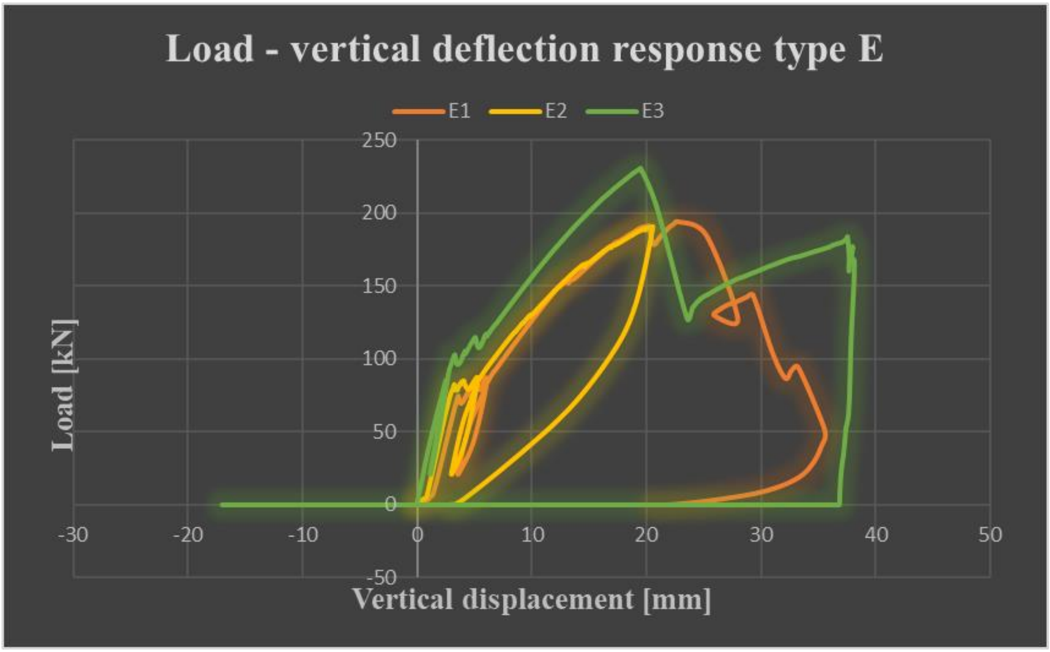

Figure 8.5 Load – vertical deflection response for type E slabs

8.3.2 Load and lateral deflection

In addition to the LVDT that was placed under the applied load, one LVDT was placed on the short side of the specimen measuring the timber element. Two LVDT`s were placed on the short side to measure the concrete element on each side. Figure 8.6 shows the results for slab A1. In Appendix D, one can find load and lateral deflection graphs for each specimen. The graphs show the movement of each element and their response to the loading that is applied.

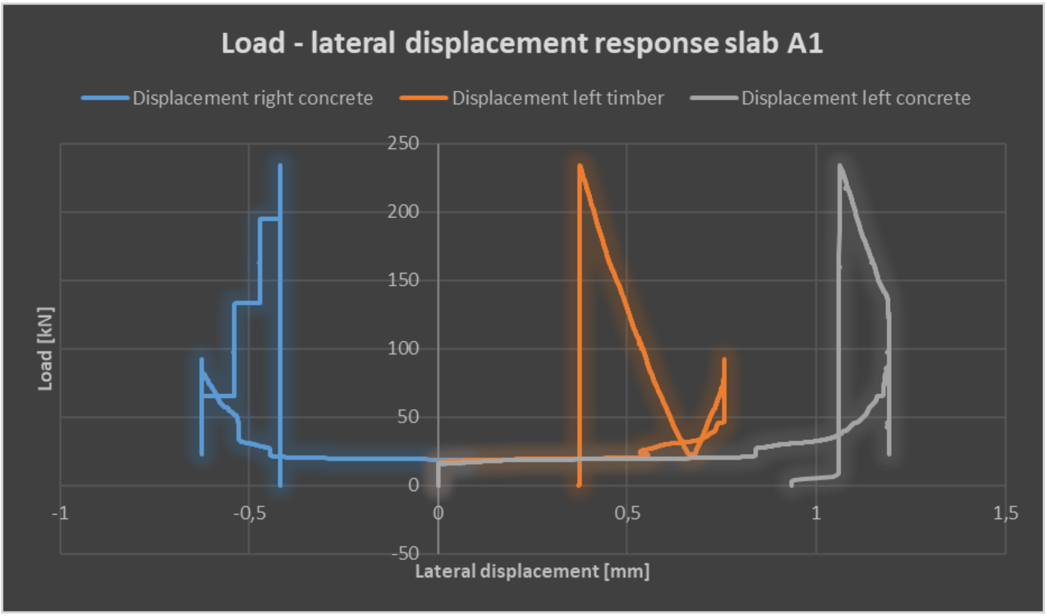

Figure 8.6 Load – lateral deflection response for slab A1

8.4 Failure modes

The failure mode was not equal in all specimens. At first glance, looking at some of the slabs, it was difficult to differentiate the tested specimens from the not-tested ones. Others had some visible cracks on the sides and under the composite.

One of the most common failure modes observed is rolling shear failure where the failure line tends to occur parallel to the applied load. This is because the shear forces acting on the CLT slab cause the individual layers to shear. The modulus of elasticity and shear modulus is much lower perpendicular to the grain than they are parallel to the grain (Chapter 3). This explains why the transverse layers are often the ones that undergo rolling shear failure.

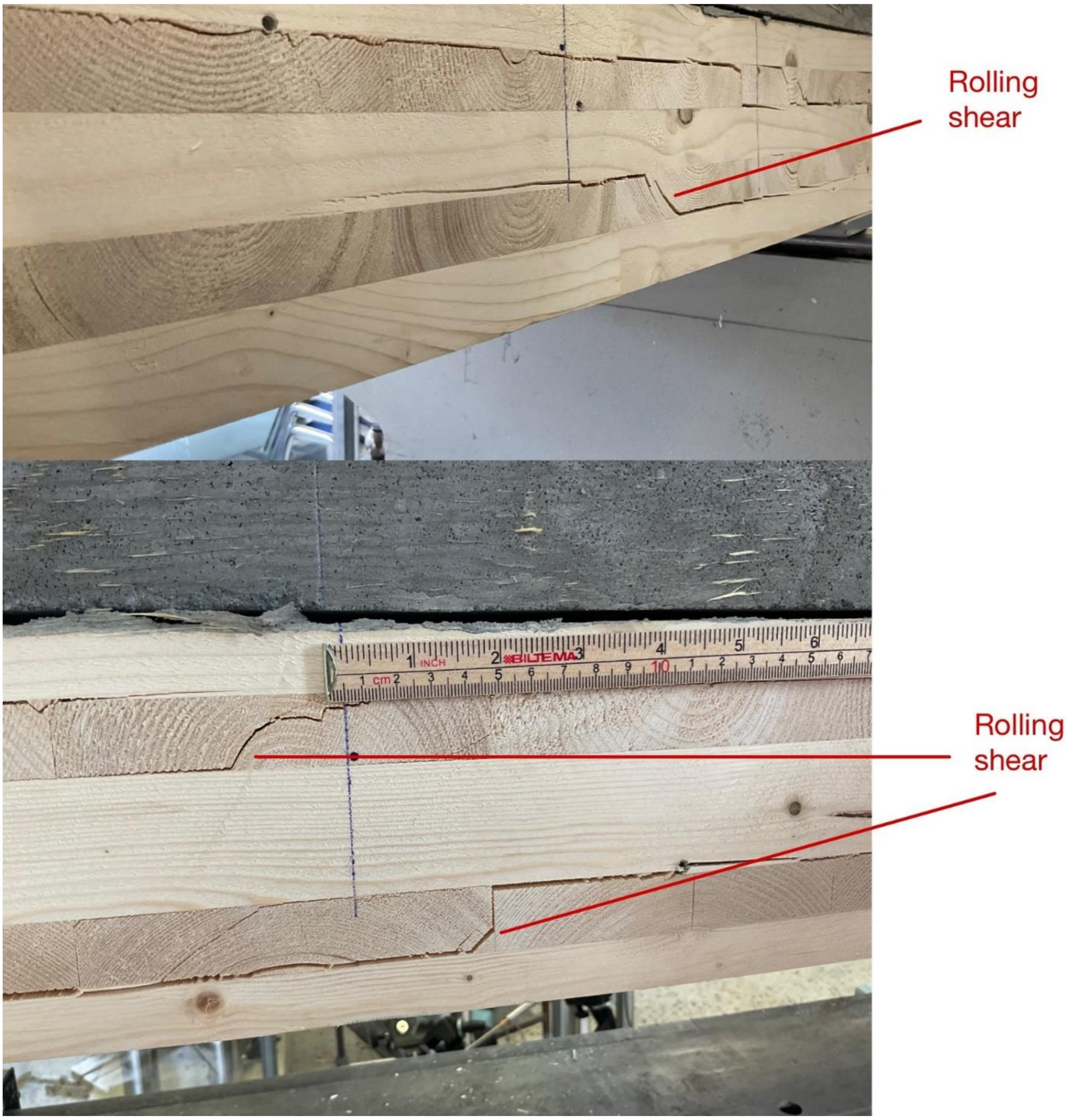

Figure 8.7 Typical rolling shear failure

Deformation in the CLT was observed, which if the applied load is high enough, it can break the bonding between lamellas and cause delamination. Some combination of rolling shear failure and delamination were common failure modes in the test. Another notable observation from the laboratory testing is the crack failure at the finger joints.

Tensile failure is common and typically occurs in the bottom layer of the CLT panel. This layer faces high tensile stresses as the specimen undergoes testing. The failure mode that led to the most visible failure was the combination of rolling shear and tension perpendicular to grain of CLT panels.

Figure 8.8 Tensile failure underneath the specimens

The concrete failure was minor compared to the timber failure. The applied load led to cracks in the concrete along the load direction and especially around where the load and the rubber pads were placed. This also resulted in some additional separation between the two materials. Pictures of the specimens after the load capacity test can be seen in Appendix F.

In the context of this thesis, lateral deflection refers to the horizontal displacement or bending of the specimen under an applied load during the four-point bending test. Slip refers to the relative movement or sliding between two materials and can be seen in Appendix F.

Figure 8.9 Concrete crack

9. Discussion

9.1 Limitations

9.1.1 Limitations of the specimen

Like other timber products, some knots and minor cracks were observed in the CLT panels. These imperfections can influence strength. The width of the CLT slabs varies slightly, with the majority measuring 59.60 cm instead of the desired 60 cm. The focus of this thesis is CLT panels with dimensions of L = 1600 mm, b=600 mm, and h_{CLT}=120 mm. Panel A2 stands out with layer 3 being 2.90 cm high, different from the other 14 panels, which are around 4 cm. The reduced height of A2 compared to theoretical predictions can have an impact on the load capacity and accuracy of the load calculations.

One limitation is the accuracy of assembling the screws and the distance between them. Despite efforts to ensure precise screw placement, some degree of variation may occur during the assembly process. It can lead to potential inconsistencies in load distribution. Some slippage and inaccuracies are unavoidable during the assembly process of the screws to achieve a 45-degree orientation, even though a jig is being used. It is worth noting that in some instances, unnecessary holes have been drilled. These additional holes and some slight inaccuracy in terms of the spacing between the screws can weaken the structural performance of the CLT panels and potentially compromise their overall performance.

In some of the composite, there is a noticeable gap between the concrete and CLT layers before the test, which may influence the structural performance of the composite as they will act less as a unit.

9.1.2 Limitations to the four-point bending test

The four-point bending test had its own limitations and it's important to find out the correct setup. An effort was made to ensure accuracy in the placement of the specimens on the supports, but slight inaccuracies in terms of millimeter positioning on both sides are expected.

The first test for slab E1 was not in accordance with [31] which is described in chapter 7.3. The specimen had two cyclic loadings instead of one. Figure 9.1 shows the graphical visualization of the 2 cycles.

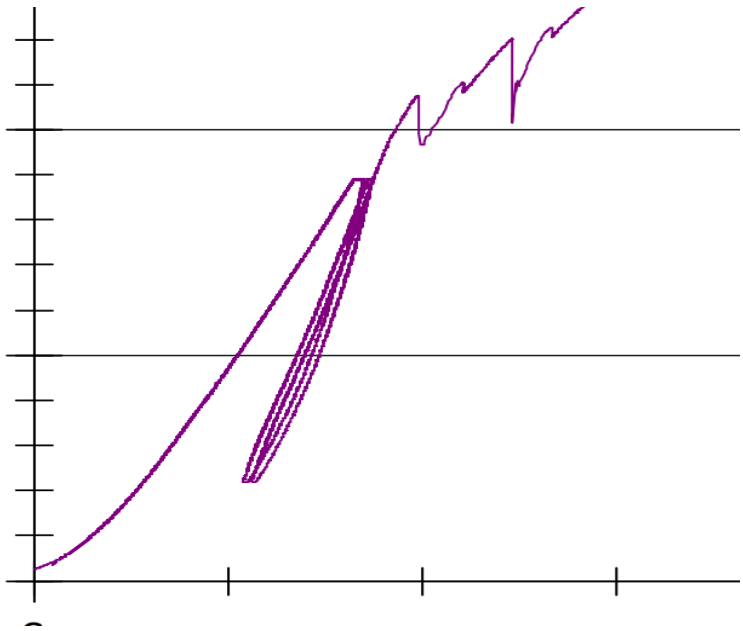

Figure 9.1 Graphical visualization of 2 cycles from Toni Technik, slab E3

After the first specimen was tested as an "pilot" test, all the other tests went according to [31] with one cyclic loading. Before this thesis there was one student that tested in the same manner and helped with the setup. Figure 9.2 shows the graphical visualization of 1 cycle.

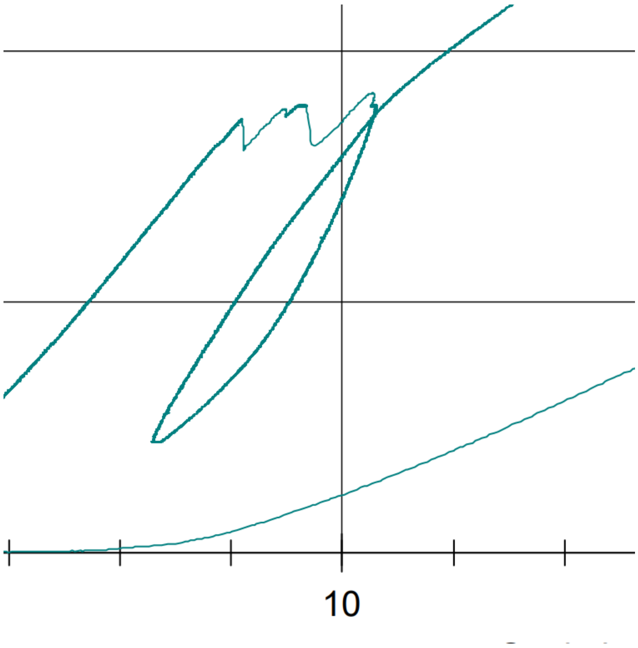

Figure 9.2 Graphical visualization of 1 cycle from Toni Technik, slab E2

The supports for the four-point bending test at the university had a circular shape. To prevent the wood from crushing and the timber element from sliding due to the smooth surface, an "L"-shaped steel profile was installed under the timber element as described in Chapter 7.2. Pictures of the support can also be seen in that chapter. Slab E3 did not have the "L"-shaped steel profile installed during the test. Slab A2 was not grinded on one of the short edges, it has a drop at the beginning of the test shown in figure 9.3.

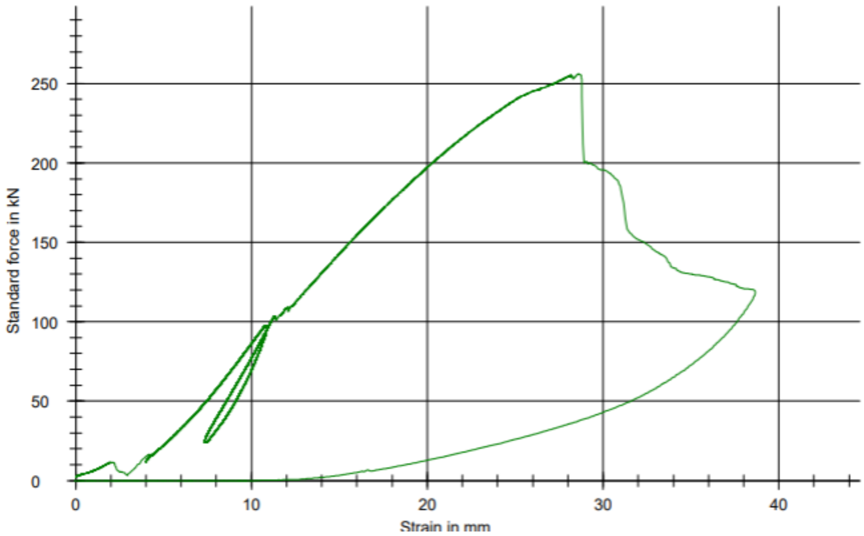

Figure 9.3 Graph of slab A2 showing a drop right after the start of the test

The LVDT`s had some limitations because they were installed incorrectly and did not measure the lateral deflection. In one of the specimens, the measurement went up to 2-3 mm and stayed there the whole time, making it unlikely that the specimen was not marked.

9.1.3 Limitations of the theoretical predictions

Limitations to the theoretical predictions are significant as there is no standard for CLT floors/slabs, and the calculations have not yet been verified and used in any standard. A design example can be found in [12, p.129-173] with all the different methods and verifications for beams and is used in this thesis. All the verifications are also considered for one element there is no verification for the whole system. The results show that the timber part always fails first.

There are no guidelines for screws with 90°angle which might explain the early drops that can be found in chapter 8.2. These screws were underperforming compared to 45° inclined crossed screws. Calculations were the same for both types of orientations and there is uncertainty if the predictions really show the true results for 90° angle screws.

The quadratic equation is used in this thesis to adjust the effective compressive height of the concrete it is taken from [12, p.134]. By doing so, the verifications of the theoretical predictions are satisfied. The only part that never got satisfied was the shear stress in the timber part, even after applying the quadratic equation. It is not clearly defined whether this is the right method for theoretical predictions of TCC.

9.2 Comparison of the results

9.2.1 Failure loads comparison

In chapter 8.3 the results from the test have been graphically presented and tabulated into tables. From the maximum applied load one can easily conclude that the screws with 45° angel crossed can withstand a much higher load. As for the 90° orientation screw the capacity was around 170-200 kN compared to 210-250 kN for 45° orientation.

Another problem that was found with the 90° screws was that they had a lot of drops before even reaching the cycle which they should do at approximately 40% of the overall capacity. This might be explained by a failure in the composite system where the composite action for 90° is not efficient enough and the load is not distributed correctly in the system.

Table 9.1 Failure loads for each slab data obtained from Catman

Specimen	Failure load [kN]
A1	234.151
A2	244.56
A3	229.032
B1	205.417
B2	245.658
B3	218.658
C1	171.688
C2	175.125
C3	166.785
D1	176.336
D2	155.357
D3	163.218
E1	194.233
E2	190.905
E3	230.891

Chapter 8.2 explains that the Catman software was not totally compatible with Toni Technik. Table 9.2 shows the failure loads obtained from Toni Technik. This shows that the actual failure loads were approximately 10 kN higher for each slab, and that the Catman software did not give the correct values.

Table 9.2 Failure load data for each slab obtained from Toni Technik

Slab	Failure load [kN]
A1	244.97

A2	255.93
A3	239.75
B1	214.97
B2	257.01
B3	228.83
C1	179.4
C2	183.3
C3	174.55
D1	184.55
D2	162.59
D3	170.83
E1	204.34
E2	199.76
E3	241.44

The best comparison between types of slabs is type B and E. They had just as many screws connecting the elements of the TCC. Type B had a spacing of 250 mm between the screws in the longitudinal direction. Compared to type E which had 125 mm spacing. On average the failure load for type B was 223.603 kN and for type E it was 215.18 kN. Overall, the 90° angel screws had an inferior failure load to the 45° screws, but type B and E had quite similar failure loads.

The maximum vertical displacement at the failure load occurs at around 15-22 mm deflection. The types that stand out are type C and D. They both have 90° orientation and a spacing of 200 and 250 mm respectively. Their failure drops occurred at a much lower value whereas all the failure drops come around 160-175 kN which can indicate an inferiority to the other types of slabs.

9.2.2 Lateral displacement comparison

Graphs of load and lateral displacement for each slab can be found in appendix D. Tables of the lateral displacement of each LVDT at the failure load can be seen bellow for each group. From the tables it is possible to see that type A and B have a very small lateral displacement and a much bigger failure load than type C, D and E. This indicated that the 90° orientation screws can`t hold the elements together and by doing so the displaced elements have a much lower capacity. Type E does have a load capacity that matches type B, but from the lateral displacement standpoint the slip is much greater in type E.

Table 9.3 Lateral displacement at the failure load for type A data obtained from Catman

Slab	Displacement right concrete [mm]	Displacement left timber [mm]	Displacement left concrete [mm]	Failure Load [kN]
A1	-0.4172	0.375	1.0614	234.151
A2	-0.7777	0.7361	0.2813	244.56
A3	-1.536	1.5542	1.1547	229.032

The lateral displacement in slab B2 is not right. The setup of LVDT`s was not done correctly, and the values are wrong.

Table 9.4 Lateral displacement and failure load for type B data obtained from Catman

Slab	Displacement right concrete [mm]	Displacement left timber [mm]	Displacement left concrete [mm]	Failure Load [kN]
B1	0.5915	-0.5309	-0.5574	205.417
B2	-4.8432	7.479	6.8588	245.658
B3	-2.0815	2.1804	2.1786	218.658

Table 9.5 Lateral displacement and failure load for type C data obtained from Catman

Slab	Displacement right concrete [mm]	Displacement left timber [mm]	Displacement left concrete [mm]	Failure Load [kN]
C1	-3.1961	3.0965	3.2699	171.688
C2	-3.6472	4.3231	3.2364	175.125
C3	-0.928	0.8908	1.0246	166.785

Table 9.6 Lateral displacement and failure load for type D data obtained from Catman

Slab	Displacement right concrete [mm]	Displacement left timber [mm]	Displacement left concrete [mm]	Failure Load [kN]
D1	-1.274	1.1998	1.0138	176.336
D2	2.7308	-2,6567	-2.865	155.357
D3	-2.4702	2.393	2.1532	163.218

Table 9.7 Lateral displacement and failure load for type E data obtained from Catman

Slab	Displacement right concrete [mm]	Displacement left timber [mm]	Displacement left concrete [mm]	Failure Load [kN]
E1	-5.4518	3.749	9.6349	194.233
E2	-1.4958	1.9362	2.0063	190.905
E3	4.1428	-4.1127	-4.5327	230.891

9.3 Comparison of theoretical predictions and test results

To present the comparison between failure loads and theoretical predictions, a graphical visualization is made. The failure load is from chapter 9.2.1. The vertical axis presents the theoretical values that are calculated. The horizontal axis represents the failure loads. For comparison, short- and long-term maximum loadings are used for theoretical predictions. Each type of specimen has its own figure below.

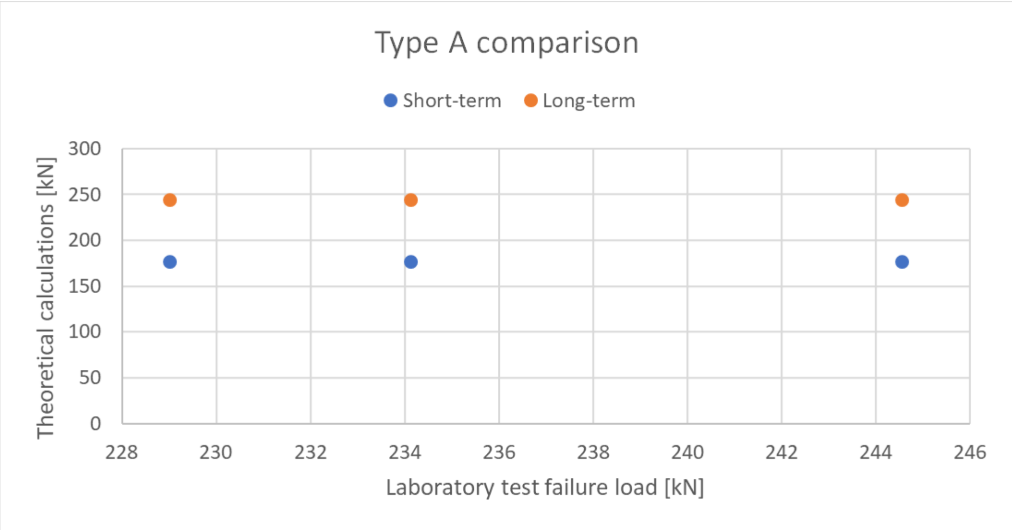

Figure 9.4 Comparison of theoretical predictions and failure loads for type A

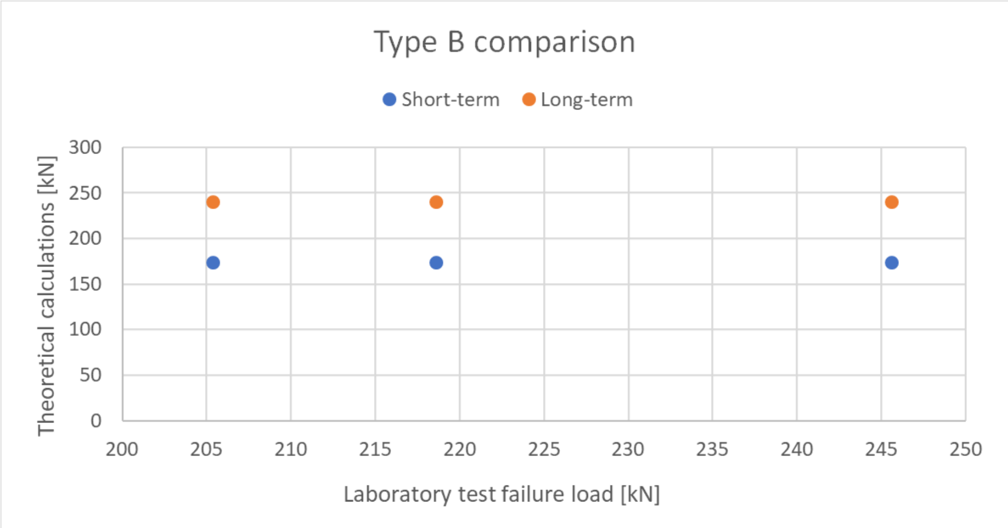

Figure 9.5 Comparison of theoretical predictions and failure loads for type B

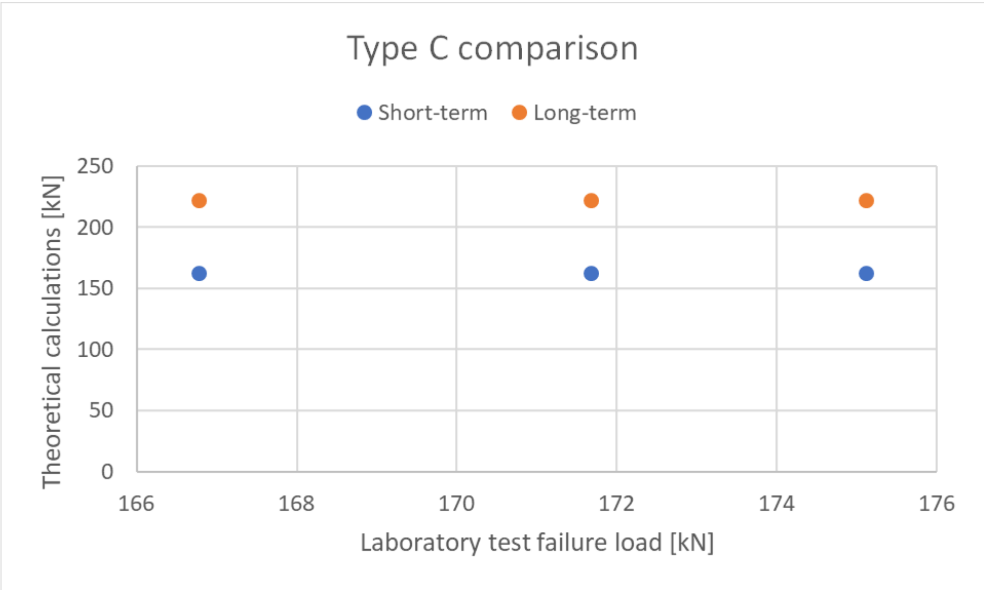

Figure 9.6 Comparison of theoretical predictions and failure loads for type C

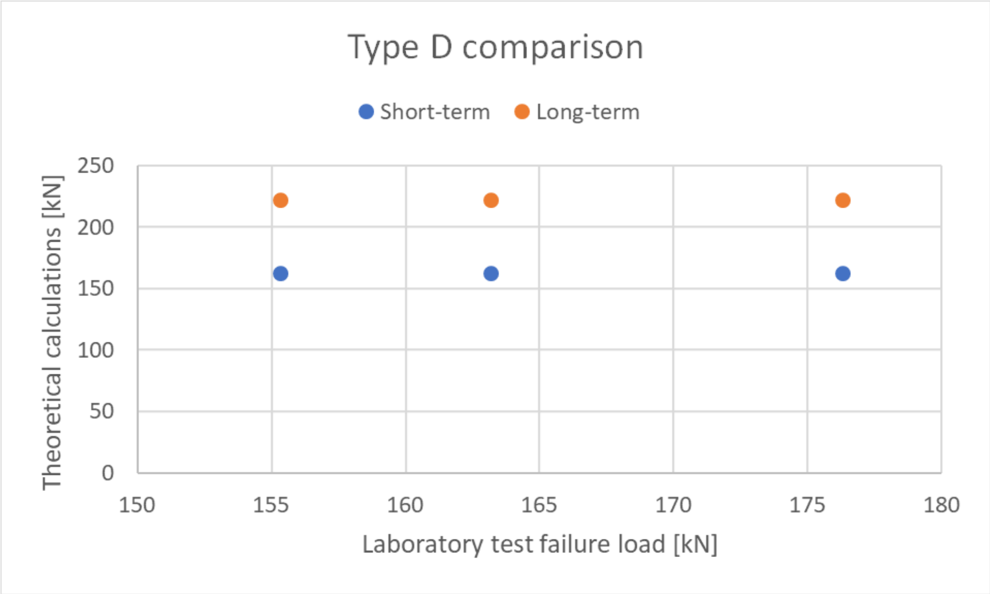

Figure 9.7 Comparison of theoretical predictions and failure loads for type D

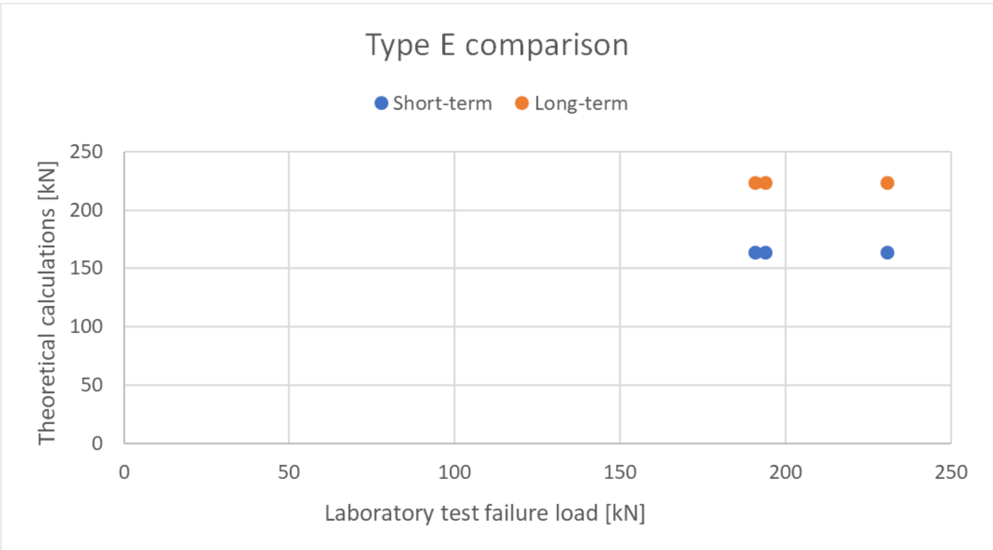

Figure 9.8 Comparison of theoretical predictions and failure loads for type E

10. Conclusion

10.1 Concluding remarks

From the load-displacement behavior graphs, a linear behavior can be seen before the first drop that happens due to interlayer slip or to a premature breakdown of the slab. The timber element, which breached during the drop, was weaker than the concrete in all the cases. A nonlinear behavior can be seen after the premature breakdown.

From the results the crossed 45° inclined screws had a much higher load capacity than the 90° single oriented screws. The load displacement behavior of 45° screws shows lower values for the lateral in-plane displacement and it might indicate that the slabs could withstand more load due to less interlayer slip between the elements compared to 90°. The theoretical predictions were conservative compared to the results and the slabs withstood a maximum applied load that was calculated for the long-term loading. There was difference of 40-50 kN in the load capacity where the 45° screws were superior to the 90° screws.

90° screws had one group with shorter spacing in the longitudinal direction. This group had a load capacity that could be compared with the maximum load applied for the long-term predictions. The rest of the specimens that had screws oriented at 90° showed a lower load capacity where they could be compared to the short-term loading predictions. Those two groups gave similar results to the theoretical predictions. The slip between the interlayers was much greater for the 90° screws.

10.2 Further study

In further study, the limitations of theoretical predictions cannot be ignored. This study is experimental, and the calculations for CLT and concrete are not given by any standard. All though the current Eurocode 5 for timber structures does not mention CLT, it is expected that this innovative product is here to stay and should be included in future versions of the code. New formulas for theoretical predictions can be found or interpreted to get a better understanding of the system. This should be done by a master's student as it is a demanding task. Reference [13] and the design example inside of the article could be great guidance.

Since the timber element is the weakest element in the TCC system a suggestion to increase or decrease the thickness of CLT could increase the capacity. Additional, gauges/sensors could be used to enhance the accuracy of the measurement of the slip between the concrete and

CLT. Embedded strain gauges could be installed between the interlayers of the timber and concrete element before the casting of concrete. Using screws with an orientation of 45° and having them crossed gave the best results overall in this study. In other articles about TCC`s orientation the screw in this manner is broadly used. It is therefore recommended to continue in the same manner. Studies about TCC`s connected with steel plates in addition to screws are also to be found and could be a new direction.

CTC screws are very solid and a great match for TCC. An idea could be to try and find screws that are easier to access and used more broadly. This would help to make the case for TCC`s being more profitable to make. An investigation of the environmental impact that a TCC floor has compared to concrete or timber floors could be interesting. The cost of the material and the time it takes to make a floor could give great indications of the practicality of a TCC floor being used.

Either way, after conducting extended research on the topic of TCC, it feels like after years with stagnant progress this topic is becoming more optional and further research articles are being made.

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12. Appendices

Appendix A. ULS, load capacity

Appendix B. SLS, maximum deflection

Appendix C. Graphs, vertical deflection Catman

Appendix D. Graphs, lateral deflection Catman

Appendix E. Graphs, Toni Technik

Appendix F. Pictures of failures

Appendix A.

ULS, load capacity

A.1 Load capacity for type A

A.2 Load capacity for type B

A.3 Load capacity for type C

A.4 Load capacity for type D

A.5 Load capacity for type E

ULS calculation predictions for CTC-screws 7-160 mm with 45 degree orientation and spacing 200 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

> $L := 1500$; #mm "lenght of the span betwen the supports"

$$L := 1500 \tag{1}$$

> $b := 600$; #mm "width of the composite"

$$b := 600 \tag{2}$$

Concrete parameters, concrete class B 35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$; #mm "height of concrete"

$$h_c := 80 \tag{3}$$

> $A_c := h_c \cdot b$; #mm^2

$$A_c := 48000 \tag{4}$$

> $I_c := \frac{(b \cdot h_c \hat{} 3)}{12}$; #mm^4

$$I_c := 25600000 \tag{5}$$

> $E_{cm,\,c} := 34000$; #MPa

$$E_{cm,c} := 34000 \tag{6}$$

> $f_{ck,\,c} := 35$; #MPa

$$f_{ck,c} := 35 \tag{7}$$

> $f_{ctk,\,0.05,\,c} := 2.2$; #MPa

$$f_{ctk,0.05,c} := 2.2 \tag{8}$$

> $\rho_c := 25.00$; #$\frac{kN}{m^3}$

$$\rho_c := 25.00 \tag{9}$$

> $\gamma_c := 1.5$;

$$\gamma_c := 1.5 \tag{10}$$

> $\varphi_c := 2.5$;

$$\varphi_c := 2.5 \tag{11}$$

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$; #mm

$$h_1 := 20 \tag{12}$$

> $h_2 := 20$; #mm

$$h_2 := 20 \tag{13}$$

> $h_3 := 40$; #mm

$$h_3 := 40 \tag{14}$$

> $h_4 := 20$; #mm

$$h_4 := 20 \tag{15}$$

> $h_5 := 20$; #mm

$$h_5 := 20 \tag{16}$$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$; #mm

$$h_t := 120 \tag{17}$$

> $\gamma_M := 1.15$ # NA in Eurocode 5 for Glued laminated timber

$$\gamma_M := 1.15 \tag{18}$$

> $K_{lima} := 1.0$; # Serice class, permanent

$$K_{lima} := 1.0 \tag{19}$$

> $k_{modi,\, t} := 0.8$; # modification factor, Swedish CLT handbook

$$k_{modi,t} := 0.8 \tag{20}$$

> $k_{def,\, t} := 0.85$; # modification factor, Swedish CLT handbook

$$k_{def,t} := 0.85 \tag{21}$$

Lamellae 1 and 5, Class T22

> $E_{0,\, mean,\, t22} := 13000$; # $\frac{N}{mm^2}$

$$E_{0,mean,t22} := 13000 \tag{22}$$

> $E_{90,\, mean,\, t22} := 430$; # $\frac{N}{mm^2}$

$$E_{90,mean,t22} := 430 \tag{23}$$

> $G_{0,mean,t22} := 810;\ \#\frac{N}{mm^2}$

$$G_{0,mean,t22} := 810 \tag{24}$$

> $G_{90,\ mean,\ t22} := 81;\ \#\frac{N}{mm^2}$

$$G_{90,mean,t22} := 81 \tag{25}$$

> $G_{R,\ t22} := G_{90,mean,t22};\ \#\frac{N}{mm^2}$

$$G_{R,t22} := 81 \tag{26}$$

> $f_{m,\ k,\ t22} := 30.5;\ \#\frac{N}{mm^2}$

$$f_{m,k,t22} := 30.5 \tag{27}$$

> $f_{t,\ 0,\ k,\ t22} := 22.0;\ \#\frac{N}{mm^2}$

$$f_{t,0,k,t22} := 22.0 \tag{28}$$

> $f_{v,\ k,\ t22} := 4.0;\ \#\frac{N}{mm^2}$

$$f_{v,k,t22} := 4.0 \tag{29}$$

> $t_{t22} := 470;\ \#\frac{kg}{m^3}$

$$t_{t22} := 470 \tag{30}$$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855 3261}{1};\ \#\frac{kN}{m^3}$

$$\rho_{t22} := 4.609118725 \tag{31}$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500;\ \#\frac{N}{mm^2}$

$$E_{0,mean,t15} := 11500 \tag{32}$$

> $E_{90,\ mean,\ t15} := 230;\ \#\frac{N}{mm^2}$

$$E_{90,mean,t15} := 230 \tag{33}$$

> $G_{0,\ mean,\ t15} := 720;\ \#\frac{N}{mm^2}$

$$G_{0,mean,t15} := 720 \tag{34}$$

> $G_{90,\ mean,\ t15} := 72;\ \#\frac{N}{mm^2}$

$$G_{90,mean,t15} := 72 \quad (35)$$

> $G_{R,\,t15} := G_{90,\,\mathrm{mean},\,t15};\ \# \frac{N}{mm^2}$

$$G_{R,t15} := 72 \quad (36)$$

> $f_{m,\,k,\,t15} := 22;\ \# \frac{N}{mm^2}$

$$f_{m,k,t15} := 22 \quad (37)$$

> $f_{t,\,0,\,k,\,t15} := 15.0;\ \ \# \frac{N}{mm^2}$

$$f_{t,0,k,t15} := 15.0 \quad (38)$$

> $f_{v,\,k,\,t15} := 4.0;\ \# \frac{N}{mm^2}$

$$f_{v,k,t15} := 4.0 \quad (39)$$

> $t_{t15} := 430;\ \# \frac{kg}{m^3}$

$$t_{t15} := 430 \quad (40)$$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.0098066355855326 1}{1};\ \# \frac{kN}{m^3}$

$$\rho_{t15} := 4.216853302 \quad (41)$$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # *Equation 6.10b give larger values*

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note

The load calculations is in kN/m, kN and kNm

There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right);\ \# \frac{kN}{m}$

$$g_{0,k} := 1.517734993 \quad (42)$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k} \cdot \gamma_{G,\,1};$ # $\frac{kN}{m}$

$$f_{d,ULS} := 1.821281992 \quad (43)$$

Modification of the shear force and moment:
The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1;$ #mm^2

$$A_1 := 12000 \quad (44)$$

> $A_5 := A_1;$ #mm^2

$$A_5 := 12000 \quad (45)$$

> $I_{t1} := \frac{\left(b \cdot h_1^{\,3}\right)}{12};$ #mm^4

$$I_{t1} := 400000 \quad (46)$$

> $I_{t5} := I_{t1};$ #mm^4

$$I_{t5} := 400000 \quad (47)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2;$ #mm^2

$$A_2 := 12000 \quad (48)$$

> $A_3 := b \cdot h_3\,;$ #mm^2

$$A_3 := 24000 \quad (49)$$

> $A_4 := A_2;$ #mm^2

$$A_4 := 12000 \quad (50)$$

> $I_{t2} := \frac{(b \cdot h_2^{\ 3})}{12}$; #mm^4

$$I_{t2} := 400000 \quad \textbf{(51)}$$

> $I_{t3} := \frac{(b \cdot h_3^{\ 3})}{12}$; #mm^4

$$I_{t3} := 3200000 \quad \textbf{(52)}$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad \textbf{(53)}$$

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 50 \quad \textbf{(54)}$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 30 \quad \textbf{(55)}$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad \textbf{(56)}$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 30 \quad \textbf{(57)}$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 50 \quad \textbf{(58)}$$

> $(EI)_1 := E_{0,\,mean,\,t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 5200000000 \quad \textbf{(59)}$$

> $(EI)_2 := E_{90,\,mean,\,t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad \textbf{(60)}$$

> $(EI)_3 := E_{0,\,mean,\,t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 36800000000 \quad \textbf{(61)}$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad \textbf{(62)}$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 5200000000 \quad \textbf{(63)}$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 47384000000 \quad (64)$$

> $(EAz\hat{}2)_1 := E_{0,\,mean,\,t22} \cdot A_1 \cdot \left(z_1^{\,2}\right)$; #Nmm^2

$$(EAz\hat{}2)_1 := 390000000000 \quad (65)$$

> $(EAz\hat{}2)_2 := E_{90,\,mean,\,t15} \cdot A_2 \cdot \left(z_2^{\,2}\right)$; #Nmm^2

$$(EAz\hat{}2)_2 := 2484000000 \quad (66)$$

> $(EAz\hat{}2)_3 := E_{0,\,mean,\,t15} \cdot A_3 \cdot \left(z_3^{\,2}\right)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad (67)$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot \left(z_4^{\,2}\right)$; #Nmm^2

$$(EAz\hat{}2)_4 := 2484000000 \quad (68)$$

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot \left(z_5^{\,2}\right)$; #Nmm^2

$$(EAz\hat{}2)_5 := 390000000000 \quad (69)$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 784968000000 \quad (70)$$

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \mathrm{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 8.323520000 \times 10^{11} \quad (71)$$

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 100 \quad (72)$$

> $(GA)_{eff} :=$

$$\mathrm{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\,mean,\,t22} \cdot b} + \frac{h_2}{G_{90,\,mean,\,t15} \cdot b} + \frac{h_3}{G_{0,\,mean,\,t15} \cdot b} + \frac{h_4}{G_{90,\,mean,\,t15} \cdot b}\right.\right.$$

$$+\frac{h_5}{2\cdot G_{0,\,mean,\,t22}\cdot b}\Big)\Big); \#N$$

$$(GA)_{eff} := 9.436893204\times 10^{6} \quad \textbf{(73)}$$

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app}=\frac{EI_{eff}}{1+\frac{K_s EI_{eff}}{GA_{eff}L^2}}$$

