University of Stavanger

Faculty of Science and Technology

MASTER'S THESIS

Study program/Specialization: Master of Science in Engineering Structures and Materials/ Structural engineering	Spring semester, 2021 Open / ~~Restricted access~~
Writer: Maria Bergfjord	Maria Bergfjord (Writer's signature)

Faculty supervisor: Sudath C. Siriwardane (UiS)

External supervisor(s): Lars Henning Krokengen (COWI AS)

Laboratory supervisor: Andreas Skaare (UiS)

Thesis title:

Load capacity and structural response of CLT-concrete composite slabs with two different types of shear fasteners

Credits (ECTS): 30

Key words: Cross laminated timber Timber-concrete composite slab Shear fastener Laboratory testing Structural response Load capacity	Pages: 167 + enclosure: 634 Stavanger, 02.07.2021 Date/year

Frontpage for master thesis
Faculty of Science and Technology

Preface

This Master thesis is the last requirement to fulfil a degree in Master of Science in Engineering Structures and Materials, University of Stavanger, Faculty Science and Technology, Norway. The thesis covers 30 ECTS and was carried out spring semester 2021.

I would like to make a special thanks to my supervisor at the University of Stavanger, Professor Sudath C. Siriwardane and my supervisor in COWI AS, Lars Henning Krokengen for guidance and counsel throughout the duration of the thesis.

Also, I would also like to thank Jarle Berge, Samdar Kakay, Jan-Tore Jakobsen and Andreas Skaare for helping me with the materials, equipment, and recommendations for the laboratory experiments. In addition, I would like to thank Eugenio Facchini, technical sales consultant in Rothoblaas, for providing me with the necessary material and equipment for the laboratory experiment.

Further, I would like to extend my gratitude to my family and close friends for encouragement and support throughout the thesis during the period of time with the uncertainty of the Covid-19 pandemic.

Abstract

The subject of this Master thesis is to investigate the performance of the behaviour of Cross Laminated Timber-concrete composite slabs with two different types of shear fasteners by performing theoretical predictions and a laboratory testing, through a four-point bending test.

The type of shear fasteners is of high importance in structural design of a Timber-Concrete Composite (TCC) structure. The shear fasteners join the two elements together and the aim of the TCC is to work as one element, fully composite action. The TCC structure generally acts partially composite and this motivate to further examine the efficiency of different shear fasteners and their arrangement to achieve fully composite action of the TCC slabs. For this thesis two types of shear fasteners were examined: type A, CTC screws using crossed parallel and arranged in angles of 45^{o} and steel mesh reinforcement and type B using KOP screws oriented in pairs with a 45^{o} angle.

Theoretical predictions are performed using a combination of the γ- method and the shear analogy method to find the load capacity and maximum deflections for Cross Laminated Timber (CLT)-concrete composite slabs, both types A and B, are performed before conducting the laboratory tests. These theoretical predictions were then compared with the laboratory test results. Additional theoretical calculations were then performed, to check if it was possible to predict the failure load of the test more precisely. The degree of composite action was also examined by determining the efficiency of the shear fasteners was also examined. The efficiency can be found through the relationship of the theoretical calculations and measured deflection at midspan for a timber beam and concrete slab.

The result obtained from laboratory testing demonstrated that slabs of type A could withstand a much higher applied load than slabs of type B. These results showed that slabs of A had a much more conservative theoretical prediction, while that slabs of type B were very similar to the theoretical predicted result. Various of limitations might have had an influential factor on the results for both theoretical predictions and the laboratory testing. Hence, further studies should be performed to make a definite conclusion of the chosen shear fasteners.

Table of Content

List of Figures

List of Tables

List of Abbreviations

CFC	Continous Flexible Connection (CFC)
CLT	Cross Laminated Timber
GLT	Glued-Laminated Timber
HBV	Holz-Beton-Verbund
LSL	Laminated Strand Lumbar
LVL	Laminated Veneer Lumbar
MUF	Melamine urea formaldehyde
NLT	Nail Laminated Timber
NA	Neutral Axis
RCC	Reinforced Cement Concrete
PSL	Parallel Strand Lumbar
SCC	Self-Compacting Concrete
SLS	Serviceability Limit State
TCC	Timber-Concrete Composite
ULS	Ultimate Limit State

1 Introduction

1.1 Background

A concern when designing a structure today includes environmental impacts which is sometimes even included in the contract. Reinforced concrete is widely used building material, in recent years it has been found to be non-environmentally friendly material, e.g. due to limitations of raw material, recycling of materials, of CO_2 emission related issues, etc. to name a few.

Timber on the other hand, is a sustainable material which is also widely used in the building industry. The disadvantage with timber, is that until recently, it has not been widely used for multi-storey buildings due to the limitations, e.g.; its strength, vibration, resistance against extreme environmental actions, to mention some.

Cross Laminated Timber (CLT) is a relatively new concept, that is increasingly used in structural engineering and architectural applications [1, p. 55]. To date the current limitation of timber buildings comes to 5 stories, but in the recent years multi-storey buildings exceeding this limitation have been successfully constructed using CLT, e.g. the Forte building, a 10-storey apartment building in Melbourne, Australia [2, p. 127] or *Treet* in Bergen, Norway, a 14-storey building a constructed by a combination of glulam and cross laminated timber.

1.2 Problem statement of CLT-concrete composites

In general, a limited amount of research has been done on the use of CLT-concrete composite in comparison with other building materials and official standards and/or guidelines does not exist for CLT-concrete composite elements. However, some manufactures have developed their own guidelines/handbooks which can be used in structural design.

Eurocode 5, provide guidelines for the theoretical predictions for a 3-layered element, hence, the application of the given guidelines are restricted for elements exceeding three layers. Simplification and modifications to the existing formulas are performed to find the theoretical predictions for load capacity and maximum deflection. One simplified approach is to neglect all layers in transverse direction and another methodology modifies the cross section and considers the shear deformation of the transverse layers in addition to the longitudinal layers.

Timber is an anisotropic material, meaning that it has different strength properties depending on the grain direction and will influence the behaviour of the structure when load is applied [3, p.4]. Flaws and other defects can affect the load capacity and may cause premature failure of the structures.

Performance of shear fasteners the shear fastener, joining the two materials together is very important for composite structures. The aim for the composite structure is to act as one element, but usually it acts partially composite. Generalised guidelines to select the best shear fasteners are not available due to lack of research in this area. Performance of the different shear fasteners has not been sufficiently compared. These research gaps are the main motivation for this master's thesis research.

1.3 Objective

To overcome above research gaps to some extent, the main objective of this thesis is to investigate the load capacity and structural response of 5-layeres CLT-concrete composite slabs with two different types of shear fasteners by medium scale laboratory testing. The validity of the modifications to the existing formulas in the Eurocode is investigated for theoretical prediction of 5-layered CLT-concrete composite slabs as a secondary objective of this thesis.

1.4 Outline of the thesis

Chapter 2 is a literature review on TCC and mainly CLT-concrete composite slabs. In this chapter the main principle of composite action and the different elements forming a CLT-concrete composite slab is described. In addition, the theory which are related to performing the laboratory testing is presented.

Chapter 3 describes the theoretical background for the theoretical predictions, necessary for the verifications of the structural design of a TCC structure.

Chapter 4 discuss the theoretical predictions performed, with the aim of predicting the load capacity and maximum deflection before conducting the laboratory testing.

Chapter 5 and 6 shows the preparation of the two types of SLT-concrete composite slabs for laboratory testing.

Chapter 7 presents the results of the laboratory testing, and then compared with the theoretical predictions and discussed in chapter 8.

Chapter 9 states the conclusion made from both the theoretical predictions and the laboratory testing as well as suggestions for further studies.

2 Timber Concrete Composite

A composite material is a combination of two or more materials having different material and physical properties. The reason this combining different materials is to improve the physical behaviour, such as load capacity and strength. Two great examples of a composite material are reinforced concrete or steel girders with concrete slab. Furthermore, there are many other composite materials such as timber-timber composite and steel and timber composite, ceramic matrix composite and much more. The aim of combining two elements is that they behave as one element when load is applied, so they have a fully composite action. The meaning of composite action will be further discussed in Chapter 2.1. This thesis will focus on the Timber-Concrete Composite (TCC) structure and primarily Cross Laminated Timber (CLT)-concrete composite slab.

TCC structure is a composite structure where timber and concrete interact together. The timber and concrete are connected and act in a composite action, where the two materials work as a single unit, to improve the efficiency of the structure [4, p. 2]. This construction method started in USA in the 1930s and 1940s, where the earliest TCC short span bridges were built. After World War II, this method spread to New Zeland and Australia, and then in the 1990s it came to Europe [5, p. 54]. The knowledge, experience and understanding of the material's properties have improved by the years. Today, there are many successful completed multistorey buildings that is composed of TCC worldwide [1, p. 55].

Until now, most of the of the research worldwide have focused on TCC structures, where the timber is a beam or a column element with concrete slab. There has especially been a limited amount of research into the performance of the CLT-concrete composite floor system. Since CLT panels is known to provide good structural performance due to higher strength and stiffness than conventional timber, this should be a very attractive material [1, p. 2].

2.1 Main principle of Timber-Concrete Composite

For a TCC structure, the two elements; timber and concrete, are connected by mechanical fasteners. Normally, the concrete is placed on top of the timber part, which means that the concrete is in the compression zone and the timber located in the tension zone, as shown in Figure 2-1 below.

The advantages of TCC compared to separate slabs of timber or concrete are many: [6, p.17]

"Compared to only timber slabs the advantages are:

- *Increased stiffness*
- *Increased load carrying capacity*
- *Improved sound insulation*
- *Reduced sensitivity concerning vibrations*
- *Simplified possibility to realize the horizontal bracing of the structure"* [6, p.17]

"Compared to a pure concrete slab the advantages are the following:

- *Reduced dead load*
- *Increase of re-growing materials and therefore less CO_2 emissions*
- *Increase of prefabricated elements leading to a faster erection of the structure and therefore to a lower influence of the surrounding conditions during the erection phase*
- *Reduced volume of concrete, which leads to a faster building process and less volume to be transported on site*
- *Reduced effort for the props/formwork since the load carrying capacity and the stiffness of the timber cross section is higher than the related properties of the prefabricated concrete elements"* [6, p.17]

These advantages are only applicable for TCC slab that fulfil all requirements for the load carrying capacity and stiffness. The tensile strength in pure concrete section is often neglected as the reinforcement transfers the stresses caused by bending due to applied load. Generally, the 2/3 of the height of the concrete section subjects to bending induced cracks under ultimate loading as shown in Figure 2-1 [6, p.17].

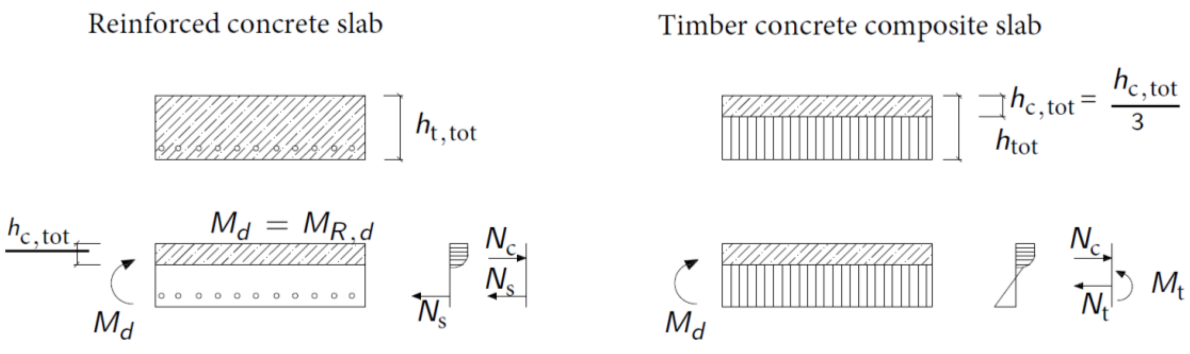


Figure 2-1 Load transfer (a) for a reinforced concrete slab, (b) for a TCC slab [6, p.18]

In practice there will be created some horizontal slip, movement at the interface since the mechanical fasteners are deformable. This phenomenon is described as the “partial composite action”, meaning the TCC is not behaving fully composite. As the load is continuously applied, the slip between the timber and concrete will continuously grow. This will affect the Neutral Axis (NA) were it goes from one single NA for both elements, to two separate NA for each component, moving further and further apart. Achieving a structure acting fully composite is a challenging, but some small slippage could be helpful for the system, because it can allow for redistribution of the shear stresses along the shear fasteners. [7, p.37]

The connection between the timber and the concrete is essential for a high degree of composite action in a TCC system. These connections transfer the shear between the two elements effectively, hence the choice of connections i.e. shear fasteners are very important for the performance of the slab. There are two bounds of the composite action of the TCC demonstrated in Figure 2-2 below. Figure 2-2 (c) shows a lower bound, a TCC slab with no composite action, where the timber and concrete works independently and there is no force transferred horizontally between the two elements through the shear fasteners. Figure 2-2 (a) shows the upper bound composite action, where it behaves fully composite and work as a single unit. This means that the TCC is rigidly connected and that there is no interlayer slip between the timber and concrete elements [7, p.37].

Composite action is often ranged from 0 to 1, where 0 means no composite action, 1 means fully composite action, and anything between means the structure behaves partially composite.

The Figure 2-2 below demonstrates the action for a TCC for a timber beam and concrete slab [7, p. 38].

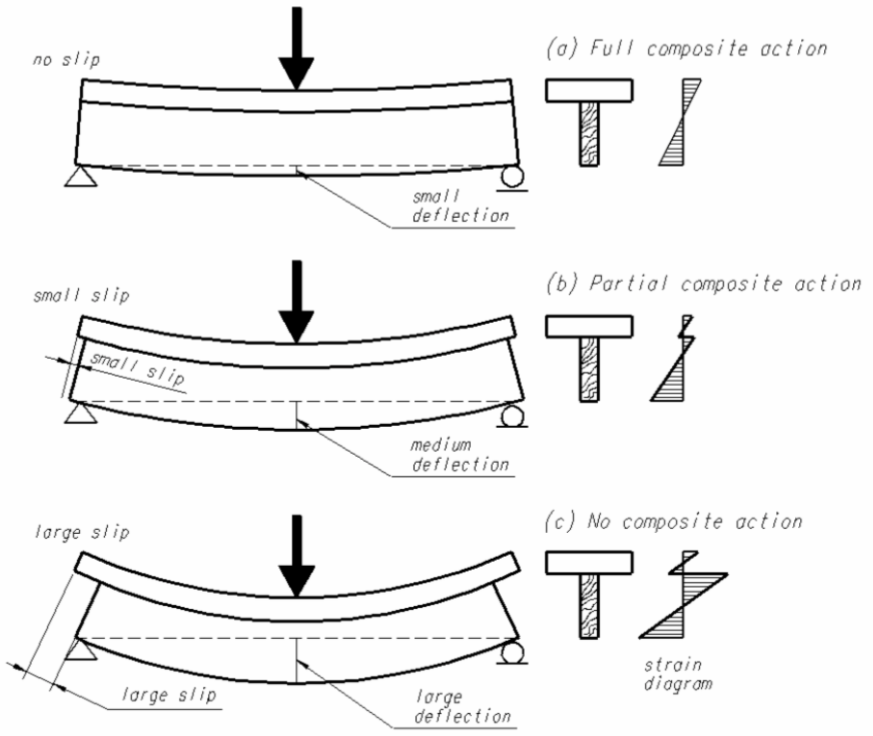


Figure 2-2 The concept of composite action (a) Fully composite action, (b) Partial composite action and (c) No composite action between the timber beam and concrete slab [7, p.38]

The choice of the shear fastener is of importance to determine the high composite action of the TCC structure for a timber beam and concrete slab. The efficiency of the shear fastener can be found through the relationship of the theoretical predictions and measured deflection at midspan for a timber beam and concrete slab, see equation 2.1 [7, p. 38]. Efficiency can vary between 0%, no composite action and 100%, fully composite action.

$$Efficiency = \frac{D_N - D_I}{D_N - D_C} \quad 2.1$$

Where:

D_N: Theoretical deflection for the corresponding TCC with **no** composite action

D_C: Theoretical deflection for the corresponding TCC with **full** composite action

D_I: The actual measured deflection of the TCC from laboratory testing

Another method to determine the efficiency of the interlayer connection is through the relationship of the bending stiffness from theoretical predictions and actual value from laboratory testing and is shown in the following equation 2.2: [7, p. 38]

$$\gamma = \frac{EI_{Real} - EI_0}{EI_\infty - EI_0} \qquad 2.2$$

Where:

γ: The efficiency of the interlayer connection in the TCC beam

EI_0: Theoretical bending stiffness of the TCC beam with **no** composite action

EI_∞: Theoretical bending stiffness of the TCC beam with **full** composite action

EI_{Real}: The actual measured bending stiffness of the TCC beam from laboratory testing

When $\gamma \rightarrow 1$ indicates that the actual measured bending stiffness is very stiff and moves toward the theoretical bending stiffness for fully composite action. Similarly, when $\gamma \rightarrow 0$ indicates that the bending stiffness is very flexible and moves theoretical bending stiffness for no composite action. Equation 2.2 has been obtained through many different research's using different types of material and geometrical conditions and is representative for a wide number of practical applications [8, p. 29].

The Figure 2-3 below demonstrates the correlation between the slip of the shear fastener and the effective bending stiffness. When the degree of composite action increases, the slip between the timber and concrete will also increase. The correlation does not increase linearly and will have a more asymptotical curve for the maximum and minimum values of the slip. Hence, after a given value of slip (K), the effective bending stiffness will be affected to a small degree [8, p. 29].

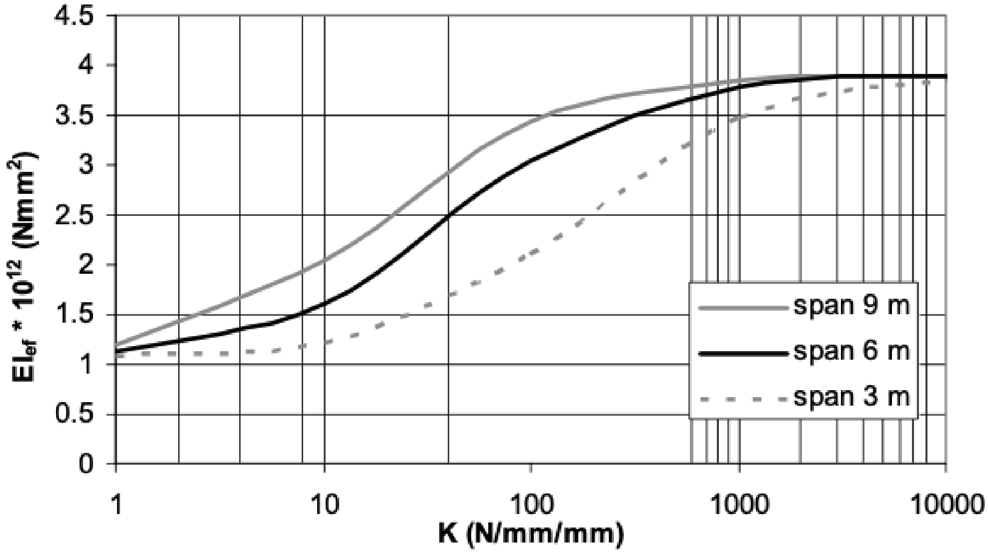


Figure 2-3 A graphical representation of the correlation of a shear fastener and the effective bending stiffness of a composite floor [8, p.29]

2.2 Types of systems

There are several different types of TCC systems which is dependent on what type of material and usage of the designed structure.

The type of material is referring to the properties of timber and concrete, as well as the shear fastener to be used. A previous survey described in the [9, p. 31-33] describes the currently used TCC structure solutions. These system names often come from the chosen type of shear fastener.

2.2.1 The Holz-Beton-Verbund system

This system comes from the type of shear fastener Holz-Beton-Verbund (HBV), which is a type of net steel plate where half is inserted into the timber in the longitudinal direction and half into the concrete part, see Figure 2-4 where the plate is coloured blue.

In addition to the HBV shear fastener, the concrete is reinforced with a mesh reinforcement where the HBV will function as support. This type of shear fastener is suitable for engineered wood beam spaced at the centres, as well as solid timber panels [10, p. 93], [9, p. 31].

The dimensions for the HBV net steel plate with thickness of 2 mm and height of 90 mm,105 mm or 120 mm, and length is usually 1 m. The material properties for the HBV system are provided by the suppliers.

An advantage using this shear fastener, is that this type is less dependent on the properties of the timber and the bending stiffness. A disadvantage is the quality control when gluing the shear fastener into the wood due to poor quality in the bonding, it will affect the behaviour of the whole system [11, p. 34].

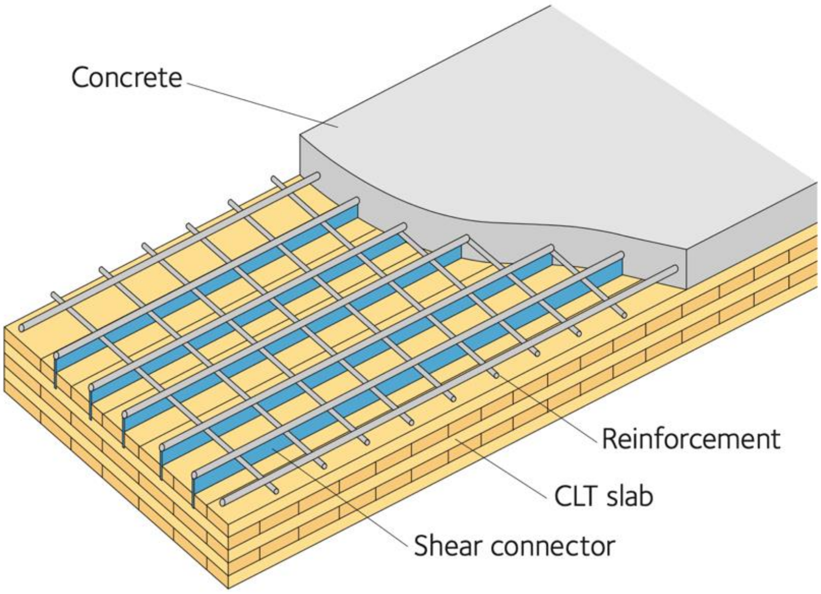


Figure 2-4 Composite floor structure in CLT and concrete, with shear fastener of type HBV [10, p. 93]

2.2.2 M-section system

This system is composed of timber beams, a plywood interlayer, and a reinforced concrete layer. Plywood interlayer will work as formwork for the concrete that will be casted in-situ. The timber used is Laminated Veneer Lumber (LVL) with the dimension; width of 63 mm and the height 400 mm. Spacing between the LVL beams are 1200 mm. The width of each section is 2400 mm, with a double LVL beam side by side in the middle part and single beam on the outermost sides. For larger area, these sections shown in Figure 2-5 are then joined together by connecting two outermost single LVL beams.

This M-section system can have a total span length of up to 10 m. A 17 mm plywood is placed on top of the LVL beams, where notched screw shear fasteners are placed along the length of the LVL beam. E.g., for a 10-meter span, six to eight notched screw fasteners are needed. Steel reinforcement and thereafter concrete is placed in-situ on top. The reinforcement will contribute with control of the shrinkage of the concrete slab [9, p. 31-32].

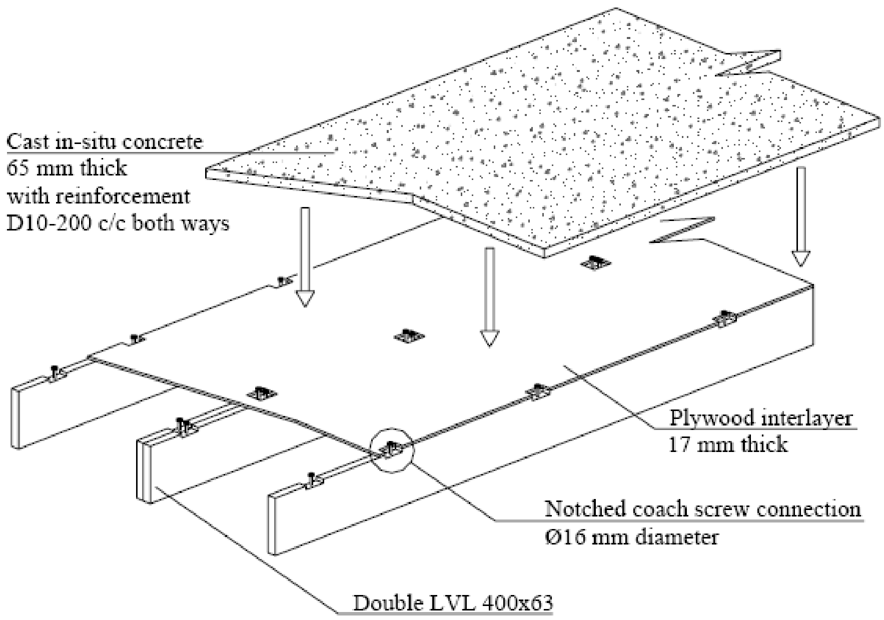


Figure 2-5 M-section TCC system [9, p. 32]

2.2.3 Tecnaria system

The Tecnaria system is another system joining timber beam and concrete slabs, forming a composite structure. Each fastener, Figure 2-6, is made up by a 40 mm long steel stud with a diameter of 12 mm, that is welded onto a steel square plate, with sides of 50 mm and thickness 4 mm. There are 2 holes where two screws with dimensions 10 mm diameter and 120 mm length is connecting the stud into the timber with a regular or variable spacing along the length [7, p. 5], [8, p. 13], [6, p. 67-68].

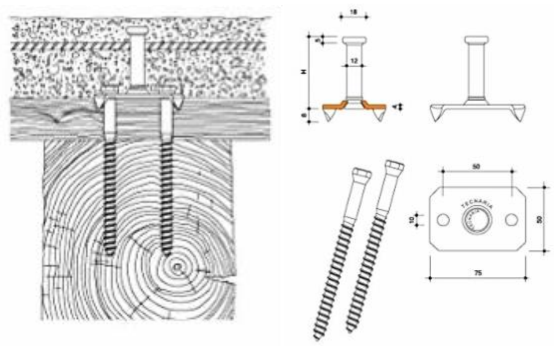


Figure 2-6 Tecnaria system (a) Cross sectional cut (b) detail drawings of the Tecnaria fastener [8, p. 13]

2.2.4 The SEPA-2000 system solution

This system is suitable for both in-situ and pre-casted concrete, and there is no need for formwork when the concrete is placed upside-down. In this system, timber truss is connected to concrete by a nail-plate-connector, see Figure 2-7.

SEPA-2000 system allow for spans up to 8 m and continuous spans having three or more supports, satisfying the conditions of strength, stiffness, vibration, and load capacity [7, p. 24], [9, p. 33].

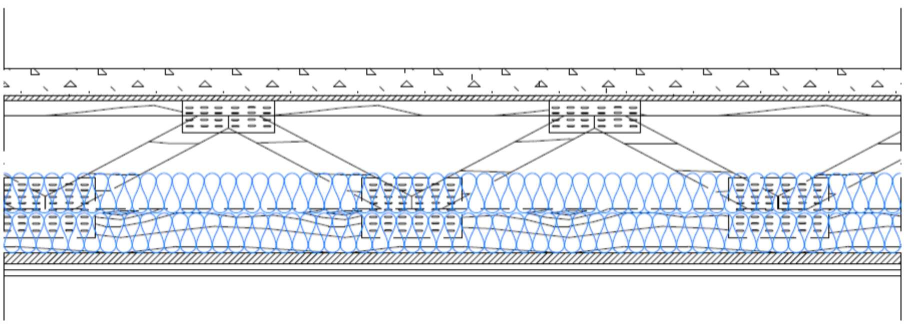


Figure 2-7 The SEPA-2000 system, side view [9, p. 33]

2.3 Timber

Timber is an anisotropic material, which means that it has different strength properties depending on the grain direction [3, p. 4]. The direction parallel to the grain fibres, the longitudinal direction, is considerably stronger than the direction that is perpendicular to the grain fibres and will affect the structure when load is applied, causing compressive, tensile, and bending stress. The strength of the timber depends partly on the density and how the grains are consistent in the direction. Fibre deviations, such as knots, can have a huge impact on how well the timber structure can handle the applied load. In addition, humidity, temperature and duration of the applied load will have an impact. E.g., if the timber is wet, it will be weaker than dry, also colder temperature make it stronger [5, p. 12].

The types of wood often used in timber beams are Glued-Laminated Timber (GLT), Parallel Strand Lumbar (PSL) and Laminated Veneer Lumbar (LVL). Types of timber, often used in mass timber panels are Nail Laminated Timber (NLT), Cross Laminated Timber (CLT) and Laminated Strand Lumbar (LSL) [12].

2.3.1 Mass Timber

In the recent years, Mass Timber has become a popular material when designing buildings or bridges. Typically, it is spruce tree that is used, connected with glue, nails, or wooden dowel. The material is high in strength and can be used in walls, floors, and roof in tall buildings. Some advantages using timber, is reduction in CO_2 emission, lower weight, aesthetic aspects [2, p. 127].

Mass Timber consists of interconnected layered lamellae or planks that are connected using glue, nails, or tree plugs. There are three different ways of assembling the lamellae; these are described below and shown in Figure 2-8.

a. *“Bordstabelelement/Kantstilte elementer”*- Vertical oriented, connecting the planks together side by side on the largest cross-sectional area surface joined together by screws, nails, glue, wooden dowels, or steel plugs
b. *“Flersjiktselement/Krysslaminert tre”* - Cross Laminated Timber, composed of planks layered on top 90 or 45 degrees in relation to each layer, usually connected using glue or wooden dowels
c. *“Hulromselement”* - Hollow Core Elements, with a variety of designs

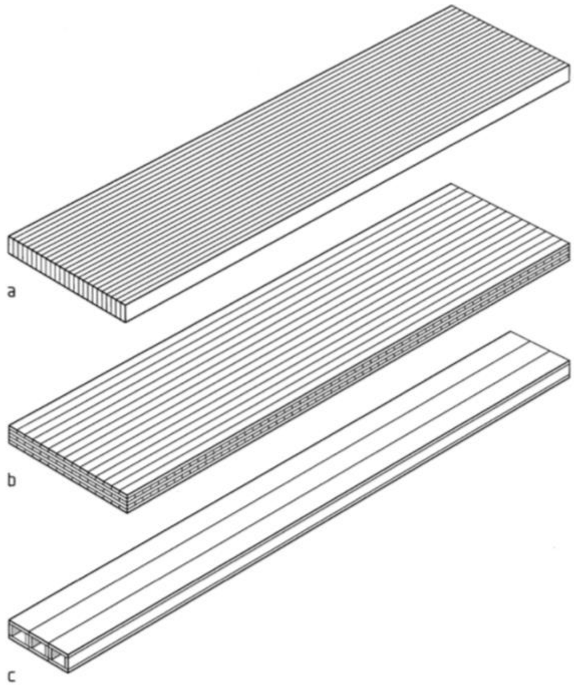


Figure 2-8 Types of Mass Timber [13]

CLT timber alone used in flooring, cannot exceed span over 7-7.5 m in length due to limitations in static and dynamic stiffness. Using CLT together with another material, e.g., concrete as a composite material, limitations can be improved. Use of CLT-concrete composite material is beneficial in terms of reduction of CO_2 emission, indoor and outdoor climate. It can also be recycled and reused [14, p. 6-7].

2.4 Concrete

Concrete is a composite material that is composed of cement paste, coarse and fine aggregate, sand, admixtures, and water. Where fine and coarse aggregate makes up to 70% of the total volume and the cement paste nearly makes 30% of the volume [15, Ch. 1, p. 3,]. The different quantity of the different materials will influence the quality and behaviour of the finished concrete. The supplier shall mix the concrete in accordance with NS-EN 206-1. In Norway, most common type of concrete (65%-70%) is of the durability class M60, meaning water/binder ratio shall be less than 0.60, which often corresponds to the strength class B25 [15, Ch. 1, p. 8,].

There are different types of concrete depending on the density and composition:

- Lightweight concrete has a density less than 2000 kg/m^3.
- Plain/ordinary concrete is commonly known as normal mix design, has a density of 22000-2500 kg/m^3.
- Heavy concrete has a density of larger than 2600 kg/m^3 [15, Ch. 1, p. 22,25].

Other types of concrete are Reinforced Cement Concrete (RCC) and Self Compacting Concrete (SCC), to mention some. Reinforcement includes steel in various forms, rebar, mesh, etc. increase the tensile strength, and make this type of concrete beneficial. SCC is a relatively new, where fresh concrete is composed of two components, matrix phase and particle phase. The motivation for developing SCC is to improve working conditions, increase quality and productivity. There is a change in the technique of concrete placement; no need for vibration using pokers, due to that compaction will done by the gravity itself because of the mix design [15, Ch. 1, p. 26].

The concrete can be casted on-site, or it can be precasted at the supplier's factory. For a TCC slab, on-site casting is preferable because this will create a natural diaphragm with the need of additional topping or post-fix connection between the slabs. Using on-site casting method reduces the weight of the prefabricated timber components and is more efficient in terms of transport [12].

"To get a successful casting it is important to follow these following points:

- *Casting technique*
- *Concrete workability*
- *Adequate formwork quality*
- *Placing of reinforcement and concrete cover thickness*
- *Curing conditons"* [15, Ch. 1, p. 44].

2.5 Survey of types on concrete experimented for TCC

A survey conducted by [9, p. 34], have proposed enhancement methods of different concrete types for TCC. The concrete type is determined by the timber and shear fastener used. For some types of shear fasteners the RCC is used, other plain concrete or SCC.

2.5.1 Lightweight concrete

Use of Lightweight concrete method, the overall weight of the TCC floor can be reduced further. From experimental testing, performed by E. Steinberg, R. Selle and T. Faust, replacing plain concrete with lightweight reduces the overall dead weight with approximately 15%. Though, this type of concrete has a lower capacity, increases the failure in the concrete and often have higher likelihood to split due to concentrated load on the shear fasteners [9, p. 34].

2.5.2 Steel-fibre-reinforced concrete

Compared to plain concrete, the steel-fibre-reinforced concrete is more ductile and can better redistribute the stresses. Consequently, this type of concrete will resist flexural action more than plain concrete. If a crack has been initiated, redistribution of stresses can avoid brittle failure to occur [9, p. 34].

Hence, using this type of concrete it is possible to reduce the height of concrete or increase spacing of the timber beam panels. Through experiments conducted by M. Tanjik, P. Dobrila and M. Premrov, increase in shear capacity together with the initial slip modulus of the shear fasteners have been proven [9, p. 34].

2.5.3 Carbon-strip-reinforcement

M. Tanjik, P. Dobrila and M. Premrov, have also examined the usage of applying a carbon strip at the bottom part of the timber component with dowels as shear fasteners. In this experiment, they found that the timber was the crucial component for the load-bearing capacity where the carbon strip behaved as tensile reinforcement. Adding carbon strip made it possible to increase the bending stiffness, both moment and shear capacity without changing the dimensions of the timber component [9, p. 34].

2.6 The shear fasteners

The connection type is the most essential component in a TCC structure. It determines the degree of composite action and performance of the composite structure. For a high degree of composite action, it is important that the shear between the timber and concrete is transferred effectively [7, p. 37]. The choice of shear fastener affects both the stress distribution and the displacement of the composite structure when load is applied.

"The ideal behaviour of the shear fasteners from the mechanical performance point of view is:

i. *strong enough to transmit the shear forces developed at the interface,*

ii. *stiff enough to transmit the load with a limited slip at the interface,*

iii. *ductile enough to allow full load distribution and avoid failure on the fasteners.* [6, p. 33]

In addition, there are some other factors to consider when deciding the shear fasteners such as cost, complexity and feasibility in practice [6, p. 33].

To date, the types of shear fasteners available does fulfil only parts of the above mentioned ideal mechanical behaviour. Since it is not possible to neglect the connection slip between the timber and the concrete, this must be considered for the overall analysis of a TCC structure.
For a simple model the transformed section method is not satisfactory. Hence, the type of shear fastener is of importance and has a significant effect on the overall behaviour of both the composite structure described earlier in this chapter [6, p. 33].

A research conducted by the Civil Engineering Department of the University of Coimbra, Portugal, is the basis of the statistic study of different types of connections and give a good indication of what type of shear fasteners to use in practice [6, p. 33-34].

Figure 2-9 and Figure 2-10 below, shows the distribution of the different type of shear fasteners used in research and the distribution of types within the dowel type shear fastener.

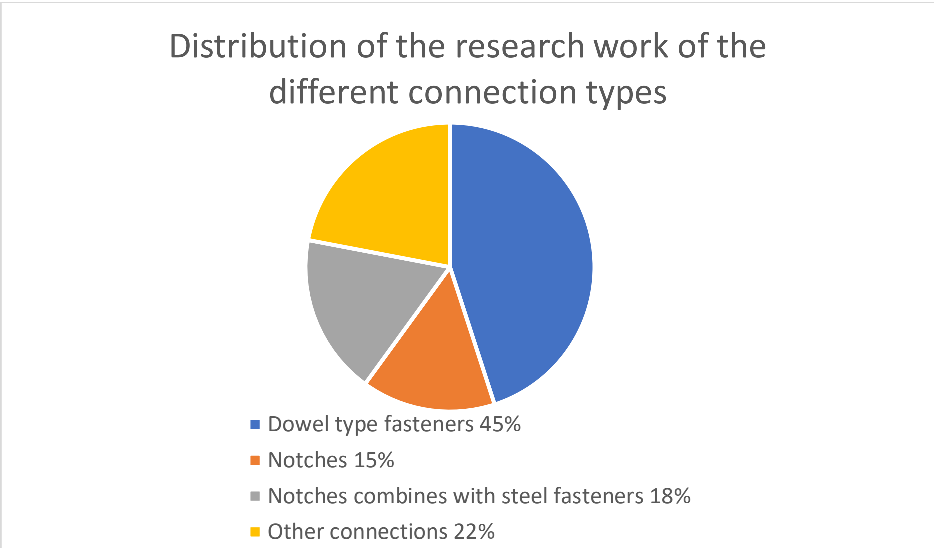


Figure 2-9 Distribution of the research work of the different connection types [6, p. 34]

Figure 2-9 above, shows that the dowel type shear fastener is the most used type, with 45% of the research cases. Notches (15%) and notches combined with steel fasteners (22%), are approximately 33% of the cases. This makes ¾ of all research cases, but do not necessarily show the distribution of the shear fastener used in practice. However, this can give an indication of what shear fastener is chosen [6, p. 33-34].

Out of the 45% dowel type of shear fasteners the Figure 2-10 below, demonstrates how this is distributed within the group of dowel type shear fasteners. Screws are most frequently type used shear fasteners within the group of dowel fasteners. Only 4% is recorded to be inclined screws.

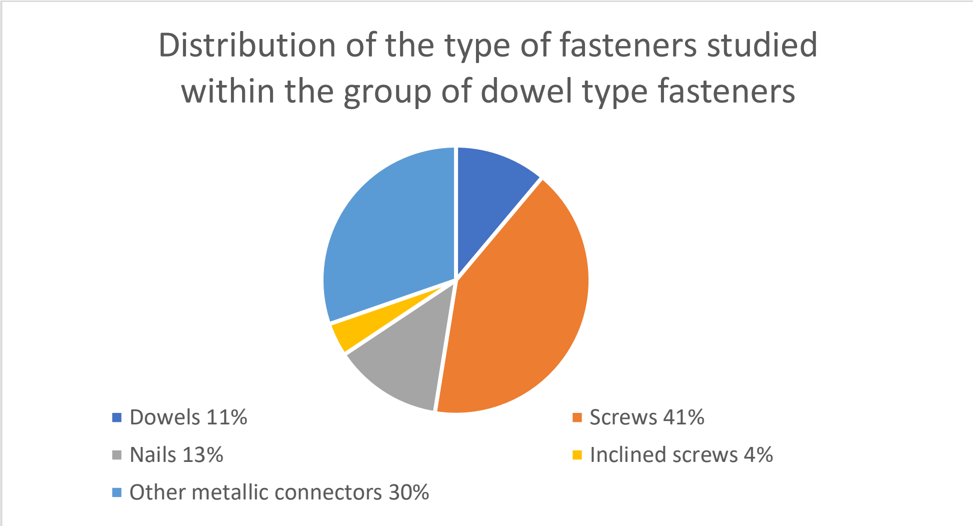


Figure 2-10 Distribution of the type of fasteners studied within the group of dowel type fasteners [6, p. 35]

[8, p. 16] conducted a laboratory testing of different types of shear fasteners and the result is shown in Figure 2-11. This figure shows that there is a huge difference in the mechanical behaviour of the different types of shear fastener. The glued and notched joints have a very high strength and stiffness, while the nail plates and dowel type fastener have a lower strength and stiffness, but have a much higher plastic deformation capacity [8, p. 16].

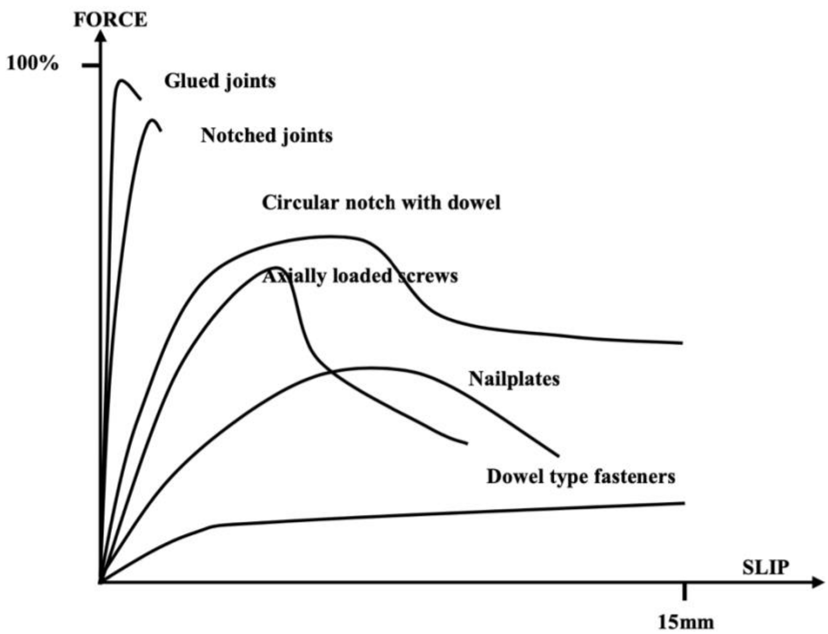


Figure 2-11 Typical load-slip behaviour for different type of joints [8, p. 16]

2.7 Theory for laboratory experiments

The performance of the different laboratory experiments follows different standards and is described in the following chapters.

2.7.1 Theory for the concrete verification

2.7.1.1 Compressive test

The compressive test is tested in accordance with NS-EN 206:2013, [16]. The load rate is set to 0.5 KN per second and is gradually increased until failure. The compressive stress is than found by equation 2.3 below.

$$\sigma = \frac{F}{A}\ [MPa] \qquad 2.3$$

Where:

F: The applied Load [N]

A : The cross-sectional area [mm^2]

2.7.1.2 Modulus of elasticity test

The modulus of elasticity test was tested in accordance with NS-EN 12390-13:2013 [17].

The purpose of this test is to find the modulus of elasticity by having three cycles with loading and unloading in accordance with the referred standard. Results received from the compressive strength tests, is used when finding the values for the preload step, lower step and upper step. These values changed with the different days of testing. Each test group, 28, 38 and 40 days after curing, dependent on the average result of the compressive test of the three cubes. The formulas below are from [17]:

Average compressive stress f_c, measured and converted from cube to cylinder.

$$f_c = \sigma_{average} \cdot 0.8 \qquad 2.4$$

Then Upper stress, σ_a, is found:

$$\sigma_a = \frac{f_c}{3} \qquad 2.5$$

Lower stress, σ_b, is found:

$$0.1 \cdot f_c < \sigma_b < 0.15 \cdot f_c \qquad 2.6$$

Preload stress,σ_p, is found:

$$0.5\ MPa < \sigma_p < \sigma_b \qquad 2.7$$

2.7.1.3 Splitting tensile test

The splitting tensile test was conducted after the modulus of elasticity test and the cylinders were placed correctly into the device. It is important to apply the load with a continuously rate according to the standard NS-EN 12390-6:2001[18], and not with any shock rate loading. The test stops automatically when the test specimen is split into two halves. After finding the breaking load, the calculations of splitting tensile strength can be found using the equation below.

$$f_{ct} = \frac{2 \cdot F}{\pi \cdot L \cdot d} \qquad 2.8$$

Where:

F: Maximum load applied [N]

L : Length of specimen [mm]

d : Diameter of the cylinder specimen [mm]

2.7.2 Four-point bending test

There are no specific standards for conducting laboratory testing for a TCC element, instead the laboratory testing is performed in accordance with NS-ISO 6891:1991 “Timber structures, Joints made with mechanical fasteners” [19].

The load procedure was performed in accordance with NS-ISO 6891:1991 [19], where the load rate is applied with one cycle and then continuously increasing until the ultimate failure. The estimated failure load, F_{est}, is obtained from the theoretical predictions using the γ- method together with the shear analogy method to predict the ultimate failure load for the CLT-concrete composite slab. The load rate is found by assuming reaching failure takes 10 minutes and where the estimated failure load is divided on 10 minutes.

$$Load\ rate = \frac{F_{est}}{10\ minutes} \quad 2.9$$

The Figure 2-12 below describes this loading process. It starts by applying the load rate until the $0.4 \cdot F_{est}$ is reached, point 04 in the figure below. Then the load is maintained for 30 seconds. After the load of $0.4 \cdot F_{est}$ is applied constant for 30 seconds, its starts to unload until reaching $0.1 \cdot F_{est}$, point 11. This load is then held constant for another 30 seconds. Thereafter, from point 21, load is applied with the constant load rate until reaching the ultimate failure load.

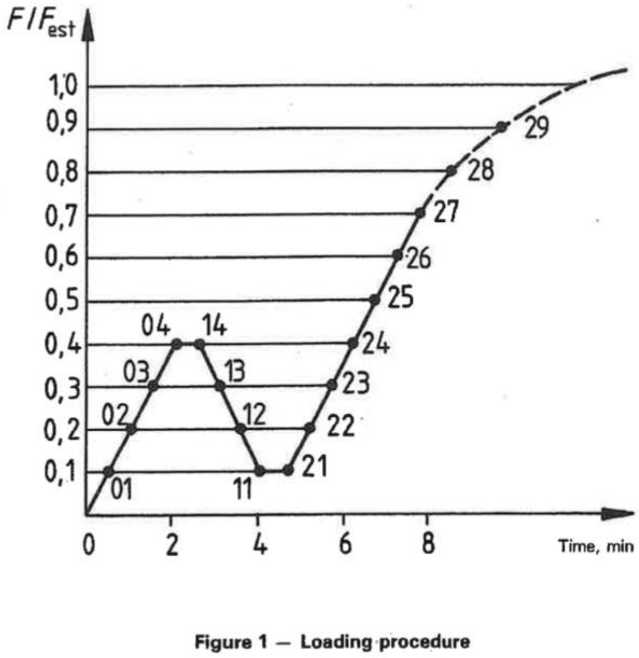


Figure 2-12 Loading procedure [19, clause 8.4]

3 Analysis of TCC elements

The design of structure shall be in accordance with "Eurocode 0, Basis of structural design" [20], in such a manner that the structure will sustain all actions and influences that is likely to occur during its design life. The structure must be designed to have an acceptable structural resistance, serviceability, and durability. Providing sufficient information related to the safety of a structure, verification of the two categories of limit state design; Ultimate Limit State (ULS) and Serviceability Limit State (SLS) must be achieved. In addition, it is essential to verify the short-term and long-term effect of the composite slab [20, clause 2.1, 3.4, 3.5].

3.1 Verification of the cross section

In addition to Eurocode 0 [20], there have been adaptions for both timber and concrete structures. The verification of concrete structure is verified in accordance with Eurocode 2 NS-EN 1992-1-1:2004 [21] and similarly, timber structure is verified in accordance with Eurocode 5 NS-EN 1995-1-1:2004+A1:2008+NA2010 [22]. Since there are no official standards for a TCC design with two separate parts having different material properties, this is discussed in the following chapters.

3.1.1 Ultimate Limit State

The safety of people and the safety of structural design should be classified as ULS. If relevant, verification of loss of equilibrium, failure due to excessive deformation or failure due to fatigue should be included [20].

3.1.1.1 Normal stresses of the concrete cross section

For both the top and bottom part of the concrete cross section is verified as follows:

Verification of the compressive stress of the top part of the concrete section: [21]

$$\frac{\sigma_{c,t}}{f_{cd}} \leq 1.0 \qquad (3.1)$$

Where:

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \quad 3.2$$

Verification of the tensile stress of the bottom part of the concrete section:

$$\frac{\sigma_{c,b}}{f_{ctd}} \leq 1.0 \quad 3.3$$

Where:

$$f_{ctd} = \frac{f_{ctk,o.05}}{\gamma_c} \quad 3.4$$

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

$\sigma_{c,b}$: Normal stress of the bottom part of 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

3.1.1.2 Normal stresses of the timber cross section

For the verification of the timber cross section, it is assumed to be subjected to stresses from combined bending and axial tension and is verified as follows: [22]

$$\frac{\sigma_2}{f_{t,0,2}} + \frac{\sigma_{m,2}}{f_{m,d}} \leq 1.0 \quad 3.5$$

Where:

$$f_{m,d} = \frac{k_{mod} f_{m,k}}{\gamma_M} \quad 3.6$$

$$f_{t,0,d} = \frac{k_{mod} f_{t,0,k}}{\gamma_M} \tag{3.7}$$

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

3.1.1.3 Shear stress of the timber cross section

Verification of the shear stress of the timber cross section:

$$\frac{\tau_{2,max}}{f_{v,d}} \leq 1.0 \tag{3.8}$$

Where:

$$f_{v,d} = \frac{k_{mod} f_{v,k}}{\gamma_M} \tag{3.9}$$

$\tau_{2,max}$: The maximum design shear of the timber

$f_{v,d}$: Design shear strength of timber

$f_{v,k}$: Characteristic shear strength of timber

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

γ_M: Partial factor of material properties

3.1.1.4 Verification of the connection between timber and concrete

Verification of the shear fastener, connecting the timber and concrete:

$$\frac{F_1}{F_{R,d}} \leq 1.0 \tag{3.10}$$

Where:

F_1: The acting load per fastener

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

3.1.2 Serviceability Limit State

The SLS takes the concerns related to function of the structure, comfort of people and the appearance of the structure into the consideration in design [20].

The composite system is verified for both timber and concrete as follows:

$$\frac{w}{L/250} \leq 1.0 \qquad 3.11$$

Where the L is the length and w is the deflection of the composite structure.

When performing structural analysis, it is important to use the appropriate design models including all relevant variables. The design model should be precise enough to predict structural behaviour and ensured it is possible to build it. The impact of deformation in the connections should be considered through their stiffness or from the prescribed slip values. It is important that the load-carrying-capacity of the shear fastener is verified when the forces and the moments between the members are determined [22, clause 5.3].

For both the ULS and SLS verification calculations, these two limit states are affected by forces that acts on the structure, the material properties from the concrete, timber and shear fasteners. For the ULS, when the stiffness distribution in the structure affects the distribution of the member forces and bending moments and for the SLS, when the structure with components with varying time-dependent properties, causing new final mean values for the modulus of elasticity, the shear modulus, and the slip modulus [22, clause 2.3.2.2].

The TCC structure must satisfy the ULS and the SLS for both short-term and long-term. From the formulas in [22, clause 2.3.2.2] a general effect, for stress and displacement, designated E^F, caused by the ULS and the SLS can be expressed as follows:

For ULS

$$E^{Fu} = E^{Fu}(E_{cm}(t_0), E_{0,mean}, k_u) \quad 3.12$$

For SLS

$$E^{Fs} = E^{Fs}(E_{cm}(t_0), E_{0,mean}, k_{ser}) \quad 3.13$$

Where:

$$k_{ser} \neq k_u$$

Where these equations are dependent on the load applied on the structure, the modulus of elasticity and the slip moduli of the components.

The design load combinations for ULS considers only one load combination, but the load combination for SLS considers three different load combinations; one for characteristic, the other for frequent and the third load combination consider quasi-permanent. These equations 3.14-3.17 are as follows: [7, p. 43-44, 161]

For ULS

$$F_{d,u} = \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,j} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad 3.14$$

For SLS

Characteristic:

$$F_{d,r} = \sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} \qquad 3.15$$

Frequent:

$$F_{d,f} = \sum_{j\geq 1} G_{k,j} + \psi_{11} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \qquad 3.16$$

Quasi-permanent:

$$F_{d,p} = \sum_{j\geq 1} G_{k,j} + \sum_{i\geq 1} \psi_{2,i} \cdot Q_{k,i} \qquad 3.17$$

Where:

G: Permanent action

Q: Variable action

γ : Partial factor (of safety or serviceability)

ψ: Factor for variable action (combination, frequent or quasi permanent)

[7, p.43-44, 161]

3.2 Short-term verification

The short-term verification, at the early state, when the load is applied instantaneously and with no creep effect when performing analysis on the stresses and loads that affects the composite cross section. For TCC, there are some different theoretical predictions used, γ-method, shear analogy Continuous Flexible Connection to mention some. For this thesis, only γ-method together with the shear analogy method will be discussed [7, p. 44].

When performing the short-term verification procedures, they are based on the modulus of elasticity for concrete and timber and including the slip-modulus of the shear fastener. The load-slip relation of the shear fastener is usually non-linear and is therefore considered by the design purpose. The slip-modulus is different for the ULS and the SLS, k_u and k_{ser} respectively. The value of the slip-modulus is dependent of which standard used. If experimental data are available from a push-out test, it is possible to choose the values for k_u and k_{ser} with accordance to [19]. For SLS, the slip-modulus k_{ser}, the value taken is the secant value at 40% ($k_{0.4}$) of the load-carrying capacity of the shear fastener from the push-out test. Similarly, for ULS, the k_u value recommended to use is the secant value at 60% ($k_{0.6}$).

See Figure 3-1 below, from [7, p. 45] presenting the load-slip relation values from experimental data. If there is not any experimental data available, it is recommended to use the formula for timber-to-timber connections in [22]. The value for the SLS is recommended to be double of the value for the slip-modulus k_{ser} [22, clause 7.1(3)] For the slip modulus for the ULS verification, 2/3 of k_{ser} [22, eq. 2.1] should be used [7, p. 44].

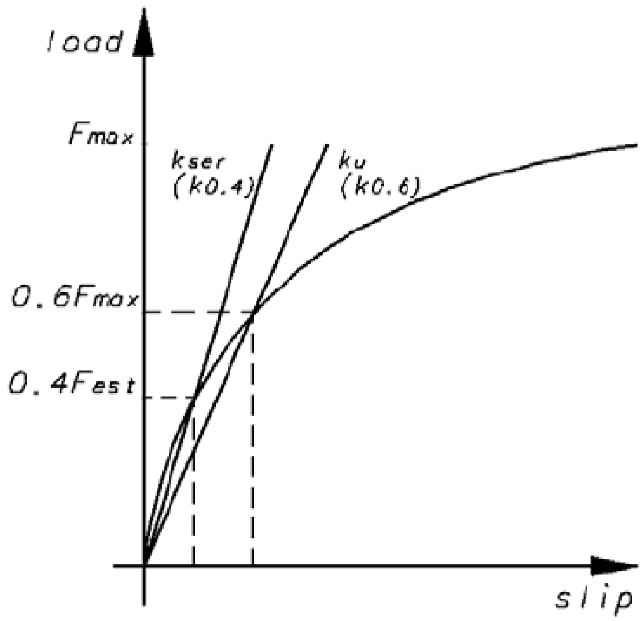


Figure 3-1 k_u and k_{ser} [7, p. 45]

The procedure for the short-term verifications for ULS and SLS can be summarised in a general approach, in terms of stress and vertical deflection as follows: [9, p. 10]

ULS – in terms of stress

$$\sigma_{inst} = \sigma^{F_{d,u}}(E_{cm}, E_{0,mean}, k_u \qquad 3.18$$

Where $F_{d,u}$ is the load combination for ULS. [9, p. 10]

SLS – in terms of vertical deflection

$$u_{inst} = u^{F_{d,r}}(E_{cm}, E_{0,mean}, k_u \qquad 3.19$$

Where $F_{d,r}$ is the characteristic load combination for SLS. [9, p. 10]

3.3 Long-term verification

It is more challenging verifying TCC for long-term verification due to effect caused by the applied load, will not necessarily show at once, but over time it can start to appear. It is very important that the system satisfies the long-term requirements in addition to the short-term requirements. The most challenging effects that may occur, are creep and shrinkage of the concrete and shrinkage or swelling of the materials, which influences the internal forces and cause deformation [6, p. 106-107].

Creep of the material is time dependent. When there is applied a load on a composite structure, it will start to deform. When this deformation increases over time and contributes to a larger deformation, is called creep [6, p. 106-107].

The creep deformation in a TCC structure can influence the deformation and internal stresses and forces. Because of the creep deformation at least one of the components of the structure; timber, concrete, or shear fastener, will increase the deformation further. Usually, this type of structure is designed with spans longer than five meters. With these ranges of distances used; the verification of the deformation is pivotal. The creep deformation can also influence internal stresses and forces. The creep strain can be understood as the reduction of the stiffness. The stiffness is of great importance when it comes to the distribution of loads in a statically undermined system. Therefore, the creep strain can adjust the distribution of loads within the system. This variation of the loads will increase the larger difference between the creep coefficients of a single component. The equilibrium of the forces will result in that the component with less creep, will receive higher loads. So, if creeping is stronger in one component than the other, the stronger creeping component will reduce its load. Furthermore, the creep strain will affect the normal force. Finally, the bending moment of the less creeping component will be increased whereas the normal force will decrease. Conclusively, the increase will occur in the component with less creep [6, p. 106-107].

Another important effect is shrinkage and/or swelling of the material. Volumes often changes if materials are hardened by means of a chemical reaction or interact with the surrounding by absorbing or emitting moisture. In the first circumstance, the volume is reduced, and the cross section shrinks due to the reaction product embeds the elements into a new order. In the second circumstance the volume increases, because the water is enclosed into the material. The volume will decrease, and the cross section be reduced if moisture is discharged. The change in temperature in a composite system, will also have a significant effect. The different material will have different thermal expansion coefficients, meaning they will react differently to different temperatures. The effect of shrinkage and swelling is a direct effect of this temperature change [6, p. 106-107]. The critical change in temperature, is if the temperature of the composite structure changes drastically in temperature during the hardening process. The inelastic strain can be a result of this temperature change. When the strain does not depend on the stresses, it is called inelastic strains. Figure 3-2, from [6, p. 75, 106-107] demonstrates how this inelastic strain is behaves due to the temperature change.

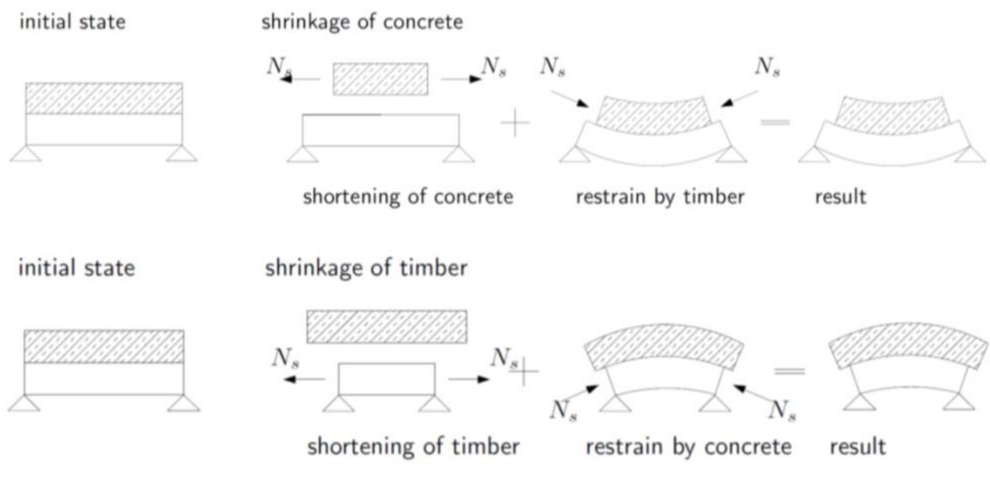


Figure 3-2 The effect of the inelastic strain on a timber-concrete composite [6, p. 75]

Figure 3-2 also demonstrates how the concrete shrinks and get a shorter length due to the change in temperature. This will then cause an increase in deflection. Increase in deflection of the concrete part, will then affect the timber part, and result in larger deformation. This will have a similar affect as applied load and therefore this deformation due to shrinkage cannot be neglected. Additionally, the effect of shrinkage may influence the internal forces. If the shrinkage occurs in the concrete part, there will be a change in the internal forces. This change will then cause a larger bending moment, an increase in stresses in the timber part, because the material will soften. Shrinkage caused by this change in stresses and eigenstresses should be considered in the structural design [6, p. 75, 106-107].

To date, there are no official standards and/or guidelines to follow when designing TCC for long-term behaviour. There are rules and regulations for the different materials behaviour individually, found in the Eurocode 5 [22] for timber and the Eurocode 2 [21] for concrete, but not for a structure consisting of two elements that behaves as one composite. These limit state calculations consider the different; material properties, time-dependent properties, and the design situations. There is just one limitation with this, and that even if the creep is considered, the change in climate and various design situation is not considered [23, p. 45].

For bridges, the Finish Transport Agency has made a guideline for the application of using the Eurocode 5, creating a rough estimation to consider the effect of creep for the long-term and short-term effect. The creep is considered by putting the modulus of elasticity of the concrete as $E_c/2$ for the long-term loading and $E_c/3$ for the short-term load duration [23, p. 45].

Absence of standards and/or guidelines explicit for TCC, the effective modulus method is recommended used by Eurocode 2 for concrete and Eurocode 5 for timber. The estimation of this long-term elastic moduli for timber and concrete uses the creep factor from previous load-duration studies. This long-term effect considers two different factors, the first one is the because of the quasi-permanent load and the other because of the difference between the short-term design load and the quasi-permanent load [9, p. 9].

The overall general effect is acquired as a sum of the effective moduli and the mean values of the modulus of elasticity. The effective moduli for timber, concrete and shear fastener are found using the equations 3.20-3.22 below [9, p. 9].

Concrete:

$$E_{c,fin} = \frac{E_{cm}(t_0)}{1 + \phi(t, t_0)} \qquad 3.20$$

Timber:

$$E_{t,fin} = \frac{E_{0,mean}}{1 + k_{def,t}} \qquad 3.21$$

Shear fastener:

$$k_{fin} = \frac{k}{1 + k_{def,t}} \qquad 3.22$$

Where:

- $E_{cm}(t_0)$: Is the Mean value of the modulus of elasticity for concrete member in compression at the time of loading t_0
- $\phi(t, t_0)$: Creep coefficient at a time t given the initial loading time t_0
- $E_{0,mean}$: Is the Mean value of the modulus for timber member in tension at the time of loading t_0
- $k_{def,t}$: Creep coefficient for timber/shear fastener
- k : The slip modulus of the shear fastener [9, p. 9]

From the Eurocode 5 [22] and Eurocode 2 [21], the recommended values are given for the different type of material. The equations 3.23 and 3.24 summarise the long-term effect.

ULS – in terms of stress

$$\sigma_{fin} = \sigma^{F_{d,p}}(E_{c,fin}, E_{t,fin}, k_{ser,fin}) + \sigma^{F_u - F_{d,p}}(E_{cm}(t), E_{0,mean}, k_u) \qquad 3.23$$

Where $F_{d,u}$ is the load combination for ULS [9, p. 10].

SLS – in terms of vertical deflection

$$u_{fin} = u^{F_{d,p}}(E_{c,fin}, E_{t,fin}, k_{ser,fin}) + u^{F_u - F_{d,p}}(E_{cm}(t), E_{0,mean}, k_{ser}) \qquad 3.24$$

Where $F_{d,r}$ is the characteristic load combination [9, p. 10].

3.4 Theoretical approaches for Timber-Concrete Composites

As mentioned previously, there is no well-defined method when it comes to theoretical predictions for TCC. In Europe there has been different methods, some are theoretical, and some are analytical methods, adapting the mechanical properties for the CLT element. The limitations with the experimental evaluation method, is that there are various of parameters that will change from project to project. This makes each project less general and more expensive. With the analytical approach, using the already known properties when predicting the strength and stiffness of the composite structure with CLT element, the analysis is more generalised and will cost less than the experimental approach [24, Ch. 3, p. 8].

The γ-method from the Eurocode 5 [22] is a common simplified approach. The shear analogy method from the American CLT handbook [24] considers the shear deformation in the transverse layers of the timber element [24, Ch. 3, p. 8]. The Continuous Flexible Connection (CFC) method assumes that the behaviour of the composite element are partial composite, were the beams have semi-rigid shear fasteners. This method is for partly composed beam made of two separate elements joined together by mechanical connections [23, p.17, 25].

Further in this chapter, the γ-method and shear analogy method will be discussed, hence in chapter 4 a combination of these two is used.

3.4.1 γ −method

The γ- method, also known as *Mechanically jointed beams* theory, is an analytical approach that has been adapted for the TCC structures. This approach is presented in Eurocode 5, Annex B of [22, Annex B]. The limitation with this approach is that it is only applicable for TCC consisting of two or maximum three layers. This method considers the effective bending stiffness of the composite structure depending greatly on the degree of composite action and the shear fasteners are assumed uniformly distributed along the length [2, p. 132-133].

The γ- method is a simplified approach for determining the maximum stresses and the maximum deflection [22, Annex B]. The method is used for mechanically jointed beams and is also suitable for slab/panels.

Then in Annex B [22, Annex B], Figure 3-3 shows which cross section this method is applicable for [22, Figure B.1, in Annex B]

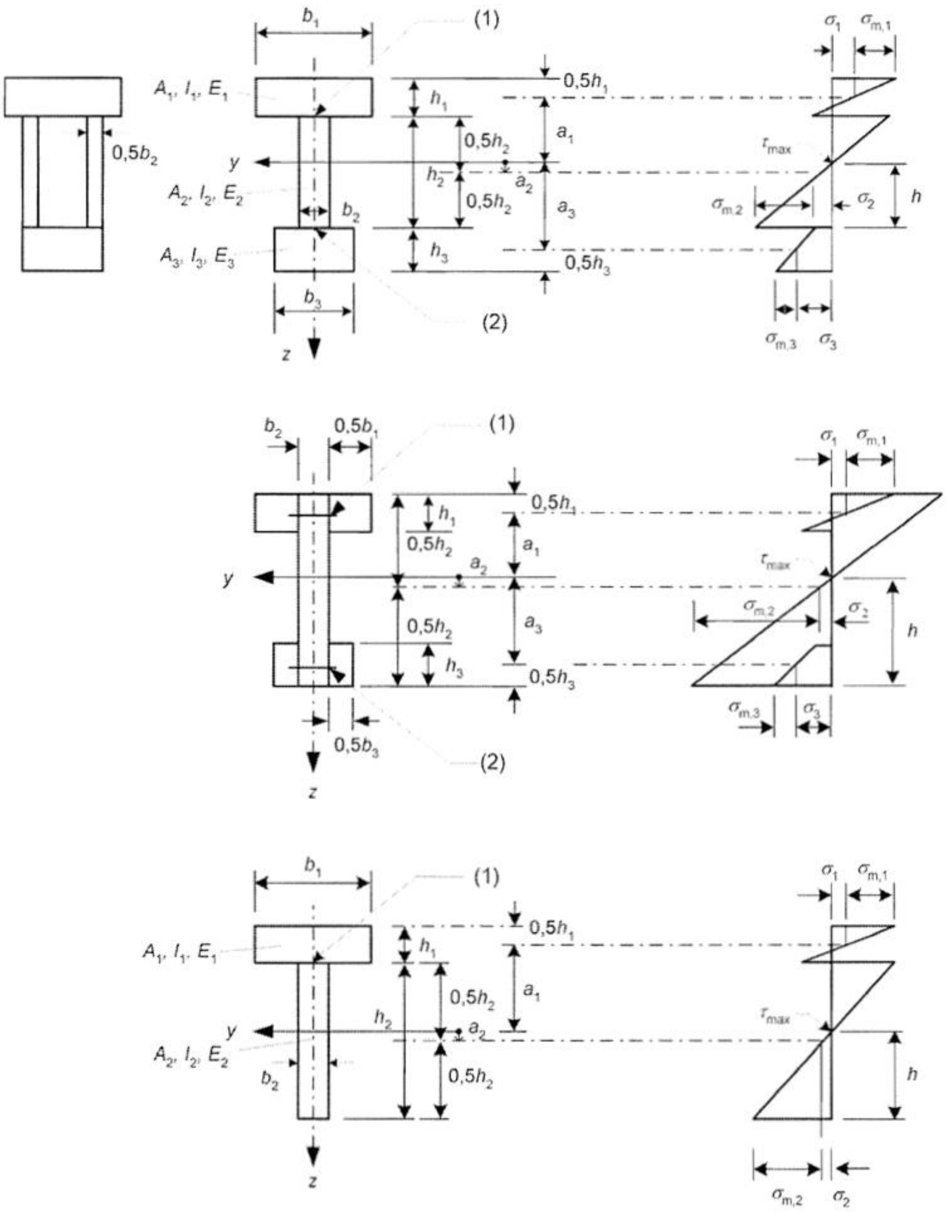


Figure 3-3 From Figure B.1 in Annex B, Cross section (left) and distribution of bending stresses (right) [22, Figure B.1]

“By using this method some assumptions must be followed.

- *The beams are simply supported with a span l. for continuous beams the expression may be used with l equal to 0,8 of the relevant span and for cantilevered beams with l equal to twice the cantilever length*
- *The individual parts (of wood, wood-based panels) are either full length or made with glued end joints*
- *The individual parts are connected to each other by mechanical fasteners with 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}, *with* $s_{max} \leq 4s_{min}$
- *The load is acting in the z – direction giving a moment M = M(x) varying sinusoidally or parabolically and shear force V(x)”* [22, clause B.1.2]

Depending on the geometry of the composite section illustrated in Figure 3-3 above, one or two flanges joined together by a web or a box beam, the spacing is determined by the sum of fasteners per unit length in the two joining elements [22, clause B.1.3]. The deflection calculations according to [22, clause B.2] uses an effective bending stiffness $(EI)_{ef}$ [22, clause B.1.3].

In equations 3.25-3.30 below $(EI)_{ef}$ is the effective bending stiffness of the entire composite section. I_i is the moment of inertia of layer “i”. E_i is the modulus of elasticity of layer “i”, while A_i is the cross-sectional area of layer “i” and a_i is the distance from the neutral axis to the centre of the layer “i”. The K_i is the stiffness of one single shear fastener and it is dependent on whether it is ULS or SLS calculations. l is referring to the length of the span of the composite sections, while the s_i is the spacing; the distance between the shear fasteners and it is determined in accordance with [22, clause B.2].

“The effective bending stiffness should be taken as:

$$(EI)_{ef} = \sum_{i=1}^{3}(E_i I_i + \gamma_i E_i A_i a_i^2) \qquad 3.25$$

Using mean values of E and where:

$$A_i = b_i h_i \tag{3.26}$$

$$I_i = \frac{b_i h_i^3}{12} \tag{3.27}$$

$$\gamma_2 = 1.0 \tag{3.28}$$

$$\gamma_1 = \left[1 + \frac{\pi^2 E_i A_i s_i}{K_i l^2}\right]^{-1} \tag{3.29}$$

For i = 1and i = 3

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^{3} \gamma_i E_i A_i} \tag{3.30}$$

Where the symbols are defined in Figure 3-3 and:

$K_i = K_{ser,i}$ for the Serviceability Limit State calculation

$K_i = K_{u,,i}$ for the Ultimate Limit State calculation

Normal stresses

The normal stresses should be taken as:

$$\sigma_i = \frac{\gamma_i E_i a_i M}{(EI)_{ef}} \tag{3.31}$$

$$\sigma_{m,i} = \frac{0.5 E_i h_i M}{(EI)_{ef}} \tag{3.32}$$

Maximum shear stress

The maximum shear stress occurs when the normal stresses are zero. The maximum shear stress in the web member (2) in Figure 3-3 should be taken as:

$$\tau_{2,max} = \frac{\gamma_3 E_3 A_3 a_3 + 0.5 E_2 b_2 h_2^2}{b_2 (EI)_{ef}} V \qquad 3.33$$

Fastener load

The load on a single shear fastener should be taken as:

$$F_i = \frac{\gamma_i E_i A_i a_i s_i}{(EI)_{ef}} V \qquad 3.34$$

Where:

i = 1 and 3, respectively;

$s_i = s_i(x)$ is the spacing of the fasteners as defined in [22, B.1.3(1)].

In the γ-method spacing between the shear fasteners is considered with equal length. If the shear fasteners have a varying length along the longitudinal direction, it should be calculated as an effective spacing, as follows: [22, clause 9.1.3(1)- 9.1.3(3)]

$$s_{ef} = 0.75 \cdot s_{min} + 0.25 \cdot s_{max} \qquad 3.35$$

Where the maximum spacing is smaller than four times the s_{min}.

It is possible to verify both the ULS and SLS using different load factors in the load calculations and use this in further calculations.

It is possible to provide new equations for CLT-concrete composite slab, by implement the above equations, for a T-section beam; one flange and one web, where the concrete section is referred to as the flange part and the timber is the web.

The effective bending stiffness for the CLT-concrete composite will then be:

$$(EI)_{ef} = E_1 I_1 + \gamma_1 E_1 A_1 a_1^2 + E_2 I_2 + \gamma_2 E_2 A_2 a_2^2 \qquad 3.36$$

Where the values for i=1 is the concrete element and i=2 is the timber element. The equations 3.37-3.40, for γ_1, γ_2, a_1 and a_2 is found with the equations below:

$$\gamma_1 = \sqrt{\frac{1}{1 + \frac{\pi E_1 A_1 s_1}{K_1 l^2}}} \qquad 3.37$$

$$\gamma_2 = 1.0 \qquad 3.38$$

$$a_2 = \frac{\gamma E_1 A_1 (h_1 + h_2)}{2(\gamma E_1 A_1 + E_2 A_2)} \qquad 3.39$$

$$a_1 = \frac{h_1 + h_2}{2} - a_2 \qquad 3.40$$

3.4.2 Shear analogy method

The Shear analogy method is another methodology that is used to perform theoretical analysis of a TCC. This method is only applicable for CLT elements manufactured with a gluing process, (i.e face-glue) and not for nailed or doweled CLT products. There is no limitation of the shear fastener used, nor restriction in the number of layers/lamellae of the CLT. This method also considers the shear deformation of the transverse layers in addition to the longitudinal layers [24, Ch 3, p. 8].

This method is a precise design methodology for CLT-concrete composite structure and has been confirmed through testing by FPInnovations. For almost any type of system configuration, this method considers the different moduli of elasticity of a single layer. By not neglecting the shear deformation of the transverse layers, the prediction will be more precise [24, Ch 3, p. 10].

The CLT element is considered as two virtual beams A and B. The sum of the inherent flexural and shear stiffness of the individual layers along their own centres is given as beam A. Beam B is given as the so-called Steiners points of the flexural and shear stiffness of the whole element. These two beams are joined together with infinitely rigid web members, so that the deflection between beams is achieved. Figure 3-4 below demonstrates beams A and B and is taken directly from [24, ch 3, p. 10].

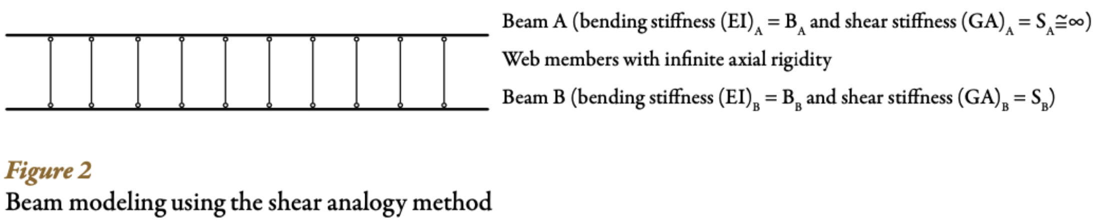


Figure 3-4 Beam modelling using the shear analogy method [24, Ch 3, p. 11]

The result of the whole CLT-concrete composite structure can be obtained when adding the bending and shear stresses for both beams together [24, ch 3, p. 10].

The equation 3.41 below expresses sum of the inherent bending stiffness from all the individual layers or cross-sections of a CLT element for the beam A: [24, ch3 p. 11]

$$(EI)_A = \sum_{i=1}^{n} E_i \cdot I_i = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{{h_i}^3}{12} \qquad 3.41$$

Where:

b_i : Width of the individual layer of the CLT element

h_i : Thickness of each individual layer

The bending stress for each individual layer of beam A is expressed as follows: [24, Ch. 3, p. 12]

$$\sigma_{A,i} = \pm \frac{M_{A,i}}{I_i} \cdot \frac{h_i}{2} \qquad 3.42$$

The shear stress for each individual layer of beam A is expressed as follows: [24, Ch3 p. 12]

$$\tau_{A,i} = \frac{E_i \cdot I_i}{(EI)_A} \cdot 1.5 \cdot \frac{V_A}{b \cdot h_i} \qquad 3.43$$

Where:

M_A : The bending forces on beam A

V_A : The shear forces on beam A

The bending and shear stresses for beam A can is shown in the Figure 3-5 below taken from [24, Ch. 3, p. 12, Figure 3].

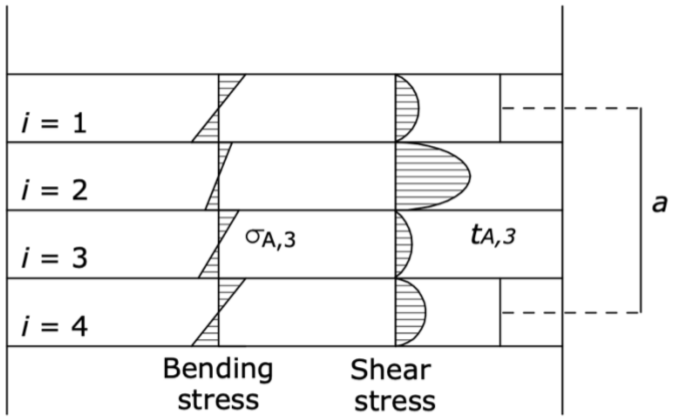


Figure 3-5 Bending and shear stresses in beam A [24, Ch. 3, p.12]

Similarly for beam B, the parallel axis theorem, which is the sum of the Steiner's points of the individual layers of the CLT element is used. The equation 3.44 below shows the bending stress for beam, where z_i is the distance from the neutral axis to the centre point of each layer [24, Ch. 3, p. 11].

$$(EI)_B = \sum_{i=1}^{n} E_i \cdot A_i \cdot {z_i}^2 \qquad 3.44$$

The bending stress for each individual layer of beam B is expressed as follows: [24, Ch. 3, p. 12]

$$\sigma_{B,i} = \frac{E_i \cdot z_i}{(EI)_B} \cdot M_B \tag{3.45}$$

The shear stress for each individual layer is expressed as follows: [24, Ch. 3, p. 12]

$$\tau_{A,i} = \frac{V_B}{(EI)_A} \cdot \sum_{j=i+1}^{n} E_j \cdot A_j \cdot z_j \tag{3.46}$$

Where:

M_B : The bending forces on beam B

V_B : The shear forces on beam B

The bending and shear stresses for beam B are shown in the Figure 3-6 below taken from Figure 4 from [24, Ch. 3, p. 13, Figure 4].

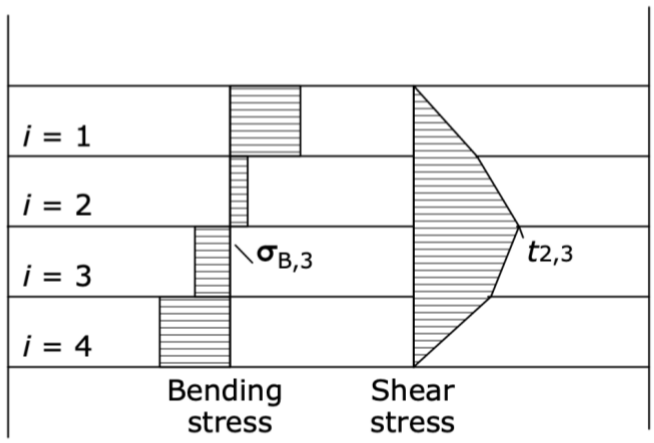


Figure 3-6 Bending and shear stresses in beam B [24, p. 13]

As mentioned earlier, the result of the whole CLT-concrete composite can be obtained when adding the bending and shear stress for both beams together [24, Ch. 3, p. 10]. This is achieved by summation of superposition of each beam. Figure 3-7 below, taken form [24, Ch. 3, p. 13, Figure 5] shows the final superposition.

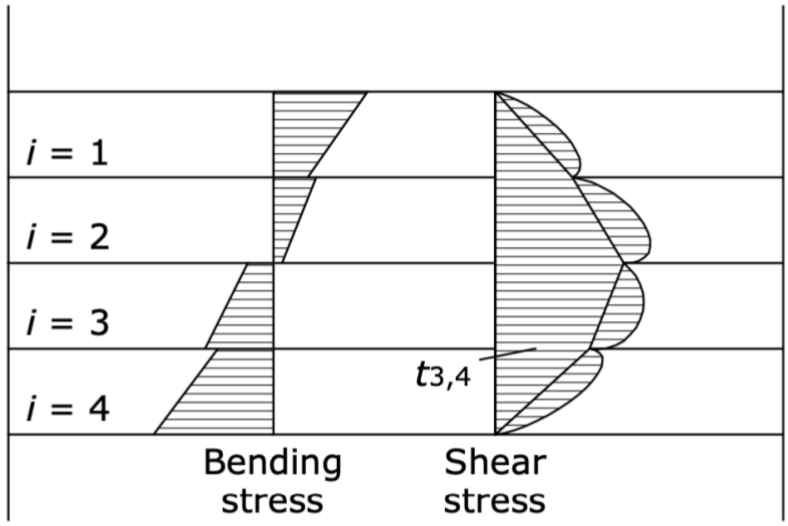


Figure 3-7 The final stress distribution obtained from the summation of result of A and B [24, Ch. 3, p. 13]

The final effective bending stiffness of the entire cross section of the CLT slab is as follows:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{{h_i}^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \qquad 3.47$$

Since the shear deflection is of significant influence, this must be included in the calculations for the effective bending stiffness. This is adjusted and a new equation 3.48 is obtained and shown below. This apparent equation reduces the effective bending stiffness, EI_{eff}.

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}} \qquad 3.48$$

Where EI_{eff} is calculated previously, L is the span length and K_s is a constant based upon the influence of the shear deformation. This constant has been solved for different loading scenarios and is expressed in the Table 3-1 below, taken form [24, Ch. 3, p. 5].

Table 3-1 Ks values for various loading conditions [24, 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

Where the effective shear stiffness, GA_{eff} is acquired as follows: [24, Ch. 3, p. 14]

$$GA_{eff} = \frac{a^2}{\left[\frac{h_i}{2 \cdot G_1 \cdot b} + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \frac{h_n}{2 \cdot G_n \cdot b}\right]} \qquad 3.49$$

In the above equations it is important to use the correct property values for the timber; for the direction that is parallel to the grain and the layers that are perpendicular to the grain. For the longitudinal lamellae, the E_0, modulus of elasticity parallel to the grain and the G, shear modulus should be used. For the transverse lamellae, the E_{90}, modulus of elasticity perpendicular to the grain and the G_R, rolling shear modulus should be used. The rolling shear is approximate on tenth of the shear modulus [24, Ch. 3, p. 11].

4 Theoretical predictions

The theoretical predictions are based on the design calculations performed by Rannveig W. Haug [25, Ch. 7]. In the beginning of the semester, the supplier (Splitkon) of the CLT slabs, could only deliver a 3-layered CLT. Theoretical predictions for these 3-layered CLT slabs can be found in Appendix F. In mid-February 2021, this purchase order of thirteen identical CLT slabs were then changed to 5-layered slabs. This affected the already performed theoretical predictions for the 3-layered slabs, since it is not possible to only use the γ-method for a 5-layered slab and the additional change in the material properties.

A more complex calculation with combination of the shear analogy and the γ-method had to be performed for CLT-concrete slab with 5-layer CLT. The laboratory experiment was designed with regards to the allowable maximum weight capacity for test machine and CLT slabs provided by Splitkon. Hence, calculations were performed to find load capacity, material dimensions and material properties of the concrete.

The maximum dimensions for the elements for the four-point bending test is weight 500 kg, length 2400 mm, width 1000 mm and applied load 400 kN. Splitkon could only deliver 13 identical CLT slab with the following dimensions; width 600 mm and length of 2100 mm, the distance between the supports will be 2000 mm. These restrictions are the basis for the theoretical predictions, type of concrete and number of shear fasteners. In the following chapters, the load capacity and maximum deflection calculations are performed, based on the theory in chapter 3.

4.1 Cross Laminated Timber

The dimensions of the received slabs were; length of 2100 mm, width of 600 mm and height of 120 mm. Figure 4-1 shows the arrangement of the lamellae of the cross section of CLT slab. Where the two outermost and the middle lamella is parallel to the grain and lamellae 2 and 4 are perpendicular to the grain. This arrangement is opposite in the longitudinal direction.

It is arranged that the two outermost and the middle lamellae is parallel to the grain, and second and fourth lamella have the orientation of perpendicular to the grain. The two outer lamellae are of strength class T22 and the three inner lamellae are of strength class T15, see Table 4-1, - Table 4-3 for material properties [26, p. 1].

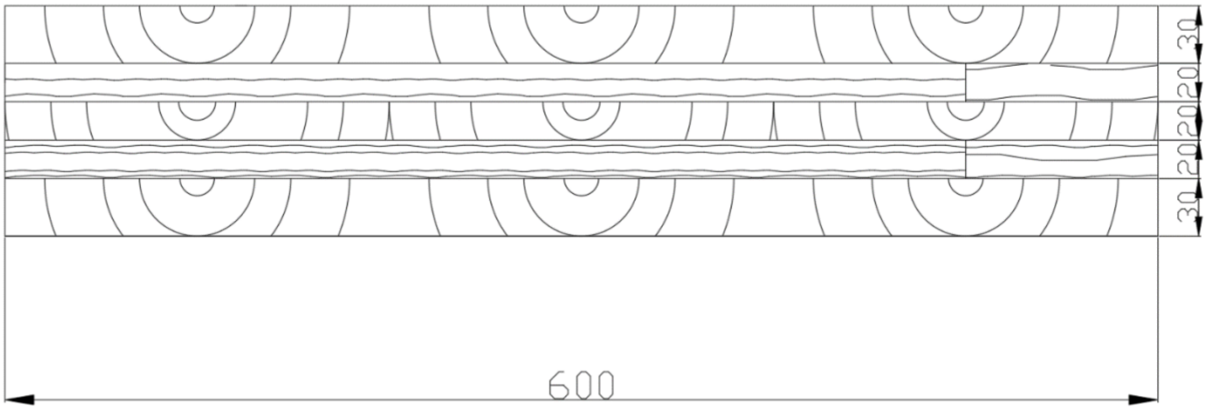


Figure 4-1 Cross sectional cut of a CLT slab [mm]

Table 4-1 Material properties and dimensions for CLT in general [26, Table 2]

Parameter	**Notation**	**Value**
Length	L	$2100\ mm$
Height, timber total	h_t	$120\ mm$
Height, Lamella 1	h_1	$30\ mm$
Height, Lamella 2	h_2	$20\ mm$
Height, Lamella 3	h_3	$20\ mm$
Height, Lamella 4	h_4	$20\ mm$
Height, Lamella 5	h_5	$30\ mm$
Width	b	$600\ mm$
Cross sectional area	A_t	$72000\ mm^2$
Moment of Inertia	I_t	$86400000\ mm^4$
Partial factor for CLT timber	γ_M	1.5
Modification factor	k_{mod}	0.8
Deformation factor	k_{def}	0.85

Table 4-2 Material properties and dimensions for strength class T22, CLT [26, Table 2]

Parameter	Notation	Value
Modulus of elasticity, parallel to grain	$E_{0,mean}$	$13000\ MPa$
Modulus of elasticity, perpendicular to grain	$E_{90,mean}$	$430\ MPa$
Shear modulus, parallel to grain	$G_{0,mean}$	$810\ MPa$
Shear modulus, perpendicular to grain	$G_{90,mean}$	$81\ MPa$
Characteristic bending strength	$f_{m,k}$	$30.5\ MPa$
Characteristic tensile strength along the grain	$f_{t,0,k}$	$22.0\ MPa$
Characteristic shear strength	f_v	$4.0\ MPa$
Density	ρ_{timber}	$470\ kg/m^3$

Table 4-3 Material properties and dimensions for strength class T15, CLT [26, table 2]

Parameter	Notation	Value
Modulus of elasticity, parallel to grain	$E_{0,mean}$	$11500\ MPa$
Modulus of elasticity, perpendicular to grain	$E_{90,mean}$	$230\ MPa$
Shear modulus, parallel to grain	$G_{0,mean}$	$720\ MPa$
Shear modulus, perpendicular to grain	$G_{90,mean}$	$72\ MPa$
Characteristic bending strength	$f_{m,k}$	$22.0\ MPa$
Characteristic tensile strength along the grain	$f_{t,0,k}$	$15.0\ MPa$
Characteristic shear strength	f_v	$4.0\ MPa$
Density	ρ_{timber}	$430\ kg/m^3$

4.2 Concrete

The type of concrete chosen for this thesis is self-compacting and have the strength class of B35. SCC is chosen, because it easier to use in larger constructions where it is harder to use vibrators for consolidation and flows under its own weight to undergo compaction without external vibration. The material properties for the concrete with strength class B35 are listed below in Table 4-4 and is taken from the [21, Table 3.1].

Table 4-4 Concrete B35, material properties and dimensions [21, Table 3.1]

Parameter	Notation	Value
Length	L	$2100\ mm$
Height	h_c	$60\ mm$
Width	b	$600\ mm$
Cross sectional area	A_c	$36000\ mm^2$
Moment of Inertia	I_c	$10800000\ mm^4$
Modulus of elasticity	E_{cm}	$34000\ MPa$
Characteristic compressive cylinder strength of concrete at 28 days	f_{ck}	$35\ MPa$
Characteristic axial tensile strength of concrete	$f_{ctk,0.05}$	$2.2\ MPa$
Partial factor for concrete	γ_c	1.5
Creep coefficient	φ_c	2.5
Density of concrete	ρ_c	$25\ kN/m^3$

The height of the concrete was determined based on 3 different references and the derivation of the height is shown in equations below:

Since the height of the CLT wood is known, rearranging the formulas, the height of concrete is found [10, p. 94].

$$h_{CLT} = 0.6 \cdot h_{tot} \tag{4.1}$$

$$h_{tot} = \frac{h_{CLT}}{0.6} = \frac{120}{0.6} = 200\ mm \quad 4.2$$

$$h_c = 0.4 \cdot h_{tot} = 80\ mm \quad 4.3$$

The research paper that are the basis of [25, Ch. 7], approximately 30% of the total height is used as the height of concrete [27, Appendix A, p. 67].

$$h_{CLT} = 0.7 \cdot h_{tot} \quad 4.4$$

$$h_{tot} = \frac{h_{CLT}}{0.7} = \frac{120}{0.7} = 171.4286\ mm \quad 4.5$$

$$h_c = 0.3 \cdot 171.4286 = 51.428mm \quad 4.6$$

In the [27, p. 227] it says that the concrete height should satisfy the following conditions:

$$50\ mm \leq h_c \leq 0.7 \cdot h_t \quad 4.7$$

$$50\ mm \leq h_c \leq 84\ mm \quad 4.8$$

Therefore, a rounded number, 60 mm for the height of concrete and then the total height is 180 mm used in this thesis.

4.3 Types of shear fasteners

The types of shear fasteners were chosen based on availability, cost and time mounting them onto the CLT slab. An interesting study was to compare two different types of dowel screws and the orientation, to investigate any effect on the CLT-concrete composite slab. UiS also got a collaboration with one of the leading developers and provider of high technology solutions, Rothoblaas. The types of shear fasteners for laboratory testing were chosen from their catalogue. CTC and KOP screw is used and is discussed below.

4.3.1 CTC screws

Figure 4-2 CTC screws

The CTC screw is Rothoblaas shear fastener for timber-concrete floor, shown in Figure 4-2. In the catalogue it has been tested and calculated with parallel and crossed parallel arrangement of both 45^o and 30^o. This type of shear fastener is self-drilling, fast and minimally invasive, making it easier and less time consuming to install. CTC screws are designed for TCC structures and was therefore chosen [28].

The CTC screws were arranged in pairs crossed parallelly of 45^o and 45/135^o angle, shown in figures below. This orientation allows for one of the screws to absorb the tensile force, while the screws in the opposite direction act as stiffeners [6, p. 18].

In addition to the CTC screws, a reinforcement mesh with dimensions of 150x150 mm, with a diameter of 5 mm was installed. Table 4-5 below lists the necessary mechanical properties.

Table 4-5 Mechanical characteristics for CTC screws [28, p.224]

Parameter	Notation	Value
Diameter of head	d	$7\ mm$
Length	l	$160\ mm$
Effective length	l_{eff}	$110\ mm$
Characteristic tensile strength	$f_{tens,k}$	$20.0\ kN$

4.3.1.1 The slip modulus of CTC screws

The slip modulus for CTC screw is determined based on laboratory testing conducted by Rothoblaas, [28, p. 225] as follows:

$$K_{ser} = n \cdot 70 \cdot l_{eff} \quad 4.9$$

Where n is number of pairs in one row, the number of pairs in the width.

The slip modulus is to be considered as relating to a single inclined shear fastener or pair of crossed shear fasteners to a parallel force at the slip surface. For three pairs of shear fasteners with the width of 600 mm CLT slab, the slip modulus is as follows:

$$K_{ser} = 3 \cdot 70 \cdot 110 = 23100\ N/mm \quad 4.10$$

For SLS, the slip modulus used for calculation is K_{ser}, but for ULS, $K_u$is used [22, clause 2.2.2(2)] as follows:

$$K_u = \frac{2}{3} \cdot K_{ser} = 15400\ N/mm \quad 4.11$$

4.3.1.2 Spacing of CTC screws

The spacing is determined on the basis the minimum spacing in accordance with the catalouge provided by Rothoblaas [28, p. 227]. The spacing can be various or continuously along the length, but for this experiment it has been chosen to use continuously spacing. Therefore, the effective spacing is not necessary to calculate.

The Figure 4-3 below shows the minimum spacings and are presented in Table 4-6 together with the chosen spacing values.

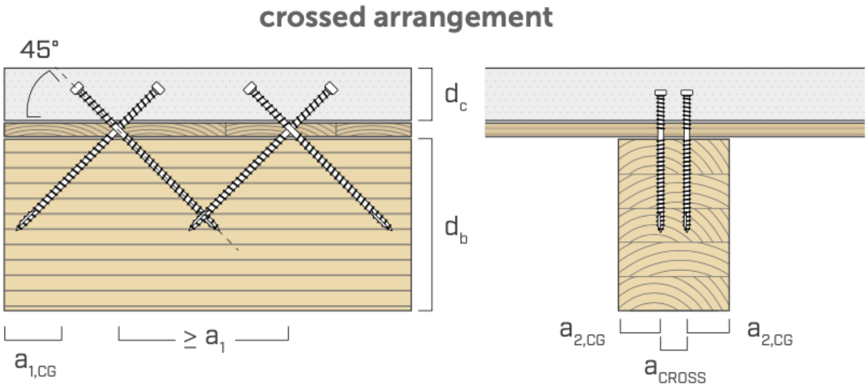


Figure 4-3 Minimum distances for axial stresses crossed arrangement for CTC screws [27, p. 227]

Table 4-6 Minimum spacing for axial stresses

The distance	Minimum spacing	Chosen spacing
$s^* = a_1$	$130 \cdot \sin(45) = 91.923\ mm$	$150\ mm$
$a_{1,CG}$	$85\ mm$	$150\ mm$
$a_{2,CG}$	$32\ mm$	$120\ mm$
a_{CROSS}	$11\ mm$	$20\ mm$

Where "s*" is the spacing between the shear fasteners in the longitudinal direction. Figure 4-4 illustrates the spacing of a cross-sectional section of the slab element type A.

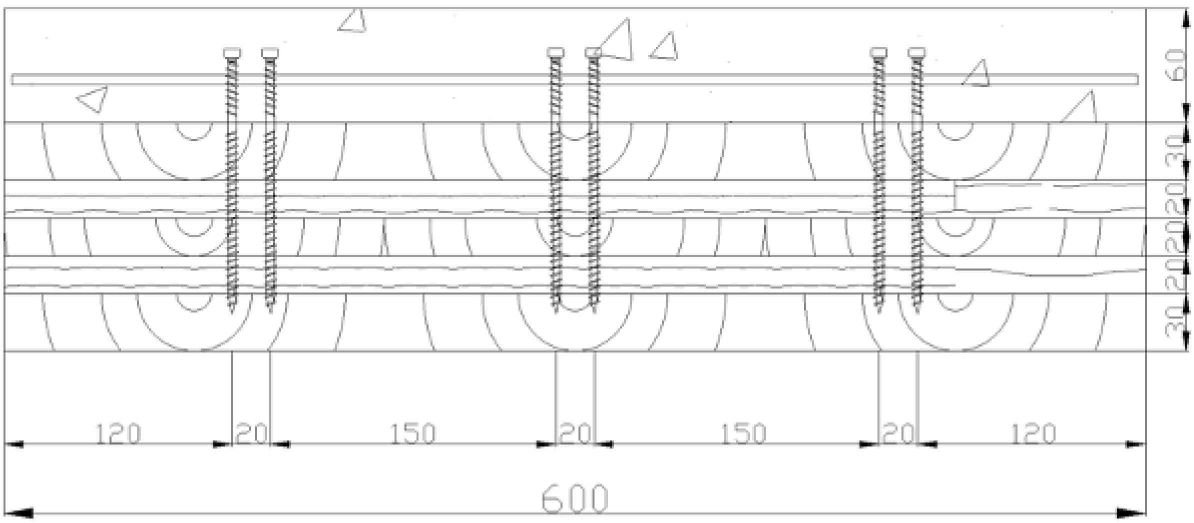


Figure 4-4 Cross section of CLT-concrete composite with CTC screws and reinforcement [mm]

4.3.1.3 Assembling of the CTC screws

This type of screws is oriented in pairs, crossed parallel and angles of 45º and 45/135º illustrated in Figure 4-5.