> $K_s := 11.5;$
 \# CLT handbook US, Ch.3, table 2, pinned − pinned support, uniformly distubuted load

$$K_s := 11.5 \quad \textbf{(74)}$$

> $EI_{app} := \frac{(EI)_{eff}}{1+\frac{K_s\cdot(EI)_{eff}}{(GA)_{eff}\cdot L\text{^}2}}; \#\frac{N}{mm^2}\cdot mm^4$

$$EI_{app} := 5.737154677\times 10^{11} \quad \textbf{(75)}$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b\cdot h_t^3}{12}}; \#\frac{N}{mm^2}$

$$E_{CLT} := 6640.225321 \quad \textbf{(76)}$$

3. γ-method from, EC5, Annex B, Maximum load capacity based on short-term verification of the slab - ULS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm,\,c}; \#\frac{N}{mm^2}$

$$E_1 := 34000 \quad \textbf{(77)}$$

> $E_2 := E_{CLT}; \#\frac{N}{mm^2}$

$$E_2 := 6640.225321 \quad \textbf{(78)}$$

> $h_1 := h_c; \#mm$

$$h_1 := 80 \quad \textbf{(79)}$$

> $h_2 := h_t; \#mm$

(80)

$$h_2 := 120 \tag{80}$$

> $A_1 := A_c$;#mm^2

$$A_1 := 48000 \tag{81}$$

> $A_2 := h_2 \cdot b$; #mm^2

$$A_2 := 72000 \tag{82}$$

> $I_1 := I_c$; #mm^4

$$I_1 := 25600000 \tag{83}$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \tag{84}$$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227. The formula is multiplied by 3, beacause there are 3 pairs of screws in each row.
Ku with secant value of 60% taken from, EC5: 2.2.2(2), eq.2.1

> $l_{eff,\ ctc} := 110$; #mm

$$l_{eff,ctc} := 110 \tag{85}$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\ ctc}$; #$\frac{N}{mm}$

$$K_{ser} := 23100 \tag{86}$$

> $K_u := \text{evalf}\left(\frac{2}{3} \cdot K_{ser}\right)$; #$\frac{N}{mm}$

$$K_u := 15400. \tag{87}$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 45;

$$angle := 45 \tag{88}$$

> k := sin(convert(angle degrees, radians));

$$k := \frac{\sqrt{2}}{2} \tag{89}$$

> $s_{min,\ 1} := \text{evalf}(130 \cdot k)$;#mm

$$s_{min,1} := 91.92388153 \tag{90}$$

> $s_{max,\ 1} := 4 \cdot s_{min,\ 1}$; #mm

(91)

$$s_{max,1} := 367.6955261 \quad (91)$$

> $s_{min} := 90$; #mm

$$s_{min} := 90 \quad (92)$$

> $s_{max} := 360$ #mm

$$s_{max} := 360 \quad (93)$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max}$; #mm

$$s := 157.50 \quad (94)$$

> $s := 200$; #mm

$$s := 200 \quad (95)$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_1 := \text{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L^2}}\right)$;

$$\gamma_1 := 0.01064160141 \quad (96)$$

> $\gamma_2 := 1.0$; #` Fully composite

$$\gamma_2 := 1.0 \quad (97)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 3.505222874 \quad (98)$$

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 96.49477713 \quad (99)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm²

$$EI_{eff,tot} := 1.611698850 \times 10^{12} \quad (100)$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.02166236236\, M_{Ed,1} \quad (101)$$

> $\sigma_{m,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.8438300990\, M_{Ed,1} \tag{102}$$

Stresses at the top of the concrete section

$\#\ \sigma c,t\ = -\ \sigma 1 - \sigma m,1 = \dfrac{fck}{\gamma c}$

$\#\ M_{Ed,1} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := \mathrm{solve}\left(M_{Ed,1} = \dfrac{f_{ck,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right)$; #Nmm

$$M_1 := 2.695960320 \times 10^7 \tag{103}$$

Stresses at the bottom of concrete section

$\#\sigma\ c,b\ = -\ \sigma\ 1\ + \sigma\ m,1 = \dfrac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \dfrac{f_{ctk,0.005,c}}{\gamma_c}$

> $M_2 := \mathrm{solve}\left(M_{Ed,1} = \dfrac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right)$; #Nmm

$$M_2 := 1.783901997 \times 10^6 \tag{104}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.01444157491\, M_{Ed,2} \tag{105}$$

> $\sigma_{m,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6$; #MPa

(106)

$$\sigma_{m,2} := 0.2472009701\, M_{Ed,2} \quad \textbf{(106)}$$

Stresses at the top of the timber section

$$\#\sigma\, t,\, t\; = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10^6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10^6\;;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> \; M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 7.939979454 \times 10^7 \quad \textbf{(107)}$$

Stresses at the bottom of the timber section

$$\#\sigma\, t,\, b\; = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> \; M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 9.339475180 \times 10^7 \quad \textbf{(108)}$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed,\,new} := \dfrac{\min(M_1, M_3, M_4)}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 26.95960320 \quad \textbf{(109)}$$

> $L_{out} := 0.3$; # m

$$L_{out} := 0.3 \quad \textbf{(110)}$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \quad \textbf{(111)}$$

> $P_{Ed} := \mathrm{solve}\left(\dfrac{P_{Ed,\,1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\;2}}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right)$; #kN

$$P_{Ed} := 175.4620583 \quad \textbf{(112)}$$

3.6 Verification of the maximum loading

3.6.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.5840086937 \quad \textbf{(113)}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 22.74932464 \quad \textbf{(114)}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333333 \quad \textbf{(115)}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.9999999999 \quad \textbf{(116)}$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 22.16531595 \quad \textbf{(117)}$$

$$> Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}; \#> 1 \text{ NOT OK}$$

$$Ver_{bottom,c} := 15.11271542 \tag{118}$$

3.6.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6; \#MPa$$

$$\sigma_2 := 0.3893391293 \tag{119}$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 6.664440065 \tag{120}$$

Stresses at the top of the timber section

$$> \sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}; \#MPa$$

$$\sigma_{t,t} := -7.053779194 \tag{121}$$

Stresses at the bottom of the timber section

$$> \sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}; \#MPa$$

$$\sigma_{b,t} := 6.275100936 \tag{122}$$

Verification of the timber section

$$> Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right); \#< 1 \text{ OK}$$

$$Ver_{timber} := -0.1651476369 \tag{123}$$

3.6.3 Shear stresses in the timber section

$$> \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2^{\,2}}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3; \#MPa$$

$$\tau_2 := 5.204926923 \tag{124}$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 1.870520614 \quad (125)$$

3.6.4 The load per shear fastener

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 36.48885782 \quad (126)$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 36.48885782 \quad (127)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (128)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.8742122187 \quad (129)$$

4. Quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(\mathrm{solve}\left(a_{1,\,1}^{\,2} \cdot \left(4 \cdot \gamma_1^{\,2} \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 138.3428530 \quad (130)$$

The effective compressed height of the concrete

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 2.944379000 \quad (131)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 0.1849575 \tag{132}$$

> $A_{1,\,eff} := b \cdot x;$

$$A_{1,eff} := 1766.627400 \tag{133}$$

> $I_{1,\,eff} := \dfrac{b \cdot x^3}{12};$

$$I_{1,eff} := 1276.295209 \tag{134}$$

New obtained effective bending stiffness

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}{}^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}{}^2;$ #Nmm²

$$EI_{eff,tot,new} := 5.860085367 \times 10^{11} \tag{135}$$

5. New short-term verification

Including the new modified parameters into the verification of the composite

5.1 Verification of the maximum loading using new parameters

5.1.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{\left(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10^6;$ #MPa

$$\sigma_1 := 2.302778606 \tag{136}$$

> $\sigma_{m,\,1} := \dfrac{\left(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10^6;$ #MPa

$$\sigma_{m,1} := 2.302778606 \tag{137}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -4.605557212 \tag{138}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}};$ #< 1 OK

$$Ver_{top,c} := -0.1973810234 \tag{139}$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 0. \tag{140}$$

Verification of the bottom part

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; # < 1 OK

$$Ver_{bottom,c} := 0. \tag{141}$$

5.1.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.05650206445 \tag{142}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 18.32920464 \tag{143}$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -18.38570670 \tag{144}$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 18.27270258 \tag{145}$$

Verification of the timber section

> $Ver_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.3401253489 \tag{146}$$

5.1.3 Shear stresses in the timber section

$$\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3; \#MPa$$

$$\tau_2 := 3.600874736 \tag{147}$$

Verification of the timber section

$$Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \# > 1.0 \text{ NOT OK}$$

$$Ver_{shear} := 1.294064358 \tag{148}$$

The results show that failure should occure in the timber section due to shear stresses

5.1.4 The load per shear fastener

$$F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_1 := 5.295376788 \tag{149}$$

$$F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_2 := 5.295372701 \tag{150}$$

$$f_{tens,k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \tag{151}$$

$$Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}; \# < 1.0 \text{ OK}$$

$$Ver_{F1} := 0.1268684022 \tag{152}$$

6. Long-term verification - ULS

6.1 Calculations of the new modulus of elasticity and slip modulus:

6.1.1 Concrete

$$E_{1,g} := \frac{E_{cm,c}}{1 + \varphi_c};$$

$$E_{1,g} := 9714.285714 \tag{153}$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \tag{154}$$

> $q_k := 0;$

$$q_k := 0 \tag{155}$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \tag{156}$$

> $E_1 := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285711 \tag{157}$$

6.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 3589.310984 \tag{158}$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 4659.807243 \tag{159}$$

> $E_2 := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 3589.310983 \tag{160}$$

6.1.3 Slip modulus Kser and Ku

> $K_{ser,g} := \frac{K_{ser}}{1+k_{def,t}};$

$$K_{ser,g} := 12486.48649 \tag{161}$$

> $K_{ser,q} := \frac{K_{ser}}{1+k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \tag{162}$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \tag{163}$$

> $K_u := \frac{2}{3} \cdot K_{ser,2}$

(164)

$$K_u := 8324.324327 \quad (164)$$

7. Long-term verification of the maximum loading - ULS

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$> \gamma_1 := \mathrm{evalf}\left(\frac{1}{1+\frac{\pi\hat{}2\cdot E_1\cdot s\cdot A_1}{K_u\cdot L\hat{}2}}\right);$$

$$\gamma_1 := 0.01994347280 \quad (165)$$

$$> \gamma_2 := 1.0;$$

$$\gamma_2 := 1.0 \quad (166)$$

$$> a_2 := \frac{\gamma_1\cdot E_1\cdot A_1\cdot(h_1+h_2)}{2\cdot(\gamma_1\cdot E_1\cdot A_1+\gamma_2\cdot E_2\cdot A_2)}; \#mm$$

$$a_2 := 3.473411734 \quad (167)$$

>

$$> a_1 := \frac{(h_1+h_2)}{2} - a_2; \#mm$$

$$a_1 := 96.52658827 \quad (168)$$

$$> EI_{eff,\ tot} := E_1\cdot I_1 + \gamma_1\cdot E_1\cdot A_1\cdot a_1^2 + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_2^2; \#Nmm^2$$

$$EI_{eff,tot} := 6.485656983\times 10^{11} \quad (169)$$

7.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_1 := \frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,\ 1})}{EI_{eff,\ tot}}\cdot 10^6; \#MPa$$

$$\sigma_1 := 0.02883398302\, M_{Ed,1} \quad (170)$$

$$> \sigma_{m,\ 1} := \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 0.5991242360\, M_{Ed,1} \quad (171)$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^6;+\ \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^6\right) \le \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\,tot}}\right)}, M_{Ed,\,1}\right); \#Nmm$$

$$M_1 := 3.715746147\times 10^7 \tag{172}$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \ \sigma\ 1\ + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6;+\ \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6\right) \le \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\,tot}}\right)}, M_{Ed,\,1}\right); \#Nmm$$

$$M_2 := 2.571789820\times 10^6 \tag{173}$$

7.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,\,2})}{EI_{eff,\,tot}}\cdot 10^6; \#MPa$$

$$\sigma_2 := 0.01922265535\ M_{Ed,2} \tag{174}$$

$$> \sigma_{m,\,2} := \frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,\,2})}{EI_{eff,tot}}\cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.3320537296\ M_{Ed,2} \tag{175}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t}\cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

> $M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_3 := 5.915025070 \times 10^7 \qquad (176)$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,\,b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 6.947316218 \times 10^7 \qquad (177)$$

7.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed,\,new} := \frac{\min(M_1, M_3, M_4)}{10^6}$; #kNm

$$M_{Ed,new} := 37.15746147 \qquad (178)$$

> $L_{out} := 0.3$; #m

$$L_{out} := 0.3 \qquad (179)$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \qquad (180)$$

> $P_{Ed} := \text{solve}\left(\frac{P_{Ed,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{2}}{8} = M_{Ed,new}, P_{Ed,1}\right)$; #kN

$$P_{Ed} := 243.4477801 \quad (181)$$

7.4 Verification of the Maximum loading

7.4.1 Normal stresses in the concrete section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 1.071397613 \quad (182)$$

> $\sigma_{m,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 22.26193571 \quad (183)$$

Stresses at the top of the concrete section

> $\sigma_{c,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333332 \quad (184)$$

Verification of the top section

> $Ver_{top,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}}$; #<1 OK

$$Ver_{top,c} := -0.9999999994 \quad (185)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 21.19053810 \quad (186)$$

> $Ver_{bottom,c} := \frac{\sigma_{b,c}}{\frac{f_{ctk,0.05,c}}{\gamma_c}}$; #<1 NOT OK

$$Ver_{bottom,c} := 14.44809416 \quad (187)$$

7.4.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.7142650756 \quad (188)$$

$$\sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 12.33827366 \quad (189)$$

Stresses at the top of the timber section

$$\sigma_{t,t} := -\sigma_2 - \sigma_{m,2}; \#MPa$$

$$\sigma_{t,t} := -13.05253874 \quad (190)$$

Stresses at the bottom of the timber section

$$\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}; \#MPa$$

$$\sigma_{b,t} := 11.62400858 \quad (191)$$

Verification of the timber section

$$Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right); \# < 1.0 \text{ OK}$$

$$Ver_{timber} := -0.3050118838 \quad (192)$$

7.4.3 Shear stresses in the timber section

$$\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{\ }2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3; \#MPa$$

$$\tau_2 := 9.700529201 \quad (193)$$

Verification of the timber section

$$Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \# > 1 \text{ NOT OK}$$

$$Ver_{shear} := 3.486127681 \quad (194)$$

7.4.4 The load per shear fastener

$$F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_1 := 67.38786389 \quad (195)$$

$$F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

(196)

$$F_2 := 67.38786392 \quad (196)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (197)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{F1} := 1.614500906 \quad (198)$$

8. Using quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(\mathrm{solve}\left(a_{1,\,1}^{\,2} \cdot \left(4 \cdot \gamma_1^{\,2} \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0,\ a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 136.9326803 \quad (199)$$

The effective compressed height of the concrete

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 5.461826370 \quad (200)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 0.3364065 \quad (201)$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 3277.095822 \quad (202)$$

> $I_{1,\,eff} := \dfrac{b \cdot x^3}{12}$

$$I_{1,eff} := 8146.736585 \quad (203)$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}^{\,2} + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}^{\,2}$; #Nmm^2

$$EI_{eff,tot,new} := 3.221294607 \times 10^{11} \quad (204)$$

9. New long-term verification

Including the new modified parameters into the verification of the composite

9.1 Verification of the maximum load using new parameters

9.1.1 Normal stresses in the concrete section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_1 := 3.060091110 \quad (205)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 3.060091112 \quad (206)$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -6.120182222 \quad (207)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.2622935238 \quad (208)$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 2. \times 10^{-9} \quad (209)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1 OK

$$Ver_{bottom,c} := 1.363636364 \times 10^{-9} \quad (210)$$

9.1.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.1392807373 \quad (211)$$

$$> \sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 24.84150645 \quad (212)$$

Stresses at the top of the timber section

$$> \sigma_{t,t} := -\sigma_2 - \sigma_{m,2}; \#MPa$$

$$\sigma_{t,t} := -24.98078719 \quad (213)$$

Stresses at the bottom of the timber section

$$> \sigma_{b,t} := -\sigma_2 + \sigma_{m,2}; \#MPa$$

$$\sigma_{b,t} := 24.70222571 \quad (214)$$

Verification of the timber section

$$> Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right); \#< 1 \text{ OK}$$

$$Ver_{timber} := -0.468023100 \quad (215)$$

9.1.3 Shear stresses in the timber section

$$> \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3; \#MPa$$

$$\tau_2 := 4.937593668 \quad (216)$$

Verification of the timber section

$$> Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \#> 1.0 \text{ NOT OK}$$

$$Ver_{shear} := 1.774447724 \quad (217)$$

Again the verifications show that failure will occure in the timber section due to shear stresses

9.1.4 The load per shear fasteners

$$> F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_1 := 13.14054191 \quad (218)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 13.14054361 \tag{219}$$

> $f_{tens,\ k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{220}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.3148254833 \tag{221}$$

>

ULS calculation predictions for CTC-screws 7-160 mm with 45 degree orientation and spacing 250 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm "lenght of the span betwen the supports"

> $b := 600$: #mm "width of the composite"

Concrete parameters, concrete class B 35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm "height of concrete"

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \frac{(b \cdot h_c\text{^}3)}{12}$: #mm^4

> $E_{cm,\,c} := 34000$: #MPa

> $f_{ck,\,c} := 35$: #MPa

> $f_{ctk,\,0.05,\,c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\frac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> $K_{lima} := 1.0$: # Serice class, permanent

> $k_{modi,\, t} := 0.8$: # modification factor, Swedish CLT handbook

> $k_{def,\, t} := 0.85$: # modification factor, Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0,\, mean,\, t22} := 13000$: # $\frac{N}{mm^2}$

> $E_{90,\, mean,\, t22} := 430$: # $\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: # $\frac{N}{mm^2}$

> $G_{90,\, mean,\, t22} := 81$: # $\frac{N}{mm^2}$

> $G_{R,\, t22} := G_{90,mean,t22}$: # $\frac{N}{mm^2}$

> $f_{m,\, k,\, t22} := 30.5$: # $\frac{N}{mm^2}$

> $f_{t,\, 0,\, k,\, t22} := 22.0$: # $\frac{N}{mm^2}$

> $f_{v,\, k,\, t22} := 4.0$: # $\frac{N}{mm^2}$

> $t_{t22} := 470$: # $\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855 3261}{1}$: # $\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\, mean,\, t15} := 11500$: # $\frac{N}{mm^2}$

> $E_{90,\, mean,\, t15} := 230$: # $\frac{N}{mm^2}$

> $G_{0,\, mean,\, t15} := 720$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t15} := 430 : \# \frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} : \# \frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # *Equation 6.10b give larger values*

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note

The load calculations is in kN/m, kN and kNm

There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right); \# \frac{kN}{m}$

$$g_{0,k} := 1.517734993 \tag{1}$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k} \cdot \gamma_{G,\,1}; \# \frac{kN}{m}$

$$f_{d,ULS} := 1.821281992 \tag{2}$$

Modification of the shear force and moment:
The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1 : \#mm^2$

> $A_5 := A_1 : \#mm^2$

> $I_{t1} := \dfrac{\left(b \cdot {h_1}^3\right)}{12} : \#mm^4$

> $I_{t5} := I_{t1} : \#mm^4$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2 : \#mm^2$

> $A_3 := b \cdot h_3 : \; \#mm^2$

> $A_4 := A_2 : \#mm^2$

> $I_{t2} := \dfrac{\left(b \cdot {h_2}^3\right)}{12} : \#mm^4$

> $I_{t3} := \dfrac{\left(b \cdot {h_3}^3\right)}{12} : \#mm^4$

> $I_{t4} := I_{t2} : \#mm^4$

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \dfrac{h_1}{2} + h_2 + \dfrac{h_3}{2} : \#mm$

> $z_2 := \dfrac{h_2}{2} + \dfrac{h_3}{2} : \#mm$

> $z_3 := 0 : \#mm$

> $z_4 := \dfrac{h_4}{2} + \dfrac{h_3}{2} : \#mm$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\text{ mean, t22}} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\text{ mean, t15}} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\text{ mean, t15}} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,\text{mean,t15}} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,\text{mean,t22}} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\hat{}2)_1 := E_{0,\text{ mean, t22}} \cdot A_1 \cdot \left(z_1^{\,2}\right)$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\text{ mean, t15}} \cdot A_2 \cdot \left(z_2^{\,2}\right)$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\text{ mean, t15}} \cdot A_3 \cdot \left(z_3^{\,2}\right)$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,\text{mean,t15}} \cdot A_4 \cdot \left(z_4^{\,2}\right)$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,\text{mean,t22}} \cdot A_5 \cdot \left(z_5^{\,2}\right)$: #Nmm^2

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$: #Nmm^2

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \text{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$: #Nmm^2

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$

$$\text{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\text{ mean, t22}} \cdot b} + \frac{h_2}{G_{90,\text{ mean, t15}} \cdot b} + \frac{h_3}{G_{0,\text{ mean, t15}} \cdot b} + \frac{h_4}{G_{90,\text{ mean, t15}} \cdot b} + \frac{h_5}{2 \cdot G_{0,\text{ mean, t22}} \cdot b}\right)\right)$$: #N

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5 :$
> # CLT handbook US, Ch.3, table 2, pinned− pinned support, uniformly distubuted load

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{}2}} : \# \dfrac{N}{mm^2} \cdot mm^4$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t^3}{12}} : \# \dfrac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum load capacity based on short-term verification of the slab - ULS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm, c} : \# \dfrac{N}{mm^2}$

> $E_2 := E_{CLT} : \# \dfrac{N}{mm^2}$

> $h_1 := h_c : \# mm$

> $h_2 := h_t : \# mm$

> $A_1 := A_c : \# mm^2$

> $A_2 := h_2 \cdot b : \# mm^2$

> $I_1 := I_c : \# mm^4$

> $I_2 := \dfrac{b \cdot h_t\hat{}3}{12} : \# mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227. The formula is multiplied by 3, beacause there are 3 pairs of screws in each row.
Ku with secant value of 60% taken from, EC5: 2.2.2(2), eq.2.1

> $l_{eff, ctc} := 110; \# mm$

$$l_{eff,ctc} := 110 \tag{3}$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff, ctc}$; # $\frac{N}{mm}$

$$K_{ser} := 23100 \quad (4)$$

> $K_u := \text{evalf}\left(\frac{2}{3} \cdot K_{ser}\right)$; # $\frac{N}{mm}$

$$K_u := 15400. \quad (5)$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 45;

$$angle := 45 \quad (6)$$

> k := sin(convert(angle degrees, radians));

$$k := \frac{\sqrt{2}}{2} \quad (7)$$

> $s_{min, 1} := \text{evalf}(130 \cdot k)$;#mm

$$s_{min,1} := 91.92388153 \quad (8)$$

> $s_{max, 1} := 4 \cdot s_{min, 1}$; #mm

$$s_{max,1} := 367.6955261 \quad (9)$$

> $s_{min} := 90$; #mm

$$s_{min} := 90 \quad (10)$$

> $s_{max} := 360$; #mm

$$s_{max} := 360 \quad (11)$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max}$; #mm

$$s := 157.50 \quad (12)$$

> s := 250; #mm

$$s := 250 \quad (13)$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_1 := \text{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L^2}}\right)$;

$$\gamma_1 := 0.008531438766 \quad (14)$$

> $\gamma_2 := 1.0$; #` Fully composite

$$\gamma_2 := 1.0 \tag{15}$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 2.829828325 \tag{16}$$

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 97.17017168 \tag{17}$$

> $EI_{\text{eff, tot}} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm^2

$$EI_{eff,tot} := 1.579408491 \times 10^{12} \tag{18}$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{\text{Ed, 1}})}{EI_{\text{eff, tot}}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.01784595101\, M_{Ed,1} \tag{19}$$

> $\sigma_{\text{m, 1}} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{\text{Ed,1}})}{EI_{\text{eff,tot}}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.8610818595\, M_{Ed,1} \tag{20}$$

Stresses at the top of the concrete section

$\sigma c,t = - \sigma 1 - \sigma m,1 = \dfrac{\text{fck}}{\gamma c}$

$M_{\text{Ed,1}} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{\text{Ed,1}})}{EI_{\text{eff,tot}}} \cdot 10\hat{}6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{\text{Ed,1}})}{EI_{\text{eff,tot}}} \cdot 10\hat{}6 \right) \leq \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := \text{solve}\left(M_{\text{Ed, 1}} = \dfrac{f_{\text{ck, c}}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{\text{eff, tot}}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{\text{eff, tot}}} \right)}, M_{\text{Ed, 1}} \right)$; #Nmm

$$M_1 := 2.654749691 \times 10^7 \tag{21}$$

Stresses at the bottom of concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1\ +\ \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6; + \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6\right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\ tot}}\right)}, M_{Ed,\ 1}\right); \#Nmm$$

$$M_2 := 1.739331369 \times 10^6 \tag{22}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,\ 2})}{EI_{eff,\ tot}}\cdot 10^6; \#MPa$$

$$\sigma_2 := 0.01189730067\ M_{Ed,2} \tag{23}$$

$$> \sigma_{m,\ 2} := \frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,\ 2})}{EI_{eff,tot}}\cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.2522548926\ M_{Ed,2} \tag{24}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\ 0,\ d}} - \frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t}\cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10^6;\right)}{\frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10^6;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_3 := 7.894876249 \times 10^7 \quad \textbf{(25)}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_4 := 8.999537508 \times 10^7 \quad \textbf{(26)}$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}; \#kNm$$

$$M_{Ed,new} := 26.54749691 \quad \textbf{(27)}$$

$$> L_{out} := 0.3; \#\ m$$

$$L_{out} := 0.3 \quad \textbf{(28)}$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \quad \textbf{(29)}$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed,\ new}, P_{Ed,\ 1} \right); \#kN$$

$$P_{Ed} := 172.7146831 \quad \textbf{(30)}$$

3.6 Verification of the maximum loading

3.6.1Normal stresses in the concrete section

$$> \sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.4737653294 \quad \textbf{(31)}$$

$$> \sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 22.85956800 \quad \textbf{(32)}$$

Stresses at the top of the concrete section

$$> \sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}; \#MPa$$

$$\sigma_{c,t} := -23.33333333 \quad \textbf{(33)}$$

Verification of the top section

$$> Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}}; \# < 1 \text{ OK}$$

$$Ver_{top,c} := -0.9999999999 \quad \textbf{(34)}$$

Stresses at the bottom of the concrete section

$$> \sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1}; \#MPa$$

$$\sigma_{b,c} := 22.38580267 \quad \textbf{(35)}$$

$$> Ver_{bottom,\ c} := \frac{\sigma_{b,\ c}}{\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c}}; \# > 1 \text{ NOT OK}$$

$$Ver_{bottom,c} := 15.26304728 \quad \textbf{(36)}$$

3.6.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10^6; \#MPa$$

$$\sigma_2 := 0.3158435528 \quad \textbf{(37)}$$

$$> \sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 6.696735980 \quad \textbf{(38)}$$

Stresses at the top of the timber section

$$> \sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2}; \#MPa$$

$$\sigma_{t,t} := -7.012579533 \tag{39}$$

Stresses at the bottom of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$; #MPa

$$\sigma_{b,t} := 6.380892427 \tag{40}$$

Verification of the timber section

> $\mathrm{Ver}_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.1574695393 \tag{41}$$

3.6.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2^{\,2}}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 5.228174858 \tag{42}$$

Verification of the timber section

> $\mathrm{Ver}_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 1.878875339 \tag{43}$$

3.6.4 The load per shear fastener

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 36.98709329 \tag{44}$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 36.98709326 \tag{45}$$

> $f_{tens,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{46}$$

> $\mathrm{Ver}_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.8861491100 \quad (47)$$

4. Quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(\mathrm{solve}\left(a_{1,\,1}^{\;2}\cdot\left(4\cdot\gamma_1^{\;2}\cdot E_1\cdot b\right) + a_{1,\,1}\cdot\left(2\cdot E_2\cdot A_2\cdot\left(1+\gamma_1\right)\right) - E_2\cdot A_2\cdot\left(2\cdot h_1 + h_2\right) = 0,\; a_{1,\,1}\right)\right);$ #mm

$$a_{1,eff} := 138.6972249 \quad (48)$$

The effective compressed height of the concrete

> $x := 2\cdot\gamma_1\cdot a_{1,\,eff};$ #mm

$$x := 2.366573762 \quad (49)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5\cdot x + 0.5\cdot h_2 - a_{1,\,eff};$

$$a_{2,new} := 0.1194882 \quad (50)$$

> $A_{1,\,eff} := b\cdot x;$

$$A_{1,eff} := 1419.944257 \quad (51)$$

> $I_{1,\,eff} := \frac{b\cdot x^3}{12};$

$$I_{1,eff} := 662.7200960 \quad (52)$$

New obtained effective bending stiffness

> $EI_{eff,\,tot,\,new} := E_1\cdot I_{1,\,eff} + \gamma_1\cdot E_1\cdot A_{1,\,eff}\cdot a_{1,\,eff}^{\;2} + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_{2,\,new}^{\;2};$ #Nmm²

$$EI_{eff,tot,new} := 5.816681615\times 10^{11} \quad (53)$$

5. New short-term verification

Including the new modified parameters into the verification of the composite

5.1 Verification of the maximum loading using new parameters

5.1.1 Normal stresses in the concrete section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_1 := 1.836188457 \quad (54)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 1.836188457 \quad (55)$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -3.672376914 \quad (56)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.1573875820 \quad (57)$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 0. \quad (58)$$

Verification of the bottom part

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; # < 1 OK

$$Ver_{bottom,c} := 0. \quad (59)$$

5.1.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.03621230099 \quad (60)$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 18.18370399 \quad (61)$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -18.21991629 \quad (62)$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 18.14749169 \quad (63)$$

Verification of the timber section

> $\mathrm{Ver}_{\mathrm{timber}} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.3351937848 \quad (64)$$

5.1.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 3.563176516 \quad (65)$$

Verification of the timber section

> $\mathrm{Ver}_{\mathrm{shear}} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1.0 NOT OK

$$Ver_{shear} := 1.280516560 \quad (66)$$

The results show that failure should occure in the timber section due to shear stresses

5.1.4 The load per shear fastener

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_1 := 4.240667659 \quad (67)$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 4.240668338 \quad (68)$$

> $f_{tens,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (69)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; #< 1.0 OK

$$Ver_{F1} := 0.1015993293 \quad (70)$$

6. Long-term verification - ULS

6.1 Calculations of the new modulus of elasticity and slip modulus:

6.1.1 Concrete

> $E_{1,g} := \dfrac{E_{cm,c}}{1 + \varphi_c}$;

$$E_{1,g} := 9714.285714 \quad (71)$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad (72)$$

> $q_k := 0$;

$$q_k := 0 \quad (73)$$

> $g_{1,k} := 0$;

$$g_{1,k} := 0 \quad (74)$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_1 := 9714.285711 \quad (75)$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1 + k_{def,t}}$;

$$E_{2,g} := 3589.310984 \quad (76)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2}$;

$$E_{2,q} := 4659.807243 \quad (77)$$

> $E_2 := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 3589.310983 \quad (78)$$

6.1.3 Slip modulus Kser and Ku

> $K_{ser,g} := \frac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (79)$$

> $K_{ser,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad (80)$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad (81)$$

> $K_u := \frac{2}{3} \cdot K_{ser,2}$

$$K_u := 8324.324327 \quad (82)$$

7. Long-term verification of the maximum loading - ULS

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

> $\gamma_1 := \mathrm{evalf}\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.01601867183 \quad (83)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (84)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)};$ #mm

$$a_2 := 2.809058722 \quad (85)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2;$ #mm

(86)

$$a_1 := 97.19094128 \quad (86)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm^2

$$EI_{eff,tot} := 6.313967974 \times 10^{11} \quad (87)$$

7.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.02395304831\, M_{Ed,1} \quad (88)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.6154155835\, M_{Ed,1} \quad (89)$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 3.649433546 \times 10^7 \quad (90)$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1 + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

> $M_2 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right)$; #Nmm

(91)

$$M_2 := 2.479728774 \times 10^6 \qquad \textbf{(91)}$$

7.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_2 := 0.01596869886\, M_{Ed,2} \qquad \textbf{(92)}$$

$$> \sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.3410829132\, M_{Ed,2} \qquad \textbf{(93)}$$

Stresses at the top of the timber section

$$\#\sigma\ t, t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.841449593 \times 10^7 \qquad \textbf{(94)}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 6.652377444 \times 10^7 \quad \textbf{(95)}$$

7.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed\ new} := \frac{\min(M_1, M_3, M_4)}{10^6}$; #kNm

$$M_{Ed,new} := 36.49433546 \quad \textbf{(96)}$$

> $L_{out} := 0.3$; #m

$$L_{out} := 0.3 \quad \textbf{(97)}$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \quad \textbf{(98)}$$

> $P_{Ed} := \text{solve}\left(\frac{P_{Ed,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{2}}{8} = M_{Ed\ new}, P_{Ed,1}\right)$; #kN

$$P_{Ed} := 239.0269401 \quad \textbf{(99)}$$

7.4 Verification of the Maximum loading

7.4.1 Normal stresses in the concrete section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.8741505802 \quad \textbf{(100)}$$

> $\sigma_{m,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 22.45918275 \quad \textbf{(101)}$$

Stresses at the top of the concrete section

> $\sigma_{c,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333333 \quad \textbf{(102)}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1 OK

$$Ver_{top,c} := -0.9999999999 \tag{103}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 21.58503217 \tag{104}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1 NOT OK

$$Ver_{bottom,c} := 14.71706739 \tag{105}$$

7.4.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.5827670532 \tag{106}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 12.44759425 \tag{107}$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -13.03036130 \tag{108}$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 11.86482720 \tag{109}$$

Verification of the timber section

> $Ver_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #<1.0 OK

$$Ver_{timber} := -0.2922127316 \tag{110}$$

7.4.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2{}^{\wedge}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 9.783360607 \qquad (111)$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 3.515895218 \qquad (112)$$

7.4.4 The load per shear fastener

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 68.70508612 \qquad (113)$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 68.70508609 \qquad (114)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \qquad (115)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{F1} := 1.646059355 \qquad (116)$$

8. Using quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(\mathrm{solve}\left(a_{1,\,1}{}^2 \cdot \left(4 \cdot \gamma_1{}^2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0,\ a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 137.5771216 \qquad (117)$$

The effective compressed height of the concrete

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 4.407605524 \tag{118}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 0.2190756 \tag{119}$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 2644.563314 \tag{120}$$

> $I_{1,\,eff} := \dfrac{b \cdot x^3}{12}$

$$I_{1,eff} := 4281.324641 \tag{121}$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}^{\;2} + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}^{\;2}$; #Nmm^2

$$EI_{eff,tot,new} := 3.179594993 \times 10^{11} \tag{122}$$

9. New long-term verification

Including the new modified parameters into the verification of the composite

9.1 Verification of the maximum load using new parameters

9.1.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{\left(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_1 := 2.457181581 \tag{123}$$

> $\sigma_{m,\,1} := \dfrac{\left(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 2.457181580 \tag{124}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -4.914363161 \tag{125}$$

Verification of the top section

> $Ver_{top, c} := \frac{\sigma_{c,t}}{\frac{f_{ck, c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.2106155640 \quad (126)$$

Stresses at the bottom of the concrete section

> $\sigma_{b, c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := -1. \times 10^{-9} \quad (127)$$

> $Ver_{bottom, c} := \frac{\sigma_{b, c}}{\frac{f_{ctk, 0.05, c}}{\gamma_c}}$; #<1 OK

$$Ver_{bottom,c} := -6.818181818 \times 10^{-10} \quad (128)$$

9.1.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2, new} \cdot M_{Ed, new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.09025239866 \quad (129)$$

> $\sigma_{m, 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 24.71815172 \quad (130)$$

Stresses at the top of the timber section

> $\sigma_{t, t} := -\sigma_2 - \sigma_{m, 2}$; #MPa

$$\sigma_{t,t} := -24.80840412 \quad (131)$$

Stresses at the bottom of the timber section

> $\sigma_{b, t} := -\sigma_2 + \sigma_{m, 2}$; #MPa

$$\sigma_{b,t} := 24.62789932 \quad (132)$$

Verification of the timber section

> $Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.460262522 \quad (133)$$

9.1.3 Shear stresses in the timber section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 4.892426488 \quad (134)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1.0 NOT OK

$$Ver_{shear} := 1.758215770 \quad (135)$$

Again the verifications show that failure will occure in the timber section due to shear stresses

9.1.4 The load per shear fasteners

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_1 := 10.64026932 \quad (136)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 10.64027005 \quad (137)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (138)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.2549231191 \quad (139)$$

>

ULS calculation predictions for CTC-screws 7-160 mm with 90 degree orientation and spacing 200 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm "lenght of the span betwen the supports"

> $b := 600$: #mm "width of the composite"

Concrete parameters, concrete class B 35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm "height of concrete"

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \dfrac{(b \cdot h_c\text{^}3)}{12}$: #mm^4

> $E_{cm, c} := 34000$: #MPa

> $f_{ck, c} := 35$: #MPa

> $f_{ctk, 0.05, c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\frac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> $K_{lima} := 1.0$: # Serice class, permanent

> $k_{modi, t} := 0.8$: # modification factor, Swedish CLT handbook

> $k_{def, t} := 0.85$: # modification factor, Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0, mean, t22} := 13000$: # $\frac{N}{mm^2}$

> $E_{90, mean, t22} := 430$: # $\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: # $\frac{N}{mm^2}$

> $G_{90, mean, t22} := 81$: # $\frac{N}{mm^2}$

> $G_{R, t22} := G_{90,mean,t22}$: # $\frac{N}{mm^2}$

> $f_{m, k, t22} := 30.5$: # $\frac{N}{mm^2}$

> $f_{t, 0, k, t22} := 22.0$: # $\frac{N}{mm^2}$

> $f_{v, k, t22} := 4.0$: # $\frac{N}{mm^2}$

> $t_{t22} := 470$: # $\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855 3261}{1}$: # $\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0, mean, t15} := 11500$: # $\frac{N}{mm^2}$

> $E_{90, mean, t15} := 230$: # $\frac{N}{mm^2}$

> $G_{0, mean, t15} := 720$: # $\frac{N}{mm^2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm^2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm^2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm^2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm^2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t15} := 430 : \# \frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} : \# \frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\ 1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\ 1} := 1.5 :$ # *Equation 6.10b give larger values*

> $\gamma_{G,\ 2} := 1.0 :$

> $\gamma_{Q,\ 2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note
The load calculations is in kN/m, kN and kNm
There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\ k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right); \# \frac{kN}{m}$

$$g_{0,k} := 1.517734993 \qquad \textbf{(1)}$$

1. 1 ULS

> $f_{d,\ ULS} := g_{0,\ k} \cdot \gamma_{G,\ 1}; \# \frac{kN}{m}$

$$f_{d,ULS} := 1.821281992 \qquad \textbf{(2)}$$

Modification of the shear force and moment:
The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1 : \#mm^2$

> $A_5 := A_1 : \#mm^2$

> $I_{t1} := \dfrac{\left(b \cdot {h_1}^3\right)}{12} : \#mm^4$

> $I_{t5} := I_{t1} : \#mm^4$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2 : \#mm^2$

> $A_3 := b \cdot h_3 : \ \#mm^2$

> $A_4 := A_2 : \#mm^2$

> $I_{t2} := \dfrac{\left(b \cdot {h_2}^3\right)}{12} : \#mm^4$

> $I_{t3} := \dfrac{\left(b \cdot {h_3}^3\right)}{12} : \#mm^4$

> $I_{t4} := I_{t2} : \#mm^4$

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \dfrac{h_1}{2} + h_2 + \dfrac{h_3}{2} : \#mm$

> $z_2 := \dfrac{h_2}{2} + \dfrac{h_3}{2} : \#mm$

> $z_3 := 0 : \#mm$

> $z_4 := \dfrac{h_4}{2} + \dfrac{h_3}{2} : \#mm$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\, mean,\, t22} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\, mean,\, t15} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\, mean,\, t15} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\text{^}2)_1 := E_{0,\, mean,\, t22} \cdot A_1 \cdot (z_1^{\,2})$: #Nmm^2

> $(EAz\text{^}2)_2 := E_{90,\, mean,\, t15} \cdot A_2 \cdot (z_2^{\,2})$: #Nmm^2

> $(EAz\text{^}2)_3 := E_{0,\, mean,\, t15} \cdot A_3 \cdot (z_3^{\,2})$: #Nmm^2

> $(EAz\text{^}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4^{\,2})$: #Nmm^2

> $(EAz\text{^}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5^{\,2})$: #Nmm^2

> $(EAz\text{^}2)_{sum} := (EAz\text{^}2)_1 + (EAz\text{^}2)_2 + (EAz\text{^}2)_3 + (EAz\text{^}2)_4 + (EAz\text{^}2)_5$: #Nmm^2

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \text{evalf}\left((EI)_{sum} + (EAz\text{^}2)_{sum} \right)$: #Nmm^2

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b} \right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i} \right) + \left(\frac{h_n}{2 \cdot G_n \cdot b} \right) \right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$

$$\text{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\, mean,\, t22} \cdot b} + \frac{h_2}{G_{90,\, mean,\, t15} \cdot b} + \frac{h_3}{G_{0,\, mean,\, t15} \cdot b} + \frac{h_4}{G_{90,\, mean,\, t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\, mean,\, t22} \cdot b} \right) \right)$$: #N

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5 :$
    # CLT handbook US, Ch.3, table 2, pinned− pinned support, uniformly distubuted load

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{}2}} : \# \dfrac{N}{mm^2} \cdot mm^4$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t^3}{12}} : \# \dfrac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum load capacity based on short-term verification of the slab - ULS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm, c} : \# \dfrac{N}{mm^2}$

> $E_2 := E_{CLT} : \# \dfrac{N}{mm^2}$

> $h_1 := h_c : \# mm$

> $h_2 := h_t : \# mm$

> $A_1 := A_c : \# mm^2$

> $A_2 := h_2 \cdot b : \# mm^2$

> $I_1 := I_c : \# mm^4$

> $I_2 := \dfrac{b \cdot h_t\hat{}3}{12} : \# mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227.
Ku with secant value of 60% taken from, EC5: 2.2.2(2), eq.2.1

> $K_{ser} := 1800; \; \# \dfrac{N}{mm}$

(3)

$$K_{ser} := 1800 \quad (3)$$

> $K_u := \mathrm{evalf}\left(\frac{2}{3} \cdot K_{ser}\right); \# \frac{N}{mm}$

$$K_u := 1200. \quad (4)$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 90;

$$angle := 90 \quad (5)$$

> k := sin(convert(angle degrees, radians));

$$k := 1 \quad (6)$$

> $s_{min,\,1} := \mathrm{evalf}(130 \cdot k)$;#mm

$$s_{min,1} := 130. \quad (7)$$

> $s_{max,\,1} := 4 \cdot s_{min,\,1}$; #mm

$$s_{max,1} := 520. \quad (8)$$

> $s_{min} := 130$; #mm

$$s_{min} := 130 \quad (9)$$

> $s_{max} := 520$ #mm

$$s_{max} := 520 \quad (10)$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max}$; #mm

$$s := 227.50 \quad (11)$$

> s := 200; #mm

$$s := 200 \quad (12)$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_1 := \mathrm{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L^2}}\right)$;

$$\gamma_1 := 0.0008374329094 \quad (13)$$

> $\gamma_2 := 1.0$; #` Fully composite

$$\gamma_2 := 1.0 \quad (14)$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 0.2850461357 \tag{15}$$

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 99.71495386 \tag{16}$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm²

$$EI_{eff,tot} := 1.457743416 \times 10^{12} \tag{17}$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.001947637576\, M_{Ed,1} \tag{18}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.9329488200\, M_{Ed,1} \tag{19}$$

Stresses at the top of the concrete section

$\sigma c,t = -\ \sigma 1 - \sigma m,1 = \dfrac{\text{fck}}{\gamma c}$

$M_{Ed,1} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\char`^6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\char`^6 \right) \leq \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := \text{solve}\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 2.495820060 \times 10^7 \tag{20}$$

Stresses at the bottom of concrete section

#σ c,b $= -\ \sigma 1 + \sigma m,1 = \dfrac{\text{fck}}{\gamma c}$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \text{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 1.575364988 \times 10^6 \tag{21}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10^6; \#MPa$$

$$\sigma_2 := 0.001298425050\, M_{Ed,2} \tag{22}$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.2733083990\, M_{Ed,2} \tag{23}$$

Stresses at the top of the timber section

$$\#\sigma\ t, t = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6; \right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

$$> M_3 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_3 := 7.712372467 \times 10^7 \tag{24}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t, b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.814637911 \times 10^7 \quad \textbf{(25)}$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 24.95820060 \quad \textbf{(26)}$$

$$> L_{out} := 0.3; \#\ m$$

$$L_{out} := 0.3 \quad \textbf{(27)}$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \quad \textbf{(28)}$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed\ new}, P_{Ed\ 1}\right); \#kN$$

$$P_{Ed} := 162.1193743 \quad \textbf{(29)}$$

3.6 Verification of the maximum loading

3.6.1 Normal stresses in the concrete section

$$> \sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff\ tot}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.04860952931 \quad \textbf{(30)}$$

$$> \sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 23.28472380 \quad \textbf{(31)}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333333 \tag{32}$$