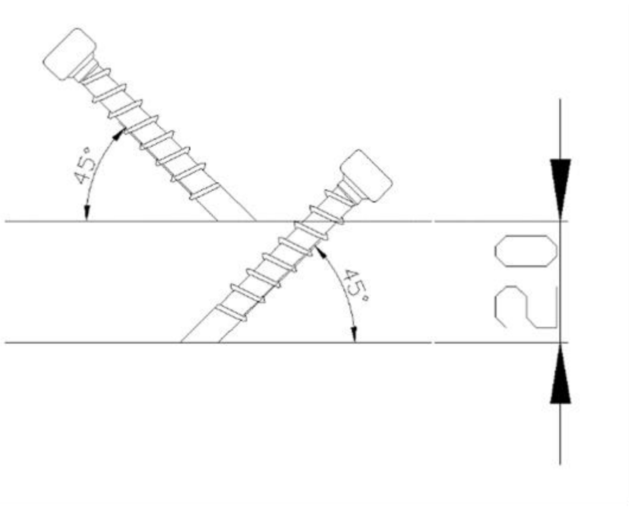


Figure 4-5 Orientation of the CTC screws [mm]

After finding the required spacing, the number of rows and required number of shear fasteners is determined as follows:

The number of screws for each slab is:

$$Number\ of\ CTC\ screws = 13\ rows\ x\ 3\ pairs\ each\ row = 78 \tag{4.12}$$

The number of screws for each slab is:

$$Number\ of\ CTC\ screws\ in\ total = 6\ slabs\ x\ 78\ screws\ per\ slab = 468 \tag{4.13}$$

The longitudinal section of the CLT-concrete composite slab with CTC screws is illustrated in Figure 4-6 below.

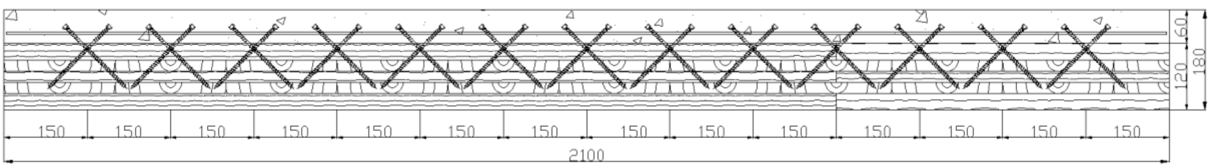


Figure 4-6 Longitudinal section of the CLT-concrete composite slab with CTC screws [mm]

Figure 4-7 shows the CLT-concrete composite slab with CTC screws seen from above, illustrating the spacing and the mesh reinforcement.

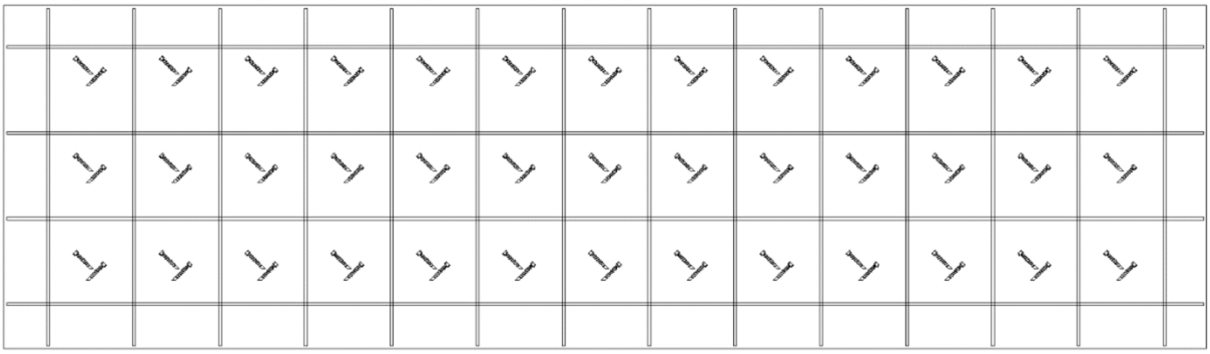


Figure 4-7 Top view of CTC screws and mesh reinforcement, before casting concrete

4.3.2 KOP screws

Figure 4-8 KOP screw

Rannveigs master thesis [25], is the basis of this laboratory testing. The KOP screws, shown in Figure 4-8, were arranged parallelly in pairs with a 45^{o} angle, opposite to the design example, to investigate whether there is a difference in the orientation of the shear fasteners. Since there were a CLT slab and KOP screws in spare, this element had KOP screws oriented crossed parallel with 45^{o} angle, shown in Figure 4-9.

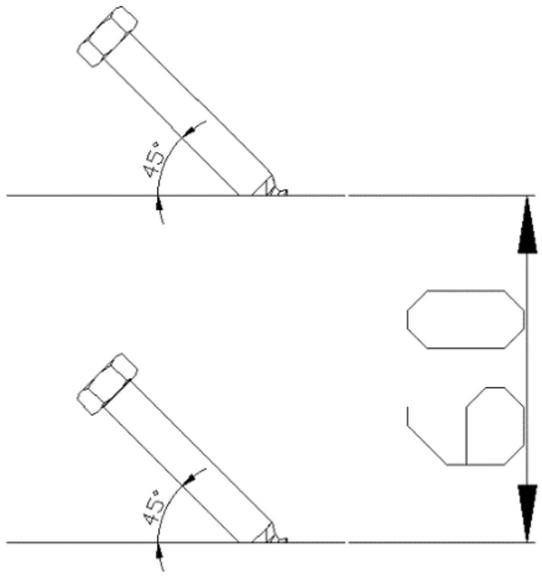


Figure 4-9 Orientation of the KOP screws [mm]

Table 4-7 below lists the necessary mechanical properties

Table 4-7 Mechanical characteristics for KOP screws

Parameter	Notation	Value
Diameter of head	d	$10\ mm$
Length	l	$140\ mm$
Threaded length	l_{eff}	$84\ mm$
Characteristic tensile strength	$f_{tens,k}$	$23.6\ kN$

4.3.2.1 The slip modulus of KOP screws

Spacing for KOP screw is calculated in accordance with [22, clause 7.1(1) - 7.1(3)].

$$K_{ser} = 3\ rows \cdot \frac{d}{23} \cdot \rho_m^3 = 12432.77728\ N/mm \qquad 4.14$$

Where:

d : Diameter of the KOP screw

ρ_m : Calculation of the mean timber densities with two different material properties found with the equation: $\rho_m = \sqrt{\rho_{T22} \cdot \rho_{T15}}$

For SLS, the slip modulus used for calculation is K_{ser}, but for ULS, $K_u$is used [22, clause 2.2.2(2)] as follows:

$$K_u = \frac{2}{3} \cdot K_{ser} = 8288.5181\ N/mm \qquad 4.15$$

4.3.2.2 Spacing of KOP screws

The spacing is determined on the basis the minimum spacing in accordance with the Eurocode 5 [22, clause 8.7.2.(1) - 8.7.2.(4), Table 8.6]. Figure 4-10 below shows the minimum spacings and are presented in Table 4-8 together with the chosen spacing values.

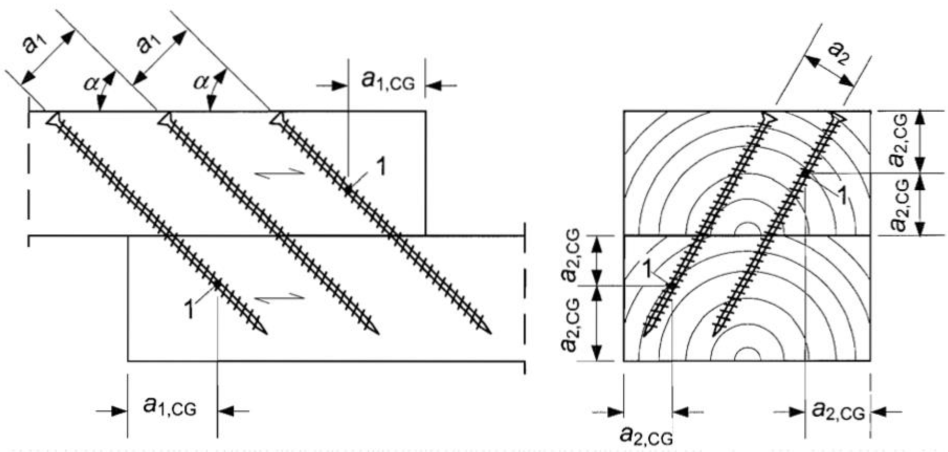


Figure 4-10 Minimum distances for axial stresses crossed arrangement [22, Figure 8.11a]

Table 4-8 Minimum spacing for axial stresses

The distance	Minimum spacing	Chosen spacing
$s^* = a_1$	$7 \cdot d = 70\ mm$	$100\ mm$
a_2	$5 \cdot d = 50\ mm$	$60\ mm$
$a_{1,CG}$	$10 \cdot d = 70\ mm$	$150\ mm$
$a_{2,CG}$	$4 \cdot d = 40\ mm$	$60\ mm$

The distance between the KOP screws of the inclined parallel pair is 60 mm centre to centre. The width of the slab is 600 mm, the minimum spacing from the edges in longitudinal direction is 40 mm and chosen spacing from edge is 120 mm. The Figure 4-11 below illustrate the pattern of spacing. However, the screws are shown with 90^o angle.

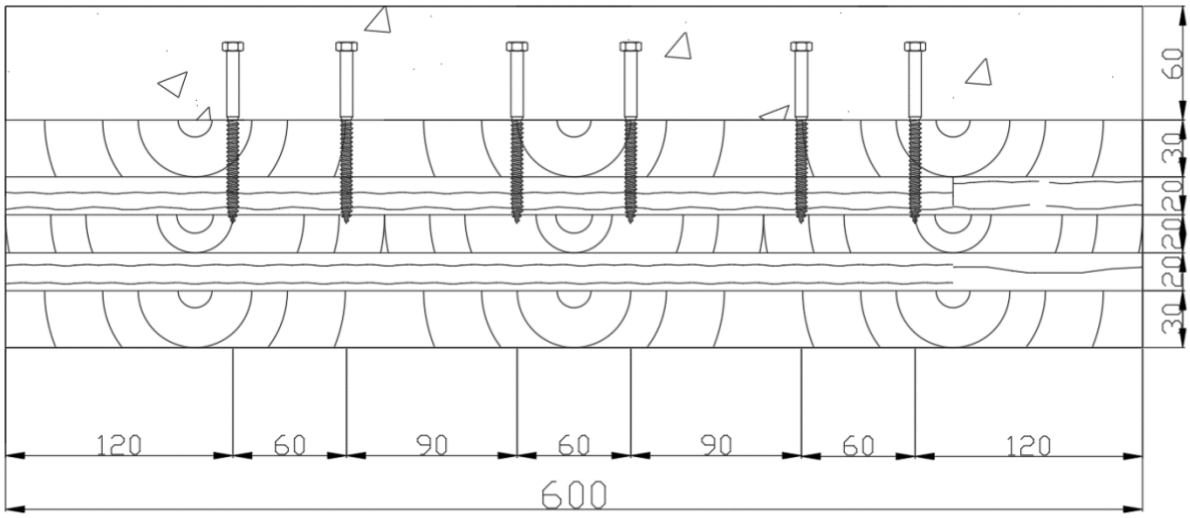


Figure 4-11 Cross section of CLT-concrete composite with KOP screws [mm]

4.3.2.3 Assembling of KOP screws

This type of screws is oriented in pairs with a 45º angle. After finding the required spacing, the number of rows and required number of shear fasteners can be determined:

The number of screws for each slab is:

$$Number\ of\ KOP\ screws = 19\ rows\ x\ 3\ pairs\ each\ row = 114 \tag{4.16}$$

The number of screws for each slab is:

$$Number\ of\ CTC\ screws\ in\ total = 6\ slabs\ x\ 184\ screws\ per\ slab = 684 \tag{4.17}$$

The longitudinal section of the CLT-concrete composite slab with KOP screws is illustrated in Figure 4-12 below.

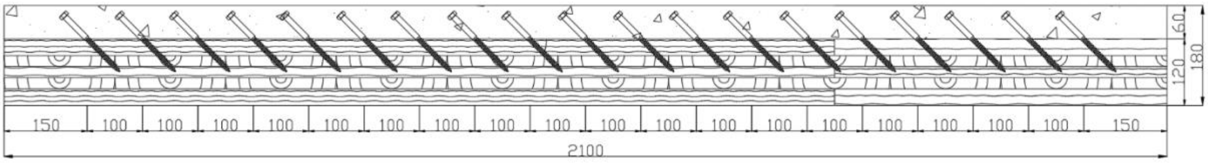


Figure 4-12 Longitudinal section of the CLT-concrete composite slab with KOP screws [mm]

Figure 4-13 shows the CLT-concrete composite slab with KOP screws seen from above, illustrating the pattern of spacing.

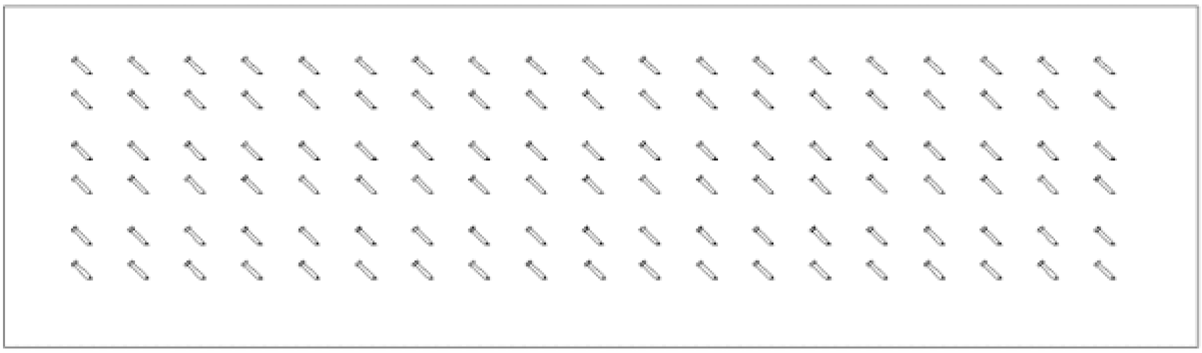


Figure 4-13 Top view of KOP screws, before casting concrete

4.4 Calculations of maximum applied load

The theoretical calculations are performed by following the theories and methods discussed in Chapter 3, using a combination of shear analogy and γ-method. The maximum load capacity is calculated and then the CLT-concrete composite slab is checked towards the ULS and SLS verification.

The following chapters shows the theoretical prediction procedure for the CLT-Concrete composite slab using the CTC-screws as shear fasteners, oriented crossed parallel with 45^{o} angle.

Eugenio Facchini, in Rothoblaas, has provided with an Excel spreadsheet for verification of the CTC screws. The spreadsheet is included in the Appendix C, and have been used to compare the theoretical predictions.

4.4.1 Load calculations

For the four-point-load test, it will be simply supported on both sides, see Figure 4-14. Besides the applied load, the dead load is the only load taking into consideration since safety factors and variable loading is not included in laboratory testing.

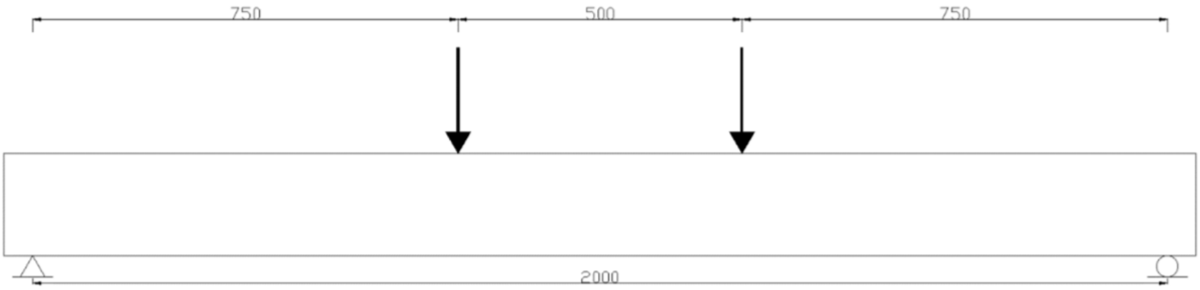


Figure 4-14 Simplified model of the four-point bending test, with pinned and roller support [mm]

Equation 4.18 shows how to calculate the characteristic value for dead load.

$$g_{0,k} = b \cdot h_c \cdot \gamma_c + b \cdot h_t \cdot \gamma_M \tag{4.18}$$

4.4.2 Load capacity for slab type A based on ULS verifications

This chapter describes the calculations to determine the maximum applied load allowed for the laboratory testing. Theoretical predictions for the CLT-concrete composite slab with CTC screws as shear fasteners is performed in the following chapters. Calculations for KOP screws are presented in a table in Chapter 4.5. All load capacity predictions based on ULS can be found in Appendix A.

4.4.2.1 Shear analogy method for CLT elements

Based on the theory described in Chapter 3.4.2, the shear analogy method includes the shear deformation in the transverse layers for an element with more than three layers. This is done by calculating the effective bending stiffness for the CLT element. For the following equations "i" describes the number of the layer. Layers 1 and 5 are the outermost layers with the martial properties of T22, and middle layers 2, 3 and 4 with material properties of T15.

The formula for the effective bending stiffness of the CLT element is:

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{{h_i}^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i z_i^2 \quad 4.19$$

Where these necessary calculations finding the effective bending stiffness are as follows:

For layer 1 and 5:

$$A_1 = b \cdot h_1 = 18000\, mm^2 \quad 4.20$$

Similarly, for A_5:

$$A_5 = A_1 = 18000\, mm^2 \quad 4.21$$

The moment of inertia for layers 1 and 5:

$$I_1 = \frac{b \cdot h_1^3}{12} = 1350000\, mm^4 \quad 4.22$$

$$I_5 = I_1 = 1350000\, mm^4 \quad 4.23$$

For layers 2, 3 and 4:

$$A_2 = b \cdot h_2 = 12000\, mm^2 \quad 4.24$$

Similarly, for $A_3\ and\ A_4$:

$$A_3 = A_4 = A_2 = 12000\, mm^2 \quad 4.25$$

The moment of inertia for layers 2, 3 and 4:

$$I_2 = \frac{b \cdot h_2^3}{12} = 400000\ mm^4 \tag{4.26}$$

$$I_3 = I_4 = I_2 = 400000\ mm^4 \tag{4.27}$$

The following equations will present the calculations for z_i, the distance from each layer to the Neutral Axis (NA):

$$z_1 = \frac{h_1}{2} + h_2 + \frac{h_3}{2} = 45\ mm \tag{4.28}$$

$$z_2 = \frac{h_2}{2} + \frac{h_3}{2} = 20\ mm \tag{4.29}$$

$$z_3 = 0\ mm \tag{4.30}$$

$$z_4 = \frac{h_4}{2} + \frac{h_3}{2} = 20\ mm \tag{4.31}$$

$$z_2 = \frac{h_5}{2} + h_4 + \frac{h_3}{2} = 45\ mm \tag{4.32}$$

The Table 4-9 below shows the answers for the necessary calculations needed to find the effective bending stiffens. For the modulus of elasticity there is a difference in the grain direction. The longitudinal layers, layer 1, 3, 5 will use the mean value for the modulus of elasticity for parallel to the grain, while layers 2 and 4 will use the values for perpendicular to the grain.

Table 4-9 Intermediate calculations for EI_{eff}

Layer	$E_{i,mean}$ $[N/mm^2]$	$E_i \cdot I_i$ $[Nmm^2]$	$E_i \cdot A_i \cdot z_i^2$ $[Nmm^2]$
$i = 1$	13000	17550000000	473850000000
$i = 2$	230	92000000	1104000000
$i = 3$	11500	4600000000	0
$i = 4$	230	92000000	1104000000
$i = 5$	13000	17550000000	473850000000
Sum of Intermediate calculations		39884000000	949908000000

From the Table 4-9 the effective bending stiffness is as follows:

$$EI_{eff} = 9.89792 \cdot 10^{11}\ Nmm^2 \qquad 4.33$$

This effective bending stiffness does not consider the shear deformation in the transverse layer. Therefore, a new adjusted effective bending stiffness is calculated; the apparent effective bending stiffness is as follows:

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s \cdot EI_{eff}}{GA_{eff} \cdot L^2}} \qquad 4.34$$

Where the effective shear stiffness is calculated as follows:

$$GA_{eff} = \frac{a^2}{\left[\frac{h_i}{2 \cdot G_1 \cdot b} + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \frac{h_n}{2 \cdot G_n \cdot b}\right]} \qquad 4.35$$

Where:

"a" is the distance between the geometrical centre of the two outermost layers as follows:

$$a = \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2} = 90\ mm \qquad 4.36$$

Table 4-10 Intermediate calculations for GA_{eff}

Layers	$\boldsymbol{G_i}$ $[\boldsymbol{N/mm^2}]$	$\boldsymbol{\frac{h_i}{G_i}}$
$i = 1$	810	0.03703703704
$i = 2$	72	0.2777777778
$i = 3$	720	0.02777777778
$i = 4$	72	0.2777777778
$i = 5$	810	0.03703703704

$$GA_{eff} = 7.834029851 \cdot 10^6\ N/mm^2 \qquad 4.37$$

Then the apparent effective bending stiffness is calculated, where K_s is taken from Table 3-1.

$$EI_{app} = 7.260572161 * 10^{11} \qquad 4.38$$

Then the modulus of elasticity for the CLT element, where the shear deformation in the transverse layers is considered as follows:

$$E_{CLT} = \frac{EI_{app}}{\frac{b \cdot h_t^3}{12}} = 8403.44\ Nmm^2 \qquad 4.39$$

4.4.2.2 Load capacity calculations using short-term verifications of the slab

After finding the modulus of elasticity of the CLT element, it is possible to determine the procedure to find the maximum allowed load and verify it with accordance standards, as described in Chapter 3.2 and the γ-method as described in Chapter 3.4.1. For the following equation “i” describes the material type, 1 is used for concrete and 2 is used for CLT.

The modulus of elasticity for concrete element is as follows:

$$E_1 = 34000\ Nmm^2 \tag{4.40}$$

The modulus of elasticity for the 5-layered CLT element is as follows:

$$E_2 = 8403.44\ Nmm^2 \tag{4.41}$$

For CTC screws the calculation for the slip modulus and spacing have been described previously in Chapter 4.3.1.1 and 4.3.1.2 respectively.

$$K_u = \frac{2}{3} \cdot 23100 = 15400\ N/mm \tag{4.42}$$

An assumption of a linear relationship between force and slip is made for the mechanical fastener [22, clause 9.1.3(2)].

The spacing "s" is calculated in accordance with [22, clause 8.7.2(2)] and the minimum spacing provided by [28, p. 225] as described in Chapter 4.3.1.2.

$$s = 150\ mm^{*} \tag{4.43}$$

* This will be performed similarly for the KOP screws as shear fasteners, but since there is not described a similar approach in the catalogue, Appendix M, it will be performed in accordance with [22, clause 8.7.2(2)] for the minimum spacing.

Since the transverse layers of the CLT have been included and the effective bending stiffness adjusted, it is now possible to find the effective bending stiffness in accordance with the γ-method [22, Annex B]. The CLT-concrete composite will be considered as one element composed of two parts, so the 5-layers CLT is then looked at as one element and concrete as the other one. The γ-factor, regards to what degree of full composite action the element has, where 0 being no composite action, while 1 equal to fully composite action.

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot A_1 \cdot s}{K_u \cdot L^2}} = 0.03287684039 \quad 4.44$$

$$\gamma_2 = 1.0 \quad 4.45$$

Then the distance from the NA to the centre of the i-layer can be determined as follows:

$$a_1 = \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)} = 5.612544326\, mm \quad 4.46$$

$$a_2 = \frac{(h_1 + h_2)}{2} - a_1 = 84.38745567\, mm \quad 4.47$$

The effective bending stiffness for the CLT-concrete composite slab can now be determined in accordance with [22, Annex B].

$$EI_{eff,tot} = E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2 \quad 4.48$$

$$EI_{eff,tot} = 1.398884340 \cdot 10^{12}\, Nmm^2 \quad 4.49$$

Now, it is possible to determine the maximum applied load by determining the moments for top and bottom of both concrete and CLT element using the verification formulas for normal stresses described in Chapter 3.4.1 γ-method.

Top part of the concrete:

$$\sigma_{c,t} = -\sigma_1 - \sigma_{m,1} = \frac{f_{ck}}{\gamma_c} \quad 4.50$$

Finding the moment for the top part of the concrete:

$$M_1 = \frac{f_{ck}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}}\right)} = 29.2917\, kNm \quad 4.51$$

For the bottom part of the concrete:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1} = \frac{f_{ctk,0.05}}{\gamma_c} \tag{4.52}$$

Finding the moment for the bottom part of the concrete:

$$M_2 = \frac{f_{ctk,0.05}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}}\right)} = 2.21664\ kNm \tag{4.53}$$

Similar moment calculation using verification calculations are performed on the CLT element.

For the top part of the timber section:

$$\sigma_{t,t} = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} \leq 1.0 \tag{4.54}$$

Where:

$$f_{t,0,d} = \frac{k_{mod} \cdot f_{t,0,k.t22}}{\gamma_M} \tag{4.55}$$

$$f_{m,d} = \frac{k_{mod} \cdot f_{m,k,t22}}{\gamma_M} \tag{4.56}$$

$$M_3 = \frac{\frac{k_{mod}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k.t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)} = 52.108496\ kNm \tag{4.57}$$

For the bottom part of the timber section:

$$\sigma_{t,b} = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} \leq 1.0 \qquad 4.58$$

$$M_4 = \frac{\frac{k_{mod}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k.t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)} = 67.63763\ kNm \qquad 4.59$$

The maximum moment is determined, neglecting the bending moment in the bottom part of the concrete, because the value of it is considerably small, because of the steel reinforcement mesh for slab type A, this will take some part of the tensile stress. A similar assumption is made for slab type B.

$$M_{Ed,new} = \min[\mathrm{M}_1, M_3, M_4] = 29.29173\ kNm \qquad 4.60$$

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

$$P_{Ed} = solve\left(\frac{P_{Ed} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L^2}{8}\right) = M_{Ed,new} \qquad 4.61$$

$$P_{Ed} = 75.67809\ kN \qquad 4.62$$

Then it is necessary to verify the top and bottom sections for both the concrete and CLT, to verify that it satisfies the maximum applied load.

Stresses in the concrete element:

$$\sigma_1 = \frac{\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new}}{EI_{eff,tot}} = 1.957516\ MPa \qquad 4.63$$

$$\sigma_{m,1} = \frac{0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new}}{EI_{eff,tot}} = 21.35813\ MPa \tag{4.64}$$

Stresses on the top part of the concrete element:

$$\sigma_{c,t} = -\sigma_1 - \sigma_{m,1} = -23.333\ MPa \tag{4.65}$$

Stresses on the bottom part of the concrete element:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1} = 19.38\ MPa \tag{4.66}$$

Verification of the top part of the concrete element:

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

$$-1.000\ \leq 1.0 \rightarrow OK \tag{4.67}$$

Verification of the bottom part of the concrete element:

$$Verification = \frac{\sigma_{c,t}}{\frac{f_{ctk,0.05}}{\gamma_c}} \leq 1.0$$

$$13.2156\ \leq 1.0 \rightarrow NOT\ OK \tag{4.68}$$

Since the bottom part of the concrete does not satisfy the verification calculations it has to be performed some modifications as proposed below.

It is possible to modify the calculations by consider only the effective compressive height of the concrete, which will further make some adjustments to the effective bending stiffness and satisfy the conditions for the tensile stress at the bottom part of the concrete. When considering only the effective compressed height of concrete some assumptions are made:

That the γ-factor is considered for the whole cross section of concrete and the tensile strength is neglected [6, p. 134]. This means that the γ-factors remain the same, but the heights a_i will be adjusted, resulting in a change in the total effective bending stiffness for the composite structure.

The quadratic equation:

$$a_{1,eff} = a_1^2(4\gamma_1^2 E_1 b) + a_1[2E_2A_2(1+\gamma_1)] + E_2A_2(2h_1 + h_2) = 0 \qquad 4.69$$

$$a_{1,eff} = 115.243\ mm$$

The effective compressed height of the concrete:

$$x = 2\gamma_1 a_{1,eff} = 7.57766\ mm \qquad 4.70$$

The equations 4.71-4.73 below presents the new modified values needed for calculating the new effective bending stiffness.

$$a_{2,new} = h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,eff} = 0.9680\ mm \qquad 4.71$$

$$A_{1,eff} = b \cdot x = 4546.597\ mm^2 \qquad 4.72$$

$$I_{1,eff} = b \cdot x^3 = 21755.829\ mm^4 \qquad 4.73$$

The modified effective bending stiffness can now be found as follows:

$$EI_{eff,tot,new} = E_1 \cdot I_{1,eff} + \gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff}^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new}^2 \qquad 4.74$$

$$EI_{eff,tot,new} = 7.94861 \cdot 10^{11}\ Nmm^2 \qquad 4.75$$

Using the new modified effective bending stiffness, to check if the top and bottom sections of the concrete and timber element satisfies the conditions. The moment and other variables will remain as previously calculated.

Stresses in the concrete element:

$$\sigma_1 = \frac{\gamma_1 \cdot E_1 \cdot a_{1,eff} \cdot M_{Ed,new}}{EI_{eff,tot,new}} = 4.74720437\ MPa \tag{4.76}$$

$$\sigma_{m,1} = \frac{0.5 \cdot E_1 \cdot x \cdot M_{Ed,new}}{EI_{eff,tot,new}} = 4.74720437\ MPa \tag{4.77}$$

Stresses on the top part of the concrete element:

$$\sigma_{c,t} = -\sigma_1 - \sigma_{m,1} = -9.49440874\ Mpa \tag{4.78}$$

Stresses on the bottom part of the concrete element:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1} = 0 \text{ MPa} \tag{4.79}$$

Verification of the top part of the concrete element:

$$Verification = \frac{\sigma_{c,t}}{\frac{f_{ck}}{\gamma_c}} \leq 1.0 \tag{4.80}$$

$$-0.406903 \ \leq 1.0 \rightarrow OK$$

Verification of the bottom part of the concrete element:

$$Verification = \frac{\sigma_{c,t}}{\frac{f_{ctk,0.05}}{\gamma_c}} \leq 1.0 \tag{4.81}$$

$$0 \ \leq 1.0 \rightarrow \ OK$$

The verification of the timber element is performed as follows:

Stresses in the timber element:

$$\sigma_2 = \frac{\gamma_2 \cdot E_2 \cdot a_{2,new} \cdot M_{Ed,new}}{EI_{eff,tot,new}} = 0.2997725500\ MPa \tag{4.82}$$

$$\sigma_{m,2} = \frac{0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new}}{EI_{eff,tot}} = 18.58070324\ MPa \tag{4.83}$$

Stresses on the top part of the timber element:

$$\sigma_{t,t} = -\sigma_2 - \sigma_{m,2} = -18.88047579\ MPa \tag{4.84}$$

Stresses on the bottom part of the timber element:

$$\sigma_{t,b} = -\sigma_2 + \sigma_{m,2} = -18.28093069\ MPa \tag{4.85}$$

Verification of the stresses in the timber element:

$$Verification = \frac{\sigma_{t,t}}{\frac{k_{mod} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{t,b}}{\frac{k_{mod} \cdot f_{m,k,t22}}{\gamma_M}} \tag{4.86}$$
$$-0.3720662106\ \leq 1.0 \rightarrow OK$$

Shear stresses on the timber element:

$$\tau_2 = \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} = 1.486952284\ MPa \tag{4.87}$$

Verification of the shear stress in the timber element:

$$Verification = \frac{\tau_2}{\frac{k_{mod} \cdot f_{v,k,t22}}{\gamma_M}} \qquad 4.88$$

$$0.5343734771 \ \leq 1.0 \rightarrow OK$$

The load per shear fastener is found using the equation:

$$F_1 = \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed} = 8.364263549 \ kN \qquad 4.89$$

Verification of the shear fastener:

$$Verification = \frac{F_1}{F_{Rd}} = \frac{F_1}{3 \cdot \frac{k_{mod} \cdot f_{tens,k}}{\gamma_M}} \qquad 4.90$$

$$0.2003938143 \ \leq 1.0 \rightarrow OK$$

4.4.2.3 Load capacity calculations using long-term verifications of the slab

This will have a similar calculation procedure as for the short-term verification calculations, but with some difference in the modulus of elasticity for both the concrete and timber element and slip modulus for the shear fasteners. As previously described in Chapter 3.3, this will consider the creep and shrinkage of the concrete and timber. The variable load is not considered and that is why these equations are neglected.

The modulus of elasticity for concrete:

$$E_1 = E_{1,g} = \frac{E_{cm}}{1 + \varphi_c} = 9714.285715 \ N/mm^2 \qquad 4.91$$

The modulus of elasticity for timber:

$$E_2 = E_{2,g} = \frac{E_{CLT}}{1 + k_{def}} = 4542.399998\ N/mm^2 \qquad 4.92$$

The slip modulus for the shear fastener:

$$K_{ser} = K_{ser,g} = \frac{K_{ser}}{1 + k_{def}} = 12486.48649\ N/mm^2 \qquad 4.93$$

For load capacity calculations based on ULS calculations, the slip modulus "K_u", considers 2/3 of the K_{ser} as follows:

$$K_u = \frac{2}{3} \cdot K_{ser} = 8324.324327\ N/mm^2 \qquad 4.94$$

Again, considering only effective compressive height of concrete is performed due to bottom part of the concrete does not satisfy the verification calculations. The performance of this calculations can be found in Appendix A, but the modified parameters are tabulated in Table 4-11 below.

Table 4-11 The modified total effective bending stiffness and adjusted parameters needed for calculations

Notation	**Value**
γ_1	0.06042753
γ_2	1.0
$a_{1,eff}$	111.632451 mm
$a_{2,new}$	1.6219025 mm
x	13.49124479 mm
$A_{1,eff}$	8094.806874 mm^2
$I_{1,eff}$	122782.2898 mm^4
$EI_{eff,tot,new}$	$4.537316051 \cdot 10^{11} Nmm^2$
$M_{Ed,new}$	45.02844547 kNm
$P_{Ed,new}$	117.6503846 kN

Similarly, as for the short-term verification, using the new modified values listed in Table 4-11, it is possible to verify the materials as follows:

Stresses in the concrete element:

$$\sigma_1 = \frac{\gamma_1 \cdot E_1 \cdot a_{1,eff} \cdot M_{Ed,new}}{EI_{eff,tot,new}} = 6.503154035\ MPa \qquad 4.95$$

$$\sigma_{m,1} = \frac{0.5 \cdot E_1 \cdot x \cdot M_{Ed,new}}{EI_{eff,tot,new}} = 6.503154036\ MPa \qquad 4.96$$

Stresses on the top part of the concrete element:

$$\sigma_{c,t} = -\sigma_1 - \sigma_{m,1} = -13.00630807\ MPa \qquad 4.97$$

Stresses on the bottom part of the concrete element:

$$\sigma_{c,b} = -\sigma_1 + \sigma_{m,1} = -1 \cdot 10^{-9}\ MPa \qquad 4.98$$

Verification of the top part of the concrete element:

$$Verification = \frac{\sigma_{c,t}}{\frac{f_{ck}}{\gamma_c}} \leq 1.0 \qquad 4.99$$

$$-0.5574132030 \ \leq 1.0 \rightarrow OK$$

Verification of the bottom part of the concrete element:

$$Verification = \frac{\sigma_{c,t}}{\frac{f_{ctk,0.05}}{\gamma_c}} \leq 1.0 \qquad 4.100$$

$$-6.81818181 \cdot 10^{-10} \ \leq 1.0 \rightarrow \ OK$$

The verification of the timber element is performed as follows:

Stresses in the timber element:

$$\sigma_2 = \frac{\gamma_2 \cdot E_2 \cdot a_{2,new} \cdot M_{Ed,new}}{EI_{eff,tot,new}} = 0.73113578\ MPa \tag{4.101}$$

$$\sigma_{m,2} = \frac{0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new}}{EI_{eff,tot}} = 27.04733922\ MPa \tag{4.102}$$

Stresses on the top part of the timber element:

$$\sigma_{t,t} = -\sigma_2 - \sigma_{m,2} = -27.778475\ MPa \tag{4.103}$$

Stresses on the bottom part of the timber element:

$$\sigma_{t,b} = -\sigma_2 + \sigma_{m,2} = 26.31620344 \tag{4.104}$$

Verification of the stresses in the timber element:

$$Verification = \frac{\sigma_{t,t}}{\frac{k_{mod} \cdot f_{t,0,k.t22}}{\gamma_M}} + \frac{\sigma_{t,b}}{\frac{k_{mod} \cdot f_{m,k,t22}}{\gamma_M}} \tag{4.105}$$

$$-0.574757942 \ \leq 1.0 \rightarrow OK$$

Shear stresses on the timber section:

$$\tau_2 = \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} = 2.23605769\ MPa \tag{4.106}$$

Verification of the shear stress in the timber element:

$$Verification = \frac{\tau_2}{\frac{k_{mod} \cdot f_{v,k,t22}}{\gamma_M}} \tag{4.107}$$

$$0.8035832032 \ \leq 1.0 \rightarrow OK$$

The load per shear fastener is found using the equation:

$$F_1 = \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed} = 20.6262215 \ kN \tag{4.108}$$

Verification of the shear fastener:

$$Verification = \frac{F_1}{F_{Rd}} = \frac{F_1}{3 \cdot \frac{k_{mod} \cdot f_{tens,k}}{\gamma_M}} \tag{4.109}$$

$$0.494251364 \ \leq 1.0 \rightarrow OK$$

4.4.3 Maximum deflection of slab type A based on SLS verifications

The SLS verification is previously described in Chapter 3.1.2. Similarly theoretical calculation as for maximum applied load calculations based on ULS, the maximum deflection based on SLS verification is found in the following chapters. In addition, the deflection is checked whether it satisfies the conditions for short-term and for long-term conditions.

For the following calculations there is no need to consider only the effective compressive height of the concrete. This means there will not be additional calculations to modify the effective bending stiffness. All maximum deflection calculations based on SLS can be found in Appendix B.

4.4.3.1 Maximum deflection calculations using short-term verifications of SLS

A similar procedure performed in chapter 4.4.2.1 using the shear analogy method to include the shear deformation in the transverse layers for the CLT to find the modified modulus of elasticity for the CLT is used.

The modulus of elasticity for concrete element:

$$E_1 = 34000\ Nmm^2 \quad 4.110$$

The modulus of elasticity for the 5-layered CLT element:

$$E_2 = 8403.44\ Nmm^2 \quad 4.111$$

The slip modulus for the shear fastener:

$$K_{ser} = 23100\ \ N/mm \quad 4.112$$

Since there is a change in value for the slip modulus for the shear fasteners, due to SLS, the required parameters used to find the effective bending stiffness will change. Table 4-12 shows the calculated value of the parameters, and the performance of the calculation can be found in Appendix B.

Table 4-12 The effective bending stiffness and adjusted parameters needed for short-term calculations

Notation	**Value**
γ_1	0.04851770613
γ_2	1.0
a_1	81.95598703 mm
a_2	8.044012970 mm
$EI_{eff,tot}$	$1.5312882410 \cdot 10^{12} Nmm^2$
P_{Ed}	80.04426452 kN
$f_{d,SLS}$	1.217734993 kN/m

Then the vertical deflection can be calculated as follows:

$$w = \frac{5 \cdot \left(\frac{P_{Ed}}{L} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot}} = 5.610726172\ mm \qquad 4.113$$

It is important to check whether the calculated deflection can satisfy the conditions for the short-term verification.

The limit for SLS short-term can be found:

$$w_{lim} = \frac{L}{250} = 8.0\ mm \qquad 4.114$$

Verification of vertical deflection:

$$Verification = \frac{w}{w_{\text{lim}}} = 0.7013407715\ mm \qquad 4.115$$

4.4.3.2 Maximum deflection calculations using long-term verifications of SLS

While it is normal to consider the variable loads for design calculations, these are neglected in this laboratory test. This will cause a more simplified formula for the long-term calculations.

A similar performance of calculations in comparison to Chapter 4.4.3.1 above, with exception of the creep and shrinkage is taken into consideration. This will have an effect on the modulus of elasticity of both timber and concrete, as well as the slip modulus for the shear fasteners. This is found by using equations from Chapter 3.3 Long-term verification as follows:

New modulus of elasticity for concrete:

$$E_{1,fin} = \frac{E_{cm}}{1 + \varphi_c} = 9714.285714\ N/mm^2 \qquad 4.116$$

New modulus of elasticity for CLT:

$$E_{2,fin} = \frac{E_{CLT}}{1 + k_{def}} = 4542.4\ N/mm^2 \qquad 4.117$$

New slip modulus for the shear fastener:

$$K_{ser,fin} = \frac{K_{ser}}{1 + k_{def}} = 12486.48649\ N/mm^2 \qquad 4.118$$

The necessary parameters to find the effective bending stiffness and then the deflection, will also be adjusted accordingly and tabulated in Table 4-13.

Table 4-13 The effective bending stiffness and adjusted parameters needed for long-term calculations

Notation	**Value**
γ_1	0.08798300593
γ_2	1.0
a_1	$82.2694404\ mm$
a_2	$7.739055957\ mm$
$EI_{eff,tot}$	$7.236339585 \cdot 10^{11} Nmm^2$
P_{Ed}	$122.3017601\ kN$
$f_{d,SLS}$	$1.217734993\ kN/m$

Then the vertical deflection can be calculated as follows:

$$w = \frac{5 \cdot \left(\frac{P_{Ed}}{L} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot}} = 17.91769486\ mm \qquad 4.119$$

It is important to check whether the calculated value can satisfy the conditions.

The limit for SLS short-term can be found:

$$w_{lim} = \frac{L}{150} = 13.33333333\ mm \tag{4.120}$$

Verification of vertical deflection:

$$Verification = \frac{w}{w_{\text{lim}}} = 1.343827115\ mm \rightarrow Not\ OK \tag{4.121}$$

4.5 Results of theoretical calculations for slab type B

The results from the different calculations are listed below in tables. A similar calculation procedure described in Chapters 4.4.2 and 4.4.3 for CTC screws are used for performing the theoretical calculations for the load capacity and maximum deflection for slabs using KOP screws. The performance of the complete calculations can be found in Appendix A and Appendix B.

4.5.1 Load capacity for slab type B based on ULS verification

The modified parameters used in the calculations are tabulated in Table 4-14 below.

Table 4-14 The effective bending stiffness and adjusted parameters needed for short- and long-term

Notation	Short-term	Long-term
γ_1	0.02671144241	0.04935924062
γ_2	1.0	1.0
$a_{1,eff}$	116.2447862 mm	113.2933254 mm
$a_{2,new}$	0.6501479 mm	1.1146021 mm
x	6.210131824 mm	11.18414502 mm
$A_{1,eff}$	3726.079094 mm^2	6710.487012 mm^2
$I_{1,eff}$	11974.91562 mm^4	69948.49480 mm^4
$EI_{eff,tot,new}$	$7.724473842 \cdot 10^{11} Nmm^2$	$4.348484069 \cdot 10^{11} Nmm^2$
M_{Ed}	32.02309981 kNm	44.23441749 kNm
P_{Ed}	82.96212951 kN	115.5229767 kN

The result of the maximum load calculations for both short-term and long-term based on the ULS verification can be found in Table 4-15.

Table 4-15 Maximum applied load calculations based on ULS verification for KOP screws, short- and long-term

KOP screws		**ULS, Short-term**	**ULS, Long-term**
Modified normal stresses in concrete cross section	$\frac{\sigma_{c,t}}{f_{c,d}}$	-0.3348508036	-0.4736519246
	$\frac{\sigma_{c,b}}{f_{c,d}}$	$-6.8181 \cdot 10^{-10}$	0
Normal stresses in CLT	$\frac{\sigma_2}{f_{t,0,2}} + \frac{\sigma_{m,2}}{f_{m,d}}$	-0.3624897309	-0.562776943
Shear stresses in CLT	$\frac{\tau_{2,max}}{f_{v,d}}$	0.5305823025	0.809885637
Shear fasteners	$\frac{F_1}{F_{RD}}$	0.09002693710	0.2320196660

4.5.2 Maximum deflection of slab type B based on SLS verification

The modified parameters used in the calculations are tabulated in Table 4-16 below.

Table 4-16 The effective bending stiffness and adjusted parameters needed for short- and long-term

Notation	**Short-term**	**Long-term**
γ_1	0.03953909054	0.07225561973
γ_2	1.0	1.0
a_1	$83.33434944\ mm$	$83.54511936\ mm$
a_2	$6.665650562\ mm$	$6.454880643\ mm$
$EI_{eff,tot}$	$1.456230493 \cdot 10^{12} Nmm^2$	$6.873754563 \cdot 10^{11} Nmm^2$
P_{Ed}	77.60657332 kN	119.742210 kN
$f_{d,SLS}$	$1.217734993\ kN/m$	$1.217734993\ kN/m$

The result of the maximum deflection calculations for both short-term and long-term based on the SLS verification can be found in Table 4-17.

Table 4-17 SLS verification of KOP screws short- and long-term

	SLS, Short-term	**Short-term**	**Long-term**
Vertical deflection	$w = \frac{5 \cdot \left(\frac{P_{Ed}}{L} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot}}$	$5.725544743\ mm$	$18.51482237\ mm$
Limitation Short-term	$w_{\lim} = \frac{L}{250}$	$8\ mm$	-
Limitation Long-term	$w_{\lim} = \frac{L}{150}$	-	$13.3333333\ mm$
Verification of the slab deflection	$\frac{w_{final}}{w_{lim}}$	0.7156930929	1.3886611678

5 Preparations for laboratory testing

5.1 Cross Laminated Timber

The CLT slab consists of 5 layers of timber boards packed in both directions. The CLT was a 5-layered element ready to use, where the glue used to join the lamellae is Melamine Urea Formaldehyde, MUF [26, p. 1]. The Figure 5-1 below shows the orientation for the cross section, where layer 1,3 and 5 is longitudinal lamella and layer 2 and 4 is transverse lamellae. The CLT slabs (approved by SINTEF) were provided by Splitkon, The CLT catalogue by Splitkon is found in Appendix N.

Figure 5-1 Cross section of 5-layered CLT used in laboratory testing

5.2 Assembling the shear fasteners

The two different types of shear fasteners were assembled a bit differently. For the spacing of the screws white chalk liner was a bit difficult to see on the wood due to the contrast. Instead, a cardboard for each screw types with the correct spacing between each row and the screw pairs was made and used see Figure 5-2.

The Figure 5-2 below shows how the spacing became as identical as it could be, when using the cardboard template. The difference for the two cardboard is the spacing between the rows and the pairs of screws, because the minimum distances appointed in the Eurocode 5 [22]. The effective length of the screws, that is fastened into the timber and the rest that will be covered by concrete is also determined in accordance with the Eurocode 5 [22].

Figure 5-2 Picture of cardboard template, to the Left CTC screw template and to the Right KOP screw template

5.2.1 CTC screws – Slab type A

The CTC screw has a diameter of 7 mm and a length of 160 mm. This screw type is self-tapping screw, which means, it can be screwed onto the timber slab without predrilling holes. Instead, the location of each screw was marked by using the cardboard a predrilling a 2-3 mm hole, to make it easier installing the screws. A jig was used, to get the 45° angle of the screws, see Figure 5-3.

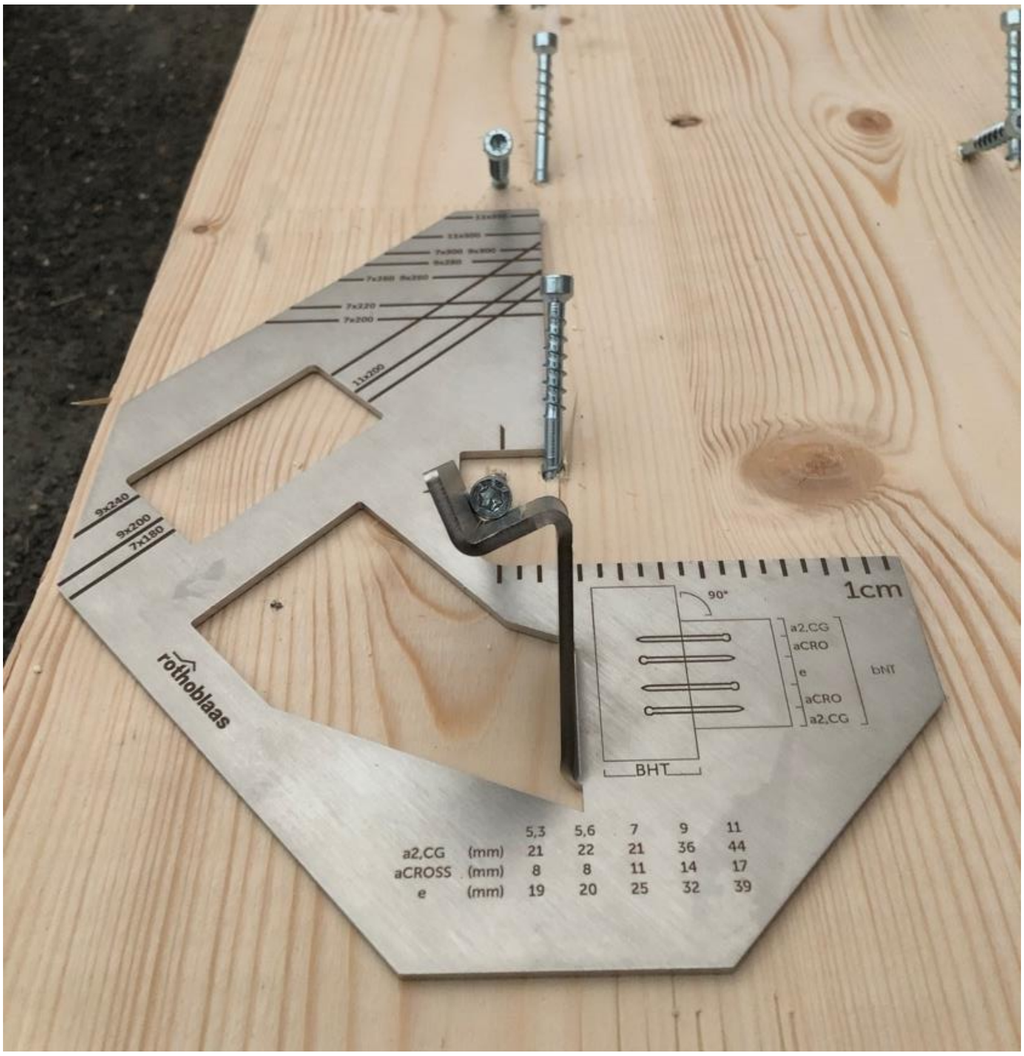


Figure 5-3 The jig used to get a 45° angle on the screws

The pairs of the CTC screws were arranged crossed parallel, shown in Figure 5-4.

Figure 5-4 A slab with CTC screws - crossed parallel with a 45º angle

In addition, a steel reinforcing mesh with the size of 150 x150 mm and diameter of 5 mm, was used to increase the strength in concrete. The steel reinforcement mesh was delivered in the size of 8 by 5 meters, and a bolt cutter was used to get the wanted length and width. The Figure 5-5 below shows the bolt cutter used and the steel reinforcement mesh.

Figure 5-5 Bolt cutter and the steel reinforcement mesh

Figure 5-6 below shows the slab with CTC screws and the steel reinforcement mesh with the spacing and numbers of screws have been determined earlier in Chapter 4.3.1.2.

Figure 5-6 A slab with CTC screws and reinforcement mesh

5.2.2 KOP screws – Slab type B

The KOP screw has a diameter of 10 mm and a total length of 140 mm. A similar procedure as for the CTC screws was used to mark the location for each screw. The KOP screw is not self-tapping, and it is therefore necessary to predrill holes with a 6 mm wood drill bit. It was necessary to pre-drill to avoid the wood drill bit get stuck. For this reason, the KOP screws are more time-consuming to assemble. The KOP screws are placed with a 45º angle parallel to each other, by use of the jig.

The Figure 5-7 shows the how to pre-drill the holes with the 6 mm wood drill bit and the jig.

Figure 5-7 Pre-drill of a hole with a 6 mm wood drill bit by using the jig

Since it is necessary to use the jig to get the 45º angle for the screws, it was necessary to pre-drill in two steps. First the location and angle were created by using the jig. The next step was to make the wanted length of hole in CLT. To prevent making the hole to deep, deeper than the actual length of the screws, a stop collar was mounted on the wood drill bit at the specified length. Figure 5-8 shows the drill with the stop collar and Figure 5-9 shows how the stop collar is preventing of drilling a hole to deep.

Figure 5-8 Drill with wood drill bit and with a mounted stop collar

Figure 5-9 Wood drill bit with stop collar in actio

The Figure 5-10 below shows how the KOP screws were fastened into the CLT.

Figure 5-10 Fastening the KOP screws into the CLT slab

Figure 5-11 below shows a CLT slab with KOP screws that are oriented parallel with a 45^o angle and the spacing and numbers of rows have been determined earlier in Chapter 4.3.2.2.

Figure 5-11 A slab with KOP screws parallel oriented with a 45^o angle

There was one remaining CLT slab and enough KOP screws in spare. This was used to make an additional slab, but the screws were placed in a similar manner as the slabs with CTC screws, crossed parallel with a 45º angle. Figure 5-12 below for the orientation of the screw types.

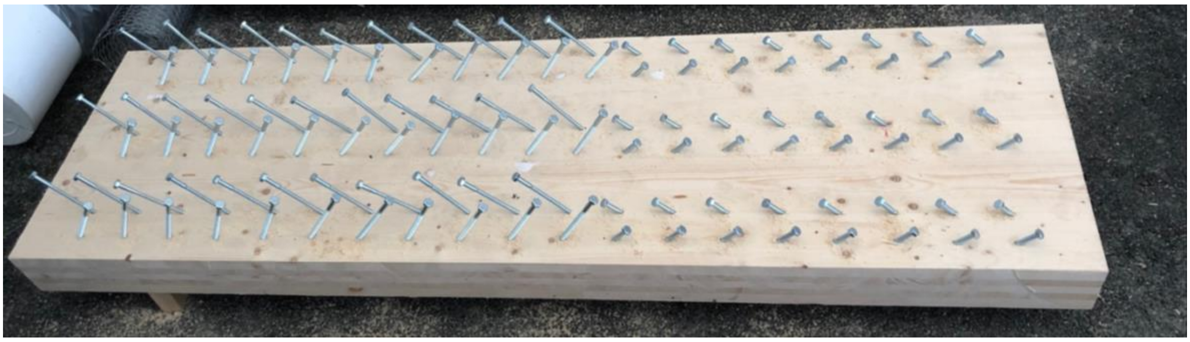


Figure 5-12 An almost ready slab with KOP screws cross-parallel oriented with a 45º angle

5.3 Formwork

Preparation of formwork was required before pouring the concrete onto the CLT slabs with the installed screws. A chop saw was used to cut the necessary lengths for the formwork. Plywood with a thickness of 16 mm was used. Since the height of concrete only was 60 mm, this was a sufficient thickness to withstand the lateral pressure occurring when the concrete was poured onto the slab. Figure 5-13 shows the chop saw and the plywood used.

Figure 5-13 Chop saw and the 16 mm plywood used for the formwork

The formwork was surrounding the CLT slab, so that the concrete would get the same width and length as the CLT slab. The formwork, two with the exact length of the slab and two with the length of the slab width plus additional 50 mm, making it easier to demoulding after the concrete was casted.

The height from the top of the CLT slab is 60 mm, making it easier to level the concrete when the concrete is poured inside the formwork. In the chance the formwork to bulge out or leak, it was used a lot of screws, approximately with spacing of 10 mm, to ensure the formwork would withstand the pressure from the pouring of concrete.

Figure 5-14 shows the two slabs with the different shear fastener and the surrounding formwork.

Figure 5-14 Slabs with different shear fastener and formwork surrounding the slab

The left slab in Figure 5-14 above, also shows the steel reinforcement mesh placed on some plastic blocks, lifting the formwork up from the top of the CLT slab.

5.4 Pouring the concrete

Mixing the concrete at the laboratory at UiS alone, would be very time consuming, because all the materials need had to be weighed separately and the mixing machine at the University is very small for the quantity needed, two cubic meters. Then it had to be mixed twenty continuous blends. Hence, of the quantity and equipment, the concrete was provided by Sola Betong. The recipe can be found in Table 5-1 and more information of the concrete is attached in the Appendix G.

Table 5-1 Material composition of concrete B35, SCC

Material	**Density [kg/m^3]**	**Quantity [kg/m^3]**	**Volume [litre m^3]**
Coarse aggregate 8-16 mm	2640.00	625.052	236.762
0.8 mm sand	2640.00	833.403	315.683
0.2 mm fine sand	2670.00	280.958	105.228
Silica	2200.00	13.310	6.050
FA from Turkey	2300.00	24.201	10.522
Standard FA	3000.00	365.835	121.945
Cold water	1000.00	179.182	179.182
Warm water	1000.00	0.00	0.00
Air	1000.00	0.403	0.403
Superplasticiser	1050.00	4.437	4.226
Air entrainment in concrete 2.0% Vol %			20.00
		2326.780	1000.00

After the slabs were ready with the surrounding formwork installed, the concrete could now be poured onto the top part of the CLT wood with the shear fasteners installed. To fill all the slabs and make some concrete cubes and cylinders testing the quality for concrete, two cubic meters of concrete is needed. Figure 5-15 below shows how the concrete was poured onto the slabs.

Figure 5-15 Pouring concrete onto the CLT slabs, both types are shown

After the slabs were poured with the concrete, even though it is self-compacting concrete it is important to get the small air pockets out of the mixture using a poke rod into the mix and a hammer on the sides of the formwork. If not performing the vibration honeycombing, voids near the surface could occur. The Figure 5-16 below shows how this was performed.

Even though it is self-compacting concrete it was important to get rid of the small air pockets in the concrete mixture. This was done by using the poker rod and/or hammer vibrating on the sides of the formwork as shown the Figure 5-16. Otherwise, honeycombs, voids near the surface and etc may affect the strength of the slab.

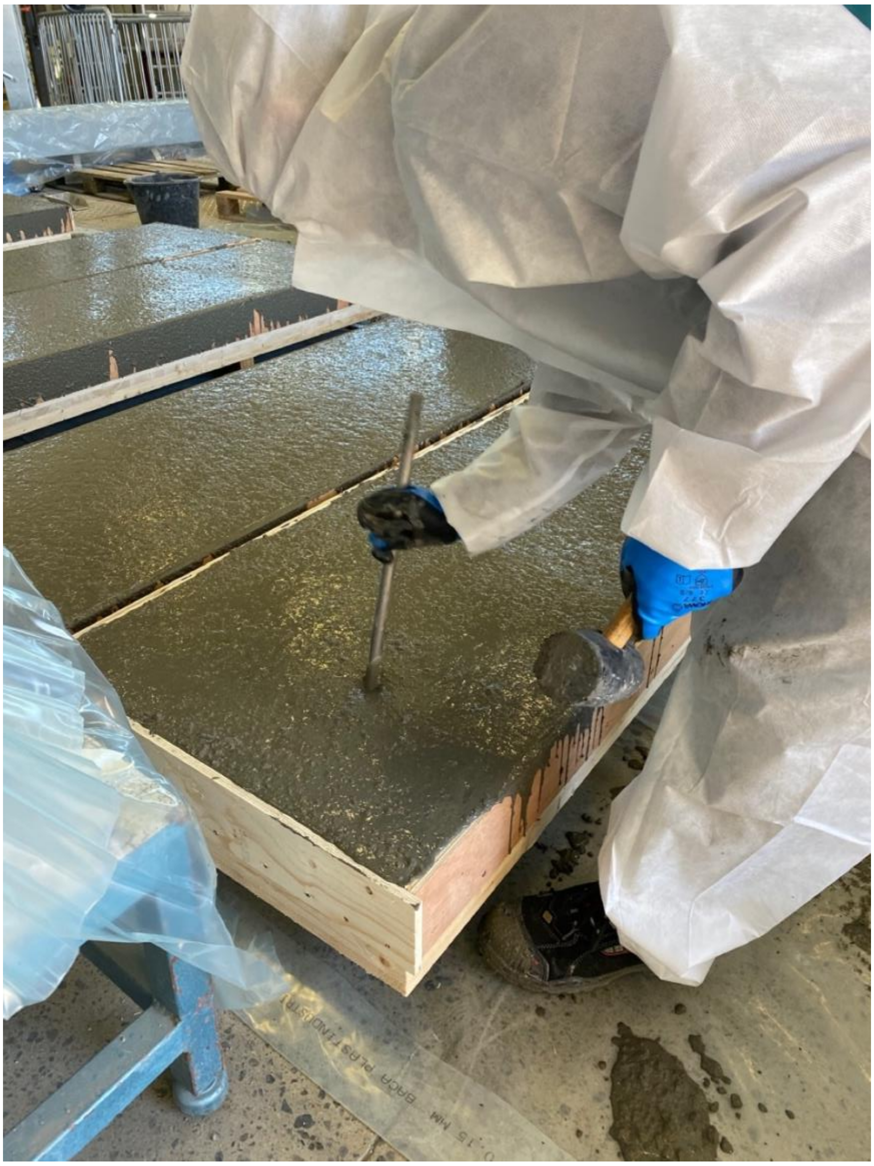


Figure 5-16 Vibration of concrete using rod and hammer

After the vibration had been performed, it was time to level the concrete. Making the top part of the concrete smooth and levelled. Figure 5-17 below demonstrates how the levelling was executed.

Figure 5-17 Making the concrete smooth and levelled

Figure 5-18 below shows some of the finished casted concrete, ready for curing.

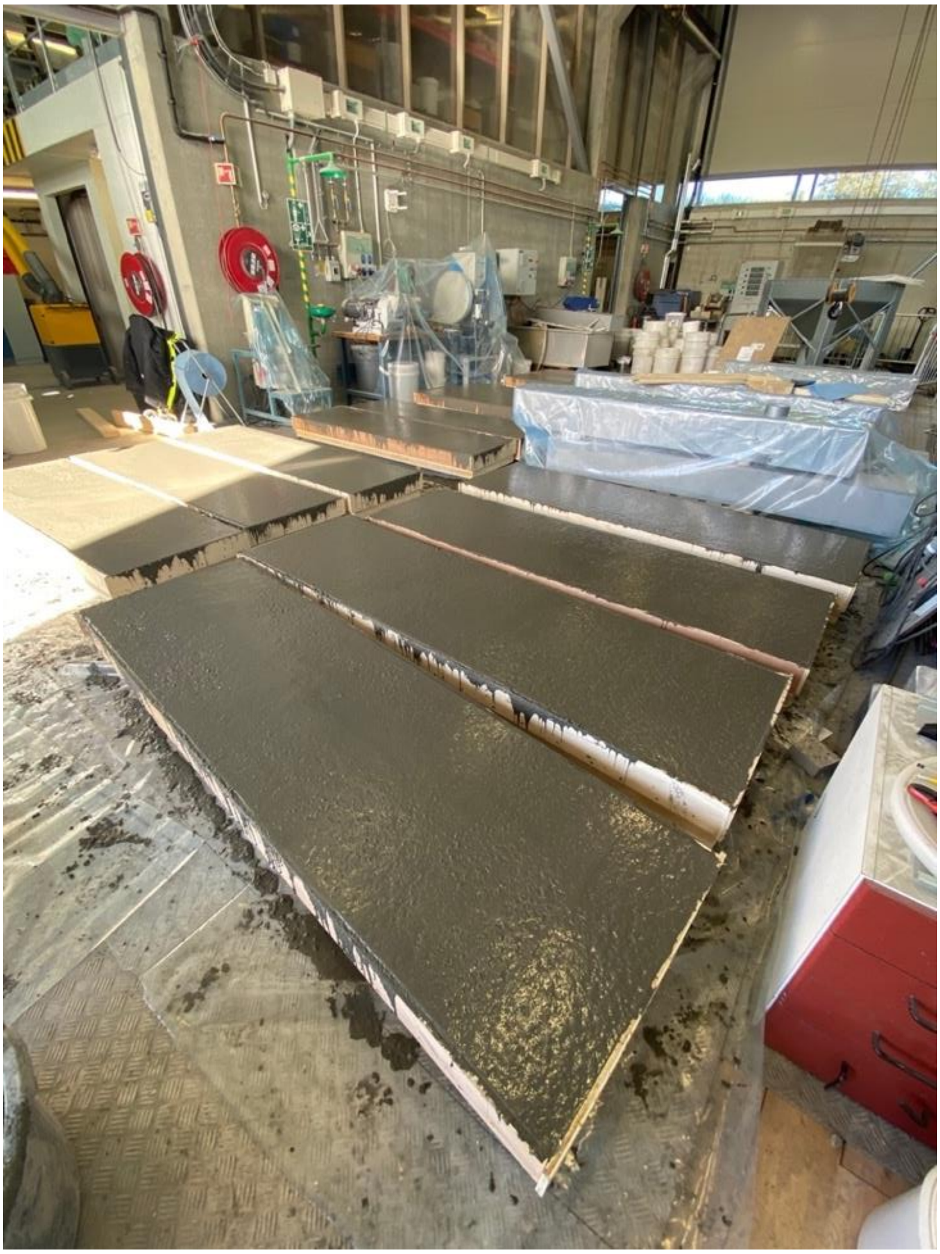


Figure 5-18 Finished casted concrete

5.5 Curing

Curing is an important process for the quality of the properties for the concrete. The usage of impermeable plastic cover is used covering the elements, to inhibit the concrete of drying out, see Figure 5-19.

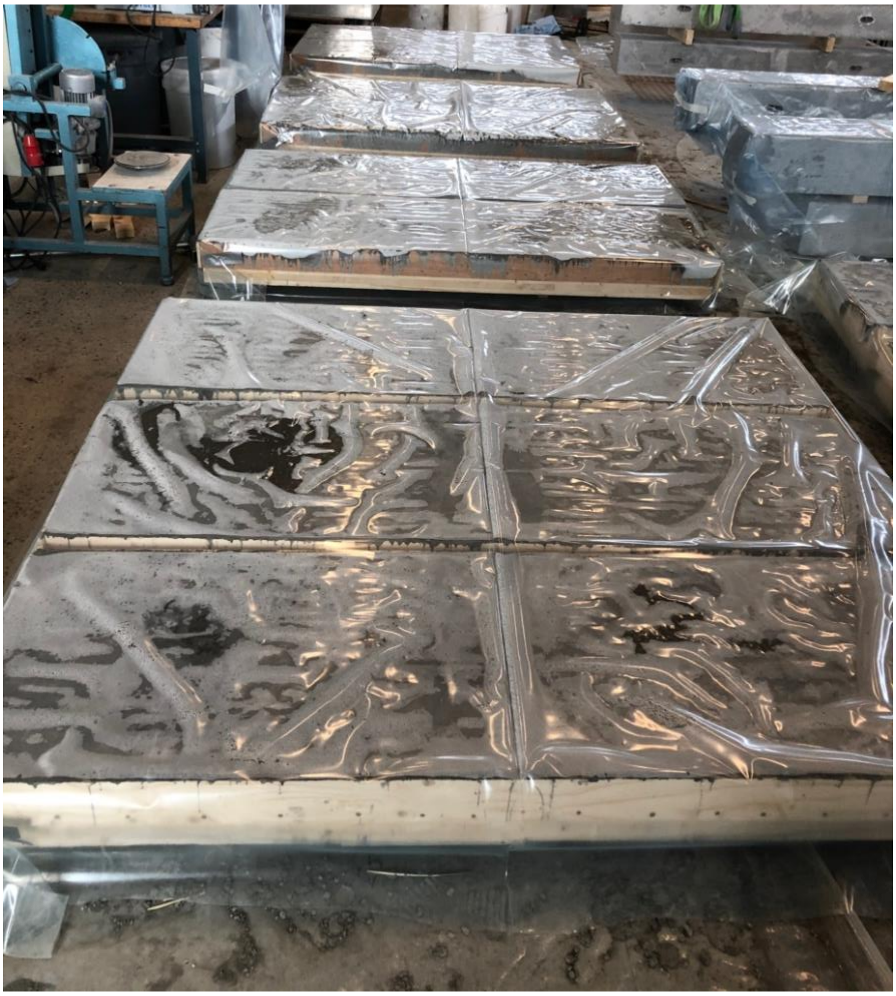


Figure 5-19 Plastic covering the CLT-concrete composite slabs for curing

In total, there were made twelve cubes and six cylinders. The reason for making the concrete cubes and cylinders was to verify that the strength class of the concrete used in the laboratory testing for the CLT-concrete slabs, satisfied the values from the strength class in Eurocode 2 [21]. Figure 5-20 below shows the cubes and cylinders poured into the formworks where they were covered in plastic the first 24 hours according to the curing procedure. After this, they were placed into water tanks for the remaining 27 days of the curing process.

Figure 5-20 Concrete cubes and cylinders covered in plastic - The first 24 hours of curing procedure

5.6 Demoulding and making ready for testing

After the CLT-concrete slabs had cured for 4 days, the formwork was removed and the covered in plastic again, see Figure 5-21.

Figure 5-21 Curing of the CLT-concrete composite slabs using plastic covers after removing the formwork

After 28 days of curing and it was time to perform the four-point bending test on the CLT-concrete composite slabs.

6 Laboratory test

6.1 Quality check of the concrete

The concrete samples, 12 cubes and 6 cylinders, were ready to be tested after 28 days of casting the concrete. Since it is not possible to test all the 13 TCC slabs in on day. It was decided to spread the concrete cube and cylinder test elements in flow with the testing of the TCC slabs.

The test days of the cubes and cylinders and TCC where performed the following days after casting the concrete:

- 28 days; 3 cubes and 2 cylinders
- 34 days; 3 cubes
- 38 days; 3 cubes and 2 cylinders
- 40 days; 3 cubes and 2 cylinders

6.1.1 Compressive strength test of concrete cubes

The compressive test of concrete cubes is tested in accordance with NS-EN 206,2013 [16] as previously described in Chapter 2.7.1.1. The concrete cube, with dimensions $100x100x100\ mm^3$, was placed into the machine and could now be tested and is loaded until failure, see Figure 6-1 below.

Figure 6-1 Toni-Technik 3000 kN compressive strength test

6.1.2 Modulus of elasticity test of concrete cylinders

The modulus of elasticity tests of the cylinders have been performed in accordance with NS-EN 12390-13:2013 [17]. First the top and bottom part of the concrete cylinder, with diameter of $150\ m$ and length of $300\ mm$, must be smooth, this performed by using a cutting machine. See Figure 6-2 below.

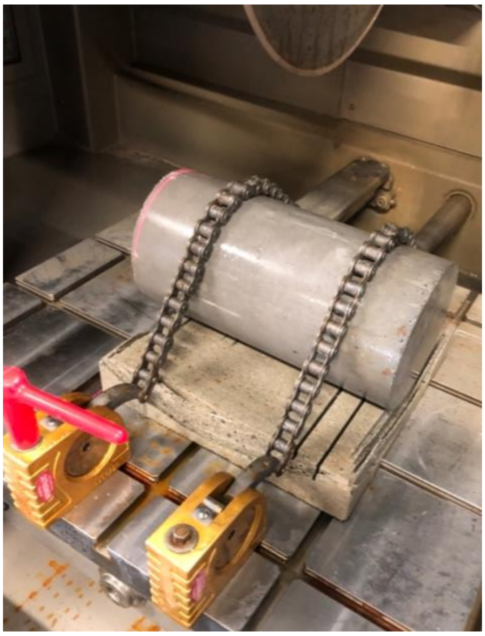


Figure 6-2 Cutting machine Brilliant 285

Each test group, 28, 38 and 40 days after casting the concrete, is dependent on the average of the result of the compressive test of the three cubes performed. The values for the upper stress, lower stress and preload are taken from the compressive test of the concrete cubes and tabulated in the Table 6-1 below. Figure 6-3 shows how the test specimen is placed into the test machine with deformation measurement device that can measure the modulus of elasticity.

Table 6-1 Values for the modulus of elasticity test

		Values [kN]
Test day 28 days	Upper	14.54
	Lower	5.45
	preload	2
Test day 38 days	Upper	15.09
	Lower	5.66
	preload	2
Test day 40 days	Upper	15.64
	Lower	5.86
	preload	2

Figure 6-3 Modulus of elasticity testing of a cylinder with deformation measurement device

6.1.3 Splitting tensile test of cylinders

The splitting tensile test of concrete cylinders is performed in accordance with NS-EN 12390-6:2001 [18]. The splitting tensile test is conducted after the modulus of elasticity test because the failure is determined when the test specimen is cut in halves and then it is not possible to perform any other test for this laboratory testing. See the Figure 6-4 below, for a cylinder placed into the testing machine.

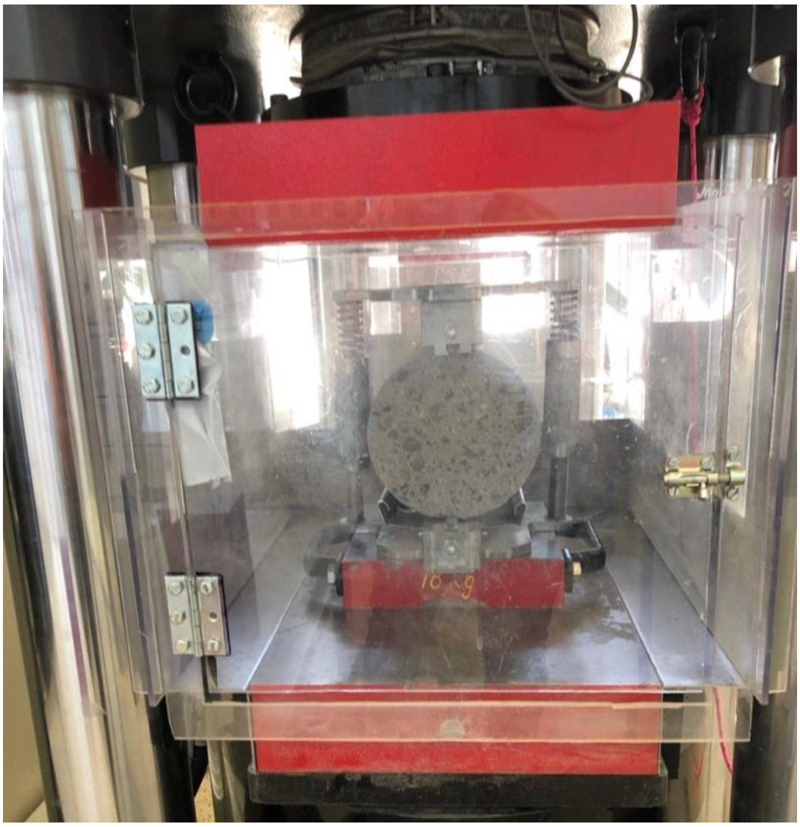


Figure 6-4 Splitting tensile test of a concrete cylinder

6.2 Preparation of the CLT-concrete composite slabs for testing

6.2.1 Four-point bending test

After the concrete in the CLT-concrete composite have cured for 28 days it was time to perform the laboratory testing. It took some additional days after 28 days of curing, to setup the format of test machine correctly and in accordance with NS-ISO 6891:1991: "Timber structures, Joints made with mechanical fasteners" [19]. The delay was due to the fact that this type of tests had not been performed at the University laboratory before. Hence, some trial and error of the setup of the four-point bending test machine occurred before getting it correct.

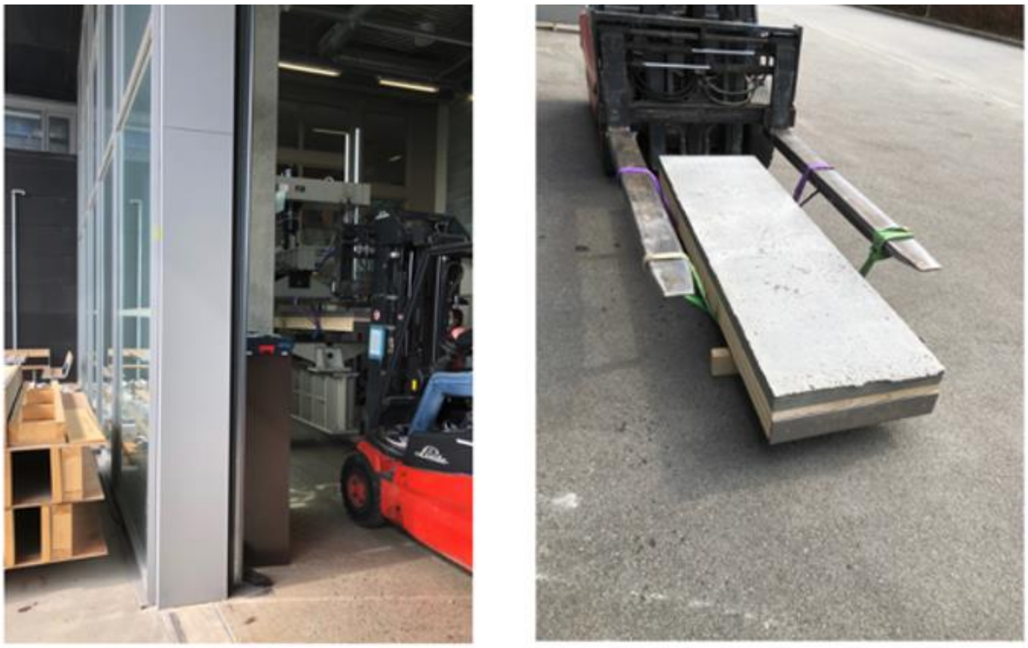
The setup for the applied load rate and duration of testing was in accordance with NS-ISO 6891:1991 “Timber structures, Joints made with mechanical fasteners” [19]. The numerical values used in the setup for the four-point bending test is presented for both slabs in Table 6-2 below.

Table 6-2 Input for slab type A and B for four-point bending test

Input	Slab A	Slab B
Cycles	1	1
Cycle speed	0.2 kN/s	0.2 kN/s
Upper step	52 kN	48 kN
Overall time upper cycle	30 s	30 s
Cycle speed	0.2 kN/s	0.2 kN/s
Upper step	13 kN	12 kN
Overall time upper cycle	30 s	30 s

6.2.2 Transportation and arrangement of the CLT-concrete composite slabs

The CLT-concrete composite slab weighed approximately 300 kg and had to be transported/moved using a forklift, see Figure 6-5 a) and b).



a) b)

Figure 6-5 a) and b) Transporting the CLT-concrete composite for testing

Before transporting it and placing the test elements onto the test machine, the two ends of bottom part of the slab had to be grinded. Two “L” shaped stainless steels profiles were installed on the short sides of the CLT-concrete composite slab. These steel profiles were used to flatten the supports preventing “crushing” of the timber when testing. Figure 6-6 below demonstrates this.

Figure 6-6 “L” shaped steel profile placed on the support, grey line marking the placement for support

6.2.3 Setup of LVDT for measuring the displacements

In addition to measure the ultimate failure load, it was also measured deflection at the midspan and under one of the applied loads. Transverse displacement on both sides of the slab, two measuring the concrete and one measuring the timber was also performed using 5 off Linear Variable Differential Transformer (LVDT). Arranging the LVDT’s are tabulated in Table 6-3 and shown in several figures below.

Table 6-3 Arrangement the LVDT's

Number	Location
1	The load cell
2	Lateral displacement on concrete, R
3	Lateral displacement on timber, L
4	Vertical displacement midpoint
5	Vertical displacement under load
6	Lateral displacement on concrete, L

Figure 6-7 shows the location of LVDT number 3, 4, 5 and 6 and the load cell, number 1.

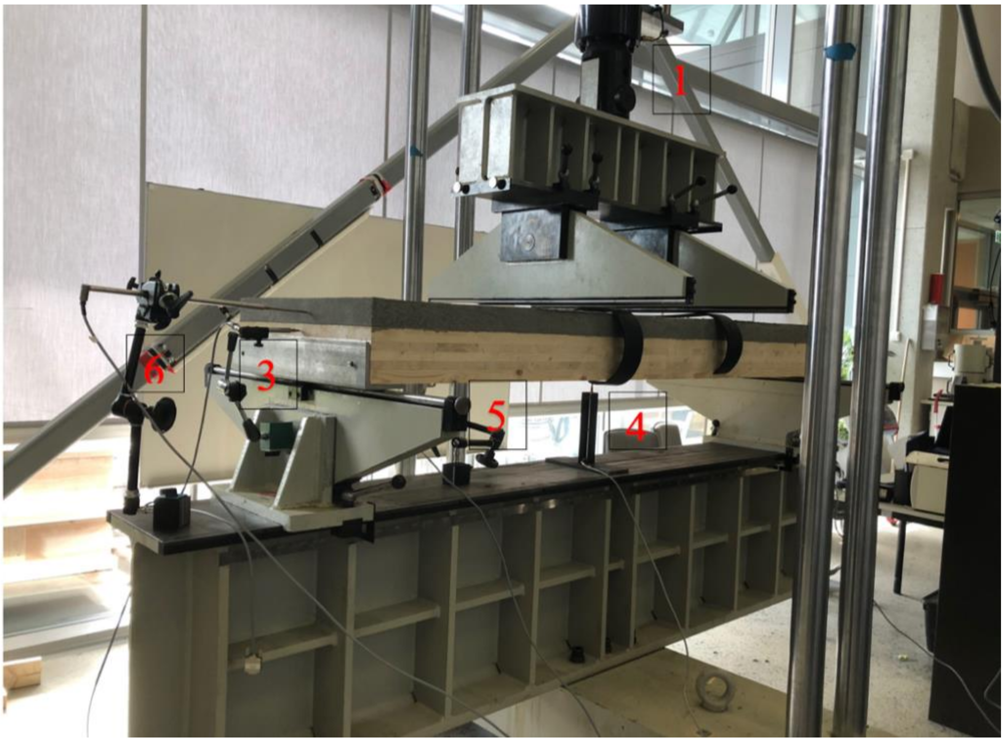


Figure 6-7 The setup for the test, LVDT's: 3, 4, 5 and 6 and load cell: 1

Figure 6-8 shows the location of LVDT number 2.

Figure 6-8 The setup for the test, LVDT 2

Figure 6-9 shows the LVDT located at midspan (5) and under the applied load (4).

Figure 6-9 The set-up for the test, LVDT's: 4 and 5 up-close

In addition to the LVDT's, in the Figure 6-8 above, a ruler/measuring tape is attached onto the test slab, to examine if it was possible to see some movements in the elements. These rulers/measuring tapes were placed near the support on both sides and at the middle, one on timber part and the other on the concrete part.

6.3 Performing the test

After the CLT-concrete composite has been placed onto the four-point-bending machine, there was placed two rubber pads on top of the top part of the concrete. This was to prevent the crushing of the concrete when load was applied. The applied load was then distributed over a bit larger area. The figures below show how this was done. The thin rubber pads in Figure 6-10 got destroyed after three test and were therefore replaced with a thicker one for the remaining ten elements, see Figure 6-11.

Figure 6-10 Thin rubber pad and placement of ruler

Figure 6-11 Thick rubber pad

After finished the test setup, it was time to perform the four-point bending test for the CLT-concrete composite slabs. Figure 6-12 below shows the test element ready for testing.

Figure 6-12 The CLT-Concrete composite slab ready for testing

7 Laboratory test results

7.1 Verification of concrete properties

It was important to verify the quality of the concrete before performing the laboratory tests. The results for compressive test of the concrete cubes are listed in the Table 7-1 below. The average values are then multiplied with 0.8 to convert the result from cubes into cylinders.

Table 7-1 Results from compressive test of concrete cubes

Days after casting concrete	Test Specimen	Test result [kN]	Average for cubes [kN]	Average for cylinder [kN]
28 days	1	54.55	55.527	43.61233
	2	55.3		
	3	53.7		
34 days	1	56.77	57.487	45.98933
	2	57.61		
	3	58.08		
38 days	1	56.86	56.59	45.272
	2	57.45		
	3	55.46		
40 days	1	57.78	58.673	46.938
	2	59.1		
	3	59.14		

The results from modulus of elasticity testing and the splitting tensile test are tabulated in Table 7-2 below.

Table 7-2 Results from modulus of elasticity and splitting-tensile test for concrete cylinders

Days after casting concrete	**Test Specimen**	**E-modulus result [GPa]**	**Splitting tensile test result f_{ct} [MPa]**
28 days	1	30.898	3.1217
	2	35.510	2.5866
38 days	1	41.863	3.4653
	2	37.924	3.1123
40 days	1	37.781	3.69423
	2	28.234	2.8328

7.2 The result of testing slabs of type A

The result from the four-point bending test for slabs of type A are listed in Table 7-3 below. The tabulated values show the different failure drops and the corresponding midspan vertical deflection.

Table 7-3 Four-point bending test for slabs of type A

		Slab A1	**Slab A2**	**Slab A3**	**Slab A4**	**Slab A5**	**Slab A6**
1st drop	Load [kN]	107.1628	128.3043	124.2517	120.7266	144.6898	130.1665
	Deflection [mm]	6.159733	7.67309	6.858697	6.997531	8.58584	8.803105
2nd drop	Load [kN]	-	138.2558	133.6174	124.3703	-	148.4118
	Deflection [mm]	-	8.987783	8.073234	8.239138	-	11.70356
3rd drop	Load [kN]	-	210.8493	-	-	-	-
	Deflection [mm]	-	20.65447	-	-	-	-
Max	Load [kN]	226.599	229.7485	245.9679	207.9605	171.6296	188.1178
	Deflection [mm]	25.03494	28.80238	26.01309	26.14769	15.85718	24.276

Testing of Slab A1, there were some troubles with the testing machine. The length of the piston (i.w. reach of the hydraulic jack/actuator) was not installed properly for the height of the test specimens. First, it seemed that the problem was the values for limitations in the setup, as previously described in Chapter 6.2.1, but after the Slab A1 had same failure without the limitations, it was discovered that the distance of the applied load had to be lowered, because the maximum length of the piston used. The height of the machine was then adjusted, so the deflection and applied load could be monitored properly. This discovery was found during the testing of the third CLT-concrete composite slab. Therefore, the result of Slab A1 is listed in an additional Table 7-4.

Table 7-4 Four-point bending test for Slab A1

		Slab A1	**Slab A1_max1**	**Slab A1_max2**
1st drop	Load [kN]	107.1628	-	-
	Deflection [mm]	6.159733	-	-
Max	Load [kN]	226.599	217.2007	240.1312
	Deflection [mm]	25.034	23.94908	29.92149

7.3 The result of testing slabs of type B

The result from the four-point bending test for slabs of type B are listed in Table 7-5 below. The tabulated values show the different failure drops and the corresponding midspan vertical deflection.

Table 7-5 Four-point bending test for slabs of type B

		Slab B1	**Slab B2**	**Slab B3**	**Slab B4**	**Slab B5**	**Slab B6**
Only drop	Load [kN]	72.40523	82.29506	85.95696	96.48657	86.87943	104.0848
	Deflection [mm]	5.037414	5.578972	5.729669	6.591282	4.760207	5.432587
Max	Load [kN]	191.8759	144.1898	191.4263	181.1175	211.7753	239.9337
	Deflection [mm]	26.9146	13.02592	22.60949	23.201448	22.69049	23.20135

Since, the rubber pads were not placed in the middle of the applied load, it was skewed. As a result, after applied the load of 144.1819 kN the rubber was squeezed out and the test stopped automatically. Hence, it was tested twice. Both performances of the testing of Slab B2 are listed in Table 7-6 below.

Table 7-6 Four-point bending test for Slab B2

		Slab B2	**Slab B2_2**
1st drop	Load [kN]	82.29506	-
	Deflection [mm]	5.578972	-
Max	Load [kN]	144.1898	163.731
	Deflection [mm]	13.02592	19.23195

7.4 The result of testing Slab C1

The result from the four-point bending test for slab type C1 is listed in Table 7-7 below. The tabulated values show the different failure drops and the corresponding midspan vertical deflection. For this Slab C1, the KOP screws, were oriented in pairs crossed parallel with a 45º angle.

Table 7-7 Four-point bending test for Slab C1

		Slab C1
1st drop	Load [kN]	105.7241
	Deflection [mm]	5.882049
2nd drop	Load [kN]	105.729
	Deflection [mm]	5.882327
Max	Load [kN]	242.9766
	Deflection [mm]	21.53667

7.5 Graphical representation of four-point bending test results

7.5.1 Load vs vertical deflection at midspan

From the four-point bending test it was possible to determine the different failure drops in addition to the maximum failure.

The following two figures, Figure 7-1 and Figure 7-2, shows the load-deflection response for slabs of type A and type B respectively.

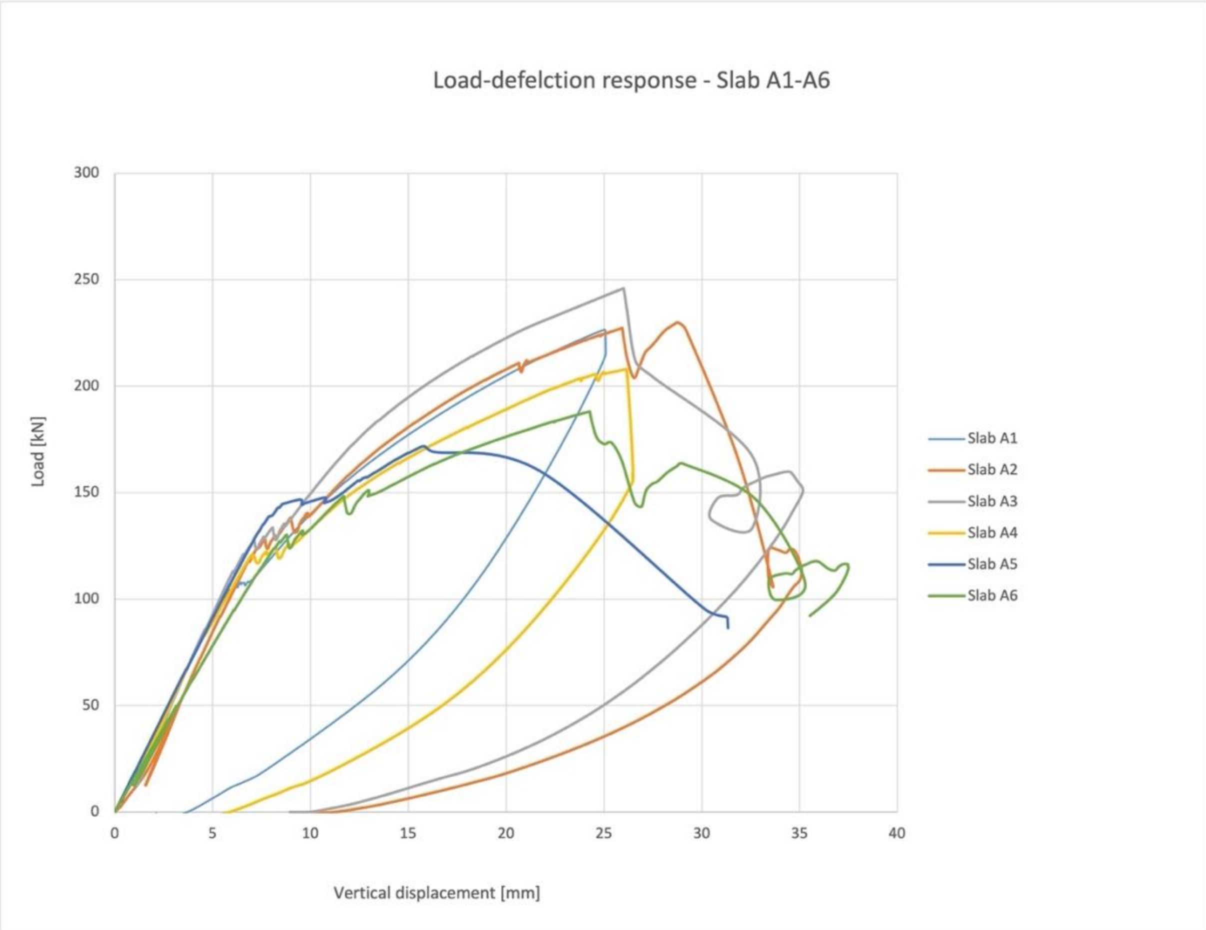


Figure 7-1 Load-deflection response for Slab A1-A6

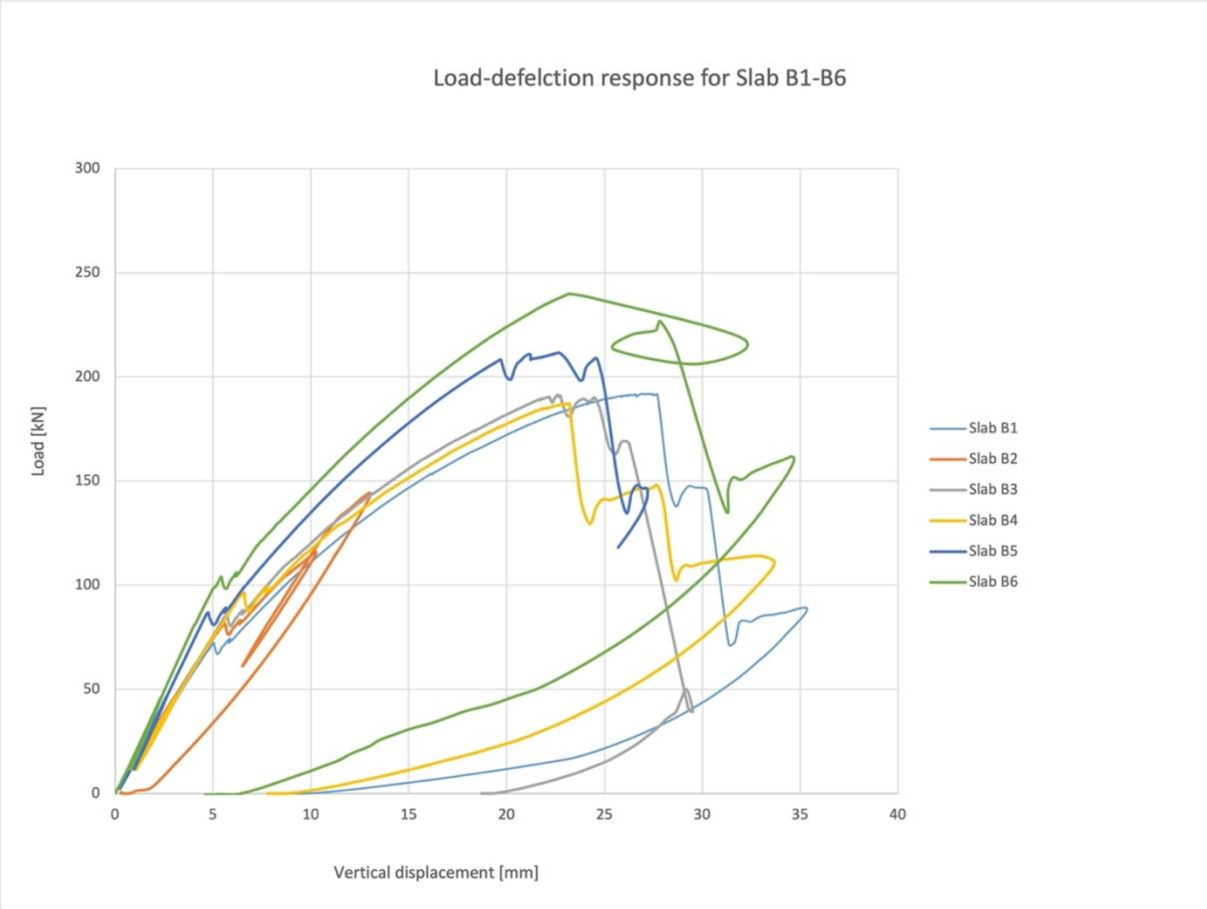


Figure 7-2 Load-deflection response for Slab B1-B6

From the two figures above, it is possible to see that there is a linearity until the first failure drop. After this first failure drop, there is a nonlinearity of load-deflection behaviour, in the global stiffness.

Figure 7-3 below shows the load-deflection response for Slab A2. This figure shows both the two vertical deflections located at mid-span and right below the applied load, and that there is a slightly difference in the location of the load. However, they react simultaneously for the failures.

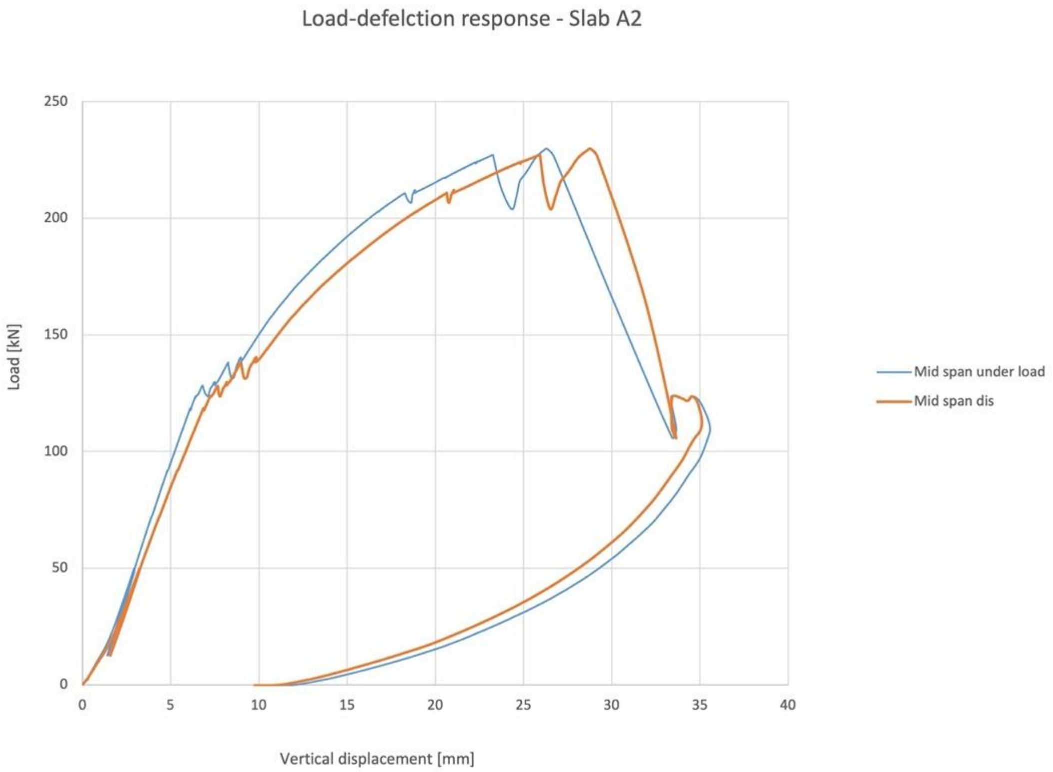


Figure 7-3 Load-deflection response for Slab A2

A similar load vertical deflection response for slab type B and Slab C1 are demonstrated below in figures. It is possible to see the similar behaviour of the response for vertical deflection of the LVDT's located at midspan and right below the applied load. Figure 7-4 demonstrates the result for Slab B6 and Figure 7-5 demonstrates for Slab C1. In Appendix H, the response of the load vertical deflection for all test specimens can be found.

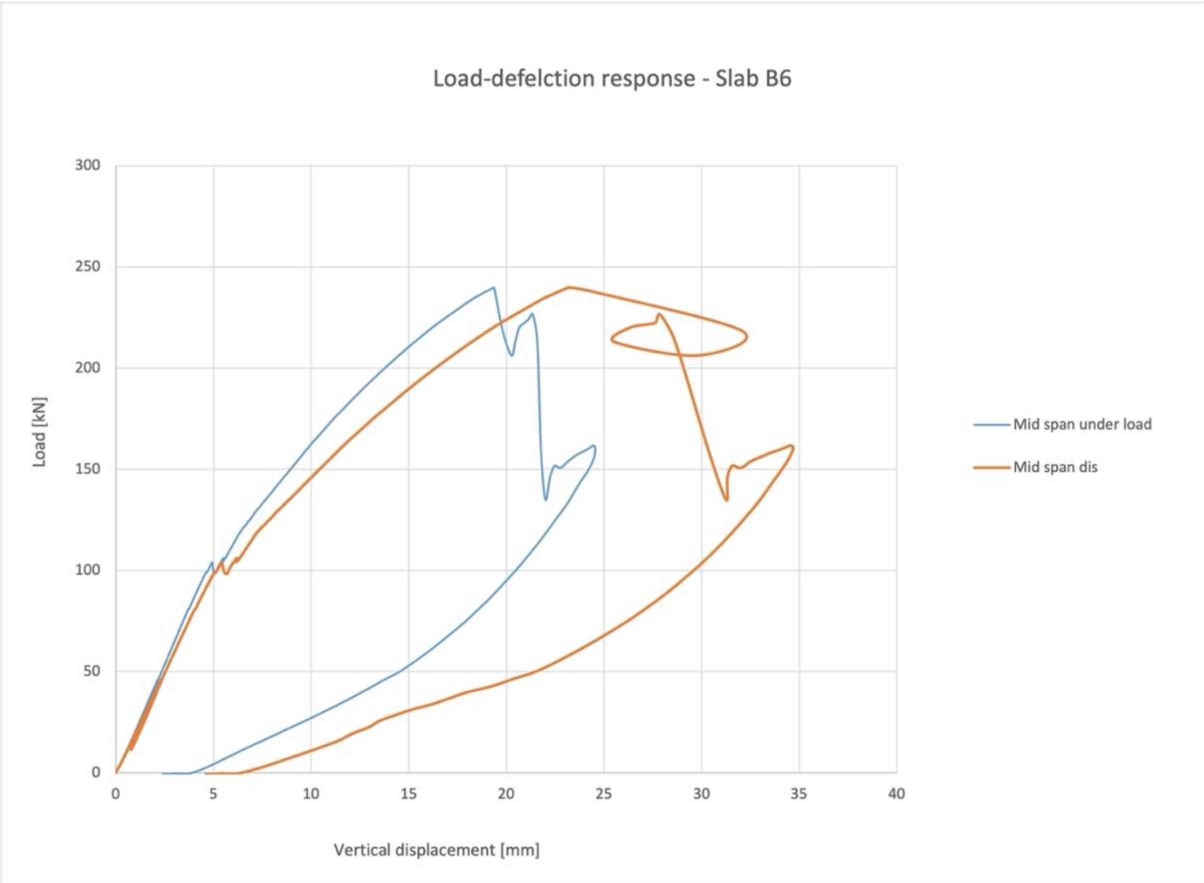


Figure 7-4 Load-deflection response for Slab B6

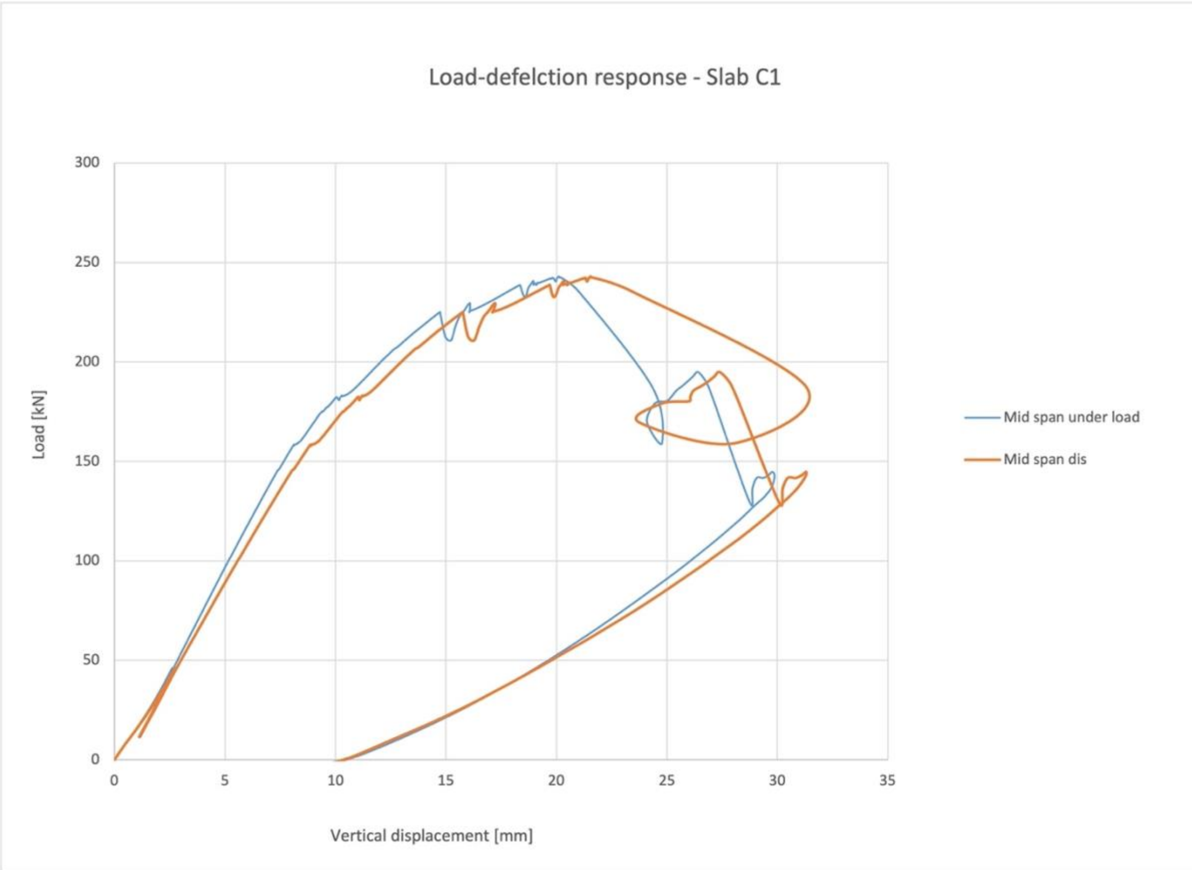


Figure 7-5 Load-deflection response for Slab C1

From the Figures 7-1 through 7-5 above, the load-displacement behaviour of the slabs shows the linear variation until first load drop point which is mostly due to the premature failure of the slabs and/or due to the interlayer slip. Then after, it behaves nonlinearly until reaching the maximum load where the collapse/fracture of the slabs are obtained.

7.5.2 Load vs lateral displacements

LVDT's were placed on the short sides of the CLT-concrete element to measure the lateral displacements. The Figure 7-6 below demonstrates the movements of the concrete and timber elements of Slab A1. It also shows the movements and behaviour of the slab when the load is applied. In Appendix I, all the load vs lateral displacements response can be found, for all slabs.

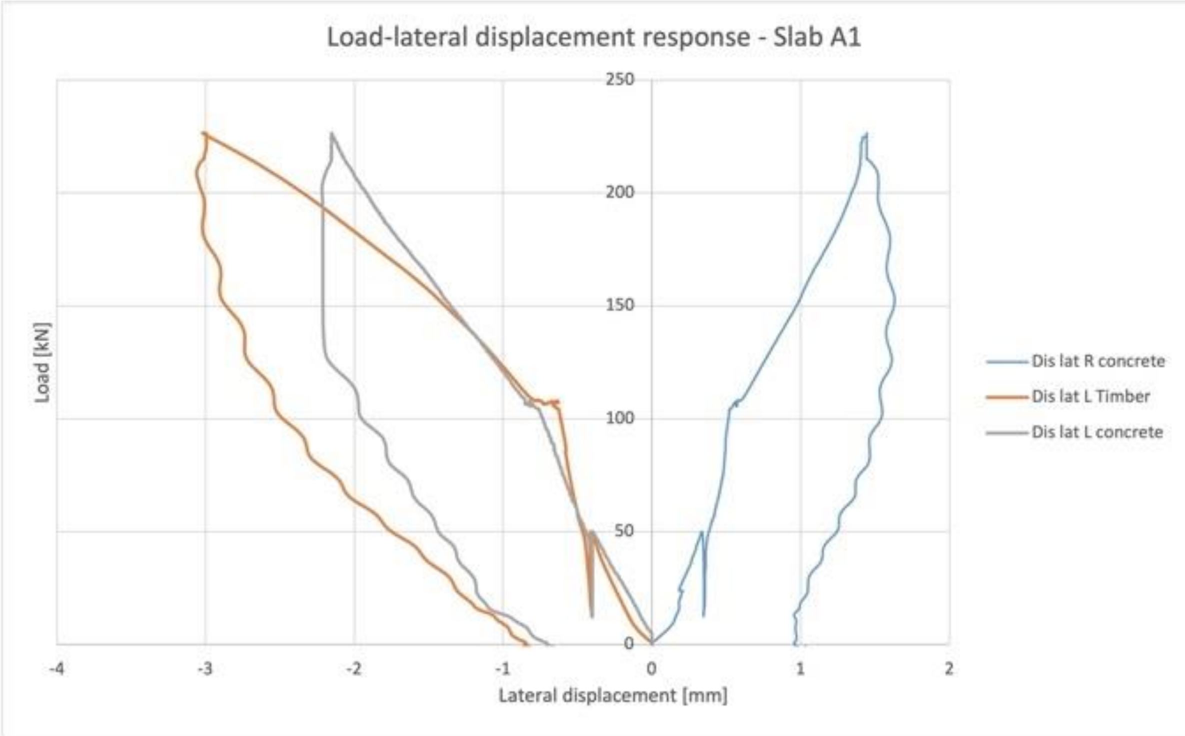


Figure 7-6 Load vs lateral displacement response for slab A1

7.6 Observation of slip between materials

The figures below show the movements/separation of the concrete and timber elements. It also shows that one element is moving in one direction and the other element in the opposite direction. The Figure 7-7 and 7-8 demonstrates the movement for Slab A4 at edge and midspan respectively.

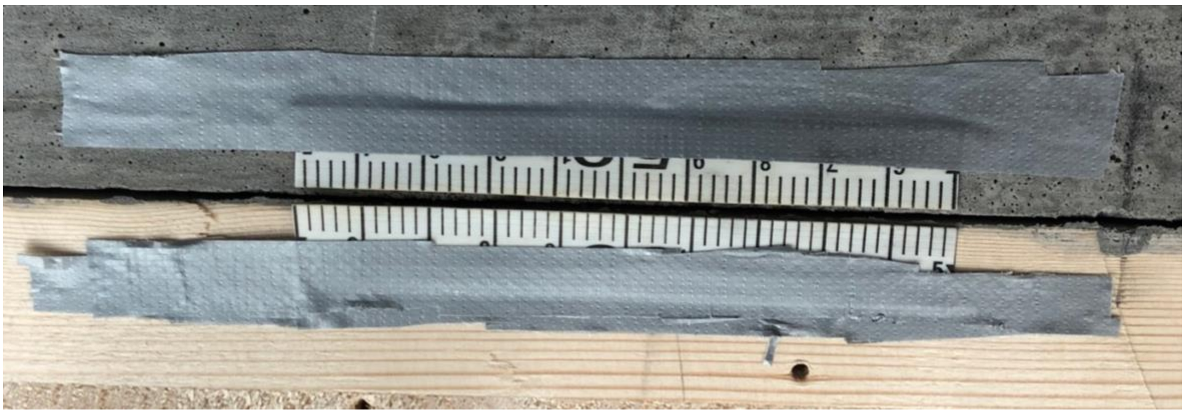


Figure 7-7 Movement in concrete and timber for Slab A4 at edge

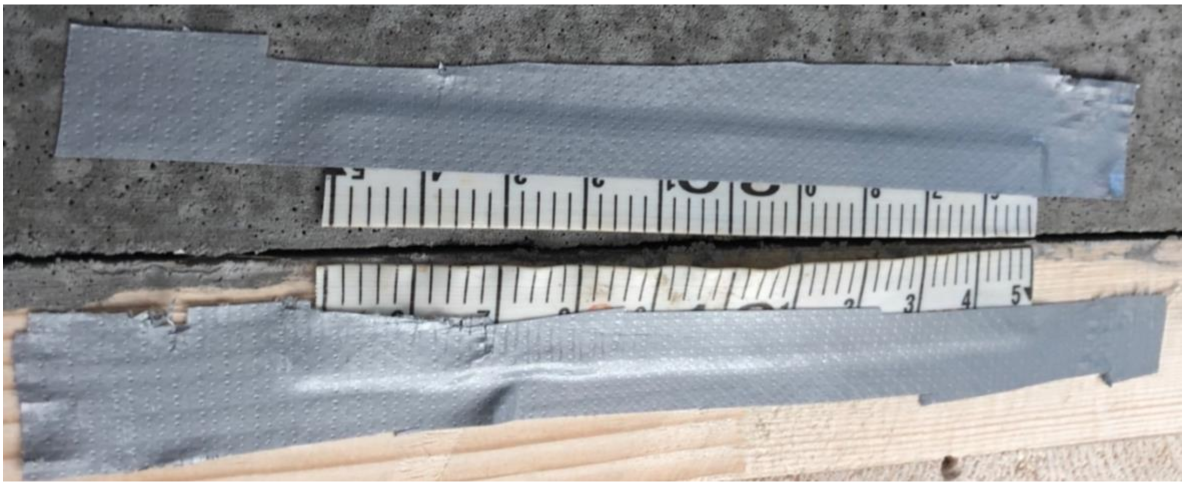


Figure 7-8 Movement in concrete and timber for Slab A4 at midspan

Figure 7-9 shows the movement for Slab B5 at edge.

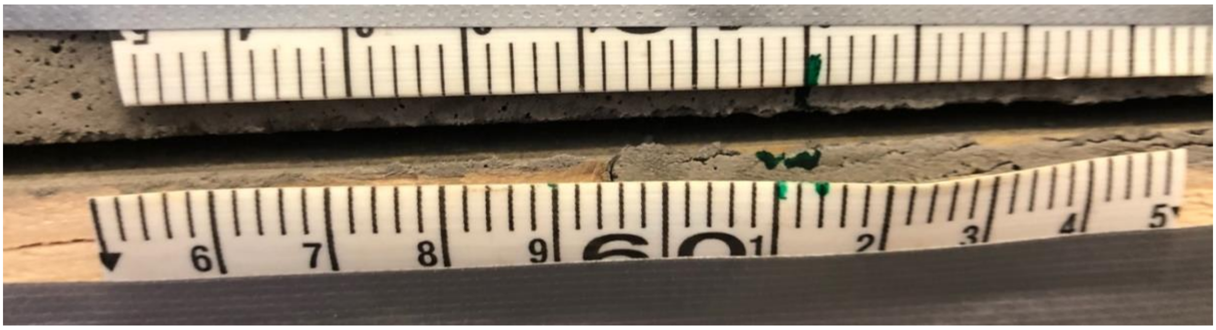


Figure 7-9 Movement in concrete and timber for Slab B5 at edge

8 Comparison and discussion of results

Since timber is an anisotropic material and none of the timber elements are completely identical due to the orientation and location of the finger joints, knots and other flaws and defects that will influence the mechanical behaviour. The figures of typical failures are presented the first Chapter 8.1. Hence, the extent of limitations affecting the results are discussed in Chapter 8.2.

The following chapters will focus on the comparison and discussion of the results from the theoretical predictions and the laboratory testing.

8.1 Typical failures

During the four-point bending test, cracking sound inside the wood, could be heard from the moment of the first failure drop until the maximum load capacity was reached. Then a distinct cracking sound could be heard. The figures shown in this chapter are taken after the maximum failure load had been reached.

In addition, the types of failures and more close-up pictures of failures to the different test elements and types of shear fasteners are included in Appendix K.

The first two figures, Figure 8-1 and Figure 8-2, are taken approximately 2 seconds apart, to demonstrate how and how quickly the failure occurs, for Slab A6. The screenshots of the loading within the 2 seconds of Slab A6 is found in Appendix K. This rate of failure is similar for all other tested elements.

Figure 8-1 2 seconds before failure of slab A6

Figure 8-2 Failure of slab A6

Performing the laboratory testing, the behaviour for the failure was not equal in all test elements. Some slabs, it was possible to see the crack in timber, either at midspan or at the edges. For some of the slabs it was not possible to see the cracks on the bottom side of the timber elements. This was the case for both type of shear fasteners.

In some of the slabs there were no visual failure underneath the timber, others the failure were visible in the fracture of the wood in the knots and finger joints, and when the knots were located right next to the finger joint. In most of the test there were cracks in the concrete in the direction and position of the applied load.

In Slab C1, there were also cracks in the longitudinal direction of the concrete, but this type of failure was not seen in slab type B (same type of shear fasteners). Further, the most common errors are presented in this chapter, and pictures of the different slab and location of failure can be found in appendix K.

The Figures 8-3 and 8-4 below shows a typical occurrence of a knot close to the finger joint failure.

Figure 8-3 Finger joint and knot failure in the timber

Figure 8-4 Failure in just finger joints and knot separately

Also, the failure in most of the slabs tested, had also cracks in the timber in the longitudinal direction. This type of crack in timber, was located at the midspan, see Figure 8-5.

Figure 8-5 Crack in timber at midspan

For some of the slabs tested, similar type of cracks occurred. The difference was that crack originated right below the applied load in the top part of the concrete and continued until the edge of the slabs in the timber part. This can be seen in Figure 8-6.

Figure 8-6 Crack in concrete and timber, side view

Further examination on the behaviour of the different shear fasteners after the laboratory testing were performed to see whether the shear fasteners were the reason of failure and therefore not withstand the applied loading. Figure 8-7 shows the position of the shear fastener CTC, after the four-point bending test.

Figure 8-7 CTC screws after four-point bending test

Figure 8-8 shows the KOP screw after testing and that the screw is embedded inside the timber and concrete and have not moved during testing.

Figure 8-8 KOP screw after four-point bending test

For the two slabs disassembled, A2 and C1, the screws looked very embedded into both the concrete and timber. Because of this and lack of time until deadline, only Slab A2 with CTC screws were dismantled entirely, and no other slabs were disassembled. Both KOP and CTC screws located at the midspan for Slab A2 and Slab C1were locked at. Since the screws located at midspan looked identical to the screws at the edge, and due to time sensitivity of this thesis, no further examinations were performed. Some more pictures of Slab A2 and Slab C1 can be found in Appendix K.

8.2 Limitations

There were many different types of limitations in this thesis and some will have a greater influence on the results and the corresponding conclusions than others. However, the limitations that have occurred will be discussed in the following chapter and later discussed how it affects the different results.

8.2.1 Limitations of provided material

For this laboratory testing there are some limitations to the material provided. These have an effect on both the theoretical predictions and the result of the laboratory testing, together with the comparison and the measured deficiencies.

First it was discovered that four out of thirteen slabs had shortage of length. To get a fair testing, it was decided to distribute two of the shorter elements to type A, one to type B and the last one to type C. The Table 8-1 below shows the exact length of these slabs.

Then, to compensate this lack in length it was decided to add a 16 mm plywood plank on each short side of the slab, before assembling the “L” shaped steel profile that prevent crushing the wood during testing.

Table 8-1 The slabs with shorter length

Slab	**Length**
A2	2,068 m
A3	2,067 m
B5	2,069 m
C1	2,068 m

At the day of casting, it was discovered during the middle of the casting process that the concrete mix contained a lot of plastic fibres. These plastic fibres were not described in the purchase order nor in the recipe for the concrete. This recipe can be found in Appendix G.

When the person delivering the concrete, was asked about these plastic fibres, the person responded with that it most likely was residue from the previous delivery. The extent of thie influence this have on the laboratory testing and results is uncertain, since the compressive strength was much greater than the strength class require.

After performing all the laboratory testing of the CLT-concrete composite slabs, it was discovered that the arrangement of the lamellae was wrong for five new slabs. The elements with lamellae 2 and 4, with height 30 mm are: A6, B1, B2, B3 and B4. Figure 8-9 demonstrates the arrangement of the CLT lamellae oriented in wrong directions compared to the Figure 8-10 showing the correct orientation of the CLT.

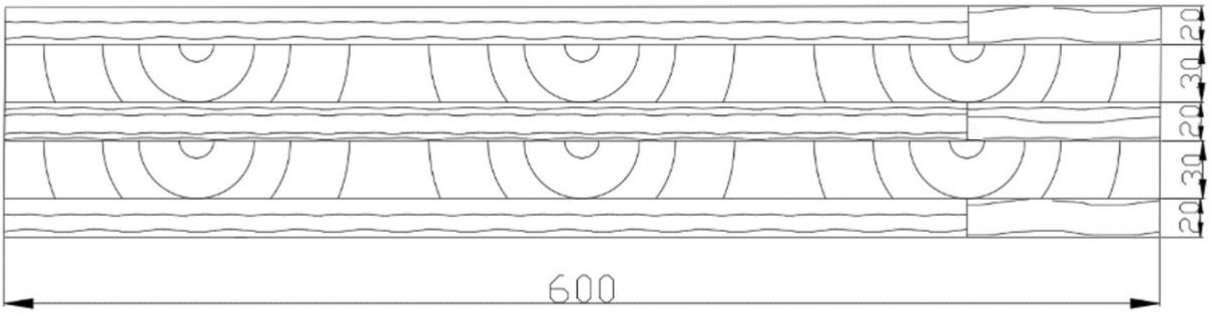


Figure 8-9 The incorrect orientation of the CLT

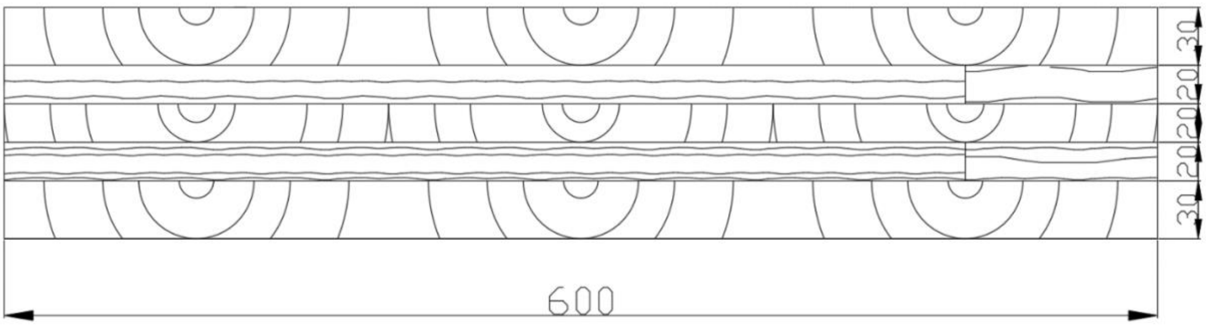


Figure 8-10 The correct orientation CLT

In a correct oriented CLT slab, the outer lamellae, layer 1 and 5, have a different material property than the inner lamellae, layers 2, 3 and 4. In addition the height of the outer lamellae is higher for the outermost lamellae. When the lamellae are oriented incorrectly, a change in both material properties and height of the longitudinal layers occurs. In addition, an increase in the transverse layers in the longitudinal direction occurs. This orientation shown in Figure 8-9 is not mentioned anywhere in the technical brochure provided by Splitkon [26], found in Appendix N.

When the two outer lamellae 1 and 5, with strength class T22, are reduced in height and at the same time lamellae 2 and 4, still being the transverse layer and having a lower strength class, T15, increased in height, the theoretical predictions and load capacity in test elements are affected.

Furthermore, as previously described in Chapter 2.3, wood in general is an anisotropic material and in addition to this, it has flaws and defects, such as knots, that influences the strengthening properties of the elements. The arrangement and location of these knots with respect to where the load is applied will have an effect on the failure load, as well as the behaviour of the failure. Another observation from the laboratory testing is that the finger joints also is the location of failure, but should in theory have a higher strengthening property.

Some, of the slabs contained holes where it was possible to see through the whole element. The Figure 8-11 (a) below demonstrates this flaw, which is marked with a circle and “X”. In additions some small, predrilled holes in one of the slabs were made because, the failure in length was not yet discovered, when placing the screws, see Figure 8-11 (b) x.

(a) (b)

Figure 8-11 (a) shows the hole through whole the CLT slab (b) shows the wrongly predrilled hole near a finger joint

The moisture content of the CLT were not measured for any of the slabs before or after casting the concrete. This could have been interested to see, if there were a change in moisture content after the concrete was casted, e.g., if some of the timber had soaked up some of the water from the concrete during curing.

The Figure 8-12 shows an element after casting and there can be seen a definite small gap between the concrete and CLT. Still, it is the shear fastener embedded inside that determines the degree of composite behaviour and it is not certainly that this gap do have any effect on this.

Figure 8-12 Minor gap between the concrete and timber elements before testing

Since it is the shear fasteners that is the main reason for the degree of composite action for the CLT-concrete composite slab, some limitations when assembling the slab together might have an effect on this behaviour. It was used a steel jig when mounting the shear fasteners, and when using the drill it sometimes slipped while fastening the screws, this may have led to that not all of the screws have an angle of 45^o or 135^o angle. Also, the distance between the shear fasteners and the spacing is not completely identical.

8.2.2 Limitations to laboratory testing of concrete verifications

For the concrete verification, compressive strength test, modulus of elasticity test and splitting tensile test. For the compressive strength test the limitations mainly relates to the placing of the test specimen correctly. If it is not placed centrically the load will not be applied at the midpoint of the test element and there will be some divergences to the results.

For the modulus of elasticity test there have been some more discrepancies to the equipment and input in the test setup. Values from concrete testing is not all satisfying in accordance with Table 3.1 in Eurocode 2 [21, Table 3.1], the modulus of elasticity test, < 34 GPa. There have been some errors with the equipment at the University laboratory for measuring the modulus of the elasticity. For this reason, the values for the theoretical predictions the values are taken from Table 3.1 in Eurocode 2 [21, Table 3.1].

Limitations for the splitting tensile test is location of the test specimen into the device and when this device is placed into the test machine. It is important to place the concrete cylinder centrically, otherwise some divergences on the result may occur.

8.2.3 Limitations to laboratory testing of four-point bending test

For the four-point bending test there are also some limitations that could affect the result. The first Slab A5, was tested with two cyclic loading, before following the correct procedure in accordance with [19].

Since this test method have not been conducted at the University before, there were some uncertainties related to the setup of the Toni Technik machine, and trial and error method was used to find the correct setup.

In the beginning the piston length, for the Toni Technik machine, were not installed for this height of test elements. Previously there have mainly been tested concrete beams with height of 300 mm. Therefore, it was not thought of and the reason for the piston length not being altered in the start of testing.

The first slab and second slabs tested, Slab A4 and Slab A5, it seemed like the test went uncomplicatedly, Slab A1 did not go to maximum failure. So, for the second try, the limitation in setup was corrected, but it would still not go to maximum failure. Then it was discovered that it had to be the piston length that was the problem, resulting in that the top part of the machine was lowered, and now the test could be performed correctly.

The test machine used in similar four-point bending test, found in previous research, the support pinned and roller were flat plates. However, the supports in machine at the University had circular shape. The “L” shaped steel profiles were assembled into the CLT material with screws and were loosened when the CLT-concrete composite element was placed onto the test machine. These steel profile inhibits the applied force from crushing the wood, due to the smooth surface the test elements could easily “slip”, since there were no grip on the round steel support.

Limitations to the other equipment used, is that one of the LVDT’s in the beginning had “duct tape”, surrounding the “LVDT number 5”, preventing from measuring vertical deflection for the LVDT located under the applied load. This particular LVDT, also had some problems with the spring and was sometimes stuck and could not measure correctly.

The rubber pads placed on top of the concrete inhibiting the concrete to crush when the load was continuously applied may also affect the results. Because the maximum load applied, made some “dents” in the rubber pads, these “dents” would increase for every slab tested and be different from the first to the last slab. This may have some influential, but still small, on the result of testing. In addition, these rubber pads were as close to identical location every time, but some e divergences have occurred.

The setup of the distances of support and the distance from the applied load was done manually, and there were some differences in millimetres in the lengths on each side.

8.2.4 Limitations to the theoretical predictions

For the theoretical predictions it is assumed the grains in the longitudinal and transverse lamellae were the correct way of orientation. This influenced the effect of the behaviour when the load was gradually applied. It will influence the calculations for efficiency of composite material when the theoretical predictions are based correct way of orientation and same length on all CLT elements, and because that the incorrect orientations of the lamellae will influence the result in the laboratory testing. This was the case for four out of six elements for slabs of type B and similarly for one out of six elements for slab of type A.

Further, the theoretical predictions performed in the [25, Ch. 7], were the basis of the load calculations in this Master thesis. When satisfying the theoretical predictions, the bottom part of the concrete is not satisfactory. Using the method adjusting the effective bending stiffness due to considering only the effective compressed height of the concrete form [6, p. 134], have been used to modify the effective bending stiffness and then performed the verification calculations once more. The question is whether these modifications calculations are possible to do for a CLT slab, when the [6, p. 134] has a different TCC structure, using timber panels, and concrete slab.

Another modification approach for the calculations for adjusting the NA and effective bending stiffness were found, and due to limited time and test period, this method was assumed to be applicable for the CLT slab used in this thesis.

The calculation for the chosen spacing have also been challenging finding the spacing for the shear fastener, since there are no standards and/or guidelines for this type of TCC element. Some also have used variation in length, where the spacing is smaller at the edges of the, ¼ of the length on the edges and then the spacing is larger at the middle, ½ of the length at the middle part of the element. Then the effective spacing is calculated in accordance with [22, clause 9.1.3(3)].

8.2.5 Limitations to the interpretation of results

When performing the laboratory testing of the CLT-concrete composite element, it has been demanding deciding what values to work with when it comes to the result of failure. The slabs of type A, with CTC shear fasteners and mesh reinforcement, have a first failure drop, some slabs also have second drops in failures before the maximum failure occurs. When this maximum failure occurred the whole slab just broke within milliseconds and made a distinct cracking sound. Hence, these maximum loads were therefore not taken as the failure load for in calculations, instead the results from the first drop were chosen. It was very difficult to determine this because, the timber material is never completely identical.

Location of the knots and other flaws in the material relative to the applied load also affects the results from laboratory testing.

Also, the input of the load procedure following the NS-ISO 6891:1991, [19], is based on the theoretical predictions and the uncertainties following these results, there could also be some uncertainties to the input before start of testing. Like, how would the result from the laboratory testing become if the theoretical predictions were not as conservative.

Additionally, the laboratory test machine available at the University have a limitation of maximum length of 2400 mm, and the limitation of the supplier of CLT elements, could only deliver elements with the length of 2100 mm, meaning that the laboratory testing is not a full-scale testing, but more a medium-sized testing. The height of the CLT element were also a limitation of delivery from the supplier. There might be some additional limitations to the execution of the laboratory testing based on this, i.e., the span to height ratio should have been taken more into considerations.

Further, the partial factors for both concrete and timber, were used in the theoretical predictions. These partial factors multiplied with the characteristic material properties will have an affect of the result of the theoretical predictions.

Also, the maximum applied load and maximum deflection calculations for Long-term verification have been compared to the laboratory testing results. Due to the consideration of creep and shrinkage in the long-term calculations, and the laboratory testing is performed 28-40 days after the casting, since these failures are time-dependent and the test specimens are fresh, this cannot be directly compared.

8.3 Comparison of the laboratory test results

From the graphical and tabulated presentation of the test results in chapter 7, it seems like the shear fastener used for slabs of type A is the type of shear fastener that can withstand the highest applied loading. In addition to slabs of type A and B, a Slab C1 using the KOP screw as shear fasteners, have a similar result of slab type A, and it seems like the orientation of the shear fastener have a great influence on the composite action and load capacity and maximum deflection.

It seems as the shar fastener arranged in pairs crossed parallelly of 45^{o} angle, based on the results of slabs of type A and Slab C1, but since there was only tested one with KOP screws in this direction and that the only slabs of type B, B6, had a similar result. Because of the limitations that have impacted the laboratory testing, the slabs of type B, might behave differently result, but this is too early to determine, and further studies should be conducted.

8.4 Additional theoretical predictions based on the results from four-point bending test

In Chapter 4.4 and Chapter 4.5 the theoretical predictions for the slabs of type A and type B were performed. After the laboratory testing it was observed that for the slabs of type A the theoretical predictions were conservative in comparison. For the slabs of type B, the theoretical prediction resembled the test result, although there are number of limitations that could affect this similarity, as previously discussed.

Therefore, additional theoretical predictions were performed in order to make a better estimate of the load capacity of the CLT-concrete composite slab. Since the laboratory testing of the CLT-concrete composite slabs, were performed in the period of 31-40 days after the casting of concrete, it can be presumed to only make comparison to the short-term verifications.

In Chapter 4 the results of the theoretical predictions are found and results of laboratory testing are found in Chapter 7. The additional theoretical predictions were performed following the same procedure, but changing the maximum moment, based on the failure. Instead of choosing the moment on top part of the concrete element, the moment of top (M3) and bottom (M4) elements were calculated. In addition, the concrete verification laboratory testing showed that the concrete had a much higher strength. Two more theoretical predictions were performed using the values from [21, Table 3.1] for concrete strength class B45 and using the total average value of the concrete strength, from the compressive strength test and partial safety factor set to one.

Table 8-2 below presents the maximum applied load for the additional theoretical predictions using different failure moments or material properties. For the type of slab with less limitations, slabs of type A, this table shows that by adjusting the calculations it will give a less conservative result, than the original theoretical predictions. The reason for slabs of type B, having closer values to the theoretical predictions and not the modified theoretical predictions, might be because of the different limitations of the CLT element described more thoroughly in the beginning of this chapter. These theoretical predictions can be found in Appendix D.

Although, keep in mind that the CLT elements, will vary and also sometimes flaws and defects exits and can lead to premature failure.

Table 8-2 Maximum applied load of different additional calculations

Max. applied load P [kN]		M3, Moment at top of timber	M4, Moment at bottom of timber	B45, concrete strength	Average Concrete strength
Short-term	CTC	$136.5205\ kN$	$177.931\ kN$	$94.35\ kN$	$136.5205\ kN$
	KOP	$133.903\ kN$	$166.443\ kN$	$91.994\ kN$	$133.903\ kN$
Long-term	CTC	$118.885\ kN$	$153.933\ kN$	$120.073\ kN$	$118.885\ kN$
	KOP	$116.0389\ kN$	$143.617\ kN$	$117.235\ kN$	$116.0389\ kN$

Similar additional theoretical predictions have been performed of the maximum deflection calculations based on SLS verification. Although the theoretical predictions and test result of the laboratory testing are more alike and are still satisfactory, additional theoretical predictions were performed and result presented in Table 8-3. These values give a much higher value for the vertical deflection that the result and will not satisfy the allowable deflection from the verification calculations, except the one using another concrete strength, B45. These theoretical predictions can be found in Appendix E.

This additional calculation for B45, give a much closer value for the actual vertical deflection result from laboratory testing for both types of shear fastener.

Table 8-3 Maximum vertical deflection of different additional calculations

Vertical deflection w [mm]		M3, Moment at top of timber	M4, Moment at bottom of timber	B45, concrete strength	Average Concrete strength	Allowable deflection [mm]
Short-term	CTC	$9.857\ mm$	$14.357\ mm$	$6.8557\ mm$	$9.857\ mm$	$\frac{w}{w_{lim}} < 8$
	KOP	$10.129\ mm$	$13.817\ mm$	$6.989\ mm$	$10.129\ mm$	
Long-term	CTC	$18.344\ mm$	$26.333\ mm$	$18.332\ mm$	$18.344\ mm$	$\frac{w}{w_{lim}} < 13.33$
	KOP	$18.818\ mm$	$25.415\ mm$	$18.809\ mm$	$18.818\ mm$	

8.5 Comparison of theoretical predictions and test results

8.5.1 Slabs A1-A6

Presenting the data described in the previous Chapter 7.2 graphically it is easier to demonstrate the outcome of the performed laboratory experiment compared to the theoretical predictions. Figure 8-13 shows the values from laboratory testing versus the theoretical predictions. It shows the theoretical predictions, dark blue dots, short-term, using the shear analogy and γ-method is very conservative, since it is very far from the line drawn into the graph. By adjusting the material properties or changing the location of the maximum moment in the theoretical predictions it is possible to receive some values that are closer to the result of the laboratory testing. A remark to the graph is that the moment at the top of the timber, M3 and the case of average compressive strength from the compressive test verifying of the concrete have the exact same values and that is why the grey dots, M3, is not seen in the figure below.

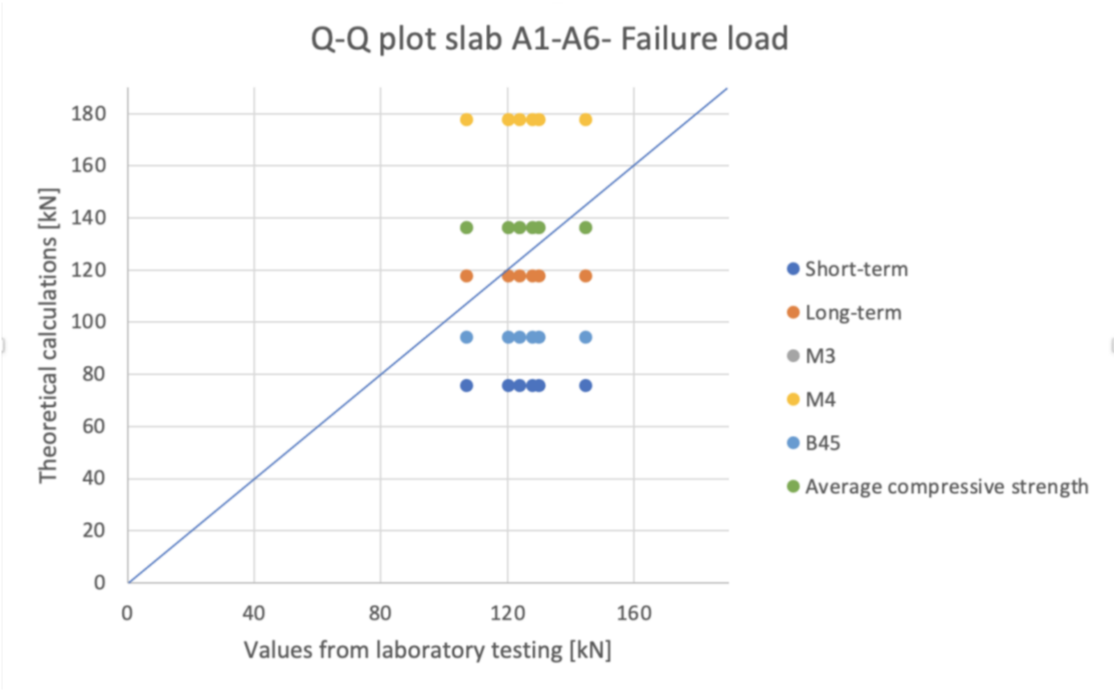


Figure 8-13 Q-Q plot of Slabs of type A

8.5.2 Slabs B1-B6 + Slab C1

A similar Q-Q graph have been performed for the slabs of type B and Slab C1. Figure 8-14 demonstrates the result of the results of the laboratory testing and the theoretical predictions. The same remark regarding M3, and average compressive strength can be seen in this graph as well. For this type of shear fastener, it shows that the theoretical prediction is much closer to the straight line, meaning that it is not as a conservative result, except for some of the test elements. The reason for this can be that five out of the six elements with KOP screws have had limitations as previously discussed in this chapter.

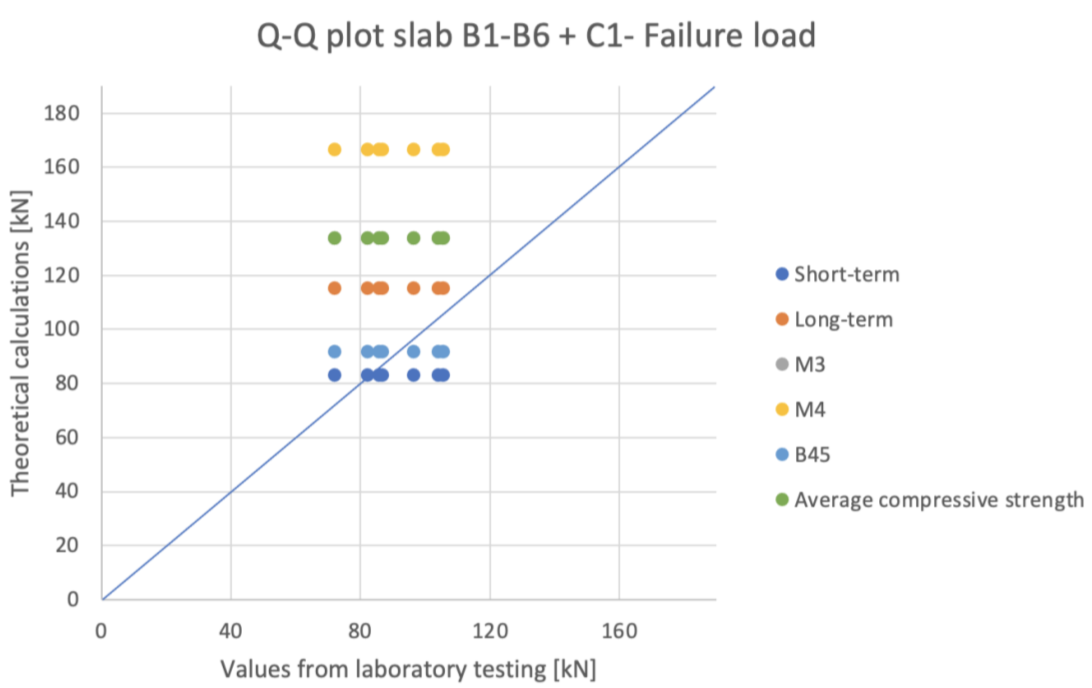


Figure 8-14 Q-Q plot of Slab of type B + Slab C1

8.6 Comparison of KOP screws from laboratory testing and design example

Another interesting comparison to make was the difference of the orientation of the KOP shear fasteners in this Master thesis in comparison to the design example [25, Ch. 7]. The design exampled used KOP screws with spacing 100 mm, but the screws were arranged in pairs crossed parallelly of 45^{o} angle.

Table 8-4 The load per fastener and design load

Slab		$F\ [kN]$	$F_{Rd}\ [kN]$	$\frac{F_1}{F_{Rd}}$
Design example		44.29	49.54	0.89
Theoretical predictions		3.75764	32	0.1174
Laboratory test results	B1	3.69	32	0.115
	B2	4.19	32	0.131
	B3	4.38	32	0.137
	B4	4.91	32	0.154
	B5	4.42	32	0.138
	B6	5.30	32	0.166
	C1	5.38	32	0.168

A limitation with this comparison, is that the theoretical predictions performed in the design example [25, Ch. 7] safety factors and other characteristic loads and variable loads are included and the concrete was of strength class B30.

By using the values obtained from the first failure drop in the four-point loading test when finding the load per shear fastener and compare this with the design load from theoretical predictions. There are some uncertainties whether this verification can be done and if this can be compared directly to the design example and theoretical predictions.

In addition, the design example is a full-scale test element, but for the theoretical predictions and laboratory testing in this master thesis uses a medium-scale test element, and for this reason, a direct comparison cannot be made.

8.7 Efficiency of composite behaviour of the CLT-concrete composite slabs

The efficiency of composite behaviour has been described earlier in chapter 2.1. The efficiency of composite behaviour of the TCC slabs is one way of determining the degree of composite behaviour. Table 8-5 and 8-6 below presents the result of the fully and no composite action from the theoretical predictions for each type of slabs.

Table 8-5 Deflection for no and fully composite for slab type A

	Notation	**Value [mm]**
Theoretical deflection for the corresponding slab with **no** composite action	D_N	5.547009131
Theoretical deflection for the corresponding slab with **full** composite action	D_C	5.610726172

Table 8-6 Deflection for no and fully composite for slab type B and C

	Notation	**Value [mm]**
Theoretical deflection for the corresponding slab with **no** composite action	D_N	6.354393607
Theoretical deflection for the corresponding slab with **full** composite action	D_C	5.725544743

Table 8-7 Efficiency for the different slab types

	D_N	D_C	D_I	Efficiency
A1	5.54700913	5.61072617	6.159733	9.61632649
A2	5.54700913	5.61072617	7.67309	33.3675393
A3	5.54700913	5.61072617	6.858697	20.5861391
A4	5.54700913	5.61072617	6.997531	22.7650538
A5	5.54700913	5.61072617	8.58584	47.6925925
A6	5.54700913	5.61072617	8.803105	51.1024338
B1	6.35439361	5.72554474	5.037414	2.09427047
B2	6.35439361	5.72554474	5.578972	1.23308103
B3	6.35439361	5.72554474	5.729669	0.99344158
B4	6.35439361	5.72554474	6.591282	-0.3767016
B5	6.35439361	5.72554474	4.760207	2.53508704
B6	6.35439361	5.72554474	5.432587	1.46586352
C1	6.35439361	5.72554474	5.882049	0.7511258

The aim is to achieve a TCC where the concrete and timber, together with the shear fastener work together as fully composite structure, meaning an efficiency of 100%. Table 8-7 above shows the calculated efficiency based on the theory using the deflection from theoretical predictions for both no and fully composite, together with the deflection from the laboratory testing.

From the efficiency calculations it is slabs of type A with the best composite behaviour, but still, it is not sufficient to call these actions fully composite, when it behaves as partial composite. For slabs of type B and Slab C1, the results shows that the two parts, timber and concrete, work almost separately and that there are no composite behaviour at all.

The problem with this efficiency calculations is that it is dependent on the theoretical predictions from both fully and no composite action, as well as the laboratory test results. Hence, the already describe limitations of the orientation of the different lamellae and orientation of these are previously described. These limitations will have an influence on the result on both the theoretical predictions for the fully and no composite action, where the outer lamellae should have the higher material properties and have a 10 mm higher height, as well as this also is an influential factor for the test result from the laboratory testing.

It has been assumed that to follow the same theoretical predictions as previously described in Chapter 4, but instead of using $\gamma_2 = 1.0$, fully composite it is used $\gamma_2 = 0$ for the theoretical predictions. Due to the all the limitations for this thesis, there will also be some uncertainties to the predicted efficiency for the shear fasteners. Hence, there are an uncertainty whether this is a good efficiency prediction for the CLT-concrete composite slabs. These calculations are found in Appendix L.

9 Conclusion

9.1 Summary

This thesis discusses the behaviour of CLT-concrete composite slabs with two different types of shear fastener. The primary objective is to investigate load capacity and structural responses of the CLT-concrete composite slabs with two different types of shear fasteners, both CTC and KOP screws. The theoretical predictions of the structural behaviour of the slabs were compared with the response of four-point bending tests.

First, theoretical predictions of the load capacity and maximum displacements were calculated for the two types of CLT-concrete composite slab with the two different types of shear fasteners, CTC and KOP screws, by following the procedure of γ-method in Eurocode 5, Annex B, [22, Annex B] for composite timber-timber with mechanically jointed beams. A modified theoretical prediction was performed by using the shear analogy method together with the γ-method. The shear analogy method includes the shear deformation in the transverse layers for composite elements with more than three layers, as an official standard and/or guideline does not exist for TCC elements.

The laboratory testing was performed in accordance with the general principles provided in the standard *NS-ISO 6891:1991 Timber structures, Joints made with mechanical fasteners* [19].

9.2 Concluding remarks

The load-displacement behaviour of the tested slabs shows the linear variation until first load drop point which is mostly due to the premature failure of the slabs and/or due to the interlayer slip. Then after, it behaves nonlinearly until reaching the maximum load where the collapse/fracture of the slabs are obtained.

Although, the theoretical predictions give a good interpretation of the behaviour of the CLT-concrete composite slab, it does provide a conservative prediction for slab type A with CTC screws as shear fasteners, but a great indication on the test result for the slab type B, with KOP screws.

By comparing the results from the laboratory testing with the theoretical predictions, it can be stated that the slab type A can withstand a much higher applied load than slab type B. It shows that the slabs of type A have a much more conservative theoretical predictions compared to actual load applied in laboratory testing and that the slabs of type B have very similar results in both the theoretical predictions and test results.

In addition, the KOP screws were compared to the design example used as the basis for this thesis [25, Ch. 7]. Due to differences in the type of scale of testing, the comparability cannot be made directly. However, the results from the laboratory testing, shows that the chosen type of shear fastener and its orientation has a considerable influence on the load capacity and maximum deflection.

There have been a lot of different limitations that have influenced both the theoretical predictions and the laboratory test results. Hence, further studies should be performed before making a definite conclusion on the behaviour of the different shear fasteners, CTC and KOP screws for this type of CLT slab.

An official standard and/or guideline does not exist for CLT-concrete slabs and most of experimental and numerical analysis focus on TCC where the timber is used as beams or columns [1, p. 56], however, some guidelines from different companies are available.

9.3 Suggestions for further study

To summarise, there were a lot of limitations that individually will have an influential factor of the theoretical predictions and the laboratory testing, and this can further make a greater divergence for the overall analysis of the CLT-concrete composite slab. To overcome above shortcomings, additional laboratory tests are recommended with the same 5-layered CLT slab, strength in concrete and shear fasteners (CTC and KOP screws) and its arrangements with additional transducers. These additional measuring gauges/sensors should be placed at the top and bottom of both the timber and concrete element, as well as on the shear fasteners, so that the slip can be measured more thoroughly. Other types and variable spacing of shear fasteners could be interesting to investigate, where the cost and time of installation of the shear fasteners are considered.

Additional theoretical analysis and parametric study should be performed to investigate performance of the composite action of CLT concrete slabs such as push-out test for the shear fasteners, since there are no official standard or guidelines for this type of material. The difference in the effective bending stiffness is also recommended to obtain from the theoretical predictions and laboratory testing, for investigating the efficiency of the composite action, instead of using the calculated and measured deflection to find the efficiency, since the aim is to achieve fully composite action.

10 References

[1] K. Q. Mai, A. Park, K. T. Nguyen and K. Lee, “Full-scale static and dynamic experiments of hybrid CLT-concrete composite floor”, *Construction and Building MATERIALS,* Vol. 170, p. 55-65, May 2019, [Online], Available: https://doi.org/10.1016/j.conbuildmat.2018.03.042

[2] Y. Jiang and R. Crocetti, “CLT-concrete composite floors with notched shear connectors”, *Construction and Building MATERIALS,* Vol. 195, p. 127- 139, Jan. 2019, [Online], Available: https://doi.org/10.1016/j.conbuildmat.2018.11.066

[3] E. Karacabeyli and C. Lum, “Structural and serviceability,”in Technical Guide for the Design and Construction of Tall Wood Buildings in Canada, 1. ed. Pointe-Claire, QC, Canada: Library and Archives Canada Cataloguing in Publication, 2014, Chapter 4, p. 1-35, (pdf: p. 163-200)

[4] M. Cepelka, “Timber-Concrete Composites – an alternative for floors and bridges?”, Trial lecture 30. Nov 2017

[5] A. Pousette, K. A. Malo, S. Thelandersson, S. Fortino, L. Salokangas and J. Wacker, “Durable timber bridges final report and guidelines”, RISE, Skellefteå, Sweeden, SP Rapport 2017:25, ISSN 0284-5172, 2017

[6] A. Dias, J. Schänzlin and P. Dietsch (eds.), “Design of timber-concrete composite structure”, COST, Aachen, Germany, Shaker Verlag GmbH, 2018

[7] E. Lukaszewska, “Development of prefabricated Timber-cocnrete floors,” Dr. Philos, Department of Civil, Mining and Environmental Engineering, Luleå University of Technology, Luleå, Sweeden, 2009

[8] A. M. P.G Dias, “Mechanical behaviour of timber-concrete connections.” Dr. Philos, Delft University of Technology, Delft, April 2005

[9] A. Manaridis, “Evaluation of timber- concrete composite floors,” Lunds Tekniska Högskola, Lund, Sverige, TVBK - 5187, ISSN 0349-4969, July 2010.

[10] A. Gustafsson + et al., “The CLT handbook CLT structures - facts and planning”, 1. Ed, Stockholm: Skogsindustrierna, 2019

[11] S. Cuerrier-Auclair, “Design Guide for Timber-Concrete Composite Floors in Canada,” FPInnovations, 1. Ed, Pointe-Claire, QC, Canada, Special Publications SP-540E, ISSN 1925-0495, June 2020

[12] StructureCraft, “Timber-cocnrete composite,” Website, Available: https://structurecraft.com/materials/mass-timber/timber-concrete-composite, 14th of June, 2021

[13]TreFokus, “Massivtre,” Available: http://www.trefokus.no/treveilederen/temaer/byggesystemer/massivtre, 16.06.2021

[14] J. Aarstad, G. Glasø and A. Bunkholt, “Fokus på tre, Massivtre”, TreFokus, Treteknisk, [Online], ISSN 1501-7427, Aug. 2011

[15] S. Jacobsen, ed., “Concrete Technology,” Trondheim, Norway: NTNU, Jan 2016, Ch. 1.

[16] *Betong – Specifikasjon, egenskaper, framstilling og samsvar*, NS-EN 206:2013

[17] *Prøving av herdnet betong Del 1: Form, mål og andre krav til prøvelegemer og former*, NS-EN 12390-1:2012

[18] *Prøving av herdnet betong Del 6: Prøvelegemers spaltestrekkfasthet (innbefattet rettelsesblad AC:2004)*, NS-EN 12390-6:2001

[19] *Trekonstruksjoner Mekaniske forbindelsesmidler Generelle prinsipper for bestemmelse av styrke og deformasjonsegenskaper (= EN 26891:1991) (ISO 6891:1983)*, NS-ISO 6891:1991

[20] *Eurocode 0: Grunnlag for prosjektering av konstruksjonerr*, NS-EN 1990:2002+A1:2005+NA2016

[21] *Eurocode 2: Design of concrete structures - Part 1-1General rules and rules for buildings,* NS-EN 1992-1-1:2004

[22] *Eurocode 5: Prosjektering av trekonstruksjoner – Del 1-1 Allmenne regler og regler for bygninger,* NS-EN 1995-1-1:2004+A1:2008+NA2010

[23] J.Jaaranen, "Analysis methods for short to medium span timber-concrete composite bridges", Master thesis, Structural Engineering and Building Technology, Aalto University, Finland, 2016

[24] B. Douglas and E. Karacabeyli, "Structural design of cross laminated timber elements," in CLT Handbook – cross-laminated timber, US ed. Pointe-Claire, QC, Canada: Library and Archives Canada Cataloguing in Publication, 2013, Chapter 3, p. 1-19 (pdf: p. 93-117)

[25] R. W. Haug, "Timber-Concrete composite floor in Norway/Scandinavia: Recent development and design methods", Master thesis, Faculty of Science and Technology, University of Stavanger, Stavanger, 2020

[26] *Teknisk Godkjenning Splitkon krysslimt tre,* SINTEF Certification Nr. 20712, first issued 07.10.2019, Valid until 01.11.2024

[27] J. Thilén, "Testing of CLT-Concrete composite decs," Lunds Tekniska Högskola, Lund, Sverige, TVBK - 5259, ISSN 0349-4969, July 2017.

[28] Rothoblaas, *Screws and connectors for timber 2021*, 2021, [Online], Available: https://issuu.com/rothoblaas/docs/screws-and-connectors-for-timber-2021-en?mode=embed

11 Appendices

A. Load capacity for Slab A and Slab B based on ULS verifications
B. Maximum deflection for Slab A and Slab B based on SLS verifications
C. Rothoblaas verification of slab, Excel spreadsheet
D. Additional load capacity for Slab A and Slab B based on ULS verifications
E. Additional maximum deflection for Slab A and Slab B based on SLS verifications
F. Load capacity calculations for Slab A and Slab B based on ULS, 3-Layerd CLT
G. Sola Betong, Concrete recipe
H. Graphs from Excel spreadsheet, Vertical deflection at Midspan response for Slab A and Slab B (Slab C1)
I. Graphs from Excel spreadsheet, Lateral displacement response for Slab A and Slab B (Slab C1)
J. Graphs from the Toni Technik machine
K. Pictures of Failures
L. Efficiency calculations
M. Rothoblaas, extract of “screws and connectors for wood” catalogue
N. Splitkon, CLT catalogue

Appendix A. Load capacity for Slab A and Slab B based on ULS verification

A.1 Load capacity for Slab A (CTC screws) based on ULS verifications

A.2 Load capacity for Slab B (KOP screws) based on ULS verifications

Load capacity predictions based on ULS
CTC-screws 7-160 mm

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

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

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

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

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

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

$$h_t := 120 \quad (1)$$

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

> $Klima := 1.0 : \#Serice\ class,\ permanent$

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

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

Lamellae 1 and 5, Class T22

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

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

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

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

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

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

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

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

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

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

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

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

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

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

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

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

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

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

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

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m^3}$

$$\rho_{t15} := 4.216853302 \qquad (3)$$

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

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

$$g_{0,k} := 1.217734993 \quad (4)$$

1. 1 ULS

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

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

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

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1; \#mm\hat{\ }2$

$$A_1 := 18000 \quad (5)$$

> $A_5 := A_1; \#mm\hat{\ }2$

$$A_5 := 18000 \quad (6)$$

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

$$I_{t1} := 1350000 \quad (7)$$

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

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

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

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2;$ #mm^2

$$A_3 := 12000 \quad (10)$$

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

$$A_4 := 12000 \quad (11)$$

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

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2};$ #mm^4

$$I_{t3} := 400000 \quad (13)$$

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

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

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

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

$$z_1 := 45 \quad (15)$$

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

$$z_2 := 20 \quad (16)$$

> $z_3 := 0;$ #mm

$$z_3 := 0 \quad (17)$$

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

$$z_4 := 20 \quad (18)$$

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

$$z_5 := 45 \quad \textbf{(19)}$$

Ei*Ii

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

$$(EI)_1 := 17550000000 \quad \textbf{(20)}$$

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

$$(EI)_2 := 92000000 \quad \textbf{(21)}$$

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

$$(EI)_3 := 4600000000 \quad \textbf{(22)}$$

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

$$(EI)_4 := 92000000 \quad \textbf{(23)}$$

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

$$(EI)_5 := 17550000000 \quad \textbf{(24)}$$

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

$$(EI)_{sum} := 39884000000 \quad \textbf{(25)}$$

Ei*Ao*zi^2

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

$$(EAz\hat{}2)_1 := 473850000000 \quad \textbf{(26)}$$

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

$$(EAz\hat{}2)_2 := 1104000000 \quad \textbf{(27)}$$

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

$$(EAz\hat{}2)_3 := 0 \quad \textbf{(28)}$$

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

$$(EAz\hat{}2)_4 := 1104000000 \quad \textbf{(29)}$$

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

$$(EAz\hat{}2)_5 := 473850000000 \quad \textbf{(30)}$$

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

$$(EAz\hat{}2)_{sum} := 949908000000 \quad \textbf{(31)}$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

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

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad \textbf{(32)}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

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

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

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

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

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (35)$$

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

$$EI_{app} := 7.260572161\ 10^{11} \quad (36)$$

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

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5

Maximum load capacity based on ULS using short-term verifications of the slab

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

General data:

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

$$E_1 := 34000 \quad (38)$$

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

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c$; #mm

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t$; #mm

$$h_2 := 120 \quad (41)$$

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

$$A_1 := 36000 \quad (42)$$

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

$$A_2 := 72000 \quad (43)$$

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

$$I_1 := 10800000 \quad (44)$$

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

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**
EC5: 2.2.2(2) Ultimate limit states equation 2.1
The values for the slip modulus (Kser) is found in the pdf about the screw types from the company

Rotho blaas page 227
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

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

$$l_{eff,ctc} := 110 \quad (46)$$

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

$$K_{ser} := 23100 \quad (47)$$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right);$ # $\frac{N}{mm}$ #equation 2.1 EC5

$$K_u := 15400. \quad (48)$$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen.

> $angle := 45;$

$$angle := 45 \quad (49)$$

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

$$k := \frac{\sqrt{2}}{2} \quad (50)$$

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

>

> $s := 150;$ #mm

$$s := 150 \quad (51)$$

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.03287684039 \quad (52)$$

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

$$\gamma_2 := 1.0 \quad (53)$$

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

$$a_2 := 5.612544326 \quad (54)$$

>

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

$$a_1 := 84.38745567 \qquad (55)$$

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

$$EI_{eff,tot} := 1.398884340\ 10^{12} \qquad (56)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 2.929172990\ 10^7 \qquad (57)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 2.216443886\ 10^6 \qquad (58)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 5.210849666\; 10^7 \quad \textbf{(59)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

(60)

$$M_4 := 6.763762779\ 10^7 \quad (60)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

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

$$M_{Ed,new} := 29.29172990 \quad (61)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (62)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (63)$$

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

$$P_{Ed} := 75.67580975 \quad (64)$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 1.975195536 \quad (65)$$

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

$$\sigma_{m,1} := 21.35813780 \quad (66)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -23.33333334 \quad (67)$$

Verification of the top section

> $Ver_{top,\ c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\ c}}{\gamma_c}}$; # <1.0 #--->OK

$$Ver_{top,c} := -1.000000000 \quad (68)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 19.38294226 \tag{69}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; # 1.0 ---> NOT OK

$$Ver_{bottom,c} := 13.21564245 \tag{70}$$

3.4.2 Normal stresses in the **TIMBER** section

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

$$\sigma_2 := 0.9875977681 \tag{71}$$

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

$$\sigma_{m,2} := 10.55775468 \tag{72}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -11.54535245 \tag{73}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 9.570156912 \tag{74}$$

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

$$Ver_{timber} := -0.3033313473 \tag{75}$$

3.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 3.273142161 \tag{76}$$

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

$$Ver_{shear} := 1.176285464 \quad (77)$$

3.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 27.55598319 \quad (78)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 27.55598319 \quad (79)$$

> $f_{tens,\, k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (80)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\, k}}{\gamma_M}}$; # < 1.0 #-->OK

$$Ver_{F1} := 0.6601954307 \quad (81)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross section has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\, eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 115.2431566 \quad (82)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\, eff}$; #mm

$$x := 7.577661732 \quad (83)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\, new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\, eff}$;

$$a_{2,new} := 0.9680125 \quad (84)$$

> $A_{1,\, eff} := b \cdot x$;

(85)

$$A_{1,eff} := 4546.597039 \quad (85)$$

> $I_{1,\,eff} := \dfrac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 21755.82954 \quad (86)$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2;$ #Nmm^2

$$EI_{eff,tot,new} := 7.948610712\ 10^{11} \quad (87)$$

5. New short-term verifications

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #MPa

$$\sigma_1 := 4.747204370 \quad (88)$$

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

$$\sigma_{m,1} := 4.747204372 \quad (89)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -9.494408742 \quad (90)$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}};$ # <1.0 #--->OK

$$Ver_{top,c} := -0.4069032318 \quad (91)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 2.\ 10^{-9} \tag{92}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom,c} := 1.363636364\ 10^{-9} \tag{93}$$

5.1.2 Normal stresses in the **TIMBER** section

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

$$\sigma_2 := 0.2997725500 \tag{94}$$

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

$$\sigma_{m,2} := 18.58070324 \tag{95}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -18.88047579 \tag{96}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 18.28093069 \tag{97}$$

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

$$Ver_{timber} := -0.3720662106 \tag{98}$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 1.486952284 \tag{99}$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; # <1.0 #-->OK

$$Ver_{shear} := 0.5343734771 \quad (100)$$

5.1.4 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

$$F_1 := 8.364263549 \quad (101)$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 8.364262875 \quad (102)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (103)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.2003938143 \quad (104)$$

6. Maximum load capacity based on ULS using long-term verification of the slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,\,g} := \dfrac{E_{cm,\,c}}{1 + \varphi_c}$;

$$E_{1,g} := 9714.285714 \quad (105)$$

> $E_{1,\,q} := \dfrac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad (106)$$

> $q_k := 0$;

$$q_k := 0 \quad (107)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (108)$$

> $E_1 := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285715 \quad (109)$$

6.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1 + k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (110)$$

> $E_{2,q} := \frac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (111)$$

> $E_2 := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad (112)$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \frac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (113)$$

> $K_{ser,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad (114)$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad (115)$$

> $K_u := \frac{2}{3} \cdot K_{ser,2}$

$$K_u := 8324.324327 \quad (116)$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.06042753609 \qquad (117)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \qquad (118)$$

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

$$a_2 := 5.462362231 \qquad (119)$$

>

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

$$a_1 := 84.53763777 \qquad (120)$$

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

$$EI_{eff,tot} := 6.581609231\ 10^{11} \qquad (121)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

$$\sigma_1 := 0.07539868541\ M_{Ed,1} \qquad (122)$$

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

$$\sigma_{m,1} := 0.4427922734\ M_{Ed,1} \qquad (123)$$

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 4.502844547\ 10^7 \quad \textbf{(124)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 3.992085640\ 10^6 \quad \textbf{(125)}$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

$$\sigma_2 := 0.03769934270\ M_{Ed,2} \quad \textbf{(126)}$$

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

$$\sigma_{m,2} := 0.4140993340\ M_{Ed,2} \quad \textbf{(127)}$$

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 4.549531312\ 10^7 \qquad \textbf{(128)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 5.863841321\ 10^7 \qquad \textbf{(129)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

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

$$M_{Ed,new} := 45.02844547 \qquad \textbf{(130)}$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \qquad \textbf{(131)}$$

> $L_{sup} := 2.0; \#m$

$$L_{sup} := 2.0 \qquad \textbf{(132)}$$

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

$$P_{Ed} := 117.6403846 \qquad \textbf{(133)}$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 3.395085595 \quad (134)$$

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

$$\sigma_{m,1} := 19.93824774 \quad (135)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -23.33333334 \quad (136)$$

Verification of the top section

> $Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}};$ *# <1.0 #--->OK*

$$Ver_{top,c} := -1.000000000 \quad (137)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 16.54316214 \quad (138)$$

> $Ver_{bottom,\ c} := \frac{\sigma_{b,\ c}}{\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c}};$ *# 1.0 ---> NOT OK*

$$Ver_{bottom,c} := 11.27942873 \quad (139)$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_2 := 1.697542797 \quad (140)$$

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

$$\sigma_{m,2} := 18.64624928 \quad (141)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -20.34379208 \quad (142)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 16.94870648 \quad (143)$$

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

$$Ver_{timber} := -0.5304698833 \quad (144)$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 5.845776587 \quad (145)$$

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

$$Ver_{shear} := 2.100825962 \quad (146)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 47.89762389 \quad (147)$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 47.89762389 \quad (148)$$

> $f_{tens,\,k} := 20.0$; #kN

(149)

$$f_{tens,k} := 20.0 \quad \textbf{(149)}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; # *1.0 ---> NOT OK*

$$Ver_{F1} := 1.147547239 \quad \textbf{(150)}$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; *#mm*

$$a_{1,eff} := 111.6324251 \quad \textbf{(151)}$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; *#mm*

$$x := 13.49134479 \quad \textbf{(152)}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 1.6219025 \quad \textbf{(153)}$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 8094.806874 \quad \textbf{(154)}$$

> $I_{1,\,eff} := \dfrac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 122782.2898 \quad \textbf{(155)}$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2$; *#Nmm^2*

$$EI_{eff,tot,new} := 4.537316051\ 10^{11} \quad \textbf{(156)}$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \dfrac{\left(\gamma_1 \cdot E_1 \cdot a_{1,\ eff} \cdot M_{Ed,\ new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_1 := 6.503154036 \quad (157)$$

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

$$\sigma_{m,1} := 6.503154035 \quad (158)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -13.00630807 \quad (159)$$

Verification of the top section

> $Ver_{top,\ c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\ c}}{\gamma_c}};$ *# <1.0 #--- >OK*

$$Ver_{top,c} := -0.5574132030 \quad (160)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := -1.\ 10^{-9} \quad (161)$$

> $Ver_{bottom,\ c} := \dfrac{\sigma_{b,\ c}}{\dfrac{f_{ctk,\ 0.05,\ c}}{\gamma_c}};$ *# <1.0 #--- >OK*

$$Ver_{bottom,c} := -6.818181818\ 10^{-10} \quad (162)$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{\left(\gamma_2 \cdot E_2 \cdot a_{2,\ new} \cdot M_{Ed,\ new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_2 := 0.7311357848 \quad (163)$$

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

$$\sigma_{m,2} := 27.04733922 \quad (164)$$

Stresses at the TOP of the timber section

> $\sigma_{t,t} := -\sigma_2 - \sigma_{m,2}$; #MPa

$$\sigma_{t,t} := -27.77847500 \quad (165)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$; #MPa

$$\sigma_{b,t} := 26.31620344 \quad (166)$$

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

$$Ver_{timber} := -0.574757942 \quad (167)$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 2.236057609 \quad (168)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{shear} := 0.8035832032 \quad (169)$$

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

$$F_1 := 20.62962215 \quad (170)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 20.62962236 \quad (171)$$

> $f_{tens,\ k} := 20.0;$ #*kN*

$$f_{tens,k} := 20.0 \qquad (172)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}};$ # *<1.0 #--> OK*

$$Ver_{F1} := 0.4942513640 \qquad (173)$$

>

>

Load capacity predictions based on ULS KOP-screws 10-140 mm

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

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

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

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

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

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

$$h_t := 120 \quad (1)$$

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

> $Klima := 1.0 : \#Serice\ class,\ permanent$

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

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

Lamellae 1 and 5, Class T22

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

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

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

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

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

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

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

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

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

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

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

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

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

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

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

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

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

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

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

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m^3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

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

$$g_{0,k} := 1.217734993 \quad \textbf{(4)}$$

1. 1 ULS

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

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

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

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1; \# mm\hat{\ }2$

$$A_1 := 18000 \quad \textbf{(5)}$$

> $A_5 := A_1; \# mm\hat{\ }2$

$$A_5 := 18000 \quad \textbf{(6)}$$

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

$$I_{t1} := 1350000 \quad \textbf{(7)}$$

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

$$I_{t5} := 1350000 \quad \textbf{(8)}$$

Layer 2, 3 and 4 (T15)

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

$$A_2 := 12000 \tag{9}$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \tag{10}$$

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

$$A_4 := 12000 \tag{11}$$

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

$$I_{t2} := 400000 \tag{12}$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \tag{13}$$

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

$$I_{t4} := 400000 \tag{14}$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24

The height to NA

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

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

$$z_1 := 45 \tag{15}$$

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

$$z_2 := 20 \tag{16}$$

> $z_3 := 0$; #mm

$$z_3 := 0 \tag{17}$$

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

$$z_4 := 20 \tag{18}$$

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

$$z_5 := 45 \quad \textbf{(19)}$$

Ei*Ii

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

$$(EI)_1 := 17550000000 \quad \textbf{(20)}$$

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

$$(EI)_2 := 92000000 \quad \textbf{(21)}$$

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

$$(EI)_3 := 4600000000 \quad \textbf{(22)}$$

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

$$(EI)_4 := 92000000 \quad \textbf{(23)}$$

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

$$(EI)_5 := 17550000000 \quad \textbf{(24)}$$

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

$$(EI)_{sum} := 39884000000 \quad \textbf{(25)}$$

Ei*Ao*zi^2

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

$$(EAz\hat{}2)_1 := 473850000000 \quad \textbf{(26)}$$

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

$$(EAz\hat{}2)_2 := 1104000000 \quad \textbf{(27)}$$

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

$$(EAz\hat{}2)_3 := 0 \quad \textbf{(28)}$$

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

$$(EAz\hat{}2)_4 := 1104000000 \quad \textbf{(29)}$$

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

$$(EAz\hat{}2)_5 := 473850000000 \quad \textbf{(30)}$$

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

$$(EAz\hat{}2)_{sum} := 949908000000 \quad \textbf{(31)}$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

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

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad \textbf{(32)}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

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

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

$$a := 90 \quad \textbf{(33)}$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad \textbf{(34)}$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

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

> $K_s := 11.5$
> #CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad \textbf{(35)}$$

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

$$EI_{app} := 7.260572161\ 10^{11} \quad \textbf{(36)}$$

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

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using verification of the slab

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

General data:

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

$$E_1 := 34000 \quad (38)$$

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

$$E_2 := 8403.440000 \quad (39)$$

$> h_1 := h_c; \# mm$

$$h_1 := 60 \quad (40)$$

$> h_2 := h_t; \# mm$

$$h_2 := 120 \quad (41)$$

$> A_1 := A_c; \# mm\hat{}2$

$$A_1 := 36000 \quad (42)$$

$> A_2 := h_2 \cdot b; \# mm\hat{}2$

$$A_2 := 72000 \quad (43)$$

$> I_1 := I_c; \# mm\hat{}4$

$$I_1 := 10800000 \quad (44)$$

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

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**
To fid Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10120 screws, we have to use the EC5 for finding the Kser and sef .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)
Sincwe there js

> $\rho_{m,\ 1} := t_{t22};$

$$\rho_{m,1} := 470 \quad (46)$$

> $\rho_{m,\ 2} := t_{t15};$

$$\rho_{m,2} := 430 \quad (47)$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\ 1} \cdot \rho_{m,\ 2}\right)\right);$

$$\rho_m := 449.5553359 \quad (48)$$

> $d_{kop} := 10 \ ;\#mm$

$$d_{kop} := 10 \quad (49)$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m \hat{} 1.5 \cdot 3\right);$

$$K_{ser} := 12432.77728 \quad (50)$$

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

$$K_u := 8288.518187 \quad (51)$$

>

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100; \#mm$

$$s := 100 \quad (52)$$

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.02671144241 \quad (53)$$

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

$$\gamma_2 := 1.0 \quad (54)$$

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

(55)

$$a_2 := 4.613982231 \quad \textbf{(55)}$$

>

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

$$a_1 := 85.38601777 \quad \textbf{(56)}$$

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

$$EI_{eff,tot} := 1.344508348\ 10^{12} \quad \textbf{(57)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 2.858362473\ 10^7 \quad \textbf{(58)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 2.092353503\ 10^6 \quad \textbf{(59)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 5.112721903\ 10^7 \quad \textbf{(60)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 6.332956592\ 10^7 \quad (61)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

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

$$M_{Ed,new} := 28.58362473 \quad (62)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (63)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (64)$$

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

$$P_{Ed} := 73.78752929 \quad (65)$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 1.648605769 \quad (66)$$

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

$$\sigma_{m,1} := 21.68472756 \quad (67)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -23.33333333 \quad (68)$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.9999999999 \tag{69}$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 20.03612179 \tag{70}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->NOT OK

$$Ver_{bottom,c} := 13.66099213 \tag{71}$$

3.4.2 Normal stresses in the **TIMBER** section

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

$$\sigma_2 := 0.8243028849 \tag{72}$$

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

$$\sigma_{m,2} := 10.71919452 \tag{73}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -11.54349740 \tag{74}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 9.894891635 \tag{75}$$

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

$$Ver_{timber} := -0.2879050168 \tag{76}$$

3.4.3 Shear stresses in the **TIMBER** section

$$> \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2 \hat{} 2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10 \hat{} 3; \#MPa$$

$$\tau_2 := 3.320542671 \quad (77)$$

Verification of the timber section

$$> Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \# <1.0 \#---> NOT\ OK$$

$$Ver_{shear} := 1.193320023 \quad (78)$$

3.4.4 The **load per shear fasteners**

$$> F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_1 := 15.32092488 \quad (79)$$

$$> F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_2 := 15.32092488 \quad (80)$$

$$> f_{tens,\, k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \quad (81)$$

$$> Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\, k}}{\gamma_M}}; \# <1.0 \#--> OK$$

$$Ver_{F1} := 0.3670638253 \quad (82)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

$$> a_{1,\, eff} := \max\left(solve\left(a_{1,\,1} \hat{} 2 \cdot \left(4 \cdot \gamma_1 \hat{} 2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right); \#mm$$

$$a_{1,eff} := 116.2447862 \quad (83)$$

The effective compressed height of the concrete:

> $x := 2\cdot\gamma_1\cdot a_{1,\ eff}$; #mm

$$x := 6.210131824 \qquad (84)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\ new} := h_1 - 0.5\cdot x + 0.5\cdot h_2 - a_{1,\ eff}$;

$$a_{2,new} := 0.6501479 \qquad (85)$$

> $A_{1,\ eff} := b\cdot x$;

$$A_{1,eff} := 3726.079094 \qquad (86)$$

> $I_{1,\ eff} := \frac{b\cdot x\hat{}3}{12}$

$$I_{1,eff} := 11974.91562 \qquad (87)$$

> $EI_{eff,\ tot,\ new} := E_1\cdot I_{1,\ eff} + \gamma_1\cdot E_1\cdot A_{1,\ eff}\cdot a_{1,\ eff}\hat{}2 + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_{2,\ new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 7.724473842\ 10^{11} \qquad (88)$$

5. New short-term verifications

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 3.906592709 \qquad (89)$$

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

$$\sigma_{m,1} := 3.906592708 \qquad (90)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -7.813185417 \qquad (91)$$

Verification of the top section

$$> \; Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}; \#<1.0 \;\#--->OK$$

$$Ver_{top,c} := -0.3348508036 \qquad (92)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := -1.\,10^{-9} \qquad (93)$$

$$> \; Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}; \#<1.0 \;\#--->OK$$

$$Ver_{bottom,c} := -6.818181818\ 10^{-10} \qquad (94)$$

5.1.2 Normal stresses in the **TIMBER** section

$$> \; \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{\ }6; \#MPa$$

$$\sigma_2 := 0.2021704428 \qquad (95)$$

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

$$\sigma_{m,2} := 18.65764170 \qquad (96)$$

Stresses at the TOP of the timber section

$$> \; \sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}; \#MPa$$

$$\sigma_{t,t} := -18.85981214 \qquad (97)$$

Stresses at the BOTTOM of the timber section

$$> \; \sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}; \#MPa$$

$$\sigma_{b,t} := 18.45547126 \qquad (98)$$

Verification of the timber section

$$> \; Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right); \#<1.0 \;\#-->OK$$

$$Ver_{timber} := -0.3624897309 \qquad (99)$$

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

$$\tau_2 := 1.476402929 \quad (100)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; # <1.0 #-->OK

$$Ver_{shear} := 0.5305823025 \quad (101)$$

5.1.4 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

$$F_1 := 3.757646070 \quad (102)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 3.757645673 \quad (103)$$

> $f_{tens,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (104)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.09002693710 \quad (105)$$

6. Maximum load capacity based on ULS using long-term verifications of the slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1 + \varphi_c}$;

(106)

$$E_{1,g} := 9714.285714 \quad (106)$$

> $E_{1,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (107)$$

> $q_k := 0;$

$$q_k := 0 \quad (108)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (109)$$

> $E_1 := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285715 \quad (110)$$

6.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1 + k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (111)$$

> $E_{2,q} := \frac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (112)$$

> $E_2 := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad (113)$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \frac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 6720.420151 \quad (114)$$

> $K_{ser,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 8724.755986 \quad (115)$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 6720.420149 \quad (116)$$

> $K_u := \frac{2}{3} \cdot K_{ser,\,2}$

$$K_u := 4480.280099 \quad (117)$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.04935924062 \quad (118)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (119)$$

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

$$a_2 := 4.512000371 \quad (120)$$

>

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

$$a_1 := 85.48799963 \quad (121)$$

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

$$EI_{eff,tot} := 6.301872574\ 10^{11} \quad (122)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 4.423441749\ 10^7 \quad \textbf{(123)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 3.690633319\ 10^6 \quad \textbf{(124)}$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 4.442789930\ 10^7 \qquad (125)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 5.476970087\ 10^7 \qquad (126)$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

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

$$M_{Ed,new} := 44.23441749 \qquad (127)$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \qquad (128)$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \qquad (129)$$

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

$$P_{Ed} := 115.5229767 \qquad (130)$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 2.877234058 \tag{131}$$

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

$$\sigma_{m,1} := 20.45609928 \tag{132}$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -23.33333334 \tag{133}$$

Verification of the top section

> $Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{top,c} := -1.000000000 \tag{134}$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 17.57886522 \tag{135}$$

> $Ver_{bottom,\ c} := \frac{\sigma_{b,\ c}}{\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c}};$ #<1.0 #--->NOT OK

$$Ver_{bottom,c} := 11.98558992 \tag{136}$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_2 := 1.438617030 \tag{137}$$

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

$$\sigma_{m,2} := 19.13054403 \quad (138)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -20.56916106 \quad (139)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 17.69192700 \quad (140)$$

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

$$Ver_{timber} := -0.5101668613 \quad (141)$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 5.995378759 \quad (142)$$

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

$$Ver_{shear} := 2.154589241 \quad (143)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 27.05115118 \quad (144)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 27.05115118 \quad (145)$$

> $f_{tens,\,k} := 20.0$; #kN

(146)

$$f_{tens,k} := 20.0 \quad (146)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; # <1.0 #-->OK

$$Ver_{F1} := 0.6481004970 \quad (147)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 113.2933254 \quad (148)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 11.18414502 \quad (149)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 1.1146021 \quad (150)$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 6710.487012 \quad (151)$$

> $I_{1,\,eff} := \dfrac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 69948.49480 \quad (152)$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 4.348484069\ 10^{11} \quad (153)$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

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

$$\sigma_1 := 5.525939120 \quad (154)$$

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

$$\sigma_{m,1} := 5.525939120 \quad (155)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -11.05187824 \quad (156)$$

Verification of the top section

> $Ver_{top, c} := \frac{\sigma_{c,t}}{\frac{f_{ck, c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{top,c} := -0.4736519246 \quad (157)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 0. \quad (158)$$

> $Ver_{bottom, c} := \frac{\sigma_{b, c}}{\frac{f_{ctk, 0.05, c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{bottom,c} := 0. \quad (159)$$

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

$$\sigma_2 := 0.5150242297 \quad (160)$$

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

$$\sigma_{m,2} := 27.72420200 \quad (161)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2};$ #*MPa*

$$\sigma_{t,t} := -28.23922623 \quad (162)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2};$ #*MPa*

$$\sigma_{b,t} := 27.20917777 \quad (163)$$

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

$$Ver_{timber} := -0.562776943 \quad (164)$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3;$ #*MPa*

$$\tau_2 := 2.253594862 \quad (165)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ #<1.0 #-->OK

$$Ver_{shear} := 0.8098856537 \quad (166)$$

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*

$$F_1 := 9.684299107 \quad (167)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed};$ #*kN*

$$F_2 := 9.684299587 \quad (168)$$

> $f_{tens,\ k} := 20.0;$ #*kN*

$$f_{tens,k} := 20.0 \tag{169}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}};$ # *<1.0 #--> NOT OK*

$$Ver_{F1} := 0.2320196660 \tag{170}$$

>

>

Appendix B. Maximum deflection for Slab A and Slab B based on SLS verification

B.1 Maximum deflection for Slab A (CTC screws) based on SLS verifications

B.2 Maximum deflection for Slab B (KOP screws) based on SLS verifications

Maximum deflection predictions based on SLS CTC-screws 7-160 mm

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$ *#mm*

$$b := 600 \quad (1)$$

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

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

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

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

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

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

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

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

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

> $h_1 := 30$: #mm

> $h_2 := 20$: #mm

> $h_3 := 20$: #mm

> $h_4 := 20$: #mm

> $h_5 := 30$: #mm

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

$$h_t := 120 \tag{2}$$

> $\gamma_M := 1.15$: #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

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

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

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

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

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

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

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

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

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

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}$; #$\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \tag{3}$$

Lamellae 2, 3 and 4, Class T15

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

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

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

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

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

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

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

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

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

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m^3}$

$$\rho_{t15} := 4.216853302 \quad (4)$$

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

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

$$g_{0,k} := 1.217734993 \qquad \textbf{(5)}$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k}\cdot\gamma_{G,\,2}:\#\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

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

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1;\#mm\hat{\ }2$

$$A_1 := 18000 \qquad \textbf{(6)}$$

> $A_5 := A_1;\#mm\hat{\ }2$

$$A_5 := 18000 \qquad \textbf{(7)}$$

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

$$I_{t1} := 1350000 \qquad \textbf{(8)}$$

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

$$I_{t5} := 1350000 \qquad \textbf{(9)}$$

Layer 2, 3 and 4 (T15)

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

$$A_2 := 12000 \qquad (10)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \qquad (11)$$

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

$$A_4 := 12000 \qquad (12)$$

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

$$I_{t2} := 400000 \qquad (13)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \qquad (14)$$

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

$$I_{t4} := 400000 \qquad (15)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

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

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

$$z_1 := 45 \qquad (16)$$

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

$$z_2 := 20 \qquad (17)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \qquad (18)$$

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

$$z_4 := 20 \qquad (19)$$

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

$$z_5 := 45 \quad (20)$$

Ei*Ii

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

$$(EI)_1 := 17550000000 \quad (21)$$

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

$$(EI)_2 := 92000000 \quad (22)$$

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

$$(EI)_3 := 4600000000 \quad (23)$$

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

$$(EI)_4 := 92000000 \quad (24)$$

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

$$(EI)_5 := 17550000000 \quad (25)$$

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

$$(EI)_{sum} := 39884000000 \quad (26)$$

Ei*Ao*zi^2

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

$$(EAz\hat{}2)_1 := 473850000000 \quad (27)$$

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

$$(EAz\hat{}2)_2 := 1104000000 \quad (28)$$

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

$$(EAz\hat{}2)_3 := 0 \quad (29)$$

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

$$(EAz\hat{}2)_4 := 1104000000 \quad (30)$$

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

$$(EAz\hat{}2)_5 := 473850000000 \quad (31)$$

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

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (32)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

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

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (33)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

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

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

$$a := 90 \quad \textbf{(34)}$$

> $(GA)_{eff} := evalf\left((a\hat{}2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^{6} \quad \textbf{(35)}$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

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

> $K_s := 11.5$
> #CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad \textbf{(36)}$$

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

$$EI_{app} := 7.260572161\ 10^{11} \quad \textbf{(37)}$$

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

$$E_{CLT} := 8403.440000 \quad \textbf{(38)}$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

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

General data:

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

$$E_1 := 34000 \quad \textbf{(39)}$$

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

$$E_2 := 8403.440000 \quad \textbf{(40)}$$

> $h_1 := h_c$; #mm

$$h_1 := 60 \quad \textbf{(41)}$$

> $h_2 := h_t$; #mm

$$h_2 := 120 \quad \textbf{(42)}$$

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

$$A_1 := 36000 \quad \textbf{(43)}$$

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

$$A_2 := 72000 \quad \textbf{(44)}$$

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

$$I_1 := 10800000 \quad \textbf{(45)}$$

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

$$I_2 := 86400000 \quad \textbf{(46)}$$

The slip modulus **Kser**

EC5: 2.2.2(2) Ultimate limit states equation 2.1
The values for the slip modulus (Kser) is found in the pdf about the screw types from the company
Rotho blaas page 227
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

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

$$l_{eff,ctc} := 110 \tag{47}$$

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

$$K_{ser} := 23100 \tag{48}$$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen

> $angle := 45;$

$$angle := 45 \tag{49}$$

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

$$k := \frac{\sqrt{2}}{2} \tag{50}$$

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

$$s_{min,1} := 91.92388153 \tag{51}$$

> $s := 150;\ \#mm$

$$s := 150 \tag{52}$$

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.04851770613 \tag{53}$$

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

$$\gamma_2 := 1.0 \tag{54}$$

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

$$a_2 := 8.044012970 \tag{55}$$

>

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

(56)

$$a_1 := 81.95598703 \quad (56)$$

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

$$EI_{eff,tot} := 1.531288241\ 10^{12} \quad (57)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 3.092990044\ 10^7 \quad (58)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 2.538286756\ 10^6 \quad (59)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 5.433818938\ 10^7 \qquad (60)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

(61)

$$M_4 := 7.914881092\ 10^7 \quad (61)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

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

$$M_{Ed,new} := 30.92990044 \quad (62)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (63)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (64)$$

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

$$P_{Ed} := 80.04426452 \quad (65)$$

3.4 Verification of the vertical defelction

> $w := \dfrac{5 \cdot \left(\dfrac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}}$;

$$w := 5.610726172 \quad (66)$$

> $w_{\lim} := evalf\left(\dfrac{L}{250}\right)$;

$$w_{\lim} := 8. \quad (67)$$

Verification of the vertical deflection

> $Ver_{deflection} := \dfrac{w}{w_{\lim}}$; #<1.0 OK

$$Ver_{deflection} := 0.7013407715 \quad (68)$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \dfrac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \quad (69)$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (70)$$

> $q_k := 0;$

$$q_k := 0 \quad (71)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (72)$$

> $E_{1,fin} := \dfrac{E_{1,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{1,fin} := 9714.285715 \quad (73)$$

4.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (74)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (75)$$

> $E_{2,fin} := \dfrac{E_{2,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4542.399998 \quad (76)$$

4.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1+k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (77)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1+k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad (78)$$

> $$K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$$

$$K_{ser,2} := 12486.48649 \quad (79)$$

> $K_{u,fin} := K_{ser,2}$

$$K_{u,fin} := 12486.48649 \quad (80)$$

5. Long-term-verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $$\gamma_{1,fin} := evalf\left(\frac{1}{1 + \frac{\pi^2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L^2}}\right);$$

$$\gamma_{1,fin} := 0.08798300593 \quad (81)$$

> $\gamma_{2,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad (82)$$

> $$a_{2,fin} := \frac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}; \#mm$$

$$a_{2,fin} := 7.739055957 \quad (83)$$

>

> $$a_{1,fin} := \frac{(h_1 + h_2)}{2} - a_{2,fin}; \#mm$$

$$a_{1,fin} := 82.26094404 \quad (84)$$

> $$EI_{eff,tot,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,fin}^2 + E_{2,fin} \cdot I_2 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2 \cdot a_{2,fin}^2; \#Nmm^2$$

$$EI_{eff,tot,fin} := 7.251748382\ 10^{11} \quad (85)$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $$\sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10^6 : \#MPa$$

> $\sigma_{m,\,1} := \dfrac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **CONCRETE** section

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

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

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

$$M_1 := 4.677646127\ 10^7 \qquad \textbf{(86)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

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

$$M_2 := 4.809994277\ 10^6 \qquad \textbf{(87)}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \dfrac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,2} := \dfrac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

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

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

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

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\ fin} \cdot a_{2,\ fin} \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_{2,\ fin} \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

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

$$M_3 := 4.789067131\ 10^7 \quad \textbf{(88)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 6.874786977\ 10^7 \quad \textbf{(89)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

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

$$M_{Ed,new} := 46.77646127 \quad \textbf{(90)}$$

> $$L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \quad \textbf{(91)}$$

> $L_{sup} := 2.0;$ #m

$$L_{sup} := 2.0 \quad \textbf{(92)}$$

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

$$P_{Ed,fin} := 122.3017601 \quad \textbf{(93)}$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot,fin}};$

$$w_{permanent} := 17.91769486 \quad \textbf{(94)}$$

>

> $w_{\lim} := evalf\left(\frac{L}{150}\right);$

$$w_{\lim} := 13.33333333 \quad \textbf{(95)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}};$ # $<$1.0 NOT OK

$$Ver_{deflection} := 1.343827115 \quad \textbf{(96)}$$

>

Maximum deflection predictions based on SLS KOP-screws 10-140 mm

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000 : \#mm\ span\ length\ between\ the\ supports$

b(c) = b(t) = 600 mm

> $b := 600 : \#mm$

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60 : \#mm$

> $A_c := h_c \cdot b : \#mm\hat{\ }2$

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

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

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

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

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

> $\gamma_c := 1.5 :$

> $\varphi_c := 2.5 :$

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

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

> $h_1 := 30$: #mm

> $h_2 := 20$: #mm

> $h_3 := 20$: #mm

> $h_4 := 20$: #mm

> $h_5 := 30$: #mm

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

$$h_t := 120 \tag{1}$$

> $\gamma_M := 1.15$: #*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

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

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

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

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

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

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

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

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

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

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}$; #$\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \tag{2}$$

Lamellae 2, 3 and 4, Class T15

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

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

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

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

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

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

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

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

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

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

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

$$g_{0,k} := 1.217734993 \qquad (4)$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k}\cdot\gamma_{G,\,2}:\#\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

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

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1;\#mm\hat{}2$

$$A_1 := 18000 \qquad (5)$$

> $A_5 := A_1;\#mm\hat{}2$

$$A_5 := 18000 \qquad (6)$$

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

$$I_{t1} := 1350000 \qquad (7)$$

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

$$I_{t5} := 1350000 \qquad (8)$$

Layer 2, 3 and 4 (T15)

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

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

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

$$A_4 := 12000 \quad (11)$$

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

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

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

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

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

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

$$z_1 := 45 \quad (15)$$

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

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

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

$$z_4 := 20 \quad (18)$$

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

$$z_5 := 45 \quad (19)$$

Ei*Ii

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

$$(EI)_1 := 17550000000 \quad (20)$$

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

$$(EI)_2 := 92000000 \quad (21)$$

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

$$(EI)_3 := 4600000000 \quad (22)$$

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

$$(EI)_4 := 92000000 \quad (23)$$

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

$$(EI)_5 := 17550000000 \quad (24)$$

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

$$(EI)_{sum} := 39884000000 \quad (25)$$

Ei*Ao*zi^2

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

$$(EAz\hat{}2)_1 := 473850000000 \quad (26)$$

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

$$(EAz\hat{}2)_2 := 1104000000 \quad (27)$$

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

$$(EAz\hat{}2)_3 := 0 \quad (28)$$

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

$$(EAz\hat{}2)_4 := 1104000000 \quad (29)$$

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

$$(EAz\hat{}2)_5 := 473850000000 \quad (30)$$

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

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (31)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

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

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (32)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

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

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

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

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

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (35)$$

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

$$EI_{app} := 7.260572161\ 10^{11} \quad (36)$$

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

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

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

General data:

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

$$E_1 := 34000 \quad (38)$$

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

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c; \#mm$

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t; \#mm$

$$h_2 := 120 \quad (41)$$

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

$$A_1 := 36000 \quad (42)$$

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

$$A_2 := 72000 \quad (43)$$

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

$$I_1 := 10800000 \quad (44)$$

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

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22};$

$$\rho_{m,1} := 470 \tag{46}$$

> $\rho_{m,\,2} := t_{t15};$

$$\rho_{m,2} := 430 \tag{47}$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right);$

$$\rho_m := 449.5553359 \tag{48}$$

> $d_{kop} := 10$;#mm

$$d_{kop} := 10 \tag{49}$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m\hat{}1.5 \cdot 3\right);$

$$K_{ser} := 12432.77728 \tag{50}$$

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;$ #mm

$$s := 100 \tag{51}$$

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.03953909054 \tag{52}$$

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

$$\gamma_2 := 1.0 \tag{53}$$

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

$$a_2 := 6.665650562 \tag{54}$$

>

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

$$a_1 := 83.33434944 \quad (55)$$

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

$$EI_{eff,tot} := 1.456230493\ 10^{12} \quad (56)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 3.001576624\ 10^7 \quad (57)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 2.352282428\ 10^6 \quad (58)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 5.310086235\ 10^7 \qquad \textbf{(59)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 7.243560277\ 10^7 \quad \textbf{(60)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

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

$$M_{Ed,new} := 30.01576624 \quad \textbf{(61)}$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \quad \textbf{(62)}$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \quad \textbf{(63)}$$

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

$$P_{Ed} := 77.60657332 \quad \textbf{(64)}$$

3.4 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}};$$

$$w := 5.725544743 \quad \textbf{(65)}$$

$$> w_{\lim} := evalf\left(\frac{L}{250}\right);$$

$$w_{\lim} := 8. \quad \textbf{(66)}$$

Verification of the vertical deflection

$$> Ver_{deflection} := \frac{w}{w_{\lim}}; \# < 1.0\ OK$$

$$Ver_{deflection} := 0.7156930929 \quad \textbf{(67)}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \quad (68)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (69)$$

> $q_k := 0;$

$$q_k := 0 \quad (70)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (71)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{1,fin} := 9714.285715 \quad (72)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (73)$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (74)$$

> $E_{2,fin} := \frac{E_{2,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4542.399998 \quad (75)$$

4.1.3 Slip modulus

>

> $K_{ser,\,g} := \dfrac{K_{ser}}{1 + k_{def,\,t}};$

$$K_{ser,g} := 6720.420151 \quad \textbf{(76)}$$

> $K_{ser,\,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 8724.755986 \quad \textbf{(77)}$$

> $K_{ser,\,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}};$

$$K_{ser,2} := 6720.420149 \quad \textbf{(78)}$$

> $K_{u,\,fin} := K_{ser,\,2}$

$$K_{u,fin} := 6720.420149 \quad \textbf{(79)}$$

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1,\,fin} := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{\ }2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,\,fin} \cdot L\hat{\ }2}}\right);$

$$\gamma_{1,fin} := 0.07225561973 \quad \textbf{(80)}$$

> $\gamma_{2,\,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad \textbf{(81)}$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)};$ #mm

$$a_{2,fin} := 6.454880643 \quad \textbf{(82)}$$

>

> $a_{1,\,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin};$ #mm

$$a_{1,fin} := 83.54511936 \quad \textbf{(83)}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}\hat{\ }2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}\hat{\ }2;$
#Nmm^2

$$EI_{eff,tot,fin} := 6.873754563\ 10^{11} \quad \textbf{(84)}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_1 := \frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,1} := \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **CONCRETE** section

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

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}} + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1\right)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 4.581588411\ 10^7 \qquad \textbf{(85)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(- \frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}} + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1\right)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 4.330780896\ 10^6 \qquad \textbf{(86)}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

> $\sigma_{m,\,2} := \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

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

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

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

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

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

$$M_3 := 4.656652667\ 10^7 \tag{87}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 6.2891909\ 10^7 \tag{88}$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

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

$$M_{Ed,new} := 45.81588411 \quad (89)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (90)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (91)$$

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

$$P_{Ed,fin} := 119.7402210 \quad (92)$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 18.51482237 \quad (93)$$

>

> $w_{\lim} := evalf\left(\frac{L}{150}\right)$;

$$w_{\lim} := 13.33333333 \quad (94)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}}$; # < 1.0 NOT OK

$$Ver_{deflection} := 1.388611678 \quad (95)$$

>

Appendix C. Rothoblaas verification of slab, Excel spreadsheet

C.1 Spreadsheet extract from Excel

rothoblaas
Solutions for Building Technology

LANGUAGE
EN

CTC_calculator
vers.2.0

ETA-19/0244

DIMENSIONING OF A TIMBER-CONCRETE COMPOSIT BEAM with CTC SCREWS

DATA RESULTS PRINT

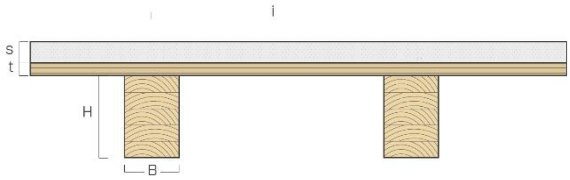
Calculation standard:	EN 1995:2014-1-	RB sales agent:	
Date:	/07/2021	Project:	
Show / Hide All		Floor n°:	

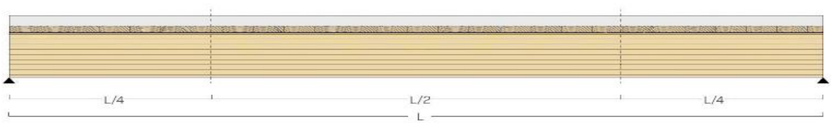
1. DESIGN DATA AND MATERIAL PROPERTIES

Service classes	1
Load-duration class	Long-term variable load
Load case category	Residential and office areas
Service limit state (deformation) [t=0]	L / 400
Service limit state (deformation) [t=∞]	L / 250

1.1 GEOMETRIC DATA

Beam width	B	600	mm
Beam height	H	120	mm
Use of SILENT FLOOR Foil:		NO	
Concrete slab thickness	s	60	mm
Formwork	t	0	mm
Beam span	L	2,10	m
Beam spacing	i	0,60	m





1.2 LOAD ANALYSIS

Uniformly distributed load on span:

LOADS ACTING ON FLOOR SURFACE	**[kN/m^2]**		
Dead load	$g_{1,k}$	1,97	kN/m^2
Permanent non-structural load	$g_{2,k}$	2,00	kN/m^2
Live load	q_k	2,00	kN/m^2
LOADS ON COMPOSIT BEAM	**[kN/m]**		
Beam spacing	i	0,60	m
Effective width	i_{eff}	0,60	m
Dead load	$G_{1,k}$	1,18	kN/m
Permanent non-structural load	$G_{2,k}$	1,20	kN/m
Live load	Q_k	1,20	kN/m

1.3 TIMBER

Timber strength class: Personalizied material

Production subject to continuous control (COV < 15%) ✓

Personalizied material

CLICK HERE →

Bending strength	$f_{m,g,k}$	30,50	N/mm^2
Tensile strength along the grain	$f_{t,0,g,k}$	22,00	N/mm^2
Tensile strength perpendicular to the grain	$f_{t,90,g,k}$	-	N/mm^2
Compressive strength along the grain	$f_{c,0,g,k}$	26,00	N/mm^2
Compressive strength perpendicular to the grain	$f_{c,90,g,k}$	0,70	N/mm^2
Shear	$f_{v,g,k}$	4,00	N/mm^2
Mean value of modulus of elasticity along the grain	$E_{0.g.mean}$	13000	N/mm^2
Characteristic value of modulus of elasticity along the grain	$E_{0.g.0.5}$	0	N/mm^2
Mean value of modulus of elasticity perpendicular the grain	$E_{90,g,mean}$	430	N/mm^2
Mean value of shear modulus	$G_{g,mean}$	720	N/mm^2
5 percentile density	$\rho_{g,k}$	470	kg/m^3
Mean density	ρ_{mean}	390	kg/m^3
Timber Safety factor	$\gamma_{m,\ timber}$	1,15	
Connection Safety factor	$\gamma_{m,\ connection}$	1,30	
Deformation factor	k_{def}	0,60	
Modification factor	k_{mod}	0,70	
Combination factor	ψ_2	0,30	

1.4 CONCRETE SLAB

Concrete strength class: Personalizied material

Personalizied material

CLICK HERE →

Welded steel mesh selection **B450C** Ø6 15x15

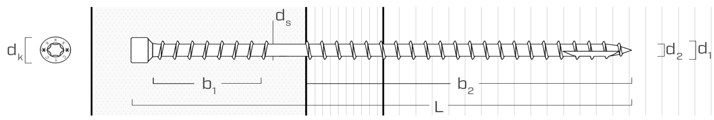
Characteristic cubic compression strength	$R_{c,k}$	37	N/mm^2
Characteristic cylindrical compression strength	$f_{c,k}$	35	N/mm^2
Design compression strenght	$f_{c,d}$	19,83	N/mm^2
Characteristic cylindrical simple tensile strength (axial)	$f_{ct,m}$	3,21	N/mm^2
Charecteristic tensile strength	$f_{ct,k,0.005}$	2,70	N/mm^2
Design tensile strength	$f_{c,t,d}$	1,80	N/mm^2
Mean secant value of modulus of elasticty	$E_{c,m}$	34000	N/mm^2
Density	ρ_k	2500	kg/m^3
Viscosity coefficient	φ	2,5	

1.5 FASTENERS TYPE

Wood-concrete fastener type: CTC7160

N° fastener rows: 3

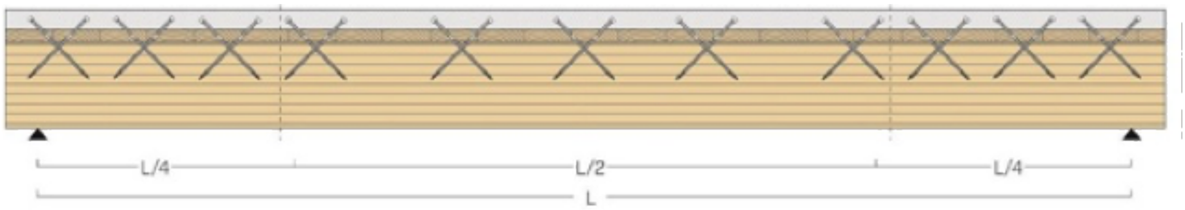
Connector Arrangement: Inclined crossed fasteners 4



Diameter	d_1	7	mm
Inner thread diameter	d_2	4,6	mm
Head diameter	d_k	9,5	mm
Length	L	160	mm
Thread lenght (head side)	b_1	40	mm
Thread lenght	b_2	110	mm
Withdrawal charecteristic parameter	$f_{ax,k}$	11,3	N/mm^2
Associated density	$\rho_{a,ax}$	350	kg/m^3
Steel tensile strength	$f_{tens,k} = R_{t,u,k}$	20,0	kN
Connection striffness	K	7700	N/mm
-Service limit state	K_{ser}	7700	N/mm
-Ultimate limit state connection strenght	K_u	5133	N/mm
Characteristic load-carryinng resistance	$R_{v,k}$	14142	N
Min spacing	s_{min}	100	mm
Max spacing (central beam part)	s_{max}	360	mm
Equivalent spacing	s_{eq}	165	mm

FASTENER DESIGN SHEAR RESISTANCE	**8 %**	**VERIFIED***
WORST CASE VERIFICATION: CONCRETE - REINFORCEMENT VERIFICATION [t=0]	**38 %**	**VERIFIED***

*Please see points 5 and 6 for more details.