Verification of the top section

> $\mathrm{Ver}_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.9999999999 \tag{33}$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 23.23611427 \tag{34}$$

> $\mathrm{Ver}_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #> 1 NOT OK

$$Ver_{bottom,c} := 15.84280518 \tag{35}$$

3.6.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{\left(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new}\right)}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.03240635287 \tag{36}$$

> $\sigma_{m,\,2} := \dfrac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new}\right)}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 6.821285850 \tag{37}$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -6.853692203 \tag{38}$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 6.788879497 \tag{39}$$

Verification of the timber section

> $Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.1278587980 \quad \textbf{(40)}$$

3.6.3 Shear stresses in the timber section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \frac{h_2}{2}^2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 5.317030396 \quad \textbf{(41)}$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 1.910807799 \quad \textbf{(42)}$$

3.6.4 The load per shear fastener

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 3.031197936 \quad \textbf{(43)}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 3.031197936 \quad \textbf{(44)}$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad \textbf{(45)}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.07262245053 \quad \textbf{(46)}$$

4. Quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

$$> a_{1,\,eff} := \max\left(\mathrm{solve}\left(a_{1,\,1}{}^{2}\cdot\left(4\cdot\gamma_1{}^{2}\cdot E_1\cdot b\right) + a_{1,\,1}\cdot\left(2\cdot E_2\cdot A_2\cdot\left(1+\gamma_1\right)\right) - E_2\cdot A_2\cdot\left(2\cdot h_1 + h_2\right) = 0,\ a_{1,\,1}\right)\right);\ \#mm$$

$$a_{1,eff} := 139.8816875 \qquad \textbf{(47)}$$

The effective compressed height of the concrete

$$> x := 2\cdot\gamma_1\cdot a_{1,\,eff};\ \#mm$$

$$x := 0.2342830570 \qquad \textbf{(48)}$$

Distance between the centre of the timber and the centre of gravity

$$> a_{2,\,new} := h_1 - 0.5\cdot x + 0.5\cdot h_2 - a_{1,\,eff};$$

$$a_{2,new} := 0.0011710 \qquad \textbf{(49)}$$

$$> A_{1,\,eff} := b\cdot x;$$

$$A_{1,eff} := 140.5698342 \qquad \textbf{(50)}$$

$$> I_{1,\,eff} := \frac{b\cdot x^3}{12};$$

$$I_{1,eff} := 0.6429728740 \qquad \textbf{(51)}$$

New obtained effective bending stiffness

$$> EI_{eff,\,tot,\,new} := E_1\cdot I_{1,\,eff} + \gamma_1\cdot E_1\cdot A_{1,\,eff}\cdot a_{1,\,eff}{}^{2} + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_{2,\,new}{}^{2};\ \#Nmm^2$$

$$EI_{eff,tot,new} := 5.737938049\times 10^{11} \qquad \textbf{(52)}$$

5. New short-term verification

Including the new modified parameters into the verification of the composite

5.1 Verification of the maximum loading using new parameters

5.1.1 Normal stresses in the concrete section

$$> \sigma_1 := \frac{\left(\gamma_1\cdot E_1\cdot a_{1,\,eff}\cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}}\cdot 10^6;\ \#MPa$$

$$\sigma_1 := 0.1732396189 \qquad \textbf{(53)}$$

$$> \sigma_{m,\,1} := \frac{\left(0.5\cdot E_1\cdot x\cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}}\cdot 10^6;\ \#MPa$$

$$\sigma_{m,1} := 0.1732396188 \quad (54)$$

Stresses at the top of the concrete section

> $\sigma_{c,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -0.3464792377 \quad (55)$$

Verification of the top section

> $Ver_{top,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.01484911019 \quad (56)$$

Stresses at the bottom of the concrete section

> $\sigma_{b,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := -1. \times 10^{-10} \quad (57)$$

Verification of the bottom part

> $Ver_{bottom,c} := \frac{\sigma_{b,c}}{\frac{f_{ctk,0.05,c}}{\gamma_c}}$; # < 1 OK

$$Ver_{bottom,c} := -6.818181818 \times 10^{-11} \quad (58)$$

5.1.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,new} \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.0003382183196 \quad (59)$$

> $\sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 17.32971749 \quad (60)$$

Stresses at the top of the timber section

> $\sigma_{t,t} := -\sigma_2 - \sigma_{m,2}$; #MPa

$$\sigma_{t,t} := -17.33005571 \quad (61)$$

Stresses at the bottom of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$; #MPa

$$\sigma_{b,t} := 17.32937927 \quad (62)$$

Verification of the timber section

$$> \ Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right); \#< 1 \text{ OK}$$

$$Ver_{timber} := -0.3156080637 \quad (63)$$

5.1.3 Shear stresses in the timber section

$$> \ \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3; \#MPa$$

$$\tau_2 := 3.377157670 \quad (64)$$

Verification of the timber section

$$> \ Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \#> 1.0 \text{ NOT OK}$$

$$Ver_{shear} := 1.213666038 \quad (65)$$

The results show that failure should occure in the timber section due to shear stresses

5.1.4 The load per shear fastener

$$> \ F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_1 := 0.03163668688 \quad (66)$$

$$> \ F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_2 := 0.03163597820 \quad (67)$$

$$> \ f_{tens,k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \quad (68)$$

$$> \ Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}; \#< 1.0 \text{ OK}$$

$$Ver_{F1} := 0.0007579622900 \quad (69)$$

6. Long-term verification - ULS

6.1 Calculations of the new modulus of elasticity and slip modulus:

6.1.1 Concrete

> $E_{1,g} := \dfrac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \tag{70}$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \tag{71}$$

> $q_k := 0;$

$$q_k := 0 \tag{72}$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \tag{73}$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285711 \tag{74}$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 3589.310984 \tag{75}$$

> $E_{2,q} := \dfrac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 4659.807243 \tag{76}$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 3589.310983 \tag{77}$$

6.1.3 Slip modulus Kser and Ku

> $K_{ser,g} := \frac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 972.9729730 \quad (78)$$

> $K_{ser,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 1263.157895 \quad (79)$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 972.9729728 \quad (80)$$

> $K_u := \frac{2}{3} \cdot K_{ser,2}$

$$K_u := 648.6486485 \quad (81)$$

7. Long-term verification of the maximum loading - ULS

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

> $\gamma_1 := \mathrm{evalf}\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.001583150078 \quad (82)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (83)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)};$ #mm

$$a_2 := 0.2848340000 \quad (84)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2;$ #mm

$$a_1 := 99.71516600 \quad (85)$$

> $EI_{eff,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2;$ #Nmm²

$$EI_{eff,tot} := 5.661631593 \times 10^{11} \quad (86)$$

7.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.002708647996\, M_{Ed,1} \quad (87)$$

> $\sigma_{m,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.6863241135\, M_{Ed,1} \quad (88)$$

Stresses at the top of the concrete section

$\#\sigma\ c,t = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \leq \frac{f_{c,k}}{\gamma_c}$

> $M_1 := \mathrm{solve}\left(M_{Ed,1} = \frac{f_{ck,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right)$; #Nmm

$$M_1 := 3.386389535 \times 10^7 \quad (89)$$

Stresses at the bottom of the concrete section

$\#\sigma\ c,b = -\ \sigma\ 1 + \sigma\ m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{ctk,0.005,c}}{\gamma_c}$

> $M_2 := \mathrm{solve}\left(M_{Ed,1} = \frac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right)$; #Nmm

$$M_2 := 2.145455656 \times 10^6 \quad (90)$$

7.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.001805765331\, M_{Ed,2} \quad (91)$$

$$> \sigma_{m,2} := \frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.3803826785\, M_{Ed,2} \qquad (92)$$

Stresses at the top of the timber section

$$\#\sigma\ t,t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.541436441 \times 10^7 \qquad (93)$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 5.614860398 \times 10^7 \qquad (94)$$

7.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed,\,new} := \dfrac{\min(M_1, M_3, M_4)}{10^6}$; #kNm

$$M_{Ed,new} := 33.86389535 \tag{95}$$

> $L_{out} := 0.3$; #m

$$L_{out} := 0.3 \tag{96}$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \tag{97}$$

> $P_{Ed} := \mathrm{solve}\left(\dfrac{P_{Ed,\,1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\;2}}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right)$; #kN

$$P_{Ed} := 221.4906727 \tag{98}$$

7.4 Verification of the Maximum loading

7.4.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.09172537228 \tag{99}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 23.24160796 \tag{100}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333333 \tag{101}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1 OK

$$Ver_{top,c} := -0.9999999999 \tag{102}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 23.14988259 \tag{103}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1 NOT OK

$$Ver_{bottom,c} := 15.78401085 \qquad (104)$$

7.4.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.06115024821 \qquad (105)$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 12.88123922 \qquad (106)$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -12.94238947 \qquad (107)$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 12.82008897 \qquad (108)$$

Verification of the timber section

> $Ver_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #<1.0 OK

$$Ver_{timber} := -0.2414419893 \qquad (109)$$

7.4.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 10.11014584 \qquad (110)$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 3.633333661 \quad \textbf{(111)}$$

7.4.4 The load per shear fastener

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 5.759426563 \quad \textbf{(112)}$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 5.759426563 \quad \textbf{(113)}$$

> $f_{tens,\, k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad \textbf{(114)}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\, k}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{F1} := 0.1379862614 \quad \textbf{(115)}$$

8. Using quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\, eff} := \max\left(\text{solve}\left(a_{1,\, 1}^{\ 2} \cdot \left(4 \cdot \gamma_1^{\ 2} \cdot E_1 \cdot b\right) + a_{1,\, 1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0,\ a_{1,\, 1}\right)\right)$; #mm

$$a_{1,eff} := 139.7765040 \quad \textbf{(116)}$$

The effective compressed height of the concrete

> $x := 2 \cdot \gamma_1 \cdot a_{1,\, eff}$; #mm

$$x := 0.4425743664 \quad \textbf{(117)}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\, new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\, eff}$;

$$a_{2,new} := 0.0022088 \tag{118}$$

> $A_{1,\,eff} := b \cdot x;$

$$A_{1,eff} := 265.5446198 \tag{119}$$

> $I_{1,\,eff} := \frac{b \cdot x^3}{12}$

$$I_{1,eff} := 4.334397859 \tag{120}$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}{}^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}{}^2;$ #Nmm^2

$$EI_{eff,tot,new} := 3.101963005 \times 10^{11} \tag{121}$$

9. New long-term verification

Including the new modified parameters into the verification of the composite

9.1 Verification of the maximum load using new parameters

9.1.1 Normal stresses in the concrete section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6;$ #MPa

$$\sigma_1 := 0.2346753275 \tag{122}$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6;$ #MPa

$$\sigma_{m,1} := 0.2346753274 \tag{123}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -0.4693506549 \tag{124}$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$ #< 1 OK

$$Ver_{top,c} := -0.02011502806 \tag{125}$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := -1. \times 10^{-10} \quad (126)$$

> $\mathrm{Ver}_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1 OK

$$Ver_{bottom,c} := -6.818181818 \times 10^{-11} \quad (127)$$

9.1.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.0008655014124 \quad (128)$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 23.51054180 \quad (129)$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -23.51140730 \quad (130)$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 23.50967630 \quad (131)$$

Verification of the timber section

> $\mathrm{Ver}_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.428218705 \quad (132)$$

9.1.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 4.613541125 \quad (133)$$

Verification of the timber section

> $$Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \#> 1.0 \;\; \text{NOT OK}$$

$$Ver_{shear} := 1.657991342 \quad \textbf{(134)}$$

Again the verifications show that failure will occure in the timber section due to shear stresses

9.1.4 The load per shear fasteners

> $$F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_1 := 0.08151799017 \quad \textbf{(135)}$$

> $$F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_2 := 0.08151711512 \quad \textbf{(136)}$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad \textbf{(137)}$$

> $$Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}; \#< 1 \; \text{OK}$$

$$Ver_{F1} := 0.001953035181 \quad \textbf{(138)}$$

>

ULS calculation predictions for CTC-screws 7-160 mm with 90 degree orientation and spacing 250 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm "lenght of the span betwen the supports"

> $b := 600$: #mm "width of the composite"

Concrete parameters, concrete class B 35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm "height of concrete"

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \frac{(b \cdot h_c\hat{}3)}{12}$: #mm^4

> $E_{cm,\, c} := 34000$: #MPa

> $f_{ck,\, c} := 35$: #MPa

> $f_{ctk,\, 0.05,\, c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\frac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> $K_{lima} := 1.0$: # Serice class, permanent

> $k_{modi,\,t} := 0.8$: # modification factor, Swedish CLT handbook

> $k_{def,\,t} := 0.85$: # modification factor, Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0,\,mean,\,t22} := 13000$: # $\frac{N}{mm^2}$

> $E_{90,\,mean,\,t22} := 430$: # $\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t22} := 81$: # $\frac{N}{mm^2}$

> $G_{R,\,t22} := G_{90,mean,t22}$: # $\frac{N}{mm^2}$

> $f_{m,\,k,\,t22} := 30.5$: # $\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t22} := 22.0$: # $\frac{N}{mm^2}$

> $f_{v,\,k,\,t22} := 4.0$: # $\frac{N}{mm^2}$

> $t_{t22} := 470$: # $\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855326 1}{1}$: # $\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\,mean,\,t15} := 11500$: # $\frac{N}{mm^2}$

> $E_{90,\,mean,\,t15} := 230$: # $\frac{N}{mm^2}$

> $G_{0,\,mean,\,t15} := 720$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t15} := 430 : \# \frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.0098066355855 3261}{1} : \# \frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # *Equation 6.10b give larger values*

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note
The load calculations is in kN/m, kN and kNm
There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right); \# \frac{kN}{m}$

$$g_{0,k} := 1.517734993 \quad (1)$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k} \cdot \gamma_{G,\,1}; \# \frac{kN}{m}$

$$f_{d,ULS} := 1.821281992 \quad (2)$$

Modification of the shear force and moment:
The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1 : \#mm^2$

> $A_5 := A_1 : \#mm^2$

> $I_{t1} := \dfrac{\left(b \cdot h_1^{\,3}\right)}{12} : \#mm^4$

> $I_{t5} := I_{t1} : \#mm^4$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2 : \#mm^2$

> $A_3 := b \cdot h_3 : \ \#mm^2$

> $A_4 := A_2 : \#mm^2$

> $I_{t2} := \dfrac{\left(b \cdot h_2^{\,3}\right)}{12} : \#mm^4$

> $I_{t3} := \dfrac{\left(b \cdot h_3^{\,3}\right)}{12} : \#mm^4$

> $I_{t4} := \ I_{t2} : \#mm^4$

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \dfrac{h_1}{2} + h_2 + \dfrac{h_3}{2} : \#mm$

> $z_2 := \dfrac{h_2}{2} + \dfrac{h_3}{2} : \#mm$

> $z_3 := 0 : \#mm$

> $z_4 := \dfrac{h_4}{2} + \dfrac{h_3}{2} : \#mm$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot \left(z_1^{\ 2}\right)$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot \left(z_2^{\ 2}\right)$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot \left(z_3^{\ 2}\right)$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot \left(z_4^{\ 2}\right)$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot \left(z_5^{\ 2}\right)$: #Nmm^2

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$: #Nmm^2

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \text{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$: #Nmm^2

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$

$$\text{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$$: #N

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1+\frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5 :$
>   # CLT handbook US, Ch.3, table 2, pinned− pinned support, uniformly distubuted load

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{}2}} : \# \frac{N}{mm^2} \cdot mm^4$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t^3}{12}} : \# \frac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum load capacity based on short-term verification of the slab - ULS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm, c} : \# \frac{N}{mm^2}$

> $E_2 := E_{CLT} : \# \frac{N}{mm^2}$

> $h_1 := h_c : \# mm$

> $h_2 := h_t : \# mm$

> $A_1 := A_c : \# mm^2$

> $A_2 := h_2 \cdot b : \# mm^2$

> $I_1 := I_c : \# mm^4$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12} : \# mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227.
Ku with secant value of 60% taken from, EC5: 2.2.2(2), eq.2.1

> $K_{ser} := 1800; \; \# \frac{N}{mm}$

(3)

$$K_{ser} := 1800 \tag{3}$$

> $K_u := \text{evalf}\left(\frac{2}{3}\cdot K_{ser}\right)$; # $\frac{N}{mm}$

$$K_u := 1200. \tag{4}$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 90;

$$angle := 90 \tag{5}$$

> k := sin(convert(angle degrees, radians));

$$k := 1 \tag{6}$$

> $s_{min,\,1} := \text{evalf}(130\cdot k)$;#mm

$$s_{min,1} := 130. \tag{7}$$

> $s_{max,\,1} := 4\cdot s_{min,\,1}$; #mm

$$s_{max,1} := 520. \tag{8}$$

> $s_{min} := 130$; #mm

$$s_{min} := 130 \tag{9}$$

> $s_{max} := 520$; #mm

$$s_{max} := 520 \tag{10}$$

> $s := 0.75\cdot s_{min} + 0.25\cdot s_{max}$; #mm

$$s := 227.50 \tag{11}$$

> s := 250; #mm

$$s := 250 \tag{12}$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3}\left(E_i I_i + \gamma_i E_i A_i a_i^2\right)$$

> $\gamma_1 := \text{evalf}\left(\frac{1}{1 + \frac{\pi^2\cdot E_1\cdot s\cdot A_1}{K_u\cdot L^2}}\right)$;

$$\gamma_1 := 0.0006700585531 \tag{13}$$

> $\gamma_2 := 1.0$; #` Fully composite

$$\gamma_2 := 1.0 \tag{14}$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 0.2282051190 \quad (15)$$

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 99.77179488 \quad (16)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm^2

$$EI_{eff,tot} := 1.455025868 \times 10^{12} \quad (17)$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.001562171618\, M_{Ed,1} \quad (18)$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.9346912860\, M_{Ed,1} \quad (19)$$

Stresses at the top of the concrete section

$$\# \sigma c,t = - \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\# M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := \text{solve}\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 2.492202634 \times 10^7 \quad (20)$$

Stresses at the bottom of concrete section

$$\# \sigma c,b = - \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6;+\frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6\right)\leq\frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\,1}=\frac{f_{ctk,\,0.05,\,c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\,tot}}+\frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\,tot}}\right)},M_{Ed,\,1}\right);\#Nmm$$

$$M_2 := 1.571772485\times 10^6 \qquad \mathbf{(21)}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,\,2})}{EI_{eff,\,tot}}\cdot 10^6;\#MPa$$

$$\sigma_2 := 0.001041447746\,M_{Ed,2} \qquad \mathbf{(22)}$$

$$> \sigma_{m,\,2} := \frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,\,2})}{EI_{eff,tot}}\cdot 10^6;\#MPa$$

$$\sigma_{m,2} := 0.2738188564\,M_{Ed,2} \qquad \mathbf{(23)}$$

Stresses at the top of the timber section

$$\#\sigma\ t,t\ = -\frac{\sigma_2}{f_{t,\,0,\,d}}-\frac{\sigma_{m,\,2}}{f_{m,\,d}}<1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t}\cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10^6;\right)}{\frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10^6;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\leq 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right),M_{Ed,2}\right);\#Nmm$$

$$M_3 := 7.708052239\times 10^7 \qquad \mathbf{(24)}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.789771042 \times 10^7 \quad (25)$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 24.92202634 \quad (26)$$

$$> L_{out} := 0.3; \#\ m$$

$$L_{out} := 0.3 \quad (27)$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \quad (28)$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed\ new}, P_{Ed\ 1}\right); \#kN$$

$$P_{Ed} := 161.8782126 \quad (29)$$

3.6 Verification of the maximum loading

3.6.1 Normal stresses in the concrete section

$$> \sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff\ tot}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.03893248220 \quad (30)$$

$$> \sigma_{m,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 23.29440086 \quad (31)$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333334 \tag{32}$$

Verification of the top section

> $\mathrm{Ver}_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -1.000000000 \tag{33}$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 23.25546838 \tag{34}$$

> $\mathrm{Ver}_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #> 1 NOT OK

$$Ver_{bottom,c} := 15.85600118 \tag{35}$$

3.6.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.02595498815 \tag{36}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 6.824120750 \tag{37}$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -6.850075738 \tag{38}$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 6.798165762 \tag{39}$$

Verification of the timber section

> $Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.1271848225 \quad \textbf{(40)}$$

3.6.3 Shear stresses in the timber section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \frac{h_2^{\,2}}{2}}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 5.319036845 \quad \textbf{(41)}$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 1.911528866 \quad \textbf{(42)}$$

3.6.4 The load per shear fastener

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 3.034578592 \quad \textbf{(43)}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 3.034578592 \quad \textbf{(44)}$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad \textbf{(45)}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.07270344543 \quad \textbf{(46)}$$

4. Quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

$$\sigma_{m,1} := 0.1384443662 \quad (54)$$

Stresses at the top of the concrete section

> $\sigma_{c,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -0.2768887324 \quad (55)$$

Verification of the top section

> $\mathrm{Ver}_{top,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.01186665996 \quad (56)$$

Stresses at the bottom of the concrete section

> $\sigma_{b,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 0. \quad (57)$$

Verification of the bottom part

> $\mathrm{Ver}_{bottom,c} := \frac{\sigma_{b,c}}{\frac{f_{ctk,0.05,c}}{\gamma_c}}$; # < 1 OK

$$Ver_{bottom,c} := 0. \quad (58)$$

5.1.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,new} \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.0002163181158 \quad (59)$$

> $\sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 17.30544926 \quad (60)$$

Stresses at the top of the timber section

> $\sigma_{t,t} := -\sigma_2 - \sigma_{m,2}$; #MPa

$$\sigma_{t,t} := -17.30566558 \quad (61)$$

Stresses at the bottom of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$; #MPa

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(\text{solve}\left(a_{1,\,1}^{\;2}\cdot\left(4\cdot\gamma_1^{\;2}\cdot E_1\cdot b\right) + a_{1,\,1}\cdot\left(2\cdot E_2\cdot A_2\cdot\left(1+\gamma_1\right)\right) - E_2\cdot A_2\cdot\left(2\cdot h_1 + h_2\right) = 0,\; a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 139.9055051 \tag{47}$$

The effective compressed height of the concrete

> $x := 2\cdot\gamma_1\cdot a_{1,\,eff}$; #mm

$$x := 0.1874897606 \tag{48}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5\cdot x + 0.5\cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 0.0007500 \tag{49}$$

> $A_{1,\,eff} := b\cdot x$;

$$A_{1,eff} := 112.4938564 \tag{50}$$

> $I_{1,\,eff} := \dfrac{b\cdot x^3}{12}$;

$$I_{1,eff} := 0.3295358498 \tag{51}$$

New obtained effective bending stiffness

> $EI_{eff,\,tot,\,new} := E_1\cdot I_{1,\,eff} + \gamma_1\cdot E_1\cdot A_{1,\,eff}\cdot a_{1,\,eff}^{\;2} + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_{2,\,new}^{\;2}$; #Nmm²

$$EI_{eff,tot,new} := 5.737656430 \times 10^{11} \tag{52}$$

5. New short-term verification

Including the new modified parameters into the verification of the composite

5.1 Verification of the maximum loading using new parameters

5.1.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{\left(\gamma_1\cdot E_1\cdot a_{1,\,eff}\cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}}\cdot 10^6$; #MPa

$$\sigma_1 := 0.1384443662 \tag{53}$$

> $\sigma_{m,\,1} := \dfrac{\left(0.5\cdot E_1\cdot x\cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}}\cdot 10^6$; #MPa

$$\sigma_{b,t} := 17.30523294 \tag{62}$$

Verification of the timber section

$$> \ Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right); \#< 1 \text{ OK}$$

$$Ver_{timber} := -0.3151524348 \tag{63}$$

5.1.3 Shear stresses in the timber section

$$> \ \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3; \#MPa$$

$$\tau_2 := 3.372252149 \tag{64}$$

Verification of the timber section

$$> \ Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \#> 1.0 \text{ NOT OK}$$

$$Ver_{shear} := 1.211903115 \tag{65}$$

The results show that failure should occure in the timber section due to shear stresses

5.1.4 The load per shear fastener

$$> \ F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_1 := 0.02529001873 \tag{66}$$

$$> \ F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_2 := 0.02529125883 \tag{67}$$

$$> \ f_{tens,\,k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \tag{68}$$

$$> \ Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}; \#< 1.0 \text{ OK}$$

$$Ver_{F1} := 0.0006059066987 \tag{69}$$

6. Long-term verification - ULS

6.1 Calculations of the new modulus of elasticity and slip modulus:

6.1.1 Concrete

> $E_{1,g} := \dfrac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \tag{70}$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \tag{71}$$

> $q_k := 0;$

$$q_k := 0 \tag{72}$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \tag{73}$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285711 \tag{74}$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 3589.310984 \tag{75}$$

> $E_{2,q} := \dfrac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 4659.807243 \tag{76}$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 3589.310983 \tag{77}$$

6.1.3 Slip modulus Kser and Ku

> $K_{ser,\,g} := \dfrac{K_{ser}}{1 + k_{def,\,t}};$

$$K_{ser,g} := 972.9729730 \tag{78}$$

> $K_{ser,\,q} := \dfrac{K_{ser}}{1 + k_{def,t}\cdot\psi_2};$

$$K_{ser,q} := 1263.157895 \tag{79}$$

> $K_{ser,\,2} := \dfrac{K_{ser,g}\cdot\left(g_{0,k} + g_{1,k}\right)\cdot\gamma_{G,1} + K_{ser,q}\cdot q_k\cdot\gamma_{Q,1}}{\left(g_{0,\,k} + g_{1,k}\right)\cdot\gamma_{G,\,1} + q_k\cdot\gamma_{Q,\,1}};$

$$K_{ser,2} := 972.9729728 \tag{80}$$

> $K_u := \dfrac{2}{3}\cdot K_{ser,\,2}$

$$K_u := 648.6486485 \tag{81}$$

7. Long-term verification of the maximum loading - ULS

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

> $\gamma_1 := \mathrm{evalf}\left(\dfrac{1}{1 + \dfrac{\pi\hat{\ }2\cdot E_1\cdot s\cdot A_1}{K_u\cdot L\hat{\ }2}}\right);$

$$\gamma_1 := 0.001266921208 \tag{82}$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \tag{83}$$

> $a_2 := \dfrac{\gamma_1\cdot E_1\cdot A_1\cdot\left(h_1 + h_2\right)}{2\cdot\left(\gamma_1\cdot E_1\cdot A_1 + \gamma_2\cdot E_2\cdot A_2\right)};$ #mm

$$a_2 := 0.2280691317 \tag{84}$$

>

> $a_1 := \dfrac{\left(h_1 + h_2\right)}{2} - a_2;$ #mm

$$a_1 := 99.77193087 \tag{85}$$

> $EI_{eff,\,tot} := E_1\cdot I_1 + \gamma_1\cdot E_1\cdot A_1\cdot a_1^2 + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_2^2;$ #Nmm²

$$EI_{eff,tot} := 5.646961826\times 10^{11} \tag{86}$$

7.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.002174472923\, M_{Ed,1} \tag{87}$$

> $\sigma_{m,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.6881070575\, M_{Ed,1} \tag{88}$$

Stresses at the top of the concrete section

$\#\sigma\ c,t\ = -\ \sigma 1 - \sigma\ m,1 = \dfrac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \leq \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := \mathrm{solve}\left(M_{Ed,1} = \dfrac{f_{ck,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right)$; #Nmm

$$M_1 := 3.380263313 \times 10^7 \tag{89}$$

Stresses at the bottom of the concrete section

$\#\sigma\ c,b\ = -\ \sigma\ 1 + \sigma\ m,1 = \dfrac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \dfrac{f_{ctk,0.005,c}}{\gamma_c}$

> $M_2 := \mathrm{solve}\left(M_{Ed,1} = \dfrac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right)$; #Nmm

$$M_2 := 2.138208185 \times 10^6 \tag{90}$$

7.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.001449648616\, M_{Ed,2} \tag{91}$$

> $$\sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.3813708426\, M_{Ed,2} \tag{92}$$

Stresses at the top of the timber section

$$\#\sigma\ t, t\ = -\ \frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

> $$M_3 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.534289488 \times 10^7 \tag{93}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t, b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $$M_4 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 5.592927465 \times 10^7 \tag{94}$$

7.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed,\,new} := \dfrac{\min(M_1, M_3, M_4)}{10^6}$; #kNm

$$M_{Ed,new} := 33.80263313 \tag{95}$$

> $L_{out} := 0.3$; #m

$$L_{out} := 0.3 \tag{96}$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \tag{97}$$

> $P_{Ed} := \text{solve}\left(\dfrac{P_{Ed,\,1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right)$; #kN

$$P_{Ed} := 221.0822579 \tag{98}$$

7.4 Verification of the Maximum loading

7.4.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.07350291048 \tag{99}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 23.25983041 \tag{100}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333332 \tag{101}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1 OK

$$Ver_{top,c} := -0.9999999994 \tag{102}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 23.18632750 \tag{103}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1 NOT OK

$$Ver_{bottom,c} := 15.80885966 \tag{104}$$

7.4.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.04900194032 \tag{105}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 12.89133868 \tag{106}$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -12.94034062 \tag{107}$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 12.84233674 \tag{108}$$

Verification of the timber section

> $Ver_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #<1.0 OK

$$Ver_{timber} := -0.2402595529 \tag{109}$$

7.4.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{\ }2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 10.11771924 \tag{110}$$

Verification of the timber section

$$> Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \#> 1 \text{ NOT OK}$$

$$Ver_{shear} := 3.636055351 \tag{111}$$

7.4.4 The load per shear fastener

$$> F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_1 := 5.768848601 \tag{112}$$

$$> F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_2 := 5.768848604 \tag{113}$$

$$> f_{tens,\,k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \tag{114}$$

$$> Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}; \#> 1 \text{ NOT OK}$$

$$Ver_{F1} := 0.1382119977 \tag{115}$$

8. Using quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

$$> a_{1,\,eff} := \max\left(\text{solve}\left(a_{1,\,1}^{\,2} \cdot \left(4 \cdot \gamma_1^{\,2} \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right); \#mm$$

$$a_{1,eff} := 139.8214418 \tag{116}$$

The effective compressed height of the concrete

$$> x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}; \#mm$$

$$x := 0.3542854998 \tag{117}$$

Distance between the centre of the timber and the centre of gravity

$$> a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff};$$

$$a_{2,new} := 0.0014154 \quad \textbf{(118)}$$

> $A_{1,\,eff} := b \cdot x;$

$$A_{1,eff} := 212.5712999 \quad \textbf{(119)}$$

> $I_{1,\,eff} := \dfrac{b \cdot x^3}{12}$

$$I_{1,eff} := 2.223464183 \quad \textbf{(120)}$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}{}^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}{}^2;$ #Nmm^2

$$EI_{eff,tot,new} := 3.101676371 \times 10^{11} \quad \textbf{(121)}$$

9. New long-term verification

Including the new modified parameters into the verification of the composite

9.1 Verification of the maximum load using new parameters

9.1.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6;$ #MPa

$$\sigma_1 := 0.1875375790 \quad \textbf{(122)}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6;$ #MPa

$$\sigma_{m,1} := 0.1875375790 \quad \textbf{(123)}$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -0.3750751580 \quad \textbf{(124)}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}};$ #< 1 OK

$$Ver_{top,c} := -0.01607464964 \quad \textbf{(125)}$$

Stresses at the bottom of the concrete section

> $\sigma_{b,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 0. \quad (126)$$

> $Ver_{bottom,c} := \dfrac{\sigma_{b,c}}{\dfrac{f_{ctk,0.05,c}}{\gamma_c}}$; #<1 OK

$$Ver_{bottom,c} := 0. \quad (127)$$

9.1.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,new} \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.0005536615058 \quad (128)$$

> $\sigma_{m,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 23.47017828 \quad (129)$$

Stresses at the top of the timber section

> $\sigma_{t,t} := -\sigma_2 - \sigma_{m,2}$; #MPa

$$\sigma_{t,t} := -23.47073194 \quad (130)$$

Stresses at the bottom of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$; #MPa

$$\sigma_{b,t} := 23.46962462 \quad (131)$$

Verification of the timber section

> $Ver_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.427448621 \quad (132)$$

9.1.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 4.605337816 \quad (133)$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1.0 NOT OK

$$Ver_{shear} := 1.655043278 \tag{134}$$

Again the verifications show that failure will occure in the timber section due to shear stresses

9.1.4 The load per shear fasteners

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_1 := 0.06518329374 \tag{135}$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 0.06518087619 \tag{136}$$

> $f_{tens,\, k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{137}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\, k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.001561683079 \tag{138}$$

>

Appendix A.5 Load capacity for type E

ULS calculation predictions for CTC-screws 7-160 mm with 90 degree orientation and spacing 125 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm "lenght of the span betwen the supports"

> $b := 600$: #mm "width of the composite"

Concrete parameters, concrete class B 35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm "height of concrete"

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \dfrac{(b \cdot h_c\hat{}3)}{12}$: #mm^4

> $E_{cm,\,c} := 34000$: #MPa

> $f_{ck,\,c} := 35$: #MPa

> $f_{ctk,\,0.05,\,c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\dfrac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> $K_{lima} := 1.0$: # Serice class, permanent

> $k_{modi,\, t} := 0.8$: # modification factor, Swedish CLT handbook

> $k_{def,\, t} := 0.85$: # modification factor, Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0,\, mean,\, t22} := 13000$: # $\frac{N}{mm^2}$

> $E_{90,\, mean,\, t22} := 430$: # $\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: # $\frac{N}{mm^2}$

> $G_{90,\, mean,\, t22} := 81$: # $\frac{N}{mm^2}$

> $G_{R,\, t22} := G_{90,mean,t22}$: # $\frac{N}{mm^2}$

> $f_{m,\, k,\, t22} := 30.5$: # $\frac{N}{mm^2}$

> $f_{t,\, 0,\, k,\, t22} := 22.0$: # $\frac{N}{mm^2}$

> $f_{v,\, k,\, t22} := 4.0$: # $\frac{N}{mm^2}$

> $t_{t22} := 470$: # $\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}$: # $\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\, mean,\, t15} := 11500$: # $\frac{N}{mm^2}$

> $E_{90,\, mean,\, t15} := 230$: # $\frac{N}{mm^2}$

> $G_{0,\, mean,\, t15} := 720$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 : \#\frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \#\frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \#\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \#\frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \#\frac{N}{mm^2}$

> $t_{t15} := 430 : \#\frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.0098066355855326 1}{1} : \#\frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # *Equation 6.10b give larger values*

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note
The load calculations is in kN/m, kN and kNm
There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right); \#\frac{kN}{m}$

$$g_{0,k} := 1.517734993 \qquad (1)$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k} \cdot \gamma_{G,\,1}; \#\frac{kN}{m}$

$$f_{d,ULS} := 1.821281992 \qquad (2)$$

Modification of the shear force and moment:
The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1 : \#mm^2$

> $A_5 := A_1 : \#mm^2$

> $I_{t1} := \dfrac{\left(b \cdot h_1^{\,3}\right)}{12} : \#mm^4$

> $I_{t5} := I_{t1} : \#mm^4$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2 : \#mm^2$

> $A_3 := b \cdot h_3 : \ \#mm^2$

> $A_4 := A_2 : \#mm^2$

> $I_{t2} := \dfrac{\left(b \cdot h_2^{\,3}\right)}{12} : \#mm^4$

> $I_{t3} := \dfrac{\left(b \cdot h_3^{\,3}\right)}{12} : \#mm^4$

> $I_{t4} := \ I_{t2} : \#mm^4$

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \dfrac{h_1}{2} + h_2 + \dfrac{h_3}{2} : \#mm$

> $z_2 := \dfrac{h_2}{2} + \dfrac{h_3}{2} : \#mm$

> $z_3 := 0 : \#mm$

> $z_4 := \dfrac{h_4}{2} + \dfrac{h_3}{2} : \#mm$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot \left(z_1^2\right)$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot \left(z_2^2\right)$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot \left(z_3^2\right)$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot \left(z_4^2\right)$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot \left(z_5^2\right)$: #Nmm^2

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$: #Nmm^2

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \mathrm{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$: #Nmm^2

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$
$$\mathrm{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$$: #N

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5 :$
CLT handbook US, Ch.3, table 2, pinned − pinned support, uniformly distubuted load

> $$EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{}2}} : \# \frac{N}{mm^2} \cdot mm^4$$

> $$E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t^3}{12}} : \# \frac{N}{mm^2}$$

3. γ-method from, EC5, Annex B, Maximum load capacity based on short-term verification of the slab - ULS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm, c} : \# \frac{N}{mm^2}$

> $E_2 := E_{CLT} : \# \frac{N}{mm^2}$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm^2$

> $A_2 := h_2 \cdot b : \#mm^2$

> $I_1 := I_c : \#mm^4$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12} : \#mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227. The formula is multiplied by 3, beacause there are 3 pairs of screws in each row.
Ku with secant value of 60% taken from, EC5: 2.2.2(2), eq.2.1

> $K_{ser} := 1800; \# \frac{N}{mm}$

(3)

$$K_{ser} := 1800 \quad (3)$$

> $K_u := \text{evalf}\left(\frac{2}{3} \cdot K_{ser}\right); \# \frac{N}{mm}$

$$K_u := 1200. \quad (4)$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> $angle := 90;$

$$angle := 90 \quad (5)$$

> $k := \sin(\text{convert}(angle\ degrees, radians));$

$$k := 1 \quad (6)$$

> $s_{min, 1} := \text{evalf}(130 \cdot k);\#mm$

$$s_{min,1} := 130. \quad (7)$$

> $s_{max, 1} := 4 \cdot s_{min, 1}; \#mm$

$$s_{max,1} := 520. \quad (8)$$

> $s_{min} := 130; \#mm$

$$s_{min} := 130 \quad (9)$$

> $s_{max} := 520 \#mm$

$$s_{max} := 520 \quad (10)$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max}; \#mm$

$$s := 227.50 \quad (11)$$

> $s := 125; \#mm$

$$s := 125 \quad (12)$$

The spacing does not satisfy the minimum spacing. We did not know the slip modulus before we chose the spacing. As for why the spacing is 125, we wanted to see the difference in capacity of 90 degree and 45 degree orientation in screws. Therefore we took as many screws in Type B as Type E the orientation of the screws is different. To see what the outcome would be.

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3}(E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_1 := \text{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L^2}}\right);$

$$\gamma_1 := 0.001339219751 \quad (13)$$

> $\gamma_2 := 1.0$; #` Fully composite

$$\gamma_2 := 1.0 \tag{14}$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 0.4550675245 \tag{15}$$

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 99.54493248 \tag{16}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^{\,2} + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^{\,2}$; #Nmm²

$$EI_{eff,tot} := 1.465872074 \times 10^{12} \tag{17}$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.003092102257\, M_{Ed,1} \tag{18}$$

> $\sigma_{m,\ 1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.9277753660\, M_{Ed,1} \tag{19}$$

Stresses at the top of the concrete section

$\sigma c,t \;=\; -\;\; \sigma 1 - \sigma m,1 = \dfrac{\text{fck}}{\gamma \text{c}}$

$M_{Ed,1} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{\textasciicircum}6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{\textasciicircum}6 \right) \le \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := \text{solve}\left(M_{Ed,\ 1} = \dfrac{f_{ck,\ c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 2.506622492 \times 10^7 \tag{20}$$

Stresses at the bottom of concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1\ +\ \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6;+\frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6\right)\leq\frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> \ M_2 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\,tot}}+\frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\,tot}}\right)}, M_{Ed,\,1}\right);\#Nmm$$

$$M_2 := 1.586128704\times 10^6 \tag{21}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \ \sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,\,2})}{EI_{eff,\,tot}}\cdot 10^6;\#MPa$$

$$\sigma_2 := 0.002061401505\,M_{Ed,2} \tag{22}$$

$$> \ \sigma_{m,\,2} := \frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,\,2})}{EI_{eff,tot}}\cdot 10^6;\#MPa$$

$$\sigma_{m,2} := 0.2717928299\,M_{Ed,2} \tag{23}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t}\cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10^6;\right)}{\frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10^6;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\leq 1.0$$

> $M_3 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_3 := 7.725227970 \times 10^7 \quad (24)$$

Stresses at the bottom of the timber section

$\#\sigma \text{ t, b } = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 7.889413075 \times 10^7 \quad (25)$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 25.06622492 \quad (26)$$

> $L_{out} := 0.3$; # m

$$L_{out} := 0.3 \quad (27)$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \quad (28)$$

> $P_{Ed} := \text{solve}\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed} := 162.8395365 \quad (29)$$

3.6 Verification of the maximum loading

3.6.1 Normal stresses in the concrete section

> $$\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6;$$ #MPa

$$\sigma_1 := 0.07750733063 \quad (30)$$

> $$\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6;$$ #MPa

$$\sigma_{m,1} := 23.25582600 \quad (31)$$

Stresses at the top of the concrete section

> $$\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$$ #MPa

$$\sigma_{c,t} := -23.33333333 \quad (32)$$

Verification of the top section

> $$Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$$ #< 1 OK

$$Ver_{top,c} := -0.9999999999 \quad (33)$$

Stresses at the bottom of the concrete section

> $$\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1};$$ #MPa

$$\sigma_{b,c} := 23.17831867 \quad (34)$$

> $$Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};$$ #> 1 NOT OK

$$Ver_{bottom,c} := 15.80339910 \quad (35)$$

3.6.2 Normal stresses in the timber section

> $$\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10^6;$$ #MPa

$$\sigma_2 := 0.05167155377 \quad (36)$$

> $$\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6;$$ #MPa

$$\sigma_{m,2} := 6.812820205 \quad (37)$$

Stresses at the top of the timber section

> $$\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2};$$ #MPa

$$\sigma_{t,t} := -6.864491759 \tag{38}$$

Stresses at the bottom of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$; #MPa

$$\sigma_{b,t} := 6.761148651 \tag{39}$$

Verification of the timber section

> $Ver_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.1298714374 \tag{40}$$

3.6.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2^{\,2}}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 5.311034212 \tag{41}$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 1.908652920 \tag{42}$$

3.6.4 The load per shear fastener

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 3.021098990 \tag{43}$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 3.021098990 \tag{44}$$

> $f_{tens,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{45}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.07238049663 \tag{46}$$

4. Quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(\mathrm{solve}\left(a_{1,\,1}{}^{2}\cdot\left(4\cdot\gamma_1{}^{2}\cdot E_1\cdot b\right)+a_{1,\,1}\cdot\left(2\cdot E_2\cdot A_2\cdot\left(1+\gamma_1\right)\right)-E_2\cdot A_2\cdot\left(2\cdot h_1+h_2\right)=0,\ a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 139.8097723 \tag{47}$$

The effective compressed height of the concrete

> $x := 2\cdot\gamma_1\cdot a_{1,\,eff}$; #mm

$$x := 0.3744720168 \tag{48}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5\cdot x + 0.5\cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 0.0029917 \tag{49}$$

> $A_{1,\,eff} := b\cdot x$;

$$A_{1,eff} := 224.6832101 \tag{50}$$

> $I_{1,\,eff} := \dfrac{b\cdot x^3}{12}$;

$$I_{1,eff} := 2.625597278 \tag{51}$$

New obtained effective bending stiffness

> $EI_{eff,\,tot,\,new} := E_1\cdot I_{1,\,eff} + \gamma_1\cdot E_1\cdot A_{1,\,eff}\cdot a_{1,\,eff}{}^{2} + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_{2,\,new}{}^{2}$; #Nmm^2

$$EI_{eff,tot,new} := 5.739155366\times 10^{11} \tag{52}$$

5. New short-term verification

Including the new modified parameters into the verification of the composite

5.1 Verification of the maximum loading using new parameters

5.1.1 Normal stresses in the concrete section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.2780412559 \quad (53)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.2780412558 \quad (54)$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -0.5560825117 \quad (55)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.02383210764 \quad (56)$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := -1. \times 10^{-10} \quad (57)$$

Verification of the bottom part

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; # < 1 OK

$$Ver_{bottom,c} := -6.818181818 \times 10^{-11} \quad (58)$$

5.1.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.0008676444803 \quad (59)$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 17.40103246 \tag{60}$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -17.40190010 \tag{61}$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 17.40016482 \tag{62}$$

Verification of the timber section

> $\mathrm{Ver}_{\mathrm{timber}} := \left(\dfrac{\sigma_{\mathrm{t,t}}}{\dfrac{k_{\mathrm{modi,t}} \cdot f_{\mathrm{t,0,k,t22}}}{\gamma_{\mathrm{M}}}} + \dfrac{\sigma_{\mathrm{b,t}}}{\dfrac{k_{\mathrm{modi,t}} \cdot f_{\mathrm{m,k,t22}}}{\gamma_{\mathrm{M}}}} \right)$; #< 1 OK

$$Ver_{timber} := -0.3169662368 \tag{63}$$

5.1.3 Shear stresses in the timber section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,\mathrm{new}}\right)^2}{b \cdot EI_{\mathrm{eff,tot,new}}} \cdot P_{\mathrm{Ed}} \cdot 10^3$; #MPa

$$\tau_2 := 3.391645911 \tag{64}$$

Verification of the timber section

> $\mathrm{Ver}_{\mathrm{shear}} := \dfrac{\tau_2}{\dfrac{k_{\mathrm{modi,t}} \cdot f_{\mathrm{v,k,t22}}}{\gamma_{\mathrm{M}}}}$; #> 1.0 NOT OK

$$Ver_{shear} := 1.218872749 \tag{65}$$

The results show that failure should occure in the timber section due to shear stresses

5.1.4 The load per shear fastener

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_{1,\mathrm{eff}} \cdot a_{1,\mathrm{eff}} \cdot s}{EI_{\mathrm{eff,tot,new}}} \cdot P_{\mathrm{Ed}}$; #kN

$$F_1 := 0.05072952548 \tag{66}$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\mathrm{new}} \cdot s}{EI_{\mathrm{eff,tot,new}}} \cdot P_{\mathrm{Ed}}$; #kN

$$F_2 := 0.05072887640 \tag{67}$$

> $f_{tens,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{68}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; #< 1.0 OK

$$Ver_{F1} := 0.001215394881 \tag{69}$$

6. Long-term verification - ULS

6.1 Calculations of the new modulus of elasticity and slip modulus:

6.1.1 Concrete

> $E_{1,g} := \dfrac{E_{cm,c}}{1 + \varphi_c}$;

$$E_{1,g} := 9714.285714 \tag{70}$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \tag{71}$$

> $q_k := 0$;

$$q_k := 0 \tag{72}$$

> $g_{1,k} := 0$;

$$g_{1,k} := 0 \tag{73}$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_1 := 9714.285711 \tag{74}$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1 + k_{def,t}}$;

$$E_{2,g} := 3589.310984 \tag{75}$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2}$;

$$E_{2,q} := 4659.807243 \tag{76}$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 3589.310983 \quad (77)$$

6.1.3 Slip modulus Kser and Ku

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 972.9729730 \quad (78)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 1263.157895 \quad (79)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 972.9729728 \quad (80)$$

> $K_u := \dfrac{2}{3} \cdot K_{ser,2}$

$$K_u := 648.6486485 \quad (81)$$

7. Long-term verification of the maximum loading - ULS

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

> $\gamma_1 := \mathrm{evalf}\left(\dfrac{1}{1 + \dfrac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.002530636297 \quad (82)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (83)$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)};$ #mm

$$a_2 := 0.4545270902 \quad (84)$$

>

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2;$ #mm

(85)

$$a_1 := 99.54547291 \quad \textbf{(85)}$$

$$> EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2; \#Nmm^2$$

$$EI_{eff,tot} := 5.705485445 \times 10^{11} \quad \textbf{(86)}$$

7.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_1 := \frac{\left(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1}\right)}{EI_{eff,\,tot}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.004289133043\, M_{Ed,1} \quad \textbf{(87)}$$

$$> \sigma_{m,\,1} := \frac{\left(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 0.6810488470\, M_{Ed,1} \quad \textbf{(88)}$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot}} \cdot 10^6; + \frac{\left(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot}} \cdot 10^6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{\left(\gamma_1 \cdot E_1 \cdot a_1\right)}{EI_{eff,\,tot}} + \frac{\left(0.5 \cdot E_1 \cdot h_1\right)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 3.404646176 \times 10^7 \quad \textbf{(89)}$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1\ + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot}} \cdot 10\hat{\ }6; + \frac{\left(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{\left(\gamma_1 \cdot E_1 \cdot a_1\right)}{EI_{eff,\,tot}} + \frac{\left(0.5 \cdot E_1 \cdot h_1\right)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

(90)

$$M_2 := 2.167189678 \times 10^6 \quad (90)$$

7.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_2 := 0.002859422029\, M_{Ed,2} \quad (91)$$

$$> \sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.3774589578\, M_{Ed,2} \quad (92)$$

Stresses at the top of the timber section

$$\#\sigma\ t, t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.562690919 \times 10^7 \quad (93)$$

Stresses at the bottom of the timber section

$$\#\sigma\ t, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 5.6807735 \times 10^7 \quad (94)$$

7.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed\ new} := \frac{\min(M_1, M_3, M_4)}{10^6}$; #kNm

$$M_{Ed,new} := 34.04646176 \quad (95)$$

> $L_{out} := 0.3$; #m

$$L_{out} := 0.3 \quad (96)$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \quad (97)$$

> $P_{Ed} := \text{solve}\left(\frac{P_{Ed\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{2}}{8} = M_{Ed\ new}, P_{Ed\ 1}\right)$; #kN

$$P_{Ed} := 222.7077821 \quad (98)$$

7.4 Verification of the Maximum loading

7.4.1 Normal stresses in the concrete section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.1460298041 \quad (99)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 23.18730353 \quad (100)$$

Stresses at the top of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.33333333 \quad (101)$$

Verification of the top section

> $Ver_{top,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}}$; #<1 OK

$$Ver_{top,c} := -0.9999999999 \quad \textbf{(102)}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 23.04127373 \quad \textbf{(103)}$$

> $Ver_{bottom,c} := \frac{\sigma_{b,c}}{\frac{f_{ctk,0.05,c}}{\gamma_c}}$; #<1 NOT OK

$$Ver_{bottom,c} := 15.70995936 \quad \textbf{(104)}$$

7.4.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.09735320278 \quad \textbf{(105)}$$

> $\sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 12.85114198 \quad \textbf{(106)}$$

Stresses at the top of the timber section

> $\sigma_{t,t} := -\sigma_2 - \sigma_{m,2}$; #MPa

$$\sigma_{t,t} := -12.94849518 \quad \textbf{(107)}$$

Stresses at the bottom of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$; #MPa

$$\sigma_{b,t} := 12.75378878 \quad \textbf{(108)}$$

Verification of the timber section

> $Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #<1.0 OK

$$Ver_{timber} := -0.2449657458 \quad \textbf{(109)}$$

7.4.3 Shear stresses in the timber section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 10.08756568 \tag{110}$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{shear} := 3.625218916 \tag{111}$$

7.4.4 The load per shear fastener

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 5.731339843 \tag{112}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 5.731339843 \tag{113}$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{114}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #> 1 NOT OK

$$Ver_{F1} := 0.1373133504 \tag{115}$$

8. Using quadratic equation

Verification of both timber and concrete section are not OK. By following " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402", on page 134. Modifications are done by considering only the effective compressed height of the concrete this is done by using the quadratic equation.