PRINT DATA ARRANGEMENT

ARRANGEMENT * Number of fastener is rounded up by excess	N° fastener rows:		3
	Spacing [mm]	**N° of pairs***	**Distribution sector [m]**
0 ≤ x ≤ L/4	100	**18**	0,53
L/4 ≤ x ≤ 3/4*L	360	**3**	1,05
3/4*L ≤ x ≤ L	100	**18**	0,53
TOTAL NUMBER OF CONNECTORS CTC PER BEAM:		**78**	**CTC7160**

2. VERIFICATIONS - SUMMARY

2.1 ULTIMATE LIMIT STATE VERIFICATION [t=0]

CONCRETE		
COMPRESSION STRESS	8 %	**VERIFIED**
TENSION STRESS	77 %	**VERIFIED**
REINFORCEMENT		
REINFORCEMENT VERIFICATION	38 %	**VERIFIED**
TIMBER		
COMBINED BENDING AND COMPRESSION STRESS	7 %	**VERIFIED**
SHEAR STRESS	10 %	**VERIFIED**
FASTENER		
FASTENER DESIGN SHEAR RESISTANCE	7 %	**VERIFIED**
Efficiency of composit section	*0,07*	

2.2 ULTIMATE LIMIT STATE VERIFICATION [t=∞]

CONCRETE		
COMPRESSION STRESS	5 %	**VERIFIED**
TENSION STRESS	35 %	**VERIFIED**
REINFORCEMENT		
REINFORCEMENT VERIFICATION	38 %	**VERIFIED**
TIMBER		
COMBINED BENDING AND COMPRESSION STRESS	8 %	**VERIFIED**
SHEAR STRESS	12 %	**VERIFIED**
FASTENER		
FASTENER DESIGN SHEAR RESISTANCE	8 %	**VERIFIED**
Efficiency of composit section	*0,11*	

2.3 SERVICE LIMIT STATE VERIFICATION [t=0]

W INST (CHARACTERISTIC COMBINATION)	$w_{g+q,inst}$	0,47 mm L / 4501	VERIFIED
W LIM (CHARACTERISTIC COMBINATION)	$w_{inst,lim}$	5,25 mm L / 400	

2.4 SERVICE LIMIT STATE VERIFICATION [t=∞]

W INST (CHARACTERISTIC COMBINATION)	$w_{g+q,fin}$	0,75 mm L / 2783	VERIFIED
W LIM (CHARACTERISTIC COMBINATION)	$w_{fin,lim}$	8,40 mm L / 250	

3. LOADS

Combination of actions (permanent and variable)	$q_{,d}$	4,90	kN/m
Max. bending moment	$M_{s,d}$	2,70	kNm
Design shear force	$V_{s,d}$	5,14	kN
	γ_{G1}	1,30	
Combination factor	γ_{G2}	1,30	
	γ_Q	1,50	

4. STIFFNESS

4.1 INITIAL STIFFNESS [t=0]

CONCRETE			
Bending stiffness	$E_c J_c$	3,67E+11	Nmm^2
Axial stiffness	$E_c A_c$	1,22E+09	N
TIMBER			
Bending stiffness	$E_L J_L$	1,12E+12	Nmm^2
Axial stiffness	$E_L A_L$	9,36E+08	N
COMPOSIT SECTION BEAM			
Bending stiffness Deformable fastener	$(E J)_{t=0}$	1,49E+12	Nmm^2
Axial stiffness	$(E A)_{t=0}$	5,30E+08	N
Distance between beam and concrete slab center of gravity	a	90	mm
Composite beam stiffness Infinitely rigid fastener	$(E J)_{t=0}$	5,79E+12	Nmm^2
parameter γ1	γ_1	0,033	
parameter γ2	γ_2	1,0	
Distance between beam and composite section center of gravity	a_2	3,7	mm
Distance between concrete slab and composit beam center of gravity	a_1	86,3	mm
EFFECTIVE BENDING STIFFNESS	$(E J)_{eff}$	1,80E+12	Nmm^2
TIMBER-CONCRETE COMPOSITE SYSTEM EFFICIENCY	η	0,07	

4.2 FINAL STIFFNESS [t=∞]

CONCRETE			
Bending stiffness	$E_{c,\infty} J_c$	1,05E+11	Nmm^2
Axial stiffness	$E_{c,\infty} A_c$	3,50E+08	N
	$E_{c,\infty}$	9,71E+03	N/mm^2
TIMBER			
Bending stiffness	$E_{L,\infty} J_L$	7,02E+11	Nmm^2
Axial stiffness	$E_{L,\infty} A_L$	5,85E+08	N
	$E_{0,mean,\infty}$	8,13E+03	N/mm^2
FASTENER			
Stiffness	$K_{u,\infty}$	3208	N/mm
COMPOSIT SECTION BEAM			
Bending stiffness Deformable fastener	$(E J)_{t=\infty}$	8,06914E+11	Nmm^2
Axial stiffness	$(E A)_{t=\infty}$	2,19E+08	N
Distance between beam and concrete slab center of gravity	a	90	mm
Composite beam stiffness Infinitely rigid fastener	$(E J)_{t=\infty}$	2,58E+12	Nmm^2
parameter γ1	γ_1	0,07	
parameter γ2	γ_2	1,0	
Distance between beam and composite section center of gravity	a_2	3,58	mm
Distance between concrete slab and composit beam center of gravity	a_1	86,42	mm
EFFECTIVE BENDING STIFFNESS	$(E J)_{eff}$	9,96E+11	Nmm^2
TIMBER-CONCRETE COMPOSITE SYSTEM EFFICIENCY	η	0,11	

5. STRENGTH VERIFICATIONS

5.1 STRENGTH VERIFICATIONS [t=0]

5.1.1 CONCRETE

Design compression force in concrete slab	$N_{c,d}$	5206	N
Design Bending moment on concrete slab	$M_{c,d}$	549380	Nmm
Max. compression stress in concrete slab borders	$\sigma_{c,compr}$	1,67	N/mm^2
Max tensile stress in concrete slab borders	$\sigma_{c,tens}$	-1,38	N/mm^2

COMPRESSION STRESS

Max. compression stress in concrete slab borders	$\sigma_{c,compr}$	2	N/mm^2
Design compression strenght	$f_{c,d}$	20	N/mm^2
$\sigma_{c,compr} \leq f_{c,d}$	1,67	≤	19,83
		VERIFIED	**8 %**

TENSION STRESS

Max tensile stress in concrete slab borders	$\sigma_{c,tens}$	-1,38	N/mm^2
Design tensile strength	$f_{c,t,d}$	1,80	N/mm^2
$\sigma_{c,tens} \leq f_{c,t,d}$	1,38	≤	1,80
		VERIFIED	**77 %**

Reinforcement verification

Concrete area	A_c	360	cm^2
Reinforcement area	A_s	1,88	cm^2/m
Required reinforcement	$A_{s,nec}$	0,29	cm^2/m
Minimum reinforcement required by calculation standard (0,002*Ac)	$A_{s,standard}$	0,72	cm^2/m
As ≥ As,standard	1,88	≥	0,72
		VERIFIED	

5.1.2 TIMBER

Design compression force in timber beam	$N_{L,d}$	5206	N
Design Bending moment on timber beam	$M_{L,d}$	1680457	Nmm
Max axial stress	$\sigma_{L,axial}$	0,07	N/mm^2
Max. bending stress	$\sigma_{L,bending}$	1,17	N/mm^2

COMBINED BENDING AND COMPRESSION STRESS

Max axial stress	$\sigma_{L,axial}$	0,07	N/mm^2
Tensile strength along the grain	$f_{t,0,d}$	13,39	N/mm^2
Max. bending stress	$\sigma_{L,bending}$	1,17	N/mm^2
Bending strength	$f_{m,d}$	18,57	Mpa
$\sigma_{L,axial} / f_{t,0,d} + \sigma_{L,bending} / f_{m,d}$	0,07	≤	1,00
		VERIFIED	**7 %**

SHEAR STRESS

Timber: max shear stress	$\tau_{L,max}$	0,25	N/mm^2
Shear strength	$f_{v,d}$	2,43	N/mm^2
$\tau_{L,max} \leq f_{v,d}$	0,25	≤	2,43
		VERIFIED	**10 %**

5.1.3 FASTENER

Fastener load	$F_{s,d}$	545	N
Characteristic load-carryinng resistance	$R_{v,k}$	14142	N
Fastener design shear resistance	$R_{v,d}$	7615	N

FASTENER VERIFICATION

$F_{s,d} \leq R_{v,d}$	545	≤	7615
		VERIFIED	**7 %**

5.2 STRENGTH VERIFICATIONS [t=∞]

5.2.1 CONCRETE

Design compression force in concrete slab	$N_{c,d}$	5682	N
Design Bending moment on concrete slab	$M_{c,d}$	284359	Nmm
Max. compression stress in concrete slab borders	$\sigma_{c,compr}$	0,95	N/mm^2
Max tensile stress in concrete slab borders	$\sigma_{c,tens}$	-0,63	N/mm^2

COMPRESSION STRESS

Max. compression stress in concrete slab borders	$\sigma_{c,compr}$	0,95	N/mm^2
Design compression strenght	$f_{c,d}$	19,83	N/mm^2
$\sigma_{c,compr} \leq f_{c,d}$	0,95	≤	19,83
		VERIFIED	**5 %**

TENSION STRESS

Max. compression stress in concrete slab borders	$\sigma_{c,tens}$	0,63	N/mm^2
Design tensile strength	$f_{c,t,d}$	1,80	N/mm^2
$\sigma_{c,tens} \leq f_{c,t,d}$	0,63	≤	1,80
		VERIFIED	**35 %**

Reinforcement verification

Concrete area	A_c	360	cm^2
Reinforcement area	A_s	1,88	cm^2/m
Required reinforcement	$A_{s,nec}$	0,12	cm^2/m
Minimum reinforcement required by calculation standard (0,002*Ac)	$A_{s,standard}$	0,72	cm^2/m
As ≥ As,standard	1,88	≥	0,72
		VERIFIED	

5.2.2 TIMBER

Design compression force in timber beam	$N_{L,d}$	5682	N
Design Bending moment on timber beam	$M_{L,d}$	1902694	Nmm
Max axial stress	$\sigma_{L,axial}$	0,08	N/mm^2
Max. bending stress	$\sigma_{L,bending}$	1,32	N/mm^2

COMBINED BENDING AND COMPRESSION STRESS

Max axial stress	$\sigma_{L,axial}$	0,08	N/mm^2
Tensile strength along the grain	$f_{t,0,d}$	13,39	N/mm^2
Max. bending stress	$\sigma_{L,bending}$	1,32	N/mm^2
Bending strength	$f_{m,d}$	18,57	Mpa
$\sigma_{L,ax} / f_{t,0,d} + \sigma_{L,bend} / f_{m,d}$	0,08	≤	1,00
		VERIFIED	**8 %**

SHEAR STRESS

Timber: max shear stress	$\tau_{L,max}$	0,28	N/mm^2
Shear strength	$f_{v,d}$	2,43	N/mm^2
$\tau_{L,max} \leq f_{v,d}$	0,28	≤	2,43
		VERIFIED	**12 %**

5.2.3 FASTENER

Fastener load	$F_{s,d}$	595	N
Characteristic load-carryinng resistance	$R_{v,k}$	14142	N
Fastener design shear resistance	$R_{v,d}$	7615	N

FASTENER VERIFICATION

$F_{s,d} \leq R_{v,d}$	595	≤	7615
		VERIFIED	**8 %**

6. SERVICE LIMIT STATE VERIFICATION

6.1 INITIAL STIFFNESS [t=0]

CONCRETE			
Bending stiffness	$E_c J_c$	3,67E+11	Nmm^2
Axial stiffness	$E_c A_c$	1,22E+09	N
TIMBER			
Bending stiffness	$E_L J_L$	1,12E+12	Nmm^2
Axial stiffness	$E_L A_L$	9,36E+08	N
COMPOSIT SECTION BEAM [t=0]			
Bending stiffness Deformable fastener	$(E J)_{t=0}$	1,49E+12	Nmm^2
Axial stiffness	$(E A)_{t=0}$	5,30E+08	N
Distance between beam and concrete slab center of gravity	a	90	mm
REAL COMPOSIT BEAM STIFFNESS [t=0]			
Composite beam stiffness Infinitely rigid fastener	$(E J)_{t=0}$	5,79E+12	Nmm^2
parameter γ1	γ_1	0,0486	
parameter γ2	γ_2	1,0	
Distance between beam and composite section center of gravity	a_2	5,4	mm
Distance between concrete slab and composit beam center of gravity	a_1	84,6	mm
EFFECTIVE BENDING STIFFNESS	$(E J)_{eff}$	1,94E+12	Nmm^2
Timber-concrete composite system efficiency	η	0,11	

6.2 FINAL STIFFNESS [t=∞]

CONCRETE			
Material Stiffness [t=∞]	$E_{c,\infty}$	9714	N/mm^2
Bending stiffness	$E_{c,\infty} J_c$	1,05E+11	Nmm^2
Axial stiffness	$E_{c,\infty} A_c$	3,50E+08	N
TIMBER			
Material Stiffness [t=∞]	$E_{0,mean,\infty}$	8125	N/mm^2
Bending stiffness	$E_{L,\infty} J_L$	7,02E+11	Nmm^2
Axial stiffness	$E_{L,\infty} A_L$	5,85E+08	N
FASTENER			
Slip modulus (Service limit state)	$K_{ser,\infty}$	4813	N/mm
COMPOSIT SECTION BEAM			
Bending stiffness	$(E J)_{t=\infty}$	8,07E+11	Nmm^2
Axial stiffness	$(E A)_{t=\infty}$	2,19E+08	N
Distance between beam and concrete slab center of gravity	a	90	mm
Composite beam stiffness [t=∞]	$(E J)_{t=\infty}$	2,58E+12	Nmm^2
parameter γ1	γ_1	0,10056	
parameter γ2	γ_2	1,0	
Distance between beam and composite section center of gravity	a_2	5,1	mm
Distance between concrete slab and composit beam center of gravity	a_1	84,9	mm
EFFECTIVE BENDING STIFFNESS	$(E J)_{eff}$	1,08E+12	Nmm^2
Timber-concrete composite system efficiency	η	0,15	

6.3 DEFLECTION - SERVICE LIMIT STATE VERIFICATION

Deflection [t=0]			
Deflection due to permanent load	$w_{g,inst}$	0,31	mm
Deflection due to variable load	$w_{q,inst}$	0,16	mm
Total deflection	$w_{g+q,inst}$	0,47	mm
SERVICE LIMIT STATE VERIFICATION [t=0]			
Service limit state (deformation)	$w_{inst,lim}$	L / 400	
	$w_{inst,lim}$	5,25	mm
Total deflection	$w_{g+q,inst}$	0,47	mm
$\mathbf{w_{g+q,inst} \leq w_{inst,lim}}$	0,47	≤	5,25
		VERIFIED	L / 4501
Deflection [t=∞]			
Deflection due to permanent load	$w_{g,fin}$	0,65	mm
Deflection due to variable load	$w_{q,fin}$	0,11	mm
Total deflection	$w_{g+q,fin}$	0,75	mm
SERVICE LIMIT STATE VERIFICATION [t=∞]			
Service limit state (deformation)	$w_{fin,lim}$	L / 250	
	$w_{fin,lim}$	8,40	mm
Total deflection	$w_{g+q,fin}$	0,75	mm
$\mathbf{w_{g+q,inst} \leq w_{inst,lim}}$	0,75	≤	8,40
		VERIFIED	L / 2783

Appendix D. Additional load capacity for Slab A and Slab B based on ULS verifications

For Slab type A (CTC screws)

D.1 Load capacity for Slab A based on ULS verifications using M3

D.2 Load capacity for Slab A based on ULS verifications using M4

D.3 Load capacity for Slab A based on ULS verifications using B45 strength

D.4 Load capacity for Slab A based on ULS verifications using avg. strength

For Slab type B (KOP screws)

D.5 Load capacity for Slab B based on ULS verifications using M3

D.6 Load capacity for Slab B based on ULS verifications using M4

D.7 Load capacity for Slab B based on ULS verifications using B45 strength

D.8 Load capacity for Slab B based on ULS verifications using avg. strength

Load capacity predictions based on ULS CTC-screws 7-160 mm

M3, choosing the moment M3, top part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\# \dfrac{kN}{m\hat{}3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

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

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 :\#mm$

> $h_5 := 30 : \#mm$

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

$$h_t := 120 \quad (1)$$

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

$$\gamma_M := 1.15 \quad (2)$$

> $Klima := 1.0 :$ #*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

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

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

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

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

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

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

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

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

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

> $\rho_{t22} := \frac{t_{t22} \cdot 0.0098066358553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (3)$$

Lamellae 2, 3 and 4, Class T15

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

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

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

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

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

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

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

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

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

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (4)$$

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

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

$$g_{0,k} := 1.217734993 \qquad (5)$$

1. 1 ULS

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

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

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

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1; \#mm\hat{\ }2$

$$A_1 := 18000 \qquad (6)$$

> $A_5 := A_1; \#mm\hat{\ }2$

$$A_5 := 18000 \qquad (7)$$

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

$$I_{t1} := 1350000 \qquad (8)$$

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

$$I_{t5} := 1350000 \qquad (9)$$

Layer 2, 3 and 4 (T15)

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

$$A_2 := 12000 \tag{10}$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \tag{11}$$

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

$$A_4 := 12000 \tag{12}$$

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

$$I_{t2} := 400000 \tag{13}$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \tag{14}$$

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

$$I_{t4} := 400000 \tag{15}$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24

The height to NA

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

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

$$z_1 := 45 \tag{16}$$

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

$$z_2 := 20 \tag{17}$$

> $z_3 := 0$; #mm

$$z_3 := 0 \tag{18}$$

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

$$z_4 := 20 \tag{19}$$

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

$$z_5 := 45 \quad (20)$$

Ei*Ii

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

$$(EI)_1 := 17550000000 \quad (21)$$

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

$$(EI)_2 := 92000000 \quad (22)$$

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

$$(EI)_3 := 4600000000 \quad (23)$$

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

$$(EI)_4 := 92000000 \quad (24)$$

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

$$(EI)_5 := 17550000000 \quad (25)$$

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

$$(EI)_{sum} := 39884000000 \quad (26)$$

Ei*Ao*zi^2

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

$$(EAz\hat{}2)_1 := 473850000000 \quad (27)$$

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

$$(EAz\hat{}2)_2 := 1104000000 \quad (28)$$

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

$$(EAz\hat{}2)_3 := 0 \quad (29)$$

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

$$(EAz\hat{}2)_4 := 1104000000 \quad (30)$$

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

$$(EAz\hat{}2)_5 := 473850000000 \quad (31)$$

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

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (32)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

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

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (33)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

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

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

$$a := 90 \quad (34)$$

> $(GA)_{eff} := evalf\left((a\hat{}2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right);$ #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (35)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

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

> $K_s := 11.5$
> #CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (36)$$

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

$$EI_{app} := 7.260572161\ 10^{11} \quad (37)$$

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

$$E_{CLT} := 8403.440000 \quad \textbf{(38)}$$

3. Gamma method, Annex B, EC5

Maximum load capacity based on ULS using short-term verifications of the slab

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

General data:

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

$$E_1 := 34000 \quad \textbf{(39)}$$

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

$$E_2 := 8403.440000 \quad \textbf{(40)}$$

> $h_1 := h_c; \# mm$

$$h_1 := 60 \quad \textbf{(41)}$$

> $h_2 := h_t; \# mm$

$$h_2 := 120 \quad \textbf{(42)}$$

> $A_1 := A_c; \# mm\hat{}2$

$$A_1 := 36000 \quad \textbf{(43)}$$

> $A_2 := h_2 \cdot b; \# mm\hat{}2$

$$A_2 := 72000 \quad \textbf{(44)}$$

> $I_1 := I_c; \# mm\hat{}4$

$$I_1 := 10800000 \quad \textbf{(45)}$$

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

$$I_2 := 86400000 \quad \textbf{(46)}$$

The slip modulus **Kser**

EC5: 2.2.2(2) Ultimate limit states equation 2.1

The values for the slip modulus (Kser) is found in the pdf about the screw types from the company

Rotho blaas page 227
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

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

$$l_{eff,ctc} := 110 \tag{47}$$

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

$$K_{ser} := 23100 \tag{48}$$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right);$ # $\frac{N}{mm}$ #equation 2.1 EC5

$$K_u := 15400. \tag{49}$$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen.

> $angle := 45;$

$$angle := 45 \tag{50}$$

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

$$k := \frac{\sqrt{2}}{2} \tag{51}$$

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

$$s_{min,1} := 91.92388153 \tag{52}$$

> $s := 150;$ #mm

$$s := 150 \tag{53}$$

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.03287684039 \tag{54}$$

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

$$\gamma_2 := 1.0 \tag{55}$$

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

$$a_2 := 5.612544326 \tag{56}$$

>

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

$$a_1 := 84.38745567 \quad (57)$$

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

$$EI_{eff,tot} := 1.398884340\ 10^{12} \quad (58)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 2.929172990\ 10^7 \quad (59)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 2.216443886\ 10^6 \quad (60)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 5.210849666\ 10^7 \tag{61}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 6.763762779\ 10^7 \quad (62)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{M_3}{10\hat{}6};$ #kNm

$$M_{Ed,new} := 52.10849666 \quad (63)$$

> $L_{out} := 0.75;$ #m

$$L_{out} := 0.75 \quad (64)$$

> $L_{sup} := 2.0;$ #m

$$L_{sup} := 2.0 \quad (65)$$

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

$$P_{Ed} := 136.5205211 \quad (66)$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 3.513772329 \quad (67)$$

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

$$\sigma_{m,1} := 37.99504010 \quad (68)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -41.50881243 \quad (69)$$

Verification of the top section

> $Ver_{top,\, c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\, c}}{\gamma_c}}$; # <1.0 #--- >OK

$$Ver_{top,c} := -1.778949104 \quad \textbf{(70)}$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 34.48126777 \quad \textbf{(71)}$$

> $Ver_{bottom,\, c} := \dfrac{\sigma_{b,\, c}}{\dfrac{f_{ctk,\, 0.05,\, c}}{\gamma_c}}$; # <1.0 #-- > NOT OK

$$Ver_{bottom,c} := 23.50995530 \quad \textbf{(72)}$$

3.4.2 Normal stresses in the **TIMBER** section

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

$$\sigma_2 := 1.756886165 \quad \textbf{(73)}$$

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

$$\sigma_{m,2} := 18.78170822 \quad \textbf{(74)}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\, t} := -\sigma_2 - \sigma_{m,\, 2}$; #MPa

$$\sigma_{t,t} := -20.53859438 \quad \textbf{(75)}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\, t} := -\sigma_2 + \sigma_{m,\, 2}$; #MPa

$$\sigma_{b,t} := 17.02482206 \quad \textbf{(76)}$$

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

$$Ver_{timber} := -0.5396110274 \quad \textbf{(77)}$$

3.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3;$ *#MPa*

$$\tau_2 := 5.904807295 \quad \textbf{(78)}$$

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*

$$Ver_{shear} := 2.122040122 \quad \textbf{(79)}$$

3.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_1 := 49.71148901 \quad \textbf{(80)}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_2 := 49.71148901 \quad \textbf{(81)}$$

> $f_{tens,\,k} := 20.0;$ *#kN*

$$f_{tens,k} := 20.0 \quad \textbf{(82)}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}};$ *# <1.0 #-->NOT OK*

$$Ver_{F1} := 1.191004424 \quad \textbf{(83)}$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right);$ *#mm*

$$a_{1,eff} := 115.2431566 \quad \textbf{(84)}$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\ eff}$; #mm

$$x := 7.577661732 \quad \textbf{(85)}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\ new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\ eff}$;

$$a_{2,new} := 0.9680125 \quad \textbf{(86)}$$

> $A_{1,\ eff} := b \cdot x$;

$$A_{1,eff} := 4546.597039 \quad \textbf{(87)}$$

> $I_{1,\ eff} := \frac{b \cdot x\hat{\ }3}{12}$

$$I_{1,eff} := 21755.82954 \quad \textbf{(88)}$$

> $EI_{eff,\ tot,\ new} := E_1 \cdot I_{1,\ eff} + \gamma_1 \cdot E_1 \cdot A_{1,\ eff} \cdot a_{1,\ eff}\hat{\ }2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\ new}\hat{\ }2$; #Nmm^2

$$EI_{eff,tot,new} := 7.948610712\ 10^{11} \quad \textbf{(89)}$$

5. New short-term verification

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 8.445034962 \quad \textbf{(90)}$$

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

$$\sigma_{m,1} := 8.445034965 \quad \textbf{(91)}$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -16.89006993 \quad \textbf{(92)}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.7238601399 \quad \textbf{(93)}$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 3.\ 10^{-9} \quad \textbf{(94)}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom,c} := 2.045454546\ 10^{-9} \quad \textbf{(95)}$$

5.1.2 Normal stresses in the **TIMBER** section

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

$$\sigma_2 := 0.5332801091 \quad \textbf{(96)}$$

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

$$\sigma_{m,2} := 33.05412538 \quad \textbf{(97)}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -33.58740549 \quad \textbf{(98)}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 32.52084527 \quad \textbf{(99)}$$

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

$$Ver_{timber} := -0.661886852 \quad \textbf{(100)}$$

5.1.3 Shear stresses in the **TIMBER** section

$$> \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^{\wedge}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^{\wedge}3; \#MPa$$

$$\tau_2 := 2.682488649 \quad \textbf{(101)}$$

Verification of the timber section

$$> Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \# < 1.0 \# {-}{-}> OK$$

$$Ver_{shear} := 0.9640193582 \quad \textbf{(102)}$$

5.1.4 The **load per shear fasteners**

$$> F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_{1,eff} \cdot a_{1,eff} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_1 := 15.08928179 \quad \textbf{(103)}$$

$$> F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}; \#kN$$

$$F_2 := 15.08928058 \quad \textbf{(104)}$$

$$> f_{tens,\,k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \quad \textbf{(105)}$$

$$> Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}; \# < 1.0 \# {-}{-}> OK$$

$$Ver_{F1} := 0.3615140430 \quad \textbf{(106)}$$

6. Maximum load capacity based on ULS using long-term verificatons of the slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

$$> E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c};$$

(107)

$$E_{1,g} := 9714.285714 \quad (107)$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (108)$$

> $q_k := 0;$

$$q_k := 0 \quad (109)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (110)$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285715 \quad (111)$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (112)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (113)$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad (114)$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1+k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (115)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1+k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad (116)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad (117)$$

> $K_u := \frac{2}{3} \cdot K_{ser,\,2}$

$$K_u := 8324.324327 \quad (118)$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.06042753609 \quad (119)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (120)$$

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

$$a_2 := 5.462362231 \quad (121)$$

>

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

$$a_1 := 84.53763777 \quad (122)$$

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

$$EI_{eff,tot} := 6.581609231\ 10^{11} \quad (123)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

$$\sigma_1 := 0.07539868541\ M_{Ed,1} \quad (124)$$

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

$$\sigma_{m,1} := 0.4427922734\ M_{Ed,1} \quad (125)$$

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 4.502844547\ 10^7 \quad (126)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 3.992085640\ 10^6 \quad (127)$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

$$\sigma_2 := 0.03769934270\ M_{Ed,2} \quad (128)$$

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

$$\sigma_{m,2} := 0.4140993340\ M_{Ed,2} \quad (129)$$

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 4.549531312\ 10^7 \tag{130}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 5.863841321\ 10^7 \tag{131}$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

$$> M_{Ed,\,new} := \frac{M_3}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 45.49531312 \tag{132}$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \tag{133}$$

> $L_{sup} := 2.0;\ \#m$

$$L_{sup} := 2.0 \qquad (134)$$

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

$$P_{Ed} := 118.8853650 \qquad (135)$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 3.430286802 \qquad (136)$$

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

$$\sigma_{m,1} := 20.14497313 \qquad (137)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -23.57525993 \qquad (138)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};\ \#<1.0\ \#--->OK$

$$Ver_{top,c} := -1.010368283 \qquad (139)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 16.71468633 \qquad (140)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};\ \#\ 1.0\ --->\ NOT\ OK$

$$Ver_{bottom,c} := 11.39637704 \qquad (141)$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{\left(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new}\right)}{EI_{eff,\ tot}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_2 := 1.715143401 \quad \textbf{(142)}$$

> $\sigma_{m,\ 2} := \frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new}\right)}{EI_{eff,tot}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_{m,2} := 18.83957886 \quad \textbf{(143)}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2};$ #*MPa*

$$\sigma_{t,t} := -20.55472226 \quad \textbf{(144)}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2};$ #*MPa*

$$\sigma_{b,t} := 17.12443546 \quad \textbf{(145)}$$

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

$$Ver_{timber} := -0.5359699440 \quad \textbf{(146)}$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3;$ #*MPa*

$$\tau_2 := 5.907642054 \quad \textbf{(147)}$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ #*MPa # 1.0 --- > NOT OK*

$$Ver_{shear} := 2.123058863 \quad \textbf{(148)}$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ #*kN*

$$F_1 := 48.40452127 \quad \textbf{(149)}$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff, tot}} \cdot P_{Ed}$; #kN

$$F_2 := 48.40452127 \quad (150)$$

> $f_{tens,\ k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (151)$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}}$; # <1.0 #-->OK

$$Ver_{F1} := 1.159691656 \quad (152)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\ eff} := \max\left(solve\left(a_{1,\ 1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\ 1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\ 1}\right)\right)$; #mm

$$a_{1,eff} := 111.6324251 \quad (153)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\ eff}$; #mm

$$x := 13.49134479 \quad (154)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\ new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\ eff}$;

$$a_{2,new} := 1.6219025 \quad (155)$$

> $A_{1,\ eff} := b \cdot x$;

$$A_{1,eff} := 8094.806874 \quad (156)$$

> $I_{1,\ eff} := \dfrac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 122782.2898 \quad (157)$$

> $EI_{eff,\ tot,\ new} := E_1 \cdot I_{1,\ eff} + \gamma_1 \cdot E_1 \cdot A_{1,\ eff} \cdot a_{1,\ eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\ new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 4.537316051\ 10^{11} \quad (158)$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 6.570580576 \quad (159)$$

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

$$\sigma_{m,1} := 6.570580575 \quad (160)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -13.14116115 \quad (161)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$ # <1.0 #---> OK

$$Ver_{top,c} := -0.5631926206 \quad (162)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := -1.\,10^{-9} \quad (163)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};$ # <1.0 #---> OK

$$Ver_{bottom,c} := -6.818181818\,10^{-10} \quad (164)$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{\left(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_2 := 0.7387164075 \quad (165)$$

> $\sigma_{m,\,2} := \frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_{m,2} := 27.32777368 \quad (166)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #*MPa*

$$\sigma_{t,t} := -28.06649009 \quad (167)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #*MPa*

$$\sigma_{b,t} := 26.58905727 \quad (168)$$

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*

$$Ver_{timber} := -0.580717197 \quad (169)$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3;$ #*MPa*

$$\tau_2 := 2.259721658 \quad (170)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #<*1.0* #-->*OK*

$$Ver_{shear} := 0.8120874707 \quad (171)$$

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*

$$F_1 := 20.84794407 \quad (172)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed};$ #kN

$$F_2 := 20.84794429 \tag{173}$$

> $f_{tens,\,k} := 20.0;$ #kN

$$f_{tens,k} := 20.0 \tag{174}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}};$ #<1.0 #-->OK

$$Ver_{F1} := 0.4994819933 \tag{175}$$

>

>

Load capacity predictions based on ULS
CTC-screws 7-160 mm

M4, choosing the moment M5, bottom part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

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

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

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

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

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

$$h_t := 120 \tag{1}$$

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

> $Klima := 1.0 : \#Serice\ class,\ permanent$

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

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

Lamellae 1 and 5, Class T22

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

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

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

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

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

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

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

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

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

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \tag{2}$$

Lamellae 2, 3 and 4, Class T15

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

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

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

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

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

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

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

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

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

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{\ }3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

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

$$g_{0,k} := 1.217734993 \quad (4)$$

1. 1 ULS

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

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

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

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1; \#mm\hat{}2$

$$A_1 := 18000 \quad (5)$$

> $A_5 := A_1; \#mm\hat{}2$

$$A_5 := 18000 \quad (6)$$

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

$$I_{t1} := 1350000 \quad (7)$$

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

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

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

$$A_2 := 12000 \tag{9}$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \tag{10}$$

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

$$A_4 := 12000 \tag{11}$$

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

$$I_{t2} := 400000 \tag{12}$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \tag{13}$$

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

$$I_{t4} := 400000 \tag{14}$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

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

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

$$z_1 := 45 \tag{15}$$

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

$$z_2 := 20 \tag{16}$$

> $z_3 := 0$; #mm

$$z_3 := 0 \tag{17}$$

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

$$z_4 := 20 \tag{18}$$

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

$$z_5 := 45 \tag{19}$$

Ei*Ii

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

$$(EI)_1 := 17550000000 \tag{20}$$

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

$$(EI)_2 := 92000000 \tag{21}$$

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

$$(EI)_3 := 4600000000 \tag{22}$$

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

$$(EI)_4 := 92000000 \tag{23}$$

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

$$(EI)_5 := 17550000000 \tag{24}$$

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

$$(EI)_{sum} := 39884000000 \tag{25}$$

Ei*Ao*zi^2

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

$$(EAz\hat{}2)_1 := 473850000000 \tag{26}$$

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

$$(EAz\hat{}2)_2 := 1104000000 \tag{27}$$

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

$$(EAz\hat{}2)_3 := 0 \tag{28}$$

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

$$(EAz\hat{}2)_4 := 1104000000 \tag{29}$$

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

$$(EAz\hat{}2)_5 := 473850000000 \tag{30}$$

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

$$(EAz\hat{}2)_{sum} := 949908000000 \tag{31}$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

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

$$(EI)_{eff} := 9.897920000\ 10^{11} \tag{32}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

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

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

$$a := 90 \qquad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \qquad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

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

> $K_s := 11.5$
#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \qquad (35)$$

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

$$EI_{app} := 7.260572161\ 10^{11} \qquad (36)$$

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

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using short-term verifications of the slabs

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

General data:

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

$$E_1 := 34000 \quad (38)$$

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

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c$; #mm

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t$; #mm

$$h_2 := 120 \quad (41)$$

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

$$A_1 := 36000 \quad (42)$$

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

$$A_2 := 72000 \quad (43)$$

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

$$I_1 := 10800000 \quad (44)$$

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

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**
EC5: 2.2.2(2) Ultimate limit states equation 2.1
The values for the slip modulus (Kser) is found in the pdf about the screw types from the company

Rotho blaas page 227
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

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

$$l_{eff,ctc} := 110 \qquad (46)$$

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

$$K_{ser} := 23100 \qquad (47)$$

> $K_u := evalf\left(\frac{2}{3}\cdot K_{ser}\right)$; #$\frac{N}{mm}$ #equation 2.1 EC5

$$K_u := 15400. \qquad (48)$$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen.

> $angle := 45$;

$$angle := 45 \qquad (49)$$

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

$$k := \frac{\sqrt{2}}{2} \qquad (50)$$

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

$$s_{min,1} := 91.92388153 \qquad (51)$$

> $s := 150$; #mm

$$s := 150 \qquad (52)$$

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.03287684039 \qquad (53)$$

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

$$\gamma_2 := 1.0 \qquad (54)$$

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

$$a_2 := 5.612544326 \qquad (55)$$

>

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

$$a_1 := 84.38745567 \quad \textbf{(56)}$$

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

$$EI_{eff,tot} := 1.398884340\ 10^{12} \quad \textbf{(57)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 2.929172990\ 10^7 \quad \textbf{(58)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 2.216443886\ 10^6 \quad \textbf{(59)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

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

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 5.210849666\ 10^7 \qquad \textbf{(60)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 6.763762779\ 10^7 \quad (61)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{M_4}{10^{\wedge}6}$; #kNm

$$M_{Ed,new} := 67.63762779 \quad (62)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (63)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (64)$$

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

$$P_{Ed} := 177.9315375 \quad (65)$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 4.560930370 \quad (66)$$

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

$$\sigma_{m,1} := 49.31814474 \quad (67)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -53.87907511 \quad (68)$$

Verification of the top section

> $Ver_{top, c} := \frac{\sigma_{c,t}}{\frac{f_{ck, c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -2.309103219 \quad (69)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 44.75721437 \quad (70)$$

> $Ver_{bottom, c} := \frac{\sigma_{b, c}}{\frac{f_{ctk, 0.05, c}}{\gamma_c}}$; #<1.0 #--> NOT OK

$$Ver_{bottom,c} := 30.51628252 \quad (71)$$

3.4.2 Normal stresses in the **TIMBER** section

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

$$\sigma_2 := 2.280465186 \quad (72)$$

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

$$\sigma_{m,2} := 24.37894530 \quad (73)$$

Stresses at the TOP of the timber section

> $\sigma_{t, t} := -\sigma_2 - \sigma_{m, 2}$; #MPa

$$\sigma_{t,t} := -26.65941049 \quad (74)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b, t} := -\sigma_2 + \sigma_{m, 2}$; #MPa

$$\sigma_{b,t} := 22.09848011 \quad (75)$$

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

$$Ver_{timber} := -0.700423390 \quad (76)$$

3.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2 \hat{} 2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10 \hat{} 3;$ *#MPa*

$$\tau_2 := 7.695923164 \quad \textbf{(77)}$$

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*

$$Ver_{shear} := 2.765722387 \quad \textbf{(78)}$$

3.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_1 := 64.79056482 \quad \textbf{(79)}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_2 := 64.79056482 \quad \textbf{(80)}$$

> $f_{tens,\,k} := 20.0;$ *#kN*

$$f_{tens,k} := 20.0 \quad \textbf{(81)}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}};$ *# <1.0 #--> NOT OK*

$$Ver_{F1} := 1.552273949 \quad \textbf{(82)}$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1} \hat{} 2 \cdot \left(4 \cdot \gamma_1 \hat{} 2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right);$ *#mm*

$$a_{1,eff} := 115.2431566 \quad \textbf{(83)}$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\, eff}$; #mm

$$x := 7.577661732 \qquad (84)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\, new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\, eff}$;

$$a_{2,new} := 0.9680125 \qquad (85)$$

> $A_{1,\, eff} := b \cdot x$;

$$A_{1,eff} := 4546.597039 \qquad (86)$$

> $I_{1,\, eff} := \dfrac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 21755.82954 \qquad (87)$$

> $EI_{eff,\, tot,\, new} := E_1 \cdot I_{1,\, eff} + \gamma_1 \cdot E_1 \cdot A_{1,\, eff} \cdot a_{1,\, eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\, new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 7.948610712\ 10^{11} \qquad (88)$$

5. New short-term verification

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_{1,\, eff} \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_1 := 10.96178489 \qquad (89)$$

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

$$\sigma_{m,1} := 10.96178490 \qquad (90)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -21.92356979 \qquad (91)$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.9395815624 \tag{92}$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 1.\ 10^{-8} \tag{93}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom,c} := 6.818181818\ 10^{-9} \tag{94}$$

5.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{\left(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 0.6922057599 \tag{95}$$

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

$$\sigma_{m,2} := 42.90476166 \tag{96}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -43.59696742 \tag{97}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 42.21255590 \tag{98}$$

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

$$Ver_{timber} := -0.859139285 \tag{99}$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^{\wedge}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^{\wedge}3;$ #MPa

$$\tau_2 := 3.496172779 \quad (100)$$

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

$$Ver_{shear} := 1.256437093 \quad (101)$$

5.1.4 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

$$F_1 := 19.66634091 \quad (102)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed};$ #kN

$$F_2 := 19.66633933 \quad (103)$$

> $f_{tens,\,k} := 20.0;$ #kN

$$f_{tens,k} := 20.0 \quad (104)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}};$ #<1.0 #--> OK

$$Ver_{F1} := 0.4711727510 \quad (105)$$

6. Maximum load capacity based on ULS using long-term verifications of slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c};$

(106)

$$E_{1,g} := 9714.285714 \quad (106)$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (107)$$

> $q_k := 0;$

$$q_k := 0 \quad (108)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (109)$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285715 \quad (110)$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (111)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (112)$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad (113)$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1+k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (114)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1+k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad (115)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad (116)$$

> $K_u := \frac{2}{3} \cdot K_{ser,\,2}$

$$K_u := 8324.324327 \quad (117)$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.06042753609 \quad (118)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (119)$$

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

$$a_2 := 5.462362231 \quad (120)$$

>

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

$$a_1 := 84.53763777 \quad (121)$$

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

$$EI_{eff,tot} := 6.581609231\ 10^{11} \quad (122)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

$$\sigma_1 := 0.07539868541\ M_{Ed,1} \quad (123)$$

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

$$\sigma_{m,1} := 0.4427922734\ M_{Ed,1} \quad (124)$$

Stresses at the **TOP** of the **CONCRETE** section

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

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

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

$$M_1 := 4.502844547\ 10^7 \quad (125)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

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

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

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

$$M_2 := 3.992085640\ 10^6 \quad (126)$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

$$\sigma_2 := 0.03769934270\ M_{Ed,2} \quad (127)$$

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

$$\sigma_{m,2} := 0.4140993340\ M_{Ed,2} \quad (128)$$

Stresses at the **TOP** of the **TIMBER** section

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

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

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

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

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

$$M_3 := 4.549531312\ 10^7 \tag{129}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

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

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

$$M_4 := 5.863841321\ 10^7 \tag{130}$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

$$> M_{Ed,\ new} := \frac{M_4}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 58.63841321 \tag{131}$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \tag{132}$$

> $L_{sup} := 2.0;$ #m

$$L_{sup} := 2.0 \quad (133)$$

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

$$P_{Ed} := 153.9336319 \quad (134)$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 4.421259271 \quad (135)$$

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

$$\sigma_{m,1} := 25.96463630 \quad (136)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c,t} := -30.38589557 \quad (137)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{top,c} := -1.302252667 \quad (138)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b,c} := 21.54337703 \quad (139)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};$ #<1.0 #--> NOT OK

$$Ver_{bottom,c} := 14.68866616 \quad (140)$$

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

$$\sigma_2 := 2.210629635 \quad (141)$$

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

$$\sigma_{m,2} := 24.28212786 \quad (142)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2}$; #MPa

$$\sigma_{t,t} := -26.49275750 \quad (143)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2}$; #MPa

$$\sigma_{b,t} := 22.07149822 \quad (144)$$

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

$$Ver_{timber} := -0.690805821 \quad (145)$$

7.4.3 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

$$\tau_2 := 7.649257731 \quad (146)$$

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

$$Ver_{shear} := 2.748951997 \quad (147)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 62.67452482 \quad (148)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 62.67452482 \quad (149)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (150)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; # <1.0 #--> NOT OK

$$Ver_{F1} := 1.501577157 \quad (151)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 111.6324251 \quad (152)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 13.49134479 \quad (153)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 1.6219025 \quad (154)$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 8094.806874 \quad (155)$$

> $I_{1,\,eff} := \frac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 122782.2898 \quad (156)$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 4.537316051\ 10^{11} \quad (157)$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

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

$$\sigma_1 := 8.468749690 \quad (158)$$

> $\sigma_{m, 1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed, new})}{EI_{eff, tot, new}} \cdot 10\hat{}6$; #*MPa*

$$\sigma_{m, 1} := 8.468749690 \quad (159)$$

Stresses at the TOP of the concrete section

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

$$\sigma_{c, t} := -16.93749938 \quad (160)$$

Verification of the top section

> $Ver_{top, c} := \frac{\sigma_{c, t}}{\frac{f_{ck, c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top, c} := -0.7258928306 \quad (161)$$

Stresses at the BOTTOM of the concrete section

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

$$\sigma_{b, c} := 0. \quad (162)$$

> $Ver_{bottom, c} := \frac{\sigma_{b, c}}{\frac{f_{ctk, 0.05, c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom, c} := 0. \quad (163)$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #MPa

$$\sigma_2 := 0.9521235260 \quad (164)$$

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

$$\sigma_{m,2} := 35.22246964 \quad (165)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2};$ #MPa

$$\sigma_{t,t} := -36.17459317 \quad (166)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2};$ #MPa

$$\sigma_{b,t} := 34.27034611 \quad (167)$$

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

$$Ver_{timber} := -0.748480065 \quad (168)$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3;$ #MPa

$$\tau_2 := 2.925903974 \quad (169)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ #<1.0 #-->OK

$$Ver_{shear} := 1.051496741 \quad (170)$$

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

$$F_1 := 26.99406903 \quad (171)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 26.99406930 \quad (172)$$

> $f_{tens,\ k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (173)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}}$; # <1.0 #-- > OK

$$Ver_{F1} := 0.6467329037 \quad (174)$$

>

>

Load capacity predictions based on ULS CTC-screws 7-160 mm B45

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B45

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

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

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

> $E_{cm, c} := 36000$: *#MPa*

> $f_{ck, c} := 45$: *#MPa*

> $f_{ctk, 0.05, c} := 2.7$: *#MPa*

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

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

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

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

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

$$h_t := 120 \quad (1)$$

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

> $Klima := 1.0 : \#Serice\ class,\ permanent$

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

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

Lamellae 1 and 5, Class T22

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

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

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

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

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

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

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

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

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

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

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

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

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

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

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

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

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

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

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

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m^3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

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

$$g_{0,k} := 1.217734993 \quad (4)$$

1. 1 ULS

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

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

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

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1;\#mm\hat{}2$

$$A_1 := 18000 \quad (5)$$

> $A_5 := A_1;\#mm\hat{}2$

$$A_5 := 18000 \quad (6)$$

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

$$I_{t1} := 1350000 \quad (7)$$

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

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

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

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

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

$$A_4 := 12000 \quad (11)$$

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

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

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

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

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

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

$$z_1 := 45 \quad (15)$$

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

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

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

$$z_4 := 20 \quad (18)$$

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

$$z_5 := 45 \quad (19)$$

Ei*Ii

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

$$(EI)_1 := 17550000000 \quad (20)$$

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

$$(EI)_2 := 92000000 \quad (21)$$

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

$$(EI)_3 := 4600000000 \quad (22)$$

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

$$(EI)_4 := 92000000 \quad (23)$$

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

$$(EI)_5 := 17550000000 \quad (24)$$

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

$$(EI)_{sum} := 39884000000 \quad (25)$$

Ei*Ao*zi^2

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

$$(EAz\hat{}2)_1 := 473850000000 \quad (26)$$

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

$$(EAz\hat{}2)_2 := 1104000000 \quad (27)$$

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

$$(EAz\hat{}2)_3 := 0 \quad (28)$$

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

$$(EAz\hat{}2)_4 := 1104000000 \quad (29)$$

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

$$(EAz\hat{}2)_5 := 473850000000 \quad (30)$$

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

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (31)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

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

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (32)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

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

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

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

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

> $K_s := 11.5$
#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (35)$$

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

$$EI_{app} := 7.260572161\ 10^{11} \quad (36)$$

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

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using short-term verifications of the slab

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

General data:

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

$$E_1 := 36000 \quad (38)$$

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

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c$; #*mm*

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t$; #*mm*

$$h_2 := 120 \quad (41)$$

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

$$A_1 := 36000 \quad (42)$$

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

$$A_2 := 72000 \quad (43)$$

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

$$I_1 := 10800000 \quad (44)$$

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

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**

EC5: 2.2.2(2) Ultimate limit states equation 2.1

The values for the slip modulus (Kser) is found in the pdf about the screw types from the company

Rotho blaas page 227
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

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

$$l_{eff,ctc} := 110 \tag{46}$$

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

$$K_{ser} := 23100 \tag{47}$$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right);$ # $\frac{N}{mm}$ #*equation 2.1 EC5*

$$K_u := 15400. \tag{48}$$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen

> $angle := 45;$

$$angle := 45 \tag{49}$$

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

$$k := \frac{\sqrt{2}}{2} \tag{50}$$

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

$$s_{min,1} := 91.92388153 \tag{51}$$

> $s := 150;$ #*mm*

$$s := 150 \tag{52}$$

Annex B, EC5
B.2 Effective bending stiffenes

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

$$\gamma_1 := 0.03110716623 \tag{53}$$

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

$$\gamma_2 := 1.0 \tag{54}$$

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

$$a_2 := 5.622172792 \tag{55}$$

>

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

$$a_1 := 84.37782721 \quad (56)$$

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

$$EI_{eff,tot} := 1.421008650\ 10^{12} \quad (57)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

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

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 3.629678972\ 10^7 \quad (58)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_2 := 2.595426370\ 10^6 \quad (59)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,\, t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \le 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$

$$M_3 := 5.292220572\ 10^7 \tag{60}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\dfrac{\dfrac{k_{modi,t}}{\gamma_M}}{\left(-\dfrac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \dfrac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 6.872493099\ 10^7 \quad \textbf{(61)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \dfrac{\min(M_1, M_3, M_4)}{10\hat{\ }6}; \#kNm$

$$M_{Ed,new} := 36.29678972 \quad \textbf{(62)}$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \quad \textbf{(63)}$$

> $L_{sup} := 2.0; \#m$

$$L_{sup} := 2.0 \quad \textbf{(64)}$$

> $P_{Ed} := solve\left(\dfrac{P_{Ed,\ 1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$

$$P_{Ed} := 94.35596927 \quad \textbf{(65)}$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{\ }6; \#MPa$

$$\sigma_1 := 2.413586024 \quad \textbf{(66)}$$

> $\sigma_{m,\ 1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{\ }6; \#MPa$

$$\sigma_{m,1} := 27.58641399 \quad \textbf{(67)}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}; \#MPa$

$$\sigma_{c,t} := -30.00000001 \quad \textbf{(68)}$$

Verification of the top section

> $Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}}$; # <1.0 #--- >OK

$$Ver_{top,c} := -1.000000000 \quad (69)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 25.17282797 \quad (70)$$

> $Ver_{bottom,\ c} := \frac{\sigma_{b,\ c}}{\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c}}$; # <1.0 #--- > NOT OK

$$Ver_{bottom,c} := 13.98490443 \quad (71)$$

3.4.2 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

$$\sigma_2 := 1.206793011 \quad (72)$$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 12.87893193 \quad (73)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2}$; #MPa

$$\sigma_{t,t} := -14.08572494 \quad (74)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2}$; #MPa

$$\sigma_{b,t} := 11.67213892 \quad (75)$$

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

$$Ver_{timber} := -0.3702527714 \quad (76)$$

3.4.3 Shear stresses in the **TIMBER** section

$$> \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\text{\^{}}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\text{\^{}}3; \#MPa$$

$$\tau_2 := 4.017558958 \qquad \textbf{(77)}$$

Verification of the timber section

$$> Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \# < 1.0 \# \text{---} > NOT\ OK$$

$$Ver_{shear} := 1.443810250 \qquad \textbf{(78)}$$

3.4.4 The **load per shear fasteners**

$$> F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_1 := 33.88111600 \qquad \textbf{(79)}$$

$$> F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_2 := 33.88111598 \qquad \textbf{(80)}$$

$$> f_{tens,\ k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \qquad \textbf{(81)}$$

$$> Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}}; \# < 1.0 \# \text{--} > OK$$

$$Ver_{F1} := 0.8117350707 \qquad \textbf{(82)}$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

$$> a_{1,\ eff} := \max\Big(solve\Big(a_{1,\ 1}\text{\^{}}2 \cdot \big(4 \cdot \gamma_1\text{\^{}}2 \cdot E_1 \cdot b\big) + a_{1,\ 1} \cdot \big(2 \cdot E_2 \cdot A_2 \cdot (1 + \gamma_1)\big) - E_2 \cdot A_2 \cdot \big(2 \cdot h_1 + h_2\big) = 0, a_{1,\ 1}\Big)\Big); \#mm$$

$$a_{1,eff} := 115.4860990 \qquad \textbf{(83)}$$

The effective compressed height of the concrete:

> $x := 2\cdot\gamma_1\cdot a_{1,\,eff};$ #mm

$$x := 7.184890558 \quad \textbf{(84)}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5\cdot x + 0.5\cdot h_2 - a_{1,\,eff};$

$$a_{2,new} := 0.9214557 \quad \textbf{(85)}$$

> $A_{1,\,eff} := b\cdot x;$

$$A_{1,eff} := 4310.934335 \quad \textbf{(86)}$$

> $I_{1,\,eff} := \frac{b\cdot x^{\wedge}3}{12}$

$$I_{1,eff} := 18545.15537 \quad \textbf{(87)}$$

> $EI_{eff,\,tot,\,new} := E_1\cdot I_{1,\,eff} + \gamma_1\cdot E_1\cdot A_{1,\,eff}\cdot a_{1,\,eff}{}^{\wedge}2 + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_{2,\,new}{}^{\wedge}2;$ #Nmm^2

$$EI_{eff,tot,new} := 7.916249222\ 10^{11} \quad \textbf{(88)}$$

5. New short-term verification

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1\cdot E_1\cdot a_{1,\,eff}\cdot M_{Ed,\,new})}{EI_{eff,tot,new}}\cdot 10^{\wedge}6;$ #MPa

$$\sigma_1 := 5.929818754 \quad \textbf{(89)}$$

> $\sigma_{m,\,1} := \frac{(0.5\cdot E_1\cdot x\cdot M_{Ed,\,new})}{EI_{eff,tot,new}}\cdot 10^{\wedge}6;$ #MPa

$$\sigma_{m,1} := 5.929818755 \quad \textbf{(90)}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -11.85963751 \quad \textbf{(91)}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.3953212503 \quad \textbf{(92)}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 1.\ 10^{-9} \quad \textbf{(93)}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom,c} := 5.555555556\ 10^{-10} \quad \textbf{(94)}$$

5.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 0.3550424825 \quad \textbf{(95)}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 23.11836472 \quad \textbf{(96)}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -23.47340720 \quad \textbf{(97)}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 22.76332224 \quad \textbf{(98)}$$

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

$$Ver_{timber} := -0.460912267 \quad \textbf{(99)}$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3;$ #MPa

$$\tau_2 := 1.858735416 \quad (100)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ # <1.0 #-- >OK

$$Ver_{shear} := 0.6679830401 \quad (101)$$

5.1.4 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

$$F_1 := 9.967936227 \quad (102)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed};$ #kN

$$F_2 := 9.967936020 \quad (103)$$

> $f_{tens,\,k} := 20.0;$ #kN

$$f_{tens,k} := 20.0 \quad (104)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}};$ # <1.0 #-- > OK

$$Ver_{F1} := 0.2388151387 \quad (105)$$

6. Maximum load capacity based on ULS using long-term verifications of slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c};$

(106)

$$E_{1,g} := 10285.71429 \quad \textbf{(106)}$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2};$

$$E_{1,q} := 16000.00000 \quad \textbf{(107)}$$

> $q_k := 0;$

$$q_k := 0 \quad \textbf{(108)}$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad \textbf{(109)}$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 10285.71429 \quad \textbf{(110)}$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1 + k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad \textbf{(111)}$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad \textbf{(112)}$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad \textbf{(113)}$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad \textbf{(114)}$$

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad \textbf{(115)}$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad \textbf{(116)}$$

> $K_u := \frac{2}{3} \cdot K_{ser,\,2}$

$$K_u := 8324.324327 \quad (117)$$

Repeating step 3-5

7. Long-term verification

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.05726268645 \quad (118)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (119)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}; \#mm$

$$a_2 := 5.479641370 \quad (120)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2; \#mm$

$$a_1 := 84.52035863 \quad (121)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2; \#Nmm\hat{}2$

$$EI_{eff,tot} := 6.648409588\ 10^{11} \quad (122)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10\hat{}6; \#MPa$

$$\sigma_1 := 0.07487722477\ M_{Ed,1} \quad (123)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; \#MPa$

$$\sigma_{m,1} := 0.4641281868\ M_{Ed,1} \quad (124)$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $$M_1 := solve\left(M_{Ed,1} = \frac{f_{ck,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_1 := 5.565806828\ 10^7 \quad \textbf{(125)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{ctk,0.005,c}}{\gamma_c}$$

> $$M_2 := solve\left(M_{Ed,1} = \frac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_2 := 4.624266028\ 10^6 \quad \textbf{(126)}$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $$\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6; \#MPa$$

$$\sigma_2 := 0.03743861238\ M_{Ed,2} \quad \textbf{(127)}$$

> $$\sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6; \#MPa$$

$$\sigma_{m,2} := 0.4099386422\ M_{Ed,2} \quad \textbf{(128)}$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,t = -\ \frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t} \cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 4.594078354\ 10^7 \tag{129}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\ b\ =\ -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 5.926064409\ 10^7 \tag{130}$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$

$$M_{Ed,new} := 45.94078354 \tag{131}$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \tag{132}$$

> $L_{sup} := 2.0;$ #m

$$L_{sup} := 2.0 \quad (133)$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right);$ #kN

$$P_{Ed} := 120.0732861 \quad (134)$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10\hat{}6;$ #MPa

$$\sigma_1 := 3.439918376 \quad (135)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6;$ #MPa

$$\sigma_{m,1} := 21.32241256 \quad (136)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -24.76233094 \quad (137)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{top,c} := -0.8254110314 \quad (138)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1};$ #MPa

$$\sigma_{b,c} := 17.88249418 \quad (139)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};$ #<1.0 #---> NOT OK

$$Ver_{bottom,c} := 9.934718990 \quad (140)$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 1.719959187 \quad (141)$$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 18.83290242 \quad (142)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2}$; #MPa

$$\sigma_{t,t} := -20.55286161 \quad (143)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2}$; #MPa

$$\sigma_{b,t} := 17.11294323 \quad (144)$$

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

$$Ver_{timber} := -0.5363900096 \quad (145)$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 5.906721583 \quad (146)$$

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

$$Ver_{shear} := 2.122728069 \quad (147)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 48.55007395 \quad (148)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 48.55007395 \quad \textbf{(149)}$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad \textbf{(150)}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{F1} := 1.163178855 \quad \textbf{(151)}$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 112.0315886 \quad \textbf{(152)}$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 12.83045946 \quad \textbf{(153)}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 1.5531817 \quad \textbf{(154)}$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 7698.275676 \quad \textbf{(155)}$$

> $I_{1,\,eff} := \frac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 105607.9544 \quad \textbf{(156)}$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 4.512475431\ 10^{11} \quad \textbf{(157)}$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #MPa

$$\sigma_1 := 6.717848685 \quad (158)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #MPa

$$\sigma_{m,1} := 6.717848685 \quad (159)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -13.43569737 \quad (160)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{top,c} := -0.4478565790 \quad (161)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1};$ #MPa

$$\sigma_{b,c} := 0. \quad (162)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{bottom,c} := 0. \quad (163)$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 0.7182757224 \quad (164)$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 27.74726444 \quad (165)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -28.46554016 \quad (166)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 27.02898872 \quad (167)$$

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

$$Ver_{timber} := -0.586057016 \quad (168)$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 2.289749318 \quad (169)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{shear} := 0.8228786613 \quad (170)$$

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

$$F_1 := 20.27509668 \quad (171)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 20.27509704 \tag{172}$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{173}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.4857575243 \tag{174}$$

>

>

Load capacity predictions based on ULS CTC-screws 7-160 mm

Using the average result from compressive test of concrete cubes

> $restart;$

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000 :$ *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600 :$ *#mm*

Concrete data, B35*

*strength average from compressive strength test of concrete cubes

*Partial factor is 1.0

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60 :$ *#mm*

> $A_c := h_c \cdot b :$ *#mm^2*

> $I_c := \frac{(b \cdot h_c\hat{}3)}{12} :$ *#mm^4*

> $E_{cm,\,c} := 34000 :$ *#MPa*

> $f_{ck,\,c} := 45.4553333 :$ *#MPa*

> $f_{ctk,\,0.05,\,c} := 2.2 :$ *#MPa*

> $\rho_c := 25.00 : \# \frac{kN}{m\hat{}3}$

> $\gamma_c := 1.0 :$

> $\varphi_c := 2.5 :$

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \qquad (1)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \# \frac{kg}{m\^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m\^3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\^2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\^2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\^2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\^2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\^2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\^2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\^2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\^2}$

> $t_{t15} := 430 : \# \frac{kg}{m\^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m\^3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5\right)\right)$;# $\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad (4)$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k} \cdot \gamma_{G,\,1}$: # $\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$; #mm^2

$$A_1 := 18000 \quad (5)$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad (6)$$

> $I_{t1} := \frac{(b \cdot h_1 \hat{} 3)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad (7)$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (11)$$

> $I_{t2} := \frac{(b \cdot h_2 \hat{} 3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (15)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (18)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad (19)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad (20)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad (21)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad (22)$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad (23)$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad (24)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (25)$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad (26)$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \quad (27)$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad (28)$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \quad (29)$$

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_5 := 473850000000 \quad (30)$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

(31)

$$(EAz\hat{\ }2)_{sum} := 949908000000 \quad (31)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{\ }2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (32)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a\hat{\ }2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^{6} \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
> *#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load*

$$K_s := 11.5 \quad (35)$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; # $\frac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \quad (36)$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t^3}{12}}$; # $\frac{N}{mm^{\wedge}}$

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}$; # $\frac{N}{mm^2}$

$$E_1 := 34000 \quad (38)$$

> $E_2 := E_{CLT}$; # $\frac{N}{mm}$

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c$; #mm

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t$; #mm

$$h_2 := 120 \quad (41)$$

> $A_1 := A_c$;#mm^2

$$A_1 := 36000 \quad (42)$$

> $A_2 := h_2 \cdot b$; #mm^2

$$A_2 := 72000 \quad (43)$$

> $I_1 := I_c$; #mm^4

$$I_1 := 10800000 \quad (44)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \quad \textbf{(45)}$$

The slip modulus **Kser**

EC5: 2.2.2(2) Ultimate limit states equation 2.1

The values for the slip modulus (Kser) is found in the pdf about the screw types from the company **Rotho blaas page 227**

There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

> $l_{eff,\ ctc} := 110$; #mm

$$l_{eff,ctc} := 110 \quad \textbf{(46)}$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\ ctc}$; $\# \frac{N}{mm}$

$$K_{ser} := 23100 \quad \textbf{(47)}$$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right)$; $\# \frac{N}{mm}$ #equation 2.1 EC5

$$K_u := 15400. \quad \textbf{(48)}$$

Finding the spacing for the slab

Rotho blaas p. 225: minimum spacing was calculated

s,min= 130*sin(45), and continuous spacing were chosen

> $angle := 45$;

$$angle := 45 \quad \textbf{(49)}$$

> $k := \sin(convert(angle\ degrees, radians))$;

$$k := \frac{\sqrt{2}}{2} \quad \textbf{(50)}$$

> $s_{\min,\ 1} := evalf(130 \cdot k)$;#mm

$$s_{min,1} := 91.92388153 \quad \textbf{(51)}$$

> $s := 150$ #mm

$$s := 150 \quad \textbf{(52)}$$

Annex B, EC5

B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right)$;

$$\gamma_1 := 0.03287684039 \quad \textbf{(53)}$$

> $\gamma_2 := 1.0$; #Fully composite

$$\gamma_2 := 1.0 \quad \textbf{(54)}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 5.612544326 \quad (55)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 84.38745567 \quad (56)$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 1.398884340\ 10^{12} \quad (57)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma\,c,t\ = -\ \sigma 1 - \sigma\,m,1 = \frac{fck}{\gamma\,c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 5.706280053\ 10^7 \quad (58)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma\,c,b\ = -\ \sigma\,1\ + \sigma\,m,1 = \frac{fck}{\gamma\,c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}}\right)}, M_{Ed,\,1}\right); \#Nmm$$

$$M_2 := 3.324665829\ 10^6 \quad \textbf{(59)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\, t = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\,;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.210849666\ 10^7 \quad \textbf{(60)}$$

\>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma t,\, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $$M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 6.763762779\ 10^7 \quad (61)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $$M_{Ed,\,new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 52.10849666 \quad (62)$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \quad (63)$$

> $L_{sup} := 2.0; \#m$

$$L_{sup} := 2.0 \quad (64)$$

> $$P_{Ed} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right); \#kN$$

$$P_{Ed} := 136.5205211 \quad (65)$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

> $$\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10\hat{}6; \#MPa$$

$$\sigma_1 := 3.513772329 \quad (66)$$

> $$\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6; \#MPa$$

$$\sigma_{m,1} := 37.99504010 \quad (67)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}; \#MPa$

$$\sigma_{c,t} := -41.50881243 \quad (68)$$

Verification of the top section

> $Ver_{top,\ c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\ c}}{\gamma_c}}$; # <1.0 #--->OK

$$Ver_{top,c} := -0.9131780457 \quad (69)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 34.48126777 \quad (70)$$

> $Ver_{bottom,\ c} := \dfrac{\sigma_{b,\ c}}{\dfrac{f_{ctk,\ 0.05,\ c}}{\gamma_c}}$; # <1.0 #--> NOT OK

$$Ver_{bottom,c} := 15.67330353 \quad (71)$$

3.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 1.756886165 \quad (72)$$

> $\sigma_{m,\ 2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 18.78170822 \quad (73)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2}$; #MPa

$$\sigma_{t,t} := -20.53859438 \quad (74)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2}$; #MPa

$$\sigma_{b,t} := 17.02482206 \quad (75)$$

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

(76)

$$Ver_{timber} := -0.5396110274 \quad (76)$$

3.4.3 Shear stresses in the **TIMBER** section

$$> \tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3; \#MPa$$

$$\tau_2 := 5.904807295 \quad (77)$$

Verification of the timber section

$$> Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \# < 1.0 \#--> NOT\ OK$$

$$Ver_{shear} := 2.122040122 \quad (78)$$

3.4.4 The **load per shear fasteners**

$$> F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_1 := 49.71148901 \quad (79)$$

$$> F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$$

$$F_2 := 49.71148901 \quad (80)$$

$$> f_{tens,\,k} := 20.0; \#kN$$

$$f_{tens,k} := 20.0 \quad (81)$$

$$> Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}; \# < 1.0 \#--> OK$$

$$Ver_{F1} := 1.191004424 \quad (82)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

$$> a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right); \#mm$$

(83)

$$a_{1,eff} := 115.2431566 \quad \textbf{(83)}$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\ eff}$; #mm

$$x := 7.577661732 \quad \textbf{(84)}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\ new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\ eff}$;

$$a_{2,new} := 0.9680125 \quad \textbf{(85)}$$

> $A_{1,\ eff} := b \cdot x$;

$$A_{1,eff} := 4546.597039 \quad \textbf{(86)}$$

> $I_{1,\ eff} := \frac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 21755.82954 \quad \textbf{(87)}$$

> $EI_{eff,\ tot,\ new} := E_1 \cdot I_{1,\ eff} + \gamma_1 \cdot E_1 \cdot A_{1,\ eff} \cdot a_{1,\ eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\ new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 7.948610712\ 10^{11} \quad \textbf{(88)}$$

5. New short-term verification

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\ eff} \cdot M_{Ed,\ new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_1 := 8.445034962 \quad \textbf{(89)}$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\ new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,1} := 8.445034965 \quad \textbf{(90)}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}$; #MPa

(91)

$$\sigma_{c,t} := -16.89006993 \quad (91)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}$; # <1.0 #--->OK

$$Ver_{top,c} := -0.3715750981 \quad (92)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 3.\ 10^{-9} \quad (93)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; # <1.0 #--->OK

$$Ver_{bottom,c} := 1.363636364\ 10^{-9} \quad (94)$$

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

$$\sigma_2 := 0.5332801091 \quad (95)$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 33.05412538 \quad (96)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -33.58740549 \quad (97)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 32.52084527 \quad (98)$$

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

$$Ver_{timber} := -0.661886852 \quad \textbf{(99)}$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 2.682488649 \quad \textbf{(100)}$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{shear} := 0.9640193582 \quad \textbf{(101)}$$

5.1.4 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

$$F_1 := 15.08928179 \quad \textbf{(102)}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 15.08928058 \quad \textbf{(103)}$$

> $f_{tens,\ k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad \textbf{(104)}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}}$; #<1.0 #--> OK

$$Ver_{F1} := 0.3615140430 \quad \textbf{(105)}$$

6. Maximum load capacity based on ULS using long-term verification of the slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \quad (106)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (107)$$

> $q_k := 0;$

$$q_k := 0 \quad (108)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (109)$$

> $E_1 := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285715 \quad (110)$$

6.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (111)$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (112)$$

> $E_2 := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad (113)$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \frac{K_{ser}}{1+k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (114)$$

> $K_{ser,q} := \frac{K_{ser}}{1+k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad \textbf{(115)}$$

$$> K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$$

$$K_{ser,2} := 12486.48649 \quad \textbf{(116)}$$

$$> K_u := \frac{2}{3} \cdot K_{ser,2}$$

$$K_u := 8324.324327 \quad \textbf{(117)}$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5

B.2 Effective bending stiffenes

$$> \gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L^2}}\right);$$

$$\gamma_1 := 0.06042753609 \quad \textbf{(118)}$$

$$> \gamma_2 := 1.0;$$

$$\gamma_2 := 1.0 \quad \textbf{(119)}$$

$$> a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}; \#mm$$

$$a_2 := 5.462362231 \quad \textbf{(120)}$$

\>

$$> a_1 := \frac{(h_1 + h_2)}{2} - a_2; \#mm$$

$$a_1 := 84.53763777 \quad \textbf{(121)}$$

$$> EI_{eff,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2; \#Nmm^2$$

$$EI_{eff,tot} := 6.581609231\ 10^{11} \quad \textbf{(122)}$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6; \#MPa$$

$$\sigma_1 := 0.07539868541\ M_{Ed,1} \quad \textbf{(123)}$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_{m,1} := 0.4427922734\, M_{Ed,1} \quad (124)$$

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right);$ #*Nmm*

$$M_1 := 8.771927130\ 10^7 \quad (125)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$

> $M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right);$ #*Nmm*

$$M_2 := 5.988128459\ 10^6 \quad (126)$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_2 := 0.03769934270\, M_{Ed,2} \quad (127)$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6;$ #*MPa*

$$\sigma_{m,2} := 0.4140993340\, M_{Ed,2} \quad (128)$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t, t = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\,;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 4.549531312\ 10^7 \qquad (129)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 5.863841321\ 10^7 \qquad (130)$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\,new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$

(131)

$$M_{Ed,new} := 45.49531312 \quad (131)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (132)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (133)$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right)$; #kN

$$P_{Ed} := 118.8853650 \quad (134)$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_1 := 3.430286802 \quad (135)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_{m,1} := 20.14497313 \quad (136)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.57525993 \quad (137)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.5186467290 \quad (138)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 16.71468633 \quad (139)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--> NOT OK

(140)

$$Ver_{bottom,c} := 7.597584695 \quad \textbf{(140)}$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 1.715143401 \quad \textbf{(141)}$$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 18.83957886 \quad \textbf{(142)}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2}$; #MPa

$$\sigma_{t,t} := -20.55472226 \quad \textbf{(143)}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2}$; #MPa

$$\sigma_{b,t} := 17.12443546 \quad \textbf{(144)}$$

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

$$Ver_{timber} := -0.5359699440 \quad \textbf{(145)}$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 5.907642054 \quad \textbf{(146)}$$

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

$$Ver_{shear} := 2.123058863 \quad \textbf{(147)}$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 48.40452127 \quad (148)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 48.40452127 \quad (149)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (150)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; # <1.0 #--> NOT OK

$$Ver_{F1} := 1.159691656 \quad (151)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 111.6324251 \quad (152)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 13.49134479 \quad (153)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 1.6219025 \quad (154)$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 8094.806874 \quad (155)$$

> $I_{1,\,eff} := \frac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 122782.2898 \quad (156)$$

> $EI_{eff,\ tot,\ new} := E_1 \cdot I_{1,\ eff} + \gamma_1 \cdot E_1 \cdot A_{1,\ eff} \cdot a_{1,\ eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\ new}\hat{}2;\ \#Nmm\hat{}2$

$$EI_{eff,tot,new} := 4.537316051\ 10^{11} \quad (157)$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\ eff} \cdot M_{Ed,\ new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;\ \#MPa$

$$\sigma_1 := 6.570580576 \quad (158)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\ new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;\ \#MPa$

$$\sigma_{m,1} := 6.570580575 \quad (159)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1};\ \#MPa$

$$\sigma_{c,t} := -13.14116115 \quad (160)$$

Verification of the top section

> $Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}};\ \# < 1.0\ \#\text{---} > OK$

$$Ver_{top,c} := -0.2891005344 \quad (161)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1};\ \#MPa$

$$\sigma_{b,c} := -1.\ 10^{-9} \quad (162)$$

> $Ver_{bottom, c} := \dfrac{\sigma_{b, c}}{\dfrac{f_{ctk, 0.05, c}}{\gamma_c}}$; # <1.0 #--->OK

$$Ver_{bottom,c} := -4.545454545\ 10^{-10} \quad (163)$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2, new} \cdot M_{Ed, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 0.7387164075 \quad (164)$$

> $\sigma_{m, 2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 27.32777368 \quad (165)$$

Stresses at the TOP of the timber section

> $\sigma_{t, t} := -\sigma_2 - \sigma_{m, 2}$; #MPa

$$\sigma_{t,t} := -28.06649009 \quad (166)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b, t} := -\sigma_2 + \sigma_{m, 2}$; #MPa

$$\sigma_{b,t} := 26.58905727 \quad (167)$$

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

$$Ver_{timber} := -0.580717197 \quad (168)$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 2.259721658 \quad (169)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; # <1.0 #-- >OK

$$Ver_{shear} := 0.8120874707 \quad (170)$$

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

$$F_1 := 20.84794407 \quad (171)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 20.84794429 \quad (172)$$

> $f_{tens,\ k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (173)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}}$; # <1.0 #-- > NOT OK

$$Ver_{F1} := 0.4994819933 \quad (174)$$

>

>

Load capacity predictions based on ULS
KOP-screws 10-140 mm

M3, choosing the moment M3, top part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\# \frac{kN}{m \hat{} 3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \tag{3}$$

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

> $g_{0,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5\right)\right)$;# $\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad \textbf{(4)}$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k} \cdot \gamma_{G,\,1}$:# $\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$; #mm^2

$$A_1 := 18000 \quad \textbf{(5)}$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad \textbf{(6)}$$

> $I_{t1} := \frac{\left(b \cdot h_1\hat{\ }3\right)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad \textbf{(7)}$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad \textbf{(8)}$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (11)$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (15)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (18)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad (19)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad (20)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad (21)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad (22)$$

> $(EI)_4 := E_{90, mean, t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad (23)$$

> $(EI)_5 := E_{0, mean, t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad (24)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (25)$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad (26)$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \quad (27)$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad (28)$$

> $(EAz\hat{}2)_4 := E_{90, mean, t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \quad (29)$$

> $(EAz\hat{}2)_5 := E_{0, mean, t22} \cdot A_5 \cdot (z_5\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_5 := 473850000000 \quad (30)$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (31)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (32)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
> #CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (35)$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; # $\frac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \quad (36)$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t\hat{}3}{12}}$; # $\frac{N}{mm\hat{}}$

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using verification of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\, c}$; # $\frac{N}{mm\hat{}2}$

$$E_1 := 34000 \quad (38)$$

> $E_2 := E_{CLT}$; # $\frac{N}{mm}$

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c$; #mm

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t$; #mm

$$h_2 := 120 \quad (41)$$

> $A_1 := A_c$;#mm^2

$$A_1 := 36000 \quad (42)$$

> $A_2 := h_2 \cdot b$; #mm^2

$$A_2 := 72000 \quad (43)$$

> $I_1 := I_c$; #mm^4

$$I_1 := 10800000 \quad (44)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22};$

$$\rho_{m,1} := 470 \quad \textbf{(46)}$$

> $\rho_{m,\,2} := t_{t15};$

$$\rho_{m,2} := 430 \quad \textbf{(47)}$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1}\cdot\rho_{m,\,2}\right)\right);$

$$\rho_m := 449.5553359 \quad \textbf{(48)}$$

> $d_{kop} := 10\ ;\#mm$

$$d_{kop} := 10 \quad \textbf{(49)}$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23}\cdot\rho_m\hat{\ }1.5\cdot 3\right);$

$$K_{ser} := 12432.77728 \quad \textbf{(50)}$$

> $K_u := evalf\left(\frac{2}{3}\cdot K_{ser}\right)\ ;\#\frac{N}{mm}$

$$K_u := 8288.518187 \quad \textbf{(51)}$$

>

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;\ \#mm$

$$s := 100 \quad \textbf{(52)}$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1+\frac{\pi\hat{\ }2\cdot E_1\cdot s\cdot A_1}{K_u\cdot L\hat{\ }2}}\right);$

$$\gamma_1 := 0.02671144241 \quad \textbf{(53)}$$

> $\gamma_2 := 1.0;\ \#Fully\ composite$

$$\gamma_2 := 1.0 \quad \textbf{(54)}$$

> $a_2 := \frac{\gamma_1\cdot E_1\cdot A_1\cdot\left(h_1+h_2\right)}{2\cdot\left(\gamma_1\cdot E_1\cdot A_1+\gamma_2\cdot E_2\cdot A_2\right)};\ \#mm$

(55)

$$a_2 := 4.613982231 \quad \textbf{(55)}$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 85.38601777 \quad \textbf{(56)}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 1.344508348\ 10^{12} \quad \textbf{(57)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 2.858362473\ 10^7 \quad \textbf{(58)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_2 := 2.092353503\ 10^6 \quad (59)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ 2})}{EI_{eff,\ tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\ 2})}{EI_{eff, tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\ 0,\ d}} - \frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t} \cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_3 := 5.112721903\ 10^7 \quad (60)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\ t,\ b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 6.332956592\ 10^7 \quad (61)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{M_3}{10\hat{\ }6}$; #kNm

$$M_{Ed,new} := 51.12721903 \quad (62)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (63)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (64)$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed} := 133.9037808 \quad (65)$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_1 := 2.948843229 \quad (66)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_{m,1} := 38.78723660 \quad (67)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -41.73607983 \quad (68)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -1.788689136 \quad (69)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 35.83839337 \quad (70)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #---> NOT OK

$$Ver_{bottom,c} := 24.43526820 \quad (71)$$

3.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 1.474421614 \quad (72)$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 19.17330680 \quad (73)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -20.64772841 \quad (74)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 17.69888519 \quad (75)$$

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

$$Ver_{timber} := -0.5149725757 \quad (76)$$

3.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3;$ *#MPa*

$$\tau_2 := 6.025858600 \qquad (77)$$

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*

$$Ver_{shear} := 2.165542934 \qquad (78)$$

3.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_1 := 27.80320451 \qquad (79)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_2 := 27.80320451 \qquad (80)$$

> $f_{tens,\, k} := 20.0;$ *#kN*

$$f_{tens,k} := 20.0 \qquad (81)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\, k}}{\gamma_M}};$ *# <1.0 #-->OK*

$$Ver_{F1} := 0.6661184413 \qquad (82)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\, eff} := \max\left(solve\left(a_{1,\, 1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\, 1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\, 1}\right)\right);$ *#mm*

$$a_{1,eff} := 116.2447862 \qquad (83)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\, eff}$; #mm

$$x := 6.210131824 \tag{84}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\, new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\, eff}$;

$$a_{2,new} := 0.6501479 \tag{85}$$

> $A_{1,\, eff} := b \cdot x$;

$$A_{1,eff} := 3726.079094 \tag{86}$$

> $I_{1,\, eff} := \frac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 11974.91562 \tag{87}$$

> $EI_{eff,\, tot,\, new} := E_1 \cdot I_{1,\, eff} + \gamma_1 \cdot E_1 \cdot A_{1,\, eff} \cdot a_{1,\, eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\, new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 7.724473842\ 10^{11} \tag{88}$$

5. New short-term verifications

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\, eff} \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_1 := 6.987679937 \tag{89}$$

> $\sigma_{m,\, 1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,1} := 6.987679940 \tag{90}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\, t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -13.97535988 \tag{91}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.5989439949 \quad (92)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 3.\ 10^{-9} \quad (93)$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom,c} := 2.045454546\ 10^{-9} \quad (94)$$

5.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 0.3616200747 \quad (95)$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 33.37272102 \quad (96)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -33.73434109 \quad (97)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 33.01110095 \quad (98)$$

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

$$Ver_{timber} := -0.648381445 \quad (99)$$

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

$$\tau_2 := 2.679259437 \quad (100)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; # <1.0 #-->OK

$$Ver_{shear} := 0.9628588601 \quad (101)$$

5.1.4 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

$$F_1 := 6.819079329 \quad (102)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 6.819078608 \quad (103)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (104)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.1633737756 \quad (105)$$

6. Maximum load capacity based on ULS using long-term verifications of the slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c}$;

(106)

$$E_{1,g} := 9714.285714 \quad \textbf{(106)}$$

> $$E_{1,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2};$$

$$E_{1,q} := 15111.11111 \quad \textbf{(107)}$$

> $q_k := 0;$

$$q_k := 0 \quad \textbf{(108)}$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad \textbf{(109)}$$

> $$E_1 := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$$

$$E_1 := 9714.285715 \quad \textbf{(110)}$$

6.1.2 CLT

> $$E_{2,g} := \frac{E_{CLT}}{1 + k_{def,t}};$$

$$E_{2,g} := 4542.400000 \quad \textbf{(111)}$$

> $$E_{2,q} := \frac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$$

$$E_{2,q} := 5897.150877 \quad \textbf{(112)}$$

> $$E_2 := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$$

$$E_2 := 4542.399998 \quad \textbf{(113)}$$

6.1.3 Slip modulus

>

> $$K_{ser,g} := \frac{K_{ser}}{1 + k_{def,t}};$$

$$K_{ser,g} := 6720.420151 \quad \textbf{(114)}$$

> $$K_{ser,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$$

$$K_{ser,q} := 8724.755986 \quad \textbf{(115)}$$

> $$K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$$

$$K_{ser,2} := 6720.420149 \quad \textbf{(116)}$$

> $K_u := \frac{2}{3} \cdot K_{ser,\,2}$

$$K_u := 4480.280099 \quad (117)$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L^2}}\right);$

$$\gamma_1 := 0.04935924062 \quad (118)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (119)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)};$ #mm

$$a_2 := 4.512000371 \quad (120)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2;$ #mm

$$a_1 := 85.48799963 \quad (121)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2^2;$ #Nmm^2

$$EI_{eff,tot} := 6.301872574\ 10^{11} \quad (122)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10^6;$ #MPa

$$\sigma_1 := 0.06504514407\ M_{Ed,1} \quad (123)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10^6;$ #MPa

$$\sigma_{m,1} := 0.4624475789\ M_{Ed,1} \quad (124)$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6;+ \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right); \#Nmm$$

$$M_1 := 4.423441749\ 10^7 \quad \textbf{(125)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6;+ \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right); \#Nmm$$

$$M_2 := 3.690633319\ 10^6 \quad \textbf{(126)}$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ 2})}{EI_{eff,\ tot}} \cdot 10\hat{}6; \#MPa$$

$$\sigma_2 := 0.03252257204\ M_{Ed,2} \quad \textbf{(127)}$$

$$> \sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\ 2})}{EI_{eff,tot}} \cdot 10\hat{}6; \#MPa$$

$$\sigma_{m,2} := 0.4324809756\ M_{Ed,2} \quad \textbf{(128)}$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t = -\ \frac{\sigma_2}{f_{t,\ 0,\ d}} - \frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t} \cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 4.442789930\ 10^7 \qquad (129)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 5.476970087\ 10^7 \qquad (130)$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

$$> M_{Ed,\ new} := \frac{M_3}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 44.42789930 \qquad (131)$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \qquad (132)$$

> $L_{sup} := 2.0;\ \#m$

$$L_{sup} := 2.0 \quad (133)$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right);\ \#kN$

$$P_{Ed} := 116.0389281 \quad (134)$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10\hat{}6;\ \#MPa$

$$\sigma_1 := 2.889819111 \quad (135)$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6;\ \#MPa$

$$\sigma_{m,1} := 20.54557446 \quad (136)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};\ \#MPa$

$$\sigma_{c,t} := -23.43539357 \quad (137)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};\ \# < 1.0\ \#{-}{-}{-} > OK$

$$Ver_{top,c} := -1.004374010 \quad (138)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1};\ \#MPa$

$$\sigma_{b,c} := 17.65575535 \quad (139)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};\ \# < 1.0\ \#{-}{-}{-} > OK$

$$Ver_{bottom,c} := 12.03801501 \quad (140)$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_2 := 1.444909556 \quad (141)$$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_{m,2} := 19.21422123 \quad (142)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2};$ *#MPa*

$$\sigma_{t,t} := -20.65913079 \quad (143)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2};$ *#MPa*

$$\sigma_{b,t} := 17.76931167 \quad (144)$$

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*

$$Ver_{timber} := -0.5123983378 \quad (145)$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3;$ *#MPa*

$$\tau_2 := 6.022155458 \quad (146)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ *#MPa*

$$Ver_{shear} := 2.164212117 \quad (147)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_1 := 27.17196766 \quad (148)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 27.17196766 \qquad (149)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \qquad (150)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{F1} := 0.6509950587 \qquad (151)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{\ }2 \cdot \left(4 \cdot \gamma_1\hat{\ }2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 113.2933254 \qquad (152)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 11.18414502 \qquad (153)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 1.1146021 \qquad (154)$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 6710.487012 \qquad (155)$$

> $I_{1,\,eff} := \frac{b \cdot x\hat{\ }3}{12}$

$$I_{1,eff} := 69948.49480 \qquad (156)$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{\ }2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{\ }2$; #Nmm^2

$$EI_{eff,tot,new} := 4.348484069\ 10^{11} \qquad (157)$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{\left(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,1} := \frac{\left(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6 : \#MPa$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}; \#MPa$

$$\sigma_{c,t} := -11.10021928 \quad (158)$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}; \# < 1.0 \# \text{---} > OK$

$$Ver_{top,c} := -0.4757236834 \quad (159)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}; \#MPa$

$$\sigma_{b,c} := 2.\ 10^{-9} \quad (160)$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}; \# < 1.0 \# \text{---} > OK$

$$Ver_{bottom,c} := 1.363636364\ 10^{-9} \quad (161)$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{\left(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{}6; \#MPa$

$$\sigma_2 := 0.5172769511 \quad (162)$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_{m,2} := 27.84546796 \quad (163)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -28.36274491 \quad (164)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 27.32819101 \quad (165)$$

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

$$Ver_{timber} := -0.565238535 \quad (166)$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{\ }2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{\ }3$; #MPa

$$\tau_2 := 2.263659920 \quad (167)$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{shear} := 0.8135027838 \quad (168)$$

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

$$F_1 := 9.727551348 \quad (169)$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 9.727551831 \quad (170)$$

> $f_{tens,\ k} := 20.0;$ #*kN*

$$f_{tens,k} := 20.0 \qquad \textbf{(171)}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}};$ # *<1.0 #--> NOT OK*

$$Ver_{F1} := 0.2330559176 \qquad \textbf{(172)}$$

>

>

Load capacity predictions based on ULS
KOP-screws 10-140 mm

M4, choosing the moment M4, bottom part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\#\dfrac{kN}{m\hat{}3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0, mean, t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90, mean, t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

> $g_{0,\,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right);\#\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \qquad (4)$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k}\cdot\gamma_{G,\,1} :\#\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1;\#mm\hat{}2$

$$A_1 := 18000 \qquad (5)$$

> $A_5 := A_1;\#mm\hat{}2$

$$A_5 := 18000 \qquad (6)$$

> $I_{t1} := \frac{(b\cdot h_1\hat{}3)}{12};\#mm\hat{}4$

$$I_{t1} := 1350000 \qquad (7)$$

> $I_{t5} := I_{t1};\#mm\hat{}4$

$$I_{t5} := 1350000 \qquad (8)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (11)$$

> $I_{t2} := \frac{(b \cdot h_2^3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (15)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (18)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad \textbf{(19)}$$

Ei*Ii

> $(EI)_1 := E_{0,\,mean,\,t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad \textbf{(20)}$$

> $(EI)_2 := E_{90,\,mean,\,t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad \textbf{(21)}$$

> $(EI)_3 := E_{0,\,mean,\,t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad \textbf{(22)}$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad \textbf{(23)}$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad \textbf{(24)}$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad \textbf{(25)}$$

Ei*Ao*zi^2

> $(EAz\hat{\ }2)_1 := E_{0,\,mean,\,t22} \cdot A_1 \cdot (z_1\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_1 := 473850000000 \quad \textbf{(26)}$$

> $(EAz\hat{\ }2)_2 := E_{90,\,mean,\,t15} \cdot A_2 \cdot (z_2\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_2 := 1104000000 \quad \textbf{(27)}$$

> $(EAz\hat{\ }2)_3 := E_{0,\,mean,\,t15} \cdot A_3 \cdot (z_3\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_3 := 0 \quad \textbf{(28)}$$

> $(EAz\hat{\ }2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_4 := 1104000000 \quad \textbf{(29)}$$

> $(EAz\hat{\ }2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_5 := 473850000000 \quad \textbf{(30)}$$

> $(EAz\hat{\ }2)_{sum} := (EAz\hat{\ }2)_1 + (EAz\hat{\ }2)_2 + (EAz\hat{\ }2)_3 + (EAz\hat{\ }2)_4 + (EAz\hat{\ }2)_5$; #Nmm^2

$$(EAz\hat{\ }2)_{sum} := 949908000000 \quad \textbf{(31)}$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{\ }2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad \textbf{(32)}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}; \#mm$

$$a := 90 \qquad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right); \#N$

$$(GA)_{eff} := 7.834029851\ 10^{6} \qquad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \qquad (35)$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}; \# \frac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \qquad (36)$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t^{\wedge}3}{12}}; \# \frac{N}{mm^{\wedge}}$

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using verification of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}; \# \frac{N}{mm^{\wedge}2}$

$$E_1 := 34000 \quad (38)$$

> $E_2 := E_{CLT}; \# \frac{N}{mm}$

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c; \#mm$

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t; \#mm$

$$h_2 := 120 \quad (41)$$

> $A_1 := A_c; \#mm^{\wedge}2$

$$A_1 := 36000 \quad (42)$$

> $A_2 := h_2 \cdot b; \#mm^{\wedge}2$

$$A_2 := 72000 \quad (43)$$

> $I_1 := I_c; \#mm^{\wedge}4$

$$I_1 := 10800000 \quad (44)$$

> $I_2 := \frac{b \cdot h_t^{\wedge}3}{12}; \#mm^{\wedge}4$

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22};$

$$\rho_{m,1} := 470 \quad \textbf{(46)}$$

> $\rho_{m,\,2} := t_{t15};$

$$\rho_{m,2} := 430 \quad \textbf{(47)}$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right);$

$$\rho_m := 449.5553359 \quad \textbf{(48)}$$

> $d_{kop} := 10$;#mm

$$d_{kop} := 10 \quad \textbf{(49)}$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m\hat{\ }1.5 \cdot 3\right);$

$$K_{ser} := 12432.77728 \quad \textbf{(50)}$$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right)$; $\#\frac{N}{mm}$

$$K_u := 8288.518187 \quad \textbf{(51)}$$

>

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;$ #mm

$$s := 100 \quad \textbf{(52)}$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{\ }2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{\ }2}}\right);$

$$\gamma_1 := 0.02671144241 \quad \textbf{(53)}$$

> $\gamma_2 := 1.0;$ #Fully composite

$$\gamma_2 := 1.0 \quad \textbf{(54)}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot \left(h_1 + h_2\right)}{2 \cdot \left(\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2\right)}$; #mm

(55)

$$a_2 := 4.613982231 \quad \textbf{(55)}$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 85.38601777 \quad \textbf{(56)}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 1.344508348\ 10^{12} \quad \textbf{(57)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c{,}t = -\ \sigma 1 - \sigma m{,}1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 2.858362473\ 10^{7} \quad \textbf{(58)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c{,}b = -\ \sigma 1 + \sigma m{,}1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_2 := 2.092353503\ 10^6 \quad \textbf{(59)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,\, t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.112721903\ 10^7 \quad \textbf{(60)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right);$ #Nmm

$$M_4 := 6.332956592\ 10^7 \quad \textbf{(61)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{M_4}{10\hat{\ }6};$ #kNm

$$M_{Ed,new} := 63.32956592 \quad \textbf{(62)}$$

> $L_{out} := 0.75;$ #m

$$L_{out} := 0.75 \quad \textbf{(63)}$$

> $L_{sup} := 2.0;$ #m

$$L_{sup} := 2.0 \quad \textbf{(64)}$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right);$ #kN

$$P_{Ed} := 166.4433725 \quad \textbf{(65)}$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{\ }6;$ #MPa

$$\sigma_1 := 3.652632887 \quad \textbf{(66)}$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{\ }6;$ #MPa

$$\sigma_{m,1} := 48.04444490 \quad \textbf{(67)}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -51.69707779 \quad \textbf{(68)}$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; # <1.0 #--- >OK

$$Ver_{top,c} := -2.215589048 \tag{69}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,\,1}$; #MPa

$$\sigma_{b,c} := 44.39181201 \tag{70}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; # <1.0 #--- >NOT OK

$$Ver_{bottom,c} := 30.26714456 \tag{71}$$

3.4.2 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

$$\sigma_2 := 1.826316444 \tag{72}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$; #MPa

$$\sigma_{m,2} := 23.74933000 \tag{73}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -25.57564644 \tag{74}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 21.92301356 \tag{75}$$

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

$$Ver_{timber} := -0.637879203 \tag{76}$$

3.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3;$ #MPa

$$\tau_2 := 7.490186025 \quad (77)$$

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

$$Ver_{shear} := 2.691785603 \quad (78)$$

3.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ #kN

$$F_1 := 34.55958523 \quad (79)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ #kN

$$F_2 := 34.55958523 \quad (80)$$

> $f_{tens,\ k} := 20.0;$ #kN

$$f_{tens,k} := 20.0 \quad (81)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}};$ #<1.0 #-->OK

$$Ver_{F1} := 0.8279900630 \quad (82)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\ eff} := \max\left(solve\left(a_{1,\ 1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\ 1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\ 1}\right)\right);$ #mm

$$a_{1,eff} := 116.2447862 \quad (83)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\ eff}$; #mm

$$x := 6.210131824 \quad (84)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\ new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\ eff}$;

$$a_{2,new} := 0.6501479 \quad (85)$$

> $A_{1,\ eff} := b \cdot x$;

$$A_{1,eff} := 3726.079094 \quad (86)$$

> $I_{1,\ eff} := \frac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 11974.91562 \quad (87)$$

> $EI_{eff,\ tot,\ new} := E_1 \cdot I_{1,\ eff} + \gamma_1 \cdot E_1 \cdot A_{1,\ eff} \cdot a_{1,\ eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\ new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 7.724473842\ 10^{11} \quad (88)$$

5. New short-term verification

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\ eff} \cdot M_{Ed,\ new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_1 := 8.655404022 \quad (89)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\ new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,1} := 8.655404020 \quad (90)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -17.31080804 \quad (91)$$

Verification of the top section

> $Ver_{top,\, c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\, c}}{\gamma_c}}$; # <1.0 #--- >OK

$$Ver_{top,c} := -0.7418917731 \qquad \textbf{(92)}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\, c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := -2.\ 10^{-9} \qquad \textbf{(93)}$$

> $Ver_{bottom,\, c} := \frac{\sigma_{b,\, c}}{\frac{f_{ctk,\, 0.05,\, c}}{\gamma_c}}$; # <1.0 #--- >OK

$$Ver_{bottom,c} := -1.363636364\ 10^{-9} \qquad \textbf{(94)}$$

5.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\, new} \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 0.4479266191 \qquad \textbf{(95)}$$

> $\sigma_{m,\, 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 41.33766662 \qquad \textbf{(96)}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\, t} := -\sigma_2 - \sigma_{m,\, 2}$; #MPa

$$\sigma_{t,t} := -41.78559324 \qquad \textbf{(97)}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\, t} := -\sigma_2 + \sigma_{m,\, 2}$; #MPa

$$\sigma_{b,t} := 40.88974000 \qquad \textbf{(98)}$$

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

$$Ver_{timber} := -0.803128281 \qquad \textbf{(99)}$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})^{\wedge}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^{\wedge}3;$ #MPa

$\tau_2 := 3.330338948$ **(100)**

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

$Ver_{shear} := 1.196840559$ **(101)**

5.1.4 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

$F_1 := 8.476165154$ **(102)**

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed};$ #kN

$F_2 := 8.476164259$ **(103)**

> $f_{tens,\ k} := 20.0;$ #kN

$f_{tens,k} := 20.0$ **(104)**

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}};$ #<1.0 #--> OK

$Ver_{F1} := 0.2030747901$ **(105)**

6. Maximum load capacity based on ULS using long-term verifications of the slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,\ g} := \frac{E_{cm,\ c}}{1 + \varphi_c};$

(106)

$$E_{1,g} := 9714.285714 \quad \mathbf{(106)}$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad \mathbf{(107)}$$

> $q_k := 0;$

$$q_k := 0 \quad \mathbf{(108)}$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad \mathbf{(109)}$$

> $E_1 := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 9714.285715 \quad \mathbf{(110)}$$

6.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad \mathbf{(111)}$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad \mathbf{(112)}$$

> $E_2 := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad \mathbf{(113)}$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \frac{K_{ser}}{1+k_{def,t}};$

$$K_{ser,g} := 6720.420151 \quad \mathbf{(114)}$$

> $K_{ser,q} := \frac{K_{ser}}{1+k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 8724.755986 \quad \mathbf{(115)}$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 6720.420149 \quad \mathbf{(116)}$$

> $K_u := \frac{2}{3} \cdot K_{ser,\,2}$

$$K_u := 4480.280099 \tag{117}$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.04935924062 \tag{118}$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \tag{119}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}; \#mm$

$$a_2 := 4.512000371 \tag{120}$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2; \#mm$

$$a_1 := 85.48799963 \tag{121}$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2; \#Nmm\hat{}2$

$$EI_{eff,tot} := 6.301872574\ 10^{11} \tag{122}$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma\, c,t = -\ \sigma 1 - \sigma\, m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 4.423441749\ 10^7 \qquad \textbf{(123)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma\, c{,}b\ = -\ \sigma\, 1 + \sigma\, m{,}1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 3.690633319\ 10^6 \qquad \textbf{(124)}$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\, t\ = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\,;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 4.442789930\ 10^7 \qquad \textbf{(125)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b \;=\; -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 5.476970087\ 10^7 \qquad \textbf{(126)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

$$> M_{Ed,\,new} := \frac{M_4}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 54.76970087 \qquad \textbf{(127)}$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \qquad \textbf{(128)}$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \qquad \textbf{(129)}$$

$$> P_{Ed} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right); \#kN$$

$$P_{Ed} := 143.6170657 \qquad \textbf{(130)}$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{\ }6;$ #*MPa*

$$\sigma_1 := 3.562503084 \quad (131)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{\ }6;$ #*MPa*

$$\sigma_{m,1} := 25.32811557 \quad (132)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1};$ #*MPa*

$$\sigma_{c,t} := -28.89061865 \quad (133)$$

Verification of the top section

> $Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}};$ #*<1.0 #--->OK*

$$Ver_{top,c} := -1.238169371 \quad (134)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1};$ #*MPa*

$$\sigma_{b,c} := 21.76561249 \quad (135)$$

> $Ver_{bottom,\ c} := \frac{\sigma_{b,\ c}}{\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c}};$ #*<1.0 #--->NOT OK*

$$Ver_{bottom,c} := 14.84019033 \quad (136)$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{\ }6;$ #*MPa*

$$\sigma_2 := 1.781251542 \quad (137)$$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{\ }6;$ #*MPa*

$$\sigma_{m,2} := 23.68685367 \quad (138)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -25.46810521 \quad (139)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 21.90560213 \quad (140)$$

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

$$Ver_{timber} := -0.631672981 \quad (141)$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 7.453397841 \quad (142)$$

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

$$Ver_{shear} := 2.678564849 \quad (143)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 33.62973382 \quad (144)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 33.62973382 \quad (145)$$

> $f_{tens,\,k} := 20.0$; #kN

(146)

$$f_{tens,k} := 20.0 \tag{146}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; # <1.0 #-->OK

$$Ver_{F1} := 0.8057123730 \tag{147}$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 113.2933254 \tag{148}$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 11.18414502 \tag{149}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 1.1146021 \tag{150}$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 6710.487012 \tag{151}$$

> $I_{1,\,eff} := \dfrac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 69948.49480 \tag{152}$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 4.348484069\ 10^{11} \tag{153}$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\, eff} \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_1 := 6.842048564 \tag{154}$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_{m,1} := 6.842048565 \tag{155}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ *#MPa*

$$\sigma_{c,t} := -13.68409713 \tag{156}$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$ *# <1.0 #--- >OK*

$$Ver_{top,c} := -0.5864613056 \tag{157}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1};$ *#MPa*

$$\sigma_{b,c} := 1.\ 10^{-9} \tag{158}$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};$ *# <1.0 #--- >OK*

$$Ver_{bottom,c} := 6.818181818\ 10^{-10} \tag{159}$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\, new} \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_2 := 0.6376872264 \tag{160}$$

> $\sigma_{m,\,2} := \frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new}\right)}{EI_{eff,tot,new}} \cdot 10\hat{\ }6;$ *#MPa*

$$\sigma_{m,2} := 34.32725776 \tag{161}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2};$ *#MPa*

$$\sigma_{t,t} := -34.96494499 \tag{162}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2};$ *#MPa*

$$\sigma_{b,t} := 33.68957053 \tag{163}$$

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*

$$Ver_{timber} := -0.696813172 \tag{164}$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)\hat{\ }2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{\ }3;$ *#MPa*

$$\tau_2 := 2.801647695 \tag{165}$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ *#<1.0 #-->OK*

$$Ver_{shear} := 1.006842141 \tag{166}$$

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*

$$F_1 := 12.03942853 \tag{167}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed};$ *#kN*

$$F_2 := 12.03942912 \tag{168}$$

> $f_{tens,\ k} := 20.0;$ #*kN*

$$f_{tens,k} := 20.0 \qquad \textbf{(169)}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}};$ # *<1.0 #--> NOT OK*

$$Ver_{F1} := 0.2884446418 \qquad \textbf{(170)}$$

>

>

>

Load capacity predictions based on ULS
KOP-screws 10-140 mm
B45

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: #*mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: #*mm*

Concrete data, B45

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: #*mm*

> $A_c := h_c \cdot b$; #*mm^2*

$$A_c := 36000 \qquad \textbf{(1)}$$

> $I_c := \frac{(b \cdot h_c\hat{\ }3)}{12}$: #*mm^4*

> $E_{cm,\,c} := 36000$: #*MPa*

> $f_{ck,\,c} := 45$: #*MPa*

> $f_{ctk,\,0.05,\,c} := 2.7$: #*MPa*

> $\rho_c := 25.00$: #$\frac{kN}{m\hat{\ }3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (2)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} :\#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (3)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (4)$$

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

> $g_{0,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right)$;# $\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad \textbf{(5)}$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,k}\cdot\gamma_{G,1}$:# $\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1$; #mm^2

$$A_1 := 18000 \quad \textbf{(6)}$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad \textbf{(7)}$$

> $I_{t1} := \frac{(b\cdot h_1\hat{\ }3)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad \textbf{(8)}$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad \textbf{(9)}$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (10)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (11)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (12)$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (13)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (14)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (15)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (16)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (17)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (18)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (19)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad (20)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad (21)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad (22)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad (23)$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad (24)$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad (25)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (26)$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad (27)$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \quad (28)$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad (29)$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \quad (30)$$

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_5 := 473850000000 \quad (31)$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (32)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (33)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad \textbf{(34)}$$

> $(GA)_{eff} := evalf\left((a\hat{\ }2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad \textbf{(35)}$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad \textbf{(36)}$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{\ }2}}$; # $\frac{N}{mm\hat{\ }2} \cdot mm\hat{\ }4$

$$EI_{app} := 7.260572161\ 10^{11} \quad \textbf{(37)}$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t\hat{}3}{12}}$; $\# \frac{N}{mm\hat{}}$

$$E_{CLT} := 8403.440000 \quad (38)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using verification of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}$; $\# \frac{N}{mm\hat{}2}$

$$E_1 := 36000 \quad (39)$$

> $E_2 := E_{CLT}$; $\# \frac{N}{mm}$

$$E_2 := 8403.440000 \quad (40)$$

> $h_1 := h_c$; $\#mm$

$$h_1 := 60 \quad (41)$$

> $h_2 := h_t$; $\#mm$

$$h_2 := 120 \quad (42)$$

> $A_1 := A_c$;$\#mm\hat{}2$

$$A_1 := 36000 \quad (43)$$

> $A_2 := h_2 \cdot b$; $\#mm\hat{}2$

$$A_2 := 72000 \quad (44)$$

> $I_1 := I_c$; $\#mm\hat{}4$

$$I_1 := 10800000 \quad (45)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; $\#mm\hat{}4$

$$I_2 := 86400000 \quad (46)$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22};$

$$\rho_{m,1} := 470 \quad \textbf{(47)}$$

> $\rho_{m,\,2} := t_{t15};$

$$\rho_{m,2} := 430 \quad \textbf{(48)}$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right);$

$$\rho_m := 449.5553359 \quad \textbf{(49)}$$

> $d_{kop} := 10$;#mm

$$d_{kop} := 10 \quad \textbf{(50)}$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m \hat{}1.5 \cdot 3\right);$

$$K_{ser} := 12432.77728 \quad \textbf{(51)}$$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right); \#\frac{N}{mm}$

$$K_u := 8288.518187 \quad \textbf{(52)}$$

>

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;$ #mm

$$s := 100 \quad \textbf{(53)}$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi \hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L \hat{}2}}\right);$

$$\gamma_1 := 0.02526496580 \quad \textbf{(54)}$$

> $\gamma_2 := 1.0;$ #Fully composite

$$\gamma_2 := 1.0 \quad \textbf{(55)}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot \left(h_1 + h_2\right)}{2 \cdot \left(\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2\right)};$ #mm

(56)

$$a_2 := 4.620487373 \quad \textbf{(56)}$$

>

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 85.37951263 \quad \textbf{(57)}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 1.366462581\ 10^{12} \quad \textbf{(58)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\ 1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \dfrac{f_{ck,\ c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 3.541110507\ 10^7 \quad \textbf{(59)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\ 1} = \dfrac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_2 := 2.453880692\ 10^6 \quad \textbf{(60)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,\, t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.195500916\ 10^7 \quad \textbf{(61)}$$

\>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 6.437449365\ 10^7 \quad (62)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}$; #kNm

$$M_{Ed,new} := 35.41110507 \quad (63)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (64)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (65)$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed} := 91.99414353 \quad (66)$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_1 := 2.012410726 \quad (67)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_{m,1} := 27.98758928 \quad (68)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -30.00000001 \quad (69)$$

Verification of the top section

> $Ver_{top,\ c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\ c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -1.000000000 \quad (70)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 25.97517855 \quad (71)$$

> $Ver_{bottom,\ c} := \dfrac{\sigma_{b,\ c}}{\dfrac{f_{ctk,\ 0.05,\ c}}{\gamma_c}}$; #<1.0 #--->NOT OK

$$Ver_{bottom,c} := 14.43065475 \quad (72)$$

3.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10^{\wedge}6$; #MPa

$$\sigma_2 := 1.006205363 \quad (73)$$

> $\sigma_{m,\ 2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^{\wedge}6$; #MPa

$$\sigma_{m,2} := 13.06622373 \quad (74)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2}$; #MPa

$$\sigma_{t,t} := -14.07242909 \quad (75)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2}$; #MPa

$$\sigma_{b,t} := 12.06001837 \quad (76)$$

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

$$Ver_{timber} := -0.3511028046 \quad (77)$$

3.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3;$ *#MPa*

$$\tau_2 := 4.073352896 \quad (78)$$

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*

$$Ver_{shear} := 1.463861197 \quad (79)$$

3.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_1 := 18.82087562 \quad (80)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_2 := 18.82087563 \quad (81)$$

> $f_{tens,\,k} := 20.0;$ *#kN*

$$f_{tens,k} := 20.0 \quad (82)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}};$ *#<1.0 #-->OK*

$$Ver_{F1} := 0.4509168117 \quad (83)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right);$ *#mm*

$$a_{1,eff} := 116.4402144 \quad (84)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\ eff}$; #mm

$$x := 5.883716070 \quad (85)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\ new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\ eff}$;

$$a_{2,new} := 0.6179276 \quad (86)$$

> $A_{1,\ eff} := b \cdot x$;

$$A_{1,eff} := 3530.229642 \quad (87)$$

> $I_{1,\ eff} := \frac{b \cdot x^3}{12}$

$$I_{1,eff} := 10184.15792 \quad (88)$$

> $EI_{eff,\ tot,\ new} := E_1 \cdot I_{1,\ eff} + \gamma_1 \cdot E_1 \cdot A_{1,\ eff} \cdot a_{1,\ eff}^2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\ new}^2$; #Nmm^2

$$EI_{eff,tot,new} := 7.701890334\ 10^{11} \quad (89)$$

5. New short-term verifications

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load 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

$$\sigma_1 := 4.869298079 \quad (90)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\ new})}{EI_{eff,tot,new}} \cdot 10^6$; #MPa

$$\sigma_{m,1} := 4.869298082 \quad (91)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -9.738596161 \quad (92)$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.3246198720 \tag{93}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 3.\,10^{-9} \tag{94}$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom,c} := 1.666666666\;10^{-9} \tag{95}$$

5.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_2 := 0.2387464083 \tag{96}$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{\ }6$; #MPa

$$\sigma_{m,2} := 23.18197875 \tag{97}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -23.42072516 \tag{98}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 22.94323234 \tag{99}$$

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

$$Ver_{timber} := -0.448990605 \tag{100}$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)^{\wedge}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10^{\wedge}3;$ #*MPa*

$$\tau_2 := 1.844132524 \qquad (101)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ #<1.0 #-->OK

$$Ver_{shear} := 0.6627351259 \qquad (102)$$

5.1.4 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*

$$F_1 := 4.465704882 \qquad (103)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed};$ #*kN*

$$F_2 := 4.465705134 \qquad (104)$$

> $f_{tens,\,k} := 20.0;$ #*kN*

$$f_{tens,k} := 20.0 \qquad (105)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}};$ #<1.0 #--> OK

$$Ver_{F1} := 0.1069908461 \qquad (106)$$

6. Maximum load capacity based on ULS using long-term verifications of the slab

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c};$

(107)

$$E_{1,g} := 10285.71429 \quad (107)$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2};$

$$E_{1,q} := 16000.00000 \quad (108)$$

> $q_k := 0;$

$$q_k := 0 \quad (109)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (110)$$

> $E_1 := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_1 := 10285.71429 \quad (111)$$

6.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1 + k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (112)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (113)$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_2 := 4542.399998 \quad (114)$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 6720.420151 \quad (115)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 8724.755986 \quad (116)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 6720.420149 \quad (117)$$

> $K_u := \frac{2}{3} \cdot K_{ser,\,2}$

$$K_u := 4480.280099 \quad (118)$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.04674524444 \quad (119)$$

> $\gamma_2 := 1.0;$

$$\gamma_2 := 1.0 \quad (120)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}; \#mm$

$$a_2 := 4.523783491 \quad (121)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2; \#mm$

$$a_1 := 85.47621651 \quad (122)$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2; \#Nmm\hat{}2$

$$EI_{eff,tot} := 6.367055192\ 10^{11} \quad (123)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,1} = \frac{f_{ck,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_1 := 5.462640462\ 10^7 \quad \textbf{(124)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,0.005,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,1} = \frac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_2 := 4.284793890\ 10^6 \quad \textbf{(125)}$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^\wedge 6;\right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^\wedge 6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

$$> \ M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 4.487636906\ 10^7 \tag{126}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\ b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> \ M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 5.535302887\ 10^7 \tag{127}$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

$$> \ M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10^\wedge 6}; \#kNm$$

$$M_{Ed,new} := 44.87636906 \tag{128}$$

$$> \ L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \tag{129}$$

$$> \ L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \tag{130}$$

$$> \ P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}{}^\wedge 2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed} := 117.2348475 \tag{131}$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10^{\wedge}6;$ #*MPa*

$$\sigma_1 := 2.896651705 \quad (132)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^{\wedge}6;$ #*MPa*

$$\sigma_{m,1} := 21.74877537 \quad (133)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1};$ #*MPa*

$$\sigma_{c,t} := -24.64542708 \quad (134)$$

Verification of the top section

> $Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}};$ # <1.0 #--->OK

$$Ver_{top,c} := -0.8215142360 \quad (135)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1};$ #*MPa*

$$\sigma_{b,c} := 18.85212366 \quad (136)$$

> $Ver_{bottom,\ c} := \frac{\sigma_{b,\ c}}{\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c}};$ # <1.0 #--->NOT OK

$$Ver_{bottom,c} := 10.47340203 \quad (137)$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10^{\wedge}6;$ #*MPa*

$$\sigma_2 := 1.448325852 \quad (138)$$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^{\wedge}6;$ #*MPa*

$$\sigma_{m,2} := 19.20948500 \quad (139)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; *#MPa*

$$\sigma_{t,t} := -20.65781085 \quad (140)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; *#MPa*

$$\sigma_{b,t} := 17.76115915 \quad (141)$$

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*

$$Ver_{timber} := -0.5126963285 \quad (142)$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{\ }2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{\ }3$; *#MPa*

$$\tau_2 := 6.021933839 \quad (143)$$

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*

$$Ver_{shear} := 2.164132474 \quad (144)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; *#kN*

$$F_1 := 27.24192489 \quad (145)$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; *#kN*

$$F_2 := 27.24192489 \quad (146)$$

> $f_{tens,\,k} := 20.0$; *#kN*

(147)

$$f_{tens,k} := 20.0 \quad \textbf{(147)}$$

$$> Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}; \# < 1.0 \ \# \text{-->} OK$$

$$Ver_{F1} := 0.6526711173 \quad \textbf{(148)}$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

$$> a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right); \#mm$$

$$a_{1,eff} := 113.6239566 \quad \textbf{(149)}$$

The effective compressed height of the concrete:

$$> x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}; \#mm$$

$$x := 10.62275925 \quad \textbf{(150)}$$

Distance between the centre of the timber and the centre of gravity

$$> a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff};$$

$$a_{2,new} := 1.0646638 \quad \textbf{(151)}$$

$$> A_{1,\,eff} := b \cdot x;$$

$$A_{1,eff} := 6373.655550 \quad \textbf{(152)}$$

$$> I_{1,\,eff} := \frac{b \cdot x\hat{}3}{12}$$

$$I_{1,eff} := 59935.20860 \quad \textbf{(153)}$$

$$> EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2; \#Nmm\hat{}2$$

$$EI_{eff,tot,new} := 4.330145630 \; 10^{11} \quad \textbf{(154)}$$

9. New long-term verification

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\, eff} \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6 : \#MPa$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}; \#MPa$

$$\sigma_{c,t} := -11.32366477 \qquad \textbf{(155)}$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}; \# < 1.0 \#{-}{-}{-}{>}OK$

$$Ver_{top,c} := -0.3774554924 \qquad \textbf{(156)}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}; \#MPa$

$$\sigma_{b,c} := 0. \qquad \textbf{(157)}$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}; \# < 1.0 \#{-}{-}{-}{>}OK$

$$Ver_{bottom,c} := 0. \qquad \textbf{(158)}$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\, new} \cdot M_{Ed,\, new})}{EI_{eff,tot,new}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6 : \#MPa$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -28.74686926 \tag{159}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 27.74446462 \tag{160}$$

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

$$Ver_{timber} := -0.570718117 \tag{161}$$

9.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \dfrac{0.5 \cdot E_2 \cdot b \cdot \left(0.5 \cdot h_2 + a_{2,new}\right)\hat{}2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 2.292923367 \tag{162}$$

Verification of the timber section

> $Ver_{shear} := \dfrac{\tau_2}{\dfrac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{shear} := 0.8240193350 \tag{163}$$

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

$$F_1 := 9.427240993 \tag{164}$$

> $F_2 := \dfrac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 9.427240342 \tag{165}$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{166}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.2258609821 \quad \textbf{(167)}$$

>

>

Load capacity predictions based on ULS
KOP-screws 10-140 mm

Using the average result from compressive test of concrete cubes

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

$$L := 2000 \tag{1}$$

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

$$b := 600 \tag{2}$$

Concrete data, B35*

*strength average from compressive strength test of concrete cubes
*Partial factor is 1.0

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

$$h_c := 60 \tag{3}$$

> $A_c := h_c \cdot b$: *#mm^2*

$$A_c := 36000 \tag{4}$$

> $I_c := \frac{(b \cdot h_c \hat{} 3)}{12}$: *#mm^4*

$$I_c := 10800000 \tag{5}$$

> $E_{cm,\,c} := 34000$: *#MPa*

$$E_{cm,c} := 34000 \tag{6}$$

> $f_{ck,\ c} := 45.4553333 : \#MPa$

$$f_{ck,c} := 45.4553333 \qquad (7)$$

> $f_{ctk,\ 0.05,\ c} := 2.2 : \#MPa$

$$f_{ctk,0.05,c} := 2.2 \qquad (8)$$

> $\rho_c := 25.00 : \#\frac{kN}{m\hat{}3}$

$$\rho_c := 25.00 \qquad (9)$$

> $\gamma_c := 1.0 :$

$$\gamma_c := 1.0 \qquad (10)$$

> $\varphi_c := 2.5 :$

$$\varphi_c := 2.5 \qquad (11)$$

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

$$h_1 := 30 \qquad (12)$$

> $h_2 := 20 : \#mm$

$$h_2 := 20 \qquad (13)$$

> $h_3 := 20 : \#mm$

$$h_3 := 20 \qquad (14)$$

> $h_4 := 20 : \#mm$

$$h_4 := 20 \qquad (15)$$

> $h_5 := 30 : \#mm$

$$h_5 := 30 \qquad (16)$$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \qquad (17)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

$$\gamma_M := 1.15 \qquad (18)$$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

$$Klima := 1.0 \quad (19)$$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

$$k_{modi,t} := 0.8 \quad (20)$$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

$$k_{def,t} := 0.85 \quad (21)$$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm^2}$

$$E_{0,mean,t22} := 13000 \quad (22)$$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm^2}$

$$E_{90,mean,t22} := 430 \quad (23)$$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm^2}$

$$G_{0,mean,t22} := 810 \quad (24)$$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm^2}$

$$G_{90,mean,t22} := 81 \quad (25)$$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm^2}$

$$G_{R,t22} := 81 \quad (26)$$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm^2}$

$$f_{m,k,t22} := 30.5 \quad (27)$$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm^2}$

$$f_{t,0,k,t22} := 22.0 \quad (28)$$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm^2}$

$$f_{v,k,t22} := 4.0 \quad (29)$$

> $t_{t22} := 470 : \#\frac{kg}{m^3}$

$$t_{t22} := 470 \quad (30)$$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m^3}$

$$\rho_{t22} := 4.609118725 \quad (31)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{}2}$

$$E_{0,mean,t15} := 11500 \quad (32)$$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\hat{}2}$

$$E_{90,mean,t15} := 230 \quad (33)$$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\hat{}2}$

$$G_{0,mean,t15} := 720 \quad (34)$$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\hat{}2}$

$$G_{90,mean,t15} := 72 \quad (35)$$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\hat{}2}$

$$G_{R,t15} := 72 \quad (36)$$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\hat{}2}$

$$f_{m,k,t15} := 22 \quad (37)$$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

$$f_{t,0,k,t15} := 15.0 \quad (38)$$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

$$f_{v,k,t15} := 4.0 \quad (39)$$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

$$t_{t15} := 430 \quad (40)$$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (41)$$

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

> $g_{0,\,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right);\#\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad \textbf{(42)}$$

1. 1 ULS

> $f_{d,\,ULS} := g_{0,\,k}\cdot\gamma_{G,\,1}:\#\frac{kN}{m}$

$$f_{d,ULS} := 1.461281992 \quad \textbf{(43)}$$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$; #mm^2

$$A_1 := 18000 \quad (44)$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad (45)$$

> $I_{t1} := \frac{(b \cdot h_1\hat{}3)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad (46)$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad (47)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (48)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (49)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (50)$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (51)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (52)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (53)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (54)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (55)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (56)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (57)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad (58)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad (59)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad (60)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad (61)$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad (62)$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad (63)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (64)$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad (65)$$

> $(EAz\hat{\ }2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_2 := 1104000000 \quad (66)$$

> $(EAz\hat{\ }2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_3 := 0 \quad (67)$$

> $(EAz\hat{\ }2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_4 := 1104000000 \quad (68)$$

> $(EAz\hat{\ }2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{\ }2)$; #Nmm^2

$$(EAz\hat{\ }2)_5 := 473850000000 \quad (69)$$

> $(EAz\hat{\ }2)_{sum} := (EAz\hat{\ }2)_1 + (EAz\hat{\ }2)_2 + (EAz\hat{\ }2)_3 + (EAz\hat{\ }2)_4 + (EAz\hat{\ }2)_5$; #Nmm^2

$$(EAz\hat{\ }2)_{sum} := 949908000000 \quad (70)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{\ }2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (71)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (72)$$

> $(GA)_{eff} := evalf\left((a\hat{\ }2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (73)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (74)$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^{\wedge}2}}; \# \frac{N}{mm^{\wedge}2} \cdot mm^{\wedge}4$

$$EI_{app} := 7.260572161\ 10^{11} \quad (75)$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t^{\wedge}3}{12}}; \# \frac{N}{mm^{\wedge}}$

$$E_{CLT} := 8403.440000 \quad (76)$$

3. Gamma method, Annex B, EC5 Maximum load capacity based on ULS using verification of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}; \# \frac{N}{mm^{\wedge}2}$

$$E_1 := 34000 \quad (77)$$

> $E_2 := E_{CLT}; \# \frac{N}{mm}$

$$E_2 := 8403.440000 \quad (78)$$

> $h_1 := h_c; \#mm$

$$h_1 := 60 \quad (79)$$

> $h_2 := h_t; \#mm$

$$h_2 := 120 \quad (80)$$

> $A_1 := A_c$;#mm^2

$$A_1 := 36000 \quad (81)$$

> $A_2 := h_2 \cdot b$; #mm^2

$$A_2 := 72000 \quad (82)$$

> $I_1 := I_c$; #mm^4

$$I_1 := 10800000 \quad (83)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \quad (84)$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22}$;

$$\rho_{m,1} := 470 \quad (85)$$

> $\rho_{m,\,2} := t_{t15}$;

$$\rho_{m,2} := 430 \quad (86)$$

> $\rho_m := evalf\left(\text{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right)$;

$$\rho_m := 449.5553359 \quad (87)$$

> $d_{kop} := 10$;#mm

$$d_{kop} := 10 \quad (88)$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m\hat{}1.5 \cdot 3\right)$;

$$K_{ser} := 12432.77728 \quad (89)$$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right)$; $\#\frac{N}{mm}$

$$K_u := 8288.518187 \quad (90)$$

>

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;$ #mm

$$s := 100 \quad (91)$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1+\frac{\pi^\wedge 2\cdot E_1\cdot s\cdot A_1}{K_u\cdot L^\wedge 2}}\right);$

$$\gamma_1 := 0.02671144241 \quad (92)$$

> $\gamma_2 := 1.0;$ #Fully composite

$$\gamma_2 := 1.0 \quad (93)$$

> $a_2 := \frac{\gamma_1\cdot E_1\cdot A_1\cdot(h_1+h_2)}{2\cdot(\gamma_1\cdot E_1\cdot A_1+\gamma_2\cdot E_2\cdot A_2)};$ #mm

$$a_2 := 4.613982231 \quad (94)$$

>

> $a_1 := \frac{(h_1+h_2)}{2} - a_2;$ #mm

$$a_1 := 85.38601777 \quad (95)$$

> $EI_{eff,\ tot} := E_1\cdot I_1 + \gamma_1\cdot E_1\cdot A_1\cdot a_1^\wedge 2 + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_2^\wedge 2;$ #Nmm^2

$$EI_{eff,tot} := 1.344508348\ 10^{12} \quad (96)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,\ 1})}{EI_{eff,\ tot}}\cdot 10^\wedge 6 :$ #MPa

$$\sigma_1 := 0.05767658177\ M_{Ed,1} \quad (97)$$

> $\sigma_{m,\ 1} := \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^\wedge 6 :$ #MPa

$$\sigma_{m,1} := 0.7586416265\ M_{Ed,1} \quad (98)$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,1} = \frac{f_{ck,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_1 := 5.568335097\ 10^7 \qquad (99)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,0.005,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,1} = \frac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_2 := 3.138530254\ 10^6 \qquad (100)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

$$\sigma_2 := 0.02883829089\ M_{Ed,2} \qquad (101)$$

$$> \sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

$$\sigma_{m,2} := 0.3750117288\ M_{Ed,2} \qquad (102)$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma t,t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 5.112721903\ 10^7 \quad \textbf{(103)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\ b\ =\ -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 6.332956592\ 10^7 \quad \textbf{(104)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}; \#kNm$

$$M_{Ed,new} := 51.12721903 \quad \textbf{(105)}$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \quad \textbf{(106)}$$

> $L_{sup} := 2.0; \#m$

$$L_{sup} := 2.0 \quad \textbf{(107)}$$

$$> P_{Ed} := solve\left(\frac{P_{Ed,\,1}\cdot L_{out}}{2} + \frac{1.5\cdot g_{0,k}\cdot L_{sup}\hat{}2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right); \#kN$$

$$P_{Ed} := 133.9037808 \qquad \textbf{(108)}$$

3.4 Verification of the Maximum load

3.4.1 Normal stresses in the **CONCRETE** section

$$> \sigma_1 := \frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,new})}{EI_{eff,\,tot}}\cdot 10\hat{}6; \#MPa$$

$$\sigma_1 := 2.948843229 \qquad \textbf{(109)}$$

$$> \sigma_{m,\,1} := \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,new})}{EI_{eff,tot}}\cdot 10\hat{}6; \#MPa$$

$$\sigma_{m,1} := 38.78723660 \qquad \textbf{(110)}$$

Stresses at the TOP of the concrete section

$$> \sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}; \#MPa$$

$$\sigma_{c,t} := -41.73607983 \qquad \textbf{(111)}$$

Verification of the top section

$$> Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}}; \# < 1.0 \ \#{-}{-}{-}> OK$$

$$Ver_{top,c} := -0.9181778418 \qquad \textbf{(112)}$$

Stresses at the BOTTOM of the concrete section

$$> \sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}; \#MPa$$

$$\sigma_{b,c} := 35.83839337 \qquad \textbf{(113)}$$

$$> Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}}; \# < 1.0 \ \#{-}{-}{-}> NOT\ OK$$

$$Ver_{bottom,c} := 16.29017880 \qquad \textbf{(114)}$$

3.4.2 Normal stresses in the **TIMBER** section

$$> \sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,new})}{EI_{eff,\,tot}}\cdot 10\hat{}6; \#MPa$$

(115)

$$\sigma_2 := 1.474421614 \quad \textbf{(115)}$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6;$ *#MPa*

$$\sigma_{m,2} := 19.17330680 \quad \textbf{(116)}$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; *#MPa*

$$\sigma_{t,t} := -20.64772841 \quad \textbf{(117)}$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; *#MPa*

$$\sigma_{b,t} := 17.69888519 \quad \textbf{(118)}$$

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*

$$Ver_{timber} := -0.5149725757 \quad \textbf{(119)}$$

3.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3;$ *#MPa*

$$\tau_2 := 6.025858600 \quad \textbf{(120)}$$

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*

$$Ver_{shear} := 2.165542934 \quad \textbf{(121)}$$

3.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed};$ *#kN*

$$F_1 := 27.80320451 \quad \textbf{(122)}$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 27.80320451 \quad (123)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (124)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{F1} := 0.6661184413 \quad (125)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

4. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2 \cdot \left(4 \cdot \gamma_1\hat{}2 \cdot E_1 \cdot b\right) + a_{1,\,1} \cdot \left(2 \cdot E_2 \cdot A_2 \cdot \left(1 + \gamma_1\right)\right) - E_2 \cdot A_2 \cdot \left(2 \cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right)$; #mm

$$a_{1,eff} := 116.2447862 \quad (126)$$

The effective compressed height of the concrete:

> $x := 2 \cdot \gamma_1 \cdot a_{1,\,eff}$; #mm

$$x := 6.210131824 \quad (127)$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5 \cdot x + 0.5 \cdot h_2 - a_{1,\,eff}$;

$$a_{2,new} := 0.6501479 \quad (128)$$

> $A_{1,\,eff} := b \cdot x$;

$$A_{1,eff} := 3726.079094 \quad (129)$$

> $I_{1,\,eff} := \frac{b \cdot x\hat{}3}{12}$

$$I_{1,eff} := 11974.91562 \quad (130)$$

> $EI_{eff,\,tot,\,new} := E_1 \cdot I_{1,\,eff} + \gamma_1 \cdot E_1 \cdot A_{1,\,eff} \cdot a_{1,\,eff}\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,\,new}\hat{}2$; #Nmm^2

$$EI_{eff,tot,new} := 7.724473842\ 10^{11} \quad (131)$$

5. New short-term verifications

Including the new modifed parameters into the verification calculations

5.1 Verification of the Maximum load using new parameters

5.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_{1,\,eff} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #MPa

$$\sigma_1 := 6.987679937 \tag{132}$$

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_1 \cdot x \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{}6;$ #MPa

$$\sigma_{m,1} := 6.987679940 \tag{133}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1};$ #MPa

$$\sigma_{c,t} := -13.97535988 \tag{134}$$

Verification of the top section

> $Ver_{top,\,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\,c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{top,c} := -0.3074525884 \tag{135}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1};$ #MPa

$$\sigma_{b,c} := 3.\ 10^{-9} \tag{136}$$

> $Ver_{bottom,\,c} := \frac{\sigma_{b,\,c}}{\frac{f_{ctk,\,0.05,\,c}}{\gamma_c}};$ #<1.0 #--->OK

$$Ver_{bottom,c} := 1.363636364\ 10^{-9} \tag{137}$$

5.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10\hat{\ }6;$ #MPa

$$\sigma_2 := 0.3616200747 \quad (138)$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10\hat{\ }6;$ #MPa

$$\sigma_{m,2} := 33.37272102 \quad (139)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2};$ #MPa

$$\sigma_{t,t} := -33.73434109 \quad (140)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2};$ #MPa

$$\sigma_{b,t} := 33.01110095 \quad (141)$$

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

$$Ver_{timber} := -0.648381445 \quad (142)$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_{2,new})\hat{\ }2}{b \cdot EI_{eff,tot,new}} \cdot P_{Ed} \cdot 10\hat{\ }3;$ #MPa

$$\tau_2 := 2.679259437 \quad (143)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}};$ #<1.0 #-->OK

$$Ver_{shear} := 0.9628588601 \quad (144)$$

5.1.4 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

$$F_1 := 6.819079329 \quad (145)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 6.819078608 \quad (146)$$

> $f_{tens,\ k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (147)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.1633737756 \quad (148)$$

6. Long-term verification - ULS

6.1 New elasticity modulus calculated:

6.1.1 Concrete

> $E_{1,\ g} := \frac{E_{cm,\ c}}{1 + \varphi_c}$;

$$E_{1,g} := 9714.285714 \quad (149)$$

> $E_{1,\ q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad (150)$$

> $q_k := 0$;

$$q_k := 0 \quad (151)$$

> $g_{1,\ k} := 0$;

$$g_{1,k} := 0 \quad (152)$$

> $E_1 := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,\ q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\ k} + g_{1,k}) \cdot \gamma_{G,\ 1} + q_k \cdot \gamma_{Q,\ 1}}$;

$$E_1 := 9714.285715 \quad (153)$$

6.1.2 CLT

> $E_{2,\ g} := \frac{E_{CLT}}{1 + k_{def,\ t}}$;

$$E_{2,g} := 4542.400000 \quad (154)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2}$;

$$E_{2,q} := 5897.150877 \quad (155)$$

> $E_2 := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_2 := 4542.399998 \quad (156)$$

6.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}}$;

$$K_{ser,g} := 6720.420151 \quad (157)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2}$;

$$K_{ser,q} := 8724.755986 \quad (158)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$K_{ser,2} := 6720.420149 \quad (159)$$

> $K_u := \dfrac{2}{3} \cdot K_{ser,2}$

$$K_u := 4480.280099 \quad (160)$$

Repeating step 3-5

7. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right)$;

$$\gamma_1 := 0.04935924062 \quad (161)$$

> $\gamma_2 := 1.0$;

$$\gamma_2 := 1.0 \quad (162)$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 4.512000371 \quad (163)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 85.48799963 \quad (164)$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 6.301872574\ 10^{11} \quad (165)$$

7.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

$$\sigma_1 := 0.06504514407\ M_{Ed,1} \quad (166)$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

$$\sigma_{m,1} := 0.4624475789\ M_{Ed,1} \quad (167)$$

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 8.617243673\ 10^{7} \quad (168)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$

> $M_2 := solve\left(M_{Ed,1} = \frac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,tot}}\right)}, M_{Ed,1}\right); \#Nmm$

$$M_2 := 5.535949978\ 10^6 \quad (169)$$

7.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

$$\sigma_2 := 0.03252257204\ M_{Ed,2} \quad (170)$$

> $\sigma_{m,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

$$\sigma_{m,2} := 0.4324809756\ M_{Ed,2} \quad (171)$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t, t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 4.442789930\ 10^7 \quad (172)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma t, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 5.476970087\ 10^7 \tag{173}$$

Need to neglect the bending moment for the bottom of concrete (M2)

7.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$

$$M_{Ed,new} := 44.42789930 \tag{174}$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \tag{175}$$

> $L_{sup} := 2.0; \#m$

$$L_{sup} := 2.0 \tag{176}$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$

$$P_{Ed} := 116.0389281 \tag{177}$$

7.4 Verification of the Maximum load

7.4.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,new})}{EI_{eff,\ tot}} \cdot 10\hat{}6; \#MPa$

$$\sigma_1 := 2.889819111 \tag{178}$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6; \#MPa$

$$\sigma_{m,1} := 20.54557446 \tag{179}$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -23.43539357 \quad (180)$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.5155697224 \quad (181)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 17.65575535 \quad (182)$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->NOT OK

$$Ver_{bottom,c} := 8.025343341 \quad (183)$$

7.4.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,new})}{EI_{eff,\,tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_2 := 1.444909556 \quad (184)$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6$; #MPa

$$\sigma_{m,2} := 19.21422123 \quad (185)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -20.65913079 \quad (186)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

$$\sigma_{b,t} := 17.76931167 \quad (187)$$

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

$$Ver_{timber} := -0.5123983378 \quad (188)$$

7.4.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot h_2\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3$; #MPa

$$\tau_2 := 6.022155458 \quad (189)$$

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

$$Ver_{shear} := 2.164212117 \quad (190)$$

7.4.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_1 := 27.17196766 \quad (191)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 27.17196766 \quad (192)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (193)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{F1} := 0.6509950587 \quad (194)$$

The Verifications of both timber section and concrete section is NOT OK. So, modifiactions due to the concrete cross ection has to be done.
Following the " Design of timber-concrete composite structures: A state-of-the-art report by COST Action FP1402" **page 134**

8. Using quadratic equation

The distance between the centroid of the concrete slab and the centre of gravity

> $a_{1,\,eff} := \max\left(solve\left(a_{1,\,1}\hat{}2\cdot\left(4\cdot\gamma_1\hat{}2\cdot E_1\cdot b\right) + a_{1,\,1}\cdot\left(2\cdot E_2\cdot A_2\cdot\left(1+\gamma_1\right)\right) - E_2\cdot A_2\cdot\left(2\cdot h_1 + h_2\right) = 0, a_{1,\,1}\right)\right); \#mm$

$$a_{1,eff} := 113.2933254 \quad \textbf{(195)}$$

The effective compressed height of the concrete:

> $x := 2\cdot\gamma_1\cdot a_{1,\,eff}; \#mm$

$$x := 11.18414502 \quad \textbf{(196)}$$

Distance between the centre of the timber and the centre of gravity

> $a_{2,\,new} := h_1 - 0.5\cdot x + 0.5\cdot h_2 - a_{1,\,eff};$

$$a_{2,new} := 1.1146021 \quad \textbf{(197)}$$

> $A_{1,\,eff} := b\cdot x;$

$$A_{1,eff} := 6710.487012 \quad \textbf{(198)}$$

> $I_{1,\,eff} := \dfrac{b\cdot x\hat{}3}{12}$

$$I_{1,eff} := 69948.49480 \quad \textbf{(199)}$$

> $EI_{eff,\,tot,\,new} := E_1\cdot I_{1,\,eff} + \gamma_1\cdot E_1\cdot A_{1,\,eff}\cdot a_{1,\,eff}\hat{}2 + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_{2,\,new}\hat{}2; \#Nmm\hat{}2$

$$EI_{eff,tot,new} := 4.348484069\ 10^{11} \quad \textbf{(200)}$$

9. New long-term verifications

Including the new modifed parameters into the verification calculations

9.1 Verification of the Maximum load using new parameters

9.1.1 Normal stresses in the **CONCRETE** section

> $\sigma_1 := \dfrac{\left(\gamma_1\cdot E_1\cdot a_{1,\,eff}\cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}}\cdot 10\hat{}6; \#MPa$

$$\sigma_1 := 5.550109638 \quad \textbf{(201)}$$

> $\sigma_{m,\,1} := \dfrac{\left(0.5\cdot E_1\cdot x\cdot M_{Ed,\,new}\right)}{EI_{eff,tot,new}}\cdot 10\hat{}6; \#MPa$

(202)

$$\sigma_{m,1} := 5.550109640 \quad (202)$$

Stresses at the TOP of the concrete section

> $\sigma_{c,\,t} := -\sigma_1 - \sigma_{m,1}$; #MPa

$$\sigma_{c,t} := -11.10021928 \quad (203)$$

Verification of the top section

> $Ver_{top,\,c} := \dfrac{\sigma_{c,t}}{\dfrac{f_{ck,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{top,c} := -0.2442005915 \quad (204)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\,c} := -\sigma_1 + \sigma_{m,1}$; #MPa

$$\sigma_{b,c} := 2.\ 10^{-9} \quad (205)$$

> $Ver_{bottom,\,c} := \dfrac{\sigma_{b,\,c}}{\dfrac{f_{ctk,\,0.05,\,c}}{\gamma_c}}$; #<1.0 #--->OK

$$Ver_{bottom,c} := 9.090909091\ 10^{-10} \quad (206)$$

9.1.2 Normal stresses in the **TIMBER** section

> $\sigma_2 := \dfrac{(\gamma_2 \cdot E_2 \cdot a_{2,\,new} \cdot M_{Ed,\,new})}{EI_{eff,tot,new}} \cdot 10^{\wedge}6$; #MPa

$$\sigma_2 := 0.5172769511 \quad (207)$$

> $\sigma_{m,\,2} := \dfrac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot,new}} \cdot 10^{\wedge}6$; #MPa

$$\sigma_{m,2} := 27.84546796 \quad (208)$$

Stresses at the TOP of the timber section

> $\sigma_{t,\,t} := -\sigma_2 - \sigma_{m,\,2}$; #MPa

$$\sigma_{t,t} := -28.36274491 \quad (209)$$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\,t} := -\sigma_2 + \sigma_{m,\,2}$; #MPa

(210)

$$\sigma_{b,t} := 27.32819101 \quad (210)$$

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

$$Ver_{timber} := -0.565238535 \quad (211)$$

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

$$\tau_2 := 2.263659920 \quad (212)$$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; #<1.0 #-->OK

$$Ver_{shear} := 0.8135027838 \quad (213)$$

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

$$F_1 := 9.727551348 \quad (214)$$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_{2,new} \cdot s}{EI_{eff,tot,new}} \cdot P_{Ed}$; #kN

$$F_2 := 9.727551831 \quad (215)$$

> $f_{tens,\,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \quad (216)$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,\,k}}{\gamma_M}}$; #<1.0 #--> NOT OK

$$Ver_{F1} := 0.2330559176 \quad (217)$$

>

>

Appendix E. Additional maximum deflection for Slab A and Slab B based on SLS verifications

For Slab type A (CTC screws)

E.1 Maximum deflection for Slab A based on SLS verifications using M3

E.2 Maximum deflection for Slab A based on SLS verifications using M4

E.3 Maximum deflection for Slab A based on SLS verifications using B45 strength

E.4 Maximum deflection for Slab A based on SLS verifications using avg. strength

For Slab type B (KOP screws)

E.5 Maximum deflection for Slab B based on SLS verifications using M3

E.6 Maximum deflection for Slab B based on SLS verifications using M4

E.7 Maximum deflection for Slab B based on SLS verifications using B45 strength

E.8 Maximum deflection for Slab B based on SLS verifications using avg. strength

Maximum deflection predictions based on SLS CTC-screws 7-160 mm

M3, choosing the moment M3, top part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000 : \#mm\ span\ length\ between\ the\ supports$

b(c) = b(t) = 600 mm

> $b := 600 : \#mm$

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60 : \#mm$

> $A_c := h_c \cdot b : \#mm\hat{}2$

> $I_c := \frac{(b \cdot h_c\hat{}3)}{12} : \#mm\hat{}4$

> $E_{cm,\ c} := 34000 : \#MPa$

> $f_{ck,\ c} := 35 : \#MPa$

> $f_{ctk,\ 0.05,\ c} := 2.2 : \#MPa$

> $\rho_c := 25.00 : \#\frac{kN}{m\hat{}3}$

> $\gamma_c := 1.5 :$

> $\varphi_c := 2.5 :$

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15$:#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

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\^2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\^2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\^2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\^2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\^2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\^2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0; \# \frac{N}{mm\^2}$

$$f_{t,0,k,t15} := 15.0 \quad (3)$$

> $f_{v,\ k,\ t15} := 4.0; \# \frac{N}{mm\^2}$

$$f_{v,k,t15} := 4.0 \quad (4)$$

> $t_{t15} := 430; \# \frac{kg}{m\^3}$

$$t_{t15} := 430 \quad (5)$$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m\^3}$

$$\rho_{t15} := 4.216853302 \quad (6)$$

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

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5\right)\right)$;# $\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad (7)$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2}$:# $\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$; #mm^2

$$A_1 := 18000 \quad (8)$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad (9)$$

> $I_{t1} := \frac{\left(b \cdot h_1\hat{\ }3\right)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad (10)$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad (11)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (12)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (13)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (14)$$

> $I_{t2} := \frac{(b \cdot h_2\text{^}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (15)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (16)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (17)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (18)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (19)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (20)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (21)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad (22)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad (23)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad (24)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad (25)$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad (26)$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad (27)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (28)$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad (29)$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \quad (30)$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad (31)$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \quad (32)$$

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_5 := 473850000000 \quad (33)$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (34)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \tag{35}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \tag{36}$$

> $(GA)_{eff} := evalf\left((a\hat{}2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^{6} \tag{37}$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
> #CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \tag{38}$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; $\# \dfrac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \quad \mathbf{(39)}$$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t^3}{12}}$; $\# \dfrac{N}{mm^{\wedge}}$

$$E_{CLT} := 8403.440000 \quad \mathbf{(40)}$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\ c}$; $\# \dfrac{N}{mm^2}$

$$E_1 := 34000 \quad \mathbf{(41)}$$

> $E_2 := E_{CLT}$; $\# \dfrac{N}{mm}$

$$E_2 := 8403.440000 \quad \mathbf{(42)}$$

> $h_1 := h_c$; $\#mm$

$$h_1 := 60 \quad \mathbf{(43)}$$

> $h_2 := h_t$; $\#mm$

$$h_2 := 120 \quad \mathbf{(44)}$$

> $A_1 := A_c$;$\#mm^2$

$$A_1 := 36000 \quad \mathbf{(45)}$$

> $A_2 := h_2 \cdot b$; $\#mm^2$

$$A_2 := 72000 \quad \mathbf{(46)}$$

> $I_1 := I_c$; $\#mm^4$

(47)

$$I_1 := 10800000 \quad (47)$$

> $I_2 := \frac{b \cdot h_t^3}{12}$; #mm^4

$$I_2 := 86400000 \quad (48)$$

The slip modulus **Kser**
EC5: 2.2.2(2) Ultimate limit states equation 2.1
The values for the slip modulus (Kser) is found in the pdf about the screw types from the company **Rotho blaas page 227**
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

> $l_{eff,\,ctc} := 110$; #mm

$$l_{eff,ctc} := 110 \quad (49)$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\,ctc}$; # $\frac{N}{mm}$

$$K_{ser} := 23100 \quad (50)$$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen

> $angle := 45$;

$$angle := 45 \quad (51)$$

> $k := \sin(convert(angle\ degrees, radians))$;

$$k := \frac{\sqrt{2}}{2} \quad (52)$$

> $s_{\min,\,1} := evalf(130 \cdot k)$;#mm

$$s_{min,1} := 91.92388153 \quad (53)$$

> $s := 150$; #mm

$$s := 150 \quad (54)$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi^2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L^2}}\right)$;

$$\gamma_1 := 0.04851770613 \quad (55)$$

> $\gamma_2 := 1.0$; #Fully composite

$$\gamma_2 := 1.0 \quad (56)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 8.044012970 \quad (57)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 81.95598703 \quad (58)$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\text{\^{}}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\text{\^{}}2$; #Nmm^2

$$EI_{eff,tot} := 1.531288241\ 10^{12} \quad (59)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\text{\^{}}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{\^{}}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = - \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{\^{}}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{\^{}}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 3.092990044\ 10^7 \quad (60)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = - \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{\^{}}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{\^{}}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}}\right)}, M_{Ed,\,1}\right);$ #Nmm

$$M_2 := 2.538286756\ 10^6 \quad \textbf{(61)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{\ }6 :$ #MPa

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6 :$ #MPa

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\,t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right);$ #Nmm

$$M_3 := 5.433818938\ 10^7 \quad \textbf{(62)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ =\ -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.914881092\ 10^7 \quad \textbf{(63)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

$$> M_{Ed,\, new} := \frac{M_3}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 54.33818938 \quad \textbf{(64)}$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \quad \textbf{(65)}$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \quad \textbf{(66)}$$

$$> P_{Ed} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right); \#kN$$

$$P_{Ed} := 142.4663684 \quad \textbf{(67)}$$

3.4 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}};$$

$$w := 9.857021750 \quad \textbf{(68)}$$

$$> w_{\lim} := evalf\left(\frac{L}{250}\right);$$

$$w_{\lim} := 8. \quad \textbf{(69)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.232127719 \quad (70)$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c}$;

$$E_{1,g} := 9714.285714 \quad (71)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad (72)$$

> $q_k := 0$;

$$q_k := 0 \quad (73)$$

> $g_{1,k} := 0$;

$$g_{1,k} := 0 \quad (74)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$;

$$E_{1,fin} := 9714.285715 \quad (75)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}}$;

$$E_{2,g} := 4542.400000 \quad (76)$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2}$;

$$E_{2,q} := 5897.150877 \quad (77)$$

> $E_{2, fin} := \frac{E_{2, g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2, q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0, k} + g_{1,k}) \cdot \gamma_{G, 1} + q_k \cdot \gamma_{Q, 1}};$

$$E_{2,fin} := 4542.399998 \quad \textbf{(78)}$$

4.1.3 Slip modulus

>

> $K_{ser, g} := \frac{K_{ser}}{1 + k_{def, t}};$

$$K_{ser,g} := 12486.48649 \quad \textbf{(79)}$$

> $K_{ser, q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad \textbf{(80)}$$

> $K_{ser, 2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0, k} + g_{1,k}) \cdot \gamma_{G, 1} + q_k \cdot \gamma_{Q, 1}};$

$$K_{ser,2} := 12486.48649 \quad \textbf{(81)}$$

> $K_{u, fin} := K_{ser, 2};$

$$K_{u,fin} := 12486.48649 \quad \textbf{(82)}$$

5. Long-term verifications

Annex B, EC5

B.2 Effective bending stiffenes

> $\gamma_{1, fin} := evalf\left(\frac{1}{1 + \frac{\pi \hat{} 2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u, fin} \cdot L \hat{} 2}}\right);$

$$\gamma_{1,fin} := 0.08798300593 \quad \textbf{(83)}$$

> $\gamma_{2, fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad \textbf{(84)}$$

> $a_{2,fin} := \frac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)};$ #mm

$$a_{2,fin} := 7.739055957 \quad \textbf{(85)}$$

>

> $a_{1, fin} := \frac{(h_1 + h_2)}{2} - a_{2,fin};$ #mm

$$a_{1,fin} := 82.26094404 \quad \textbf{(86)}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}\hat{}2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}\hat{}2;$ #Nmm^2

$$EI_{eff,tot,fin} := 7.251748382\ 10^{11} \quad \mathbf{(87)}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 :$ #MPa

> $\sigma_{m,\,1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 :$ #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right);$ #Nmm

$$M_1 := 4.677646127\ 10^{7} \quad \mathbf{(88)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right);$ #Nmm

$$M_2 := 4.809994277\ 10^{6} \quad \mathbf{(89)}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,2} := \frac{(0.5 \cdot E_{2,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t, t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$$

$$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_{2,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 4.789067131\ 10^7 \quad \textbf{(90)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

(91)

$$M_4 := 6.874786977\ 10^7 \quad (91)$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\ new} := \frac{M_3}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 47.89067131 \quad (92)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (93)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (94)$$

> $P_{Ed,\ fin} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed,fin} := 125.2729868 \quad (95)$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 18.34449232 \quad (96)$$

>

> $w_{\lim} := evalf\left(\frac{L}{150}\right)$;

$$w_{\lim} := 13.33333333 \quad (97)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}}$; # < 1.0 NOT OK

$$Ver_{deflection} := 1.375836924 \quad (98)$$

>

Maximum deflection predictions based on SLS CTC-screws 7-160 mm

M4, choosing the moment M4, bottom part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

$$L := 2000 \tag{1}$$

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

$$b := 600 \tag{2}$$

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

$$h_c := 60 \tag{3}$$

> $A_c := h_c \cdot b$: *#mm^2*

$$A_c := 36000 \tag{4}$$

> $I_c := \frac{(b \cdot h_c{}^3)}{12}$: *#mm^4*

$$I_c := 10800000 \tag{5}$$

> $E_{cm,\,c} := 34000$: *#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*

$$f_{ctk,0.05,c} := 2.2 \tag{8}$$

> $\rho_c := 25.00 : \# \frac{kN}{m\hat{}3}$

$$\rho_c := 25.00 \quad \textbf{(9)}$$

> $\gamma_c := 1.5 :$

$$\gamma_c := 1.5 \quad \textbf{(10)}$$

> $\varphi_c := 2.5 :$

$$\varphi_c := 2.5 \quad \textbf{(11)}$$

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

$$h_1 := 30 \quad \textbf{(12)}$$

> $h_2 := 20 : \#mm$

$$h_2 := 20 \quad \textbf{(13)}$$

> $h_3 := 20 : \#mm$

$$h_3 := 20 \quad \textbf{(14)}$$

> $h_4 := 20 : \#mm$

$$h_4 := 20 \quad \textbf{(15)}$$

> $h_5 := 30 : \#mm$

$$h_5 := 30 \quad \textbf{(16)}$$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad \textbf{(17)}$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

$$\gamma_M := 1.15 \quad \textbf{(18)}$$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

$$Klima := 1.0 \quad \textbf{(19)}$$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

$$k_{modi,t} := 0.8 \quad \textbf{(20)}$$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

$$k_{def,t} := 0.85 \quad (21)$$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \# \frac{N}{mm\hat{\ }2}$

$$E_{0,mean,t22} := 13000 \quad (22)$$

> $E_{90,\ mean,\ t22} := 430 : \# \frac{N}{mm\hat{\ }2}$

$$E_{90,mean,t22} := 430 \quad (23)$$

> $G_{0,mean,t22} := 810 : \# \frac{N}{mm\hat{\ }2}$

$$G_{0,mean,t22} := 810 \quad (24)$$

> $G_{90,\ mean,\ t22} := 81 : \# \frac{N}{mm\hat{\ }2}$

$$G_{90,mean,t22} := 81 \quad (25)$$

> $G_{R,\ t22} := G_{90,mean,t22} :\# \frac{N}{mm\hat{\ }2}$

$$G_{R,t22} := 81 \quad (26)$$

> $f_{m,\ k,\ t22} := 30.5 : \# \frac{N}{mm\hat{\ }2}$

$$f_{m,k,t22} := 30.5 \quad (27)$$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \# \frac{N}{mm\hat{\ }2}$

$$f_{t,0,k,t22} := 22.0 \quad (28)$$

> $f_{v,\ k,\ t22} := 4.0 : \# \frac{N}{mm\hat{\ }2}$

$$f_{v,k,t22} := 4.0 \quad (29)$$

> $t_{t22} := 470 : \# \frac{kg}{m\hat{\ }3}$

$$t_{t22} := 470 \quad (30)$$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{\ }3}$

$$\rho_{t22} := 4.609118725 \quad (31)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{\ }2}$

(32)

$$E_{0,mean,t15} := 11500 \quad (32)$$

> $E_{90,\,mean,\,t15} := 230 : \# \frac{N}{mm\hat{\ }2}$

$$E_{90,mean,t15} := 230 \quad (33)$$

> $G_{0,\,mean,\,t15} := 720 : \# \frac{N}{mm\hat{\ }2}$

$$G_{0,mean,t15} := 720 \quad (34)$$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm\hat{\ }2}$

$$G_{90,mean,t15} := 72 \quad (35)$$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm\hat{\ }2}$

$$G_{R,t15} := 72 \quad (36)$$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm\hat{\ }2}$

$$f_{m,k,t15} := 22 \quad (37)$$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm\hat{\ }2}$

$$f_{t,0,k,t15} := 15.0 \quad (38)$$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm\hat{\ }2}$

$$f_{v,k,t15} := 4.0 \quad (39)$$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{\ }3}$

$$t_{t15} := 430 \quad (40)$$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{\ }3}$

$$\rho_{t15} := 4.216853302 \quad (41)$$

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

> $g_{0,\,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right);\#\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad \textbf{(42)}$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k}\cdot\gamma_{G,\,2}:\#\frac{kN}{m}$

$$f_{d,SLS} := 1.217734993 \quad \textbf{(43)}$$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1;\#mm\hat{}2$

$$A_1 := 18000 \quad \textbf{(44)}$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \tag{45}$$

> $I_{t1} := \frac{(b \cdot h_1{}^3)}{12}$;#mm^4

$$I_{t1} := 1350000 \tag{46}$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \tag{47}$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \tag{48}$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \tag{49}$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \tag{50}$$

> $I_{t2} := \frac{(b \cdot h_2{}^3)}{12}$;#mm^4

$$I_{t2} := 400000 \tag{51}$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \tag{52}$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \tag{53}$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24

The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \tag{54}$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (55)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (56)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (57)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad (58)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad (59)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad (60)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad (61)$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad (62)$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad (63)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (64)$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad (65)$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \quad (66)$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad (67)$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \quad (68)$$

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_5 := 473850000000 \quad \textbf{(69)}$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 949908000000 \quad \textbf{(70)}$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf((EI)_{sum} + (EAz\hat{}2)_{sum})$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad \textbf{(71)}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad \textbf{(72)}$$

> $(GA)_{eff} := evalf\left((a\hat{}2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad \textbf{(73)}$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \tag{74}$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{}2}}$; $\# \dfrac{N}{mm\hat{}2} \cdot mm\hat{}4$

$$EI_{app} := 7.260572161\ 10^{11} \tag{75}$$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t\hat{}3}{12}}$; $\# \dfrac{N}{mm\hat{}}$

$$E_{CLT} := 8403.440000 \tag{76}$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\ c}$; $\# \dfrac{N}{mm\hat{}2}$

$$E_1 := 34000 \tag{77}$$

> $E_2 := E_{CLT}$; $\# \dfrac{N}{mm}$

$$E_2 := 8403.440000 \tag{78}$$

> $h_1 := h_c$; $\#mm$

$$h_1 := 60 \tag{79}$$

> $h_2 := h_t$; $\#mm$

$$h_2 := 120 \tag{80}$$

> $A_1 := A_c$;$\#mm\hat{}2$

$$A_1 := 36000 \tag{81}$$

> $A_2 := h_2 \cdot b$; $\#mm\hat{}2$

$$A_2 := 72000 \quad \textbf{(82)}$$

> $I_1 := I_c$; #mm^4

$$I_1 := 10800000 \quad \textbf{(83)}$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \quad \textbf{(84)}$$

The slip modulus **Kser**

EC5: 2.2.2(2) Ultimate limit states equation 2.1

The values for the slip modulus (Kser) is found in the pdf about the screw types from the company **Rotho blaas page 227**

There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

> $l_{eff,\, ctc} := 110$; #mm

$$l_{eff,ctc} := 110 \quad \textbf{(85)}$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\, ctc}$; #$\frac{N}{mm}$

$$K_{ser} := 23100 \quad \textbf{(86)}$$

Finding the spacing for the slab

Rotho blaas p. 225: minimum spacing was calculated

s,min= 130*sin(45), and a continuous

> $angle := 45$;

$$angle := 45 \quad \textbf{(87)}$$

> $k := \sin(convert(angle\ degrees, radians))$;

$$k := \frac{\sqrt{2}}{2} \quad \textbf{(88)}$$

> $s_{\min,\, 1} := evalf(130 \cdot k)$;#mm

$$s := 157.50 \quad \textbf{(89)}$$

> $s := 150$; #mm

$$s := 150 \quad \textbf{(90)}$$

Annex B, EC5

B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L\hat{}2}}\right)$;

$$\gamma_1 := 0.04851770613 \quad \textbf{(91)}$$

> $\gamma_2 := 1.0$; #Fully composite

(92)

$$\gamma_2 := 1.0 \quad \textbf{(92)}$$

> $a_2 := \dfrac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 8.044012970 \quad \textbf{(93)}$$

>

> $a_1 := \dfrac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 81.95598703 \quad \textbf{(94)}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 1.531288241\ 10^{12} \quad \textbf{(95)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \dfrac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

$$\sigma_1 := 0.08828825115\ M_{Ed,1} \quad \textbf{(96)}$$

> $\sigma_{m,\ 1} := \dfrac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

$$\sigma_{m,1} := 0.6661058140\ M_{Ed,1} \quad \textbf{(97)}$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \dfrac{f_{ck,\ c}}{\gamma_c \cdot \left(\dfrac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \dfrac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 3.092990044\ 10^7 \quad \textbf{(98)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

$$> \ M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right); \#Nmm$$

$$M_2 := 2.538286756\ 10^6 \tag{99}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \ \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ 2})}{EI_{eff,\ tot}} \cdot 10\hat{}6 : \#MPa$$

$$\sigma_2 := 0.04414412554\ M_{Ed,2} \tag{100}$$

$$> \ \sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\ 2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

$$\sigma_{m,2} := 0.3292694259\ M_{Ed,2} \tag{101}$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\ 0,\ d}} - \frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t} \cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_3 := 5.433818938\ 10^7 \quad (102)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$\#\sigma t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 7.914881092\ 10^7 \quad (103)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{M_4}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 79.14881092 \quad (104)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (105)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (106)$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed} := 208.6280258 \quad (107)$$

3.4 Verification of the vertical defelction

> $w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot}};$

$$w := 14.35770237 \quad \textbf{(108)}$$

> $w_{\lim} := evalf\left(\frac{L}{250}\right);$

$$w_{\lim} := 8. \quad \textbf{(109)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}};$ # <1.0 NOT OK

$$Ver_{deflection} := 1.794712796 \quad \textbf{(110)}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1 + \varphi_c};$

$$E_{1,g} := 9714.285714 \quad \textbf{(111)}$$

> $E_{1,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad \textbf{(112)}$$

> $q_k := 0;$

$$q_k := 0 \quad \textbf{(113)}$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad \textbf{(114)}$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

(115)

$$E_{1,fin} := 9714.285715 \quad (115)$$

4.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1 + k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (116)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \Psi_2};$

$$E_{2,q} := 5897.150877 \quad (117)$$

> $E_{2,fin} := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4542.399998 \quad (118)$$

4.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (119)$$

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \Psi_2};$

$$K_{ser,q} := 16210.52632 \quad (120)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad (121)$$

> $K_{u,fin} := K_{ser,2};$

$$K_{u,fin} := 12486.48649 \quad (122)$$

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1,fin} := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{\ }2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L\hat{\ }2}}\right);$

$$\gamma_{1,fin} := 0.08798300593 \quad (123)$$

> $\gamma_{2,\,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \tag{124}$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}; \#mm$

$$a_{2,fin} := 7.739055957 \tag{125}$$

>

> $a_{1,\,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin}; \#mm$

$$a_{1,fin} := 82.26094404 \tag{126}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}\hat{}2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}\hat{}2;$
> $\#Nmm\hat{}2$

$$EI_{eff,tot,fin} := 7.251748382\ 10^{11} \tag{127}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

$$\sigma_1 := 0.09695286133\ M_{Ed,1} \tag{128}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

$$\sigma_{m,1} := 0.4018735291\ M_{Ed,1} \tag{129}$$

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma\, c,t\ =\ -\ \ \sigma 1 - \sigma\, m,1 = \dfrac{fck}{\gamma\, c}$

$\#M_{Ed,1} \cdot \left(\dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$

$$M_1 := 4.677646127\ 10^7 \tag{130}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

> $$M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 4.809994277\ 10^6 \tag{131}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $$\sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$\sigma_2 := 0.04847643066\ M_{Ed,2} \tag{132}$$

> $$\sigma_{m,\,2} := \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$\sigma_{m,2} := 0.3758321242\ M_{Ed,2} \tag{133}$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma t,t = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2} \left(\frac{\left(\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_3 := 4.789067131\ 10^7 \quad \textbf{(134)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$\#\sigma t,\ b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 6.874786977\ 10^7 \quad \textbf{(135)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\ new} := \frac{M_4}{10\hat{\ }6}$; #kNm

$$M_{Ed,new} := 68.74786977 \quad \textbf{(136)}$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad \textbf{(137)}$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad \textbf{(138)}$$

> $P_{Ed,\ fin} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed,fin} := 180.8921827 \quad \textbf{(139)}$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\,fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot,fin}};$

$$w_{permanent} := 26.33382941 \quad \textbf{(140)}$$

>

> $w_{\lim} := evalf\left(\frac{L}{150}\right);$

$$w_{\lim} := 13.33333333 \quad \textbf{(141)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}};$ # <1.0 NOT OK

$$Ver_{deflection} := 1.975037206 \quad \textbf{(142)}$$

>

Maximum deflection predictions based on SLS CTC-screws 7-160 mm B45

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B45

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#mm^2*

> $I_c := \frac{(b \cdot h_c\hat{\ }3)}{12}$: *#mm^4*

> $E_{cm,\,c} := 36000$: *#MPa*

> $f_{ck,\,c} := 45$: *#MPa*

> $f_{ctk,\,0.05,\,c} := 2.7$: *#MPa*

> $\rho_c := 25.00$: $\# \frac{kN}{m\hat{\ }3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30$: #mm

> $h_2 := 20$: #mm

> $h_3 := 20$: #mm

> $h_4 := 20$: #mm

> $h_5 := 30$: #mm

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5$; #mm

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15$: #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

> $E_{0,\ mean,\ t22} := 13000$: #$\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430$: #$\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810$: #$\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81$: #$\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22}$:#$\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5$:#$\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0$: #$\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0$: #$\frac{N}{mm\hat{}2}$

> $t_{t22} := 470$: #$\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}$; #$\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\,mean,\,t15} := 11500 : \# \frac{N}{mm\hat{}2}$

> $E_{90,\,mean,\,t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\,mean,\,t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

> $g_{0,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right)$;# $\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad (4)$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k}\cdot\gamma_{G,\,2}$: # $\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1$; #mm^2

$$A_1 := 18000 \quad (5)$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad (6)$$

> $I_{t1} := \frac{\left(b\cdot h_1\hat{}3\right)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad (7)$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (11)$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (15)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (18)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \tag{19}$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \tag{20}$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \tag{21}$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \tag{22}$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \tag{23}$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \tag{24}$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \tag{25}$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \tag{26}$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \tag{27}$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \tag{28}$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \tag{29}$$

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_5 := 473850000000 \tag{30}$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 949908000000 \tag{31}$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \tag{32}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \middle/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (35)$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; $\# \frac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \quad (36)$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t\hat{}3}{12}}$; # $\frac{N}{mm\hat{}}$

$$E_{CLT} := 8403.440000 \quad (37)$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}$; # $\frac{N}{mm\hat{}2}$

$$E_1 := 36000 \quad (38)$$

> $E_2 := E_{CLT}$; # $\frac{N}{mm}$

$$E_2 := 8403.440000 \quad (39)$$

> $h_1 := h_c$; #mm

$$h_1 := 60 \quad (40)$$

> $h_2 := h_t$; #mm

$$h_2 := 120 \quad (41)$$

> $A_1 := A_c$;#mm^2

$$A_1 := 36000 \quad (42)$$

> $A_2 := h_2 \cdot b$; #mm^2

$$A_2 := 72000 \quad (43)$$

> $I_1 := I_c$; #mm^4

$$I_1 := 10800000 \quad (44)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**

EC5: 2.2.2(2) Ultimate limit states equation 2.1
The values for the slip modulus (Kser) is found in the pdf about the screw types from the company **Rotho blaas page 227**
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

> $l_{eff,\ ctc} := 110$; #mm

$$l_{eff,ctc} := 110 \quad (46)$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\ ctc}$; #$\frac{N}{mm}$

$$K_{ser} := 23100 \quad (47)$$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen

> $angle := 45$;

$$angle := 45 \quad (48)$$

> $k := \sin(convert(angle\ degrees, radians))$;

$$k := \frac{\sqrt{2}}{2} \quad (49)$$

> $s_{\min,\ 1} := evalf(130 \cdot k)$;#mm

$$s_{min,1} := 91.92388153 \quad (50)$$

> $s := 150$; #mm

$$s := 150 \quad (51)$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{\ }2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L\hat{\ }2}}\right)$;

$$\gamma_1 := 0.04594612250 \quad (52)$$

> $\gamma_2 := 1.0$; #Fully composite

$$\gamma_2 := 1.0 \quad (53)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 8.063805721 \quad (54)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 81.93619428 \quad (55)$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2;$ #Nmm^2

$$EI_{eff,tot} := 1.553966041\ 10^{12} \quad \textbf{(56)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6 :$ #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 :$ #MPa

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma c,t\ = -\ \ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right);$ #Nmm

$$M_1 := 3.835288341\ 10^7 \quad \textbf{(57)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b\ = -\ \sigma 1\ + \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right);$ #Nmm

$$M_2 := 2.961588212\ 10^6 \quad \textbf{(58)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{\left(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,\, t\ =\ -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 5.512165960\ 10^7 \qquad \textbf{(59)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ =\ -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_2 \cdot E_2 \cdot a_2\right)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_2 \cdot h_2\right)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 8.036611981\ 10^7 \qquad \textbf{(60)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 38.35288341 \tag{61}$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \tag{62}$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \tag{63}$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed} := 99.83888577 \tag{64}$$

3.4 Verification of the vertical defelction

> $w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}}$;

$$w := 6.855734579 \tag{65}$$

> $w_{\lim} := evalf\left(\frac{L}{250}\right)$;

$$w_{\lim} := 8. \tag{66}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}}$; # < 1.0 OK

$$Ver_{deflection} := 0.8569668224 \tag{67}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 10285.71429 \quad (68)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 16000.00000 \quad (69)$$

> $q_k := 0;$

$$q_k := 0 \quad (70)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (71)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{1,fin} := 10285.71429 \quad (72)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (73)$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (74)$$

> $E_{2,fin} := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4542.399998 \quad (75)$$

4.1.3 Slip modulus

>

> $K_{ser,g} := \frac{K_{ser}}{1+k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (76)$$

> $K_{ser,q} := \frac{K_{ser}}{1+k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad (77)$$

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad (78)$$

> $K_{u,fin} := K_{ser,2};$

$$K_{u,fin} := 12486.48649 \quad (79)$$

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1,fin} := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{}2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L\hat{}2}}\right);$

$$\gamma_{1,fin} := 0.08350322022 \quad (80)$$

> $\gamma_{2,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad (81)$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}; \#mm$

$$a_{2,fin} := 7.773786378 \quad (82)$$

>

> $a_{1,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin}; \#mm$

$$a_{1,fin} := 82.22621362 \quad (83)$$

> $EI_{eff,tot,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,fin}\hat{}2 + E_{2,fin} \cdot I_2 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2 \cdot a_{2,fin}\hat{}2;$
> $\#Nmm\hat{}2$

$$EI_{eff,tot,fin} := 7.323685481\ 10^{11} \quad (84)$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,1} := \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = - \sigma l - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> \; M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}} + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1\right)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 5.794135599\ 10^7 \tag{85}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = - \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6; + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> \; M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(- \frac{\left(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}} + \frac{\left(0.5 \cdot E_{1,fin} \cdot h_1\right)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 5.540129116\ 10^6 \tag{86}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \; \sigma_2 := \frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6 : \#MPa$$

$$> \; \sigma_{m,\,2} := \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{\ }6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,\, t = - \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6\,;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 4.833284247\ 10^7 \quad \textbf{(87)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 6.949776168\ 10^7 \quad \textbf{(88)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\,new} := \frac{\min\left(M_1, M_3, M_4\right)}{10\hat{}6}; \#kNm$

$$M_{Ed,new} := 48.33284247 \quad \textbf{(89)}$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \quad \textbf{(90)}$$

> $L_{sup} := 2.0; \#m$

$$L_{sup} := 2.0 \quad \textbf{(91)}$$

> $P_{Ed,\ fin} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right);$ #kN

$$P_{Ed,fin} := 126.4521099 \tag{92}$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot,fin}};$

$$w_{permanent} := 18.33201276 \tag{93}$$

>

> $w_{\lim} := evalf\left(\frac{L}{150}\right);$

$$w_{\lim} := 13.33333333 \tag{94}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}};$ # < 1.0 NOT OK

$$Ver_{deflection} := 1.374900957 \tag{95}$$

>

Maximum deflection predictions based on SLS CTC-screws 7-160 mm

Using the average result from compressive test of concrete cubes

> $restart;$

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35*

*strength average from compressive strength test of concrete cubes

*Partial factor is 1.0

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#mm^2*

> $I_c := \frac{(b \cdot h_c\hat{}3)}{12}$: *#mm^4*

> $E_{cm,\, c} := 34000$: *#MPa*

> $f_{ck,\, c} := 45.4553333$: *#MPa*

> $f_{ctk,\, 0.05,\, c} := 2.2$: *#MPa*

> $\rho_c := 25.00$: $\# \frac{kN}{m\hat{}3}$

> $\gamma_c := 1.0$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad \textbf{(1)}$$

> $\gamma_M := 1.15$ #*NA in Eurocode 5 for Glued laminated timber*

$$\gamma_M := 1.15 \quad \textbf{(2)}$$

> $Klima := 1.0;$ #*Serice class, permanent*

$$Klima := 1.0 \quad \textbf{(3)}$$

> $k_{modi,\ t} := 0.8;$ #*modification factor,Swedish CLT handbook*

$$k_{modi,t} := 0.8 \quad \textbf{(4)}$$

> $k_{def,\ t} := 0.85;$ #*modification factor,Swedish CLT handbook*

$$k_{def,t} := 0.85 \quad \textbf{(5)}$$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\,k,\,t22} := 30.5 : \# \frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t22} := 22.0 : \# \frac{N}{mm^2}$

> $f_{v,\,k,\,t22} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t22} := 470 : \# \frac{kg}{m^3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m^3}$

$$\rho_{t22} := 4.609118725 \quad (6)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\,mean,\,t15} := 11500 : \# \frac{N}{mm^2}$

> $E_{90,\,mean,\,t15} := 230 : \# \frac{N}{mm^2}$

> $G_{0,\,mean,\,t15} := 720 : \# \frac{N}{mm^2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm^2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm^2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm^2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm^2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t15} := 430 : \# \frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m^3}$

$$\rho_{t15} := 4.216853302 \quad (7)$$

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

> $g_{0,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right); \# \frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad \textbf{(8)}$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,k} \cdot \gamma_{G,2} : \# \frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1; \#mm\hat{\ }2$

(9)

$$A_1 := 18000 \quad (9)$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad (10)$$

> $I_{t1} := \frac{(b \cdot h_1\hat{}3)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad (11)$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad (12)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (13)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (14)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (15)$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (16)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (17)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (18)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

(19)

$$z_1 := 45 \quad (19)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (20)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (21)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (22)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad (23)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad (24)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad (25)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad (26)$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad (27)$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad (28)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (29)$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad (30)$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \quad (31)$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad (32)$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

(33)

$$(EAz^2)_4 := 1104000000 \quad (33)$$

> $(EAz^2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5^2)$; #Nmm^2

$$(EAz^2)_5 := 473850000000 \quad (34)$$

> $(EAz^2)_{sum} := (EAz^2)_1 + (EAz^2)_2 + (EAz^2)_3 + (EAz^2)_4 + (EAz^2)_5$; #Nmm^2

$$(EAz^2)_{sum} := 949908000000 \quad (35)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf((EI)_{sum} + (EAz^2)_{sum})$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (36)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (37)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (38)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \tag{39}$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; $\# \dfrac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \tag{40}$$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t^3}{12}}$; $\# \dfrac{N}{mm^{\wedge}}$

$$E_{CLT} := 8403.440000 \tag{41}$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}$; $\# \dfrac{N}{mm^2}$

$$E_1 := 34000 \tag{42}$$

> $E_2 := E_{CLT}$; $\# \dfrac{N}{mm}$

$$E_2 := 8403.440000 \tag{43}$$

> $h_1 := h_c$; #mm

$$h_1 := 60 \tag{44}$$

> $h_2 := h_t$; #mm

$$h_2 := 120 \tag{45}$$

> $A_1 := A_c$;#mm^2

$$A_1 := 36000 \tag{46}$$

> $A_2 := h_2 \cdot b$; #mm^2

$$A_2 := 72000 \quad (47)$$

> $I_1 := I_c$; #mm^4

$$I_1 := 10800000 \quad (48)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \quad (49)$$

The slip modulus **Kser**

EC5: 2.2.2(2) Ultimate limit states equation 2.1

The values for the slip modulus (Kser) is found in the pdf about the screw types from the company **Rotho blaas page 227**

There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

> $l_{eff,\ ctc} := 110$; #mm

$$l_{eff,ctc} := 110 \quad (50)$$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\ ctc}$; #$\frac{N}{mm}$

$$K_{ser} := 23100 \quad (51)$$

Finding the spacing for the slab

Rotho blaas p. 225: minimum spacing was calculated

s,min= 130*sin(45), and a continuous

> $angle := 45$;

$$angle := 45 \quad (52)$$

> $k := \sin(convert(angle\ degrees, radians))$;

$$k := \frac{\sqrt{2}}{2} \quad (53)$$

> $s_{\min,\ 1} := evalf(130 \cdot k)$; #mm

$$s_{min,1} := 91.92388153 \quad (54)$$

> $s := 150$; #mm

$$s := 150 \quad (55)$$

Annex B, EC5

B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L\hat{}2}}\right)$;

$$\gamma_1 := 0.04851770613 \quad (56)$$

> $\gamma_2 := 1.0$; #Fully composite

$$\gamma_2 := 1.0 \qquad (57)$$

$$> a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}; \#mm$$

$$a_2 := 8.044012970 \qquad (58)$$

>

$$> a_1 := \frac{(h_1 + h_2)}{2} - a_2; \#mm$$

$$a_1 := 81.95598703 \qquad (59)$$

$$> EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2; \#Nmm\hat{}2$$

$$EI_{eff,tot} := 1.531288241\ 10^{12} \qquad (60)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = - \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right); \#Nmm$$

$$M_1 := 6.025409716\ 10^7 \qquad (61)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = - \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1}\cdot\left(\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6;+\frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6\right)\leq\frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $$M_2 := solve\left(M_{Ed,\ 1}=\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1)}{EI_{eff,\ tot}}+\frac{(0.5\cdot E_1\cdot h_1)}{EI_{eff,\ tot}}\right)}, M_{Ed,\ 1}\right);\#Nmm$$

$$M_2 := 3.807430133\ 10^6 \quad \textbf{(62)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $$\sigma_2 := \frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,\ 2})}{EI_{eff,\ tot}}\cdot 10\hat{}6:\#MPa$$

> $$\sigma_{m,\ 2} := \frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,\ 2})}{EI_{eff,tot}}\cdot 10\hat{}6:\#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t\ =-\frac{\sigma_2}{f_{t,\ 0,\ d}}-\frac{\sigma_{m,\ 2}}{f_{m,\ d}}<1.0$$

$$\#f_{m,\ d}:=\frac{k_{modi,t}\cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d}:=\frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6;\right)}{\frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\leq 1.0$$

> $$M_3 := solve\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right);\#Nmm$$

$$M_3 := 5.433818938\ 10^7 \quad \textbf{(63)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma t, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.914881092\ 10^7 \quad (64)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}; \#kNm$$

$$M_{Ed,new} := 54.33818938 \quad (65)$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \quad (66)$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \quad (67)$$

$$> P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed} := 142.4663684 \quad (68)$$

3.4 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{\ }4}{384 \cdot EI_{eff,tot}};$$

$$w := 9.857021750 \quad (69)$$

$$> w_{\lim} := evalf\left(\frac{L}{250}\right);$$

$$w_{\lim} := 8. \quad (70)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.232127719 \quad \textbf{(71)}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1+\varphi_c}$;

$$E_{1,g} := 9714.285714 \quad \textbf{(72)}$$

> $E_{1,\,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad \textbf{(73)}$$

> $q_k := 0$;

$$q_k := 0 \quad \textbf{(74)}$$

> $g_{1,\,k} := 0$;

$$g_{1,k} := 0 \quad \textbf{(75)}$$

> $E_{1,\,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,\,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}}$;

$$E_{1,fin} := 9714.285715 \quad \textbf{(76)}$$

4.1.2 CLT

> $E_{2,\,g} := \frac{E_{CLT}}{1+k_{def,\,t}}$;

$$E_{2,g} := 4542.400000 \quad \textbf{(77)}$$

> $E_{2,\,q} := \frac{E_{CLT}}{1+k_{def,\,t} \cdot \psi_2}$;

$$E_{2,q} := 5897.150877 \quad \textbf{(78)}$$

> $E_{2,fin} := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4542.399998 \quad (79)$$

4.1.3 Slip modulus

>

> $K_{ser,g} := \frac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 12486.48649 \quad (80)$$

> $K_{ser,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 16210.52632 \quad (81)$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 12486.48649 \quad (82)$$

> $K_{u,fin} := K_{ser,2};$

$$K_{u,fin} := 12486.48649 \quad (83)$$

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1,fin} := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L\hat{}2}}\right);$

$$\gamma_{1,fin} := 0.08798300593 \quad (84)$$

> $\gamma_{2,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad (85)$$

> $a_{2,fin} := \frac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)};$ #mm

$$a_{2,fin} := 7.739055957 \quad (86)$$

>

> $a_{1,fin} := \frac{(h_1 + h_2)}{2} - a_{2,fin};$ #mm

$$a_{1,fin} := 82.26094404 \quad (87)$$

> $EI_{eff,\ tot,\ fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\ fin}\hat{}2 + E_{2,\ fin} \cdot I_2 + \gamma_{2,\ fin} \cdot E_{2,\ fin} \cdot A_2 \cdot a_{2,fin}\hat{}2;$ #Nmm^2

$$EI_{eff,tot,fin} := 7.251748382\ 10^{11} \tag{88}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin} \cdot M_{Ed,\ 1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \frac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\ 1} \right); \#Nmm$

$$M_1 := 9.112455591\ 10^7 \tag{89}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\ 1} \right); \#Nmm$

$$M_2 := 7.214991414\ 10^6 \tag{90}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\; t,\, t\; = -\; \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6\,; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$

$$M_3 := 4.789067131\ 10^7 \quad \textbf{(91)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\; t,\, b\; = -\; \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$

(92)

$$M_4 := 6.874786977\ 10^7 \quad (92)$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}$; #kNm

$$M_{Ed,new} := 47.89067131 \quad (93)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (94)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (95)$$

> $P_{Ed,\ fin} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed,fin} := 125.2729868 \quad (96)$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 18.34449232 \quad (97)$$

>

> $w_{\lim} := evalf\left(\frac{L}{150}\right)$;

$$w_{\lim} := 13.33333333 \quad (98)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.375836924 \quad (99)$$

>

Maximum deflection predictions based on SLS KOP-screws 10-140 mm

M3, choosing the moment M3, top part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\# \frac{kN}{m\hat{}3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\,t} := 0.8 : \#modification\ factor,Swedish\ CLT\ handbook$

> $k_{def,\,t} := 0.8 : \#modification\ factor,Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\,mean,\,t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\,mean,\,t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\,mean,\,t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\,t22} := G_{90,mean,t22} :\#\frac{N}{mm\hat{}2}$

> $f_{m,\,k,\,t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\,0,\,k,\,t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\,k,\,t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 :\#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22}\cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \tag{3}$$

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

> $g_{0,\,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right);\#\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad (4)$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k}\cdot\gamma_{G,\,2} : \#\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1;\#mm\hat{}2$

$$A_1 := 18000 \quad (5)$$

> $A_5 := A_1;\#mm\hat{}2$

$$A_5 := 18000 \quad (6)$$

> $I_{t1} := \frac{\left(b\cdot h_1\hat{}3\right)}{12};\#mm\hat{}4$

$$I_{t1} := 1350000 \quad (7)$$

> $I_{t5} := I_{t1};\#mm\hat{}4$

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (11)$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24

The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (15)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (18)$$

$> z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}; \#mm$

$$z_5 := 45 \quad (19)$$

Ei*Ii

$> (EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}; \#Nmm\hat{}2$

$$(EI)_1 := 17550000000 \quad (20)$$

$> (EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}; \#Nmm\hat{}2$

$$(EI)_2 := 92000000 \quad (21)$$

$> (EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}; \#Nmm\hat{}2$

$$(EI)_3 := 4600000000 \quad (22)$$

$> (EI)_4 := E_{90,mean,t15} \cdot I_{t4}; \#Nmm\hat{}2$

$$(EI)_4 := 92000000 \quad (23)$$

$> (EI)_5 := E_{0,mean,t22} \cdot I_{t5}; \#Nmm\hat{}2$

$$(EI)_5 := 17550000000 \quad (24)$$

$> (EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5; \#Nmm\hat{}2$

$$(EI)_{sum} := 39884000000 \quad (25)$$

Ei*Ao*zi^2

$> (EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2); \#Nmm\hat{}2$

$$(EAz\hat{}2)_1 := 473850000000 \quad (26)$$

$> (EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2); \#Nmm\hat{}2$

$$(EAz\hat{}2)_2 := 1104000000 \quad (27)$$

$> (EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2); \#Nmm\hat{}2$

$$(EAz\hat{}2)_3 := 0 \quad (28)$$

$> (EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2); \#Nmm\hat{}2$

$$(EAz\hat{}2)_4 := 1104000000 \quad (29)$$

$> (EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2); \#Nmm\hat{}2$

$$(EAz\hat{}2)_5 := 473850000000 \quad (30)$$

$> (EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5; \#Nmm\hat{}2$

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (31)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

$> (EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum} \right); \#Nmm\hat{}2$

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (32)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad \textbf{(33)}$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^{6} \quad \textbf{(34)}$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad \textbf{(35)}$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; # $\frac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \quad \textbf{(36)}$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t\hat{}3}{12}}; \# \frac{N}{mm\hat{}}$

$$E_{CLT} := 8403.440000 \quad \textbf{(37)}$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}; \# \frac{N}{mm\hat{}2}$

$$E_1 := 34000 \quad \textbf{(38)}$$

> $E_2 := E_{CLT}; \# \frac{N}{mm}$

$$E_2 := 8403.440000 \quad \textbf{(39)}$$

> $h_1 := h_c; \# mm$

$$h_1 := 60 \quad \textbf{(40)}$$

> $h_2 := h_t; \# mm$

$$h_2 := 120 \quad \textbf{(41)}$$

> $A_1 := A_c;\# mm\hat{}2$

$$A_1 := 36000 \quad \textbf{(42)}$$

> $A_2 := h_2 \cdot b; \# mm\hat{}2$

$$A_2 := 72000 \quad \textbf{(43)}$$

> $I_1 := I_c; \# mm\hat{}4$

$$I_1 := 10800000 \quad \textbf{(44)}$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}; \# mm\hat{}4$

$$I_2 := 86400000 \quad \textbf{(45)}$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22};$

$$\rho_{m,1} := 470 \quad \textbf{(46)}$$

> $\rho_{m,\,2} := t_{t15};$

$$\rho_{m,2} := 430 \quad \textbf{(47)}$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1}\cdot\rho_{m,\,2}\right)\right);$

$$\rho_m := 449.5553359 \quad \textbf{(48)}$$

> $d_{kop} := 10$;#mm

$$d_{kop} := 10 \quad \textbf{(49)}$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23}\cdot\rho_m\hat{}1.5\cdot 3\right);$

$$K_{ser} := 12432.77728 \quad \textbf{(50)}$$

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;$ #mm

$$s := 100 \quad \textbf{(51)}$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1+\frac{\pi\hat{}2\cdot E_1\cdot s\cdot A_1}{K_{ser}\cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.03953909054 \quad \textbf{(52)}$$

> $\gamma_2 := 1.0;$ #Fully composite

$$\gamma_2 := 1.0 \quad \textbf{(53)}$$

> $a_2 := \frac{\gamma_1\cdot E_1\cdot A_1\cdot\left(h_1+h_2\right)}{2\cdot\left(\gamma_1\cdot E_1\cdot A_1+\gamma_2\cdot E_2\cdot A_2\right)};$ #mm

$$a_2 := 6.665650562 \quad \textbf{(54)}$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 83.33434944 \quad (55)$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 1.456230493\ 10^{12} \quad (56)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 3.001576624\ 10^{7} \quad (57)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_2 := 2.352282428\ 10^{6} \quad (58)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ 2})}{EI_{eff,\ tot}} \cdot 10^{\wedge}6 : \#MPa$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\ 2})}{EI_{eff,tot}} \cdot 10^{\wedge}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\ 0,\ d}} - \frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$$

$$\#f_{m,\ d} := \frac{k_{modi,t} \cdot f_{m,\ k,\ t22}}{\gamma_M}$$

$$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^{\wedge}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10^{\wedge}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 5.310086235\ 10^7 \tag{59}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\ t,\ b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 7.243560277\ 10^7 \quad \textbf{(60)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{M_3}{10\hat{}6}; \#kNm$

$$M_{Ed,new} := 53.10086235 \quad \textbf{(61)}$$

> $L_{out} := 0.75; \#m$

$$L_{out} := 0.75 \quad \textbf{(62)}$$

> $L_{sup} := 2.0; \#m$

$$L_{sup} := 2.0 \quad \textbf{(63)}$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$

$$P_{Ed} := 139.1668296 \quad \textbf{(64)}$$

3.4 Verification of the vertical defelction

> $w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}};$

$$w := 10.12905554 \quad \textbf{(65)}$$

> $w_{\lim} := evalf\left(\frac{L}{250}\right);$

$$w_{\lim} := 8. \quad \textbf{(66)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}}; \# < 1.0\ OK$

$$Ver_{deflection} := 1.266131942 \quad \textbf{(67)}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \dfrac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 9714.285714 \qquad (68)$$

> $E_{1,q} := \dfrac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \qquad (69)$$

> $q_k := 0;$

$$q_k := 0 \qquad (70)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \qquad (71)$$

> $E_{1,fin} := \dfrac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{1,fin} := 9714.285715 \qquad (72)$$

4.1.2 CLT

> $E_{2,g} := \dfrac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4668.577778 \qquad (73)$$

> $E_{2,q} := \dfrac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 6002.457143 \qquad (74)$$

> $E_{2,fin} := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4668.577777 \qquad (75)$$

4.1.3 Slip modulus

>

> $K_{ser,\,g} := \dfrac{K_{ser}}{1 + k_{def,\,t}}$;

$$K_{ser,g} := 6907.098489 \quad (76)$$

> $K_{ser,\,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2}$;

$$K_{ser,q} := 8880.555200 \quad (77)$$

> $K_{ser,\,2} := \dfrac{K_{ser,g} \cdot \left(g_{0,k} + g_{1,k}\right) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{\left(g_{0,\,k} + g_{1,k}\right) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}}$;

$$K_{ser,2} := 6907.098490 \quad (78)$$

> $K_{u,\,fin} := K_{ser,\,2}$

$$K_{u,fin} := 6907.098490 \quad (79)$$

5. Long-term verifications

Annex B, EC5

B.2 Effective bending stiffenes

> $\gamma_{1,\,fin} := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{}2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,\,fin} \cdot L\hat{}2}}\right)$;

$$\gamma_{1,fin} := 0.07411396611 \quad (80)$$

> $\gamma_{2,\,fin} := 1.0$;

$$\gamma_{2,fin} := 1.0 \quad (81)$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot \left(h_1 + h_2\right)}{2 \cdot \left(\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2\right)}$; #mm

$$a_{2,fin} := 6.442876599 \quad (82)$$

>

> $a_{1,\,fin} := \dfrac{\left(h_1 + h_2\right)}{2} - a_{2,fin}$; #mm

$$a_{1,fin} := 83.55712340 \quad (83)$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}\hat{}2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}\hat{}2$;
#Nmm^2

$$EI_{eff,tot,fin} := 7.031917825\ 10^{11} \quad (84)$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = - \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,1} = \frac{f_{ck,c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_1 := 4.666788497\ 10^7 \tag{85}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = - \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,0.005,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,1} = \frac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,1} \right); \#Nmm$$

$$M_2 := 4.459489032\ 10^6 \tag{86}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,\, t\; = -\; \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6\,; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \le 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_3 := 4.636168643\ 10^7 \qquad \textbf{(87)}$$

\>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\; = -\; \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$$

$$M_4 := 6.257974216\ 10^7 \qquad \textbf{(88)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,new} := \frac{M_3}{10^6}$; #kNm

$$M_{Ed,new} := 46.36168643 \quad (89)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (90)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (91)$$

> $P_{Ed,\,fin} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}^2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right)$; #kN

$$P_{Ed,fin} := 121.1956938 \quad (92)$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\,fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 18.31398852 \quad (93)$$

>

> $w_{\lim} := evalf\left(\frac{L}{150}\right)$;

$$w_{\lim} := 13.33333333 \quad (94)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.373549139 \quad (95)$$

>

Maximum deflection predictions based on SLS KOP-screws 10-140 mm

M4, choosing the moment M4, bottom part of timber as the maximum value.