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(\mathrm{solve}\left(a_{1,\,1}{}^2 \cdot \left(4 \cdot \gamma_1{}^2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 139.6409865 \tag{116}$$

The effective compressed height of the concrete

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 0.7067610980 \qquad (117)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 0.0056329 \qquad (118)$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 424.0566588 \qquad (119)$$

> $I_{1,\,eff} := \dfrac{b \cdot x^3}{12}$

$$I_{1,eff} := 17.65175597 \qquad (120)$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}^{\;2} + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}^{\;2}$; #Nmm^2

$$EI_{eff,tot,new} := 3.103199265 \times 10^{11} \qquad (121)$$

9. New long-term verification

Including the new modified parameters into the verification of the composite

9.1 Verification of the maximum load using new parameters

9.1.1 Normal stresses in the concrete section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.3766308020 \qquad (122)$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.3766308018 \qquad (123)$$

Stresses at the top of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -0.7532616038 \qquad (124)$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #< 1 OK

$$Ver_{top,c} := -0.03228264016 \tag{125}$$

Stresses at the bottom of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := -2. \times 10^{-10} \tag{126}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1 OK

$$Ver_{bottom,c} := -1.363636364 \times 10^{-10} \tag{127}$$

9.1.2 Normal stresses in the timber section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.002218224259 \tag{128}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 23.62787473 \tag{129}$$

Stresses at the top of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -23.63009295 \tag{130}$$

Stresses at the bottom of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 23.62565651 \tag{131}$$

Verification of the timber section

> $Ver_{timber} := \left(\dfrac{\sigma_{t,t}}{\dfrac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \dfrac{\sigma_{b,t}}{\dfrac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right)$; #< 1 OK

$$Ver_{timber} := -0.430507453 \tag{132}$$

9.1.3 Shear stresses in the timber section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)^2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^3$; #MPa

$$\tau_2 := 4.637574112 \tag{133}$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #> 1.0 NOT OK

$$Ver_{shear} := 1.666628196 \tag{134}$$

Again the verifications show that failure will occure in the timber section due to shear stresses

9.1.4 The load per shear fasteners

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_1 := 0.1305909686 \tag{135}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 0.1305904348 \tag{136}$$

> $f_{tens,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{137}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; #< 1 OK

$$Ver_{F1} := 0.003128741957 \tag{138}$$

>

Appendix B.

SLS, maximum deflection

Appendix B.1 Maximum deflection for type A

SLS deflection predictions for CTC-screws 7-160 mm 45 degree orientation and spacing 200 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm span length between the supports

> $b := 600$: #mm

Concrete parameters, concrete class B35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \frac{\left(b \cdot h_c^3\right)}{12}$: #mm^4

> $E_{cm,\,c} := 34000$: #MPa

> $f_{ck,\,c} := 35$:#MPa

> $f_{ctk,\,0.05,\,c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\frac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> Klima := 1.0 : #` `Serice class, permanent

> $k_{modi, t} := 0.8$: # modification factor,Swedish CLT handbook

> $k_{def, t} := 0.85$: # modification factor,Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0, mean, t22} := 13000$: # $\frac{N}{mm^2}$

> $E_{90, mean, t22} := 430$: # $\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: # $\frac{N}{mm^2}$

> $G_{90, mean, t22} := 81$: # $\frac{N}{mm^2}$

> $G_{R, t22} := G_{90,mean,t22}$: # $\frac{N}{mm^2}$

> $f_{m, k, t22} := 30.5$: # $\frac{N}{mm^2}$

> $f_{t, 0, k, t22} := 22.0$: # $\frac{N}{mm^2}$

> $f_{v, k, t22} := 4.0$: # $\frac{N}{mm^2}$

> $t_{t22} := 470$: # $\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855326}{1}$: # $\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0, mean, t15} := 11500$: # $\frac{N}{mm^2}$

> $E_{90, mean, t15} := 230$: # $\frac{N}{mm^2}$

> $G_{0, mean, t15} := 720$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 :\ \#\frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} :\ \#\frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \#\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \#\frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \#\frac{N}{mm^2}$

> $t_{t15} := 430 : \#\frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.0098066355855326l}{1} : \#\frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :\ $ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # Equation 6.10b give larger values

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note
The load calculations is in kN/m, kN and kNm
There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right); \#\frac{kN}{m}$

$$g_{0,k} := 1.517734993 \quad \textbf{(1)}$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2}; \#\frac{kN}{m}$

$$f_{d,SLS} := 1.517734993 \quad \textbf{(2)}$$

Modification of the shear force and moment:

The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1 : \#mm^2$

> $A_5 := A_1 : \#mm^2$

> $I_{t1} := \frac{\left(b \cdot h_1^{\ 3}\right)}{12} :\#mm^4$

> $I_{t5} := I_{t1} : \#mm^4$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2 : \#mm^2$

> $A_3 := b \cdot h_3 : \ \#mm^2$

> $A_4 := A_2 : \ \#mm^2$

> $I_{t2} := \frac{\left(b \cdot h_2^{\ 3}\right)}{12} : \#mm^4$

> $I_{t3} := \frac{\left(b \cdot h_3^{\ 3}\right)}{12} : \#mm^4$

> $I_{t4} := I_{t2} : \#mm^4$

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2} : \#mm$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2} : \#mm$

> $z_3 := 0 : \#mm$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2} : \#mm$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 784968000000 \tag{3}$$

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \text{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum} \right)$; #Nmm^2

$$(EI)_{eff} := 8.323520000 \times 10^{11} \tag{4}$$

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$

$$\text{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} \right.\right.$$

$$+ \frac{h_5}{2 \cdot G_{0,\,mean,\,t22} \cdot b} \Big) \Big); \#N$$

$$(GA)_{eff} := 9.436893204 \times 10^6 \qquad (5)$$

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$ #CLT handbook US, Ch.3, table2, pinned - pinned support, uniformly distributed load

$$K_s := 11.5 \qquad (6)$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}} : \# \dfrac{N}{mm^4} \cdot mm^4$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t^3}{12}} : \# \dfrac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum deflection prediction using short-term verifications - SLS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm,\,c} : \# \dfrac{N}{mm^2}$

> $E_2 := E_{CLT} : \# \dfrac{N}{mm^2}$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm^2$

> $A_2 := h_2 \cdot b : \#mm^2$

> $I_1 := I_c : \#mm^4$

> $I_2 := \dfrac{b \cdot h_t^3}{12} : \#mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227. The formula is multiplied by 3, beacause there are 3 pairs of screws in each row.

> $l_{eff,\ ctc} := 110$; #mm

$$l_{eff,ctc} := 110 \tag{7}$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\ ctc}$; #$\frac{N}{mm}$

$$K_{ser} := 23100 \tag{8}$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 45;

$$angle := 45 \tag{9}$$

> k := sin(convert(angle degrees, radians));

$$k := \frac{\sqrt{2}}{2} \tag{10}$$

> $s_{min,\ 1} := \text{evalf}(130 \cdot k)$;#mm

$$s_{min,1} := 91.92388153 \tag{11}$$

> $s_{max,\ 1} := 4 \cdot s_{min,\ 1}$; #mm

$$s_{max,1} := 367.6955261 \tag{12}$$

> $s_{min} := 90$; #mm

$$s_{min} := 90 \tag{13}$$

> $s_{max} := 360$; #mm

$$s_{max} := 360 \tag{14}$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max}$; #mm

$$s := 157.50 \tag{15}$$

> $s := 200$; #mm

$$s := 200 \tag{16}$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} \left(E_i I_i + \gamma_i E_i A_i a_i^2\right)$$

> $\gamma_1 := \text{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L^2}}\right)$;

(17)

$$\gamma_1 := 0.01587791888 \quad (17)$$

> $\gamma_2 := 1.0$; #Fully composite

$$\gamma_2 := 1.0 \quad (18)$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 5.141329679 \quad (19)$$

>

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 94.85867032 \quad (20)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm^2

$$EI_{eff,tot} := 1.689920498 \times 10^{12} \quad (21)$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.03030283455\, M_{Ed,1} \quad (22)$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.8047715865\, M_{Ed,1} \quad (23)$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \ \sigma 1 - \ \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 ; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \le \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 2.794162142 \times 10^7 \quad (24)$$

Stresses at the bottom of concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1\ +\ \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^6;+\ \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^6\right) \le \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\,tot}}\right)}, M_{Ed,\,1}\right); \#Nmm$$

$$M_2 := 1.893771263\times 10^6 \quad (25)$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,\,2})}{EI_{eff,\,tot}}\cdot 10^6; \#MPa$$

$$\sigma_2 := 0.02020188971\ M_{Ed,2} \quad (26)$$

$$> \sigma_{m,\,2} := \frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,\,2})}{EI_{eff,tot}}\cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.2357587352\ M_{Ed,2} \quad (27)$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t}\cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6;\right)}{\frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_3 := 8.044023605 \times 10^7 \quad \textbf{(28)}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_4 := 1.021286689 \times 10^8 \quad \textbf{(29)}$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 27.94162142 \quad \textbf{(30)}$$

$$> L_{out} := 0.3; \#m$$

$$L_{out} := 0.3 \quad \textbf{(31)}$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \quad \textbf{(32)}$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed,\ new}, P_{Ed,\ 1} \right); \#kN$$

$$P_{Ed} := 182.0088465 \quad \textbf{(33)}$$

3.6 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS} \right) \cdot L^4}{384 \cdot EI_{eff,tot}};$$

(34)

$$w := 4.792226413 \quad (34)$$

> $w_{lim} := \mathrm{evalf}\left(\frac{L}{250}\right);$

$$w_{lim} := 6. \quad (35)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{lim}};$ #<1.0 OK

$$Ver_{deflection} := 0.7987044022 \quad (36)$$

4. Maximum deflection prediction using long-term verifications - SLS

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \quad (37)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (38)$$

> $q_k := 0;$

$$q_k := 0 \quad (39)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (40)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{1,fin} := 9714.285711 \quad (41)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 3589.310984 \quad (42)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 4659.807243 \tag{43}$$

> $E_{2,fin} := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 3589.310983 \tag{44}$$

4.1.3 Slip modulus

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 12486.48649 \tag{45}$$

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \tag{46}$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \tag{47}$$

> $K_{u,fin} := K_{ser,2}$

$$K_{u,fin} := 12486.48649 \tag{48}$$

5. Long-term verifications

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_{1,fin} := \mathrm{evalf}\left(\dfrac{1}{1 + \dfrac{\pi^2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L^2}} \right);$

$$\gamma_{1,fin} := 0.02961984787 \tag{49}$$

> $\gamma_{2,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \tag{50}$$

$$> a_{2,fin} := \frac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}; \#mm$$

$$a_{2,fin} := 5.073180113 \tag{51}$$

$$> a_{1,\,fin} := \frac{(h_1 + h_2)}{2} - a_{2,fin}; \#mm$$

$$a_{1,fin} := 94.92681989 \tag{52}$$

$$> EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}^{\ 2} + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}^{\ 2}; \#Nmm^2$$

$$EI_{eff,tot,fin} := 6.899085749 \times 10^{11} \tag{53}$$

5.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.03959050899\, M_{Ed,1} \tag{54}$$

$$> \sigma_{m,\,1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 0.5632216245\, M_{Ed,1} \tag{55}$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 3.870747126 \times 10^7 \tag{56}$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1 + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{\left(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,\,fin}\cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}}\cdot 10\hat{}6;+\ \frac{\left(0.5\cdot E_{1,fin}\cdot h_1\cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}}\cdot 10\hat{}6\right)\le\frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\,1}=\frac{f_{ctk,\,0.05,\,c}}{\gamma_c\cdot\left(-\frac{\left(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}}+\frac{\left(0.5\cdot E_{1,fin}\cdot h_1\right)}{EI_{eff,tot,fin}}\right)},M_{Ed,\,1}\right);\#Nmm$$

$$M_2 := 2.800953999\times 10^6 \tag{57}$$

5.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{\left(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}}\cdot 10^6;\ \#MPa$$

$$\sigma_2 := 0.02639367267\,M_{Ed,2} \tag{58}$$

$$> \sigma_{m,\,2} := \frac{\left(0.5\cdot E_{2,\,fin}\cdot h_2\cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}}\cdot 10^6;\ \#MPa$$

$$\sigma_{m,2} := 0.3121553592\,M_{Ed,2} \tag{59}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}}-\frac{\sigma_{m,\,2}}{f_{m,\,d}}<1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t}\cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}}\cdot 10\hat{}6;\right)}{\frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}}+\frac{\left(\frac{\left(0.5\cdot E_{2,\,fin}\cdot h_2\cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}}\cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\le 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin}\cdot f_{t,0,k,t22}}+\frac{\left(0.5\cdot E_{2,\,fin}\cdot h_2\right)}{EI_{eff,tot,fin}\cdot f_{m,k,t22}}\right)}\right),M_{Ed,2}\right);\#Nmm$$

$$M_3 := 6.083899339\times 10^7 \tag{60}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin})}{EI_{eff,tot,fin}\cdot f_{t,0,k,t22}} + \frac{(0.5\cdot E_{2,\,fin}\cdot h_2)}{EI_{eff,tot,fin}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.699620127 \times 10^7 \quad (61)$$

5.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed,\,new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 38.70747126 \quad (62)$$

$$> L_{out} := 0.3; \#m$$

$$L_{out} := 0.3 \quad (63)$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \quad (64)$$

$$> P_{Ed,\,fin} := \mathrm{solve}\left(\frac{P_{Ed,\,1}\cdot L_{out}}{2} + \frac{1.5\cdot g_{0,k}\cdot L_{sup}^{\ 2}}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right); \#kN$$

$$P_{Ed,fin} := 253.7811787 \quad (65)$$

5.4 Verification of the vertical deflection

Creep is included in the calculations

$$> w_{permanent} := \frac{5\cdot\left(\frac{P_{Ed,\,fin}}{L_{sup}} + f_{d,SLS}\right)\cdot L^4}{384\cdot EI_{eff,tot,fin}};$$

$$w_{permanent} := 16.31018894 \quad (66)$$

>

$$> w_{lim} := \mathrm{evalf}\left(\frac{L}{150}\right);$$

$$w_{lim} := 10. \quad (67)$$

Verification of the vertical deflection

> $\mathrm{Ver}_{\mathrm{deflection}} := \frac{\mathrm{w}_{\mathrm{permanent}}}{\mathrm{w}_{\mathrm{lim}}}$; # < 1.0 NOT OK

$$Ver_{deflection} := 1.631018894 \quad \textbf{(68)}$$

>

Appendix B.2 Maximum deflection for type B

SLS deflection predictions for CTC-screws 7-160 mm 45 degree orientation and spacing 250 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm span length between the supports

> $b := 600$: #mm

Concrete parameters, concrete class B35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \frac{(b \cdot h_c^3)}{12}$: #mm^4

> $E_{cm,c} := 34000$: #MPa

> $f_{ck,c} := 35$:#MPa

> $f_{ctk,0.05,c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\frac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> $Klima := 1.0$: #` `Serice class, permanent

> $k_{modi,\,t} := 0.8$: # modification factor,Swedish CLT handbook

> $k_{def,\,t} := 0.85$: # modification factor,Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0,\,mean,\,t22} := 13000$: # $\frac{N}{mm^2}$

> $E_{90,\,mean,\,t22} := 430$: # $\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t22} := 81$: # $\frac{N}{mm^2}$

> $G_{R,\,t22} := G_{90,mean,t22}$: # $\frac{N}{mm^2}$

> $f_{m,\,k,\,t22} := 30.5$: # $\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t22} := 22.0$: # $\frac{N}{mm^2}$

> $f_{v,\,k,\,t22} := 4.0$: # $\frac{N}{mm^2}$

> $t_{t22} := 470$: # $\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855 3261}{1}$: # $\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\,mean,\,t15} := 11500$: # $\frac{N}{mm^2}$

> $E_{90,\,mean,\,t15} := 230$: # $\frac{N}{mm^2}$

> $G_{0,\,mean,\,t15} := 720$: # $\frac{N}{mm^2}$

> $G_{90,\ mean,\ t15} := 72 :\ \# \frac{N}{mm^2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} :\ \# \frac{N}{mm^2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm^2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm^2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t15} := 430 : \# \frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.0098066355855326 1}{1} : \# \frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\ 1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\ 1} := 1.5 :$ # Equation 6.10b give larger values

> $\gamma_{G,\ 2} := 1.0 :$

> $\gamma_{Q,\ 2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note
The load calculations is in kN/m, kN and kNm
There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\ k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right) ;\# \frac{kN}{m}$

$$g_{0,k} := 1.517734993 \quad \textbf{(1)}$$

1. 1 SLS

> $f_{d,\ SLS} := g_{0,\ k} \cdot \gamma_{G,\ 2} ; \# \frac{kN}{m}$

$$f_{d,SLS} := 1.517734993 \quad \textbf{(2)}$$

Modification of the shear force and moment:

The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$: #mm²

> $A_5 := A_1$: #mm²

> $I_{t1} := \frac{(b \cdot h_1^3)}{12}$:#mm⁴

> $I_{t5} := I_{t1}$: #mm⁴

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$: #mm²

> $A_3 := b \cdot h_3$: #mm²

> $A_4 := A_2$: #mm²

> $I_{t2} := \frac{(b \cdot h_2^3)}{12}$: #mm⁴

> $I_{t3} := \frac{(b \cdot h_3^3)}{12}$: #mm⁴

> $I_{t4} := I_{t2}$: #mm⁴

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$: #mm

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$: #mm

> $z_3 := 0$: #mm

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$: #mm

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 784968000000 \qquad (3)$$

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \mathrm{evalf}((EI)_{sum} + (EAz\hat{}2)_{sum})$; #Nmm^2

$$(EI)_{eff} := 8.323520000 \times 10^{11} \qquad (4)$$

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$

$$\mathrm{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b}\right.\right.$$

$$+\frac{h_5}{2\cdot G_{0,\,mean,\,t22}\cdot b}\Bigg)\Bigg); \#N$$

$$(GA)_{eff} := 9.436893204\times 10^{6} \quad (5)$$

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1+\frac{K_s EI_{eff}}{GA_{eff}L^2}}$$

> $K_s := 11.5$ #CLT handbook US, Ch.3, table2, pinned - pinned support, uniformly distributed load

$$K_s := 11.5 \quad (6)$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1+\dfrac{K_s\cdot(EI)_{eff}}{(GA)_{eff}\cdot L^2}} : \#\dfrac{N}{mm^4}\cdot mm^4$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b\cdot h_t^{3}}{12}} : \#\dfrac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum deflection prediction using short-term verifications - SLS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm,\,c} : \#\dfrac{N}{mm^2}$

> $E_2 := E_{CLT} : \#\dfrac{N}{mm^2}$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm^2$

> $A_2 := h_2\cdot b : \#mm^2$

> $I_1 := I_c : \#mm^4$

> $I_2 := \dfrac{b\cdot h_t^{3}}{12} : \#mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227. The formula is multiplied by 3, beacause there are 3 pairs of screws in each row.

> $l_{eff,\ ctc} := 110;$ #mm

$$l_{eff,ctc} := 110 \tag{7}$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\ ctc};$ #$\frac{N}{mm}$

$$K_{ser} := 23100 \tag{8}$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> $angle := 90;$

$$angle := 90 \tag{9}$$

> $k := \sin(\mathrm{convert}(angle\ degrees, radians));$

$$k := 1 \tag{10}$$

> $s_{min,\ 1} := \mathrm{evalf}(130 \cdot k);$#mm

$$s_{min,1} := 130. \tag{11}$$

> $s_{max,\ 1} := 4 \cdot s_{min,\ 1};$ #mm

$$s_{max,1} := 520. \tag{12}$$

> $s_{min} := 90;$ #mm

$$s_{\min} := 90 \tag{13}$$

> $s_{max} := 360;$ #mm

$$s_{\max} := 360 \tag{14}$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max};$ #mm

$$s := 157.50 \tag{15}$$

> $s := 250;$ #mm

$$s := 250 \tag{16}$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3}(E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_1 := \mathrm{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L^2}}\right);$

$$\gamma_1 := 0.01274280093 \tag{17}$$

> $\gamma_2 := 1.0;$ #Fully composite

$$\gamma_2 := 1.0 \tag{18}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 4.168483631 \tag{19}$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 95.83151637 \tag{20}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm²

$$EI_{eff,tot} := 1.643409096 \times 10^{12} \tag{21}$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.02526425460\ M_{Ed,1} \tag{22}$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.8275480545\ M_{Ed,1} \tag{23}$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \le \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := \text{solve}\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 2.736045561 \times 10^7 \tag{24}$$

Stresses at the bottom of concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1\ +\ \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \le \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

$$> \ M_2 := \mathrm{solve}\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right); \#Nmm$$

$$M_2 := 1.828114524 \times 10^6 \tag{25}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \ \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ 2})}{EI_{eff,\ tot}} \cdot 10^6; \#MPa$$

$$\sigma_2 := 0.01684283641\ M_{Ed,2} \tag{26}$$

$$> \ \sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\ 2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.2424311270\ M_{Ed,2} \tag{27}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\ 0,\ d}} - \frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t} \cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \le 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_3 := 7.983022693 \times 10^7 \tag{28}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_4 := 9.684731979 \times 10^7 \tag{29}$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 27.36045561 \tag{30}$$

$$> L_{out} := 0.3; \#m$$

$$L_{out} := 0.3 \tag{31}$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \tag{32}$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed,\ new}, P_{Ed,\ 1} \right); \#kN$$

$$P_{Ed} := 178.1344077 \tag{33}$$

3.6 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS} \right) \cdot L^4}{384 \cdot EI_{eff,tot}};$$

(34)

$$w := 4.824251150 \quad (34)$$

> $w_{lim} := \text{evalf}\left(\frac{L}{250}\right);$

$$w_{lim} := 6. \quad (35)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{lim}};$ # < 1.0 OK

$$Ver_{deflection} := 0.8040418583 \quad (36)$$

4. Maximum deflection prediction using long-term verifications - SLS

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \quad (37)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (38)$$

> $q_k := 0;$

$$q_k := 0 \quad (39)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (40)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{1,fin} := 9714.285711 \quad (41)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 3589.310984 \quad (42)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 4659.807243 \qquad (43)$$

> $E_{2,fin} := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 3589.310983 \qquad (44)$$

4.1.3 Slip modulus

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 12486.48649 \qquad (45)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \qquad (46)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \qquad (47)$$

> $K_{u,fin} := K_{ser,2}$

$$K_{u,fin} := 12486.48649 \qquad (48)$$

5. Long-term verifications

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_{1,fin} := \text{evalf}\left(\dfrac{1}{1 + \dfrac{\pi^2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L^2}} \right);$

$$\gamma_{1,fin} := 0.02383708849 \qquad (49)$$

> $\gamma_{2,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \qquad (50)$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}$; #mm

$$a_{2,fin} := 4.123571979 \quad (51)$$

> $a_{1,\,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin}$; #mm

$$a_{1,fin} := 95.87642802 \quad (52)$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}^{\;2} + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}^{\;2}$; #Nmm^2

$$EI_{eff,tot,fin} := 6.653678149 \times 10^{11} \quad (53)$$

5.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.03336676766\, M_{Ed,1} \quad (54)$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.5839949270\, M_{Ed,1} \quad (55)$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := \text{solve}\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 3.779523986 \times 10^7 \quad (56)$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1\ + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,\,fin}\cdot M_{Ed,1})}{EI_{eff,tot,fin}}\cdot 10\hat{}6;+\frac{(0.5\cdot E_{1,fin}\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot,fin}}\cdot 10\hat{}6\right)\le\frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\,1}=\frac{f_{ctk,\,0.05,\,c}}{\gamma_c\cdot\left(-\frac{(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,\,fin})}{EI_{eff,tot,fin}}+\frac{(0.5\cdot E_{1,fin}\cdot h_1)}{EI_{eff,tot,fin}}\right)},M_{Ed,\,1}\right);\#Nmm$$

$$M_2 := 2.663624521\times 10^6 \tag{57}$$

5.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\cdot M_{Ed\,2})}{EI_{eff,tot,fin}}\cdot 10^6;\#MPa$$

$$\sigma_2 := 0.02224451177\,M_{Ed,2} \tag{58}$$

$$> \sigma_{m,\,2} := \frac{(0.5\cdot E_{2,\,fin}\cdot h_2\cdot M_{Ed\,2})}{EI_{eff,tot,fin}}\cdot 10^6;\#MPa$$

$$\sigma_{m,2} := 0.3236685848\,M_{Ed,2} \tag{59}$$

Stresses at the top of the timber section

$$\#\sigma\ t,t=-\frac{\sigma_2}{f_{t,\,0,\,d}}-\frac{\sigma_{m,\,2}}{f_{m,\,d}}<1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t}\cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\cdot M_{Ed,2})}{EI_{eff,tot,fin}}\cdot 10\hat{}6;\right)}{\frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_{2,\,fin}\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot,fin}}\cdot 10\hat{}6\,;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\le 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin})}{EI_{eff,tot,fin}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_{2,\,fin}\cdot h_2)}{EI_{eff,tot,fin}\cdot f_{m,k,t22}}\right)}\right),M_{Ed,2}\right);\#Nmm$$

$$M_3 := 5.985031998\times 10^7 \tag{60}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t, b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $$M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin}\cdot E_{2,\ fin}\cdot a_{2,\ fin})}{EI_{eff,tot,fin}\cdot f_{t,0,k,t22}} + \frac{(0.5\cdot E_{2,\ fin}\cdot h_2)}{EI_{eff,tot,fin}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.245644270 \times 10^7 \quad \textbf{(61)}$$

5.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $$M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}; \#kNm$$

$$M_{Ed,new} := 37.79523986 \quad \textbf{(62)}$$

> $L_{out} := 0.3; \#m$

$$L_{out} := 0.3 \quad \textbf{(63)}$$

> $L_{sup} := 1.5; \#m$

$$L_{sup} := 1.5 \quad \textbf{(64)}$$

> $$P_{Ed,\ fin} := \mathrm{solve}\left(\frac{P_{Ed,\ 1}\cdot L_{out}}{2} + \frac{1.5\cdot g_{0,k}\cdot L_{sup}^{\ 2}}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed,fin} := 247.6996361 \quad \textbf{(65)}$$

5.4 Verification of the vertical deflection

Creep is included in the calculations

> $$w_{permanent} := \frac{5\cdot\left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right)\cdot L^4}{384\cdot EI_{eff,tot,fin}};$$

$$w_{permanent} := 16.51009212 \quad \textbf{(66)}$$

>

> $$w_{lim} := \mathrm{evalf}\left(\frac{L}{150}\right);$$

$$w_{lim} := 10. \quad \textbf{(67)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{lim}}$; # < 1.0 NOT OK

$$Ver_{deflection} := 1.651009212 \tag{68}$$

>

Appendix B.3 Maximum deflection for type C

SLS deflection predictions for CTC-screws 7-160 mm 90 degree orientation and spacing 200 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm span length between the supports

> $b := 600$: #mm

Concrete parameters, concrete class B35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \dfrac{\left(b \cdot h_c^{\,3}\right)}{12}$: #mm^4

> $E_{cm,\,c} := 34000$: #MPa

> $f_{ck,\,c} := 35$:#MPa

> $f_{ctk,\,0.05,\,c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\dfrac{\mathrm{kN}}{\mathrm{m}^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> $Klima := 1.0$: # Serice class, permanent

> $k_{modi,\, t} := 0.8$: # modification factor,Swedish CLT handbook

> $k_{def,\, t} := 0.85$: # modification factor,Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0,\, mean,\, t22} := 13000$: #$\frac{N}{mm^2}$

> $E_{90,\, mean,\, t22} := 430$: #$\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: #$\frac{N}{mm^2}$

> $G_{90,\, mean,\, t22} := 81$: #$\frac{N}{mm^2}$

> $G_{R,\, t22} := G_{90,mean,t22}$: #$\frac{N}{mm^2}$

> $f_{m,\, k,\, t22} := 30.5$: #$\frac{N}{mm^2}$

> $f_{t,\, 0,\, k,\, t22} := 22.0$: #$\frac{N}{mm^2}$

> $f_{v,\, k,\, t22} := 4.0$: #$\frac{N}{mm^2}$

> $t_{t22} := 470$: #$\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855 3261}{1}$: #$\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\, mean,\, t15} := 11500$: #$\frac{N}{mm^2}$

> $E_{90,\, mean,\, t15} := 230$: #$\frac{N}{mm^2}$

> $G_{0,\, mean,\, t15} := 720$: #$\frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 : \#\frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \#\frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \#\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \#\frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \#\frac{N}{mm^2}$

> $t_{t15} := 430 : \#\frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} : \#\frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # Equation 6.10b give larger values

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note
The load calculations is in kN/m, kN and kNm
There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right);\#\frac{kN}{m}$

$$g_{0,k} := 1.517734993 \tag{1}$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2};\#\frac{kN}{m}$

$$f_{d,SLS} := 1.517734993 \tag{2}$$

Modification of the shear force and moment:

The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$: #mm^2

> $A_5 := A_1$: #mm^2

> $I_{t1} := \dfrac{(b \cdot h_1^{\,3})}{12}$:#mm^4

> $I_{t5} := I_{t1}$: #mm^4

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$: #mm^2

> $A_3 := b \cdot h_3$: #mm^2

> $A_4 := A_2$: #mm^2

> $I_{t2} := \dfrac{(b \cdot h_2^{\,3})}{12}$: #mm^4

> $I_{t3} := \dfrac{(b \cdot h_3^{\,3})}{12}$: #mm^4

> $I_{t4} := I_{t2}$: #mm^4

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \dfrac{h_1}{2} + h_2 + \dfrac{h_3}{2}$: #mm

> $z_2 := \dfrac{h_2}{2} + \dfrac{h_3}{2}$: #mm

> $z_3 := 0$: #mm

> $z_4 := \dfrac{h_4}{2} + \dfrac{h_3}{2}$: #mm

$$> z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2} : \#mm$$

$$> (EI)_1 := E_{0,\text{ mean, t22}} \cdot I_{t1} : \#Nmm^2$$

$$> (EI)_2 := E_{90,\text{ mean, t15}} \cdot I_{t2} : \#Nmm^2$$

$$> (EI)_3 := E_{0,\text{ mean, t15}} \cdot I_{t3} : \#Nmm^2$$

$$> (EI)_4 := E_{90,\text{mean,t15}} \cdot I_{t4} : \#Nmm^2$$

$$> (EI)_5 := E_{0,\text{mean,t22}} \cdot I_{t5} : \#Nmm^2$$

$$> (EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5 : \#Nmm^2$$

$$> (EAz\hat{}2)_1 := E_{0,\text{ mean, t22}} \cdot A_1 \cdot \left(z_1^{\,2}\right) : \#Nmm^2$$

$$> (EAz\hat{}2)_2 := E_{90,\text{ mean, t15}} \cdot A_2 \cdot \left(z_2^{\,2}\right) : \#Nmm^2$$

$$> (EAz\hat{}2)_3 := E_{0,\text{ mean, t15}} \cdot A_3 \cdot \left(z_3^{\,2}\right) : \#Nmm^2$$

$$> (EAz\hat{}2)_4 := E_{90,\text{mean,t15}} \cdot A_4 \cdot \left(z_4^{\,2}\right) : \#Nmm^2$$

$$> (EAz\hat{}2)_5 := E_{0,\text{mean,t22}} \cdot A_5 \cdot \left(z_5^{\,2}\right) : \#Nmm^2$$

$$> (EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5; \#Nmm^2$$

$$(EAz\hat{}2)_{sum} := 784968000000 \tag{3}$$

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

$$> (EI)_{eff} := \text{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right); \#Nmm^2$$

$$(EI)_{eff} := 8.323520000 \times 10^{11} \tag{4}$$

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

$$> a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2} : \#mm$$

$$> (GA)_{eff} :=$$

$$\text{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\text{ mean, t22}} \cdot b} + \frac{h_2}{G_{90,\text{ mean, t15}} \cdot b} + \frac{h_3}{G_{0,\text{ mean, t15}} \cdot b} + \frac{h_4}{G_{90,\text{ mean, t15}} \cdot b}\right.\right.$$

$$+ \frac{h_5}{2 \cdot G_{0,\,mean,\,t22} \cdot b} \Big) \Big) ; \#N$$

$$(GA)_{eff} := 9.436893204 \times 10^{6} \qquad (5)$$

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$ #CLT handbook US, Ch.3, table2, pinned - pinned support, uniformly distributed load

$$K_s := 11.5 \qquad (6)$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}} : \# \dfrac{N}{mm^4} \cdot mm^4$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t^3}{12}} : \# \dfrac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum deflection prediction using short-term verifications - SLS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm,\,c} : \# \dfrac{N}{mm^2}$

> $E_2 := E_{CLT} : \# \dfrac{N}{mm^2}$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm^2$

> $A_2 := h_2 \cdot b : \#mm^2$

> $I_1 := I_c : \#mm^4$

> $I_2 := \dfrac{b \cdot h_t^3}{12} : \#mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227.