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

$$L := 2000 \quad (1)$$

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

$$b := 600 \quad (2)$$

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

$$h_c := 60 \quad (3)$$

> $A_c := h_c \cdot b$: *#mm^2*

$$A_c := 36000 \quad (4)$$

> $I_c := \frac{(b \cdot h_c\text{^}3)}{12}$: *#mm^4*

$$I_c := 10800000 \quad (5)$$

> $E_{cm,\,c} := 34000$: *#MPa*

$$E_{cm,c} := 34000 \quad (6)$$

> $f_{ck,\,c} := 35$: *#MPa*

$$f_{ck,c} := 35 \quad (7)$$

> $f_{ctk,\,0.05,\,c} := 2.2$: *#MPa*

$$f_{ctk,0.05,c} := 2.2 \quad (8)$$

> $\rho_c := 25.00 : \# \frac{kN}{m\hat{}3}$

$$\rho_c := 25.00 \quad \textbf{(9)}$$

> $\gamma_c := 1.5 :$

$$\gamma_c := 1.5 \quad \textbf{(10)}$$

> $\varphi_c := 2.5 :$

$$\varphi_c := 2.5 \quad \textbf{(11)}$$

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

$$h_1 := 30 \quad \textbf{(12)}$$

> $h_2 := 20 : \#mm$

$$h_2 := 20 \quad \textbf{(13)}$$

> $h_3 := 20 : \#mm$

$$h_3 := 20 \quad \textbf{(14)}$$

> $h_4 := 20 : \#mm$

$$h_4 := 20 \quad \textbf{(15)}$$

> $h_5 := 30 : \#mm$

$$h_5 := 30 \quad \textbf{(16)}$$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad \textbf{(17)}$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

$$\gamma_M := 1.15 \quad \textbf{(18)}$$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

$$Klima := 1.0 \quad \textbf{(19)}$$

> $k_{modi,\ t} := 0.8 : \#modification\ factor,Swedish\ CLT\ handbook$

$$k_{modi,t} := 0.8 \quad \textbf{(20)}$$

> $k_{def,\ t} := 0.85 : \#modification\ factor,Swedish\ CLT\ handbook$

$$k_{def,t} := 0.85 \quad (21)$$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \# \frac{N}{mm^2}$

$$E_{0,mean,t22} := 13000 \quad (22)$$

> $E_{90,\ mean,\ t22} := 430 : \# \frac{N}{mm^2}$

$$E_{90,mean,t22} := 430 \quad (23)$$

> $G_{0,mean,t22} := 810 : \# \frac{N}{mm^2}$

$$G_{0,mean,t22} := 810 \quad (24)$$

> $G_{90,\ mean,\ t22} := 81 : \# \frac{N}{mm^2}$

$$G_{90,mean,t22} := 81 \quad (25)$$

> $G_{R,\ t22} := G_{90,mean,t22} : \# \frac{N}{mm^2}$

$$G_{R,t22} := 81 \quad (26)$$

> $f_{m,\ k,\ t22} := 30.5 : \# \frac{N}{mm^2}$

$$f_{m,k,t22} := 30.5 \quad (27)$$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \# \frac{N}{mm^2}$

$$f_{t,0,k,t22} := 22.0 \quad (28)$$

> $f_{v,\ k,\ t22} := 4.0 : \# \frac{N}{mm^2}$

$$f_{v,k,t22} := 4.0 \quad (29)$$

> $t_{t22} := 470 : \# \frac{kg}{m^3}$

$$t_{t22} := 470 \quad (30)$$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m^3}$

$$\rho_{t22} := 4.609118725 \quad (31)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm^2}$

(32)

$$E_{0,mean,t15} := 11500$$ **(32)**

> $E_{90,\,mean,\,t15} := 230 : \# \frac{N}{mm^{\wedge}2}$

$$E_{90,mean,t15} := 230$$ **(33)**

> $G_{0,\,mean,\,t15} := 720 : \# \frac{N}{mm^{\wedge}2}$

$$G_{0,mean,t15} := 720$$ **(34)**

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm^{\wedge}2}$

$$G_{90,mean,t15} := 72$$ **(35)**

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm^{\wedge}2}$

$$G_{R,t15} := 72$$ **(36)**

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm^{\wedge}2}$

$$f_{m,k,t15} := 22$$ **(37)**

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm^{\wedge}2}$

$$f_{t,0,k,t15} := 15.0$$ **(38)**

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm^{\wedge}2}$

$$f_{v,k,t15} := 4.0$$ **(39)**

> $t_{t15} := 430 : \# \frac{kg}{m^{\wedge}3}$

$$t_{t15} := 430$$ **(40)**

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m^{\wedge}3}$

$$\rho_{t15} := 4.216853302$$ **(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

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right) ;\# \frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad \textbf{(42)}$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2} : \# \frac{kN}{m}$

$$f_{d,SLS} := 1.217734993 \quad \textbf{(43)}$$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1 ; \# mm\hat{\ }2$

$$A_1 := 18000 \quad \textbf{(44)}$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \tag{45}$$

> $I_{t1} := \frac{(b \cdot h_1\hat{}3)}{12}$;#mm^4

$$I_{t1} := 1350000 \tag{46}$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \tag{47}$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \tag{48}$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \tag{49}$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \tag{50}$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \tag{51}$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \tag{52}$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \tag{53}$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \tag{54}$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \tag{55}$$

> $z_3 := 0$; #mm

$$z_3 := 0 \tag{56}$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \tag{57}$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \tag{58}$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \tag{59}$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \tag{60}$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \tag{61}$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \tag{62}$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \tag{63}$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \tag{64}$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \tag{65}$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \tag{66}$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \tag{67}$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \tag{68}$$

> $(EAz^2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5^2)$; #Nmm^2

$$(EAz^2)_5 := 473850000000 \quad (69)$$

> $(EAz^2)_{sum} := (EAz^2)_1 + (EAz^2)_2 + (EAz^2)_3 + (EAz^2)_4 + (EAz^2)_5$; #Nmm^2

$$(EAz^2)_{sum} := 949908000000 \quad (70)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf((EI)_{sum} + (EAz^2)_{sum})$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (71)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (72)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (73)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (74)$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{\ }2}}; \# \dfrac{N}{mm\hat{\ }2} \cdot mm\hat{\ }4$

$$EI_{app} := 7.260572161\ 10^{11} \quad (75)$$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t\hat{\ }3}{12}}; \# \dfrac{N}{mm\hat{\ }}$

$$E_{CLT} := 8403.440000 \quad (76)$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\ c}; \# \dfrac{N}{mm\hat{\ }2}$

$$E_1 := 34000 \quad (77)$$

> $E_2 := E_{CLT}; \# \dfrac{N}{mm}$

$$E_2 := 8403.440000 \quad (78)$$

> $h_1 := h_c; \# mm$

$$h_1 := 60 \quad (79)$$

> $h_2 := h_t; \# mm$

$$h_2 := 120 \quad (80)$$

> $A_1 := A_c; \# mm\hat{\ }2$

$$A_1 := 36000 \quad (81)$$

> $A_2 := h_2 \cdot b; \# mm\hat{\ }2$

$$A_2 := 72000 \tag{82}$$

> $I_1 := I_c$; #mm^4

$$I_1 := 10800000 \tag{83}$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; #mm^4

$$I_2 := 86400000 \tag{84}$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22}$;

$$\rho_{m,1} := 470 \tag{85}$$

> $\rho_{m,\,2} := t_{t15}$;

$$\rho_{m,2} := 430 \tag{86}$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right)$;

$$\rho_m := 449.5553359 \tag{87}$$

> $d_{kop} := 10$;#mm

$$d_{kop} := 10 \tag{88}$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m\hat{}1.5 \cdot 3\right)$;

$$K_{ser} := 12432.77728 \tag{89}$$

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100$; #mm

$$s := 100 \tag{90}$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L\hat{}2}}\right)$;

$$\gamma_1 := 0.03953909054 \tag{91}$$

> $\gamma_2 := 1.0;$ #*Fully composite*

$$\gamma_2 := 1.0 \qquad \textbf{(92)}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)};$ #*mm*

$$a_2 := 6.665650562 \qquad \textbf{(93)}$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2;$ #*mm*

$$a_1 := 83.33434944 \qquad \textbf{(94)}$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2;$ #*Nmm^2*

$$EI_{eff,tot} := 1.456230493\ 10^{12} \qquad \textbf{(95)}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6 :$ #*MPa*

$$\sigma_1 := 0.07693067110\ M_{Ed,1} \qquad \textbf{(96)}$$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 :$ #*MPa*

$$\sigma_{m,1} := 0.7004385670\ M_{Ed,1} \qquad \textbf{(97)}$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right);$ #*Nmm*

$$M_1 := 3.001576624\ 10^7 \qquad \textbf{(98)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b = - \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$

$> M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(- \frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right); \#Nmm$

$$M_2 := 2.352282428\ 10^6 \quad (99)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ 2})}{EI_{eff,\ tot}} \cdot 10\hat{}6 : \#MPa$

$$\sigma_2 := 0.03846533556\ M_{Ed,2} \quad (100)$$

$> \sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\ 2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

$$\sigma_{m,2} := 0.3462407926\ M_{Ed,2} \quad (101)$$

Stresses at the **TOP** of the **TIMBER** section

$\#\sigma\ t,t = - \frac{\sigma_2}{f_{t,\ 0,\ d}} - \frac{\sigma_{m,\ 2}}{f_{m,\ d}} < 1.0$

$\#f_{m,\ d} := \frac{k_{modi,t} \cdot f_{m,\ k,\ t22}}{\gamma_M}$

$\#f_{t,\ d} := \frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}$

$\#M_{Ed,2} \left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right);$ #Nmm

$$M_3 := 5.310086235\ 10^7 \tag{102}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$\#\sigma t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right);$ #Nmm

$$M_4 := 7.243560277\ 10^7 \tag{103}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{M_4}{10\hat{}6};$ #kNm

$$M_{Ed,new} := 72.43560277 \tag{104}$$

> $L_{out} := 0.75;$ #m

$$L_{out} := 0.75 \tag{105}$$

> $L_{sup} := 2.0;$ #m

$$L_{sup} := 2.0 \tag{106}$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right);$ #kN

$$P_{Ed} := 190.7261374 \tag{107}$$

3.4 Verification of the vertical defelction

> $w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS} \right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}};$

$$w := 13.81718133 \quad (108)$$

> $w_{\mathrm{lim}} := evalf\left(\frac{L}{250} \right);$

$$w_{\mathrm{lim}} := 8. \quad (109)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\mathrm{lim}}};$ # <1.0 NOT OK

$$Ver_{deflection} := 1.727147666 \quad (110)$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c};$

$$E_{1,g} := 9714.285714 \quad (111)$$

> $E_{1,\,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2};$

$$E_{1,q} := 15111.11111 \quad (112)$$

> $q_k := 0;$

$$q_k := 0 \quad (113)$$

> $g_{1,\,k} := 0;$

$$g_{1,k} := 0 \quad (114)$$

> $E_{1,\,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,\,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}};$

(115)

$$E_{1,fin} := 9714.285715 \quad (115)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1 + k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (116)$$

> $E_{2,q} := \frac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (117)$$

> $E_{2,fin} := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4542.399998 \quad (118)$$

4.1.3 Slip modulus

>

> $K_{ser,g} := \frac{K_{ser}}{1 + k_{def,t}};$

$$K_{ser,g} := 6720.420151 \quad (119)$$

> $K_{ser,q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 8724.755986 \quad (120)$$

> $K_{ser,2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$K_{ser,2} := 6720.420149 \quad (121)$$

> $K_{u,fin} := K_{ser,2};$

$$K_{u,fin} := 6720.420149 \quad (122)$$

5. Long-term verifications

B.2 Effective bending stiffenes

> $\gamma_{1,fin} := evalf\left(\frac{1}{1 + \frac{\pi^2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L^2}}\right);$

$$\gamma_{1,fin} := 0.07225561973 \quad (123)$$

> $\gamma_{2,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad (124)$$

> $a_{2,\,fin} := \dfrac{\gamma_{1,\,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,\,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,\,fin} \cdot A_2)}$; #mm

$$a_{2,fin} := 6.454880643 \quad \textbf{(125)}$$

>

> $a_{1,\,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin}$; #mm

$$a_{1,fin} := 83.54511936 \quad \textbf{(126)}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}\hat{}2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}\hat{}2$;
#Nmm^2

$$EI_{eff,tot,fin} := 6.873754563\ 10^{11} \quad \textbf{(127)}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6$: #MPa

$$\sigma_1 := 0.08531189048\ M_{Ed,1} \quad \textbf{(128)}$$

> $\sigma_{m,\,1} := \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6$: #MPa

$$\sigma_{m,1} := 0.4239729085\ M_{Ed,1} \quad \textbf{(129)}$$

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \dfrac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \dfrac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\,1} = \dfrac{f_{ck,\,c}}{\gamma_c \cdot \left(\dfrac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \dfrac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right)$; #Nmm

$$M_1 := 4.581588411\ 10^7 \quad \textbf{(130)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \dfrac{fck}{\gamma c}$

$$\#M_{Ed,1}\cdot\left(\frac{\left(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,fin}\cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}}\cdot 10^{\wedge}6;+\frac{\left(0.5\cdot E_{1,fin}\cdot h_{1}\cdot M_{Ed,1}\right)}{EI_{eff,tot,fin}}\cdot 10^{\wedge}6\right)\leq\frac{f_{ctk,\,0.005,\,c}}{\gamma_{c}}$$

$$> M_2 := solve\left(M_{Ed,\,1}=\frac{f_{ctk,\,0.05,\,c}}{\gamma_c\cdot\left(-\frac{\left(\gamma_{1,fin}\cdot E_{1,fin}\cdot a_{1,\,fin}\right)}{EI_{eff,tot,fin}}+\frac{\left(0.5\cdot E_{1,fin}\cdot h_1\right)}{EI_{eff,tot,fin}}\right)},M_{Ed,\,1}\right);\#Nmm$$

$$M_2 := 4.330780896\ 10^6 \quad \textbf{(131)}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{\left(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}}\cdot 10^{\wedge}6:\#MPa$$

$$\sigma_2 := 0.04265594524\ M_{Ed,2} \quad \textbf{(132)}$$

$$> \sigma_{m,\,2} := \frac{\left(0.5\cdot E_{2,\,fin}\cdot h_2\cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}}\cdot 10^{\wedge}6:\#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,t = -\frac{\sigma_2}{f_{t,\,0,\,d}}-\frac{\sigma_{m,\,2}}{f_{m,\,d}}<1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t}\cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}}\cdot 10^{\wedge}6;\right)}{\frac{k_{modi,t}\cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}}+\frac{\left(\frac{\left(0.5\cdot E_{2,\,fin}\cdot h_2\cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}}\cdot 10^{\wedge}6;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\leq 1.0$$

$$> M_3 := solve\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_{2,fin}\cdot E_{2,\,fin}\cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin}\cdot f_{t,0,k,t22}}+\frac{\left(0.5\cdot E_{2,\,fin}\cdot h_2\right)}{EI_{eff,tot,fin}\cdot f_{m,k,t22}}\right)}\right),M_{Ed,2}\right);\#Nmm$$

$$M_3 := 4.656652667\ 10^7 \quad \textbf{(133)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma t,\ b\ = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin}\cdot E_{2,\ fin}\cdot a_{2,\ fin})}{EI_{eff,tot,fin}\cdot f_{t,0,k,t22}} + \frac{(0.5\cdot E_{2,\ fin}\cdot h_2)}{EI_{eff,tot,fin}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 6.2891909\ 10^7 \quad (134)$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

$$> M_{Ed,new} := \frac{M_4}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 62.89190900 \quad (135)$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \quad (136)$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \quad (137)$$

$$> P_{Ed,\ fin} := solve\left(\frac{P_{Ed,\ 1}\cdot L_{out}}{2} + \frac{1.5\cdot g_{0,k}\cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed,fin} := 165.2762873 \quad (138)$$

5.4 Verification of the vertical deflection

Where creep is included.

$$> w_{permanent} := \frac{5\cdot\left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right)\cdot L\hat{}4}{384\cdot EI_{eff,tot,fin}};$$

$$w_{permanent} := 25.41547643 \quad (139)$$

>

$$> w_{\lim} := evalf\left(\frac{L}{150}\right);$$

$$w_{\lim} := 13.33333333 \quad (140)$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.906160733 \quad \textbf{(141)}$$

>

Maximum deflection predictions based on SLS KOP-screws 10-140 mm B45

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B45

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#mm^2*

> $I_c := \frac{(b \cdot h_c\hat{}3)}{12}$: *#mm^4*

> $E_{cm,\ c} := 36000$: *#MPa*

> $f_{ck,\ c} := 45$: *#MPa*

> $f_{ctk,\ 0.05,\ c} := 2.7$: *#MPa*

> $\rho_c := 25.00$: $\#\frac{kN}{m\hat{}3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm^2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm^2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm^2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm^2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm^2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm^2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm^2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm^2}$

> $t_{t15} := 430 : \# \frac{kg}{m^3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m^3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

> $g_{0,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right)$;# $\frac{kN}{m}$

$$g_{0,k} := 1.217734993 \quad (4)$$

1. 1 SLS

> $f_{d,SLS} := g_{0,k}\cdot\gamma_{G,2}$: # $\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1$; #mm^2

$$A_1 := 18000 \quad (5)$$

> $A_5 := A_1$; #mm^2

$$A_5 := 18000 \quad (6)$$

> $I_{t1} := \frac{\left(b\cdot h_1{}^{\wedge}3\right)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad (7)$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (11)$$

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24

The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (15)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (16)$$

> $z_3 := 0$; #mm

$$z_3 := 0 \quad (17)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2}$; #mm

$$z_4 := 20 \quad (18)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2}$; #mm

$$z_5 := 45 \quad \textbf{(19)}$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1}$; #Nmm^2

$$(EI)_1 := 17550000000 \quad \textbf{(20)}$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2}$; #Nmm^2

$$(EI)_2 := 92000000 \quad \textbf{(21)}$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3}$; #Nmm^2

$$(EI)_3 := 4600000000 \quad \textbf{(22)}$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4}$; #Nmm^2

$$(EI)_4 := 92000000 \quad \textbf{(23)}$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5}$; #Nmm^2

$$(EI)_5 := 17550000000 \quad \textbf{(24)}$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5$; #Nmm^2

$$(EI)_{sum} := 39884000000 \quad \textbf{(25)}$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_1 := 473850000000 \quad \textbf{(26)}$$

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_2 := 1104000000 \quad \textbf{(27)}$$

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_3 := 0 \quad \textbf{(28)}$$

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_4 := 1104000000 \quad \textbf{(29)}$$

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2)$; #Nmm^2

$$(EAz\hat{}2)_5 := 473850000000 \quad \textbf{(30)}$$

> $(EAz\hat{}2)_{sum} := (EAz\hat{}2)_1 + (EAz\hat{}2)_2 + (EAz\hat{}2)_3 + (EAz\hat{}2)_4 + (EAz\hat{}2)_5$; #Nmm^2

$$(EAz\hat{}2)_{sum} := 949908000000 \quad \textbf{(31)}$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad \textbf{(32)}$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a^2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^6 \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
> #CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

$$K_s := 11.5 \quad (35)$$

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; # $\frac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \quad (36)$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t\hat{}3}{12}}; \# \frac{N}{mm\hat{}}$

$$E_{CLT} := 8403.440000 \qquad (37)$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\ c}; \# \frac{N}{mm\hat{}2}$

$$E_1 := 36000 \qquad (38)$$

> $E_2 := E_{CLT}; \# \frac{N}{mm}$

$$E_2 := 8403.440000 \qquad (39)$$

> $h_1 := h_c; \# mm$

$$h_1 := 60 \qquad (40)$$

> $h_2 := h_t; \# mm$

$$h_2 := 120 \qquad (41)$$

> $A_1 := A_c; \# mm\hat{}2$

$$A_1 := 36000 \qquad (42)$$

> $A_2 := h_2 \cdot b; \# mm\hat{}2$

$$A_2 := 72000 \qquad (43)$$

> $I_1 := I_c; \# mm\hat{}4$

$$I_1 := 10800000 \qquad (44)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}; \# mm\hat{}4$

$$I_2 := 86400000 \qquad (45)$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22};$

$$\rho_{m,1} := 470 \quad (46)$$

> $\rho_{m,\,2} := t_{t15};$

$$\rho_{m,2} := 430 \quad (47)$$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right);$

$$\rho_m := 449.5553359 \quad (48)$$

> $d_{kop} := 10$;#mm

$$d_{kop} := 10 \quad (49)$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m\hat{}1.5 \cdot 3\right);$

$$K_{ser} := 12432.77728 \quad (50)$$

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;$ #mm

$$s := 100 \quad (51)$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.03742468206 \quad (52)$$

> $\gamma_2 := 1.0;$ #Fully composite

$$\gamma_2 := 1.0 \quad (53)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot \left(h_1 + h_2\right)}{2 \cdot \left(\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2\right)};$ #mm

$$a_2 := 6.679235652 \quad (54)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 83.32076435 \quad (55)$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\text{^}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\text{^}2$; #Nmm^2

$$EI_{eff,tot} := 1.478570259\ 10^{12} \quad (56)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\text{^}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{^}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{^}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{^}6 \right) \le \frac{f_{c,k}}{\gamma_c}$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 3.720431383\ 10^7 \quad (57)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$

$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{^}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\text{^}6 \right) \le \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_2 := 2.750137975\ 10^6 \quad (58)$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{\ }6 : \#MPa$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\, t,\, t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6; \right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{\ }6\ ; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$

$$M_3 := 5.390081175\ 10^7 \qquad \textbf{(59)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 7.357412351\ 10^7 \quad \textbf{(60)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}$; #kNm

$$M_{Ed,new} := 37.20431383 \quad \textbf{(61)}$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad \textbf{(62)}$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad \textbf{(63)}$$

> $P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed} := 96.77603356 \quad \textbf{(64)}$$

3.4 Verification of the vertical defelction

> $w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{\ }4}{384 \cdot EI_{eff,tot}}$;

$$w := 6.989543821 \quad \textbf{(65)}$$

> $w_{\lim} := evalf\left(\frac{L}{250}\right)$;

$$w_{\lim} := 8. \quad \textbf{(66)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}}$; #<1.0 OK

$$Ver_{deflection} := 0.8736929776 \quad \textbf{(67)}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1+\varphi_c};$

$$E_{1,g} := 10285.71429 \quad (68)$$

> $E_{1,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2};$

$$E_{1,q} := 16000.00000 \quad (69)$$

> $q_k := 0;$

$$q_k := 0 \quad (70)$$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad (71)$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{1,fin} := 10285.71429 \quad (72)$$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1+k_{def,t}};$

$$E_{2,g} := 4542.400000 \quad (73)$$

> $E_{2,q} := \frac{E_{CLT}}{1+k_{def,t} \cdot \psi_2};$

$$E_{2,q} := 5897.150877 \quad (74)$$

> $E_{2,fin} := \frac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}};$

$$E_{2,fin} := 4542.399998 \quad (75)$$

4.1.3 Slip modulus

>

> $K_{ser,\,g} := \dfrac{K_{ser}}{1 + k_{def,\,t}};$

$$K_{ser,g} := 6720.420151 \quad \textbf{(76)}$$

> $K_{ser,\,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 8724.755986 \quad \textbf{(77)}$$

> $K_{ser,\,2} := \dfrac{K_{ser,g} \cdot \left(g_{0,k} + g_{1,k}\right) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{\left(g_{0,\,k} + g_{1,k}\right) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}};$

$$K_{ser,2} := 6720.420149 \quad \textbf{(78)}$$

> $K_{u,\,fin} := K_{ser,\,2};$

$$K_{u,fin} := 6720.420149 \quad \textbf{(79)}$$

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1,\,fin} := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{\ }2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,\,fin} \cdot L\hat{\ }2}}\right);$

$$\gamma_{1,fin} := 0.06851645740 \quad \textbf{(80)}$$

> $\gamma_{2,\,fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad \textbf{(81)}$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot \left(h_1 + h_2\right)}{2 \cdot \left(\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2\right)};$ #mm

$$a_{2,fin} := 6.479023419 \quad \textbf{(82)}$$

>

> $a_{1,\,fin} := \dfrac{\left(h_1 + h_2\right)}{2} - a_{2,fin};$ #mm

$$a_{1,fin} := 83.52097658 \quad \textbf{(83)}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}\hat{\ }2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}\hat{\ }2;$
#Nmm^2

$$EI_{eff,tot,fin} := 6.942575216\ 10^{11} \quad \textbf{(84)}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma\, c,t \;=\; -\;\; \sigma 1 - \sigma\, m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 5.668456262\ 10^7 \tag{85}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma\, c,b \;=\; -\;\; \sigma\, 1 \;+\; \sigma\, m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \le \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(- \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 5.004443360\ 10^6 \tag{86}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 4.700993431\ 10^7 \quad \textbf{(87)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\ t,\ b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,\,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 6.356326244\ 10^7 \quad \textbf{(88)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\ new} := \dfrac{\min\left(M_1, M_3, M_4\right)}{10^\wedge 6}$; #kNm

$$M_{Ed,new} := 47.00993431 \quad (89)$$

> $L_{out} := 0.75$; #m

$$L_{out} := 0.75 \quad (90)$$

> $L_{sup} := 2.0$; #m

$$L_{sup} := 2.0 \quad (91)$$

> $P_{Ed,\ fin} := solve\left(\dfrac{P_{Ed,\ 1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}{}^\wedge 2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed,fin} := 122.9243548 \quad (92)$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \dfrac{5 \cdot \left(\dfrac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L^\wedge 4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 18.80903652 \quad (93)$$

>

> $w_{\lim} := evalf\left(\dfrac{L}{150}\right)$;

$$w_{\lim} := 13.33333333 \quad (94)$$

Verification of the vertical deflection

> $Ver_{deflection} := \dfrac{w_{permanent}}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.410677739 \quad (95)$$

>

Maximum deflection predictions based on SLS KOP-screws 10-140 mm

Using the average result from compressive test of concrete cubes

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35*

*strength average from compressive strength test of concrete cubes

*Partial factor is 1.0

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#mm^2*

> $I_c := \frac{(b \cdot h_c\hat{}3)}{12}$: *#mm^4*

> $E_{cm,\,c} := 34000$: *#MPa*

> $f_{ck,\,c} := 45.4553333$: *#MPa*

> $f_{ctk,\,0.05,\,c} := 2.2$: *#MPa*

> $\rho_c := 25.00$: $\# \frac{kN}{m\hat{}3}$

> $\gamma_c := 1.0$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \qquad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1}; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \qquad (3)$$

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

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5\right)\right); \# \frac{kN}{m}$

$$g_{0,k} := 1.217734993 \qquad (4)$$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2} : \# \frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1; \# mm\hat{}2$

$$A_1 := 18000 \qquad (5)$$

> $A_5 := A_1; \# mm\hat{}2$

$$A_5 := 18000 \qquad (6)$$

> $I_{t1} := \frac{(b \cdot h_1 \hat{} 3)}{12}$;#mm^4

$$I_{t1} := 1350000 \quad (7)$$

> $I_{t5} := I_{t1}$; #mm^4

$$I_{t5} := 1350000 \quad (8)$$

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$; #mm^2

$$A_2 := 12000 \quad (9)$$

> $A_3 := A_2$; #mm^2

$$A_3 := 12000 \quad (10)$$

> $A_4 := A_2$; #mm^2

$$A_4 := 12000 \quad (11)$$

> $I_{t2} := \frac{(b \cdot h_2 \hat{} 3)}{12}$;#mm^4

$$I_{t2} := 400000 \quad (12)$$

> $I_{t3} := I_{t2}$; #mm^4

$$I_{t3} := 400000 \quad (13)$$

> $I_{t4} := I_{t2}$; #mm^4

$$I_{t4} := 400000 \quad (14)$$

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$; #mm

$$z_1 := 45 \quad (15)$$

> $z_2 := \frac{h_2}{2} + \frac{h_3}{2}$; #mm

$$z_2 := 20 \quad (16)$$

> $z_3 := 0;$ #mm

$$z_3 := 0 \quad (17)$$

> $z_4 := \frac{h_4}{2} + \frac{h_3}{2};$ #mm

$$z_4 := 20 \quad (18)$$

> $z_5 := \frac{h_5}{2} + h_4 + \frac{h_3}{2};$ #mm

$$z_5 := 45 \quad (19)$$

Ei*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1};$ #Nmm^2

$$(EI)_1 := 17550000000 \quad (20)$$

> $(EI)_2 := E_{90,\ mean,\ t15} \cdot I_{t2};$ #Nmm^2

$$(EI)_2 := 92000000 \quad (21)$$

> $(EI)_3 := E_{0,\ mean,\ t15} \cdot I_{t3};$ #Nmm^2

$$(EI)_3 := 4600000000 \quad (22)$$

> $(EI)_4 := E_{90,mean,t15} \cdot I_{t4};$ #Nmm^2

$$(EI)_4 := 92000000 \quad (23)$$

> $(EI)_5 := E_{0,mean,t22} \cdot I_{t5};$ #Nmm^2

$$(EI)_5 := 17550000000 \quad (24)$$

> $(EI)_{sum} := (EI)_1 + (EI)_2 + (EI)_3 + (EI)_4 + (EI)_5;$ #Nmm^2

$$(EI)_{sum} := 39884000000 \quad (25)$$

Ei*Ao*zi^2

> $(EAz\hat{\ }2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{\ }2);$ #Nmm^2

$$(EAz\hat{\ }2)_1 := 473850000000 \quad (26)$$

> $(EAz\hat{\ }2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{\ }2);$ #Nmm^2

$$(EAz\hat{\ }2)_2 := 1104000000 \quad (27)$$

> $(EAz\hat{\ }2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{\ }2);$ #Nmm^2

$$(EAz\hat{\ }2)_3 := 0 \quad (28)$$

> $(EAz\hat{\ }2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{\ }2);$ #Nmm^2

$$(EAz\hat{\ }2)_4 := 1104000000 \quad (29)$$

> $(EAz\hat{\ }2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{\ }2);$ #Nmm^2

$$(EAz\hat{\ }2)_5 := 473850000000 \quad (30)$$

> $(EAz\hat{\ }2)_{sum} := (EAz\hat{\ }2)_1 + (EAz\hat{\ }2)_2 + (EAz\hat{\ }2)_3 + (EAz\hat{\ }2)_4 + (EAz\hat{\ }2)_5;$ #Nmm^2

(31)

$$(EAz\hat{\ }2)_{sum} := 949908000000 \quad (31)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{\ }2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (32)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$; #mm

$$a := 90 \quad (33)$$

> $(GA)_{eff} := evalf\left((a\hat{\ }2) \middle/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^{6} \quad (34)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$
> *#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load*

$$K_s := 11.5 \quad (35)$$

> $EI_{app} := \dfrac{(EI)_{eff}}{1 + \dfrac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L^2}}$; $\# \dfrac{N}{mm^2} \cdot mm^4$

$$EI_{app} := 7.260572161\ 10^{11} \qquad \textbf{(36)}$$

> $E_{CLT} := \dfrac{EI_{app}}{\dfrac{b \cdot h_t^3}{12}}$; $\# \dfrac{N}{mm^{\wedge}}$

$$E_{CLT} := 8403.440000 \qquad \textbf{(37)}$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c}$; $\# \dfrac{N}{mm^2}$

$$E_1 := 34000 \qquad \textbf{(38)}$$

> $E_2 := E_{CLT}$; $\# \dfrac{N}{mm}$

$$E_2 := 8403.440000 \qquad \textbf{(39)}$$

> $h_1 := h_c$; $\#mm$

$$h_1 := 60 \qquad \textbf{(40)}$$

> $h_2 := h_t$; $\#mm$

$$h_2 := 120 \qquad \textbf{(41)}$$

> $A_1 := A_c$;$\#mm^2$

$$A_1 := 36000 \qquad \textbf{(42)}$$

> $A_2 := h_2 \cdot b$; $\#mm^2$

$$A_2 := 72000 \qquad \textbf{(43)}$$

> $I_1 := I_c$; $\#mm^4$

(44)

$$I_1 := 10800000 \quad (44)$$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12}$; *#mm^4*

$$I_2 := 86400000 \quad (45)$$

The slip modulus **Kser**

To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22}$;

$$\rho_{m,1} := 470 \quad (46)$$

> $\rho_{m,\,2} := t_{t15}$;

$$\rho_{m,2} := 430 \quad (47)$$

> $\rho_m := evalf\left(\text{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right)$;

$$\rho_m := 449.5553359 \quad (48)$$

> $d_{kop} := 10$;*#mm*

$$d_{kop} := 10 \quad (49)$$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m\hat{}1.5 \cdot 3\right)$;

$$K_{ser} := 12432.77728 \quad (50)$$

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100$; *#mm*

$$s := 100 \quad (51)$$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L\hat{}2}}\right)$;

$$\gamma_1 := 0.03953909054 \quad (52)$$

> $\gamma_2 := 1.0$; *#Fully composite*

$$\gamma_2 := 1.0 \quad (53)$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2)}$; #mm

$$a_2 := 6.665650562 \quad (54)$$

>

> $a_1 := \frac{(h_1 + h_2)}{2} - a_2$; #mm

$$a_1 := 83.33434944 \quad (55)$$

> $EI_{eff,\ tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2$; #Nmm^2

$$EI_{eff,tot} := 1.456230493\ 10^{12} \quad (56)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,\ tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = - \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\ tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\ tot}} \right)}, M_{Ed,\ 1} \right)$; #Nmm

$$M_1 := 5.847328537\ 10^7 \quad (57)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = - \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}}\right)}, M_{Ed,\,1}\right); \#Nmm$

$$M_2 := 3.528423641\ 10^6 \qquad \textbf{(58)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$

$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$

$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$

$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \leq 1.0$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 5.310086235\ 10^7 \qquad \textbf{(59)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ =\ -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 7.243560277\ 10^7 \qquad \textbf{(60)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6}; \#kNm$$

$$M_{Ed,new} := 53.10086235 \qquad \textbf{(61)}$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \qquad \textbf{(62)}$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \qquad \textbf{(63)}$$

$$> P_{Ed} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed} := 139.1668296 \qquad \textbf{(64)}$$

3.4 Verification of the vertical defelction

$$> w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}};$$

$$w := 10.12905554 \qquad \textbf{(65)}$$

$$> w_{\lim} := evalf\left(\frac{L}{250}\right);$$

$$w_{\lim} := 8. \qquad \textbf{(66)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.266131942 \quad (67)$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1+\varphi_c}$;

$$E_{1,g} := 9714.285714 \quad (68)$$

> $E_{1,\,q} := \frac{E_{cm,c}}{1+\varphi_c \cdot \psi_2}$;

$$E_{1,q} := 15111.11111 \quad (69)$$

> $q_k := 0$;

$$q_k := 0 \quad (70)$$

> $g_{1,\,k} := 0$;

$$g_{1,k} := 0 \quad (71)$$

> $E_{1,\,fin} := \frac{E_{1,g} \cdot (g_{0,k}+g_{1,k}) \cdot \gamma_{G,1} + E_{1,\,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k}+g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}}$;

$$E_{1,fin} := 9714.285715 \quad (72)$$

4.1.2 CLT

> $E_{2,\,g} := \frac{E_{CLT}}{1+k_{def,\,t}}$;

$$E_{2,g} := 4542.400000 \quad (73)$$

> $E_{2,\,q} := \frac{E_{CLT}}{1+k_{def,\,t} \cdot \psi_2}$;

$$E_{2,q} := 5897.150877 \quad (74)$$

> $E_{2, fin} := \frac{E_{2, g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2, q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0, k} + g_{1,k}) \cdot \gamma_{G, 1} + q_k \cdot \gamma_{Q, 1}};$

$$E_{2,fin} := 4542.399998 \quad (75)$$

4.1.3 Slip modulus

>

> $K_{ser, g} := \frac{K_{ser}}{1 + k_{def, t}};$

$$K_{ser,g} := 6720.420151 \quad (76)$$

> $K_{ser, q} := \frac{K_{ser}}{1 + k_{def,t} \cdot \psi_2};$

$$K_{ser,q} := 8724.755986 \quad (77)$$

> $K_{ser, 2} := \frac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0, k} + g_{1,k}) \cdot \gamma_{G, 1} + q_k \cdot \gamma_{Q, 1}};$

$$K_{ser,2} := 6720.420149 \quad (78)$$

> $K_{u, fin} := K_{ser, 2};$

$$K_{u,fin} := 6720.420149 \quad (79)$$

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1, fin} := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u, fin} \cdot L\hat{}2}}\right);$

$$\gamma_{1,fin} := 0.07225561973 \quad (80)$$

> $\gamma_{2, fin} := 1.0;$

$$\gamma_{2,fin} := 1.0 \quad (81)$$

> $a_{2,fin} := \frac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)};$ #mm

$$a_{2,fin} := 6.454880643 \quad (82)$$

>

> $a_{1, fin} := \frac{(h_1 + h_2)}{2} - a_{2,fin};$ #mm

$$a_{1,fin} := 83.54511936 \quad (83)$$

> $EI_{eff,\ tot,\ fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\ fin}\hat{}2 + E_{2,\ fin} \cdot I_2 + \gamma_{2,\ fin} \cdot E_{2,\ fin} \cdot A_2 \cdot a_{2,fin}\hat{}2;$ #Nmm^2

$$EI_{eff,tot,fin} := 6.873754563\ 10^{11} \quad \mathbf{(84)}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin} \cdot M_{Ed,\ 1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 :$ #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 :$ #MPa

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

> $M_1 := solve\left(M_{Ed,\ 1} = \frac{f_{ck,\ c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\ 1} \right);$ #Nmm

$$M_1 := 8.925326927\ 10^7 \quad \mathbf{(85)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\ 0.005,\ c}}{\gamma_c}$$

> $M_2 := solve\left(M_{Ed,\ 1} = \frac{f_{ctk,\ 0.05,\ c}}{\gamma_c \cdot \left(-\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\ fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\ 1} \right);$ #Nmm

$$M_2 := 6.496171343\ 10^6 \quad \mathbf{(86)}$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

$$> M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 4.656652667\ 10^7 \qquad \textbf{(87)}$$

\>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\ t,\ b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

$$> M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

(88)

$$M_4 := 6.2891909\ 10^7 \quad (88)$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

$$> M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}; \#kNm$$

$$M_{Ed,new} := 46.56652667 \quad (89)$$

$$> L_{out} := 0.75; \#m$$

$$L_{out} := 0.75 \quad (90)$$

$$> L_{sup} := 2.0; \#m$$

$$L_{sup} := 2.0 \quad (91)$$

$$> P_{Ed,\ fin} := solve\left(\frac{P_{Ed,\ 1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed,fin} := 121.7419345 \quad (92)$$

5.4 Verification of the vertical deflection

Where creep is included.

$$> w_{permanent} := \frac{5 \cdot \left(\frac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{\ }4}{384 \cdot EI_{eff,tot,fin}};$$

$$w_{permanent} := 18.81816725 \quad (93)$$

>

$$> w_{\lim} := evalf\left(\frac{L}{150}\right);$$

$$w_{\lim} := 13.33333333 \quad (94)$$

Verification of the vertical deflection

$$> Ver_{deflection} := \frac{w_{permanent}}{w_{\lim}}; \# < 1.0\ NOT\ OK$$

$$Ver_{deflection} := 1.411362544 \quad (95)$$

>

Appendix F. Load capacity calculations for Slab A and Slab B based on ULS, 3-layered CLT

Load capacity predictions based on ULS
3-Layered CLT
Assumed using the CTC-screws 7-160 mm
This was performed in mid-February

> *restart*;

General data

[c] : Concrete B35
[t] : Timber T22 &T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

>

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\#\frac{kN}{m\hat{\ }3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber data C24 data taken from CLT handbook, make sure if this is the correct value compared to splitkon

3-LAYERED CLT

> $h_1 := 33 : \#mm$

> $h_2 := 33 : \#mm$

> $h_3 := 33 : \#mm$

>

Not usinhg the c/s area or moment of inertia for the whole timber structure, only need for each layer**

> $h_t := h_1 + h_2 + h_3; \#mm$

$$h_t := 99 \tag{1}$$

>

Lamelle 1 and 3 have class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \tag{2}$$

>

Lamelle 2 have class T15

> $E_{0,\ mean,\ t15} := 11500 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \#\frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \qquad (3)$$

>

>

> $\gamma_M := 1.15$: #*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*

>

LOAD - Calculations

Category

> $\gamma_{G,\,1} := 1.2$: # *Equation 6.10b give larger values*

> $\gamma_{Q,\,1} := 1.5$: # *Equation 6.10b give larger values*

> $\gamma_{G,\,2} := 1.0$:

> $\gamma_{Q,\,2} := 1.0$:

> $\psi_1 := 0.7$:

> $\psi_2 := 0.5$:

> $\psi_3 := 0.3$:

>

>

> $g_{0,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\frac{2}{3}\cdot\rho_{t22} + \frac{1}{3}\cdot\rho_{t15}\right)\right);\#\frac{kN}{m}$

$$g_{0,k} := 1.166014797 \qquad (4)$$

>

>

Values ULS

>

> $f_{d,ULS} := g_{0,k}\cdot\gamma_{G,1} :\#\frac{kN}{m}$

Gamma method, using Annex B in EC5 Maximum load capacity based on ULS using short-term verifications of the slab

Short-time verification using the gamma method for ULS

> *#EI(eff,KLT)=sum EiIi + sum EiAizi^2*

The slip modulus **Kser**
EC5: 2.2.2(2) Ultimate limit states equation 2.1
The values for the slip modulus (Kser) is found in the pdf about the screw types from the company **Rotho blaas page 227**
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

> $l_{eff,\,ctc} := 110 : \#mm$

> $K_{ser} := 3\cdot 70\cdot l_{eff,\,ctc} : \#\frac{N}{mm}$

> $K_u := evalf\left(\frac{2}{3}\cdot K_{ser}\right) : \#\frac{N}{mm}$ #equation 2.1 EC5

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen.

> $angle := 45 :$

> $k := \sin(convert(angle\ degrees, radians)) :$

> $s_{\min,\,1} := evalf(130\cdot k) :\#mm$

>

> $s := 150 : \#mm$

General data:

> $E_1 := E_{cm,\, c} : \# \frac{N}{mm\hat{\ }2}$

> $E_2 := E_{0,\, mean,\, t22} :\# \frac{N}{mm}$

> $E_3 := E_{0,mean,t22} : \# \frac{N}{mm}$

> $h_1 := h_c : \#mm$

> $h_2 := h_2 : \#mm$

> $h_3 := h_3 : \#mm$

>

> $A_1 := A_c : \#mm\hat{\ }2$

> $A_2 := h_2 \cdot b : \#mm\hat{\ }2$

> $A_3 := h_3 \cdot b : \#mm\hat{\ }2$

> $I_1 := I_c : \#mm\hat{\ }4$

> $I_2 := \frac{b \cdot h_2\hat{\ }3}{12} : \#mm\hat{\ }4$

> $I_3 := \frac{b \cdot h_2\hat{\ }3}{12} : \#mm\hat{\ }4$

> $h_4 := 33 : \#mm$

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{\ }2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{\ }2}}\right);$

$$\gamma_1 := 0.03287684039 \quad (5)$$

> $\gamma_2 := 1.0 : \#Fully\ composite$

> $\gamma_3 := evalf\left(\frac{1}{1 + \frac{\pi\hat{\ }2 \cdot E_3 \cdot s \cdot A_3}{K_u \cdot L\hat{\ }2}}\right) :$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2) - \gamma_3 \cdot E_3 \cdot A_3 \cdot \left(\frac{h_2}{2} + \frac{h_3}{2} + h_4\right)}{(\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2 + \gamma_3 \cdot E_3 \cdot A_3)} : \#mm$

>

> $a_1 := \frac{h_1 + h_2}{2} - a_2 : \#mm$

> $a_3 := \frac{(h_2 + h_3 + 2 \cdot h_4)}{2} + a_2$: #mm

> $In_1 := \frac{1}{12} \cdot b \cdot h_1\hat{}3$: #mm^4

> $In_2 := \frac{1}{12} \cdot b \cdot h_2\hat{}3$: #mm^4

> $In_3 := \frac{1}{12} \cdot b \cdot h_3\hat{}3$: #mm^4

> $EI_{eff,\, tot} := E_1 \cdot In_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot In_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2 + E_3 \cdot In_3 + \gamma_3 \cdot E_3 \cdot A_3 \cdot a_3\hat{}2$: #Nmm^2

> $gEa_{sum} := \gamma_1 \cdot E_1 \cdot a_1 + \gamma_2 \cdot E_2 \cdot a_2 + \gamma_3 \cdot E_3 \cdot a_3$:

OBS, above the MEd and VEd is in KNm and KN, have to multiply with 10^6

MED,1 is UNKNOWN!!

Normal stresses in the **concrete** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\, 1})}{EI_{eff, tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\, 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,\, 1})}{EI_{eff, tot}} \cdot 10\hat{}6$: #MPa

Stresses at the top of the concrete section

#sigma[c,t]#= [-sigma[1] - sigma[m,1] | #`=`fc,d#= f[ck]/g[c]

>

$$\#M[Ed, 1] \cdot \left(\frac{(g[1] \cdot E[1] \cdot a[1] \cdot M[Ed, 1])}{EI[eff, tot]} \cdot 10\hat{}6 + \frac{(0.5 \cdot E[1] \cdot h[1] \cdot M[Ed, 1])}{EI[eff, tot]} \cdot 10\hat{}6 \right) \leq \frac{f[ck]}{g[c]}$$

> $M_1 := solve\left(M_{Ed,1} = \frac{f_{ck,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff, tot}} \cdot 10\hat{}6 + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff, tot}} \cdot 10\hat{}6 \right)}, M_{Ed,1} \right)$: # kNm

> $M_1 := 3.817756159 \cdot 10\hat{}6;$

$$M_1 := 3.817756159\ 10^6 \quad (6)$$

Stresses at the bottom of the concrete section

#sigma[c,t]#= | -sigma[1] + sigma[m,1] | # `= `fc,d#= f[ck]/g[c]

$$\#M[Ed,1]\cdot\left(\frac{(g[1]\cdot E[1]\cdot a[1]\cdot M[Ed,1])}{EI[eff,tot]}\cdot 10\hat{}6+\frac{(0.5\cdot E[1]\cdot h[1]\cdot M[Ed,1])}{EI[eff,tot]}\cdot 10\hat{}6\right)\leq\frac{f[ctk,0.05,c]}{g[c]}$$

$$> M_2 := solve\left(\left\{M_{Ed,1}=\frac{f_{ctk,0.05,c}}{\gamma_c\cdot\left(-\frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6+\frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10\hat{}6\right)},\ M_{Ed,1}>0\right\}\right):\#\ kNm$$

$$> M_2 := 1.002683652\cdot 10\hat{}6;$$

$$M_2 := 1.002683652\ 10^6 \quad (7)$$

Normal stresses in the **timber** section

$$> \sigma_2 := \frac{(\gamma_3\cdot E_3\cdot a_3\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6:\#MPa$$

$$> \sigma_{m,2} := \frac{(0.5\cdot E_3\cdot h_3\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6:\#MPa$$

Stresses at the second layer h2

#sigma[c,t]#= | -sigma[1] - sigma[m,1] | # `= `fc,d#= f[ck]/g[c]

>

$$\#M[Ed,2]\cdot\left(\frac{\left(\frac{(g[3]\cdot E[3]\cdot a[3]\cdot M[Ed,2])}{EI[eff,tot]}\cdot 10\hat{}6\right)}{\frac{km[modi,t]\cdot f[t,0,k,t22]}{g[M]}}+\frac{\left(\frac{(0.5\cdot E[3]\cdot h[3]\cdot M[Ed,2])}{EI[eff,tot]}\cdot 10\hat{}6\right)}{\frac{km[modi,t]\cdot f[m,k,t,t22]}{g[M]}}\right)\leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_3 \cdot E_3 \cdot a_3)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_3 \cdot h_3)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 3.623582836\ 10^7 \quad (8)$$

>

Stresses at the third layer h3

#sigma[c,t]#= | -sigma[1] - sigma[m,1] | #`=`fc,d#= f[ck]/g[c]

>

$$\#M[Ed,3] \cdot \left(\frac{\left(\frac{(g[2] \cdot E[2] \cdot a[2] \cdot M[Ed,3])}{EI[eff,tot]} \cdot 10\hat{\ }6\right)}{\frac{km[modi,t] \cdot f[t,0,k,t22]}{g[M]}} + \frac{\left(\frac{(0.5 \cdot E[2] \cdot h[2] \cdot M[Ed,3])}{EI[eff,tot]} \cdot 10\hat{\ }6\right)}{\frac{km[modi,t] \cdot f[m,k,t,t22]}{g[M]}}\right) \leq 1.0$$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 4.894929279\ 10^7 \quad (9)$$

OBS: Have neglected to consider the M2, bottom bending momnet on the concrete. SHould this be included?? se calculations below then....

> $M_{Ed,new} := \frac{\min(M_1, M_3, M_4)}{10\hat{\ }6}; \#kNm$

$$M_{Ed,new} := 3.817756159 \quad (10)$$

> $L := 2.0 :\#m$

> $b := 0.6 :$

>

> $P_{Ed} := solve\left(\frac{P_{Ed,1} \cdot L}{4} + \frac{1.5 \cdot g_{0,k} \cdot L\hat{\ }2}{8} = M_{Ed,new}, P_{Ed,1}\right); \#kN$

$$P_{Ed} := 5.886490122 \quad (11)$$

Verification of the Maximum load using new parameters

Normal stresses in the **CONCRETE** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ new})}{EI_{eff,tot}} \cdot 10\text{\^{}}6 : \#MPa$

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,\ new})}{EI_{eff,tot}} \cdot 10\text{\^{}}6 : \#MPa$

Stresses at the TOP of the concrete section

> $\sigma_{c,\ t} := -\sigma_1 - \sigma_{m,1} : \#MPa$

Verification of the top section

> $Ver_{top,\ c} := \frac{\sigma_{c,t}}{\frac{f_{ck,\ c}}{\gamma_c}}; \# < 1.0 \#{-}{-}{-} > OK$

$$Ver_{top,c} := -0.2619339628 \tag{12}$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,\ c} := -\sigma_1 + \sigma_{m,1} : \#MPa$

> $Ver_{bottom,\ c} := \frac{\sigma_{b,\ c}}{\frac{f_{ctk,\ 0.05,\ c}}{\gamma_c}}; \# < 1.0 \#{-}{-}{-} > OK$

$$Ver_{bottom,c} := 3.797347298 \tag{13}$$

Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ new})}{EI_{eff,tot}} \cdot 10\text{\^{}}6 : \#MPa$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\text{\^{}}6 : \#MPa$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2} : \#MPa$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,t} := -\sigma_2 + \sigma_{m,2}$: #MPa

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

$$Ver_{timber} := -0.05697372838 \tag{14}$$

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_2)^{\wedge}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10^{\wedge}3$:#MPa

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{shear} := 0.008780450773 \tag{15}$$

5.1.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$: #kN

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_2 := 1.409068007 \tag{16}$$

> $F_3 := \frac{\gamma_3 \cdot E_3 \cdot A_3 \cdot a_3 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}$; #kN

$$F_3 := 3.326884252 \tag{17}$$

> $f_{tens,k} := 20.0$; #kN

$$f_{tens,k} := 20.0 \tag{18}$$

> $Ver_{F1} := \frac{F_1}{3 \cdot \frac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.05409387010 \tag{19}$$

> $Ver_{F1} := \dfrac{F_2}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.03375892100 \quad \textbf{(20)}$$

> $Ver_{F1} := \dfrac{F_3}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}}$; # <1.0 #--> OK

$$Ver_{F1} := 0.07970660190 \quad \textbf{(21)}$$

>

>

>

Load capacity predictions based on ULS
3-Layered CLT
Assumed using the KOP-screws 10-140 mm
This was performed in mid-February

> *restart*;

General data

[c] : Concrete B35
[t] : Timber T22 &T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

>

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\#\dfrac{kN}{m\hat{}3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber data C24 data taken from CLT handbook, make sure if this is the correct value compared to splitkon

3-LAYERED CLT

> $h_1 := 33 : \#mm$

> $h_2 := 33 : \#mm$

> $h_3 := 33 : \#mm$

>

Not usinhg the c/s area or moment of inertia for the whole timber structure, only need for each layer**

> $h_t := h_1 + h_2 + h_3; \#mm$

$$h_t := 99 \tag{1}$$

>

Lamelle 1 and 3 have class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \tag{2}$$

>

Lamelle 2 have class T15

> $E_{0,\ mean,\ t15} := 11500 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \#\frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 :\# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

>

>

> $\gamma_M := 1.15$: *#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*

>

LOAD - Calculations

Category

> $\gamma_{G,\,1} := 1.2$: *# Equation 6.10b give larger values*

> $\gamma_{Q,\,1} := 1.5$: *# Equation 6.10b give larger values*

> $\gamma_{G,\,2} := 1.0$:

> $\gamma_{Q,\,2} := 1.0$:

> $\psi_1 := 0.7$:

> $\psi_2 := 0.5$:

> $\psi_3 := 0.3$:

>

>

> $g_{0,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\frac{2}{3} \cdot \rho_{t22} + \frac{1}{3} \cdot \rho_{t15}\right)\right);\# \frac{kN}{m}$

$$g_{0,k} := 1.166014797 \quad (4)$$

>

>

Values ULS

>

> $f_{d,ULS} := g_{0,k} \cdot \gamma_{G,1} :\# \frac{kN}{m}$

Gamma method, using Annex B in EC5 Maximum load capacity based on ULS using short-term verifications of the slab

Short-time verification using the gamma method for ULS

> *#EI(eff,KLT)=sum EiIi + sum EiAizi^2*

The slip modulus **Kser**
To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\ 1} := t_{t22}:$

> $\rho_{m,\ 2} := t_{t15}:$

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\ 1} \cdot \rho_{m,\ 2}\right)\right):$

> $d_{kop} := 10 : \#mm$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m\hat{}1.5 \cdot 3\right):$

> $K_u := evalf\left(\frac{2}{3} \cdot K_{ser}\right); \# \frac{N}{mm}$

$$K_u := 8288.518187 \quad (5)$$

>

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100;$ #*mm*

$$s := 100 \quad \textbf{(6)}$$

General data:

> $E_1 := E_{cm,\ c}$: #$\frac{N}{mm\hat{}2}$

> $E_2 := E_{0,\ mean,\ t22}$:#$\frac{N}{mm}$

> $E_3 := E_{0, mean, t22}$: #$\frac{N}{mm}$

> $h_1 := h_c$: #*mm*

> $h_2 := h_2$: #*mm*

> $h_3 := h_3$: #*mm*

>

> $A_1 := A_c$: #*mm^2*

> $A_2 := h_2 \cdot b$: #*mm^2*

> $A_3 := h_3 \cdot b$: #*mm^2*

> $I_1 := I_c$: #*mm^4*

> $I_2 := \frac{b \cdot h_2\hat{}3}{12}$: #*mm^4*

> $I_3 := \frac{b \cdot h_2\hat{}3}{12}$:#*mm^4*

> $h_4 := 33$: #*mm*

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_1 \cdot s \cdot A_1}{K_u \cdot L\hat{}2}}\right);$

$$\gamma_1 := 0.02671144241 \quad \textbf{(7)}$$

> $\gamma_2 := 1.0$: #*Fully composite*

> $\gamma_3 := evalf\left(\frac{1}{1 + \frac{\pi\hat{}2 \cdot E_3 \cdot s \cdot A_3}{K_u \cdot L\hat{}2}}\right)$:

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot (h_1 + h_2) - \gamma_3 \cdot E_3 \cdot A_3 \cdot \left(\frac{h_2}{2} + \frac{h_3}{2} + h_4\right)}{(\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2 + \gamma_3 \cdot E_3 \cdot A_3)}$: #mm

>

> $a_1 := \frac{h_1 + h_2}{2} - a_2$: #mm

> $a_3 := \frac{(h_2 + h_3 + 2 \cdot h_4)}{2} + a_2$: #mm

> $In_1 := \frac{1}{12} \cdot b \cdot h_1\hat{}3$: #mm^4

> $In_2 := \frac{1}{12} \cdot b \cdot h_2\hat{}3$: #mm^4

> $In_3 := \frac{1}{12} \cdot b \cdot h_3\hat{}3$: #mm^4

> $EI_{eff,\ tot} := E_1 \cdot In_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1\hat{}2 + E_2 \cdot In_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2\hat{}2 + E_3 \cdot In_3 + \gamma_3 \cdot E_3 \cdot A_3 \cdot a_3\hat{}2$: #Nmm^2

> $gEa_{sum} := \gamma_1 \cdot E_1 \cdot a_1 + \gamma_2 \cdot E_2 \cdot a_2 + \gamma_3 \cdot E_3 \cdot a_3$:

OBS, above the MEd and VEd is in KNm and KN, have to multiply with 10^6

MED,1 is UNKNOWN!!

Normal stresses in the **concrete** section

> $\sigma_1 := \frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\ 1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

> $\sigma_{m,\ 1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,\ 1})}{EI_{eff,tot}} \cdot 10\hat{}6$: #MPa

Stresses at the top of the concrete section

#sigma[c,t]#= [-sigma[1] - sigma[m,1] | #`=`fc,d#= f[ck]/g[c]

>

$$\#M[Ed, 1] \cdot \left(\frac{(g[1] \cdot E[1] \cdot a[1] \cdot M[Ed, 1])}{EI[eff, tot]} \cdot 10\hat{}6 + \frac{(0.5 \cdot E[1] \cdot h[1] \cdot M[Ed, 1])}{EI[eff, tot]} \cdot 10\hat{}6 \right) \leq \frac{f[ck]}{g[c]}$$

> $M_1 := solve\left(M_{Ed,1} = \frac{f_{ck,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6\right)}, M_{Ed,1}\right) : \# kNm$

> $M_1 := 3.697736206 \cdot 10\hat{}6;$

$$M_1 := 3.697736206\ 10^6 \quad \textbf{(8)}$$

Stresses at the bottom of the concrete section

#sigma[c,t]#= | -sigma[1] + sigma[m,1] | # `= `fc,d#= f[ck]/g[c]

$\#M[Ed, 1] \cdot \left(\frac{(g[1] \cdot E[1] \cdot a[1] \cdot M[Ed, 1])}{EI[eff, tot]} \cdot 10\hat{}6 + \frac{(0.5 \cdot E[1] \cdot h[1] \cdot M[Ed, 1])}{EI[eff, tot]} \cdot 10\hat{}6\right) \leq \frac{f[ctk, 0.05, c]}{g[c]}$

> $M_2 := solve\left(M_{Ed,1} = \frac{f_{ctk,0.05,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6\right)}, M_{Ed,1}\right) : \# kNm$

> $M_2 := 0.9633790713 \cdot 10\hat{}6;$

$$M_2 := 963379.0713 \quad \textbf{(9)}$$

Normal stresses in the **timber** section

> $\sigma_2 := \frac{(\gamma_3 \cdot E_3 \cdot a_3 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,2} := \frac{(0.5 \cdot E_3 \cdot h_3 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

Stresses at the second layer h2

#sigma[c,t]#= | -sigma[1] - sigma[m,1] | # `= `fc,d#= f[ck]/g[c]

>

$$\#M[Ed,2]\cdot\left(\frac{\left(\frac{(g[3]\cdot E[3]\cdot a[3]\cdot M[Ed,2])}{EI[eff,tot]}\cdot 10\hat{}6\right)}{\frac{km[modi,t]\cdot f[t,0,k,t22]}{g[M]}}+\frac{\left(\frac{(0.5\cdot E[3]\cdot h[3]\cdot M[Ed,2])}{EI[eff,tot]}\cdot 10\hat{}6\right)}{\frac{km[modi,t]\cdot f[m,k,t,t22]}{g[M]}}\right)\leq 1.0$$

> $M_3 := solve\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_3\cdot E_3\cdot a_3)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_3\cdot h_3)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_3 := 3.669880347\ 10^7 \quad (10)$$

>

Stresses at the third layer h3

#sigma[c,t]#= | -sigma[1] - sigma[m,1] | # `= `fc,d#= f[ck]/g[c]

>

$$\#M[Ed,3]\cdot\left(\frac{\left(\frac{(g[2]\cdot E[2]\cdot a[2]\cdot M[Ed,3])}{EI[eff,tot]}\cdot 10\hat{}6\right)}{\frac{km[modi,t]\cdot f[t,0,k,t22]}{g[M]}}+\frac{\left(\frac{(0.5\cdot E[2]\cdot h[2]\cdot M[Ed,3])}{EI[eff,tot]}\cdot 10\hat{}6\right)}{\frac{km[modi,t]\cdot f[m,k,t,t22]}{g[M]}}\right)\leq 1.0$$

> $M_4 := solve\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right)$; #Nmm

$$M_4 := 4.782903275\ 10^7 \quad (11)$$

OBS: Have neglected to consider the M2, bottom bending momnet on the concrete. SHould this be included?? se calculations below then....

> $M_{Ed,new} := \frac{\min(M_1, M_3, M_4)}{10^6}$; #kNm

$$M_{Ed,new} := 3.697736206 \quad (12)$$

> $L := 2.0$:#m

> $b := 0.6$:

>

> $P_{Ed} := solve\left(\frac{P_{Ed,1} \cdot L}{4} + \frac{1.5 \cdot g_{0,k} \cdot L^2}{8} = M_{Ed,new}, P_{Ed,1}\right)$; #kN

$$P_{Ed} := 5.646450216 \quad (13)$$

Verification of the Maximum load using new parameters

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

> $\sigma_{m,1} := \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10^6$: #MPa

Stresses at the TOP of the concrete section

> $\sigma_{c,t} := -\sigma_1 - \sigma_{m,1}$: #MPa

Verification of the top section

> $Ver_{top,c} := \frac{\sigma_{c,t}}{\frac{f_{ck,c}}{\gamma_c}}$; # <1.0 #--->OK

$$Ver_{top,c} := -0.2704357326 \quad (14)$$

Stresses at the BOTTOM of the concrete section

> $\sigma_{b,c} := -\sigma_1 + \sigma_{m,1}$: #MPa

> $Ver_{bottom,c} := \frac{\sigma_{b,c}}{\frac{f_{ctk,0.05,c}}{\gamma_c}}$; # <1.0 #--->OK

$$Ver_{bottom,c} := 3.984203486 \quad (15)$$

Normal stresses in the **TIMBER** section

> $\sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\ new})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,\ 2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,new})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$

Stresses at the TOP of the timber section

> $\sigma_{t,\ t} := -\sigma_2 - \sigma_{m,\ 2} : \#MPa$

Stresses at the BOTTOM of the timber section

> $\sigma_{b,\ t} := -\sigma_2 + \sigma_{m,\ 2} : \#MPa$

Verification of the timber section

> $Ver_{timber} := \left(\frac{\sigma_{t,t}}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\sigma_{b,t}}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right); \# < 1.0 \# \text{-->} OK$

$Ver_{timber} := -0.05267341500$ **(16)**

5.1.3 Shear stresses in the **TIMBER** section

> $\tau_2 := \frac{0.5 \cdot E_2 \cdot b \cdot (0.5 \cdot h_2 + a_2)\hat{}2}{b \cdot EI_{eff,tot}} \cdot P_{Ed} \cdot 10\hat{}3 : \#MPa$

Verification of the timber section

> $Ver_{shear} := \frac{\tau_2}{\frac{k_{modi,t} \cdot f_{v,k,t22}}{\gamma_M}}; \# < 1.0 \# \text{-->} OK$

$Ver_{shear} := 0.008394771596$ **(17)**

5.1.4 The **load per shear fasteners**

> $F_1 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \cdot s}{EI_{eff,tot}} \cdot P_{Ed} : \#kN$

> $F_2 := \frac{\gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$

$F_2 := 0.7904022750$ **(18)**

> $F_3 := \frac{\gamma_3 \cdot E_3 \cdot A_3 \cdot a_3 \cdot s}{EI_{eff,tot}} \cdot P_{Ed}; \#kN$

$F_3 := 1.875368276$ **(19)**

> $f_{tens,\ k} := 20.0;$ #*kN*

$$f_{tens,k} := 20.0 \tag{20}$$

> $Ver_{F1} := \dfrac{F_1}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,\ k}}{\gamma_M}};$ # <1.0 #--> OK

$$Ver_{F1} := 0.03073104382 \tag{21}$$

> $Ver_{F1} := \dfrac{F_2}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}};$ # <1.0 #--> OK

$$Ver_{F1} := 0.01893672117 \tag{22}$$

> $Ver_{F1} := \dfrac{F_3}{3 \cdot \dfrac{k_{modi,t} \cdot f_{tens,k}}{\gamma_M}};$ # <1.0 #--> OK

$$Ver_{F1} := 0.04493069827 \tag{23}$$

>

>

>

Appendix G. Sola Betong, Concrete recipe

G.1 The Concrete recipe used for the laboratory testing

Appendix G.1 The Concrete recipe used for the laboratory testing

Tel.: +47 51 64 49 49
W..: - @..: post@sola-betong.no

Resept opplysninger

Resept	: 251 ~ B35 M45 SKB dmax 16 std FA SF2		
Opprettet av	: Rune	Dato	: 18-10-2016 13:02:41
Redigert av	: Rune	Dato	: 07-06-2021 11:24:39
Resepttype	: Fast verdi	Status	: Aktiv
Konsistenstype	: Synkudbredningsmål		
Varepris navn	:	Varepris	: B23516003000
Familie	: B	Familie navn	: standard fa u/luft
Tilslagsspec.	: 11 SKB ~ SKB 16		
Bindemiddel spec.	: 71 ~ Std Fa 90 10 Flyveaske 3,3% SILICA		
Vannspec.	: 01 ~ Kaldt Vann		
Kjemispec.	: 31 B35 SKB ~ SX 23 1,0 %+ luft 0,1%		

Standard : NS206

VC spec.nr.	:	V/C-Forhold	: 0,447
Bestandighetsklasse	: M45	Ameringstål	: Ingen valgt
Kloridklasse	: Cl 0,10	Kontrollklasse	: Normal
Modenhetsminutter	:	Klassifikasjon	: Designet
Fasthetsklasse	: B35	Manuel børverdi	: 60
M³ siden sidste prøve(fam.)	: 143,30	M³ siden siste prøve	: 29,90
Rct.prv.hyp. i periode	: 23,97		
Eksponeringsklasse	: X0, XC1, XC2, XC3, XC4, XF1, XD1, XS1, XA1, XA2, XA4		

Stamopplysninger

Min. sement innhold	: Nei		
Min. sement innhold	:	Max	:
Min. filler innhold	:	Max	:
Synkutbredelsesinterval	: 500 - 700	Betongtype	:
Bruk tilstrebt synkutbredels	: Ja	Tilstræbt synkmål	: 630
Ekstra Specifikationer	:	Sertifiseringsorgan	:
Auto % andel af vann ved fl	: 100,00		
Salgs	:	Prod. pris	:

Prøvning

Uttak prøve	: Nei	Dato	: 07-10-2016
Prøvehyppighet	:		
Uttak prøve bemerkninger	:		
Forprøve gruppenr.	: Ingen valgt	Forældr.	:
Dato for siste prøve	: 27-01-2021	Dato for siste produksjon	: 26.05.2021
Siste forprøve	: 45580		

Blanderdata

Blandernavn	Blandetid	Tømmetid	Deltatid	Blander korr.
1 (Blander 1)	40,00	7,00	0,00	0,00
2 (Blander 2)	40,00	7,00	0,00	0,00

Vekt forsinkelse

Blander: Blander 1

Vækt:	A1-Tilslag 1	A1-Tilslag 2	A1-Pulver	A1-Vann	A1-Kjemi 1	A1-Kjemi 2	A1-Fiber
Sek:	0	0	10	15	16	16	0

Resept flyt synkmål:

Install:	550	600	650	700
VannBehov:	176,00	179,00	183,00	187,00
Luftinnhold %:	2,00	2,00	2,00	2,00

Tilslag

Synkmål

Materialer	Alle
Velde 8-16mm	36,00
Velde 08mm sand	48,00
Velde 02mm fin sand	16,00

Tel.: +47 51 64 49 49
W..: - @..: post@sola-betong.no

Bindemiddel

Materiale	Synkmål Alle
Silika	3,30
K-verdi	2,00
Tyrkisk flyveaske	6,00
K-verdi	0,70
Standard sement FA	90,70
K-verdi	1,00

Vann

Materiale	Procent
varmt vann	
Kaldt vann	100,00

Kjemi

Synkudbredningsmål

Materiale:	Av materiale	Forsinkelse	550	600	650	700
Mapeair 25 1:19	% av bindemiddel		0,10	0,10	0,10	0,10
Dynamon SX-23	% av bindemiddel	10,00	0,98	1,02	1,10	1,15

Proporsjonering

Synkudbredningsmål : 650
Luft : 2,0
Ekv. sement : 409,396
Samlet vannbehov : 183,000

Materialer	Kilo/m³ VOT	Vanninnhold	Kilo/m³	Pris/Kg	Pris/m3	CO2/m³
Velde 8-16mm	625,052	0,50	628,165			2,22
Velde 08mm sand	833,403	1,50	845,780			2,96
Velde 02mm fin sand	280,958	1,50	285,114			1,00
Silika	13,310	0,00	13,310			0,00
Tyrkisk flyveaske	24,201	0,00	24,201			0,00
Standard sement FA	365,835	0,00	365,835			215,57
Kaldt vann	179,182	100,00	159,535			0,00
varmt vann	0,000	100,00	0,000			0,00
Mapeair 25 1:19	0,403	99,70	0,403			0,01
Dynamon SX-23	4,437	77,00	4,437			4,44
	2326,780			2326,780		226,19

Min/max sementinnhold er anvendt under proporsjoneringen

NS206

	Resultat	Krav	Ok
Vannbehov (Fri)	183,000	-	
Effektiv bindemiddel (Fri)	409,396	-	
V/C fri beregning	0,447	-	
Vannbehov (EN206)	183,000	-	
Effektiv Bindemiddel (EN206)	409,396	300,000	✓
V/C i henhold til EN206	0,447	0,454	✓
Eff. Bindemiddel mængde fratrukket k	0,000	-	
Bindemiddel (total kg)	403,346	-	
Luft %	2,000	-	
Beregnet m³	1,000	-	
Kloridinnhold	0,078	0,100	✓
Andel reaktiv tilslag %	0,000	-	
Alkaliinnhold	5,211	-	
Flyveaske/bindemiddel forhold	0,223	0,350	✓
Silika/bindemiddel forhold	0,033	0,110	✓
Flyveaske, Ren sement andel	70,746	65,000	✓
Slagg, Ren sement andel	0,000	-	
Matriksvolum eks. luft (l)	425,243	-	
Sementpastavolum (l)	321,517	-	
Samlet vurdering			✓

Tel.: +47 51 64 49 49
W..: - @..: post@sola-betong.no

BRANEL

Blanket

Resept nr.	: 251 ~ B35 M45 SKB dmax 16 std FA SF2
Familie	: B
Anvendelse 1	:
Anvendelse 2	:

Klassifikasjon

Bestandighetsklasse	: M45	Eksponeringsklasse	: X0, XC1, XC2, XC3, XC4, XF1, XD1, XS1, : XA1, XA2, XA4
Fasthetsklasse	: B35	Tilstræbt kons.	: 630
Kontrollklasse	: Normal	Ekstra Specifikationer	:
Max. Steinstørrelse	: 16	Sertifiseringsorgan	:

Materiale sammensetning

Forkortelse	Materiale	Densitet Kg/m³	Mengde Kg/m³	Volum Liter m³	Dekl.dato
V16	Velde 8-16mm	2640,000	625,052	236,762	17-10-2016
V08	Velde 08mm sand	2640,000	833,403	315,683	17-10-2016
V02	Velde 02mm fin sand	2670,000	280,958	105,228	17-10-2016
silika	Silika	2200,000	13,310	6,050	07-10-2016
flyveaske	Tyrkisk flyveaske	2300,000	24,201	10,522	07-10-2016
Std FA Brevik Norcem	Standard sement FA	3000,000	365,835	121,945	07-10-2016
K-Vann	Kaldt vann	1000,000	179,182	179,182	17-10-2016
V-Vann	varmt vann	1000,000	0,000	0,000	17-10-2016
Luft	Mapeair 25 1:19	1000,000	0,403	0,403	07-10-2016
SX-23	Dynamon SX-23	1050,000	4,437	4,226	07-10-2016
Tilstræbt luft i betong (2,0 Vol %)				20,000	
			2326,780	1000,000	

Sand	V08	V02	Stein	V16
Materialeklasse			Materialeklasse	
Humus			Lette korn < 2200 kg/m³	
Kjemisk svind Ml/kg			Lette korn < 2400 kg/m³	
Innhold av reaktive korn			Lette korn < 2500 kg/m³	
Mørtelekspansjon % Uge			Kritisk absorbtion av 10 Pct.	
Acc. mørtelekspansion % Ug			Acc. mørtelekspansion % Ug	
Absorbtsjon %	1,00	1,40	Absorbtsjon %	0,40
D_{Lower}			D_{Lower}	
D^{Upper}			D^{Upper}	

Sement	Sulfatres.
Standard sement FA	Nei

Andre tilsetninger	Tilsetningsstoffer	Luft	SX-23
Tørstofinnhold %	Tørstofinnhold %	0,30	23,00

Fibre	Vann	K-Vann	V-Vann
Type	Materialeklasse		
Fiber tversnit	Tørstofinnhold %	0,00	0,00
Fiber lengde			

Klorid/Alkali regnskab		Kloridberegning		Alkaliberegning	
Innhold av delmaterialer	Kg/m³	% cl	Kg/m³	% Ekv. Alk	Kg/m³
Velde 8-16mm	625,052	0,000	0,000		
Velde 08mm sand	833,403	0,000	0,000		
Velde 02mm fin sand	280,958	0,000	0,000		
Silika	13,310	0,000	0,000	0,000	0,000
Tyrkisk flyveaske	24,201	0,000	0,000	0,000	0,000
Standard sement FA	365,835	0,085	0,311	1,400	5,122
Kaldt vann	179,182	0,000	0,000	0,000	0,000
varmt vann	0,000	0,000	0,000	0,000	0,000
Mapeair 25 1:19	0,403	0,050	0,000	0,200	0,001
Dynamon SX-23	4,437	0,050	0,002	2,000	0,089
Total			0,313		5,211

Tel.: +47 51 64 49 49
W..: - @..: post@sola-betong.no

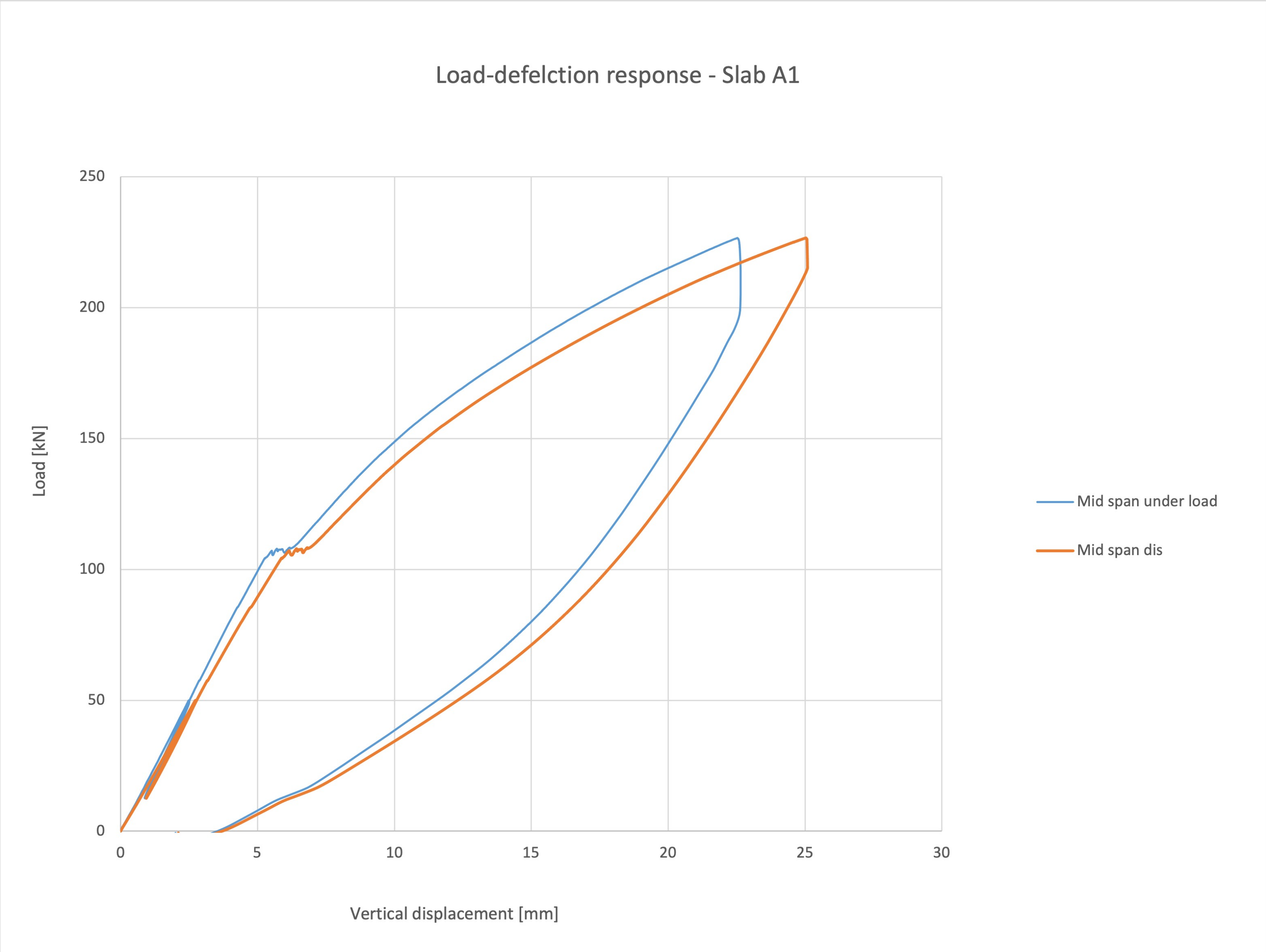
KornKurver, gjennomgang i %				
	V16	**V08**	**V02**	**Total**
Mengde, Kg	625,052	833,403	280,958	1739,412
Vol.-%	36,000	48,000	16,000	100,000
Sikt, mm				
64,000	100,000	100,000	100,000	100,000
32,000	100,000	100,000	100,000	100,000
16,000	99,324	100,000	100,000	99,757
8,000	89,654	97,800	100,000	95,220
4,000	78,578	75,300	100,000	80,432
2,000	67,461	63,300	94,000	69,710
1,000	56,313	45,600	71,000	53,521
0,500	45,138	29,700	50,000	38,506
0,250	33,934	16,900	32,000	25,448
0,125	22,699	9,400	19,300	15,772
0,063	11,425	5,300	10,000	8,257
0,000	0,000	0,000	0,000	0,000

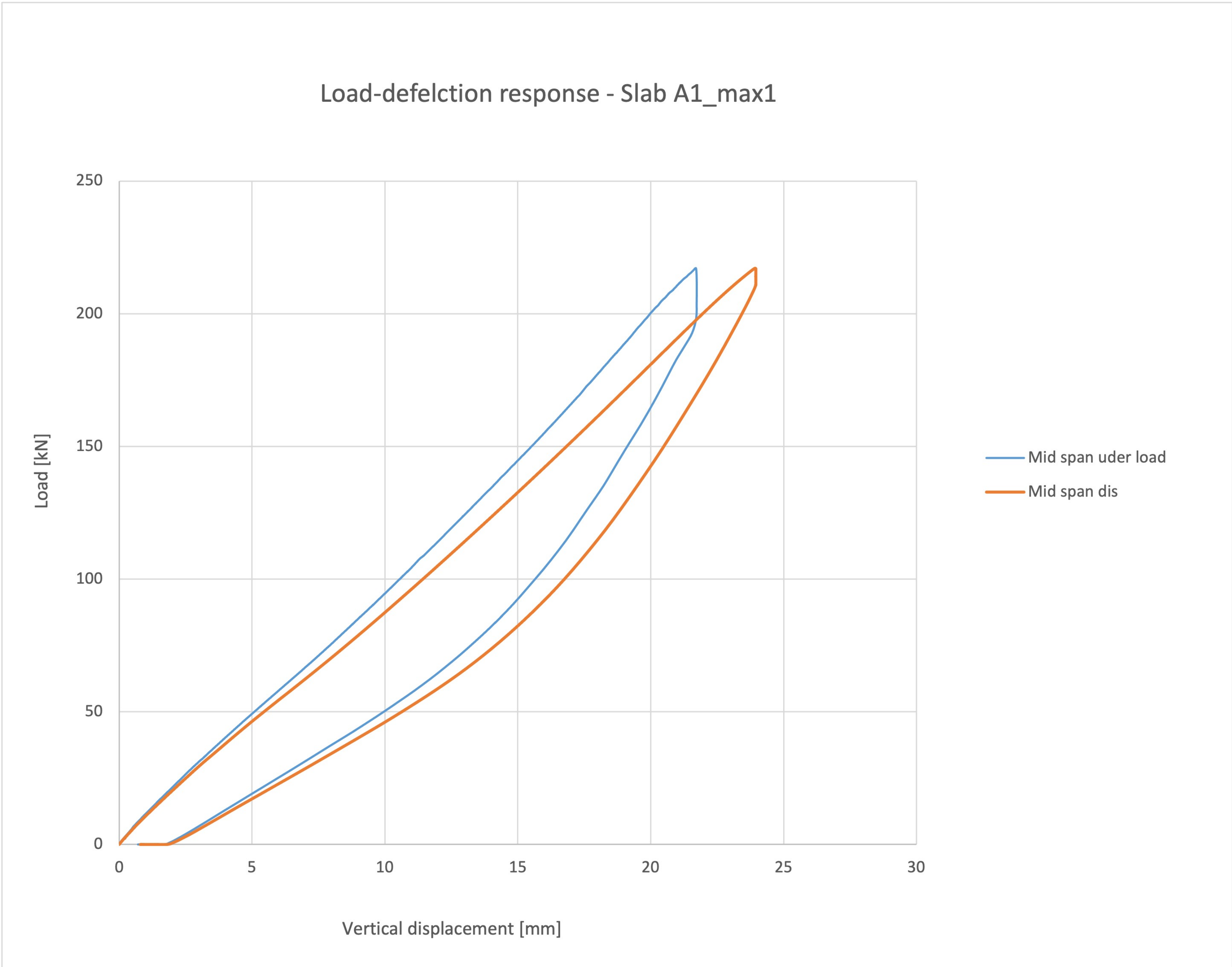
Appendix H. Graphs from Excel spreasheet, Vertical deflection at Midspan response for Slab A and Slab B (Slab C1)

H.1 Vertical deflection at midspan response for Slab A

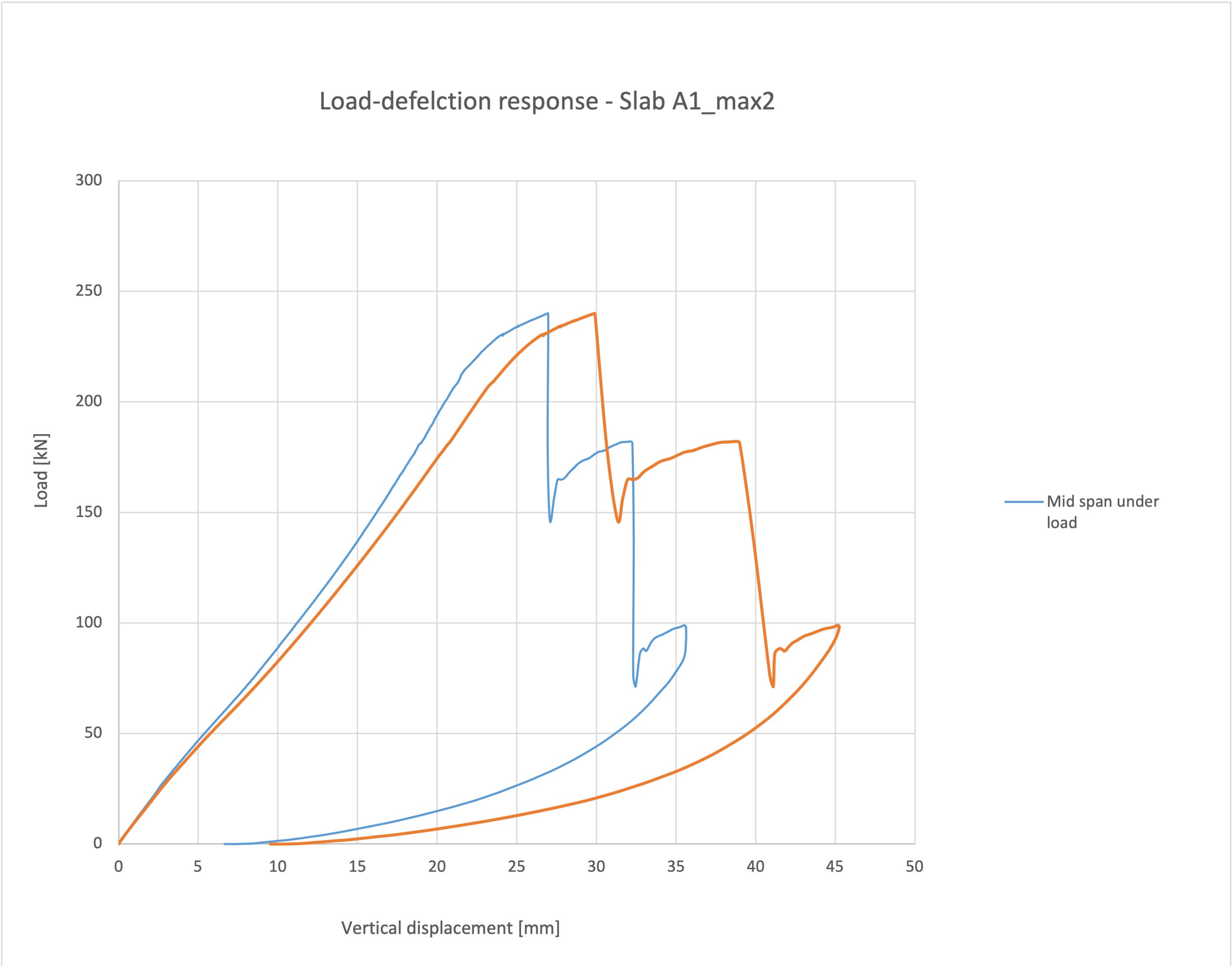
H.2 Vertical deflection at midspan response for Slab B + Slab C1

Appendix H.1 Vertical deflection at midspan response for Slab A

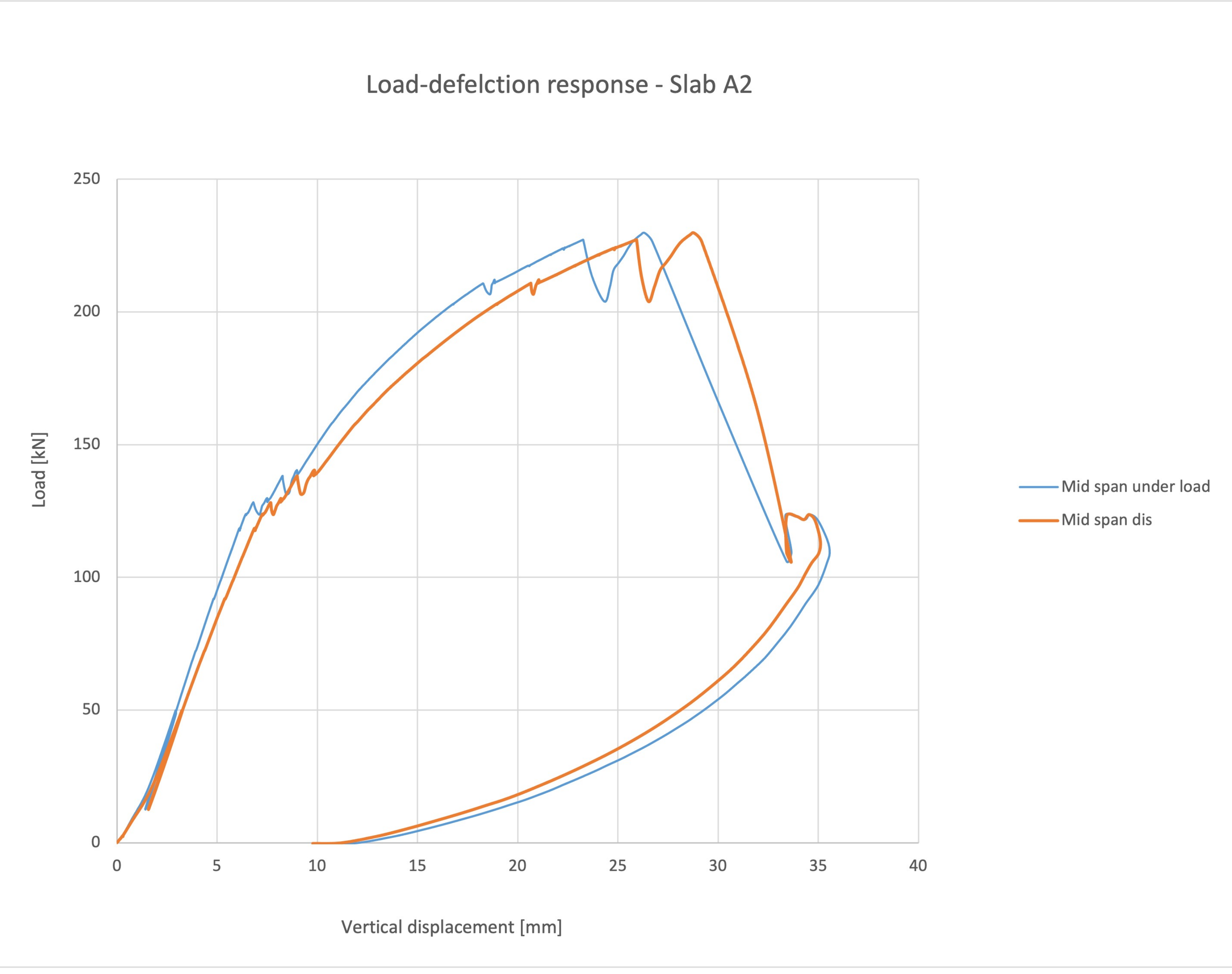




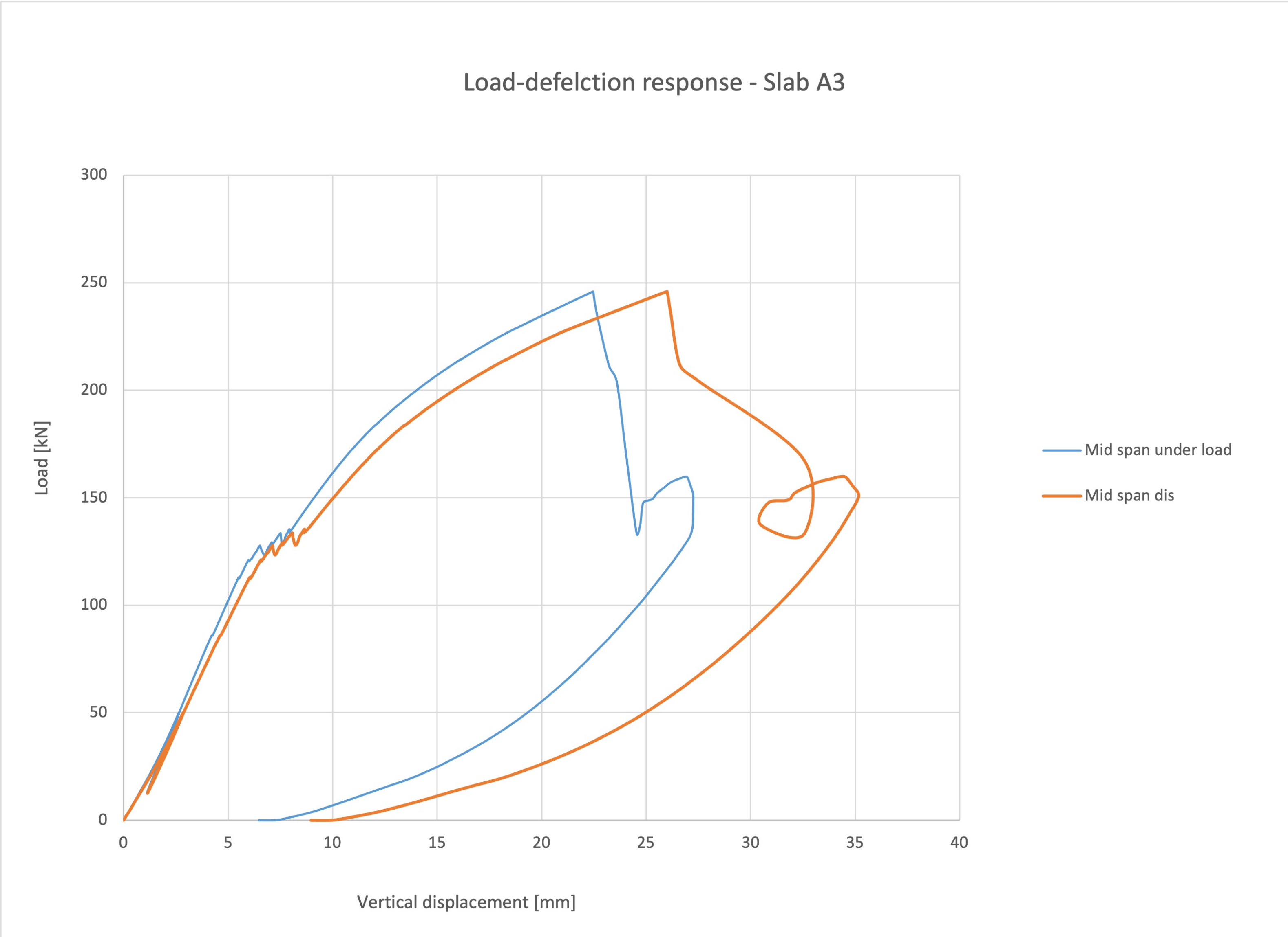
Load-defelction response - Slab A1_max1
250
200
150
100
50
0
Load [kN]
0
5
10
15
20
25
30
Vertical displacement [mm]
Mid span uder load
Mid span dis



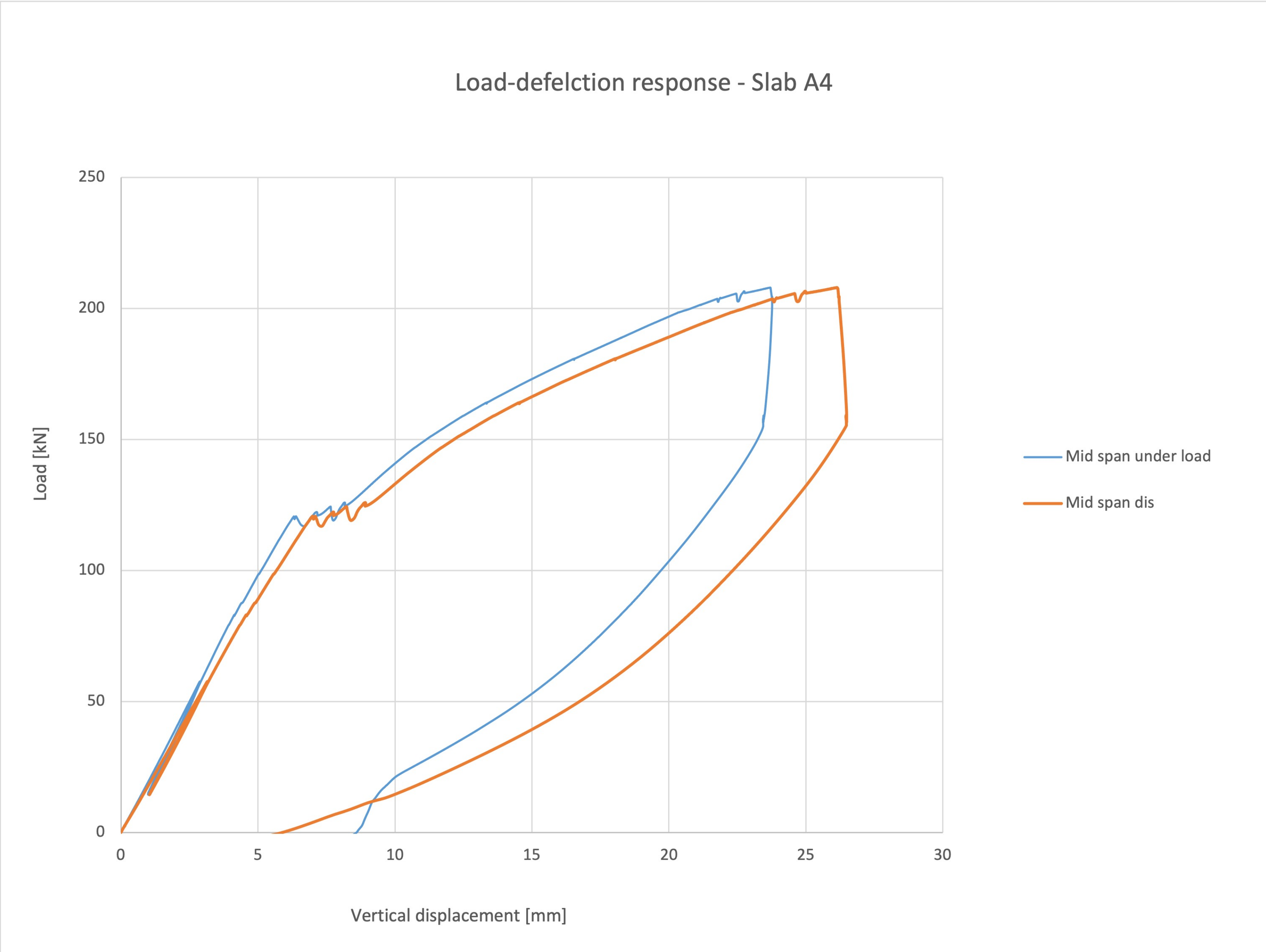
Load-defelction response - Slab A1_max2
Load [kN]
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
45
50
Vertical displacement [mm]
Mid span under load

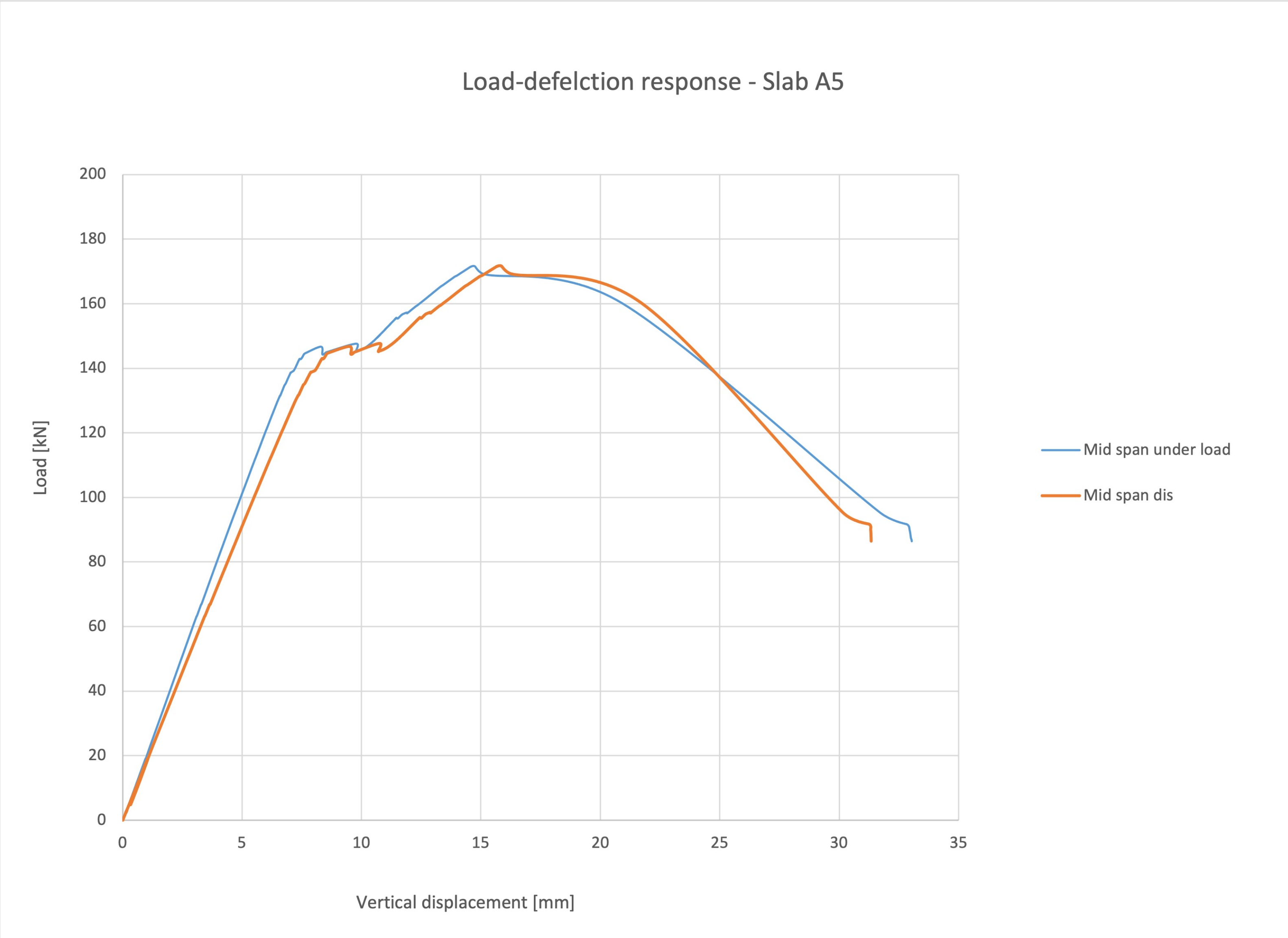


Load-defelction response - Slab A2
Load [kN]
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
Vertical displacement [mm]
Mid span under load
Mid span dis

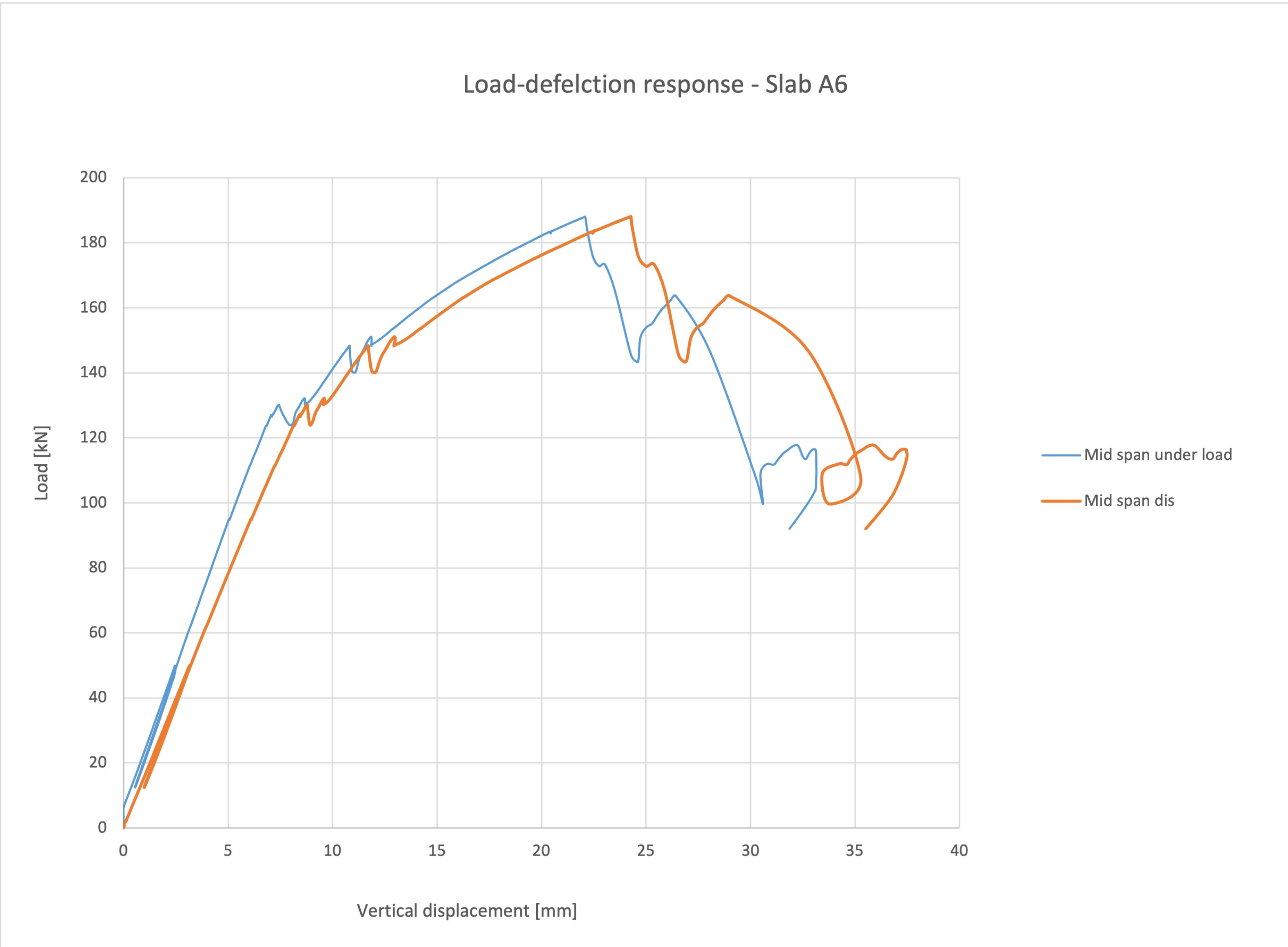


Load-defelction response - Slab A3
300
250
200
150
100
50
0
Load [kN]
0
5
10
15
20
25
30
35
40
Vertical displacement [mm]
Mid span under load
Mid span dis

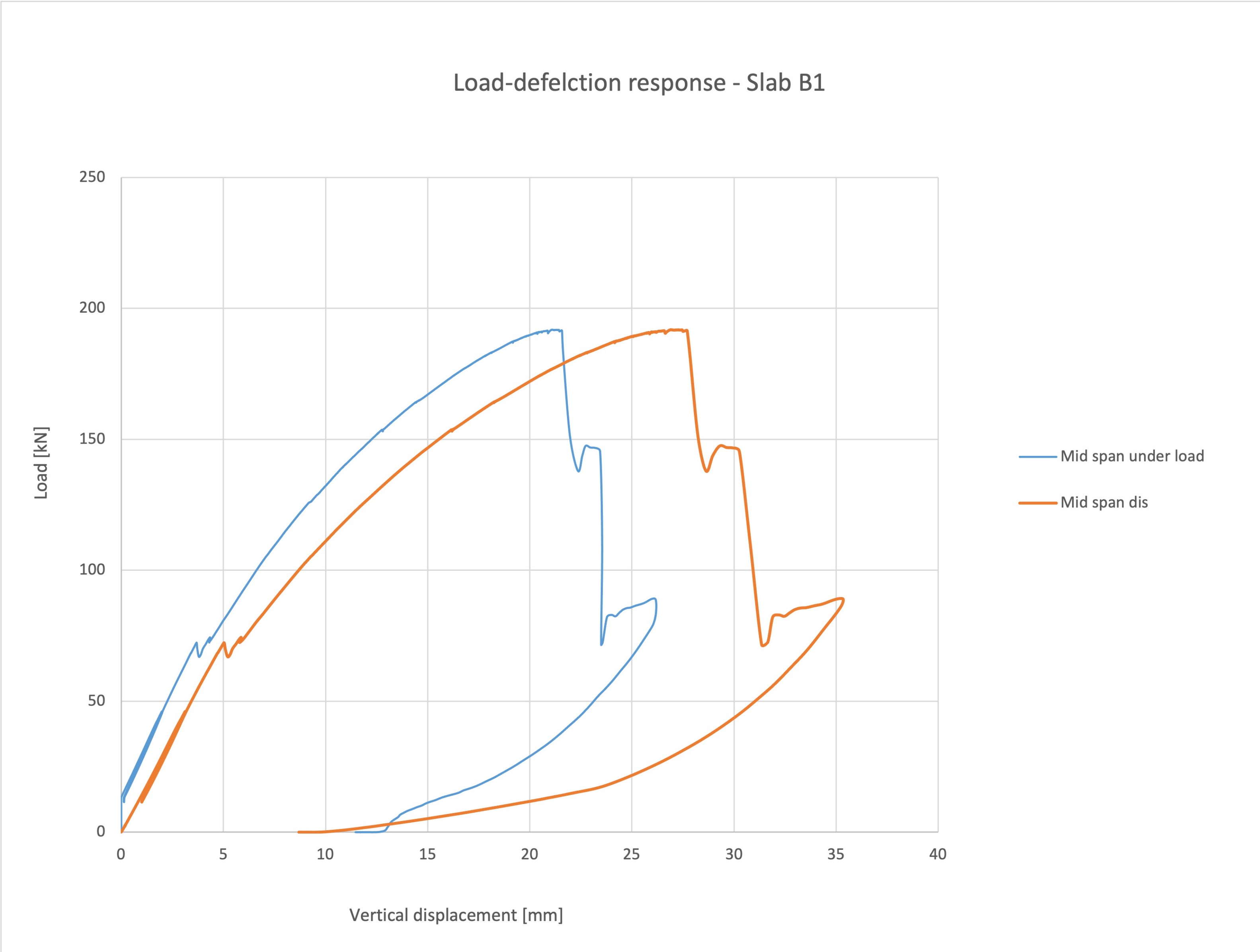

Load-defelction response - Slab A4
Load [kN]
250
200
150
100
50
0
0
5
10
15
20
25
30
Vertical displacement [mm]
Mid span under load
Mid span dis

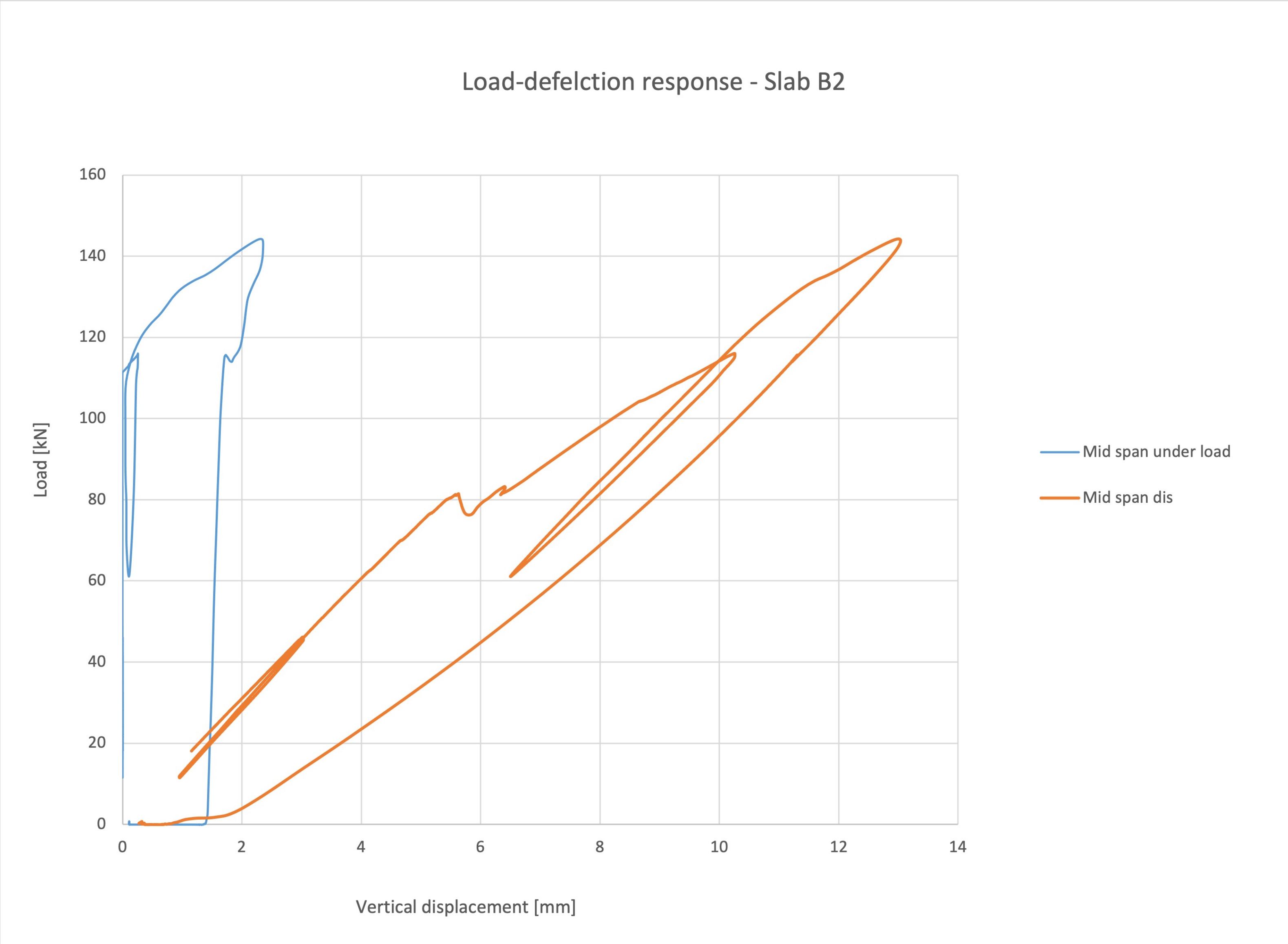


Load-defelction response - Slab A5
200
180
160
140
120
100
80
60
40
20
0
Load [kN]
0
5
10
15
20
25
30
35
Vertical displacement [mm]
Mid span under load
Mid span dis

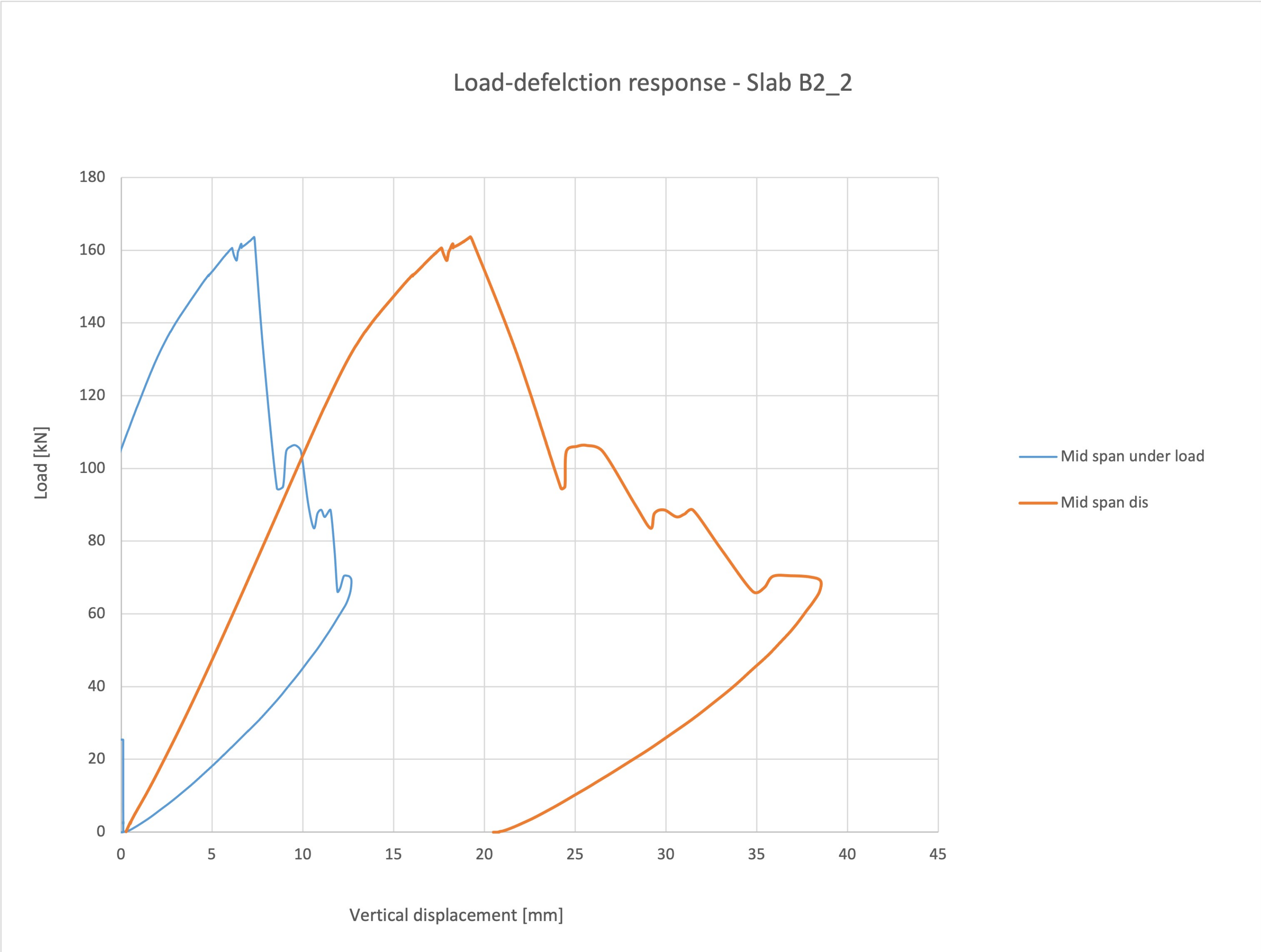

Load-defelction response - Slab A6
Load [kN]
Vertical displacement [mm]
0
20
40
60
80
100
120
140
160
180
200
0
5
10
15
20
25
30
35
40
Mid span under load
Mid span dis

Appendix H.2 Vertical deflection at midspan response for Slab B + Slab C1

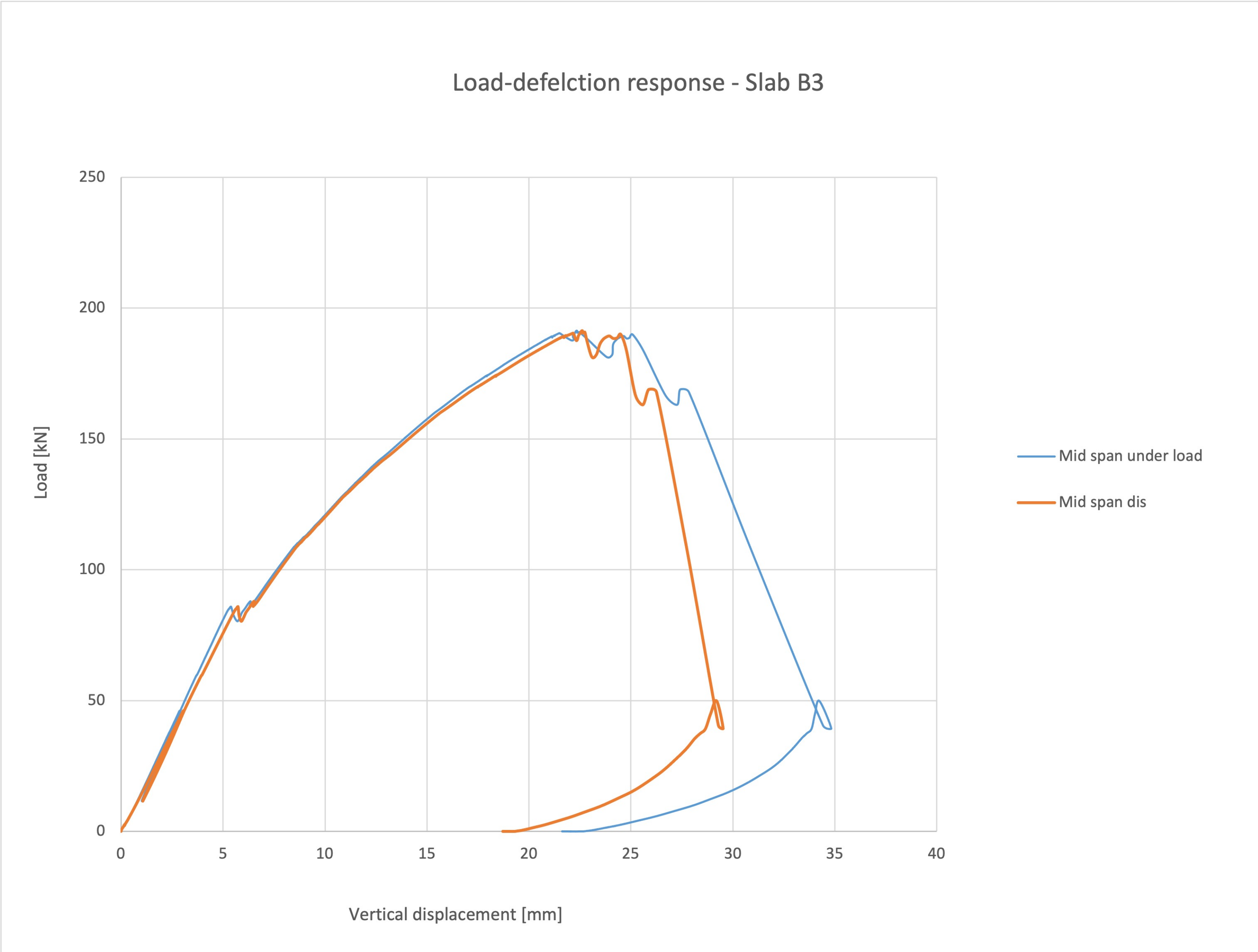


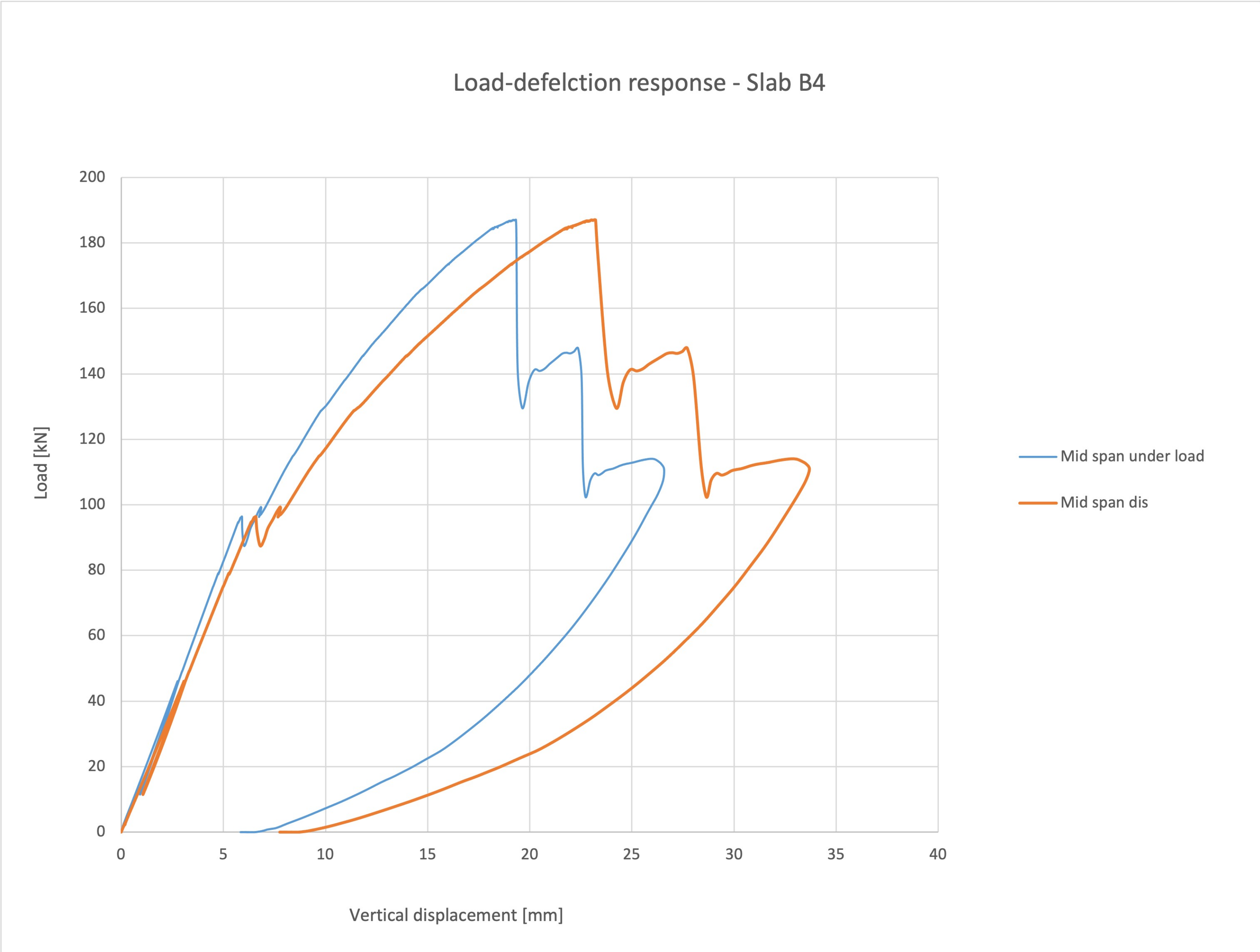


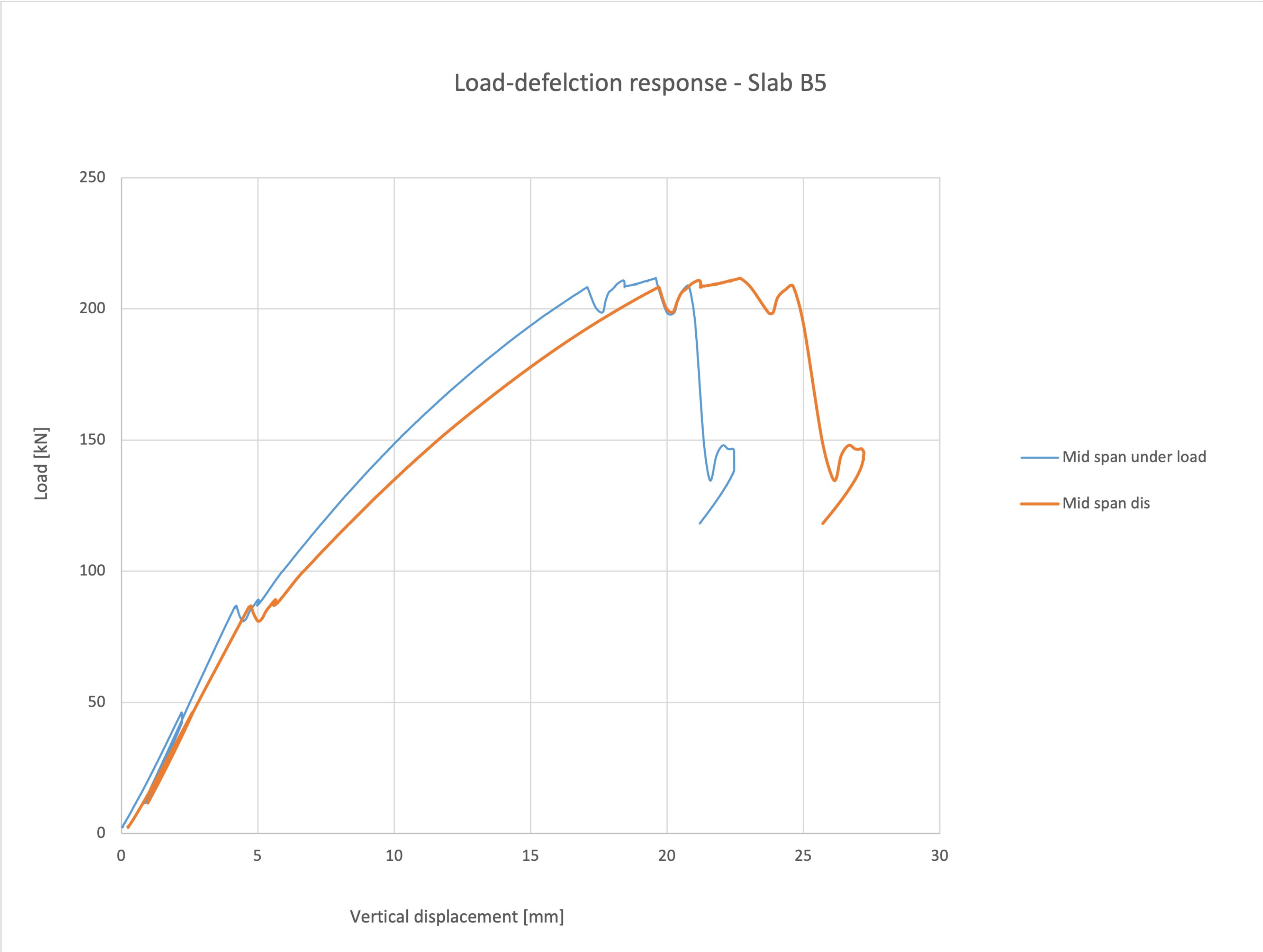
Load-defelction response - Slab B2
Load [kN]
Vertical displacement [mm]
0
20
40
60
80
100
120
140
160
0
2
4
6
8
10
12
14
Mid span under load
Mid span dis



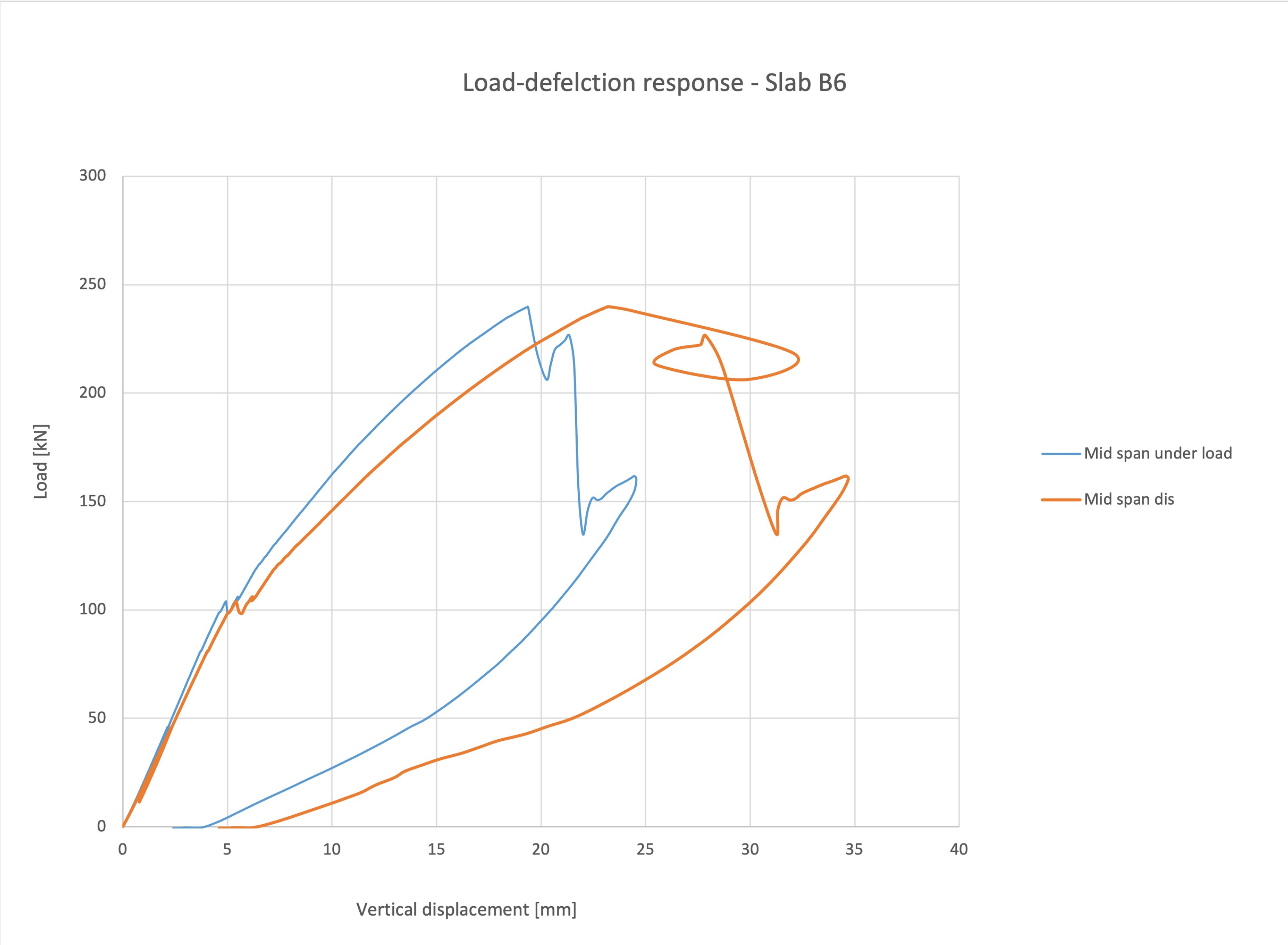
Load-defelction response - Slab B2_2
Load [kN]
180
160
140
120
100
80
60
40
20
0
0
5
10
15
20
25
30
35
40
45
Vertical displacement [mm]
Mid span under load
Mid span dis

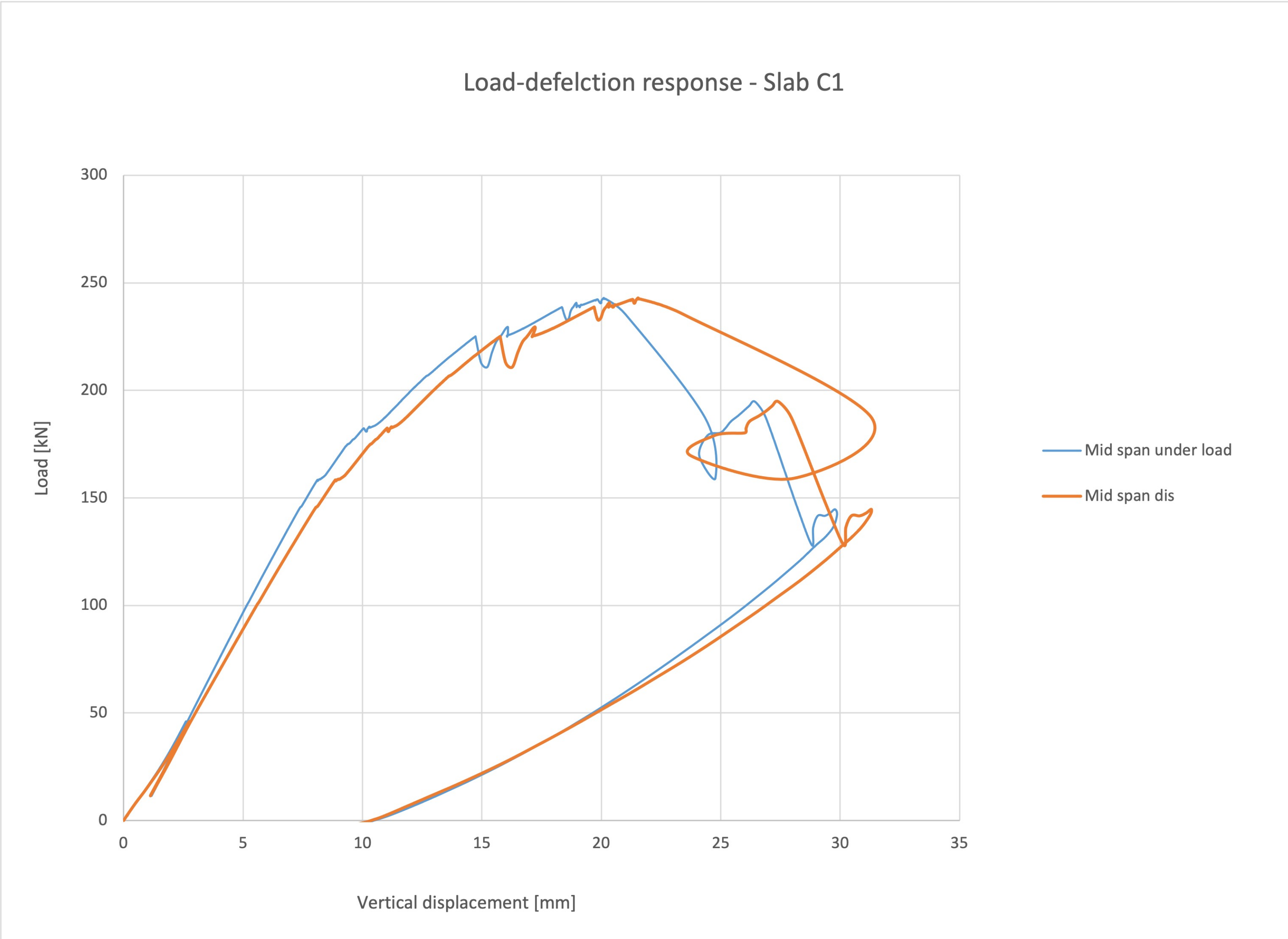

Load-defelction response - Slab B3
Load [kN]
250
200
150
100
50
0
0
5
10
15
20
25
30
35
40
Vertical displacement [mm]
Mid span under load
Mid span dis


Load-defelction response - Slab B4
Load [kN]
200
180
160
140
120
100
80
60
40
20
0
0
5
10
15
20
25
30
35
40
Vertical displacement [mm]
Mid span under load
Mid span dis



Load-defelction response - Slab B5
Load [kN]
250
200
150
100
50
0
0
5
10
15
20
25
30
Vertical displacement [mm]
Mid span under load
Mid span dis


Load-defelction response - Slab B6
Load [kN]
Vertical displacement [mm]
0
50
100
150
200
250
300
0
5
10
15
20
25
30
35
40
Mid span under load
Mid span dis



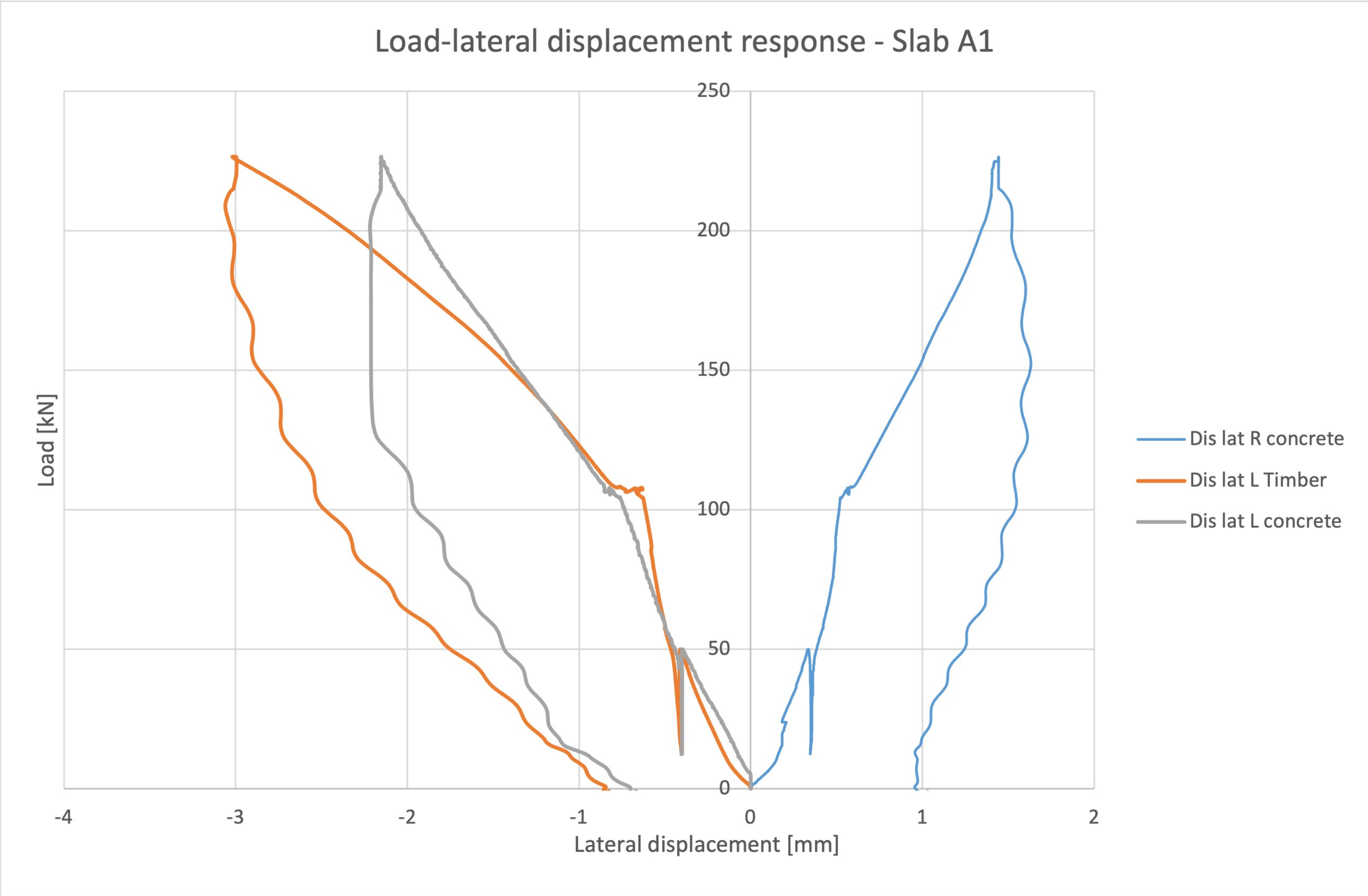
Load-defelction response - Slab C1
Load [kN]
300
250
200
150
100
50
0
0
5
10
15
20
25
30
35
Vertical displacement [mm]
Mid span under load
Mid span dis

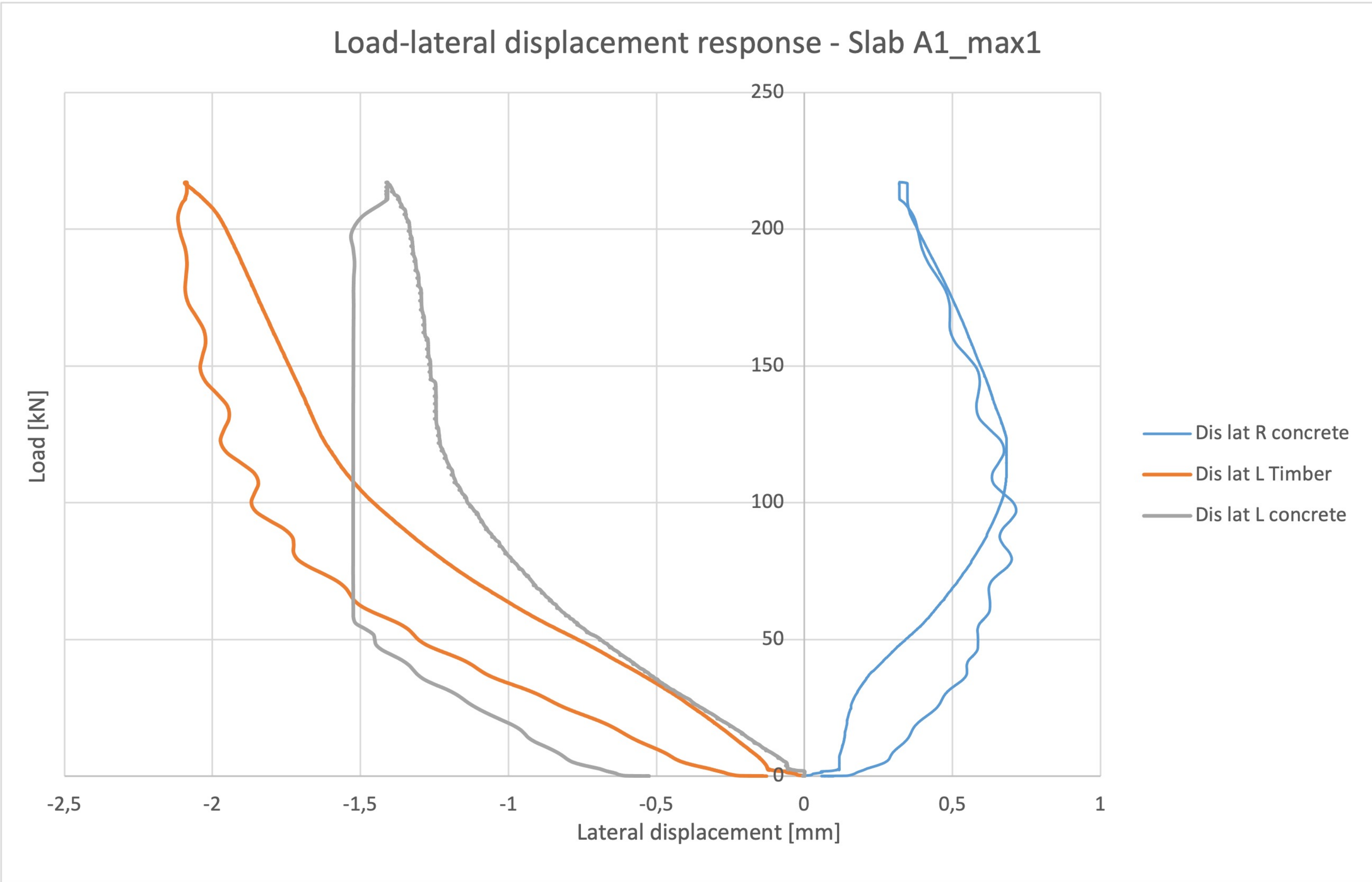
Appendix I. Graphs from Excel spreadsheet, Lateral displacement response for Slab A and Slab B

I.1 Lateral displacement responses for Slab A

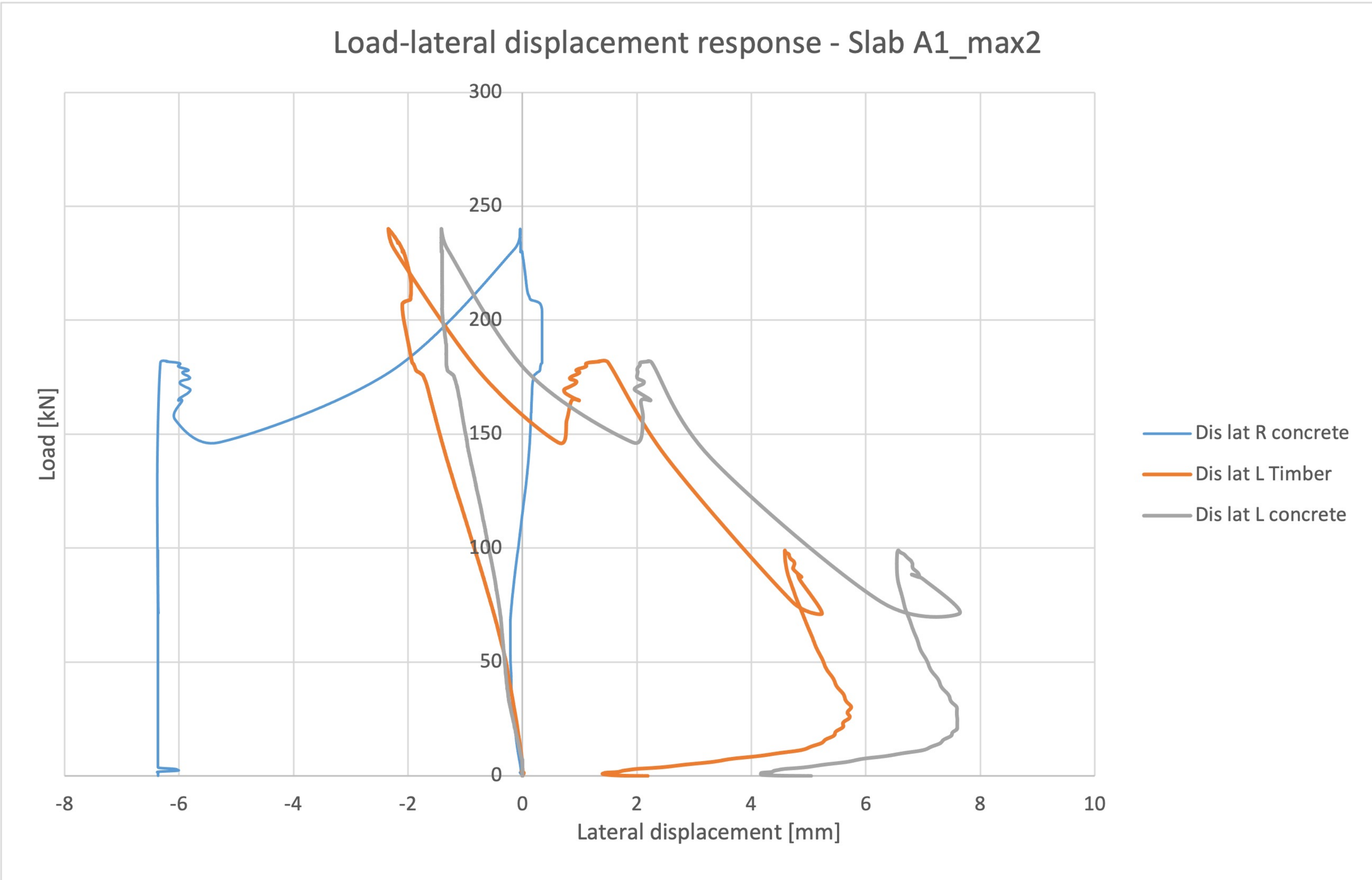
I.2 Lateral displacement responses for Slab B + Slab C1

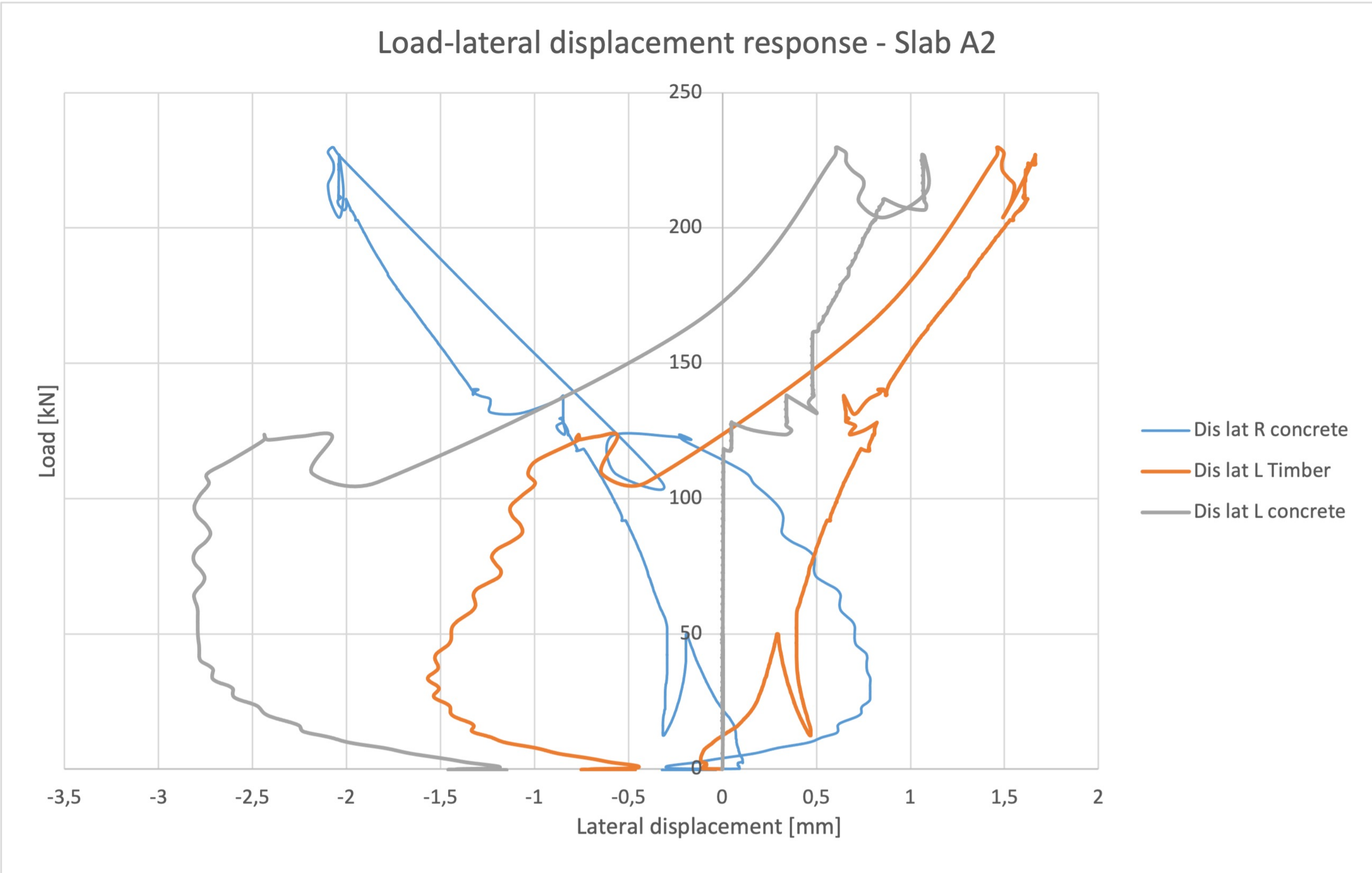
Appendix I.1 Lateral displacement responses for Slab A



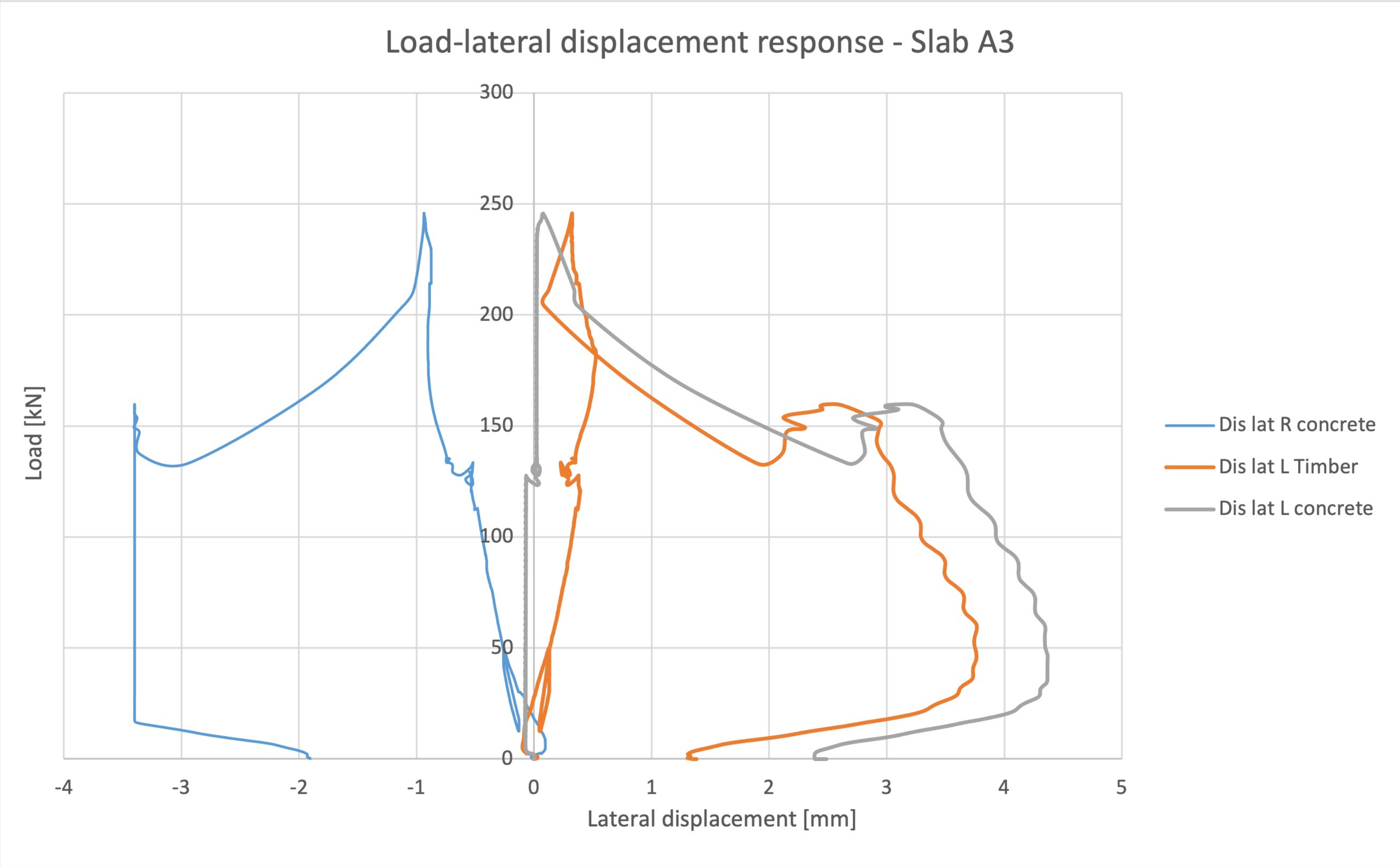


Load-lateral displacement response - Slab A1_max1
250
200
150
100
50
0
Load [kN]
-2,5
-2
-1,5
-1
-0,5
0
0,5
1
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete

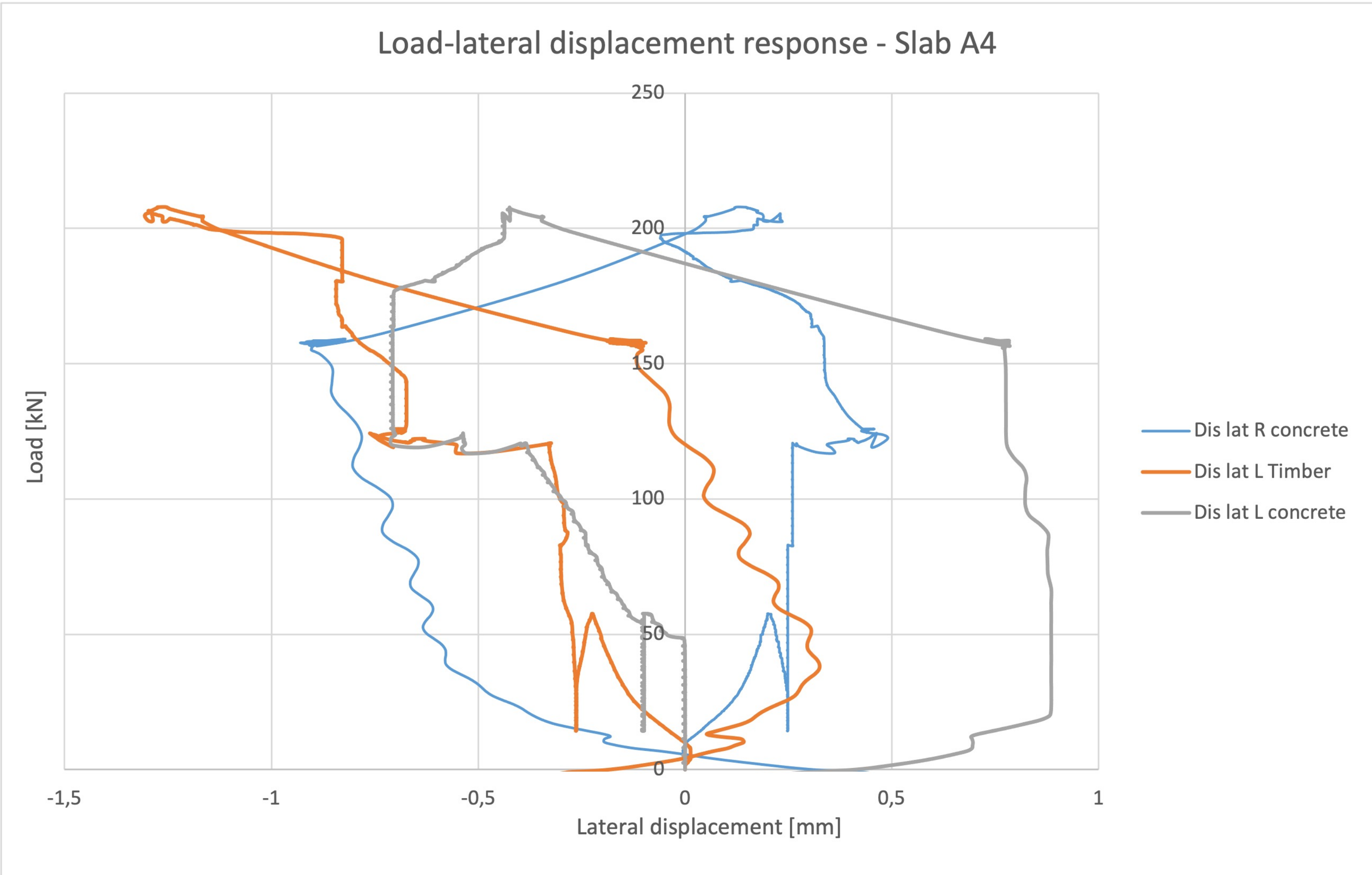

Load-lateral displacement response - Slab A1_max2
300
250
200
150
100
50
0
-8
-6
-4
-2
0
2
4
6
8
10
Lateral displacement [mm]
Load [kN]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



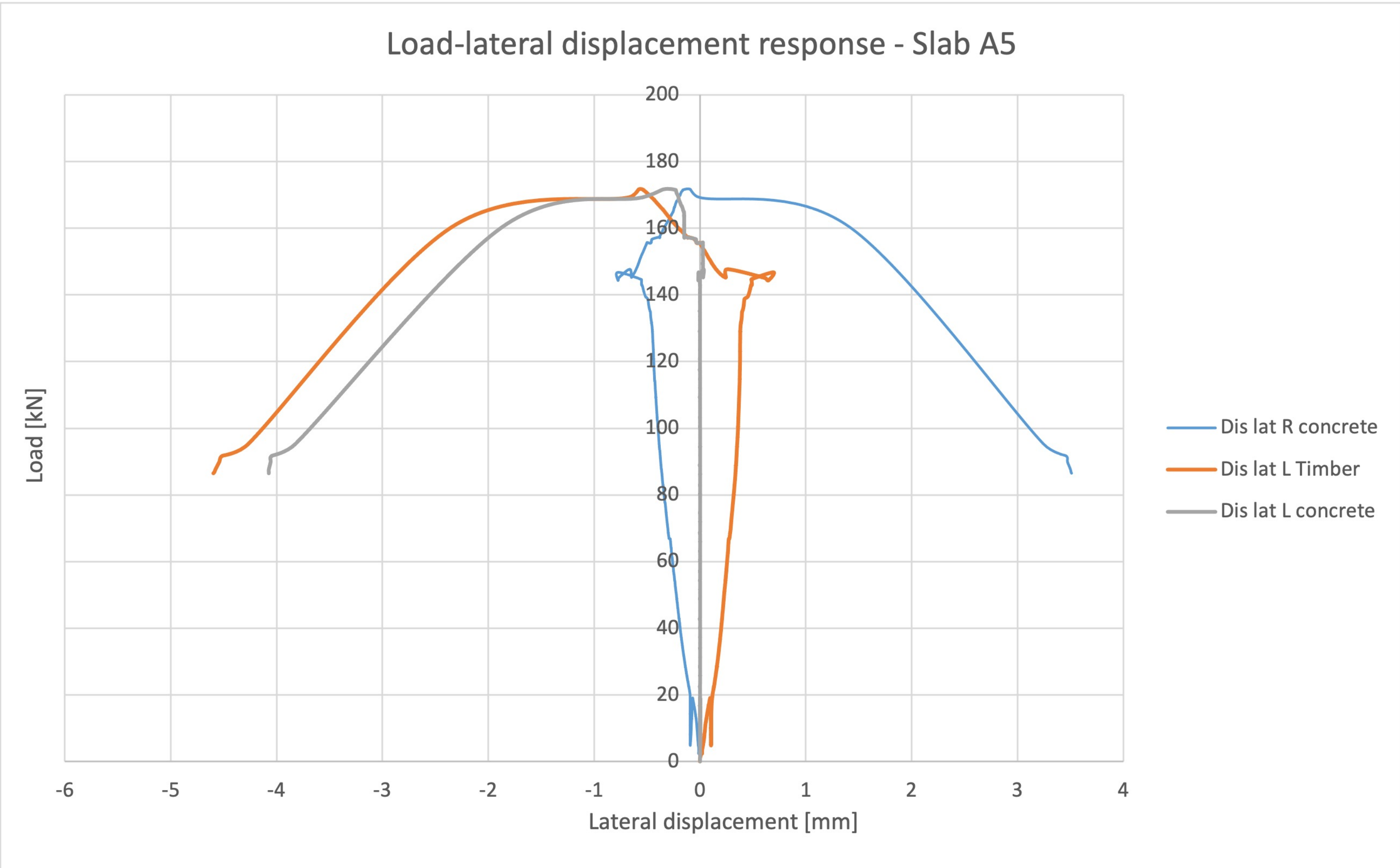
Load-lateral displacement response - Slab A2
250
200
150
100
50
0
Load [kN]
-3,5
-3
-2,5
-2
-1,5
-1
-0,5
0
0,5
1
1,5
2
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



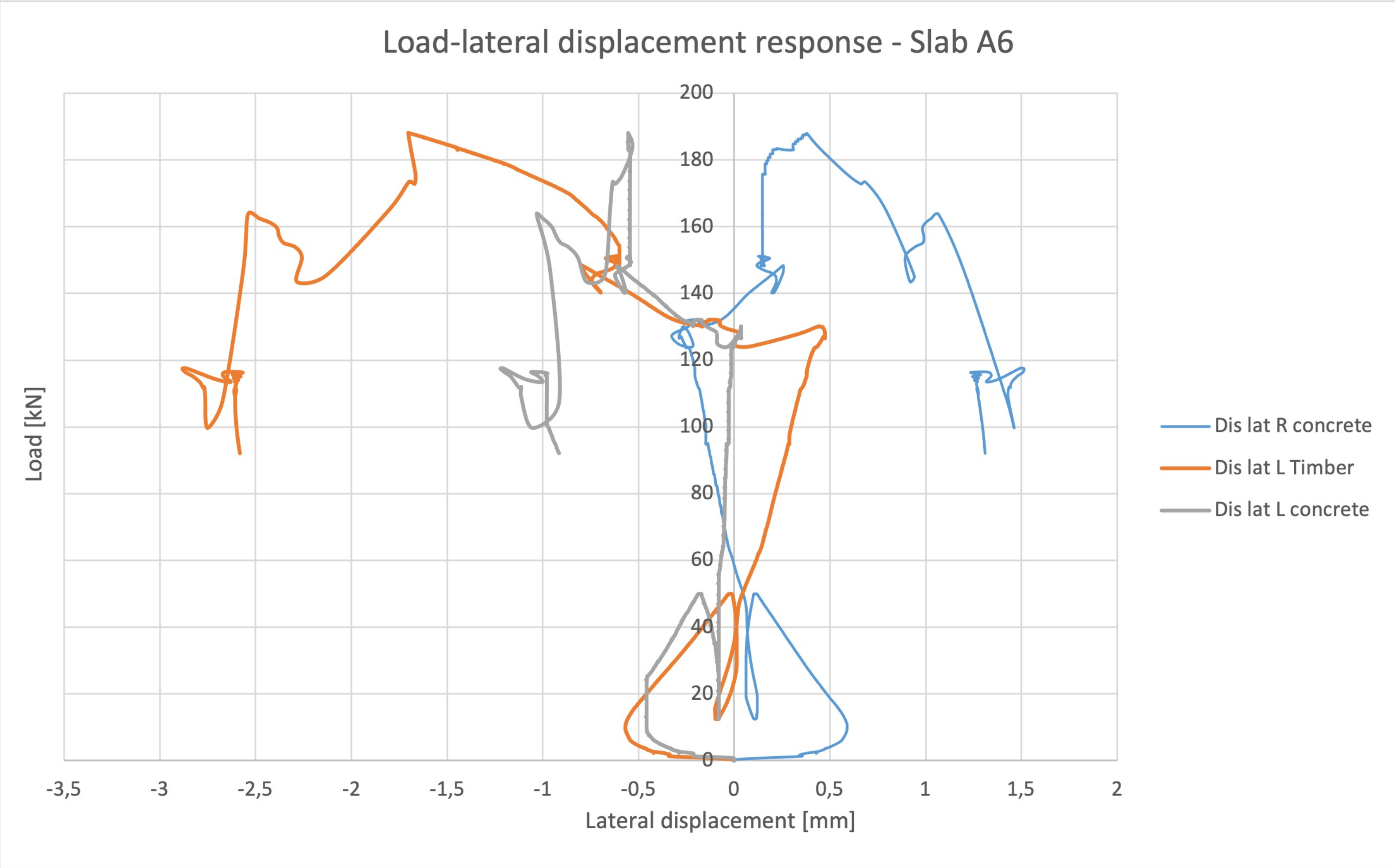
Load-lateral displacement response - Slab A3
300
250
200
150
100
50
0
-4
-3
-2
-1
0
1
2
3
4
5
Load [kN]
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



Load-lateral displacement response - Slab A4
250
200
150
100
50
0
Load [kN]
-1,5
-1
-0,5
0
0,5
1
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete

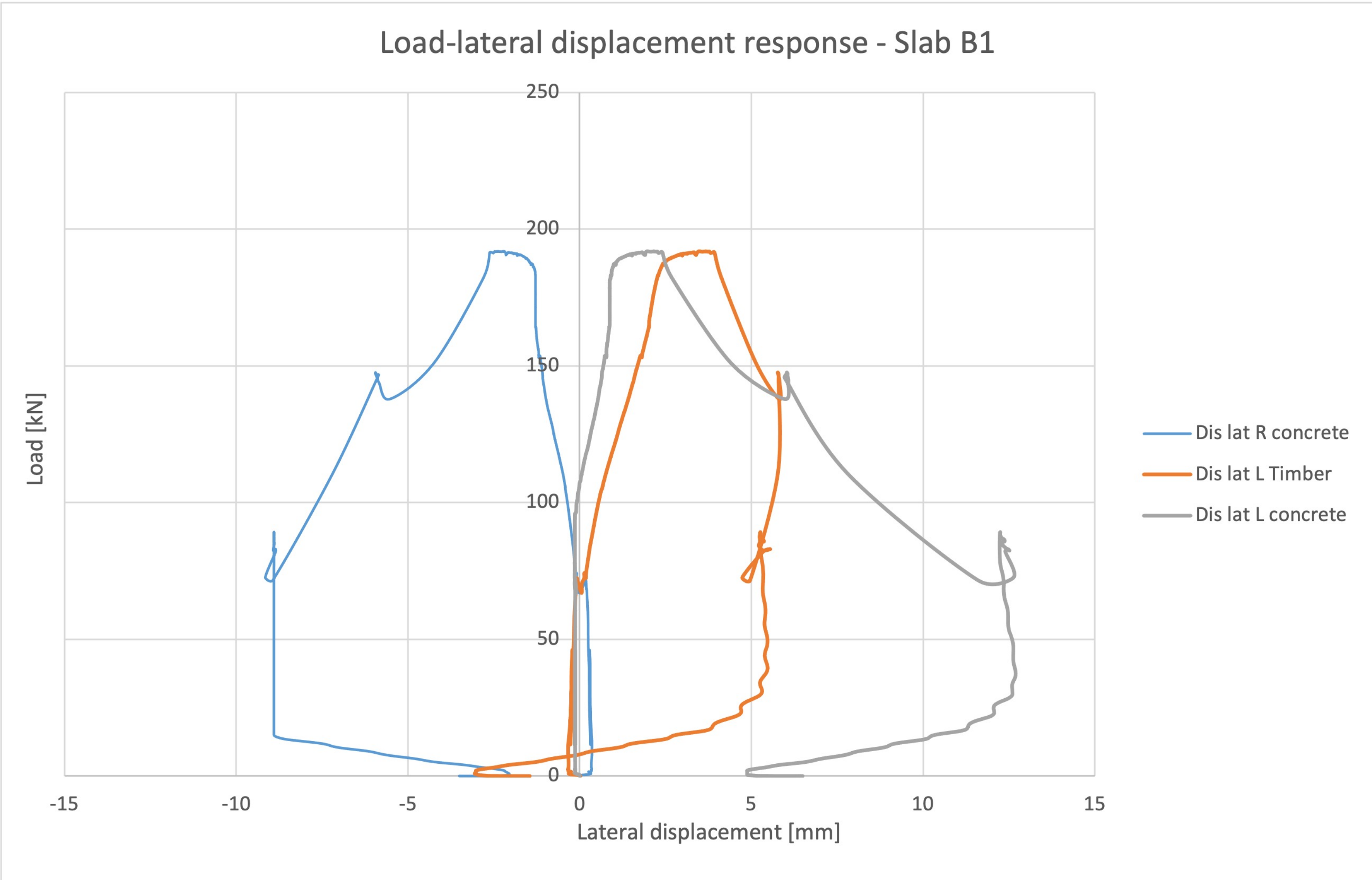


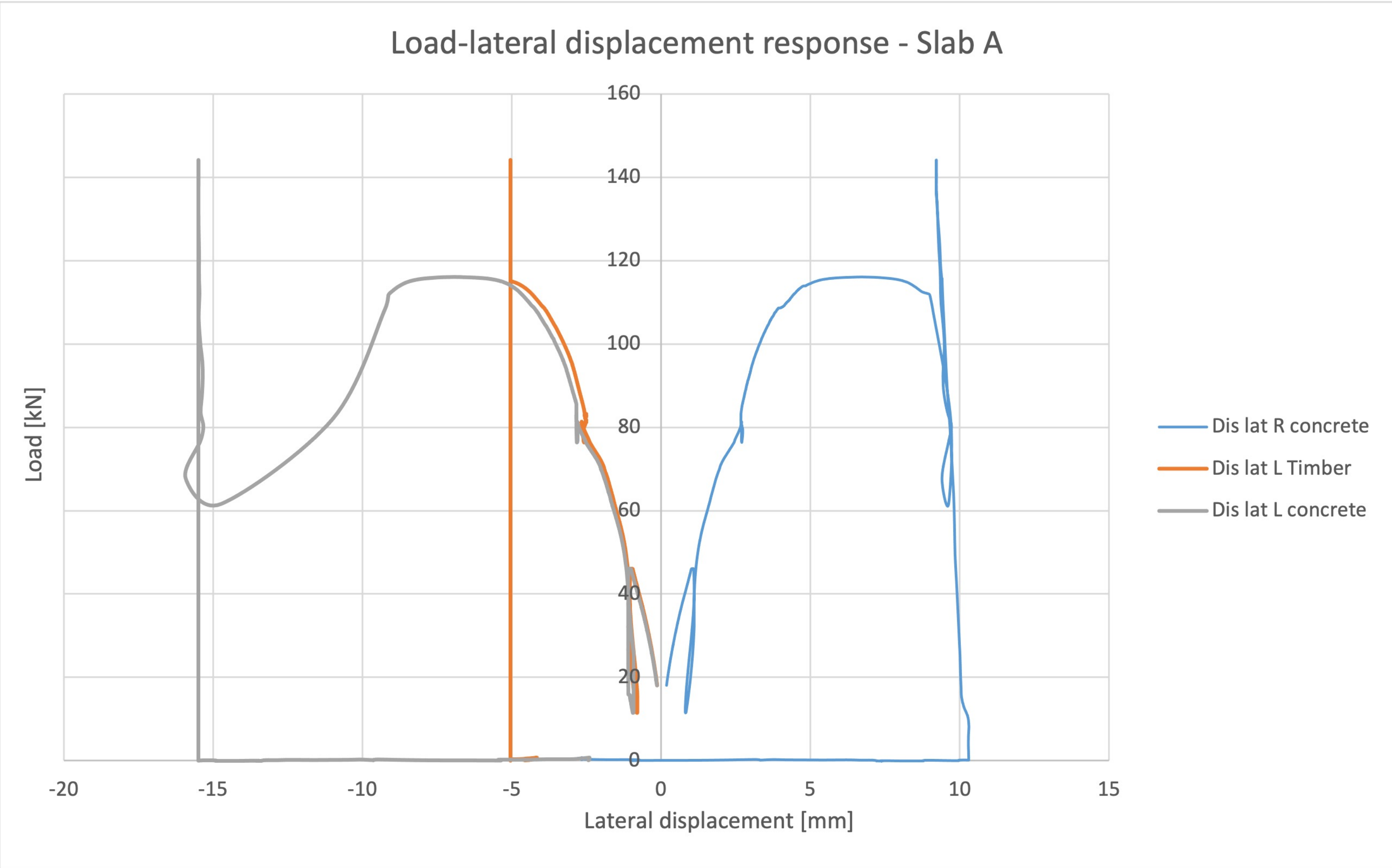
Load-lateral displacement response - Slab A5
Load [kN]
Lateral displacement [mm]
200
180
160
140
120
100
80
60
40
20
0
-6
-5
-4
-3
-2
-1
0
1
2
3
4
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



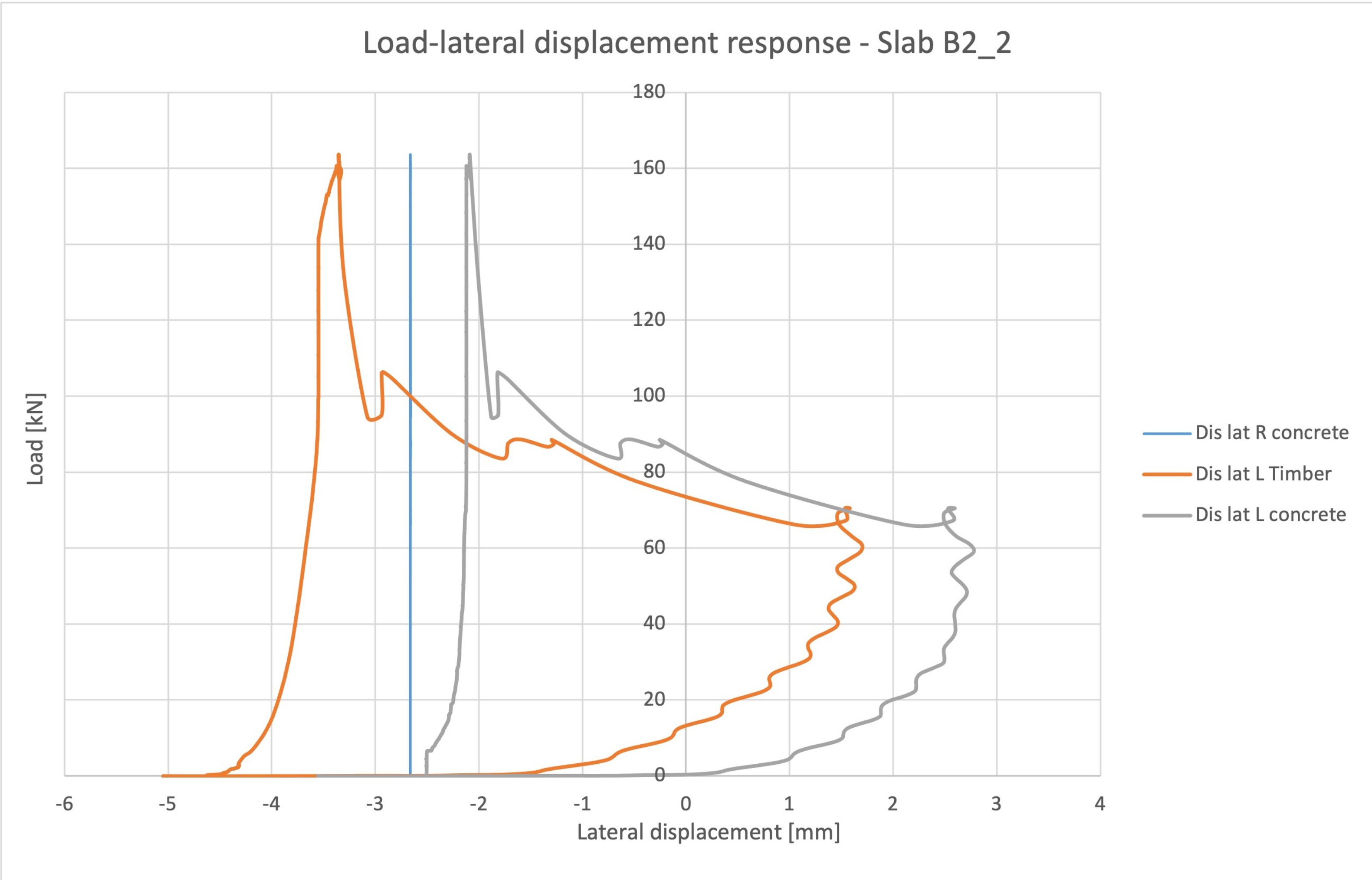
Load-lateral displacement response - Slab A6
200
180
160
140
120
100
80
60
40
20
0
-3,5
-3
-2,5
-2
-1,5
-1
-0,5
0
0,5
1
1,5
2
Lateral displacement [mm]
Load [kN]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete

Appendix I.2 Lateral displacement responses for Slab B + Slab C1

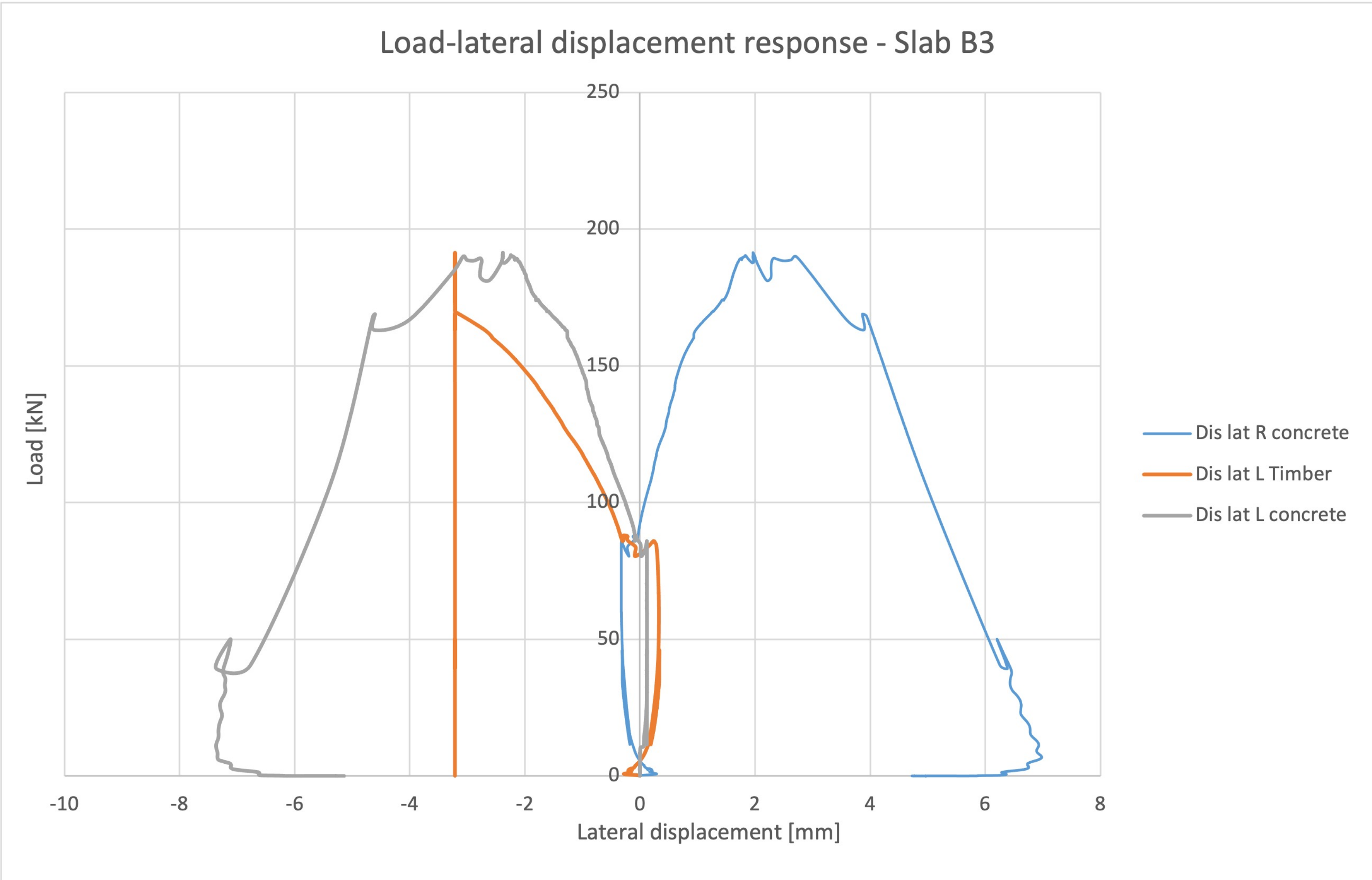




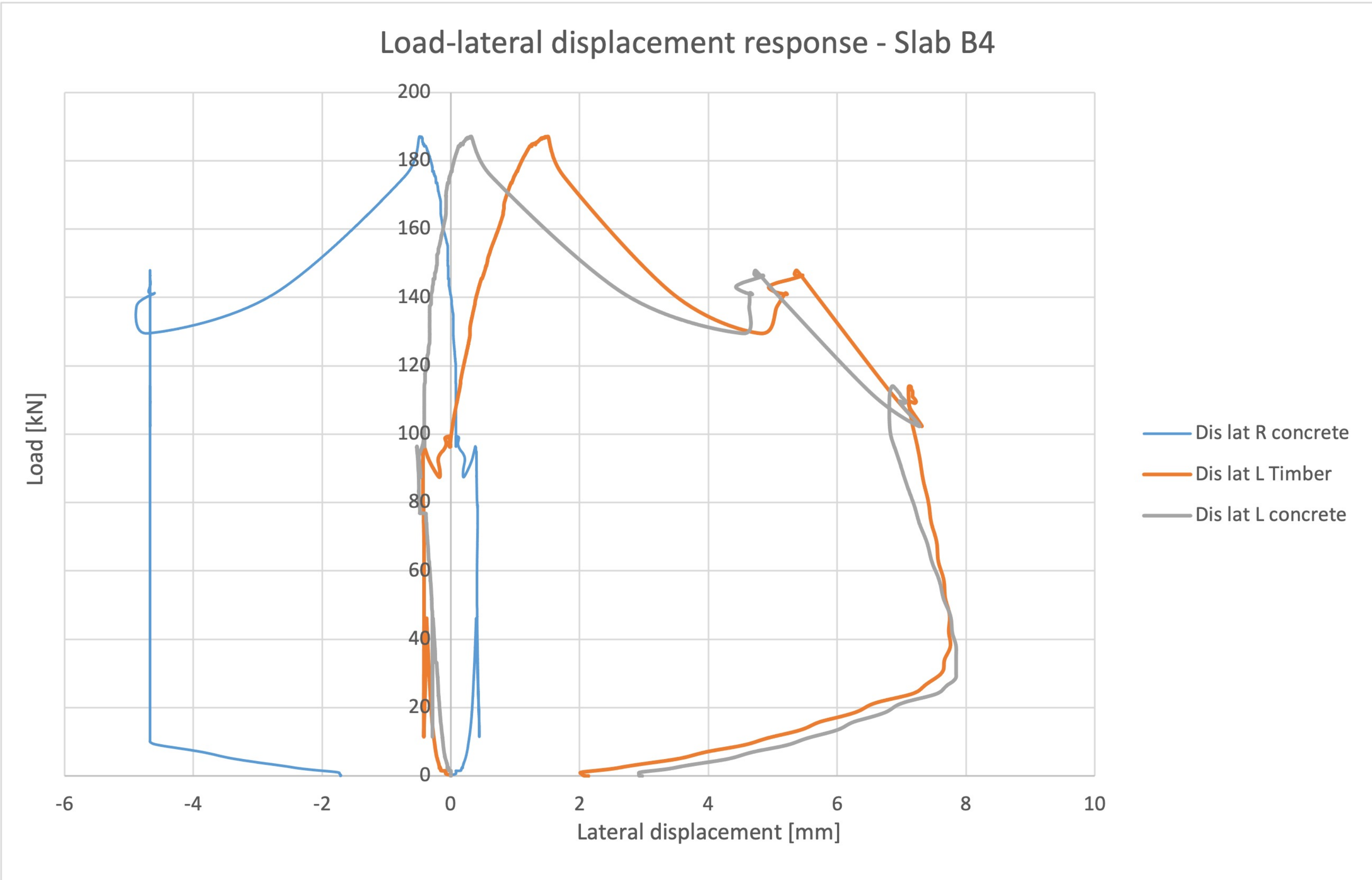
Load-lateral displacement response - Slab A
Load [kN]
160
140
120
100
80
60
40
20
0
-20
-15
-10
-5
0
5
10
15
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