> $K_{ser} := 1800;\ \#\frac{N}{mm}$

$$K_{ser} := 1800 \tag{7}$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 90;

$$angle := 90 \tag{8}$$

> k := sin(convert(angle degrees, radians));

$$k := 1 \tag{9}$$

> $s_{min,\,1} := \mathrm{evalf}(130 \cdot k);\#mm$

$$s_{min,1} := 130. \tag{10}$$

> $s_{max,\,1} := 4 \cdot s_{min,\,1};\ \#mm$

$$s_{max,1} := 520. \tag{11}$$

> $s_{min} := 130;\ \#mm$

$$s_{min} := 130 \tag{12}$$

> $s_{max} := 520;\ \#mm$

$$s_{max} := 520 \tag{13}$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max};\ \#mm$

$$s := 227.50 \tag{14}$$

> s := 200; #mm

$$s := 200 \tag{15}$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_1 := \mathrm{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L^2}}\right);$

$$\gamma_1 := 0.001255623614 \tag{16}$$

> $\gamma_2 := 1.0;\ \#\text{Fully composite}$

$$\gamma_2 := 1.0 \tag{17}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 0.4267827483 \qquad (18)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 99.57321725 \qquad (19)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2$; #Nmm^2

$$EI_{eff,tot} := 1.464519790 \times 10^{12} \qquad (20)$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.002902589947\, M_{Ed,1} \qquad (21)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.9286320400\, M_{Ed,1} \qquad (22)$$

Stresses at the top of the concrete section

$\#\sigma\ c,t\ = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \le \frac{f_{c,k}}{\gamma_c}$

> $M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 2.504827258 \times 10^7 \qquad (23)$$

Stresses at the bottom of concrete section

$\#\sigma\ c,b\ = -\ \sigma\ 1\ + \sigma\ m,1 = \frac{fck}{\gamma c}$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^6;+\ \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^6\right)\leq\frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

$$> M_2 := \text{solve}\left(M_{Ed,\ 1}=\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\ tot}}+\frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\ tot}}\right)},M_{Ed,\ 1}\right);\#Nmm$$

$$M_2 := 1.584336187\times 10^6 \qquad \textbf{(24)}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,\ 2})}{EI_{eff,\ tot}}\cdot 10^6;\ \#MPa$$

$$\sigma_2 := 0.001935059964\ M_{Ed,2} \qquad \textbf{(25)}$$

$$> \sigma_{m,\ 2} := \frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,\ 2})}{EI_{eff,tot}}\cdot 10^6;\ \#MPa$$

$$\sigma_{m,2} := 0.2720437934\ M_{Ed,2} \qquad \textbf{(26)}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\frac{\sigma_2}{f_{t,\ 0,\ d}}-\frac{\sigma_{m,\ 2}}{f_{m,\ d}}<1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t}\cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{\ }6\ ;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\leq 1.0$$

$$> M_3 := \text{solve}\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right),M_{Ed,2}\right);\#Nmm$$

$$M_3 := 7.723096263\times 10^7 \qquad \textbf{(27)}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t, b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.876932326 \times 10^7 \quad (28)$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 25.04827258 \quad (29)$$

$$> L_{out} := 0.3; \#m$$

$$L_{out} := 0.3 \quad (30)$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \quad (31)$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed} := 162.7198542 \quad (32)$$

3.6 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot}};$$

$$w := 4.950988646 \quad (33)$$

$$> w_{lim} := \mathrm{evalf}\left(\frac{L}{250}\right);$$

$$w_{lim} := 6. \quad (34)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{lim}}$; #<1.0 OK

$$Ver_{deflection} := 0.8251647743 \quad (35)$$

4. Maximum deflection prediction using long-term verifications - SLS

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1 + \varphi_c}$;

$$E_{1,g} := 9714.285714 \quad (36)$$

> $E_{1,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad (37)$$

> $q_k := 0$;

$$q_k := 0 \quad (38)$$

> $g_{1,k} := 0$;

$$g_{1,k} := 0 \quad (39)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_{1,fin} := 9714.285711 \quad (40)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1 + k_{def,t}}$;

$$E_{2,g} := 3589.310984 \quad (41)$$

> $E_{2,q} := \frac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2}$;

$$E_{2,q} := 4659.807243 \quad (42)$$

> $E_{2,fin} := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_{2,fin} := 3589.310983 \quad (43)$$

4.1.3 Slip modulus

$$> K_{ser,\,g} := \frac{K_{ser}}{1 + k_{def,\,t}};$$

$$K_{ser,g} := 972.9729730 \quad \textbf{(44)}$$

$$> K_{ser,\,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$$

$$K_{ser,q} := 1263.157895 \quad \textbf{(45)}$$

$$> K_{ser,\,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}};$$

$$K_{ser,2} := 972.9729728 \quad \textbf{(46)}$$

$$> K_{u,\,fin} := K_{ser,\,2}$$

$$K_{u,fin} := 972.9729728 \quad \textbf{(47)}$$

5. Long-term verifications

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

$$> \gamma_{1,\,fin} := \mathrm{evalf}\left(\frac{1}{1 + \frac{\pi^2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,\,fin} \cdot L^2}} \right);$$

$$\gamma_{1,fin} := 0.002372846829 \quad \textbf{(48)}$$

$$> \gamma_{2,\,fin} := 1.0;$$

$$\gamma_{2,fin} := 1.0 \quad \textbf{(49)}$$

$$> a_{2,fin} := \frac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}; \#mm$$

$$a_{2,fin} := 0.4263073724 \quad \textbf{(50)}$$

$$> a_{1,\,fin} := \frac{(h_1 + h_2)}{2} - a_{2,fin}; \#mm$$

$$a_{1,fin} := 99.57369263 \quad \textbf{(51)}$$

$$> EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}^{\ 2} + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}^{\ 2}; \#Nmm^2$$

(52)

$$EI_{eff,tot,fin} := 5.698192612 \times 10^{11} \tag{52}$$

5.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_1 := \frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1}\right)}{EI_{eff,tot,fin}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.004027987045\, M_{Ed,1} \tag{53}$$

$$> \sigma_{m,\,1} := \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 0.6819204875\, M_{Ed,1} \tag{54}$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}} + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1\right)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 3.401616039 \times 10^7 \tag{55}$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \ \sigma\ 1\ + \sigma\ m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \mathrm{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}} + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1\right)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 2.163568215 \times 10^6 \tag{56}$$

5.2 Normal stresses in the timber section

> $\sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.002685324695\, M_{Ed,2} \tag{57}$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 0.3779420488\, M_{Ed,2} \tag{58}$$

Stresses at the top of the timber section

$\#\sigma\ t,\ t\ = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$

$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$

$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$

$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$

> $M_3 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_3 := 5.559167776 \times 10^7 \tag{59}$$

Stresses at the bottom of the timber section

$\#\sigma\ t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 5.669776058 \times 10^7 \quad \textbf{(60)}$$

5.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed,\ new} := \dfrac{\min(M_1, M_3, M_4)}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 34.01616039 \quad \textbf{(61)}$$

> $L_{out} := 0.3$; #m

$$L_{out} := 0.3 \quad \textbf{(62)}$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \quad \textbf{(63)}$$

> $P_{Ed,\ fin} := \mathrm{solve}\left(\dfrac{P_{Ed,\ 1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed,fin} := 222.5057729 \quad \textbf{(64)}$$

5.4 Verification of the vertical deflection

Creep is included in the calculations

> $w_{permanent} := \dfrac{5 \cdot \left(\dfrac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 17.33555251 \quad \textbf{(65)}$$

>

> $w_{lim} := \mathrm{evalf}\left(\dfrac{L}{150}\right)$;

$$w_{\lim} := 10. \quad \textbf{(66)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \dfrac{w_{permanent}}{w_{lim}}$; # < 1.0 NOT OK

$$Ver_{deflection} := 1.733555251 \quad \textbf{(67)}$$

>

SLS deflection predictions for CTC-screws 7-160 mm 90 degree orientation and spacing 250 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm span length between the supports

> $b := 600$: #mm

Concrete parameters, concrete class B35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \frac{\left(b \cdot h_c^3\right)}{12}$: #mm^4

> $E_{cm,\,c} := 34000$: #MPa

> $f_{ck,\,c} := 35$:#MPa

> $f_{ctk,\,0.05,\,c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\frac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20 : \#mm$

> $h_5 := 20 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5 : \#mm$

> $\gamma_M := 1.15 :$ # NA in Eurocode 5 for Glued laminated timber

> $Klima := 1.0 :$ # Serice class, permanent

> $k_{modi,\,t} := 0.8 :$ # modification factor,Swedish CLT handbook

> $k_{def,\,t} := 0.85 :$ # modification factor,Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0,\,mean,\,t22} := 13000 : \#\frac{N}{mm^2}$

> $E_{90,\,mean,\,t22} := 430 : \#\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm^2}$

> $G_{90,\,mean,\,t22} := 81 : \#\frac{N}{mm^2}$

> $G_{R,\,t22} := G_{90,mean,t22} : \#\frac{N}{mm^2}$

> $f_{m,\,k,\,t22} := 30.5 : \#\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t22} := 22.0 : \#\frac{N}{mm^2}$

> $f_{v,\,k,\,t22} := 4.0 : \#\frac{N}{mm^2}$

> $t_{t22} := 470 : \#\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066355855 3261}{1} : \#\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\,mean,\,t15} := 11500 : \#\frac{N}{mm^2}$

> $E_{90,\,mean,\,t15} := 230 : \#\frac{N}{mm^2}$

> $G_{0,\,mean,\,t15} := 720 : \#\frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 : \#\frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \#\frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \#\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \#\frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \#\frac{N}{mm^2}$

> $t_{t15} := 430 : \#\frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.0098066355855326\,1}{1} : \#\frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # Equation 6.10b give larger values

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note

The load calculations is in kN/m, kN and kNm

There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right) ;\#\frac{kN}{m}$

$$g_{0,k} := 1.517734993 \quad (1)$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2} ; \#\frac{kN}{m}$

$$f_{d,SLS} := 1.517734993 \quad (2)$$

Modification of the shear force and moment:

The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$: #mm²

> $A_5 := A_1$: #mm²

> $I_{t1} := \frac{(b \cdot h_1^{\,3})}{12}$:#mm⁴

> $I_{t5} := I_{t1}$: #mm⁴

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$: #mm²

> $A_3 := b \cdot h_3$: #mm²

> $A_4 := A_2$: #mm²

> $I_{t2} := \frac{(b \cdot h_2^{\,3})}{12}$: #mm⁴

> $I_{t3} := \frac{(b \cdot h_3^{\,3})}{12}$: #mm⁴

> $I_{t4} := I_{t2}$: #mm⁴

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$: #mm

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$: #mm

> $z_3 := 0$: #mm

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$: #mm

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 784968000000 \tag{3}$$

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \text{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 8.323520000 \times 10^{11} \tag{4}$$

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$

$$\text{evalf}\left(a^2 \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b}\right.\right.$$

$$+\frac{h_5}{2\cdot G_{0,\,mean,\,t22}\cdot b}\Bigg)\Bigg); \#N$$

$$(GA)_{eff} := 9.436893204\times 10^{6} \quad (5)$$

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app}=\frac{EI_{eff}}{1+\frac{K_s EI_{eff}}{GA_{eff}L^2}}$$

> $K_s := 11.5$ #CLT handbook US, Ch.3, table2, pinned - pinned support, uniformly distributed load

$$K_s := 11.5 \quad (6)$$

> $EI_{app} := \frac{(EI)_{eff}}{1+\frac{K_s\cdot(EI)_{eff}}{(GA)_{eff}\cdot L^2}} : \#\frac{N}{mm^4}\cdot mm^4$

> $E_{CLT} := \frac{EI_{app}}{\frac{b\cdot h_t^3}{12}} : \#\frac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum deflection prediction using short-term verifications - SLS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm,\,c} : \#\frac{N}{mm^2}$

> $E_2 := E_{CLT} : \#\frac{N}{mm^2}$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm^2$

> $A_2 := h_2\cdot b : \#mm^2$

> $I_1 := I_c : \#mm^4$

> $I_2 := \frac{b\cdot h_t^3}{12} : \#mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227.

> $K_{ser} := 1800; \# \frac{N}{mm}$

$$K_{ser} := 1800 \tag{7}$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 90;

$$angle := 90 \tag{8}$$

> k := sin(convert(angle degrees, radians));

$$k := 1 \tag{9}$$

> $s_{min,\,1} := evalf(130 \cdot k);\#mm$

$$s_{min,1} := 130. \tag{10}$$

> $s_{max,\,1} := 4 \cdot s_{min,\,1}; \#mm$

$$s_{max,1} := 520. \tag{11}$$

> $s_{min} := 130; \#mm$

$$s_{min} := 130 \tag{12}$$

> $s_{max} := 520; \#mm$

$$s_{max} := 520 \tag{13}$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max}; \#mm$

$$s := 227.50 \tag{14}$$

> s := 250; #mm

$$s := 250 \tag{15}$$

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} \left(E_i I_i + \gamma_i E_i A_i a_i^2\right)$$

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L^2}}\right);$

$$\gamma_1 := 0.001004751209 \tag{16}$$

> $\gamma_2 := 1.0; \#Fully\ composite$

$$\gamma_2 := 1.0 \tag{17}$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 0.3418034194 \quad \textbf{(18)}$$

>

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 99.65819658 \quad \textbf{(19)}$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^{\,2} + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^{\,2}$; #Nmm^2

$$EI_{eff,tot} := 1.460456961 \times 10^{12} \quad \textbf{(20)}$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6$; #MPa

$$\sigma_1 := 0.002331104353\, M_{Ed,1} \quad \textbf{(21)}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 0.9312153910\, M_{Ed,1} \quad \textbf{(22)}$$

Stresses at the top of the concrete section

#σ c,t $= - \;\; \sigma 1 - \sigma$ m,1 $= \dfrac{\text{fck}}{\gamma\,\text{c}}$

#$M_{Ed,1} \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \leq \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := \text{solve}\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 2.499429160 \times 10^7 \quad \textbf{(23)}$$

Stresses at the bottom of concrete section

#σ c,b $= - \;\; \sigma\, 1 + \sigma$ m,1 $= \dfrac{\text{fck}}{\gamma\,\text{c}}$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \text{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 1.578955192 \times 10^6 \qquad \textbf{(24)}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10^6; \#MPa$$

$$\sigma_2 := 0.001554069569\, M_{Ed,2} \qquad \textbf{(25)}$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_{m,2} := 0.2728005890\, M_{Ed,2} \qquad \textbf{(26)}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2} \left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6; \right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6\ ; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

$$> M_3 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_3 := 7.716675092 \times 10^7 \qquad \textbf{(27)}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.839533839 \times 10^7 \quad (28)$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 24.99429160 \quad (29)$$

$$> L_{out} := 0.3; \#m$$

$$L_{out} := 0.3 \quad (30)$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \quad (31)$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^{\ 2}}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed} := 162.3599810 \quad (32)$$

3.6 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot}};$$

$$w := 4.953933118 \quad (33)$$

$$> w_{lim} := \mathrm{evalf}\left(\frac{L}{250}\right);$$

$$w_{lim} := 6. \quad (34)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{lim}}$; # < 1.0 OK

$$Ver_{deflection} := 0.8256555197 \quad (35)$$

4. Maximum deflection prediction using long-term verifications - SLS

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c}$;

$$E_{1,g} := 9714.285714 \quad (36)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad (37)$$

> $q_k := 0$;

$$q_k := 0 \quad (38)$$

> $g_{1,k} := 0$;

$$g_{1,k} := 0 \quad (39)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_{1,fin} := 9714.285711 \quad (40)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}}$;

$$E_{2,g} := 3589.310984 \quad (41)$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2}$;

$$E_{2,q} := 4659.807243 \quad (42)$$

> $E_{2,fin} := \frac{E_{2,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_{2,fin} := 3589.310983 \quad (43)$$

4.1.3 Slip modulus

> $K_{ser,\,g} := \dfrac{K_{ser}}{1 + k_{def,\,t}}$;

$$K_{ser,g} := 972.9729730 \quad \textbf{(44)}$$

> $K_{ser,\,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2}$;

$$K_{ser,q} := 1263.157895 \quad \textbf{(45)}$$

> $K_{ser,\,2} := \dfrac{K_{ser,g} \cdot \left(g_{0,k} + g_{1,k}\right) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{\left(g_{0,\,k} + g_{1,k}\right) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}}$;

$$K_{ser,2} := 972.9729728 \quad \textbf{(46)}$$

> $K_{u,\,fin} := K_{ser,\,2}$

$$K_{u,fin} := 972.9729728 \quad \textbf{(47)}$$

5. Long-term verifications

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} \left(E_i I_i + \gamma_i E_i A_i a_i^2\right)$$

> $\gamma_{1,\,fin} := \text{evalf}\left(\dfrac{1}{1 + \dfrac{\pi^2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,\,fin} \cdot L^2}}\right)$;

$$\gamma_{1,fin} := 0.001899178755 \quad \textbf{(48)}$$

> $\gamma_{2,\,fin} := 1.0$;

$$\gamma_{2,fin} := 1.0 \quad \textbf{(49)}$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot \left(h_1 + h_2\right)}{2 \cdot \left(\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2\right)}$; #mm

$$a_{2,fin} := 0.3414984384 \quad \textbf{(50)}$$

> $a_{1,\,fin} := \dfrac{\left(h_1 + h_2\right)}{2} - a_{2,fin}$; #mm

$$a_{1,fin} := 99.65850156 \quad \textbf{(51)}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}^{\;2} + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}^{\;2}$; #Nmm^2

(52)

$$EI_{eff,tot,fin} := 5.676275406 \times 10^{11} \tag{52}$$

5.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

$$> \sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.003239124270\, M_{Ed,1} \tag{53}$$

$$> \sigma_{m,\,1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10^6; \#MPa$$

$$\sigma_{m,1} := 0.6845535155\, M_{Ed,1} \tag{54}$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t\ = -\ \ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := \text{solve}\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 3.392495351 \times 10^7 \tag{55}$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \ \sigma\ 1\ + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := \text{solve}\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(- \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 2.152701728 \times 10^6 \tag{56}$$

5.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10^6;\ \#MPa$$

$$\sigma_2 := 0.002159416181\, M_{Ed,2} \qquad (57)$$

$$> \sigma_{m,\,2} := \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10^6;\ \#MPa$$

$$\sigma_{m,2} := 0.3794013567\, M_{Ed,2} \qquad (58)$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right);\ \#Nmm$$

$$M_3 := 5.548552195 \times 10^7 \qquad (59)$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right);\ \#Nmm$$

$$M_4 := 5.636812367 \times 10^7 \qquad (60)$$

5.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{\mathrm{Ed,\,new}} := \dfrac{\min(M_1, M_3, M_4)}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 33.92495351 \qquad (61)$$

> $L_{\mathrm{out}} := 0.3$; #m

$$L_{out} := 0.3 \qquad (62)$$

> $L_{\mathrm{sup}} := 1.5$; #m

$$L_{sup} := 1.5 \qquad (63)$$

> $P_{\mathrm{Ed,\,fin}} := \mathrm{solve}\left(\dfrac{P_{\mathrm{Ed,\,1}} \cdot L_{\mathrm{out}}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{\mathrm{sup}}^{\;2}}{8} = M_{\mathrm{Ed,\,new}}, P_{\mathrm{Ed,\,1}}\right)$; #kN

$$P_{Ed,fin} := 221.8977271 \qquad (64)$$

5.4 Verification of the vertical deflection

Creep is included in the calculations

> $w_{\mathrm{permanent}} := \dfrac{5 \cdot \left(\dfrac{P_{\mathrm{Ed,\,fin}}}{L_{\mathrm{sup}}} + f_{\mathrm{d,SLS}}\right) \cdot L^4}{384 \cdot EI_{\mathrm{eff,tot,fin}}}$;

$$w_{permanent} := 17.35541401 \qquad (65)$$

>

> $w_{\mathrm{lim}} := \mathrm{evalf}\left(\dfrac{L}{150}\right)$;

$$w_{\mathrm{lim}} := 10. \qquad (66)$$

Verification of the vertical deflection

> $Ver_{\mathrm{deflection}} := \dfrac{w_{\mathrm{permanent}}}{w_{\mathrm{lim}}}$; # < 1.0 NOT OK

$$Ver_{deflection} := 1.735541401 \qquad (67)$$

>

Appendix B.5 Maximum deflection for type E

SLS deflection predictions for CTC-screws 7-160 mm 90 degree orientation and spacing 125 mm

> restart;

General data:
Concrete class: B35
Timber class: T22 and T15

Note: Some of the values that are identical in every calculation are not going be shown in the middle "blue text" they can be found in ULS calculations for type A (45 degree orientation and spacing 200 mm)

> $L := 1500$: #mm span length between the supports

> $b := 600$: #mm

Concrete parameters, concrete class B35

All parameters are taken from Eurocode 2 (NS-EN 1992-1-1:2004+A1:2014+NA:2021 tabel 3.1)

> $h_c := 80$: #mm

> $A_c := h_c \cdot b$: #mm^2

> $I_c := \frac{\left(b \cdot h_c^3\right)}{12}$: #mm^4

> $E_{cm, c} := 34000$: #MPa

> $f_{ck, c} := 35$:#MPa

> $f_{ctk, 0.05, c} := 2.2$: #MPa

> $\rho_c := 25.00$: #$\frac{kN}{m^3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

CLT (cross-laminated timber)

All parameters are taken from several sources they are from Splitkon (SINTEF certification Nr. 20712) and Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+NA:2010) and the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT Handbook", Swedish Wood, 2019)

The timber used in the laboratory testing is 5-layered the outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 20$: #mm

> $h_2 := 20$: #mm

> $h_3 := 40$: #mm

> $h_4 := 20$: #mm

> $h_5 := 20$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$: #mm

> $\gamma_M := 1.15$: # NA in Eurocode 5 for Glued laminated timber

> $Klima := 1.0$: # Serice class, permanent

> $k_{modi,\,t} := 0.8$: # modification factor,Swedish CLT handbook

> $k_{def,\,t} := 0.85$: # modification factor,Swedish CLT handbook

Lamellae 1 and 5, Class T22

> $E_{0,\,mean,\,t22} := 13000$: # $\frac{N}{mm^2}$

> $E_{90,\,mean,\,t22} := 430$: # $\frac{N}{mm^2}$

> $G_{0,mean,t22} := 810$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t22} := 81$: # $\frac{N}{mm^2}$

> $G_{R,\,t22} := G_{90,mean,t22}$: # $\frac{N}{mm^2}$

> $f_{m,\,k,\,t22} := 30.5$: # $\frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t22} := 22.0$: # $\frac{N}{mm^2}$

> $f_{v,\,k,\,t22} := 4.0$: # $\frac{N}{mm^2}$

> $t_{t22} := 470$: # $\frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}$: # $\frac{kN}{m^3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\,mean,\,t15} := 11500$: # $\frac{N}{mm^2}$

> $E_{90,\,mean,\,t15} := 230$: # $\frac{N}{mm^2}$

> $G_{0,\,mean,\,t15} := 720$: # $\frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t15} := 430 : \# \frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} : \# \frac{kN}{m^3}$

1. Load calculations

Safety factors:

> $\gamma_{G,\,1} := 1.2 :$ # Equation 6.10b give larger values

> $\gamma_{Q,\,1} := 1.5 :$ # Equation 6.10b give larger values

> $\gamma_{G,\,2} := 1.0 :$

> $\gamma_{Q,\,2} := 1.0 :$

> $\psi_1 := 0.7 :$

> $\psi_2 := 0.5 :$

> $\psi_3 := 0.3 :$

Note

The load calculations is in kN/m, kN and kNm

There is only characteristic dead weight of the slab, for laboratory testing there is no other characteristic dead weight from anything else or variable loading

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right); \# \frac{kN}{m}$

$$g_{0,k} := 1.517734993 \tag{1}$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2}; \# \frac{kN}{m}$

$$f_{d,SLS} := 1.517734993 \tag{2}$$

Modification of the shear force and moment:

The results above are to small to compare them to the actual maximum loading that the timber concrete composite can withstand.
Therefore the Gamma method (Eurocode 5 - Annex B) and Shear Analogy method (CLT handbook US version) have been applied to find the maximum loading. As for the Gamma method it is applicable for a 3 layered element because of this the Shear analogy method has been included in the calculations to get a better understanding of the composite and make better predictions.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$: #mm^2

> $A_5 := A_1$: #mm^2

> $I_{t1} := \dfrac{(b \cdot h_1^{\,3})}{12}$:#mm^4

> $I_{t5} := I_{t1}$: #mm^4

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$: #mm^2

> $A_3 := b \cdot h_3$: #mm^2

> $A_4 := A_2$: #mm^2

> $I_{t2} := \dfrac{(b \cdot h_2^{\,3})}{12}$: #mm^4

> $I_{t3} := \dfrac{(b \cdot h_3^{\,3})}{12}$: #mm^4

> $I_{t4} := I_{t2}$: #mm^4

2.1 The effectiv bending stiffeness for the CLT element:

> $z_1 := \dfrac{h_1}{2} + h_2 + \dfrac{h_3}{2}$: #mm

> $z_2 := \dfrac{h_2}{2} + \dfrac{h_3}{2}$: #mm

> $z_3 := 0$: #mm

> $z_4 := \dfrac{h_4}{2} + \dfrac{h_3}{2}$: #mm

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$: #mm

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$: #Nmm^2

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$: #Nmm^2

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$: #Nmm^2

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$: #Nmm^2

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$: #Nmm^2

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$: #Nmm^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5^{\ 2})$: #Nmm^2

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 784968000000 \tag{3}$$

The effective bending stiffnes using the shear analogy method. CLT handbook US, Ch.3, eq.24:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $(EI)_{eff} := \text{evalf}\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 8.323520000 \times 10^{11} \tag{4}$$

2.2 The effectiv shear stiffeness for the CLT element:

The effective shear stiffeness using the shear analogy method. CLT handbook US, Ch.3, eq.25:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} :=$

$$\text{evalf}\left(a^2 \middle/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} \right.\right.$$

$$+\frac{h_5}{2\cdot G_{0,\,mean,\,t22}\cdot b}\Bigg)\Bigg); \#N$$

$$(GA)_{eff} := 9.436893204\times 10^{6} \qquad (5)$$

2.3 The apparent bending stiffness

By reducing the effective bending stiffnes using CLT handbook US, Ch.3, eq.28 we get the following apparent bending stiffness:

$$EI_{app} = \frac{EI_{eff}}{1+\frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$ #CLT handbook US, Ch.3, table2, pinned - pinned support, uniformly distributed load

$$K_s := 11.5 \qquad (6)$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1+\dfrac{K_s\cdot(EI)_{eff}}{(GA)_{eff}\cdot L^2}} : \#\dfrac{N}{mm^4}\cdot mm^4$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b\cdot h_t^3}{12}} : \#\dfrac{N}{mm^2}$

3. γ-method from, EC5, Annex B, Maximum deflection prediction using short-term verifications - SLS

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

> $E_1 := E_{cm,\,c} : \#\dfrac{N}{mm^2}$

> $E_2 := E_{CLT} : \#\dfrac{N}{mm^2}$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm^2$

> $A_2 := h_2\cdot b : \#mm^2$

> $I_1 := I_c : \#mm^4$

> $I_2 := \dfrac{b\cdot h_t^3}{12} : \#mm^4$

3.1 Slip modulus Kser and Ku

Values for the slip modulus Kser are taken from Rothoblass pdfs, both from the ETA p.9 and CTC type p.227.

> $K_{ser} := 1800; \# \frac{N}{mm}$

$$K_{ser} := 1800 \quad \textbf{(7)}$$

3.2 Minimum and Maximum spacing of the screws

Formulas for the minimum spacing are taken from Rothoblass pdf for CTC screws, ETA p.7. Formulas for maximum and effective spacing is taken from EC5 9.1.3(3), eq. (9.17)

> angle := 90;

$$angle := 90 \quad \textbf{(8)}$$

> k := sin(convert(angle degrees, radians));

$$k := 1 \quad \textbf{(9)}$$

> $s_{min, 1} := evalf(130 \cdot k);\#mm$

$$s_{min,1} := 130. \quad \textbf{(10)}$$

> $s_{max, 1} := 4 \cdot s_{min, 1}; \#mm$

$$s_{max,1} := 520. \quad \textbf{(11)}$$

> $s_{min} := 130; \#mm$

$$s_{min} := 130 \quad \textbf{(12)}$$

> $s_{max} := 520; \#mm$

$$s_{max} := 520 \quad \textbf{(13)}$$

> $s := 0.75 \cdot s_{min} + 0.25 \cdot s_{max}; \#mm$

$$s := 227.50 \quad \textbf{(14)}$$

> s := 125; #mm

$$s := 125 \quad \textbf{(15)}$$

The spacing does not satisfy the minimum spacing. We did not know the slip modulus before we chose the spacing. As for why the spacing is 125, we wanted to see the difference in capacity of 90 degree and 45 degree orientation in screws. Therefore we took as many screws in Type B as Type E the orientation of the screws is different. To see what the outcome would be.

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_1 := \text{evalf}\left(\frac{1}{1+\frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L^2}}\right);$

$$\gamma_1 := 0.002007485394 \tag{16}$$

> $\gamma_2 := 1.0;$ #Fully composite

$$\gamma_2 := 1.0 \tag{17}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)};$ #mm

$$a_2 := 0.6805990325 \tag{18}$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2;$ #mm

$$a_1 := 99.31940097 \tag{19}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2;$ #Nmm²

$$EI_{eff,tot} := 1.476654650 \times 10^{12} \tag{20}$$

3.3 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10^6;$ #MPa

$$\sigma_1 := 0.004590779835\ M_{Ed,1} \tag{21}$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6;$ #MPa

$$\sigma_{m,1} := 0.9210007230\ M_{Ed,1} \tag{22}$$

Stresses at the top of the concrete section

$\#\sigma\ c,t\ = -\ \ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6\right) \le \frac{f_{c,k}}{\gamma_c}$

> $M_1 := \text{solve}\left(M_{Ed,1} = \dfrac{f_{ck,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}}\right)}, M_{Ed,1}\right)$; #Nmm

$$M_1 := 2.520910494 \times 10^7 \qquad \textbf{(23)}$$

Stresses at the bottom of concrete section

$$\#\sigma\ c,b = -\ \sigma\ 1 + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6\right) \leq \frac{f_{ctk,0.005,c}}{\gamma_c}$$

> $M_2 := \text{solve}\left(M_{Ed,1} = \dfrac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}}\right)}, M_{Ed,1}\right)$; #Nmm

$$M_2 := 1.600448225 \times 10^6 \qquad \textbf{(24)}$$

3.4 Normal stresses in the timber section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_2 := 0.003060519891\ M_{Ed,2} \qquad \textbf{(25)}$$

> $\sigma_{m,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 0.2698081906\ M_{Ed,2} \qquad \textbf{(26)}$$

Stresses at the top of the timber section

$$\#\sigma\ t,t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6;\right)}{\frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6\;;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\leq 1.0$$

$$> M_3 := \mathrm{solve}\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 7.742127213\times 10^7 \tag{27}$$

Stresses at the bottom of the timber section

$$\#\sigma\ t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}}+\frac{\sigma_{m,2}}{f_{m,d}}<1.0$$

$$> M_4 := \mathrm{solve}\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.989522547\times 10^7 \tag{28}$$

3.5 The maxiumum loading, Ped

Neglecting the bending moment for the bottom part of the concrete section (M2)

$$> M_{Ed,\,new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 25.20910494 \tag{29}$$

$$> L_{out} := 0.3; \#m$$

$$L_{out} := 0.3 \tag{30}$$

$$> L_{sup} := 1.5; \#m$$

$$L_{sup} := 1.5 \tag{31}$$

$$> P_{Ed} := \mathrm{solve}\left(\frac{P_{Ed,\,1}\cdot L_{out}}{2}+\frac{1.5\cdot g_{0,k}\cdot L_{sup}^{\ 2}}{8}=M_{Ed,\,new}, P_{Ed,\,1}\right); \#kN$$

$$P_{Ed} := 163.7920699 \tag{32}$$

3.6 Verification of the vertical defelction

$$> \ w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS} \right) \cdot L^4}{384 \cdot EI_{eff,tot}};$$

$$w := 4.942211577 \quad (33)$$

$$> \ w_{lim} := \mathrm{evalf}\left(\frac{L}{250} \right);$$

$$w_{lim} := 6. \quad (34)$$

Verification of the vertical deflection

$$> \ Ver_{deflection} := \frac{w}{w_{lim}}; \# < 1.0 \text{ OK}$$

$$Ver_{deflection} := 0.8237019295 \quad (35)$$

4. Maximum deflection prediction using long-term verifications - SLS

4.1 New elasticity modulus calculated:

4.1.1 Concrete

$$> \ E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c};$$

$$E_{1,g} := 9714.285714 \quad (36)$$

$$> \ E_{1,\,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2};$$

$$E_{1,q} := 15111.11111 \quad (37)$$

$$> \ q_k := 0;$$

$$q_k := 0 \quad (38)$$

$$> \ g_{1,\,k} := 0;$$

$$g_{1,k} := 0 \quad (39)$$

$$> \ E_{1,\,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,\,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}};$$

$$E_{1,fin} := 9714.285711 \quad (40)$$

4.1.2 CLT

> $E_{2,\,g} := \dfrac{E_{CLT}}{1 + k_{def,\,t}};$

$$E_{2,g} := 3589.310984 \tag{41}$$

> $E_{2,\,q} := \dfrac{E_{CLT}}{1 + k_{def,\,t} \cdot \psi_2};$

$$E_{2,q} := 4659.807243 \tag{42}$$

> $E_{2,\,fin} := \dfrac{E_{2,\,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,\,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}};$

$$E_{2,fin} := 3589.310983 \tag{43}$$

4.1.3 Slip modulus

> $K_{ser,\,g} := \dfrac{K_{ser}}{1 + k_{def,\,t}};$

$$K_{ser,g} := 972.9729730 \tag{44}$$

> $K_{ser,\,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 1263.157895 \tag{45}$$

> $K_{ser,\,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}};$

$$K_{ser,2} := 972.9729728 \tag{46}$$

> $K_{u,\,fin} := K_{ser,\,2}$

$$K_{u,fin} := 972.9729728 \tag{47}$$

5. Long-term verifications

Now we repeat the steps for short-term verification

From EC5, Annex B, eq.B.1 by using the γ-method we get the effective bending stiffness:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2)$$

> $\gamma_{1,\,fin} := \mathrm{evalf}\left(\dfrac{1}{1+\dfrac{\pi^2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,\,fin} \cdot L^2}}\right);$

$$\gamma_{1,fin} := 0.003791157425 \quad \textbf{(48)}$$

> $\gamma_{2,\,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad \textbf{(49)}$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)};$ #mm

$$a_{2,fin} := 0.6793908889 \quad \textbf{(50)}$$

> $a_{1,\,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin};$ #mm

$$a_{1,fin} := 99.32060911 \quad \textbf{(51)}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}{}^2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}{}^2;$ #Nmm^2

$$EI_{eff,tot,fin} := 5.763597084 \times 10^{11} \quad \textbf{(52)}$$

5.1 Normal stresses in the concrete section

As Med is unknown we need to find the maximum loading for the CLT-concrete composite

> $\sigma_1 := \dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10^6;$ #MPa

$$\sigma_1 := 0.006346414777\, M_{Ed,1} \quad \textbf{(53)}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10^6;$ #MPa

$$\sigma_{m,1} := 0.6741821520\, M_{Ed,1} \quad \textbf{(54)}$$

Stresses at the top of the concrete section

$$\#\sigma\ c,t = -\ \sigma 1 - \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6\right) \le \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := \mathrm{solve}\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}}\right)}, M_{Ed,\,1}\right);$ #Nmm

$$M_1 := 3.4287074 \times 10^{7} \quad \textbf{(55)}$$

Stresses at the bottom of the concrete section

$$\#\sigma\ c,b\ = -\ \sigma\ 1 + \sigma\ m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,\ fin}\cdot M_{Ed,1})}{EI_{eff,tot,fin}}\cdot 10\hat{\ }6;+\frac{(0.5\cdot E_{1,fin}\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot,fin}}\cdot 10\hat{\ }6\right)\le\frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

$$> M_2 := \operatorname{solve}\left(M_{Ed\ 1}=\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c\cdot\left(-\frac{(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,\ fin})}{EI_{eff,tot,fin}}+\frac{(0.5\cdot E_{1,fin}\cdot h_1)}{EI_{eff,tot,fin}}\right)}, M_{Ed\ 1}\right);\ \#Nmm$$

$$M_2 := 2.196148821 \times 10^{6} \quad \textbf{(56)}$$

5.2 Normal stresses in the timber section

$$> \sigma_2 := \frac{(\gamma_{2,fin}\cdot E_{2,\ fin}\cdot a_{2,\ fin}\cdot M_{Ed\ 2})}{EI_{eff,tot,fin}}\cdot 10^{6};\ \#MPa$$

$$\sigma_2 := 0.004230943183\ M_{Ed,2} \quad \textbf{(57)}$$

$$> \sigma_{m,\ 2} := \frac{(0.5\cdot E_{2,\ fin}\cdot h_2\cdot M_{Ed\ 2})}{EI_{eff,tot,fin}}\cdot 10^{6};\ \#MPa$$

$$\sigma_{m,2} := 0.3736532166\ M_{Ed,2} \quad \textbf{(58)}$$

Stresses at the top of the timber section

$$\#\sigma\ t,\ t\ = -\frac{\sigma_2}{f_{t,\ 0,\ d}}-\frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t}\cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_{2,fin}\cdot E_{2,\ fin}\cdot a_{2,\ fin}\cdot M_{Ed,2})}{EI_{eff,tot,fin}}\cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_{2,\ fin}\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot,fin}}\cdot 10\hat{\ }6\ ;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\le 1.0$$

> $M_3 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_3 := 5.590602870 \times 10^7 \quad (59)$$

Stresses at the bottom of the timber section

$\#\sigma\ t, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := \text{solve}\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 5.768925278 \times 10^7 \quad (60)$$

5.3 The maxiumum loading, Ped, Long-term

Neglecting the bending moment for the bottom part of the concrete section (M2)

> $M_{Ed,new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 34.28707400 \quad (61)$$

> $L_{out} := 0.3$; #m

$$L_{out} := 0.3 \quad (62)$$

> $L_{sup} := 1.5$; #m

$$L_{sup} := 1.5 \quad (63)$$

> $P_{Ed,fin} := \text{solve}\left(\frac{P_{Ed,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^2}{8} = M_{Ed,new}, P_{Ed,1}\right)$; #kN

$$P_{Ed,fin} := 224.3118637 \quad (64)$$

5.4 Verification of the vertical deflection

Creep is included in the calculations

> $w_{permanent} := \dfrac{5 \cdot \left(\dfrac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS} \right) \cdot L^4}{384 \cdot EI_{eff,tot,fin}};$

$$w_{permanent} := 17.27653896 \tag{65}$$

>

> $w_{lim} := \text{evalf}\left(\dfrac{L}{150} \right);$

$$w_{lim} := 10. \tag{66}$$

Verification of the vertical deflection

> $Ver_{deflection} := \dfrac{w_{permanent}}{w_{lim}};$ # < 1.0 NOT OK

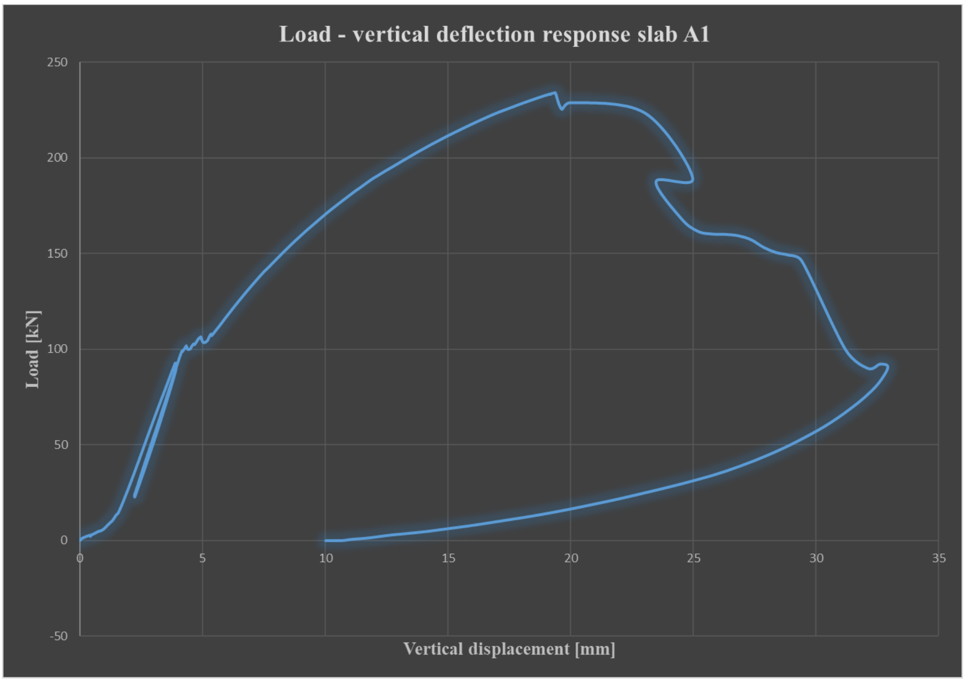
$$Ver_{deflection} := 1.727653896 \tag{67}$$

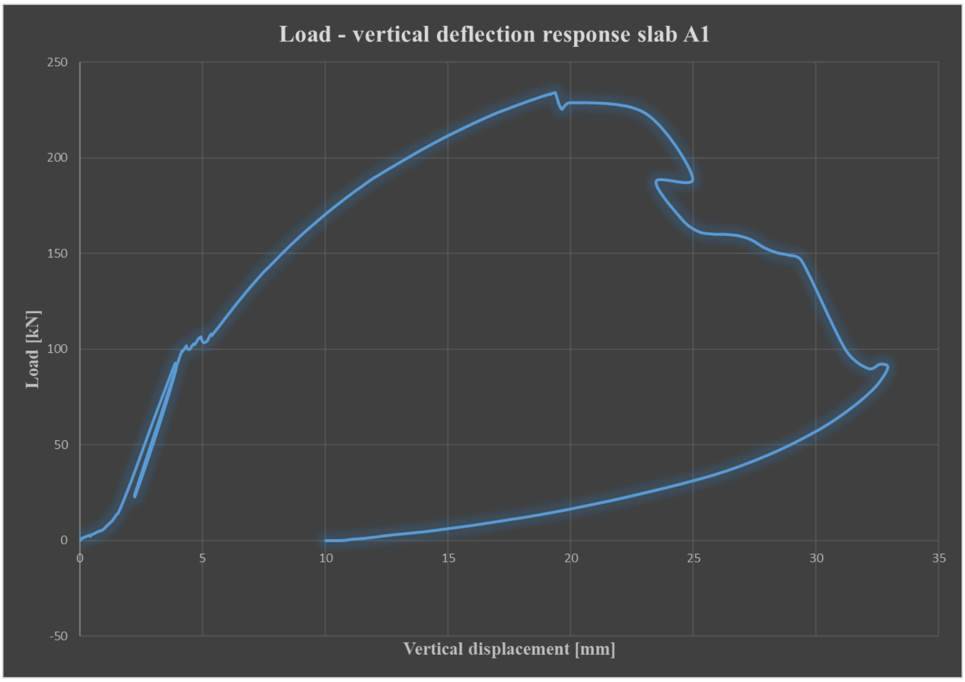
>

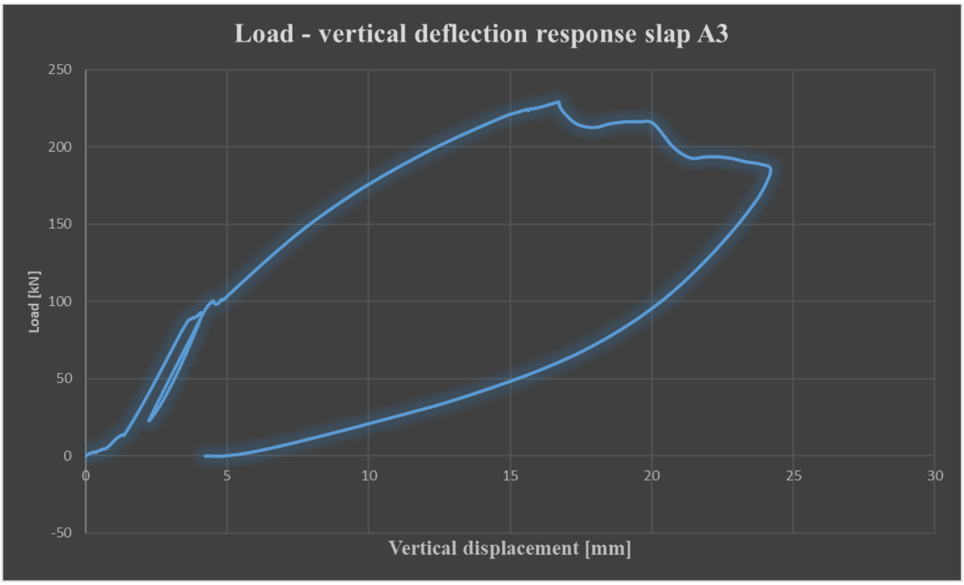
Appendix C.

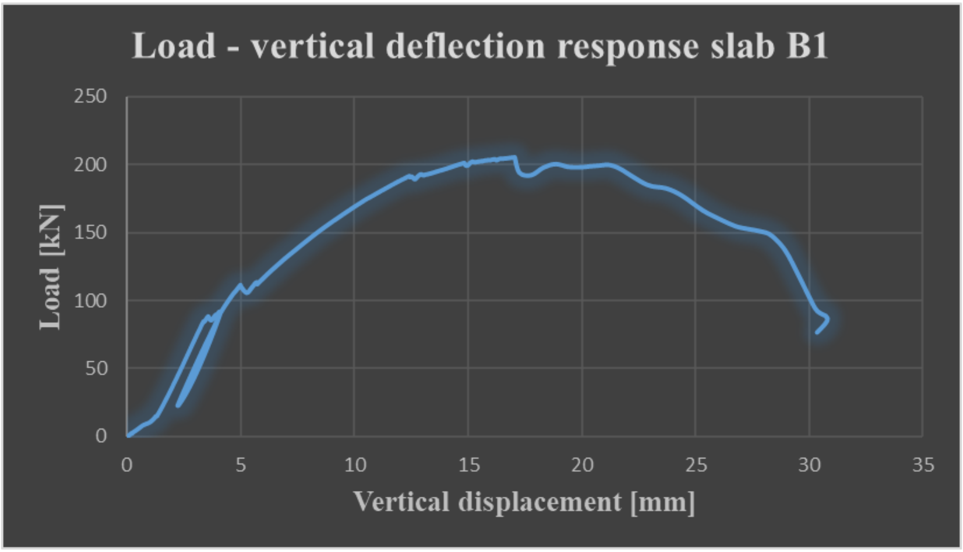
Graphs, vertical deflection Catman

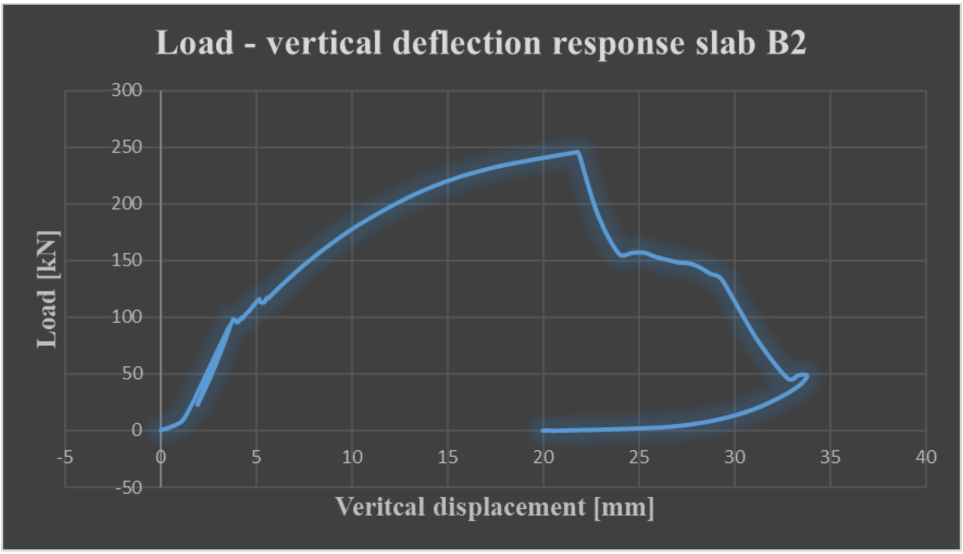
Appendix C. Graphs, vertical deflection Catman

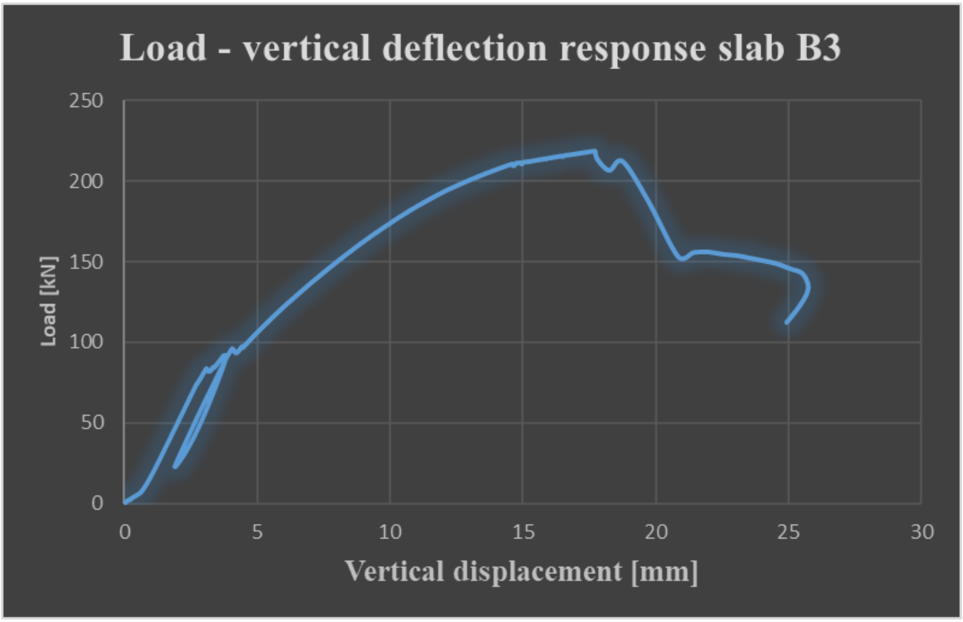


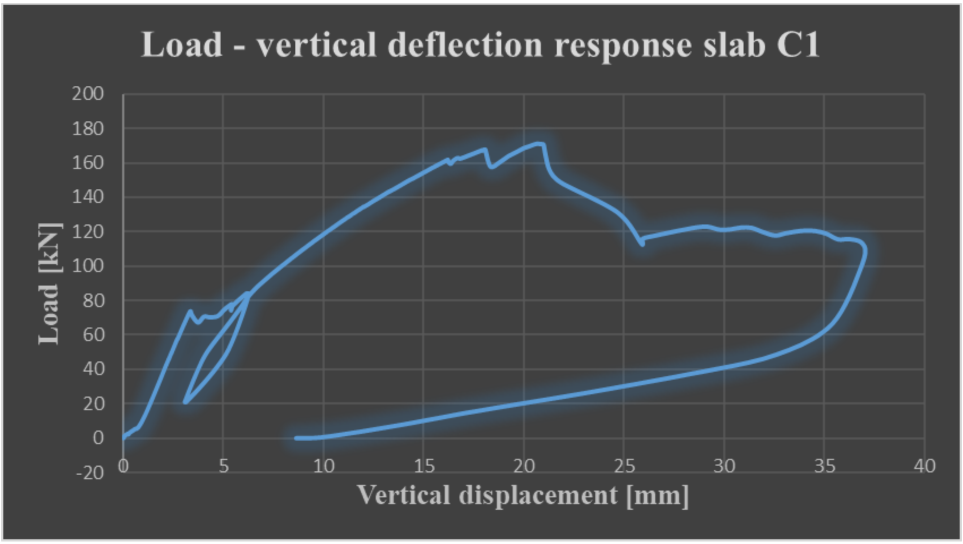


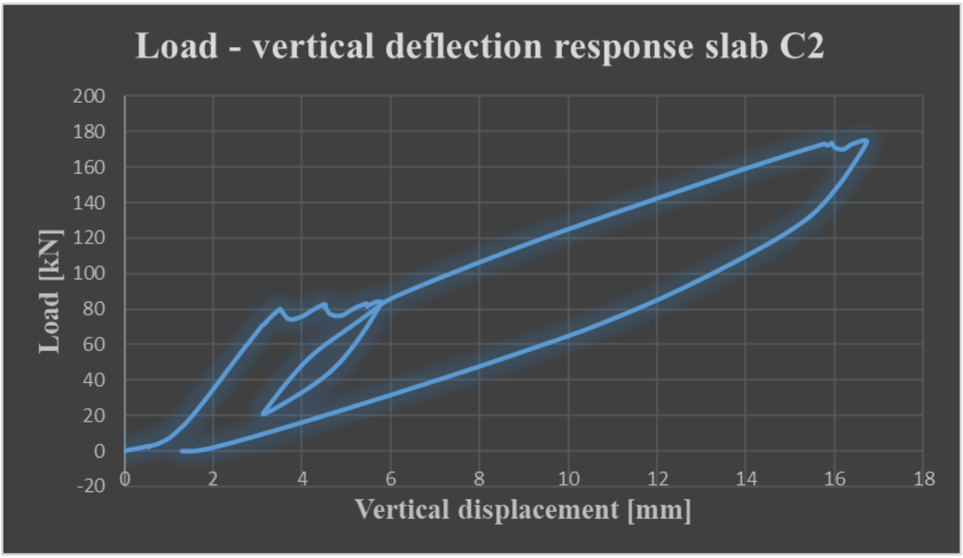

Load - vertical deflection response slap A3
Load [kN]
250
200
150
100
50
0
-50
0
5
10
15
20
25
30
Vertical displacement [mm]

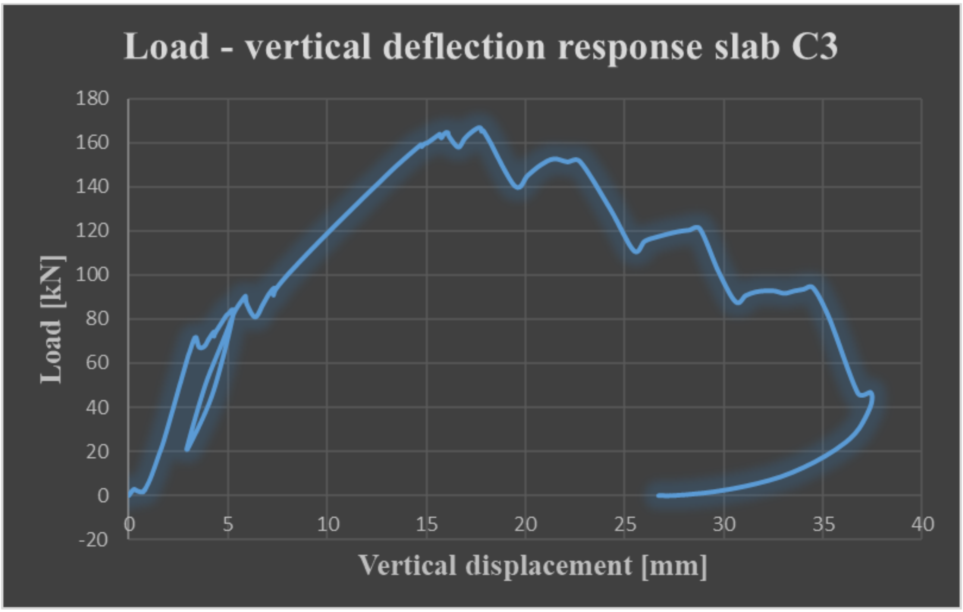

Load - vertical deflection response slab B1
Load [kN]
250
200
150
100
50
0
0
5
10
15
20
25
30
35
Vertical displacement [mm]

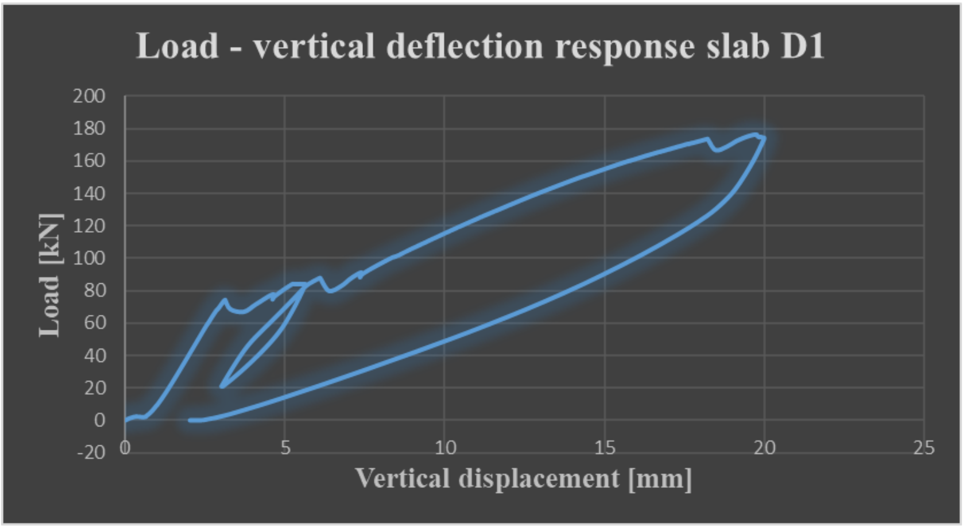

Load - vertical deflection response slab B2
Load [kN]
300
250
200
150
100
50
0
-50
-5
0
5
10
15
20
25
30
35
40
Veritcal displacement [mm]

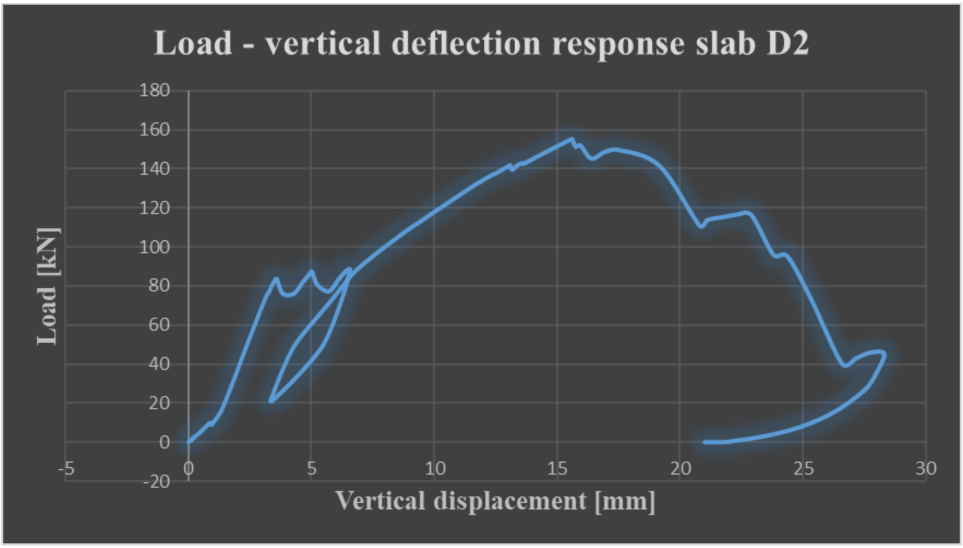

Load - vertical deflection response slab B3
Load [kN]
250
200
150
100
50
0
0
5
10
15
20
25
30
Vertical displacement [mm]

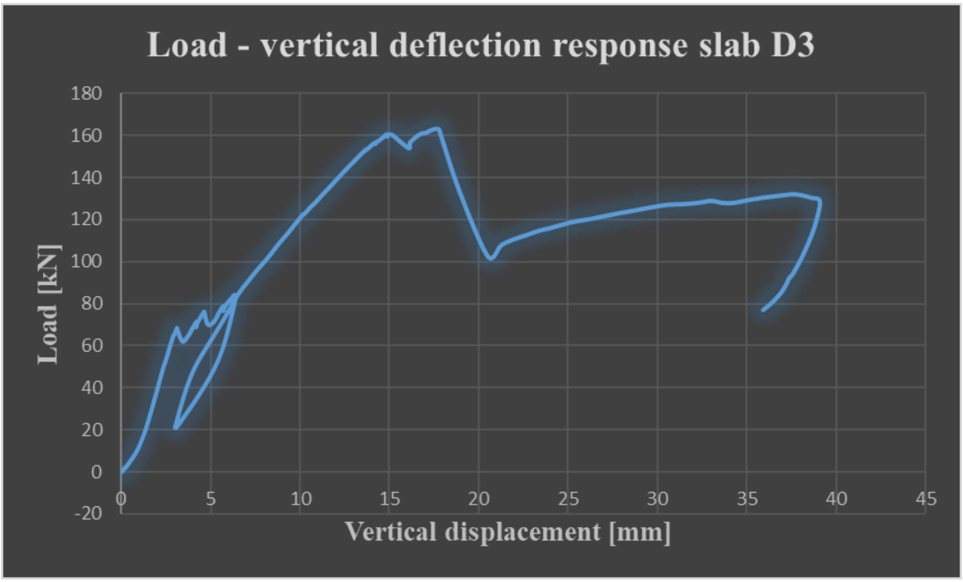

Load - vertical deflection response slab C1
Load [kN]
Vertical displacement [mm]
200
180
160
140
120
100
80
60
40
20
0
-20
0
5
10
15
20
25
30
35
40

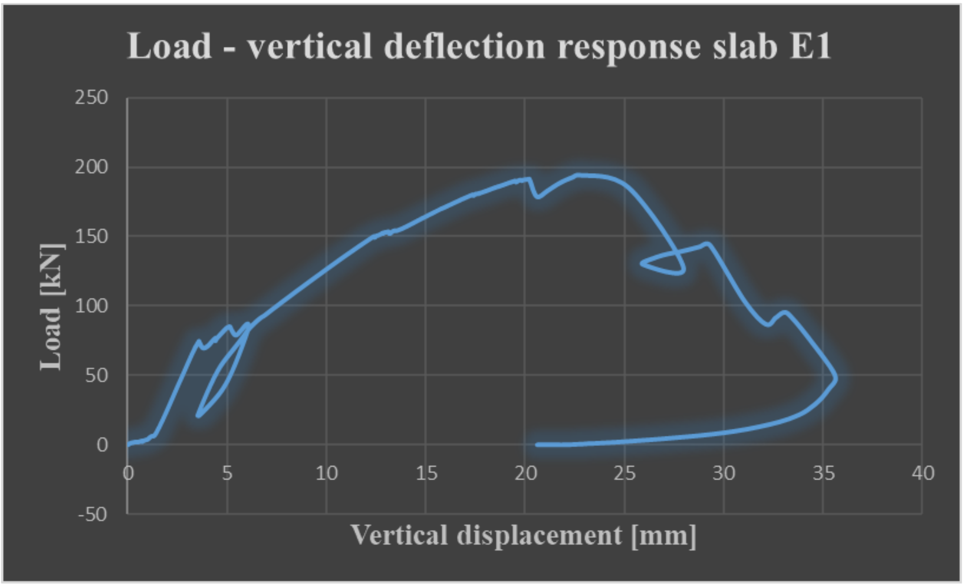

Load - vertical deflection response slab C2
Load [kN]
Vertical displacement [mm]
200
180
160
140
120
100
80
60
40
20
0
-20
0
2
4
6
8
10
12
14
16
18

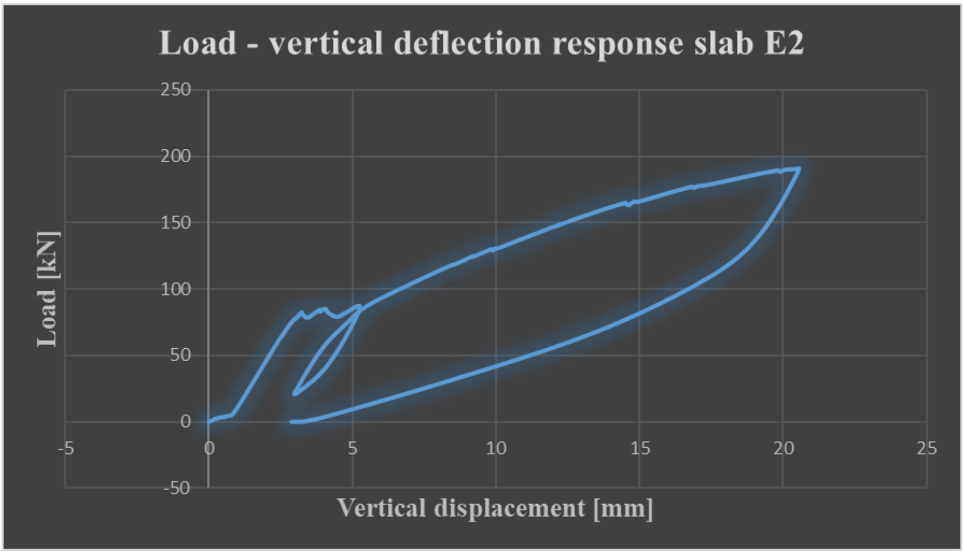

Load - vertical deflection response slab C3
Load [kN]
180
160
140
120
100
80
60
40
20
0
-20
0
5
10
15
20
25
30
35
40
Vertical displacement [mm]

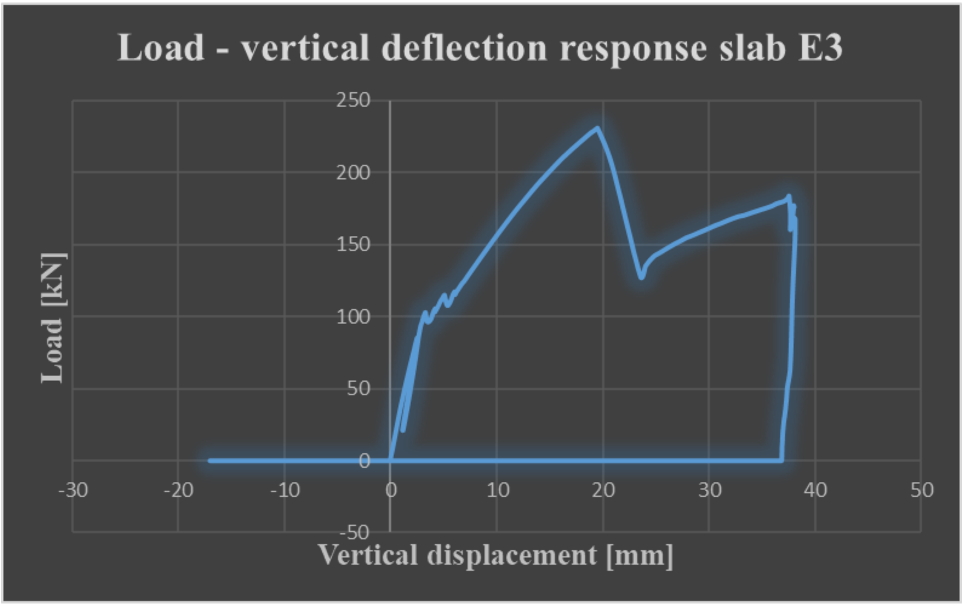

Load - vertical deflection response slab D1
Load [kN]
200
180
160
140
120
100
80
60
40
20
0
-20
0
5
10
15
20
25
Vertical displacement [mm]


Load - vertical deflection response slab D2
Load [kN]
Vertical displacement [mm]
180
160
140
120
100
80
60
40
20
0
-20
-5
0
5
10
15
20
25
30


Load - vertical deflection response slab D3
Load [kN]
Vertical displacement [mm]
180
160
140
120
100
80
60
40
20
0
-20
0
5
10
15
20
25
30
35
40
45


Load - vertical deflection response slab E1
Load [kN]
250
200
150
100
50
0
-50
0
5
10
15
20
25
30
35
40
Vertical displacement [mm]

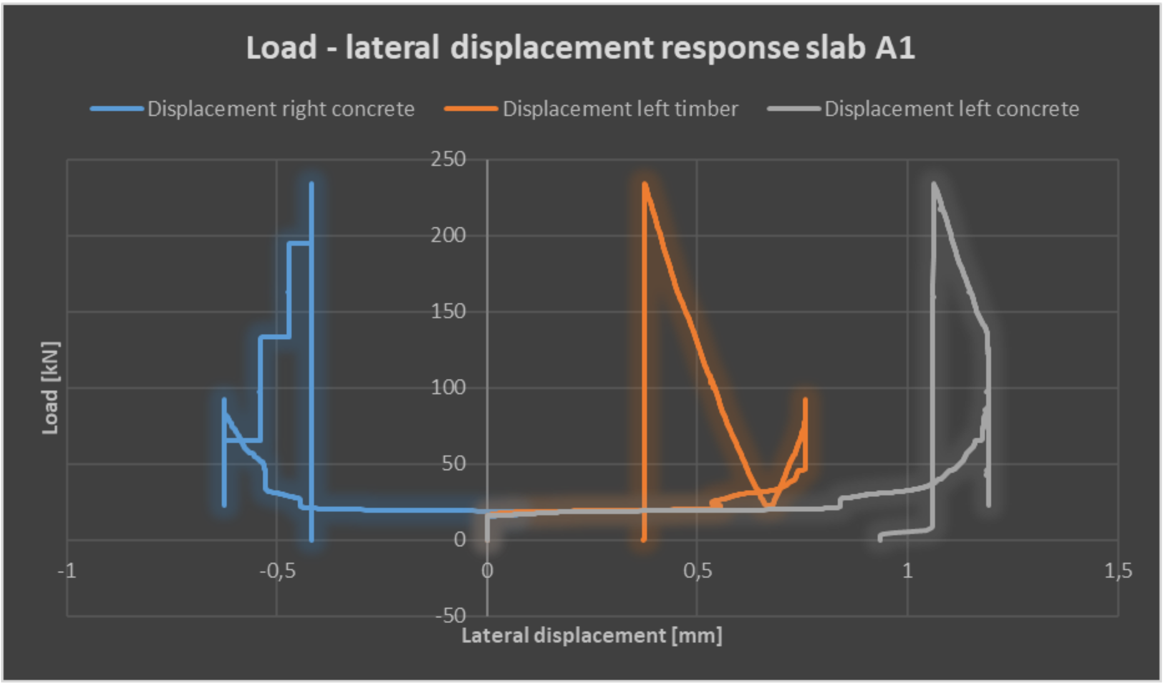

Load - vertical deflection response slab E2
Load [kN]
250
200
150
100
50
0
-50
-5
0
5
10
15
20
25
Vertical displacement [mm]

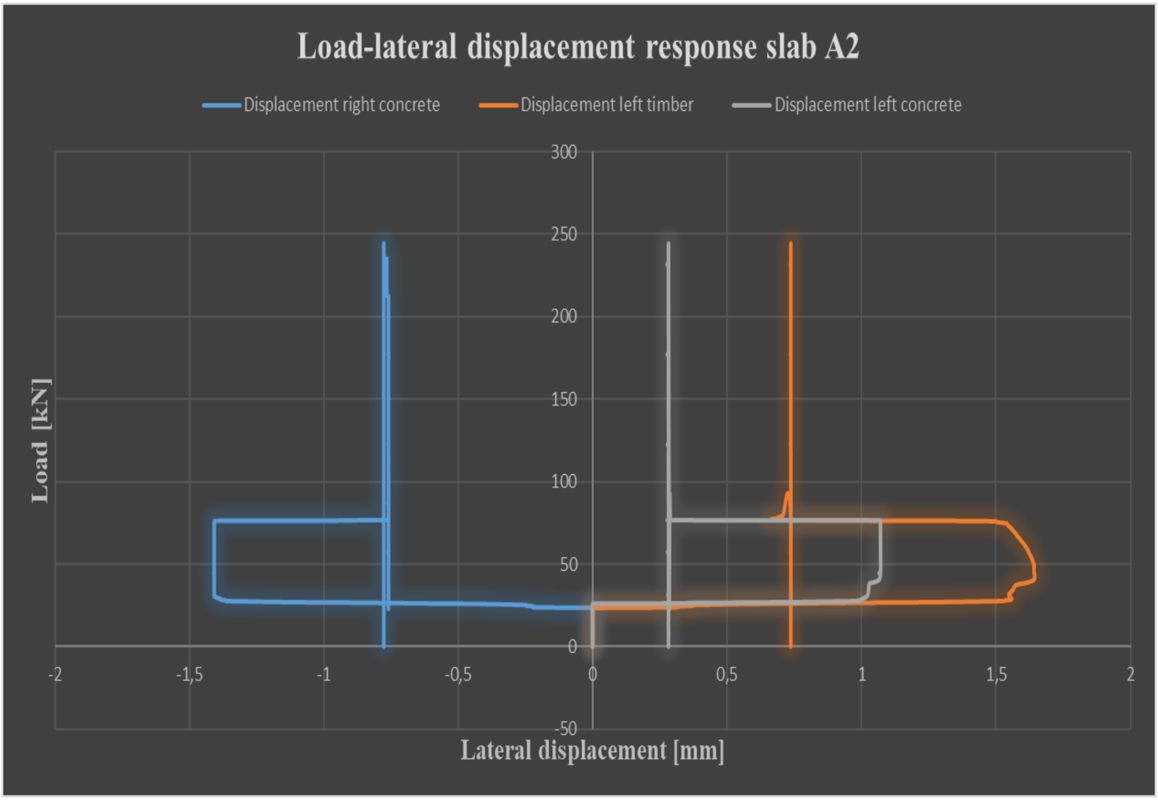

Load - vertical deflection response slab E3
Load [kN]
250
200
150
100
50
0
-50
-30
-20
-10
0
10
20
30
40
50
Vertical displacement [mm]

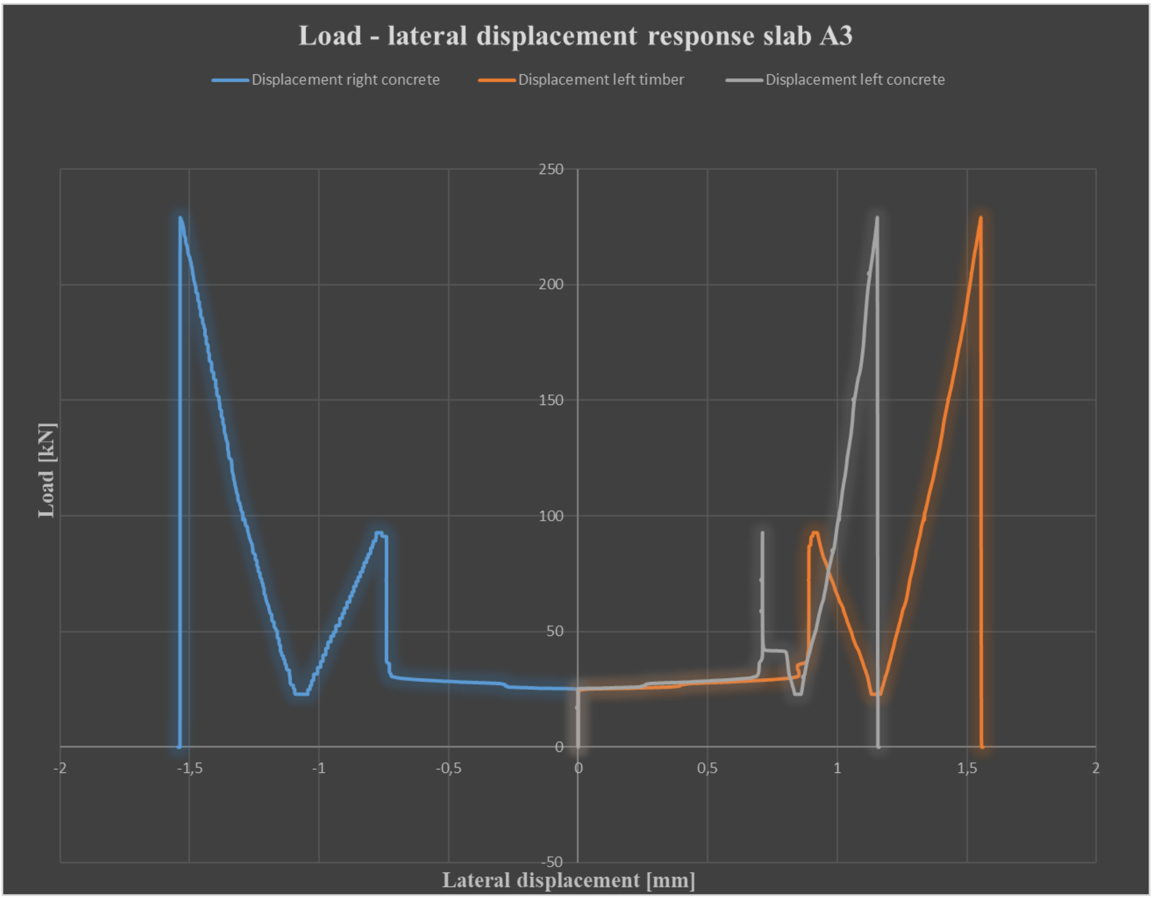
Appendix D.

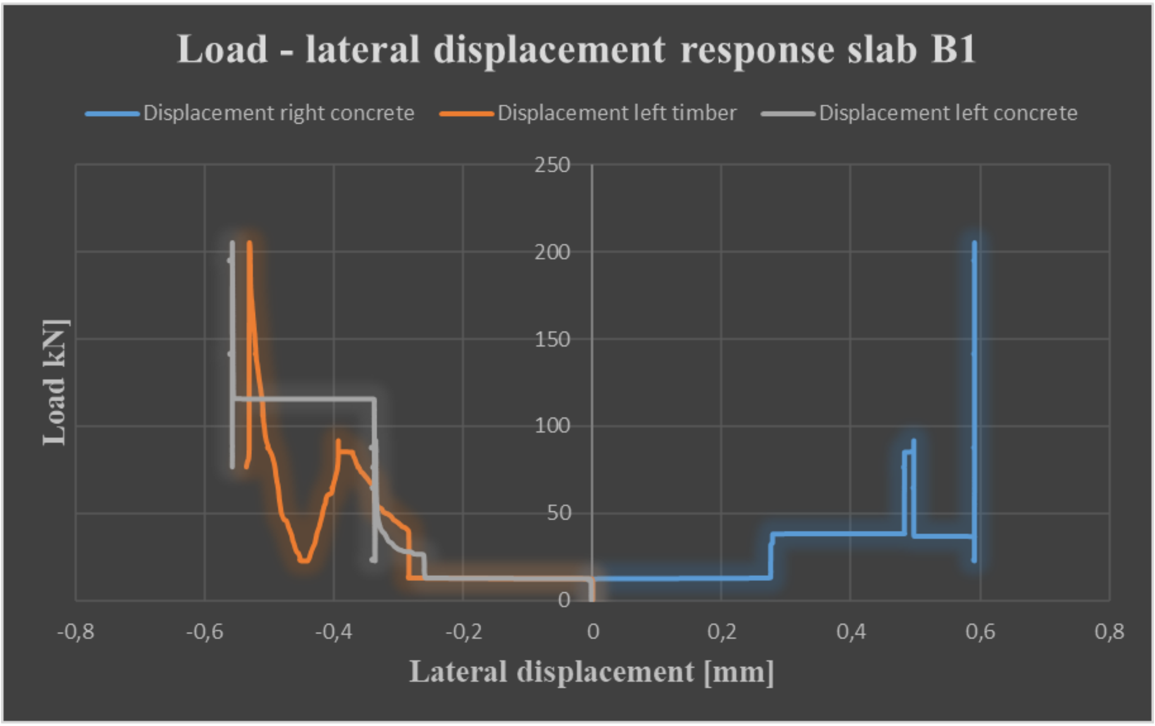
Graphs, lateral deflection Catman

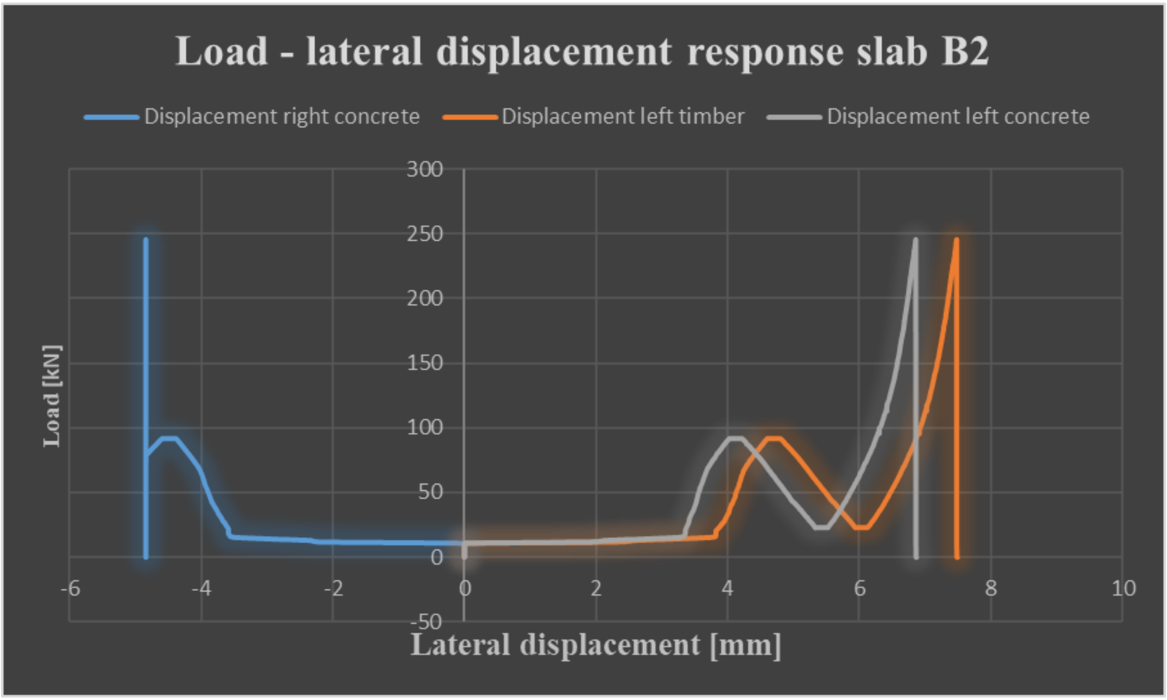
Appendix D. Graphs, lateral deflection Catman

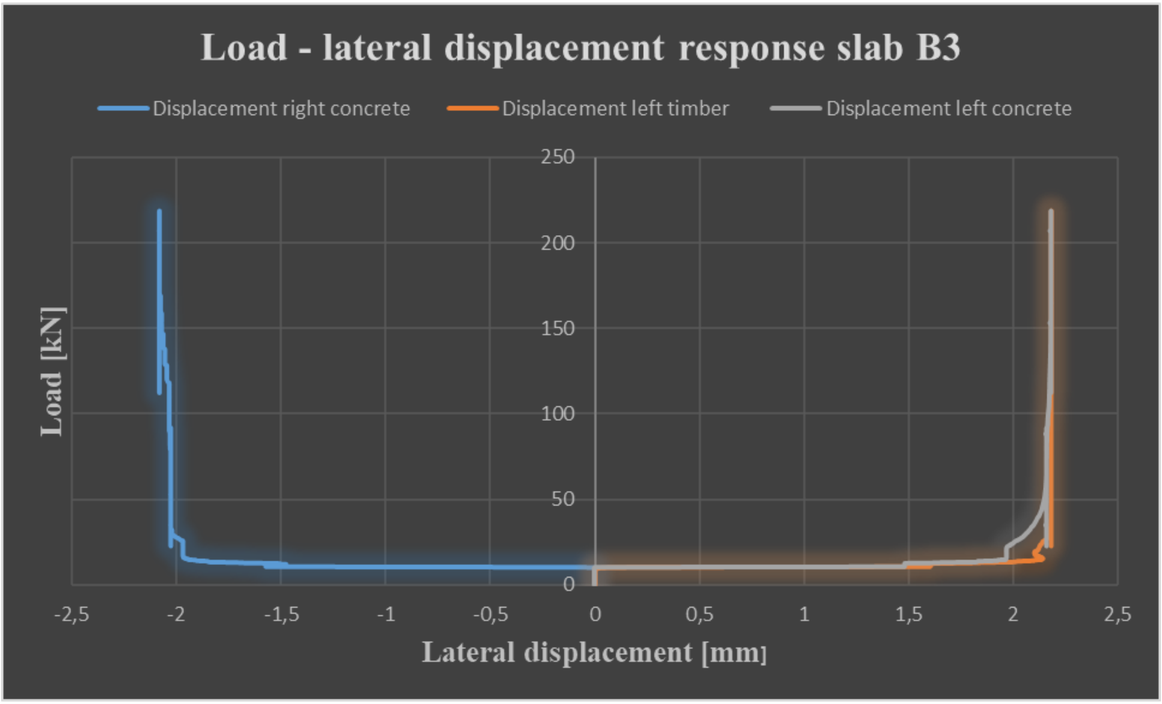


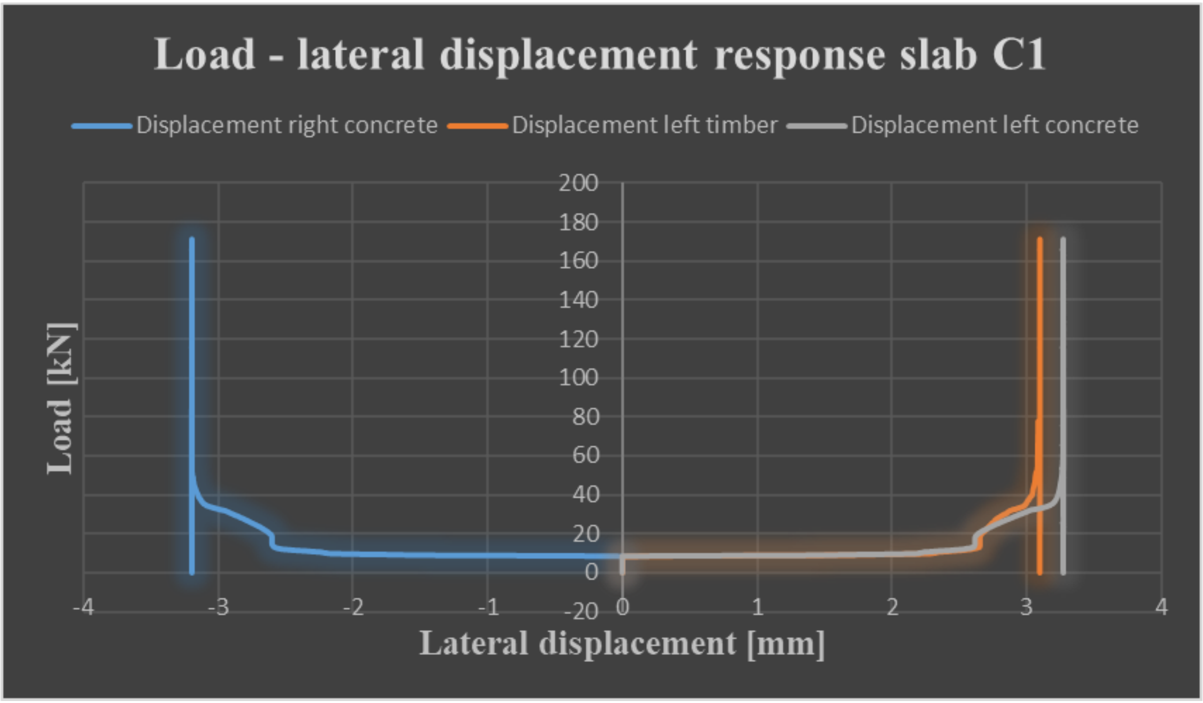



Load - lateral displacement response slab A3
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
Lateral displacement [mm]

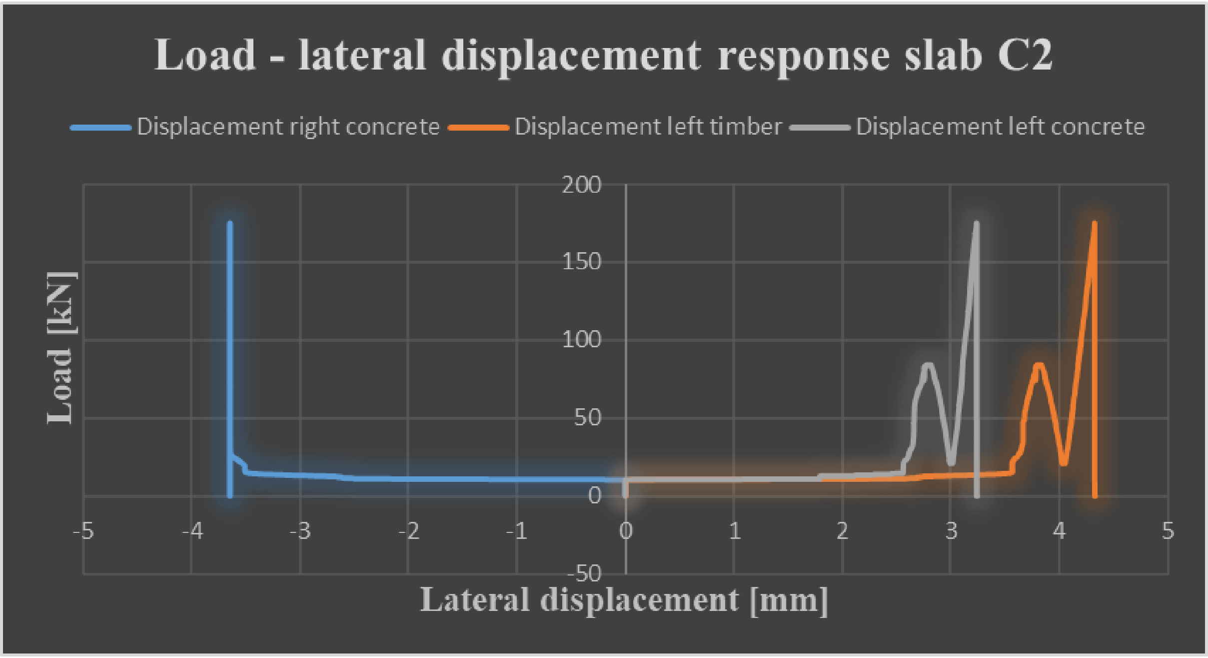

Load - lateral displacement response slab B1
Displacement right concrete
Displacement left timber
Displacement left concrete
Load kN]
Lateral displacement [mm]


Load - lateral displacement response slab B2
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
300
250
200
150
100
50
0
-50
-6
-4
-2
0
2
4
6
8
10
Lateral displacement [mm]

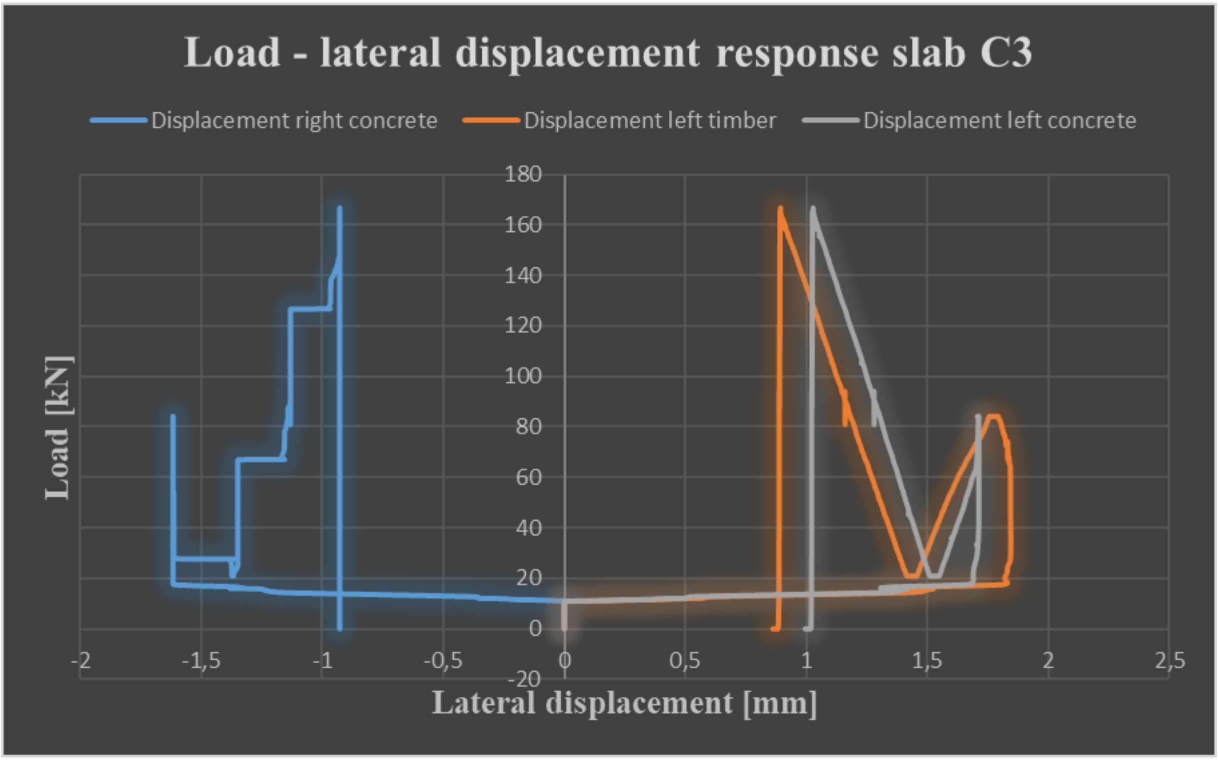

Load - lateral displacement response slab B3
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
250
200
150
100
50
0
-2,5
-2
-1,5
-1
-0,5
0
0,5
1
1,5
2
2,5
Lateral displacement [mm]

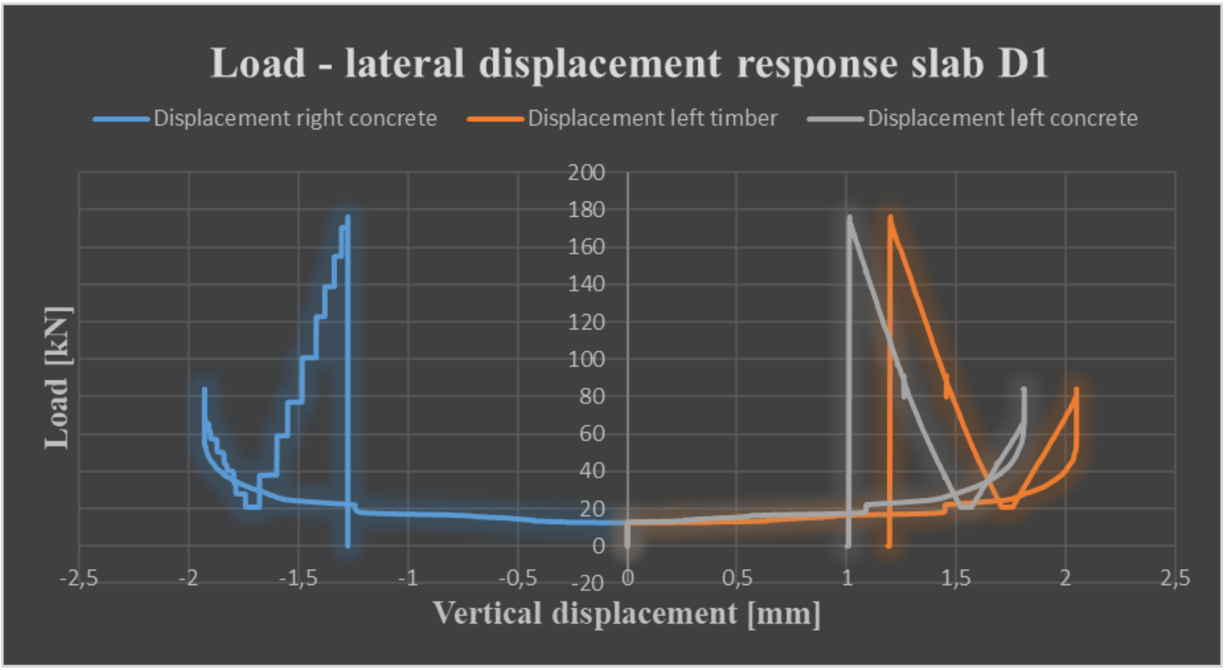


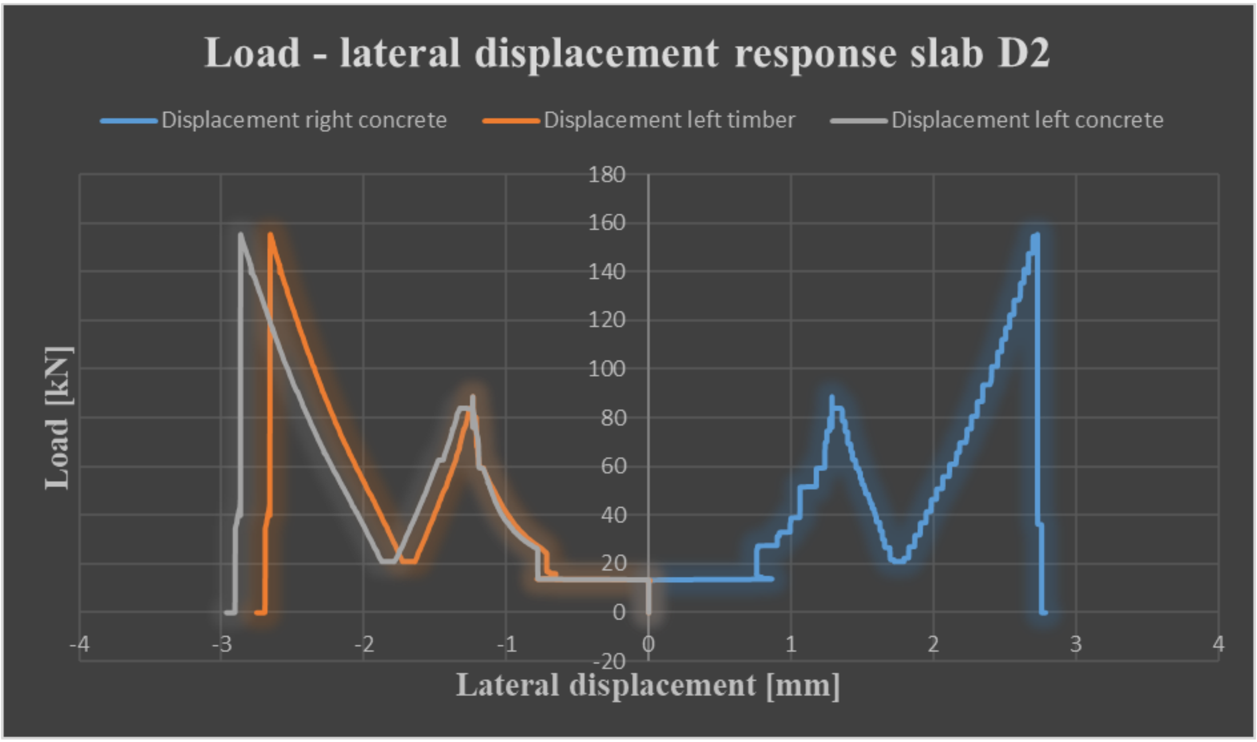
Load - lateral displacement response slab C1
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
Lateral displacement [mm]

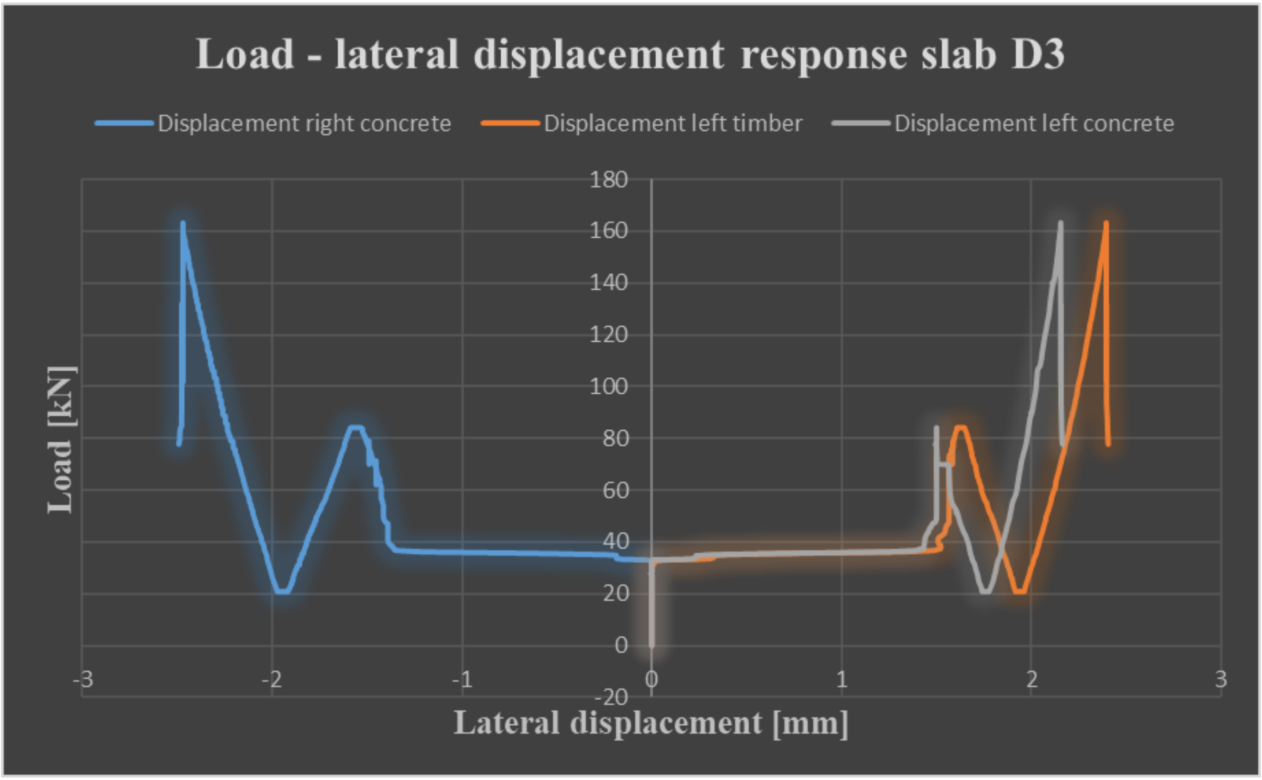


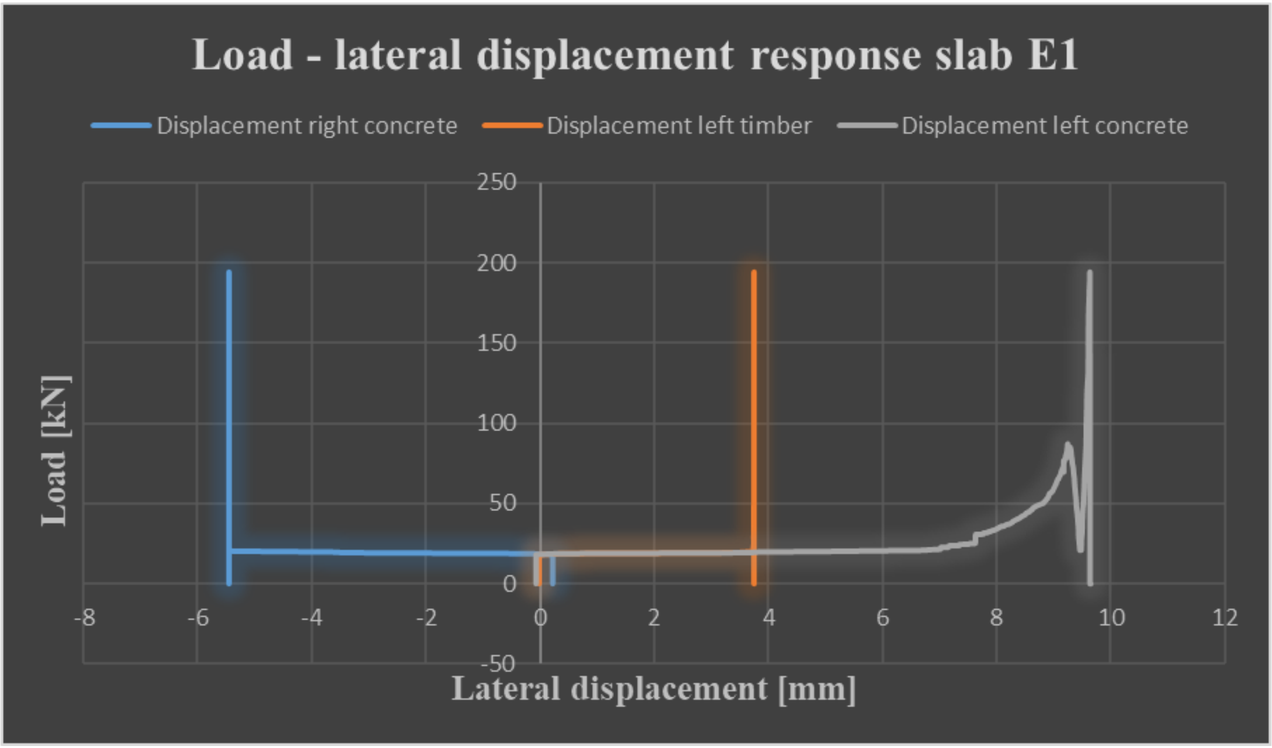
Load - lateral displacement response slab C2
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
Lateral displacement [mm]

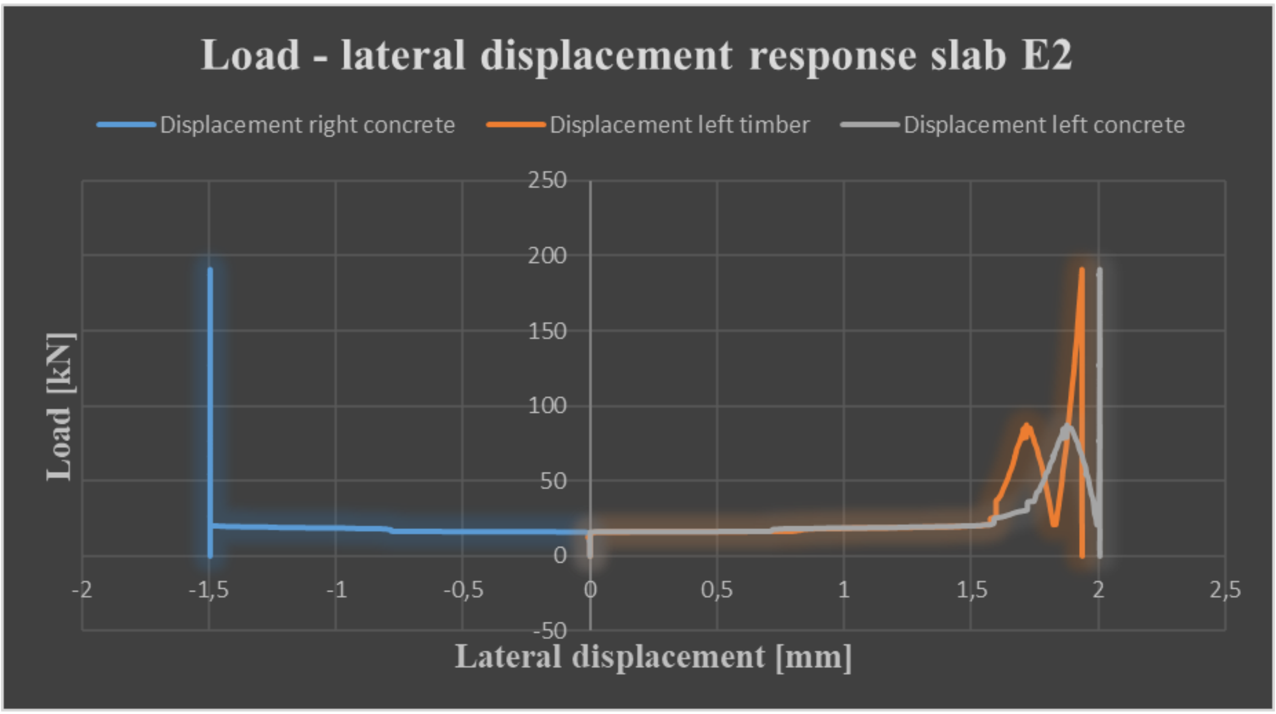

Load - lateral displacement response slab C3
Displacement right concrete
Displacement left timber
Displacement left concrete
180
160
140
120
100
80
60
40
20
0
-20
-2
-1,5
-1
-0,5
0
0,5
1
1,5
2
2,5
Load [kN]
Lateral displacement [mm]

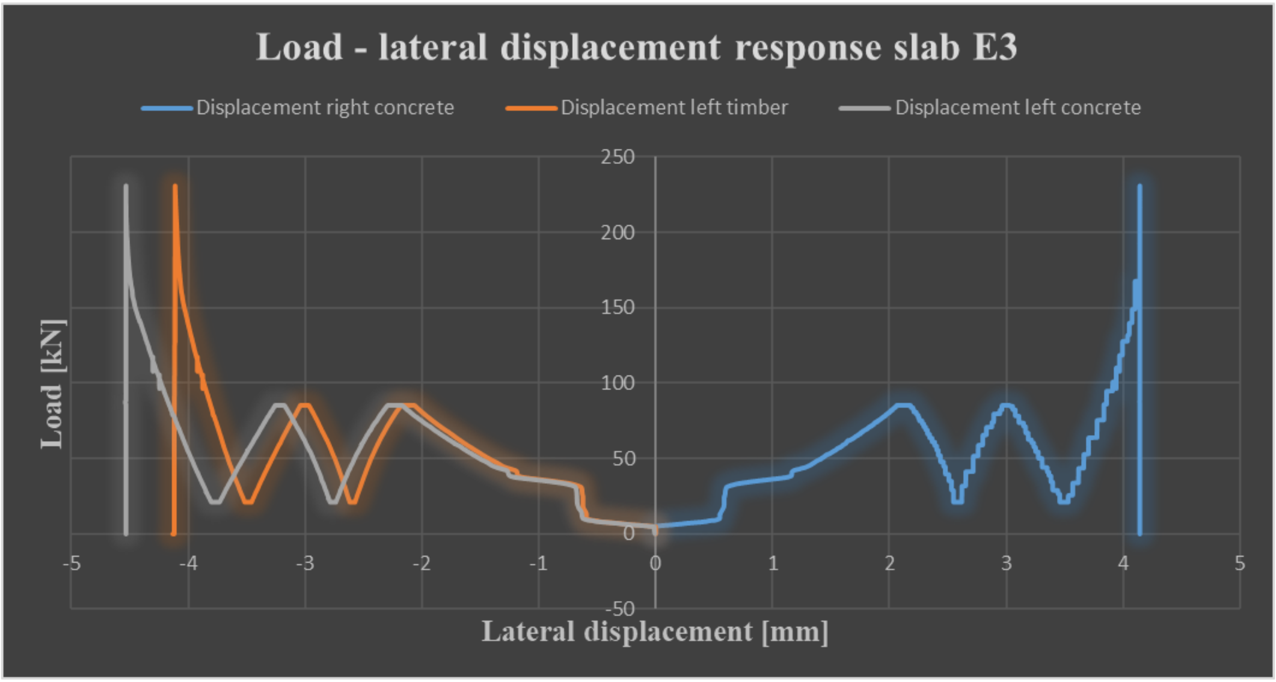

Load - lateral displacement response slab D1
Displacement right concrete
Displacement left timber
Displacement left concrete
200
180
160
140
120
100
80
60
40
20
0
-20
-2,5
-2
-1,5
-1
-0,5
0
0,5
1
1,5
2
2,5
Load [kN]
Vertical displacement [mm]


Load - lateral displacement response slab D2
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
Lateral displacement [mm]
180
160
140
120
100
80
60
40
20
0
-20
-4
-3
-2
-1
0
1
2
3
4


Load - lateral displacement response slab D3
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
Lateral displacement [mm]
180
160
140
120
100
80
60
40
20
0
-20
-3
-2
-1
0
1
2
3


Load - lateral displacement response slab E1
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
250
200
150
100
50
0
-50
-8
-6
-4
-2
0
2
4
6
8
10
12
Lateral displacement [mm]


Load - lateral displacement response slab E2
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
250
200
150
100
50
0
-50
-2
-1,5
-1
-0,5
0
0,5
1
1,5
2
2,5
Lateral displacement [mm]


Load - lateral displacement response slab E3
Displacement right concrete
Displacement left timber
Displacement left concrete
Load [kN]
250
200
150
100
50
0
-50
-5
-4
-3
-2
-1
0
1
2
3
4
5
Lateral displacement [mm]

Appendix E.

Graphs, Toni Technik

E.1 Compressive strength of cubes
E.2 CLT-concrete slabs

Toni Technik

11.05.2023

Simple standard protocol

Parameter table:

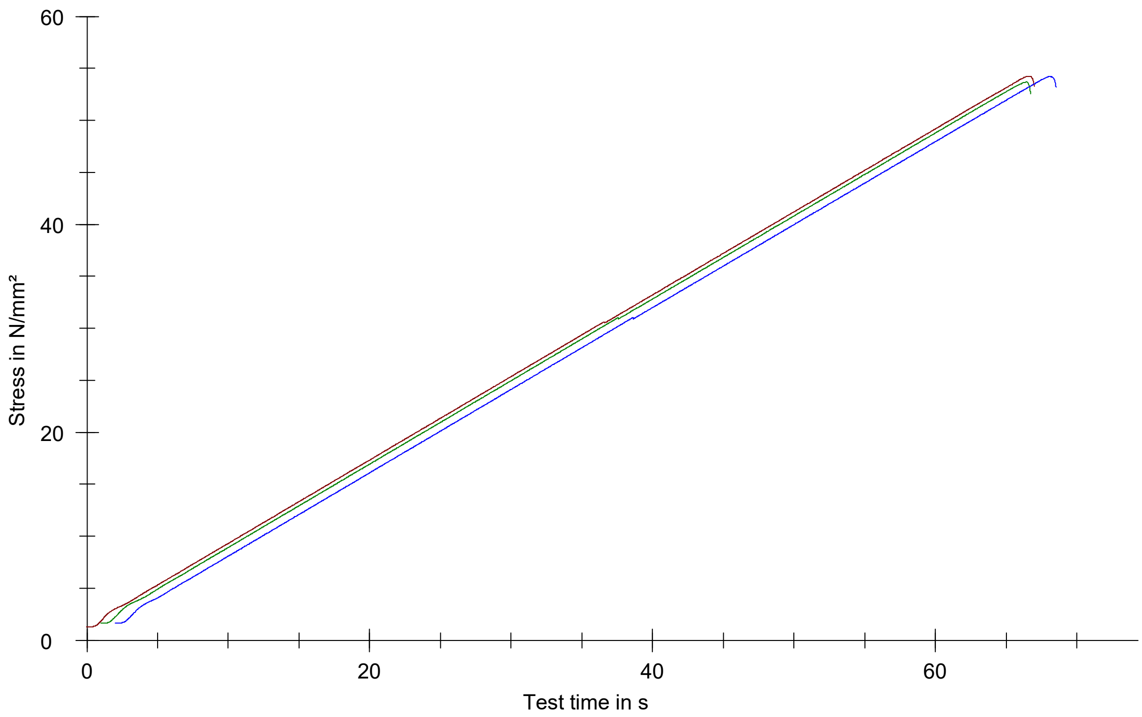
Test protocol : Compression test for cubes
Tester : Tollak -V2023
Customer :
Test standard : NS-EN 12390-3:2019
Strength grade :
Creation date : April 2023
Age : 28 T
Other :

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 3 MN

Results:

Nr	Date	ID	a mm	b mm	A mm²	h mm	F_m kN	σ_m N/mm²
1	10.05.2023	1	100,0	100,0	10000,0	100,0	542,45	54,25
2	10.05.2023	2	100,0	100,0	10000,0	100,0	536,77	53,68
3	10.05.2023	2	100,0	100,0	10000,0	100,0	542,09	54,21

Series graphics:



Statistics:

Series n = 3	a mm	b mm	A mm²	h mm	F_m kN	σ_m N/mm²
$\bar{x}$	100,0	100,0	10000,0	100,0	540,44	54,04
s	0,0	0,0	0,0	0,0	3,18	0,32
ν	0,00	0,00	0,00	0,00	0,59	0,59

Parameter table:

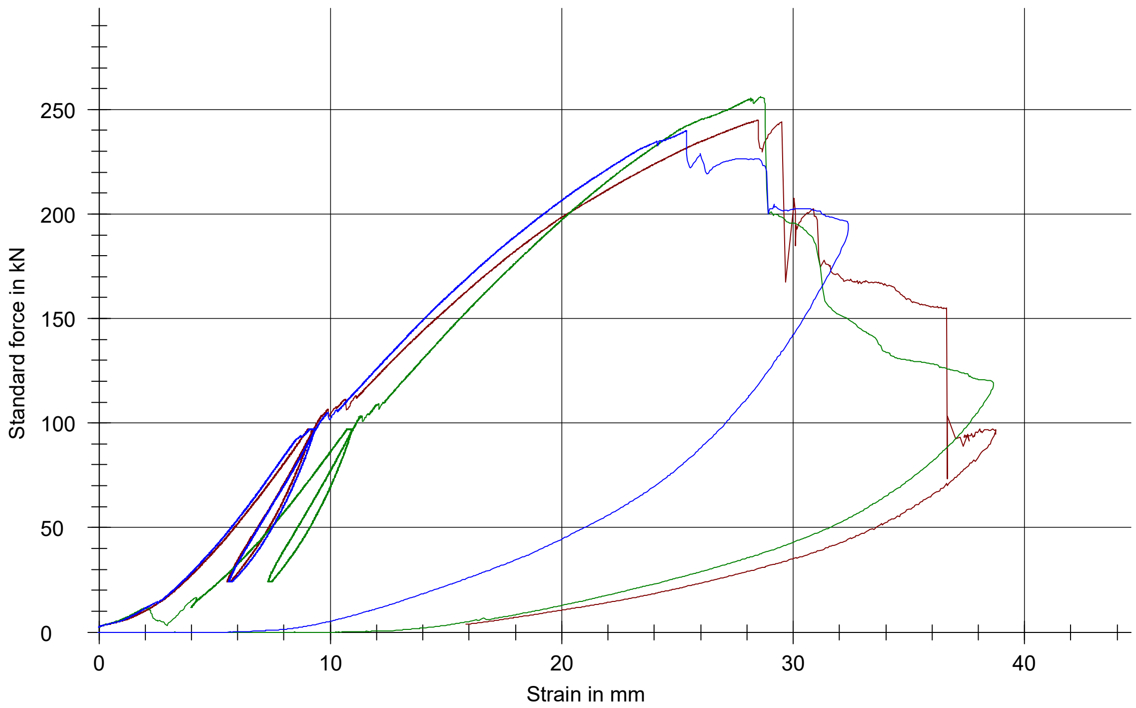
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
12.05.2023	Plate _A1	1600,0	600,0	200,0	244,97
11.05.2023	Plate _A2	1600,0	600,0	200,0	255,94
11.05.2023	Plate _A3	1600,0	600,0	200,0	239,75

Series graphics:



Statistics:

Series n = 3	a mm	b mm	h mm	F_m kN
$\overline{x}$	1600,0	600,0	200,0	246,88
s	0,0	0,0	0,0	8,26
ν	0,00	0,00	0,00	3,35

Parameter table:

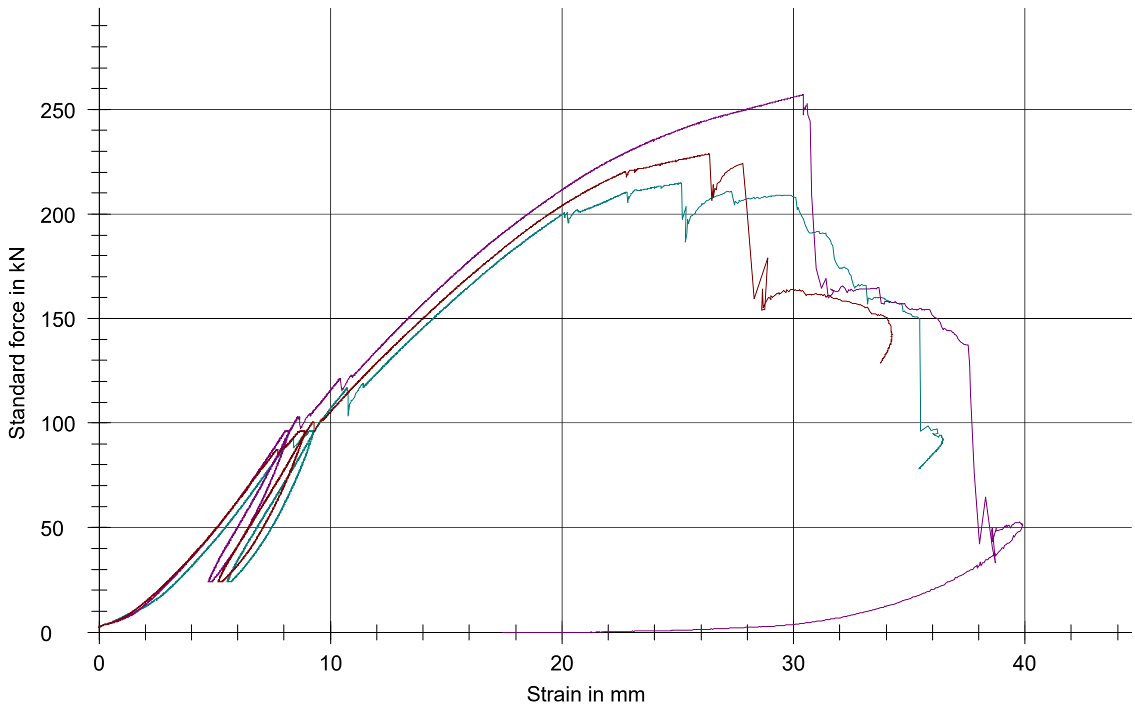
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _B1	1600,0	600,0	200,0	214,97
11.05.2023	Plate _B2	1600,0	600,0	200,0	257,01
11.05.2023	Plate _B3	1600,0	600,0	200,0	228,83

Series graphics:



Statistics:

Series n = 3	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	233,60
s	0,0	0,0	0,0	21,43
ν	0,00	0,00	0,00	9,17

Parameter table:

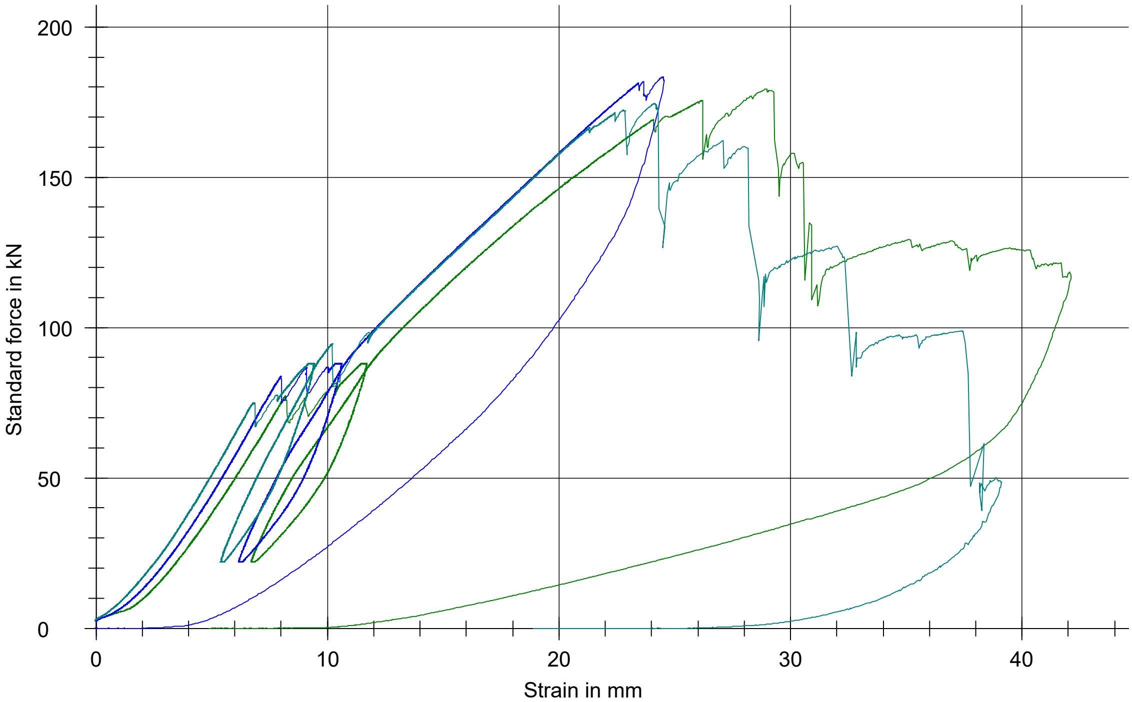
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _C1	1600,0	600,0	200,0	179,40
11.05.2023	Plate _C2	1600,0	600,0	200,0	183,30
10.05.2023	Plate _C3	1600,0	600,0	200,0	174,55

Series graphics:



Statistics:

Series n = 3	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	179,08
s	0,0	0,0	0,0	4,38
ν	0,00	0,00	0,00	2,45

Parameter table:

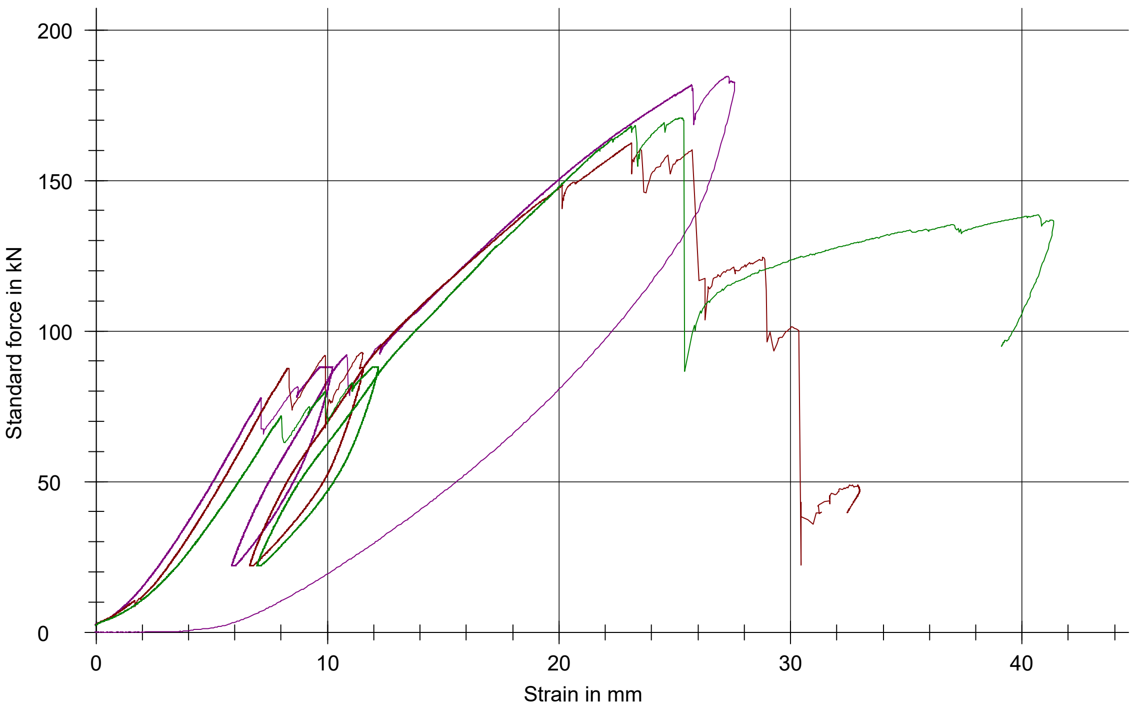
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
10.05.2023	Plate _D1	1600,0	600,0	200,0	184,55
10.05.2023	Plate _D2	1600,0	600,0	200,0	162,59
10.05.2023	Plate _D3	1600,0	600,0	200,0	170,83

Series graphics:



Statistics:

Series n = 3	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	172,66
s	0,0	0,0	0,0	11,09
ν	0,00	0,00	0,00	6,42

Toni Technik

Simple standard protocol

12.05.2023

Parameter table:

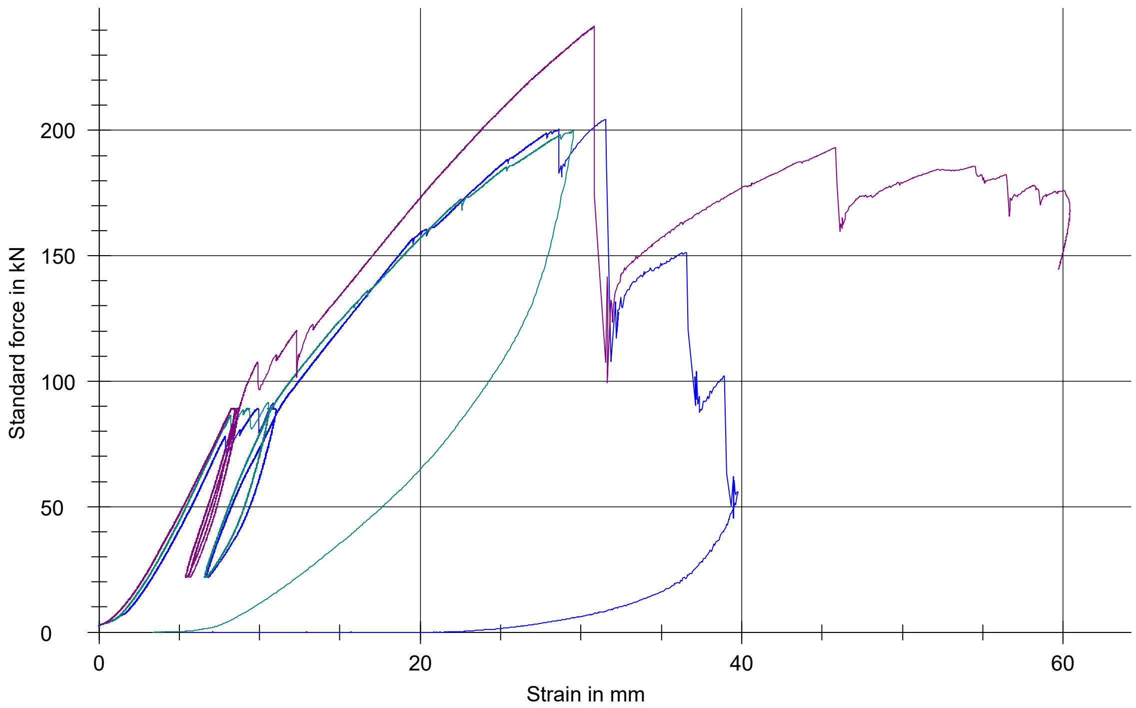
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
09.05.2023	Plate _E1	1600,0	600,0	200,0	204,34
08.05.2023	Plate _E2	1600,0	600,0	200,0	199,76
04.05.2023	Plate _E3	1600,0	600,0	200,0	241,44

Series graphics:



Statistics:

Series n = 3	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	215,18
s	0,0	0,0	0,0	22,86
ν	0,00	0,00	0,00	10,62

Parameter table:

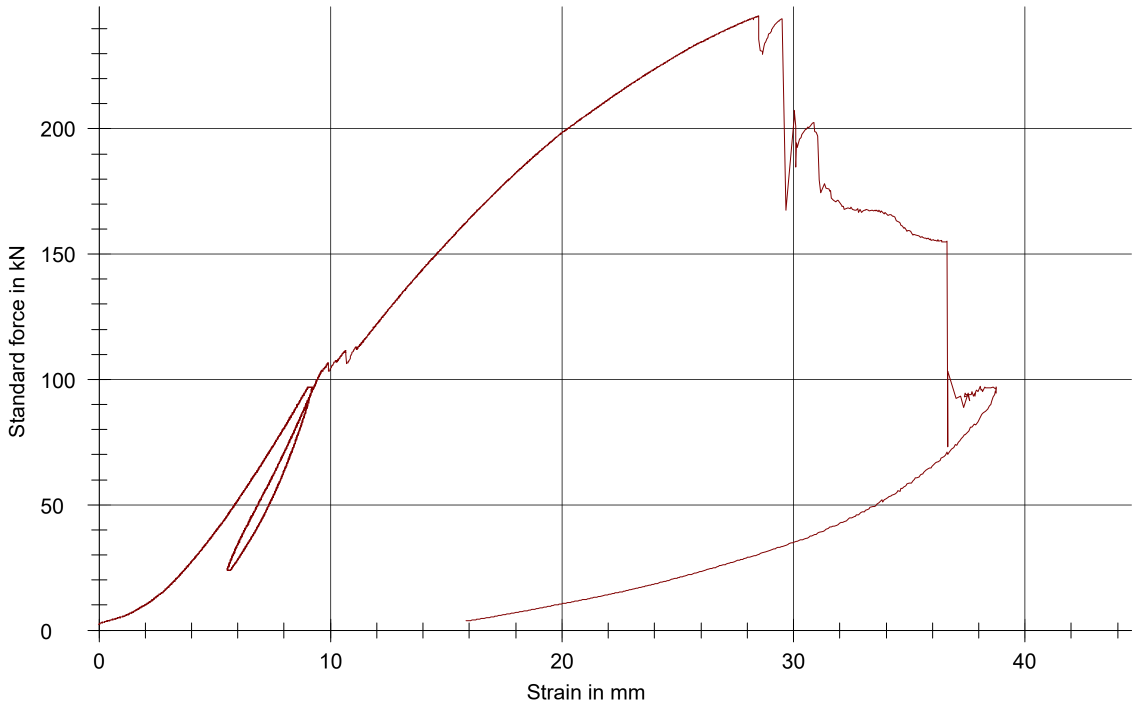
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
12.05.2023	Plate _A1	1600,0	600,0	200,0	244,97

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	244,97
s	-	-	-	-
ν	-	-	-	-

Parameter table:

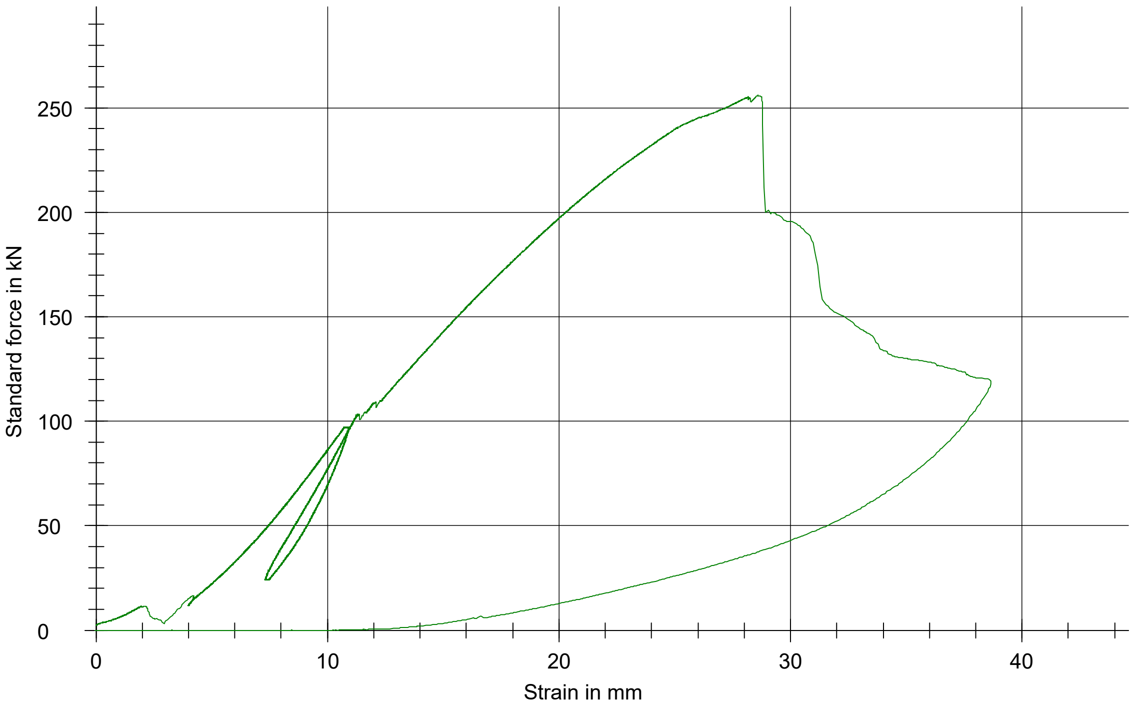
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _A2	1600,0	600,0	200,0	255,94

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	255,94
s	-	-	-	-
ν	-	-	-	-

Parameter table:

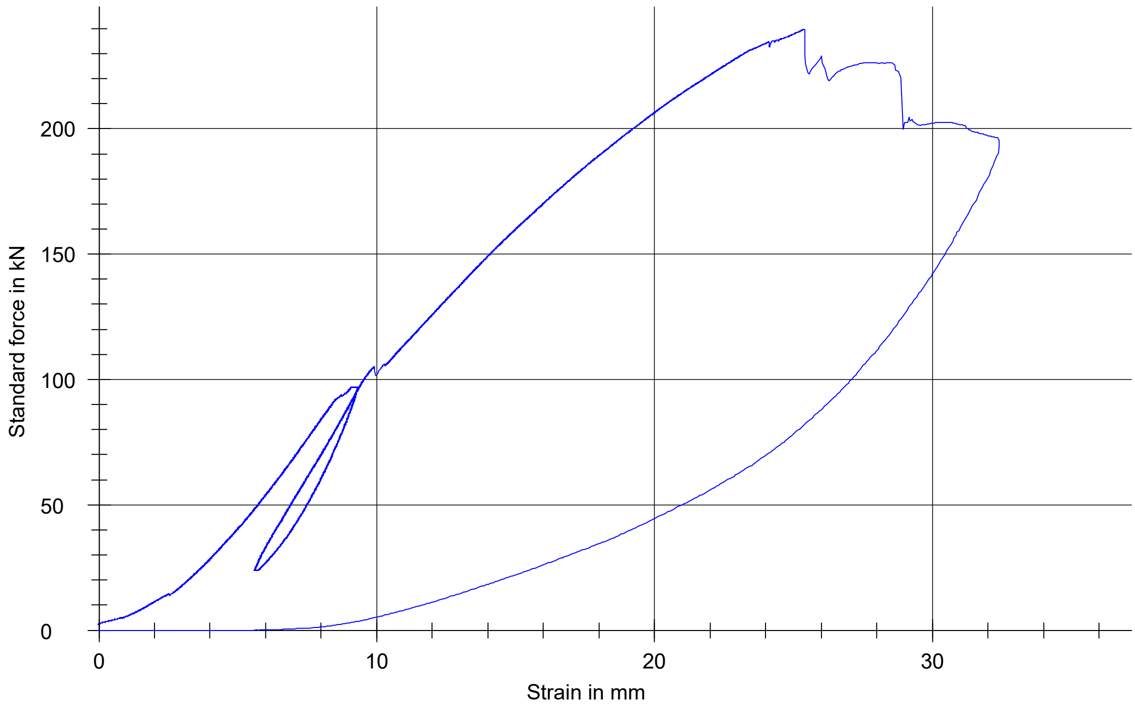
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _A3	1600,0	600,0	200,0	239,75

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	239,75
s	-	-	-	-
ν	-	-	-	-

Parameter table:

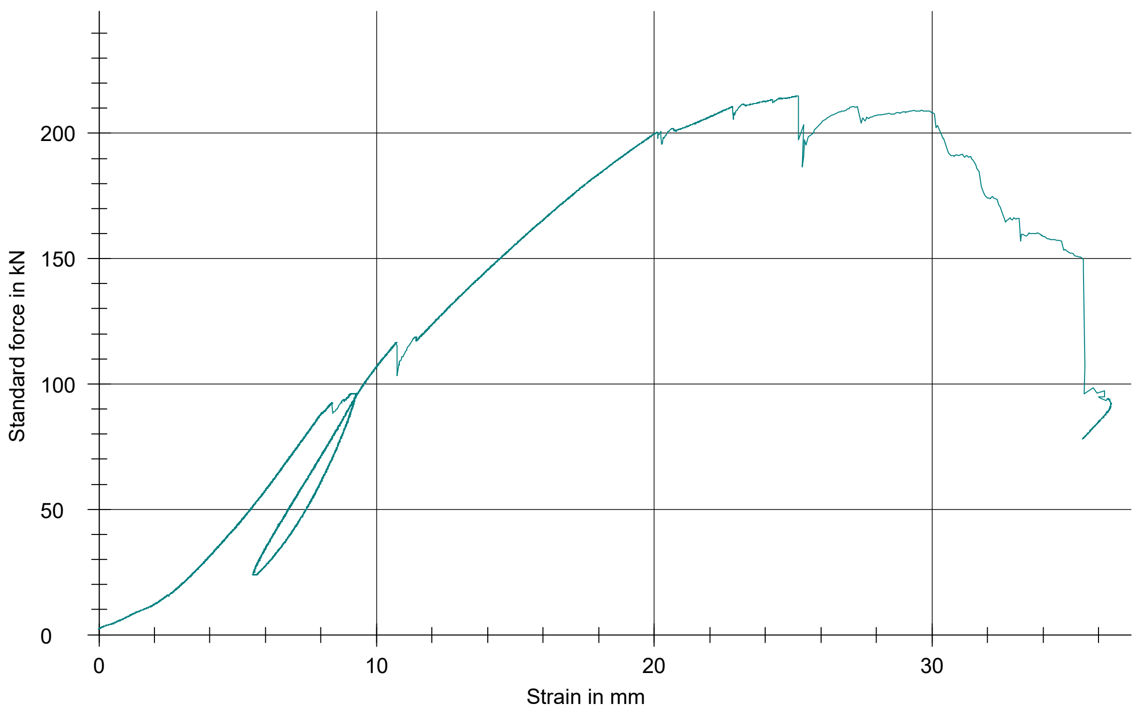
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _B1	1600,0	600,0	200,0	214,97

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	214,97
s	-	-	-	-
ν	-	-	-	-

Parameter table:

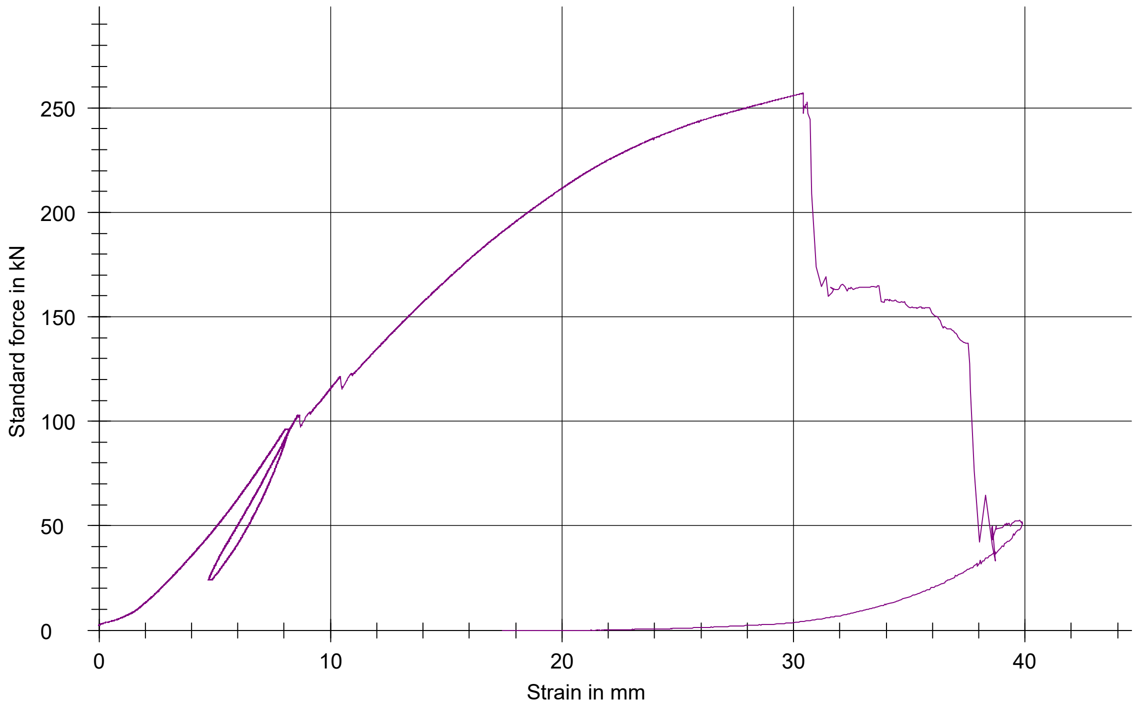
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _B2	1600,0	600,0	200,0	257,01

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	257,01
s	-	-	-	-
ν	-	-	-	-

Parameter table:

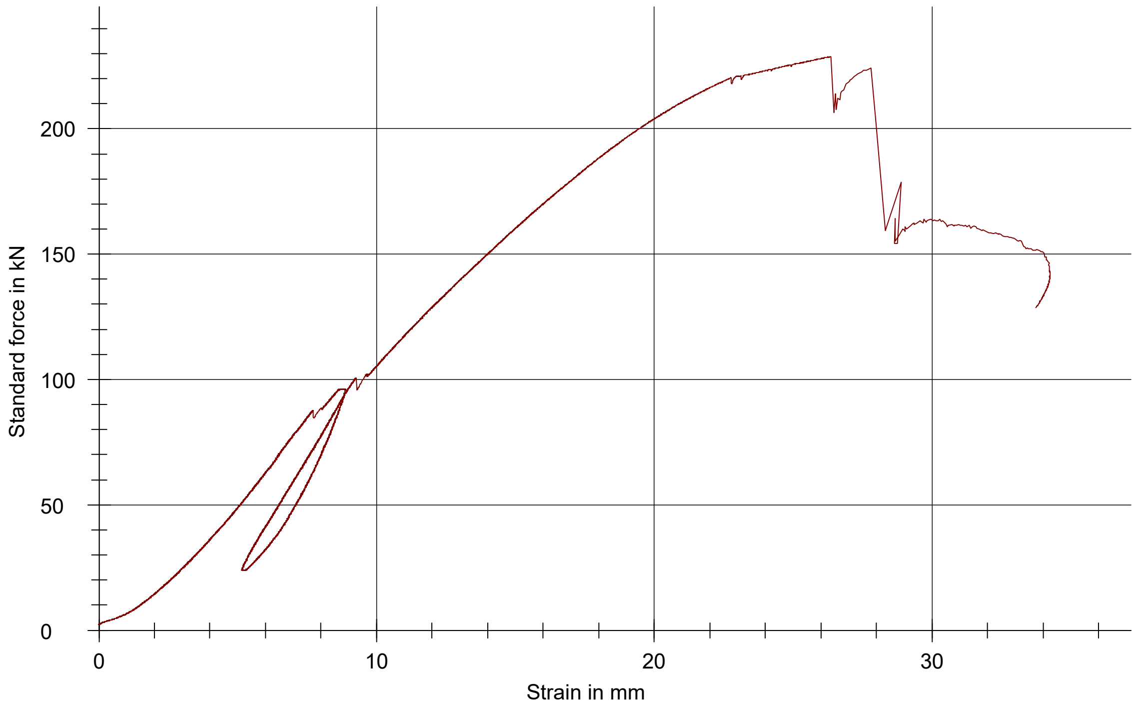
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _B3	1600,0	600,0	200,0	228,83

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\overline{x}$	1600,0	600,0	200,0	228,83
s	-	-	-	-
ν	-	-	-	-

Parameter table:

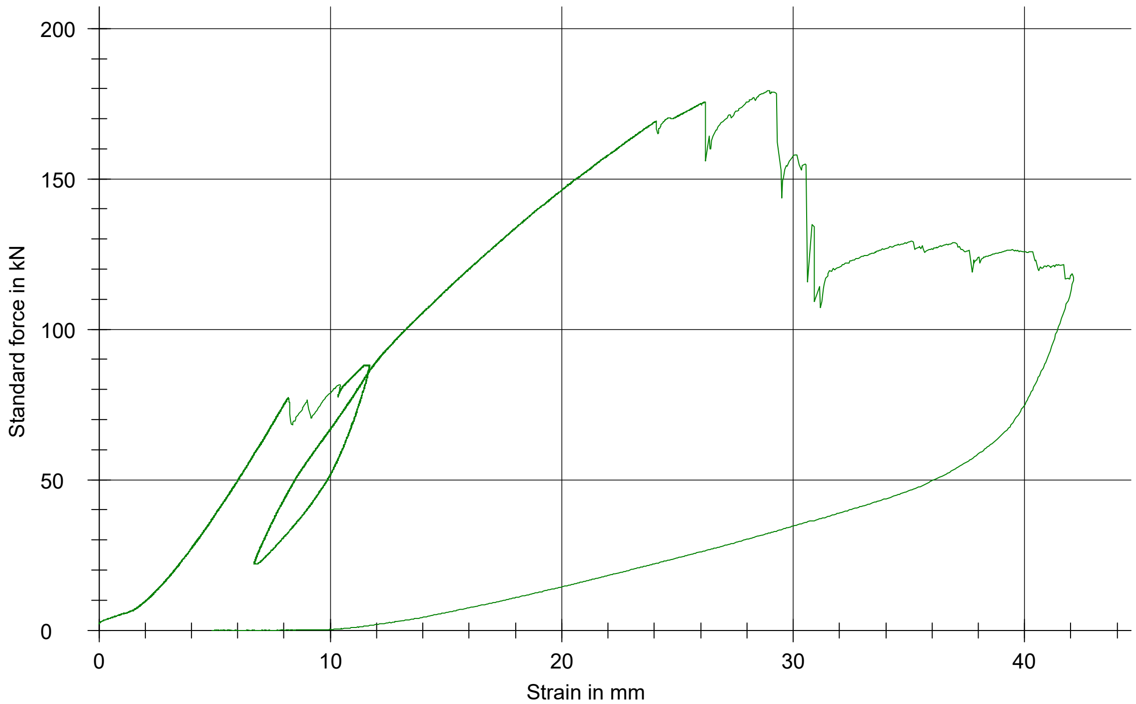
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _C1	1600,0	600,0	200,0	179,40

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	179,40
s	-	-	-	-
ν	-	-	-	-

Parameter table:

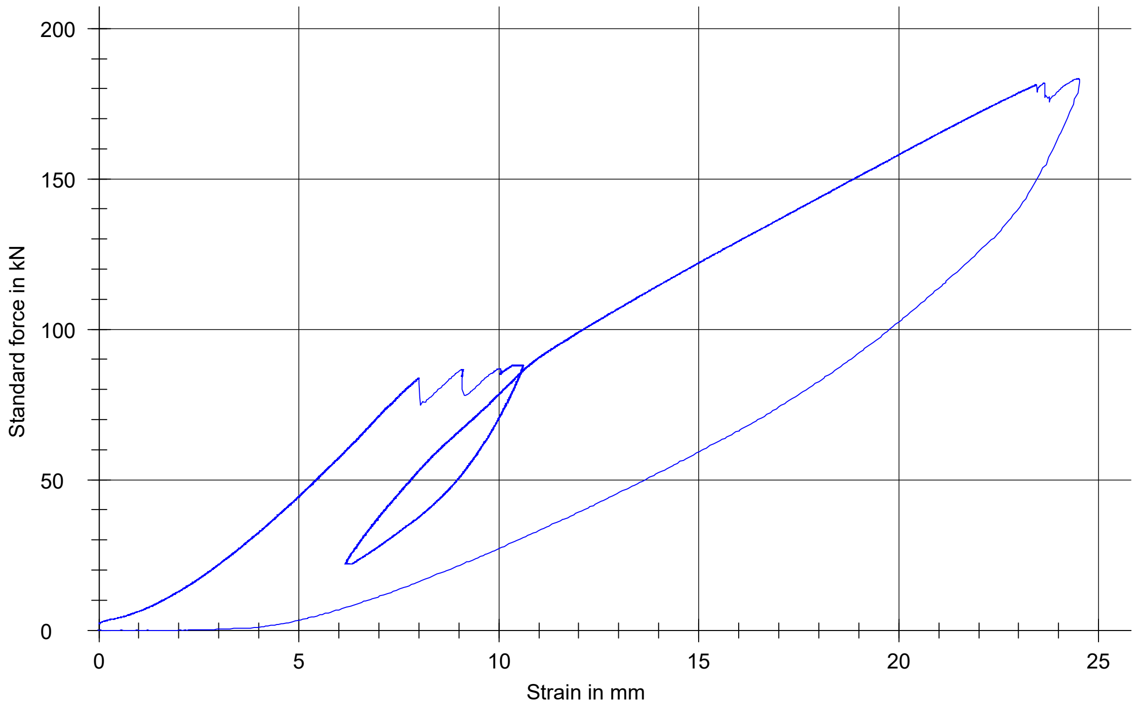
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
11.05.2023	Plate _C2	1600,0	600,0	200,0	183,30

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	183,30
s	-	-	-	-
ν	-	-	-	-

Parameter table:

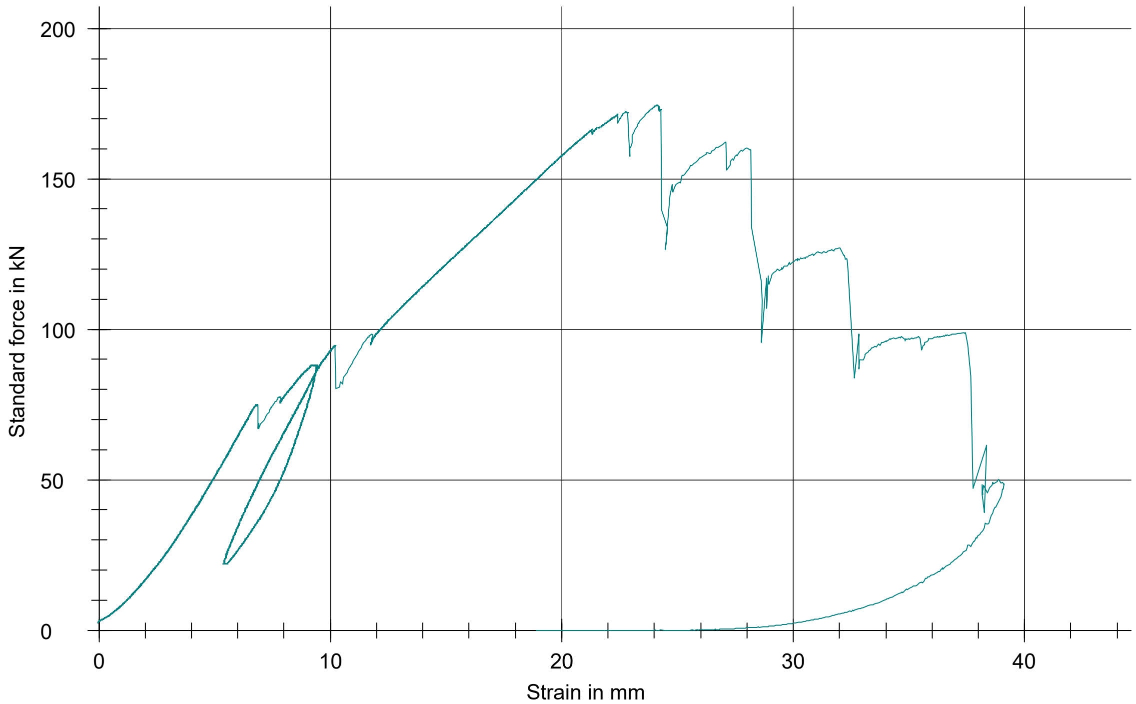
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
10.05.2023	Plate _C3	1600,0	600,0	200,0	174,55

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	174,55
s	-	-	-	-
ν	-	-	-	-

Parameter table:

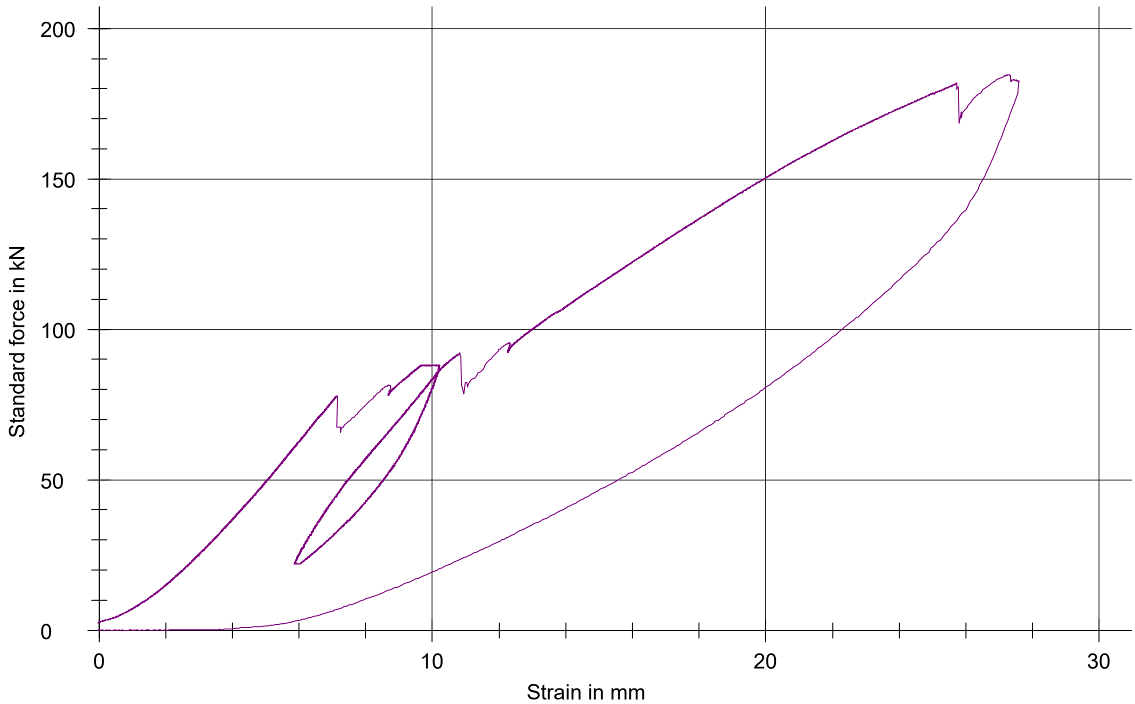
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
10.05.2023	Plate _D1	1600,0	600,0	200,0	184,55

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	184,55
s	-	-	-	-
ν	-	-	-	-

Parameter table:

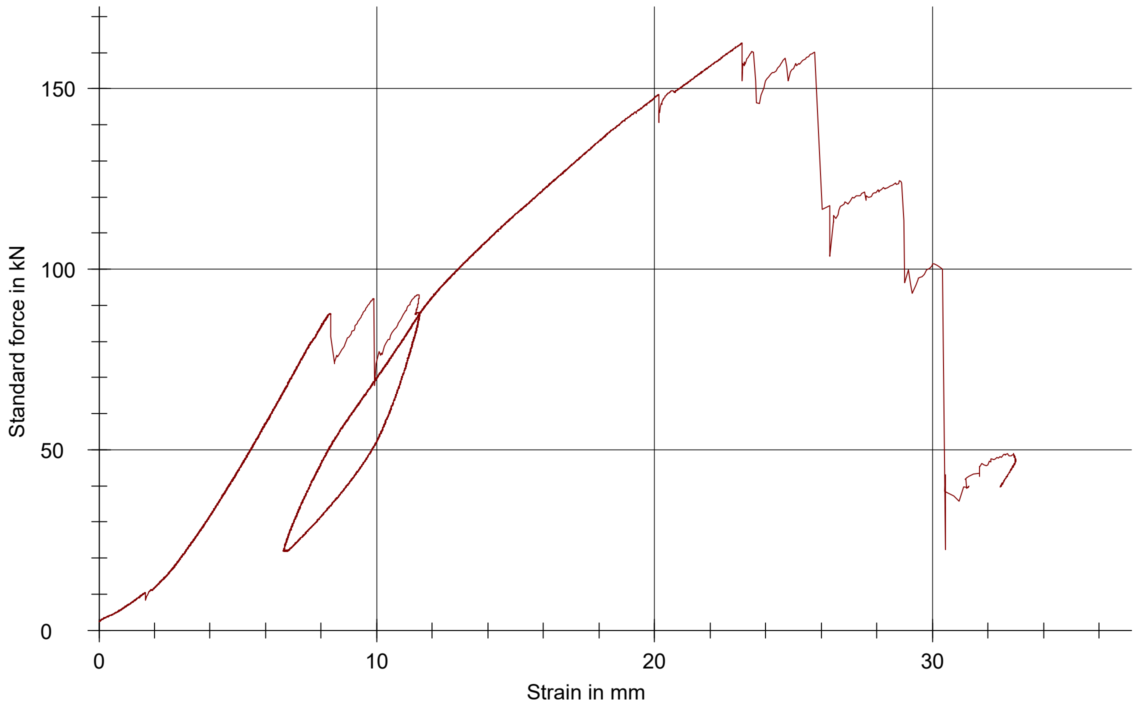
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
10.05.2023	Plate _D2	1600,0	600,0	200,0	162,59

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	162,59
s	-	-	-	-
ν	-	-	-	-

Parameter table:

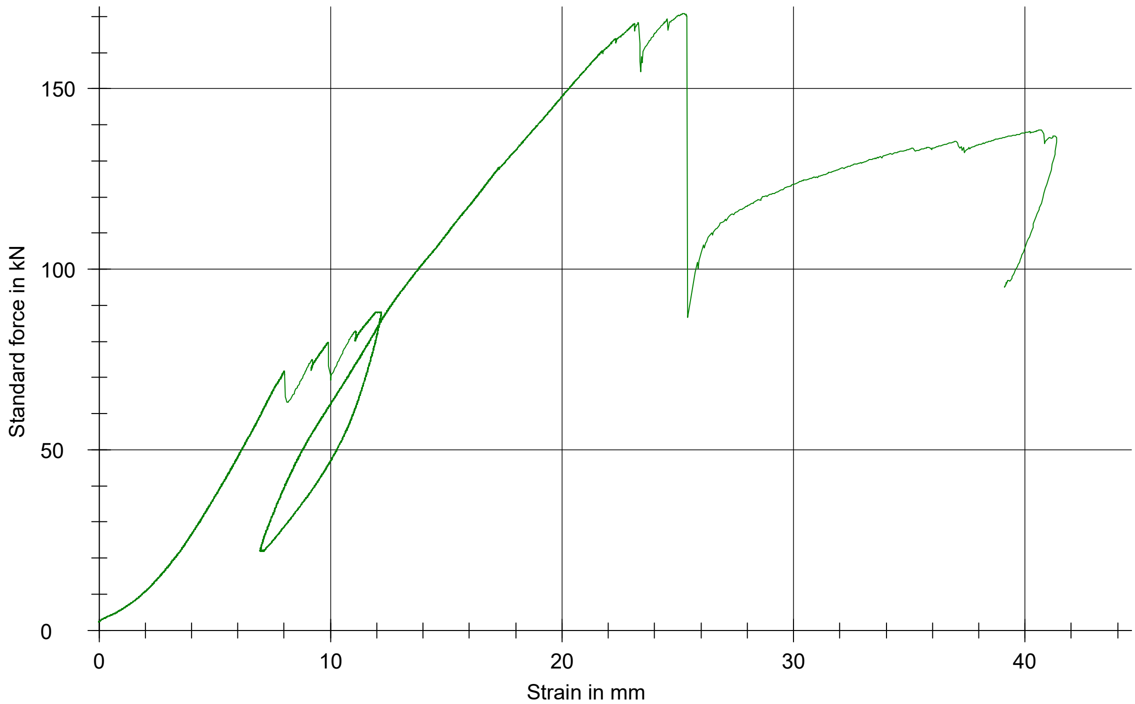
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
10.05.2023	Plate_D3	1600,0	600,0	200,0	170,83

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	170,83
s	-	-	-	-
ν	-	-	-	-

Parameter table:

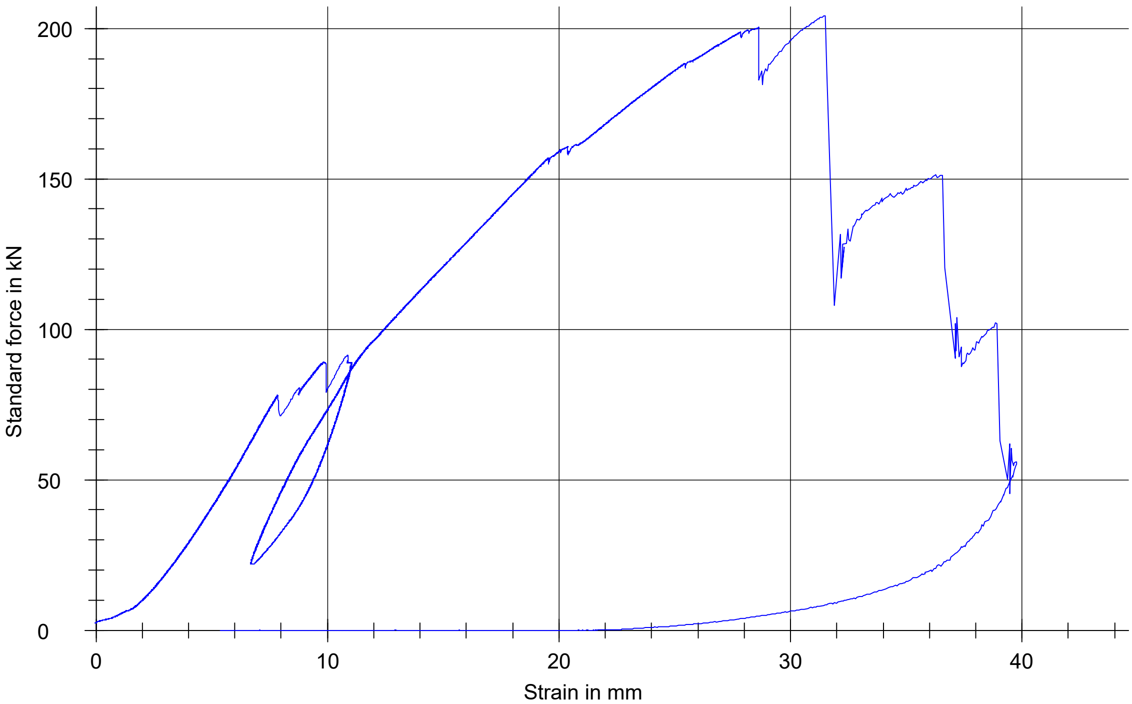
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
09.05.2023	Plate _E1	1600,0	600,0	200,0	204,34

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	204,34
s	-	-	-	-
ν	-	-	-	-

Parameter table:

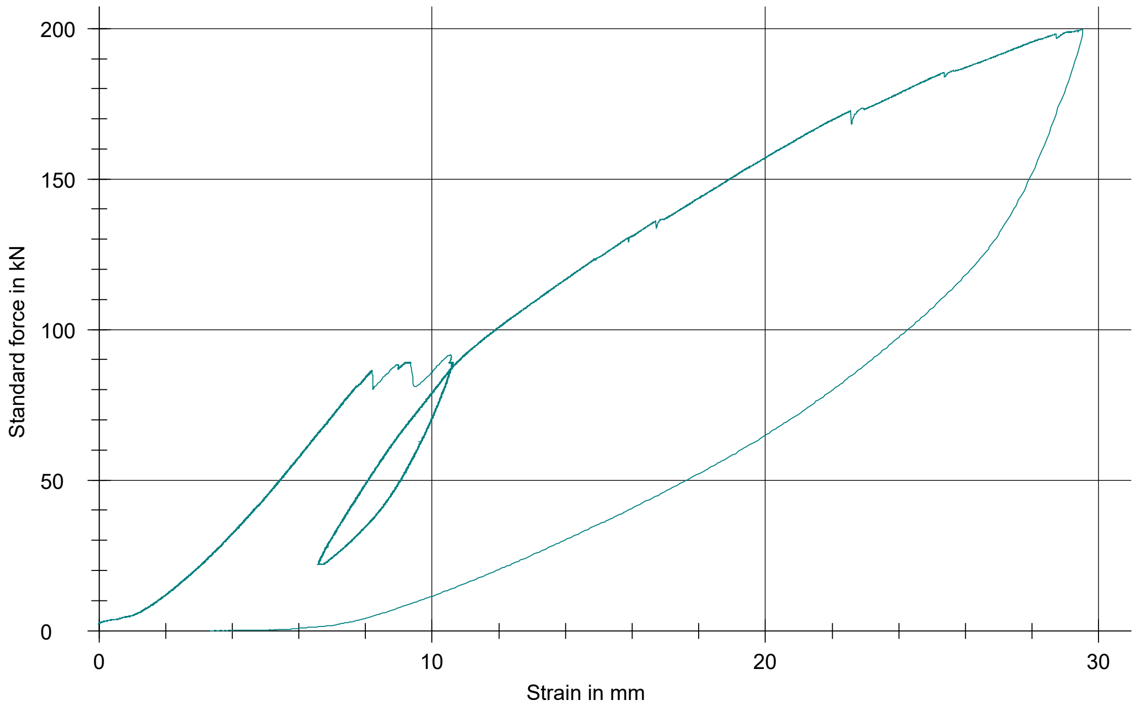
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
08.05.2023	Plate _E2	1600,0	600,0	200,0	199,76

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	199,76
s	-	-	-	-
ν	-	-	-	-

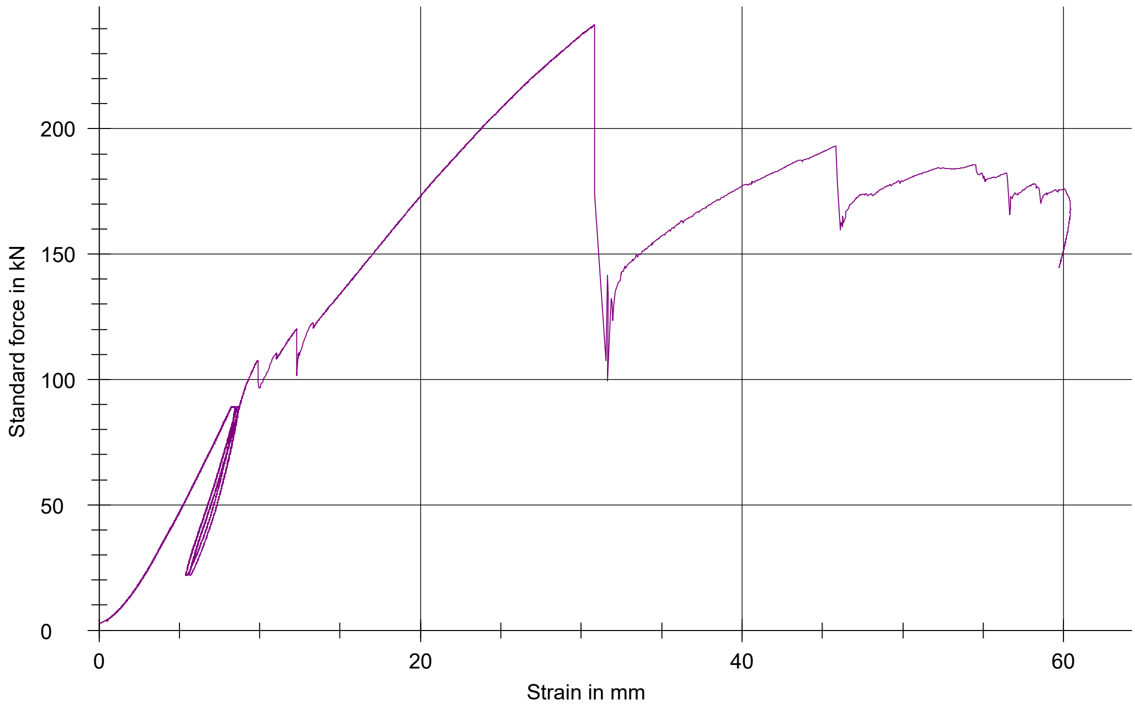
Parameter table:

Test protocol	: Antoni-Mohamaed	Type strain extensometer	:
Tester	: TCC testing	Machine data	: Controller TT0322 PistonStroke LoadCell 400 kN
Customer	: Bachelor oppgave V2023		
Creation date	: 27.04.2023		

Results:

Date	ID	a mm	b mm	h mm	F_m kN
04.05.2023	Plate _E3	1600,0	600,0	200,0	241,44

Series graphics:



Statistics:

Series n = 1	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	241,44
s	-	-	-	-
ν	-	-	-	-

Parameter table:

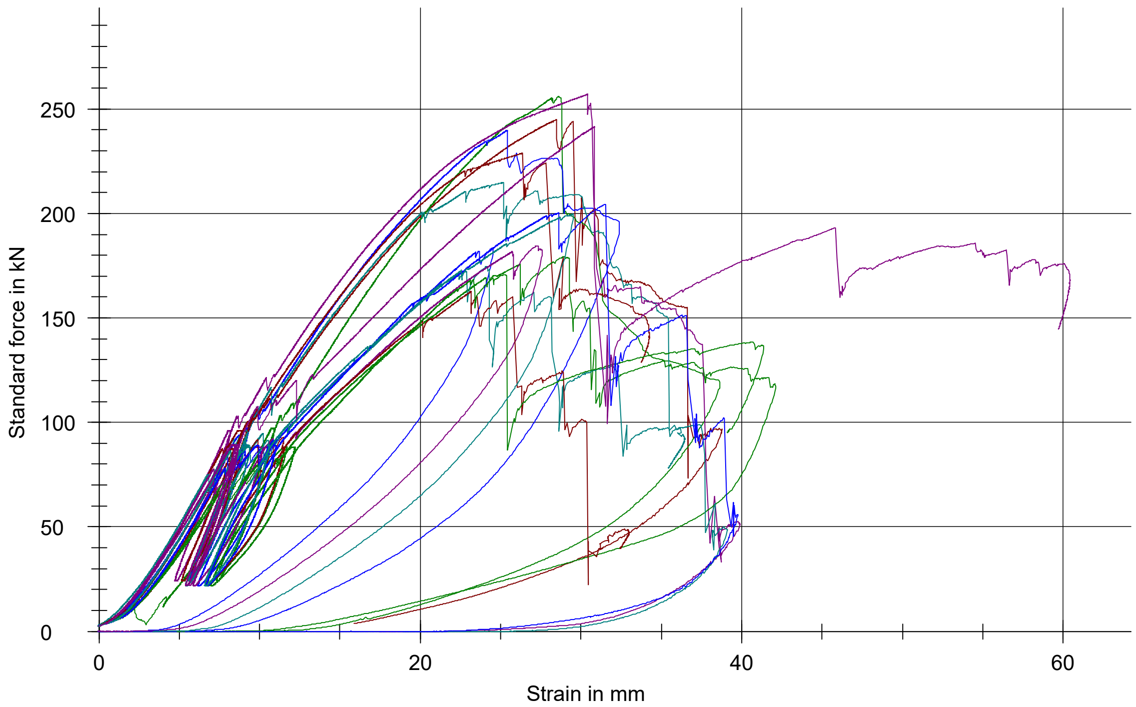
Test protocol : Antoni-Mohamaed
Tester : TCC testing
Customer : Bachelor oppgave V2023
Creation date : 27.04.2023

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN

Results:

Date	ID	a mm	b mm	h mm	F_m kN
12.05.2023	Plate _A1	1600,0	600,0	200,0	244,97
11.05.2023	Plate _A2	1600,0	600,0	200,0	255,94
11.05.2023	Plate _A3	1600,0	600,0	200,0	239,75
11.05.2023	Plate _B1	1600,0	600,0	200,0	214,97
11.05.2023	Plate _B2	1600,0	600,0	200,0	257,01
11.05.2023	Plate _B3	1600,0	600,0	200,0	228,83
11.05.2023	Plate _C1	1600,0	600,0	200,0	179,40
11.05.2023	Plate _C2	1600,0	600,0	200,0	183,30
10.05.2023	Plate _C3	1600,0	600,0	200,0	174,55
10.05.2023	Plate _D1	1600,0	600,0	200,0	184,55
10.05.2023	Plate _D2	1600,0	600,0	200,0	162,59
10.05.2023	Plate _D3	1600,0	600,0	200,0	170,83
09.05.2023	Plate _E1	1600,0	600,0	200,0	204,34
08.05.2023	Plate _E2	1600,0	600,0	200,0	199,76
04.05.2023	Plate _E3	1600,0	600,0	200,0	241,44

Series graphics:



Statistics:

Series n = 15	a mm	b mm	h mm	F_m kN
$\bar{x}$	1600,0	600,0	200,0	209,48
s	0,0	0,0	0,0	33,02
ν	0,00	0,00	0,00	15,76

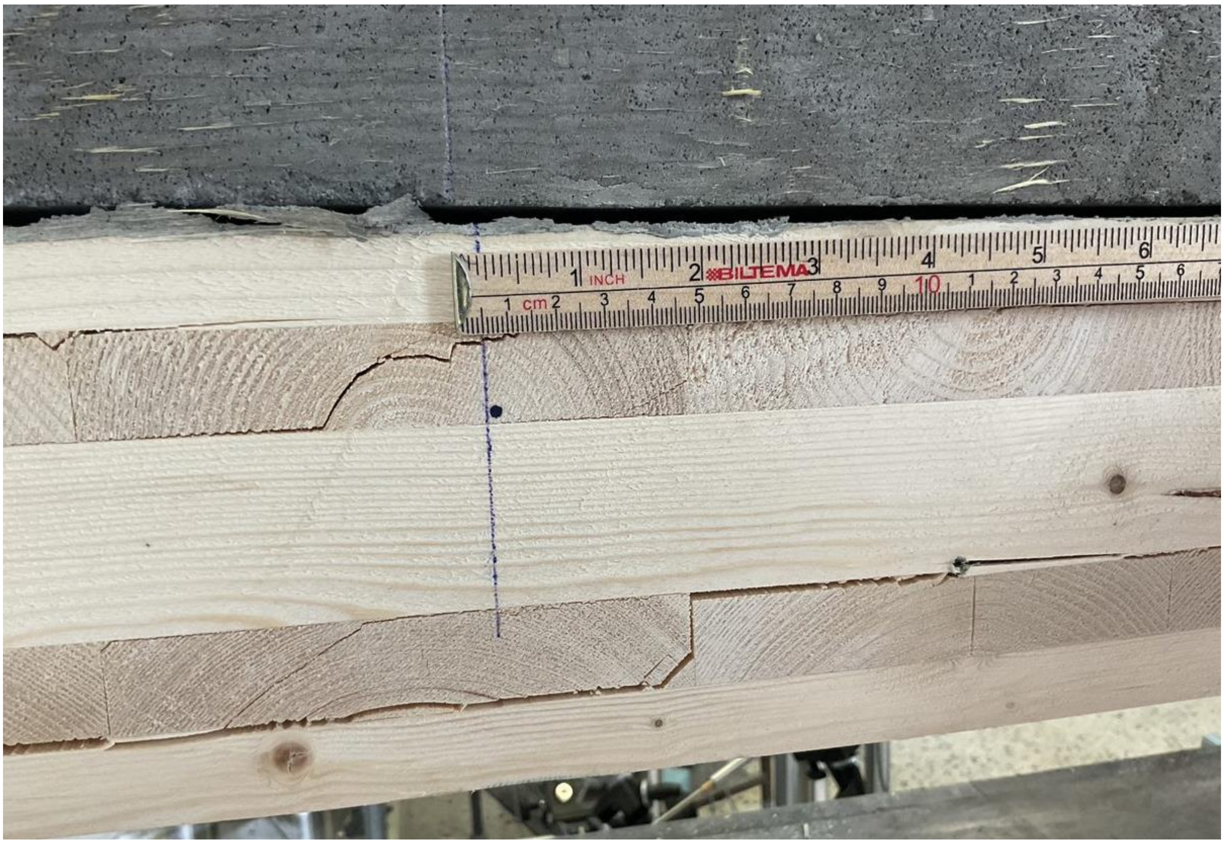
Appendix F.

Failures modes and pictures

Appendix F.1 Failure modes and pictures of type A

Specimen	Failure Modes
A1	• Rolling shear failure • Slip • Delamination, CLT layers • Small crack underneath
A2	• Rolling shear failure • Slip • Delamination, CLT layers • Crack concrete
A3	• Rolling shear failure • Crack concrete • Small delamination, CLT layers

Pictures of A1



Pictures of A2

Pictures of A3

Appendix F.2 Failure modes and pictures of type B

Specimen	Failure Modes
B1	• Rolling shear failure • Slip • Delamination, CLT layers • Finger joint failure • Crack underneath • Tensile failure and crack on timber • Crack concrete
B2	• Finger joint failure • Tensile failure and crack on timber • Slip • Delamination, CLT layers • Crack, concrete
B3	• Rolling shear failure • Slip • Delamination, CLT layers • Small failure underneath • Crack concrete

Pictures of B1

Pictures of B2


20230511_100


B2

Pictures of B3

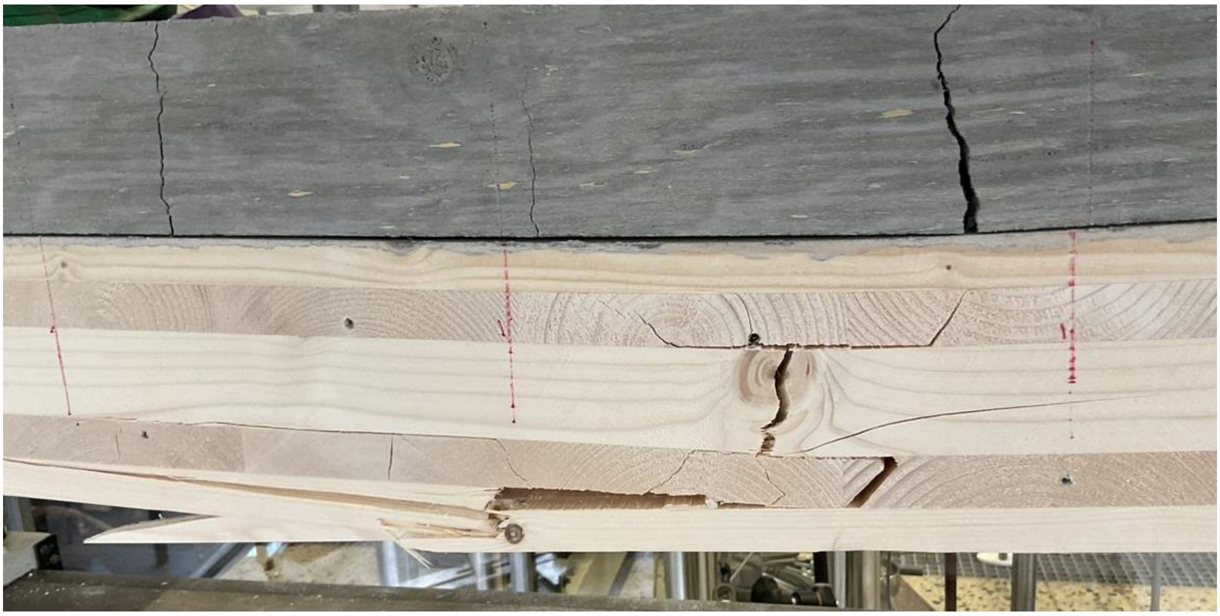
Appendix F.3 Failure modes and pictures of type C

Specimen	Failure Modes
C1	• Rolling shear failure • Slip • Delamination, CLT layers • Crack underneath • Finger joint failure underneath • Crack concrete
C2	• Small crushing failure • No other notable failure
C3	• Rolling shear failure both transverse and longitudinal CLT layers. • Knot failure • Tensile failure • Delamination CLT layers • Finger joint failure underneath • Crack concrete

Pictures of C1

Picture of C2

Pictures of C3



Appendix F.4 Failure modes and pictures of type D

Specimen	Failure Modes
D1	• Small crushing failure • No other notable failure on either concrete or timber
D2	• Rolling shear failure • Small Delamination • Finger joint failure underneath • Crushing failure
D3	• Crack concrete • Finger joint failure • Crack timber side • Crack timber underneath

Picture of D1

Pictures of D2

Pictures of D3

Appendix F.5 Failure modes and pictures of type E

Specimen	Failure Modes
E1	• Rolling shear failure • Finger joint failure, underneath • Tensile failure, bottom CLT layer • Delamination, CLT layers • Crack concrete
E2	• Small crushing failure • No other notable failure on either concrete or timber
E3	• Rolling shear failure • Finger joint failure, underneath • Finger joint failure, transverse layer • Delamination, CLT layers • Crack concrete

Pictures of E1

Pictures of E3