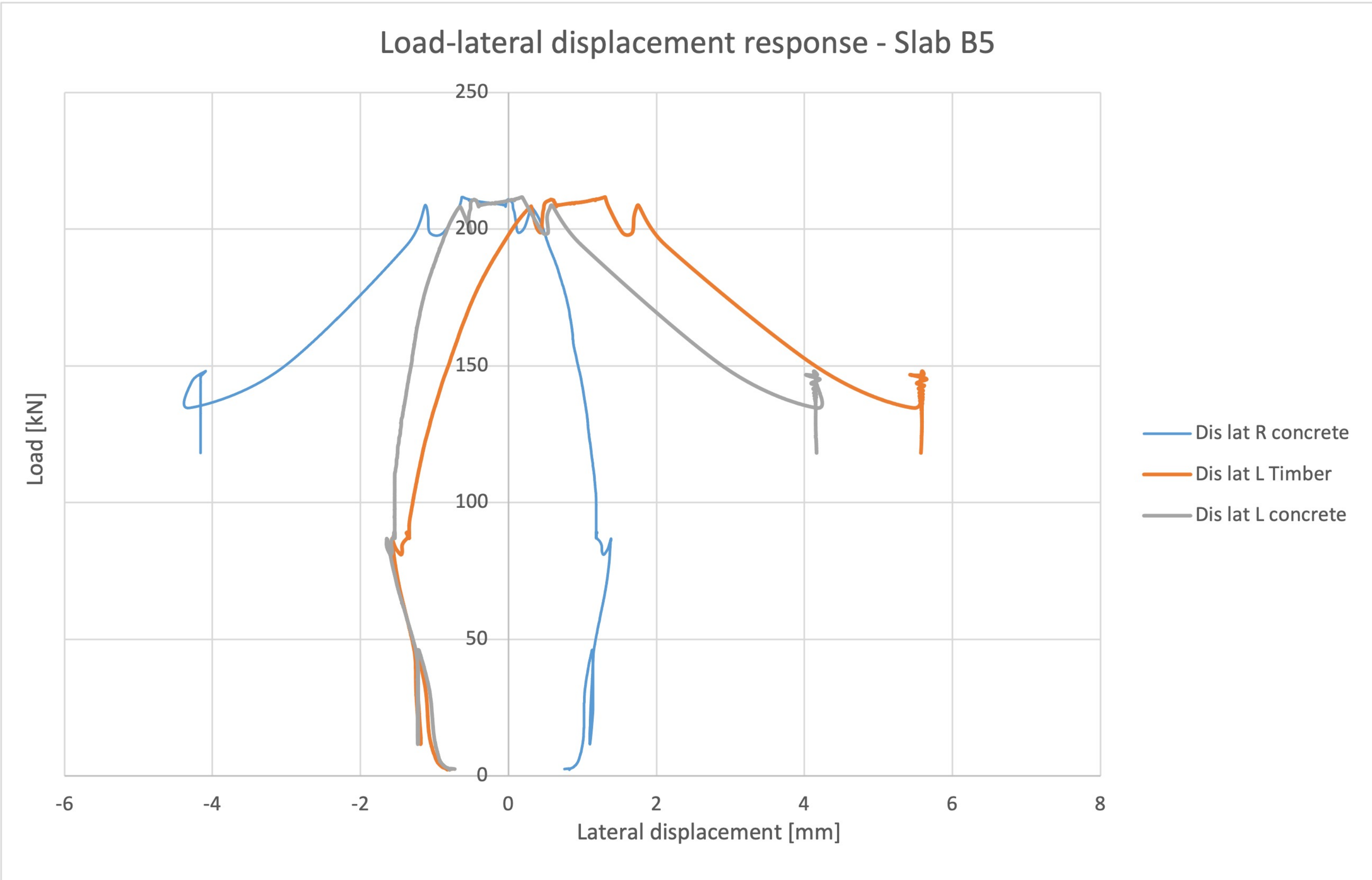
Load-lateral displacement response - Slab B2_2
Load [kN]
Lateral displacement [mm]
180
160
140
120
100
80
60
40
20
0
-6
-5
-4
-3
-2
-1
0
1
2
3
4
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



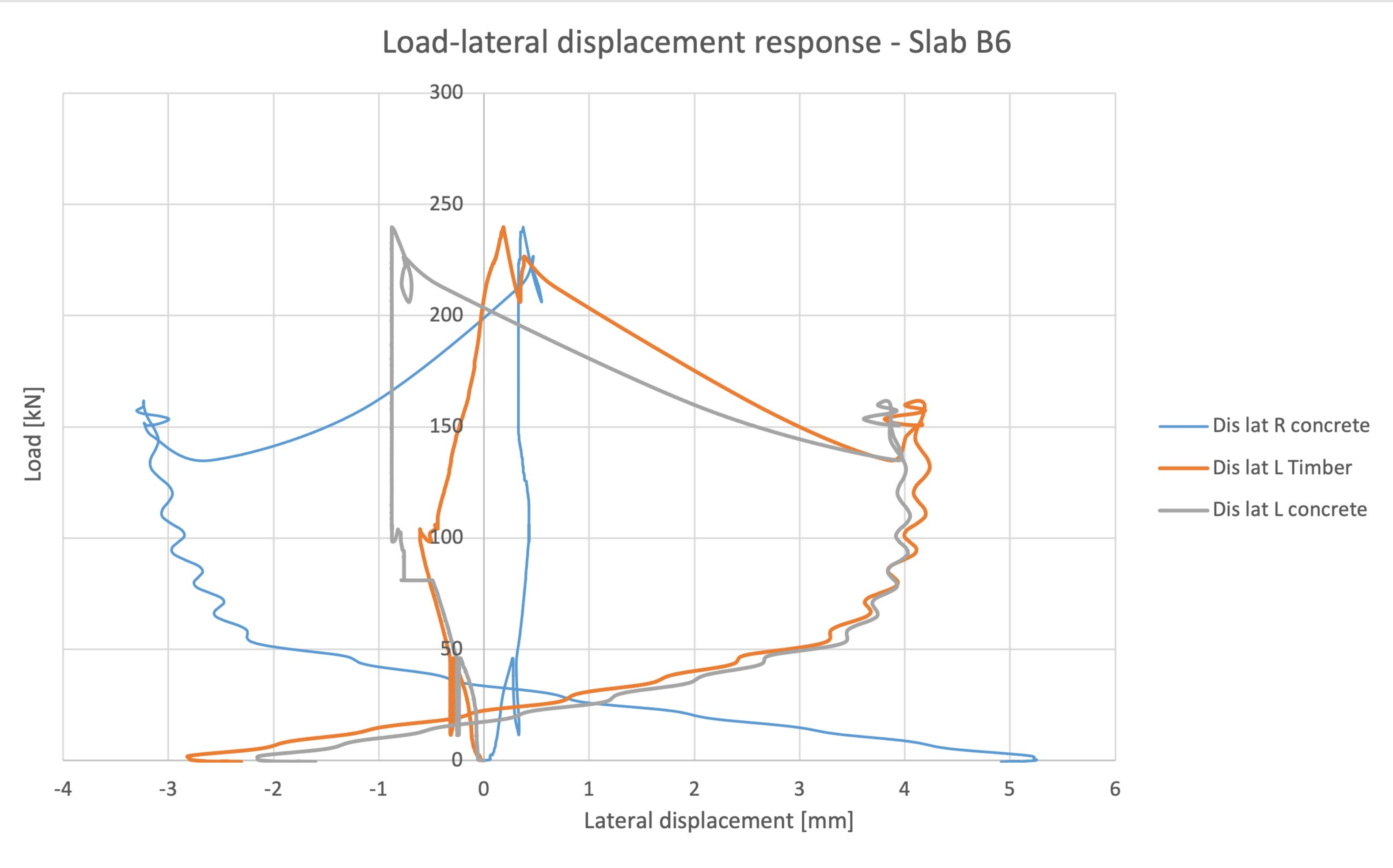
Load-lateral displacement response - Slab B3
Load [kN]
250
200
150
100
50
0
-10
-8
-6
-4
-2
0
2
4
6
8
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete

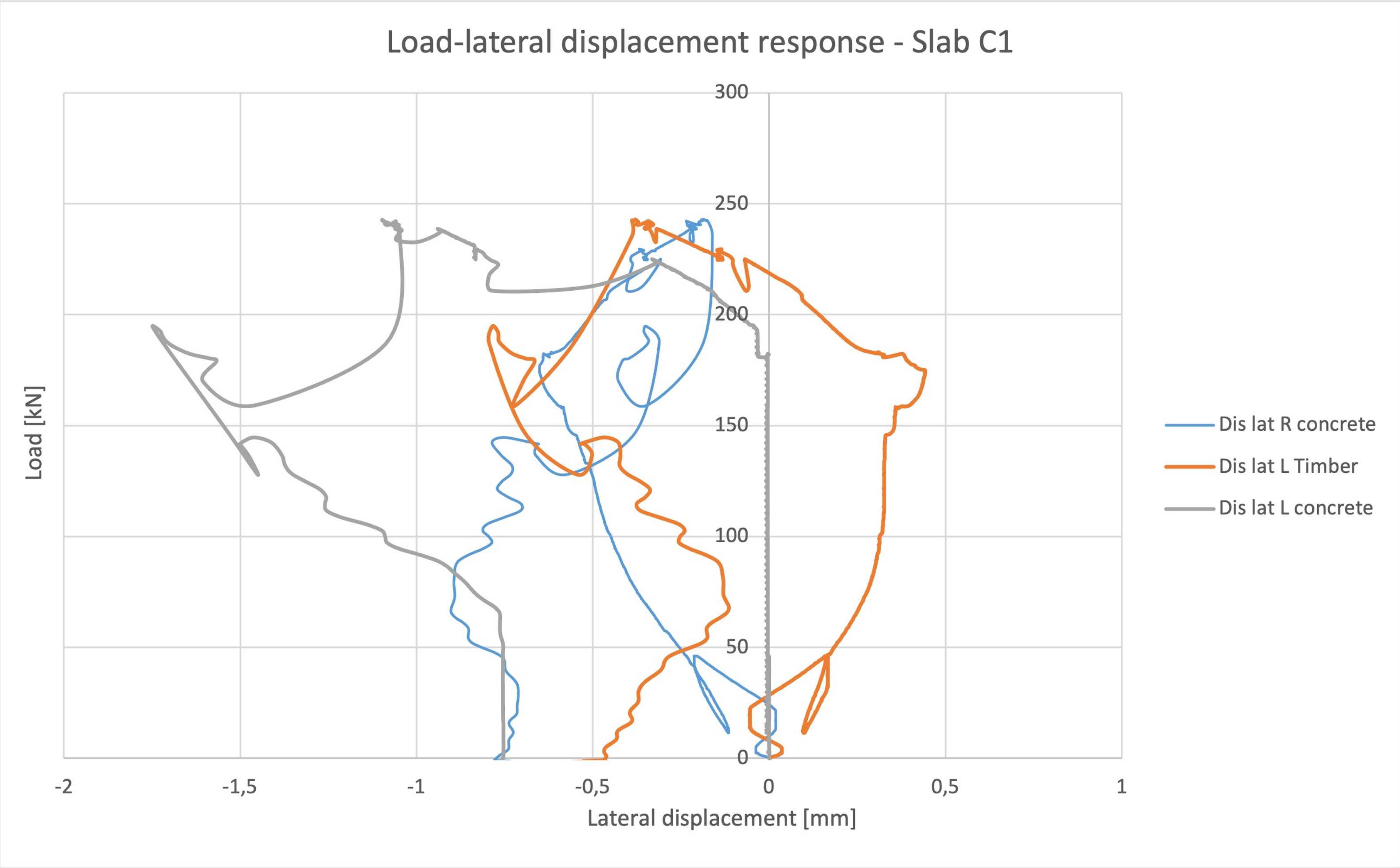


Load-lateral displacement response - Slab B4
Load [kN]
Lateral displacement [mm]
200
180
160
140
120
100
80
60
40
20
0
-6
-4
-2
0
2
4
6
8
10
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



Load-lateral displacement response - Slab B5
250
200
150
100
50
0
-6
-4
-2
0
2
4
6
8
Lateral displacement [mm]
Load [kN]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete


Load-lateral displacement response - Slab B6
Load [kN]
300
250
200
150
100
50
0
-4
-3
-2
-1
0
1
2
3
4
5
6
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete



Load-lateral displacement response - Slab C1
Load [kN]
300
250
200
150
100
50
0
-2
-1,5
-1
-0,5
0
0,5
1
Lateral displacement [mm]
Dis lat R concrete
Dis lat L Timber
Dis lat L concrete

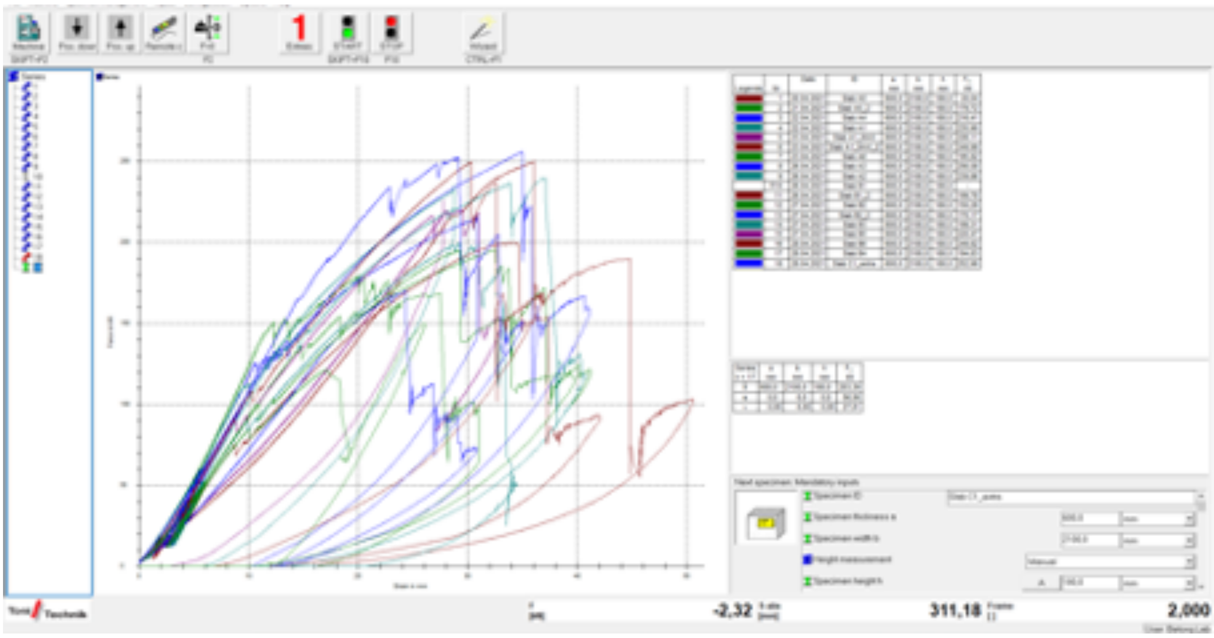
Appendix J. Graphs from the Toni Technik machine

J.1 Graphs from the Toni Technik machine for all test specimens

Appendix J.1 Graphs from the Toni Technik machine for all test specimens

Legends	Nr	Date	ID	a mm	b mm	h mm	F_m kN
	1	20.04.2021	Slab A5	600,0	2100,0	180,0	20,04
	2	21.04.2021	Slab A5_2	600,0	2100,0	180,0	179,72
	3	22.04.2021	Slab A4	600,0	2100,0	180,0	216,41
	4	22.04.2021	Slab A1	600,0	2100,0	180,0	235,90
	5	23.04.2021	Slab A1_MAX	600,0	2100,0	180,0	226,11
	6	23.04.2021	Slab A1_MAX_2	600,0	2100,0	180,0	249,99
	7	23.04.2021	Slab A6	600,0	2100,0	180,0	195,82
	8	26.04.2021	Slab A3	600,0	2100,0	180,0	256,08
	9	26.04.2021	Slab A2	600,0	2100,0	180,0	239,86
	✢10	26.04.2021	Slab B1	600,0	2100,0	180,0	-
	11	26.04.2021	Slab B1_2	600,0	2100,0	180,0	199,78
	12	27.04.2021	Slab B2	600,0	2100,0	180,0	150,28
	13	27.04.2021	Slab B2_2	600,0	2100,0	180,0	170,17
	14	27.04.2021	Slab B3	600,0	2100,0	180,0	199,31
	15	27.04.2021	Slab B5	600,0	2100,0	180,0	220,47
	16	28.04.2021	Slab B6	600,0	2100,0	180,0	249,82
	17	28.04.2021	Slab B4	600,0	2100,0	180,0	194,83
	18	28.04.2021	Slab C1_extra	600,0	2100,0	180,0	252,98

A graph of all test specimens together



Parameter table:

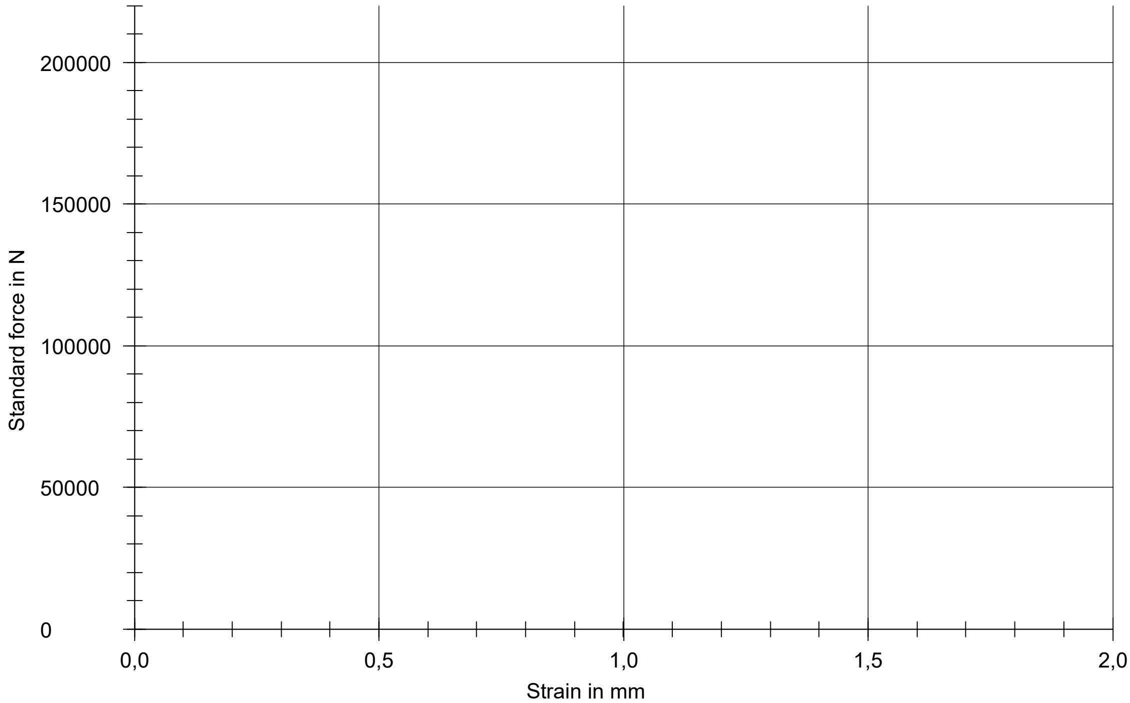
Test protocol : Masterthesis
Tester : Test 1 Capasity
Creation date : 1.12.2015

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN
Extensometer
Extensometer2

Results:

Date	ID	a mm	b mm	h mm	F_m kN
21.04.2021	Slab A5_2	600,0	2100,0	180,0	179,72
22.04.2021	Slab A4	600,0	2100,0	180,0	216,41
22.04.2021	Slab A1	600,0	2100,0	180,0	235,90
23.04.2021	Slab A1_MAX_2	600,0	2100,0	180,0	249,99
23.04.2021	Slab A6	600,0	2100,0	180,0	195,82
26.04.2021	Slab A3	600,0	2100,0	180,0	256,08
26.04.2021	Slab A2	600,0	2100,0	180,0	239,86

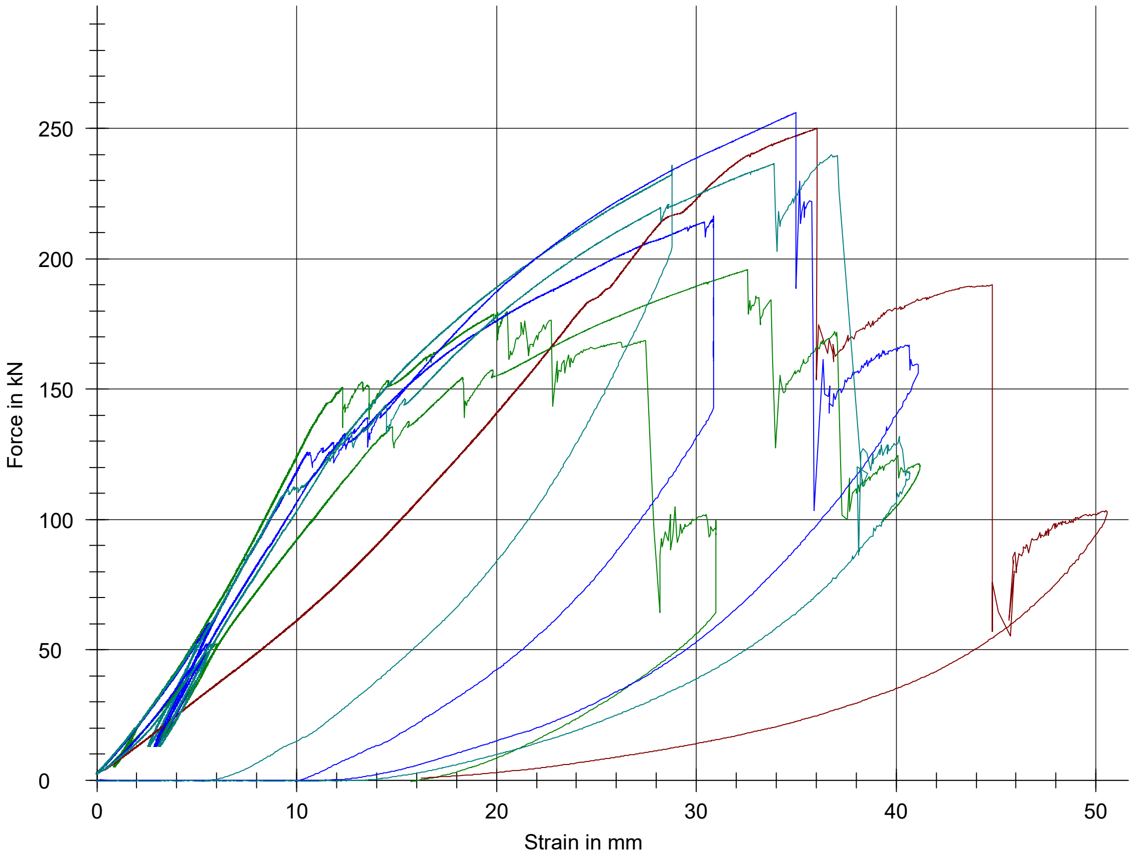
Series graphics:



Statistics:

Series n = 7	a mm	b mm	h mm	F_m kN
$\bar{x}$	600,0	2100,0	180,0	224,83
s	0,0	0,0	0,0	28,59
ν	0,00	0,00	0,00	12,72

Graph of load capacity for Slab A



Parameter table:

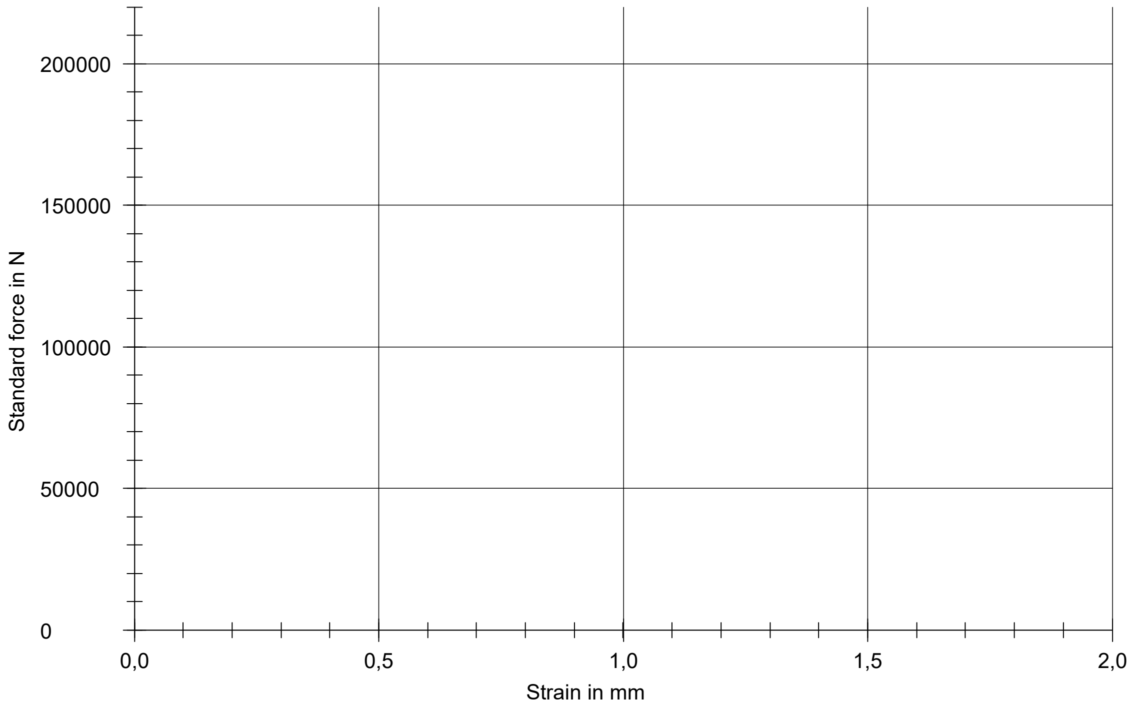
Test protocol : Masterthesis
Tester : Test 1 Capasity
Creation date : 1.12.2015

Type strain extensometer :
Machine data : Controller TT0322
PistonStroke
LoadCell 400 kN
Extensometer
Extensometer2

Results:

Date	ID	a mm	b mm	h mm	F_m kN
26.04.2021	Slab B1_2	600,0	2100,0	180,0	199,78
27.04.2021	Slab B2	600,0	2100,0	180,0	150,28
27.04.2021	Slab B3	600,0	2100,0	180,0	199,31
27.04.2021	Slab B5	600,0	2100,0	180,0	220,47
28.04.2021	Slab B6	600,0	2100,0	180,0	249,82
28.04.2021	Slab B4	600,0	2100,0	180,0	194,83

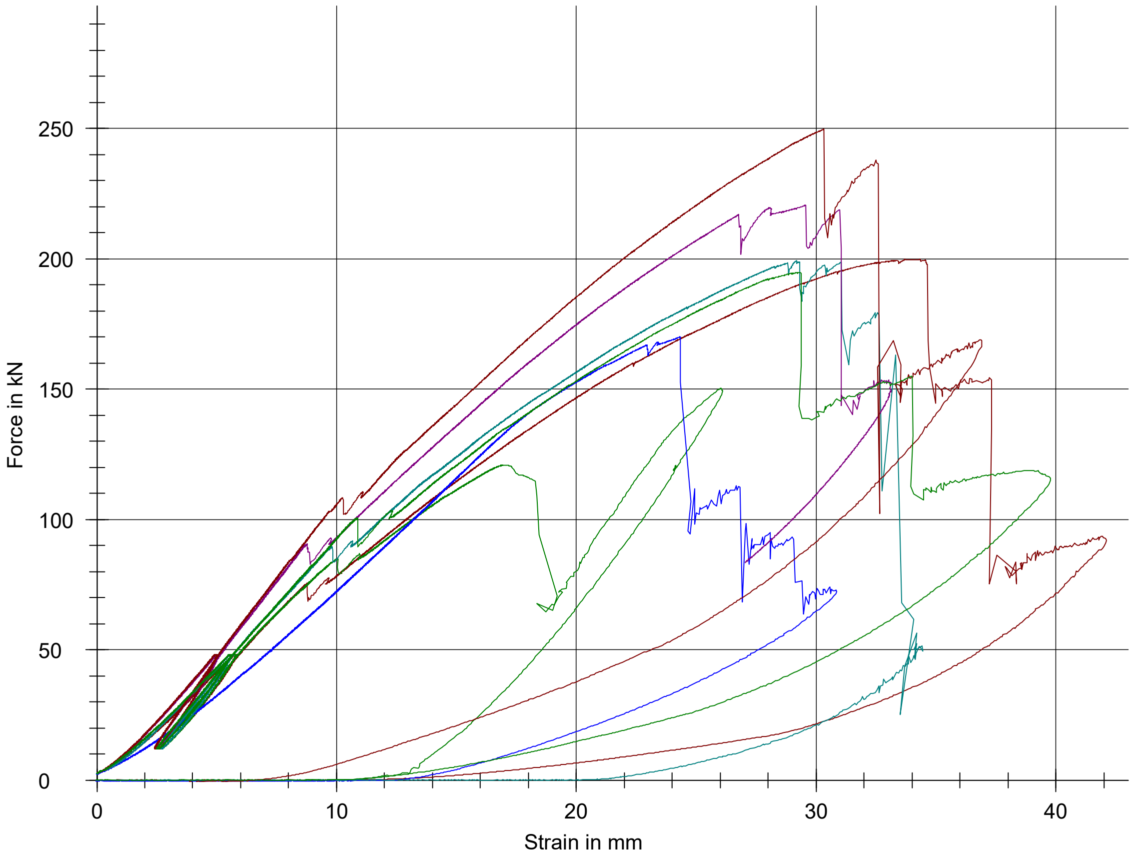
Series graphics:



Statistics:

Series n = 6	a mm	b mm	h mm	F_m kN
$\bar{x}$	600,0	2100,0	180,0	202,42
s	0,0	0,0	0,0	32,76
ν	0,00	0,00	0,00	16,18

Graph of load capacity for Slab B + C1



Appendix K. Pictures of Failures

K.1 Pictures of Failures for slabs of type A

K.2 Pictures of Failures for slabs of type B + Slab C1

K.3 Pictures of Failures for slab A6

K.4 Pictures of Failures shear fastener

Pictures of Failures, slab type A

The pictures are converted to a pdf file and the list and short description of failure is tabulated below for slab A.

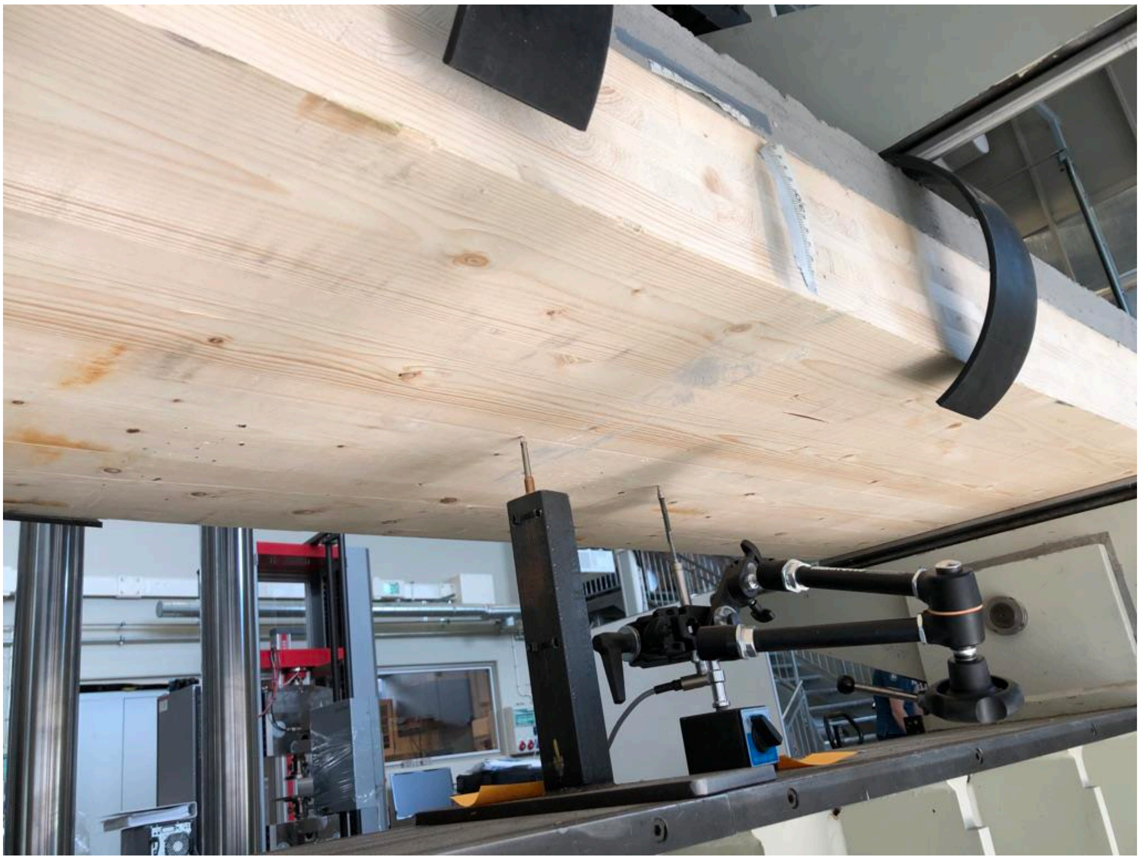
* Lamella 1 and 5 (T22) have a 20 mm in height and lamellae 2 and 4 (T15) have a 30 mm height (Slab: A6, B1, B2, B3 and B4)

** Shorter in length. (Slab A3, A3, B5 and C1)

Picture number	Slab	Failure
1	A1_max_2	Timber
2		No failure underneath
3	A2**	Concrete and timber, side view
4		Timber
5		Timber, underneath, close-up
6	A3**	Crushed concrete and timber, side view
7		Crushed concrete
8		Timber, underneath, knots
9		Slip
10		Timber, failure with ruler
11	A4	Knot (not failure in finger joint)
12		Timber and concrete, side view
13	A5_2	Knot and finger joint
14		Timber, midspan
15	A6*	Timber, underneath, finger joint
16		Timber, with, ruler

Picture 1

Picture 2

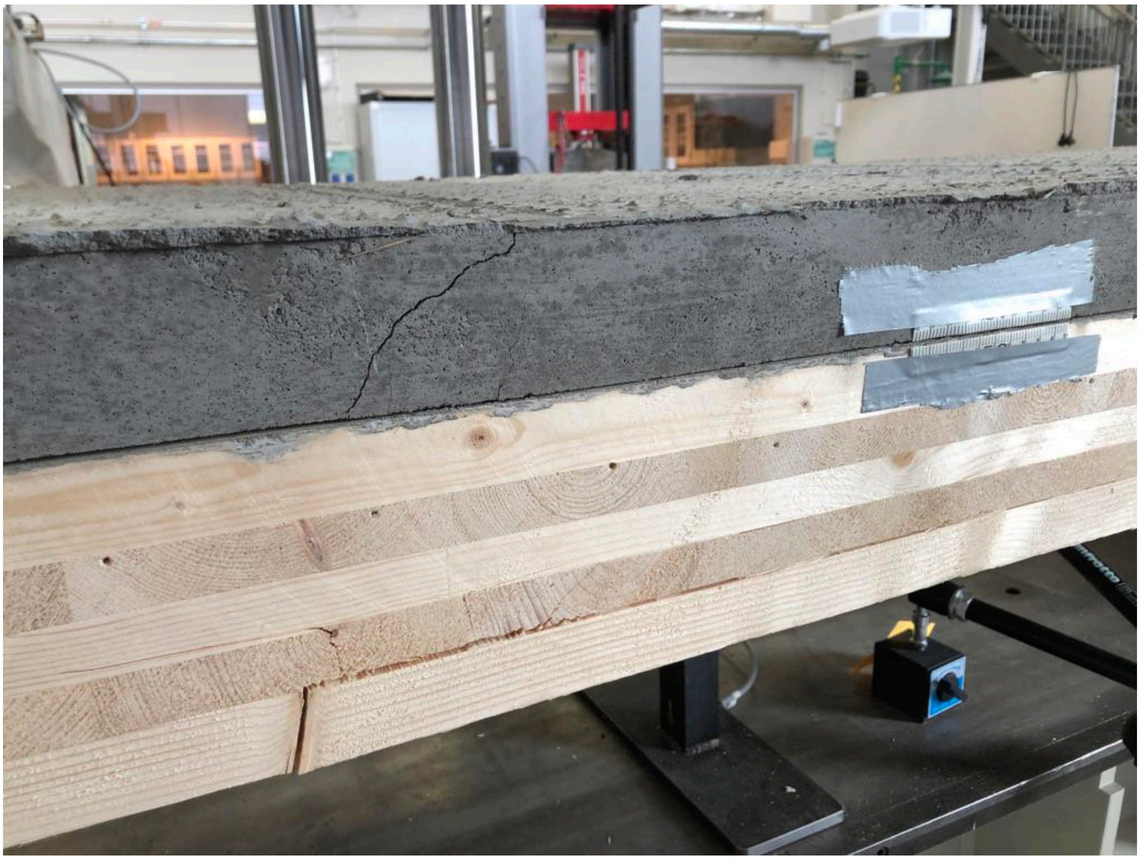


Picture 3

Picture 4

Picture 5

Picture 6

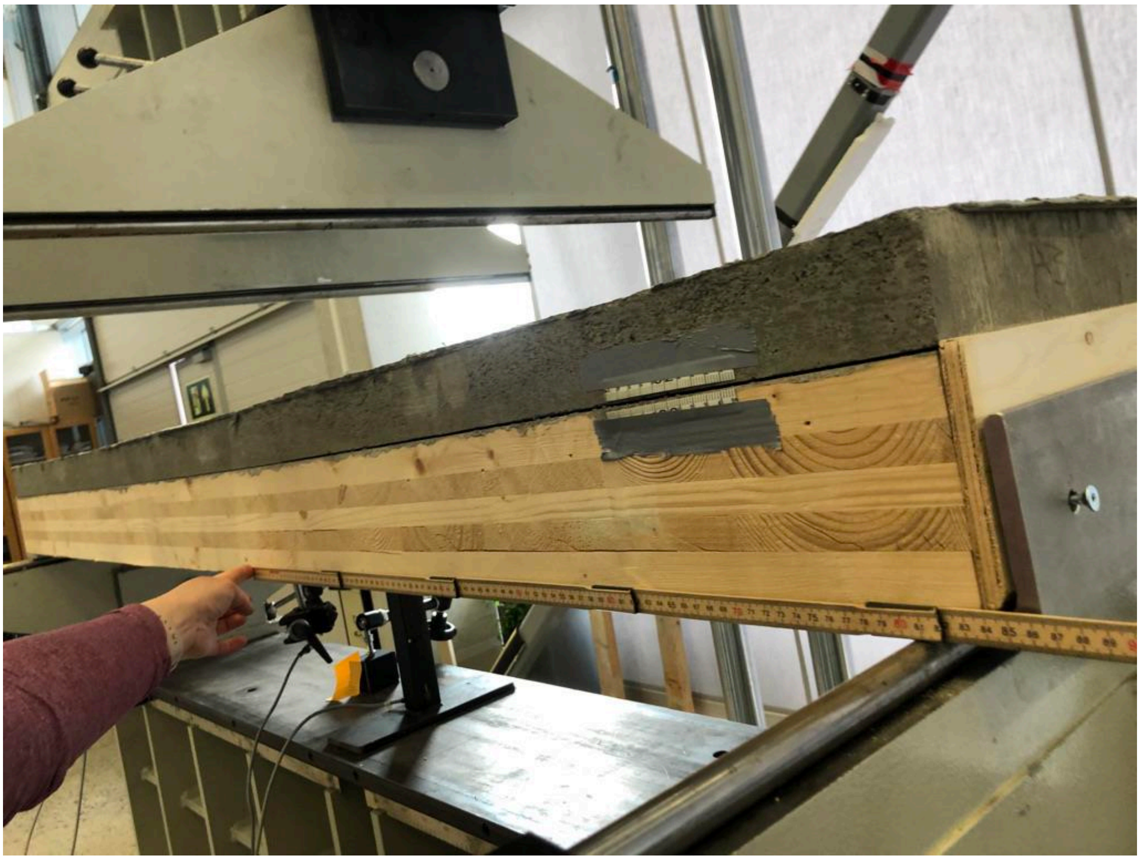


Picture 7

Picture 8

Picture 9

Picture 10



Picture 11

Picture 12

Picture 13

Picture 14

Picture 15

Picture 16

Pictures of Failures, slab type B + C1

The pictures are converted to a pdf file and the list and short description of failure is tabulated below for slab B +C1.

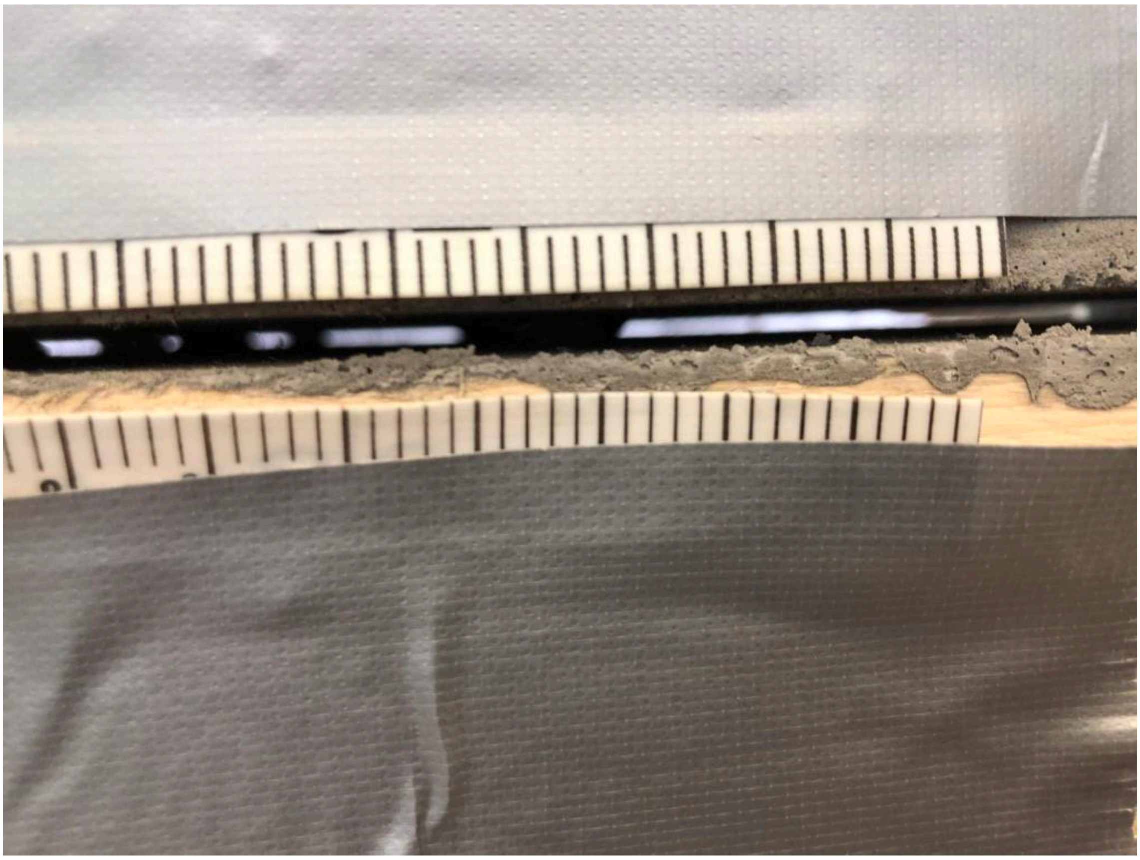
* Lamella 1 and 5 (T22) have a 20 mm in height and lamellae 2 and 4 (T15) have a 30 mm height (Slab: A6, B1, B2, B3 and B4)
** Shorter in length. (Slab A3, A3, B5 and C1)

Picture number	Slab	Failure
1	B1*	Concrete
2		Slip edge
3		Slip edge, screws
4		Timber, concrete, slip
5		Timber, underneath, knot
6	B2_2*	Sideview, midspan, timber, concrete
7		Timber, underneath, knot
8	B3*	Sideview, midspan, timber, concrete
9		Slip
10		Timber, underneath, finger joint
11		Timber, underneath, knot
12	B4*	Sideview, midspan, timber, concrete
13		Sideview, midspan, timber, concrete
14		No failure underneath
15	B5**	Crack concrete
16		Slip, timber failure
17		Slip, timber, concrete failure
18		Timber, underneath, knot
19	B6	Crack concrete
20		Slip
21		Sideview, midspan, timber, concrete
22	C1**	Concrete, longitudinal crack
23		Timber, underneath, knot
24		Sideview, midspan, timber, concrete
25		Timber, underneath
26		Timber, underneath, knot

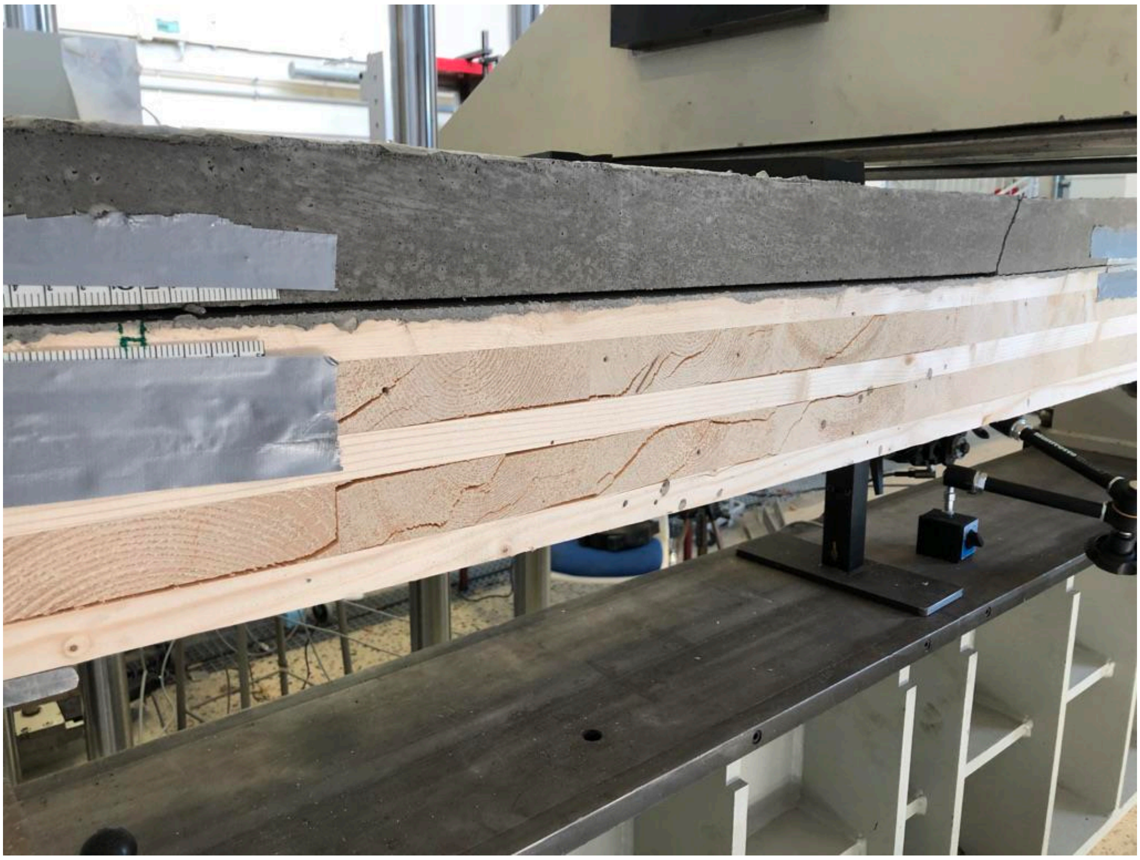
Picture 1

Picture 2

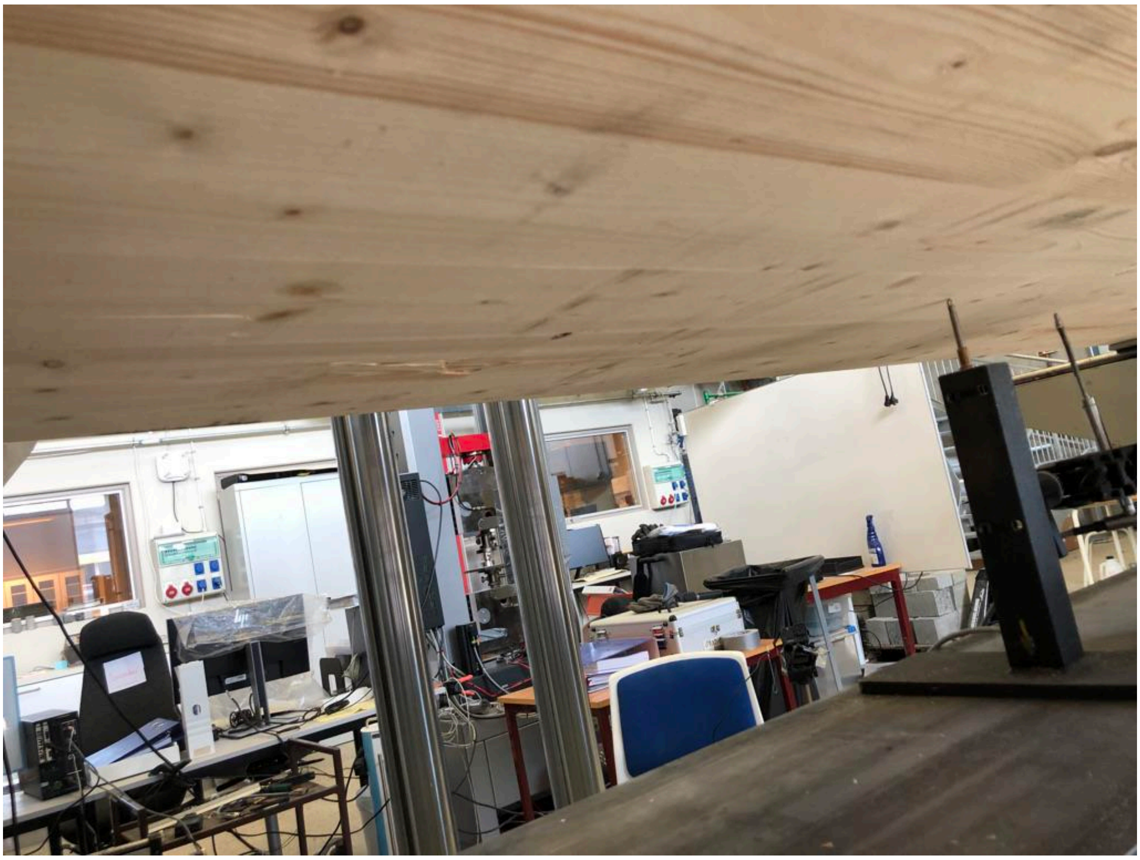
Picture 3



Picture 4



Picture 5



Picture 6

Picture 7

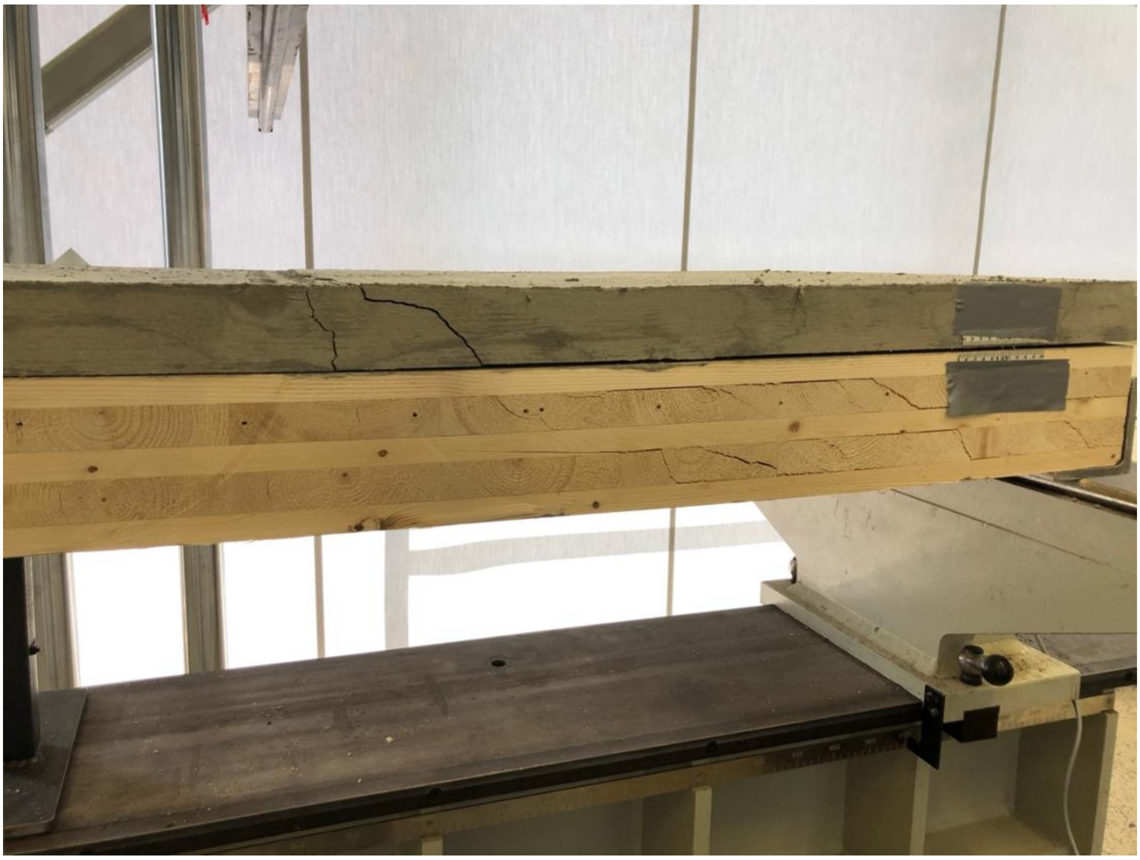
Picture 8

Picture 9

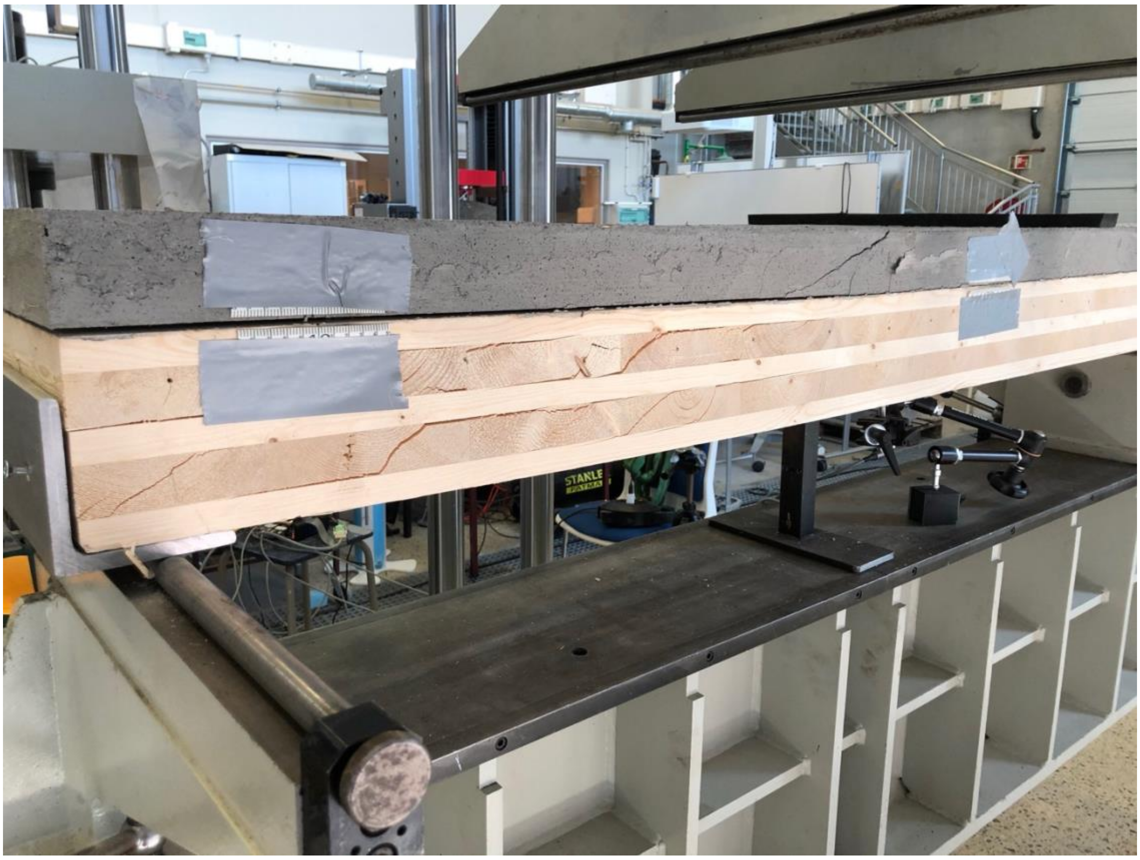
Picture 10

Picture 11

Picture 12



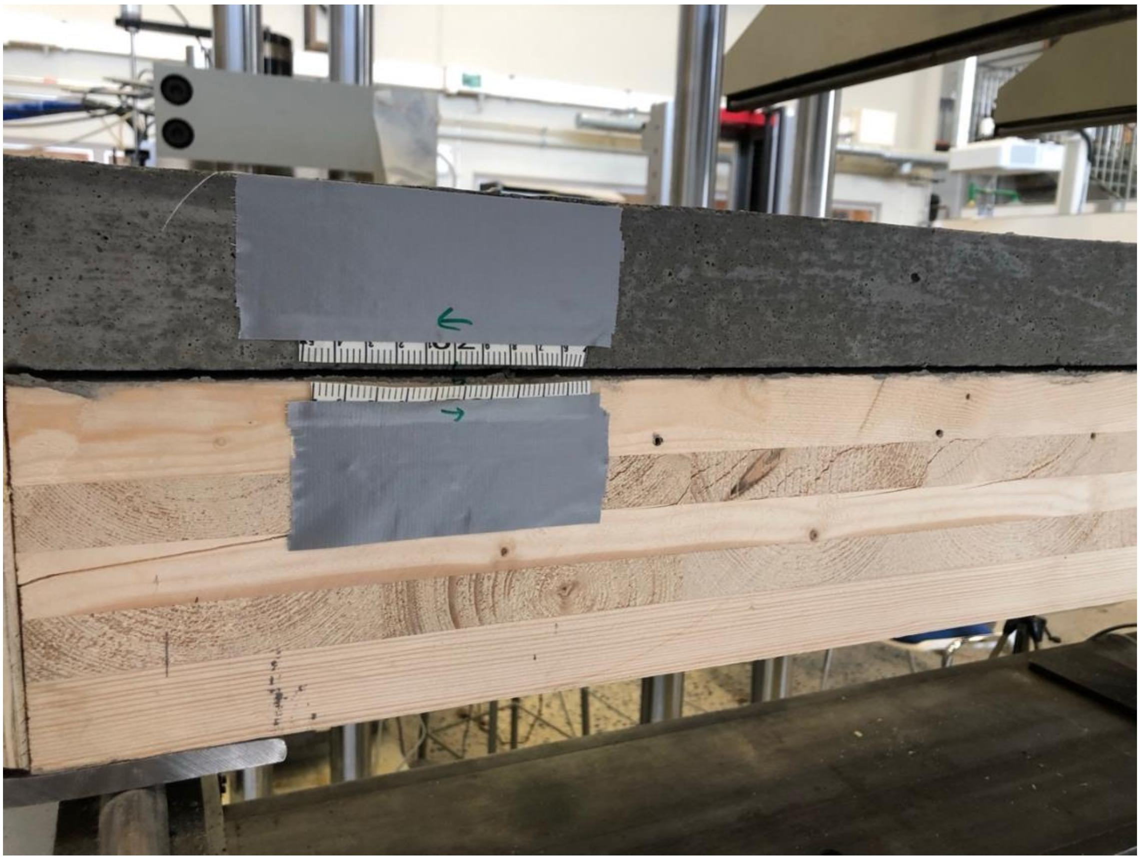
Picture 13



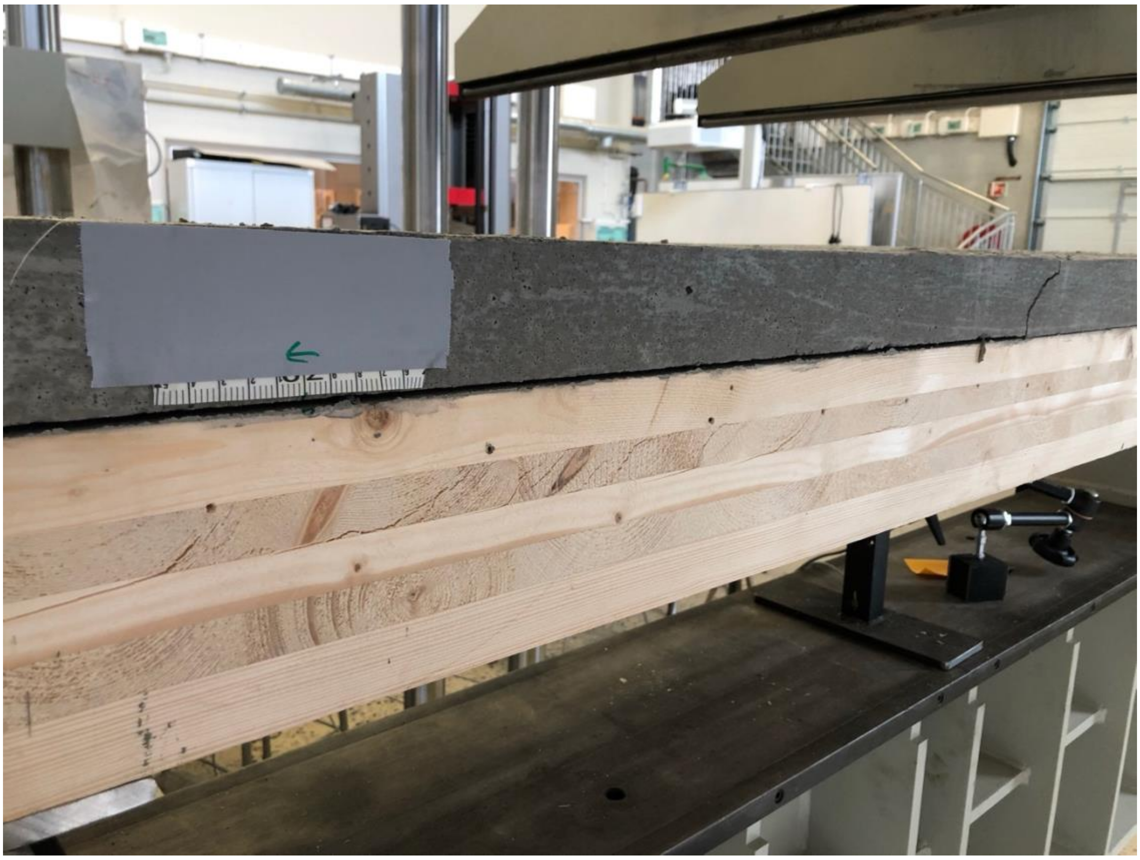
Picture 14

Picture 15

Picture 16



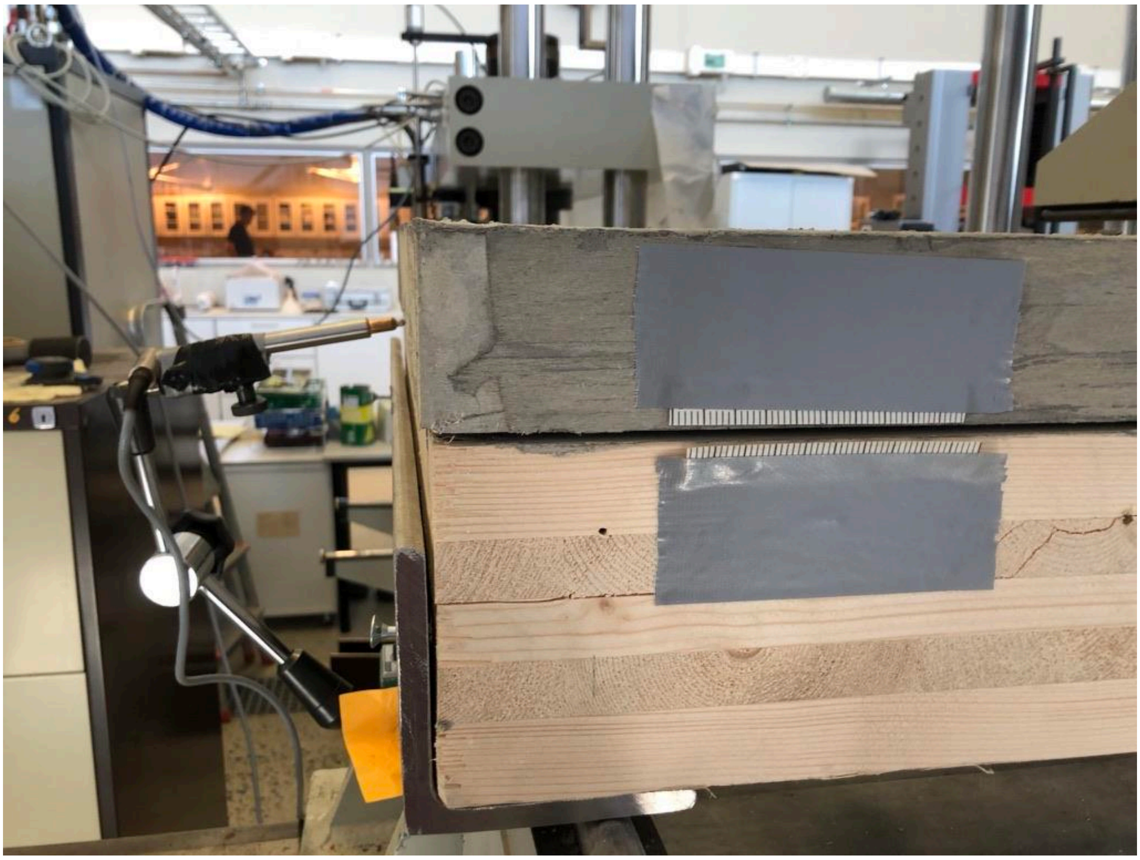
Picture 17



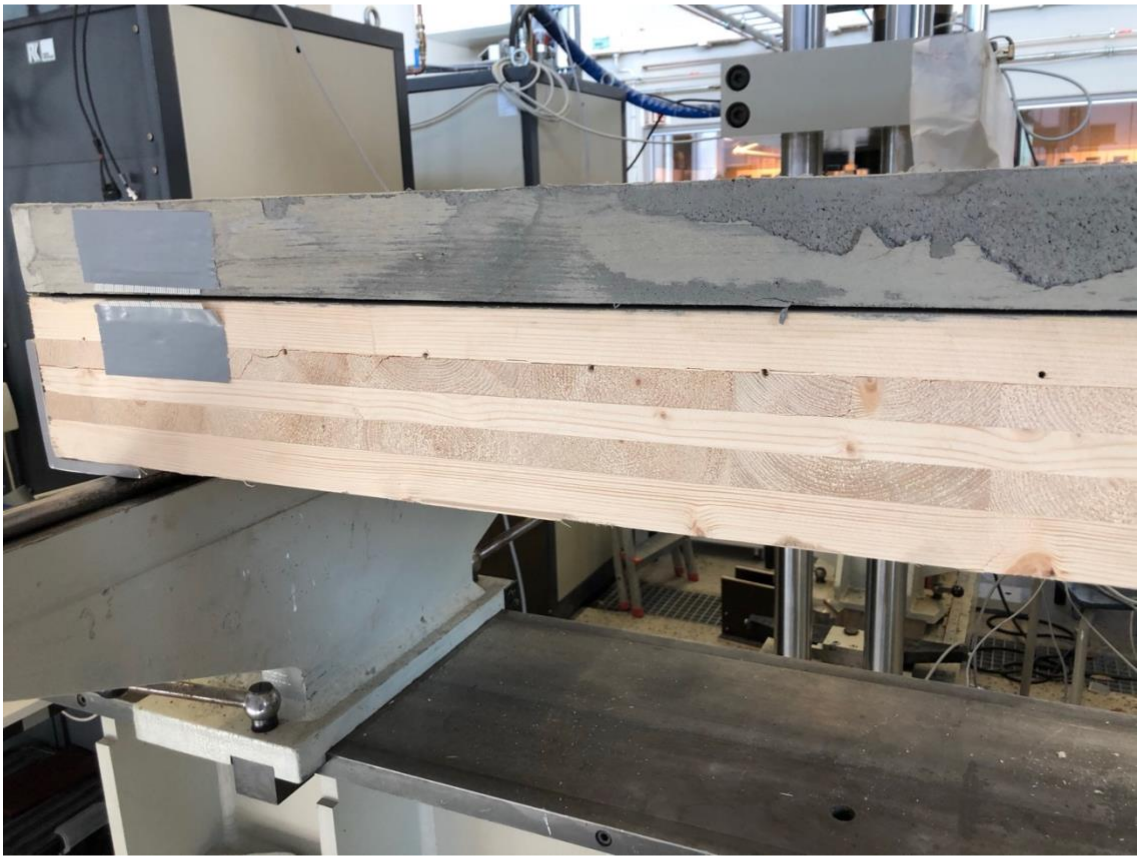
Picture 18

Picture 19

Picture 20



Picture 21



Slab C1

Picture 22

Picture 23

Picture 24

Picture 25

Picture 26

Pictures of Failure, slab A6

The picture series shows the timeline with seconds of testing thee slabA6. I have included the display for some, showing the timestamp of how fast it collapses, since it only take approximately 2 seconds. This was filmed with Gopro camera Hero 9.

Pictures of Failures, shear fasteners

The pictures are converted to a pdf file and shows the two types of shear fasteners, after investigation of there were any failure to the screws. 3 pictures of each type of shear fastener

CTC screws – Slab A2 – 3 pictures

Picture 1

Picture 2

Picture 3

KOP screws – Slab A2 – 3 pictures

Picture 1

Picture 2

Picture 3

Appendix L. Efficiency calculations

L.1 Efficiency calculations – Excel spreadsheet

L2 Maximum deflection for Slab A based on SLS verifications, NO composite Action

L.3 Maximum deflection for Slab B based on SLS verifications, NO composite Action

Appendix L.1 Efficiency calculations - Excel spreadsheet

Efficiency calculations

1st DROP

	DN	DC	Di	Efficiency
A1	5,54700913	5,61072617	6,159733	9,61632649
A2	5,54700913	5,61072617	7,67309	33,3675393
A3	5,54700913	5,61072617	6,858697	20,5861391
A4	5,54700913	5,61072617	6,997531	22,7650538
A5	5,54700913	5,61072617	8,58584	47,6925925
A6	5,54700913	5,61072617	8,803105	51,1024338
B1	6,35439361	5,72554474	5,037414	2,09427047
B2	6,35439361	5,72554474	5,578972	1,23308103
B3	6,35439361	5,72554474	5,729669	0,99344158
B4	6,35439361	5,72554474	6,591282	-0,3767016
B5	6,35439361	5,72554474	4,760207	2,53508704
B6	6,35439361	5,72554474	5,432587	1,46586352
C1	6,35439361	5,72554474	5,882049	0,7511258

Maximum deflection predictions based on SLS CTC-screws 7-160 mm

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\# \frac{kN}{m\hat{}3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5 : \#mm$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor, Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor, Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 : \#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1} : \#\frac{kN}{m\hat{}3}$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\,mean,\,t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\,mean,\,t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\,mean,\,t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\,t15} := G_{90,\,mean,\,t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\,k,\,t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\,0,\,k,\,t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\,k,\,t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} : \# \frac{kN}{m\hat{}3}$

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

> $g_{0,\,k} := \left(\frac{b}{1000} \cdot \frac{h_c}{1000} \cdot \rho_c + \frac{b}{1000} \cdot \frac{h_t}{1000} \left(\rho_{t22} \cdot 0.5 + \rho_{t15} \cdot 0.5 \right) \right) : \# \frac{kN}{m}$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k} \cdot \gamma_{G,\,2} : \# \frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b \cdot h_1$: #mm^2

> $A_5 := A_1$: #mm^2

> $I_{t1} := \frac{(b \cdot h_1\hat{}3)}{12}$: #mm^4

> $I_{t5} := I_{t1}$: #mm^4

Layer 2, 3 and 4 (T15)

> $A_2 := b \cdot h_2$: #mm^2

> $A_3 := A_2$: #mm^2

> $A_4 := A_2$: #mm^2

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12}$: #mm^4

> $I_{t3} := I_{t2}$: #mm^4

> $I_{t4} := I_{t2}$: #mm^4

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2}$: #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*Ii

> $(EI)_1 := E_{0,\, mean,\, t22} \cdot I_{t1}$: #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

$$(EI)_{sum} := 39884000000 \quad \textbf{(1)}$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\, mean,\, t22} \cdot A_1 \cdot (z_1\hat{}2)$: #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\, mean,\, t15} \cdot A_2 \cdot (z_2\hat{}2)$: #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\, mean,\, t15} \cdot A_3 \cdot (z_3\hat{}2)$: #Nmm^2

> $(EAz\hat{}2)_4 := E_{90, mean, t15} \cdot A_4 \cdot (z_4\hat{}2)$: #Nmm^2

> $(EAz\hat{}2)_5 := E_{0, mean, t22} \cdot A_5 \cdot (z_5\hat{}2)$: #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

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad \textbf{(2)}$$

2.2 The effectiv shear stiffeness for the CLT

element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} := evalf\left((a\hat{}2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b} \right) \right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^{6} \qquad (3)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$:
> #CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

> $EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{}2}}$; #$\frac{N}{mm\hat{}2} \cdot mm\hat{}4$

$$EI_{app} := 7.260572161\ 10^{11} \qquad (4)$$

> $E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t\hat{}3}{12}}$; #$\frac{N}{mm\hat{}}$

$$E_{CLT} := 8403.440000 \qquad (5)$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $E_1 := E_{cm,\,c} : \#\frac{N}{mm\hat{}2}$

> $E_2 := E_{CLT} : \#\frac{N}{mm}$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm\hat{}2$

> $A_2 := h_2 \cdot b : \#mm\hat{}2$

> $I_1 := I_c : \#mm\hat{}4$

> $I_2 := \frac{b \cdot h_t\hat{}3}{12} : \#mm\hat{}4$

The slip modulus **Kser**
EC5: 2.2.2(2) Ultimate limit states equation 2.1
The values for the slip modulus (Kser) is found in the pdf about the screw types from the company **Rotho blaas page 227**
There are 3three pair of screws in each row, in the width. Thats why the formula in ROthoblaas is multiplied with three.

> $l_{eff,\,ctc} := 110 : \#mm$

> $K_{ser} := 3 \cdot 70 \cdot l_{eff,\,ctc} : \#\frac{N}{mm}$

Finding the spacing for the slab
Rotho blaas p. 225: minimum spacing was calculated
s,min= 130*sin(45), and a continuous spacing were chosen

> $angle := 45 :$

> $k := \sin(convert(angle\ degrees,\ radians)) :$

> $s_{\min,\,1} := evalf(130 \cdot k) : \#mm$

> $s := 150 : \#mm$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1+\frac{\pi^\wedge 2\cdot E_1\cdot s\cdot A_1}{K_{ser}\cdot L^\wedge 2}}\right);$

$$\gamma_1 := 0.04851770613 \qquad (6)$$

>

> $\gamma_2 := 0; \#No\ composite$

$$\gamma_2 := 0 \qquad (7)$$

> $a_2 := \frac{\gamma_1\cdot E_1\cdot A_1\cdot(h_1+h_2)}{2\cdot(\gamma_1\cdot E_1\cdot A_1+\gamma_2\cdot E_2\cdot A_2)}; \#mm$

$$a_2 := 89.99999997 \qquad (8)$$

> $a_2 := 0;$

$$a_2 := 0 \qquad (9)$$

>

> $a_1 := \frac{(h_1+h_2)}{2} - a_2; \#mm$

$$a_1 := 90 \qquad (10)$$

> $EI_{eff,\ tot} := E_1\cdot I_1 + \gamma_1\cdot E_1\cdot A_1\cdot a_1{}^\wedge 2 + E_2\cdot I_2 + \gamma_2\cdot E_2\cdot A_2\cdot a_2{}^\wedge 2; \#Nmm^\wedge 2$

$$EI_{eff,tot} := 1.574281162\ 10^{12} \qquad (11)$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{(\gamma_1\cdot E_1\cdot a_1\cdot M_{Ed,\ 1})}{EI_{eff,\ tot}}\cdot 10^\wedge 6 : \#MPa$

> $\sigma_{m,\ 1} := \frac{(0.5\cdot E_1\cdot h_1\cdot M_{Ed,1})}{EI_{eff,tot}}\cdot 10^\wedge 6 : \#MPa$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> \; M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 3.143718714\;10^7 \tag{12}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma\, c,b \;=\; -\;\; \sigma\, 1 \;+\; \sigma\, m,1 \;=\; \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> \; M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 2.649283775\;10^6 \tag{13}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \; \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \; \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\; t,\,t \;=\; -\; \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2\cdot E_2\cdot a_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6;\right)}{\frac{k_{modi,t}\cdot f_{t,\ 0,\ k,\ t22}}{\gamma_M}}+\frac{\left(\frac{(0.5\cdot E_2\cdot h_2\cdot M_{Ed,2})}{EI_{eff,tot}}\cdot 10\hat{}6\ ;\right)}{\frac{k_{modi,t}\cdot f_{m,k,t22}}{\gamma_M}}\right)\leq 1.0$$

> $$M_3 := solve\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 6.624695649\ 10^7 \qquad (14)$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\ t,\ b\ =\ -\frac{\sigma_2}{f_{t,0,d}}+\frac{\sigma_{m,2}}{f_{m,d}}<1.0$$

> $$M_4 := solve\left(M_{Ed,2}=\left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2\cdot E_2\cdot a_2)}{EI_{eff,tot}\cdot f_{t,0,k,t22}}+\frac{(0.5\cdot E_2\cdot h_2)}{EI_{eff,tot}\cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 6.624695649\ 10^7 \qquad (15)$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $$M_{Ed,\ new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6} : \#kNm$$

> $$L_{out} := 0.75 : \#m$$

> $$L_{sup} := 2.0 : \#m$$

> $$P_{Ed} := solve\left(\frac{P_{Ed,\ 1}\cdot L_{out}}{2}+\frac{1.5\cdot g_{0,k}\cdot L_{sup}\hat{}2}{8}=M_{Ed,\ new}, P_{Ed,\ 1}\right); \#kN$$

$$P_{Ed} := 81.39702905 \qquad (16)$$

3.4 Verification of the vertical defelction

> $w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot}};$

$$w := 5.547009131 \quad \textbf{(17)}$$

> $w_{\lim} := evalf\left(\frac{L}{250}\right);$

$$w_{\lim} := 8. \quad \textbf{(18)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}}; \# < 1.0\ OK$

$$Ver_{deflection} := 0.6933761414 \quad \textbf{(19)}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,\,g} := \frac{E_{cm,\,c}}{1 + \varphi_c}:$

> $E_{1,\,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2}:$

> $q_k := 0:$

> $g_{1,\,k} := 0:$

> $E_{1,\,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,\,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}}:$

4.1.2 CLT

> $E_{2,\,g} := \frac{E_{CLT}}{1 + k_{def,\,t}}:$

> $E_{2,q} := \dfrac{E_{CLT}}{1 + k_{def,t} \cdot \psi_2}$:

> $E_{2,fin} := \dfrac{E_{2,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$:

4.1.3 Slip modulus

>

> $K_{ser,g} := \dfrac{K_{ser}}{1 + k_{def,t}}$:

> $K_{ser,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2}$:

> $K_{ser,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}$:

> $K_{u,fin} := K_{ser,2}$:

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1,fin} := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{}2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,fin} \cdot L\hat{}2}}\right)$;

$$\gamma_{1,fin} := 0.08798300593 \quad \textbf{(20)}$$

>

> $\gamma_{2,fin} := 0$;

$$\gamma_{2,fin} := 0 \quad \textbf{(21)}$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}$; #mm

$$a_{2,fin} := 89.99999998 \quad \textbf{(22)}$$

> $a_{2,fin} := 0$;

$$a_{2,fin} := 0 \quad \textbf{(23)}$$

> $a_{1,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin}$; #mm

$$a_{1,fin} := 90 \quad \textbf{(24)}$$

> $EI_{eff,tot,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,fin}\hat{}2 + E_{2,fin} \cdot I_2 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2 \cdot a_{2,fin}\hat{}2$;

#Nmm^2

$$EI_{eff,tot,fin} := 7.466058495\ 10^{11} \quad (25)$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma\, c,t\ = -\ \sigma 1 - \sigma\, m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 4.729404815\ 10^{7} \quad (26)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma\, c,b\ = -\ \sigma\, 1 + \sigma\, m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 5.104847658\ 10^{6} \quad (27)$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

> $\sigma_{m,2} := \frac{(0.5 \cdot E_{2,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$\#\sigma\, t, t = -\frac{\sigma_2}{f_{t,0,d}} - \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

$\#f_{m,d} := \frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}$

$\#f_{t,d} := \frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin} \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; \right)}{\frac{k_{modi,t} \cdot f_{t,0,k,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_{2,fin} \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 ; \right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}} \right) \leq 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$

$M_3 := 5.812282958\ 10^7$ **(28)**

>

Stresses at the **BOTTOM** of the **TIMBER** section

$\#\sigma\, t, b = -\frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_{2,fin} \cdot E_{2,fin} \cdot a_{2,fin})}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_{2,fin} \cdot h_2)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}} \right)} \right), M_{Ed,2} \right); \#Nmm$

$M_4 := 5.812282958\ 10^7$ **(29)**

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\ new} := \dfrac{\min(M_1, M_3, M_4)}{10\hat{}6}$: #kNm

> $L_{out} := 0.75$: #m

> $L_{sup} := 2.0$: #m

> $P_{Ed,\ fin} := solve\left(\dfrac{P_{Ed,\ 1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\ new}, P_{Ed,\ 1}\right)$; #kN

$$P_{Ed,fin} := 123.6819917 \quad (30)$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \dfrac{5 \cdot \left(\dfrac{P_{Ed,\ fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{}4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 17.59594516 \quad (31)$$

>

> $w_{\lim} := evalf\left(\dfrac{L}{150}\right)$;

$$w_{\lim} := 13.33333333 \quad (32)$$

Verification of the vertical deflection

> $Ver_{deflection} := \dfrac{w_{permanent}}{w_{\lim}}$; # <1.0 NOT OK

$$Ver_{deflection} := 1.319695887 \quad (33)$$

>

>

>

>

>

Maximum deflection predictions based on SLS KOP-screws 10-140 mm

> *restart*;

General data:
c: Concrete B35
t: Timber T22 and T15

> $L := 2000$: *#mm span length between the supports*

b(c) = b(t) = 600 mm

> $b := 600$: *#mm*

Concrete data, B35

Information of the concrete is taken from Eurocode 2 (NS-EN 1992-1-1:2004+NA:2008 tabel 3.1)

> $h_c := 60$: *#mm*

> $A_c := h_c \cdot b$: *#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*

> $\rho_c := 25.00$: $\# \frac{kN}{m\hat{}3}$

> $\gamma_c := 1.5$:

> $\varphi_c := 2.5$:

Timber, CLT (cross-laminated timber)

The information of the timber is provided by Splitkon (SINTEF certification Nr. 20712)
And some information of timber is taken from Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)
And some information of timber is taken from the Swedish handbook of CLT (E. Borgström and J. Fröbel,"The CLT HAndbook", Swedish Wood, 2019)

The timber used in the laboratroy testing is a 5-layered elements
The outermost layers (layer 1 and 5) has the class T22 and the middle layers has the class T15.

> $h_1 := 30 : \#mm$

> $h_2 := 20 : \#mm$

> $h_3 := 20 : \#mm$

> $h_4 := 20 : \#mm$

> $h_5 := 30 : \#mm$

> $h_t := h_1 + h_2 + h_3 + h_4 + h_5; \#mm$

$$h_t := 120 \quad (1)$$

> $\gamma_M := 1.15 : \#NA\ in\ Eurocode\ 5\ for\ Glued\ laminated\ timber$

> $Klima := 1.0 : \#Serice\ class,\ permanent$

> $k_{modi,\ t} := 0.8 : \#modification\ factor,Swedish\ CLT\ handbook$

> $k_{def,\ t} := 0.85 : \#modification\ factor,Swedish\ CLT\ handbook$

Lamellae 1 and 5, Class T22

> $E_{0,\ mean,\ t22} := 13000 : \#\frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t22} := 430 : \#\frac{N}{mm\hat{}2}$

> $G_{0,mean,t22} := 810 : \#\frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t22} := 81 : \#\frac{N}{mm\hat{}2}$

> $G_{R,\ t22} := G_{90,mean,t22} : \#\frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t22} := 30.5 : \#\frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t22} := 22.0 : \#\frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t22} := 4.0 :\#\frac{N}{mm\hat{}2}$

> $t_{t22} := 470 : \#\frac{kg}{m\hat{}3}$

> $\rho_{t22} := \frac{t_{t22} \cdot 0.00980663558553261}{1}; \#\frac{kN}{m\hat{}3}$

$$\rho_{t22} := 4.609118725 \quad (2)$$

Lamellae 2, 3 and 4, Class T15

> $E_{0,\ mean,\ t15} := 11500 : \# \frac{N}{mm\hat{}2}$

> $E_{90,\ mean,\ t15} := 230 : \# \frac{N}{mm\hat{}2}$

> $G_{0,\ mean,\ t15} := 720 : \# \frac{N}{mm\hat{}2}$

> $G_{90,\ mean,\ t15} := 72 : \# \frac{N}{mm\hat{}2}$

> $G_{R,\ t15} := G_{90,\ mean,\ t15} : \# \frac{N}{mm\hat{}2}$

> $f_{m,\ k,\ t15} := 22 : \# \frac{N}{mm\hat{}2}$

> $f_{t,\ 0,\ k,\ t15} := 15.0 : \# \frac{N}{mm\hat{}2}$

> $f_{v,\ k,\ t15} := 4.0 : \# \frac{N}{mm\hat{}2}$

> $t_{t15} := 430 : \# \frac{kg}{m\hat{}3}$

> $\rho_{t15} := \frac{t_{t15} \cdot 0.00980663558553261}{1} ; \# \frac{kN}{m\hat{}3}$

$$\rho_{t15} := 4.216853302 \quad (3)$$

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

> $g_{0,\,k} := \left(\frac{b}{1000}\cdot\frac{h_c}{1000}\cdot\rho_c + \frac{b}{1000}\cdot\frac{h_t}{1000}\left(\rho_{t22}\cdot 0.5 + \rho_{t15}\cdot 0.5\right)\right) : \#\frac{kN}{m}$

1. 1 SLS

> $f_{d,\,SLS} := g_{0,\,k}\cdot\gamma_{G,\,2} : \#\frac{kN}{m}$

I have to find the the shear force and moment.

The result above is a bit small, I have to find the maximum loading the timber-concrete composite slab can withhold.
I have to use a combination of "Gamma method" (EC5-Annex b) and "Shear Analogy method" (From the CLT handbook US version) for my procedure, because the "Gamma Method" alone is only applicable for a 3 layered element.

2. Shear Analogy method for CLT elements

For a 5 layered CLT Element, using the theory from the CLT handbook US edition

EI[eff,CLT]= sum Ei*Ii + sum EiAi*zi^2

Layer 1 and 5 (T22)

> $A_1 := b\cdot h_1 : \#mm\hat{}2$

> $A_5 := A_1 : \#mm\hat{}2$

> $I_{t1} := \frac{(b\cdot h_1\hat{}3)}{12} :\#mm\hat{}4$

> $I_{t5} := I_{t1} : \#mm\hat{}4$

Layer 2, 3 and 4 (T15)

> $A_2 := b\cdot h_2 : \#mm\hat{}2$

> $A_3 := A_2;: \#mm\hat{}2$

$$A_3 := 12000 \qquad (4)$$

> $A_4 := A_2 :$ #mm^2

> $I_{t2} := \frac{(b \cdot h_2\hat{}3)}{12} :$ #mm^4

> $I_{t3} := I_{t2} :$ #mm^4

> $I_{t4} := I_{t2} :$ #mm^4

2.1 The effectiv bending stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 24
The height to NA

$$EI_{eff} = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$$

> $z_1 := \frac{h_1}{2} + h_2 + \frac{h_3}{2} :$ #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*Ii

> $(EI)_1 := E_{0,\ mean,\ t22} \cdot I_{t1} :$ #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

$$(EI)_{sum} := 39884000000 \quad \textbf{(5)}$$

Ei*Ao*zi^2

> $(EAz\hat{}2)_1 := E_{0,\ mean,\ t22} \cdot A_1 \cdot (z_1\hat{}2) :$ #Nmm^2

> $(EAz\hat{}2)_2 := E_{90,\ mean,\ t15} \cdot A_2 \cdot (z_2\hat{}2) :$ #Nmm^2

> $(EAz\hat{}2)_3 := E_{0,\ mean,\ t15} \cdot A_3 \cdot (z_3\hat{}2) :$ #Nmm^2

> $(EAz\hat{}2)_4 := E_{90,mean,t15} \cdot A_4 \cdot (z_4\hat{}2) :$ #Nmm^2

> $(EAz\hat{}2)_5 := E_{0,mean,t22} \cdot A_5 \cdot (z_5\hat{}2)$: #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

$$(EAz\hat{}2)_{sum} := 949908000000 \quad (6)$$

The effective bending stiffeness for the CLT

CLT handbook US-chapter 3- equation 24

> $(EI)_{eff} := evalf\left((EI)_{sum} + (EAz\hat{}2)_{sum}\right)$; #Nmm^2

$$(EI)_{eff} := 9.897920000\ 10^{11} \quad (7)$$

2.2 The effectiv shear stiffeness for the CLT element:

CLT handbook US-chapter 3- equation 25

$$(GA)_{eff} = \frac{a^2}{\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=1}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)}$$

> $a := \frac{h_1}{2} + h_2 + h_3 + h_4 + \frac{h_5}{2}$: #mm

> $(GA)_{eff} := evalf\left((a\hat{}2) \Big/ \left(\frac{h_1}{2 \cdot G_{0,\ mean,\ t22} \cdot b} + \frac{h_2}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_3}{G_{0,\ mean,\ t15} \cdot b} + \frac{h_4}{G_{90,\ mean,\ t15} \cdot b} + \frac{h_5}{2 \cdot G_{0,\ mean,\ t22} \cdot b}\right)\right)$; #N

$$(GA)_{eff} := 7.834029851\ 10^{6} \quad (8)$$

2.3 The apparent bending stiffness

It can be found from reducing the effective bending stiffness per the following:
CLT handbook US-chapter 3- equation 28

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

> $K_s := 11.5$

#CLT handbook US-chapter 3-table2, pinned-pinned support, uniformly distributed load

(9)

$$K_s := 11.5$$ **(9)**

> $$EI_{app} := \frac{(EI)_{eff}}{1 + \frac{K_s \cdot (EI)_{eff}}{(GA)_{eff} \cdot L\hat{}2}} : \# \frac{N}{mm\hat{}2} \cdot mm\hat{}4$$

> $$E_{CLT} := \frac{EI_{app}}{\frac{b \cdot h_t\hat{}3}{12}} : \# \frac{N}{mm\hat{}}$$

3. Gamma method, Annex B, EC5 Maximum deflection prediction based on SLS using short-term verifications of the slab

Eurocode 5 (NS-EN 1995-1-1:2004+A1:2008+Na 2010)

General data:

> $$E_1 := E_{cm,\,c} : \# \frac{N}{mm\hat{}2}$$

> $$E_2 := E_{CLT} : \# \frac{N}{mm}$$

> $h_1 := h_c : \#mm$

> $h_2 := h_t : \#mm$

> $A_1 := A_c : \#mm\hat{}2$

> $A_2 := h_2 \cdot b : \#mm\hat{}2$

> $I_1 := I_c : \#mm\hat{}4$

> $$I_2 := \frac{b \cdot h_t\hat{}3}{12} : \#mm\hat{}4$$

The slip modulus **Kser**
To find Kser for KOP screw, could not find this in a similar manner in the Rothoblaas booklets.

For the calculation for the KOP10140 screws, we have to use the EC5 for finding the Kser and Ku .

Kser:
7.1 table 7.1 and use the formula for density 7.1.(2) and the multiply by 2, since timber-concrete composite. 7.1(3)

> $\rho_{m,\,1} := t_{t22}$:

> $\rho_{m,\,2} := t_{t15}$:

> $\rho_m := evalf\left(\mathrm{sqrt}\left(\rho_{m,\,1} \cdot \rho_{m,\,2}\right)\right)$:

> $d_{kop} := 10 : \#mm$

> $K_{ser} := evalf\left(\frac{d_{kop}}{23} \cdot \rho_m \hat{}\, 1.5 \cdot 3\right)$:

This is based on Rannveigs design example, slab 2. using spacing 100, for KOP screws. adjusted the length of the screw, to fit the height of the timber-concrete composite slab.

> $s := 100 : \#mm$

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_1 := evalf\left(\frac{1}{1 + \frac{\pi \hat{}\, 2 \cdot E_1 \cdot s \cdot A_1}{K_{ser} \cdot L \hat{}\, 2}}\right);$

$$\gamma_1 := 0.03953909054 \tag{10}$$

> $\gamma_2 := 0; \#NO\ composite$

$$\gamma_2 := 0 \tag{11}$$

> $a_2 := \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot \left(h_1 + h_2\right)}{2 \cdot \left(\gamma_1 \cdot E_1 \cdot A_1 + \gamma_2 \cdot E_2 \cdot A_2\right)}; \#mm$

$$a_2 := 89.99999998 \tag{12}$$

>

> $a_1 := \frac{\left(h_1 + h_2\right)}{2} - a_2; \#mm$

$$a_1 := 2.\ 10^{-8} \tag{13}$$

> $EI_{eff,\,tot} := E_1 \cdot I_1 + \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 \hat{}\, 2 + E_2 \cdot I_2 + \gamma_2 \cdot E_2 \cdot A_2 \cdot a_2 \hat{}\, 2; \#Nmm \hat{}\, 2$

$$EI_{eff,tot} := 1.093257216\ 10^{12} \tag{14}$$

3.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

> $\sigma_1 := \frac{\left(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,\,1}\right)}{EI_{eff,\,tot}} \cdot 10 \hat{}\, 6 : \#MPa$

> $\sigma_{m,\,1} := \frac{\left(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1}\right)}{EI_{eff,tot}} \cdot 10 \hat{}\, 6 : \#MPa$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma c,t = -\ \sigma 1 - \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 2.5009152\ 10^7 \quad \textbf{(15)}$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma c,b = -\ \sigma 1 + \sigma m,1 = \frac{fck}{\gamma c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_1 \cdot E_1 \cdot a_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_1 \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(-\frac{(\gamma_1 \cdot E_1 \cdot a_1)}{EI_{eff,\,tot}} + \frac{(0.5 \cdot E_1 \cdot h_1)}{EI_{eff,\,tot}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 1.572003841\ 10^6 \quad \textbf{(16)}$$

3.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,\,2})}{EI_{eff,\,tot}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,2} := \frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,\,2})}{EI_{eff,tot}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **TIMBER** section

$$\#\sigma\ t,\ t = -\frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$$

$$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$$

$$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$$

$$\#M_{Ed,2}\left(\frac{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{(0.5 \cdot E_2 \cdot h_2 \cdot M_{Ed,2})}{EI_{eff,tot}} \cdot 10\hat{}6\;;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

> $$M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_3 := 4.600510059\ 10^7 \qquad \textbf{(17)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$$\#\sigma\, t,\, b\ =\ -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$$

> $$M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{(\gamma_2 \cdot E_2 \cdot a_2)}{EI_{eff,tot} \cdot f_{t,0,k,t22}} + \frac{(0.5 \cdot E_2 \cdot h_2)}{EI_{eff,tot} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$$

$$M_4 := 4.600510059\ 10^7 \qquad \textbf{(18)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

3.3 The maxiumum loading, Ped

> $$M_{Ed,\,new} := \frac{\min(M_1, M_3, M_4)}{10\hat{}6} : \#kNm$$

> $$L_{out} := 0.75 : \#m$$

> $$L_{sup} := 2.0 : \#m$$

> $$P_{Ed} := solve\left(\frac{P_{Ed,\,1} \cdot L_{out}}{2} + \frac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{}2}{8} = M_{Ed,\,new}, P_{Ed,\,1}\right); \#kN$$

$$P_{Ed} := 64.25560201 \qquad \textbf{(19)}$$

3.4 Verification of the vertical defelction

> $w := \frac{5 \cdot \left(\frac{P_{Ed}}{L_{sup}} + f_{d,SLS}\right) \cdot L^4}{384 \cdot EI_{eff,tot}};$

$$w := 6.354393607 \quad \textbf{(20)}$$

> $w_{\lim} := evalf\left(\frac{L}{250}\right);$

$$w_{\lim} := 8. \quad \textbf{(21)}$$

Verification of the vertical deflection

> $Ver_{deflection} := \frac{w}{w_{\lim}};$ # <1.0 OK

$$Ver_{deflection} := 0.7942992009 \quad \textbf{(22)}$$

4. Maximum deflection prediction based on SLS using long-term verifications of the slab

4.1 New elasticity modulus calculated:

4.1.1 Concrete

> $E_{1,g} := \frac{E_{cm,c}}{1 + \varphi_c}:$

> $E_{1,q} := \frac{E_{cm,c}}{1 + \varphi_c \cdot \psi_2}:$

> $q_k := 0:$

> $g_{1,k} := 0;$

$$g_{1,k} := 0 \quad \textbf{(23)}$$

> $E_{1,fin} := \frac{E_{1,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{1,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + q_k \cdot \gamma_{Q,1}}:$

4.1.2 CLT

> $E_{2,g} := \frac{E_{CLT}}{1 + k_{def,t}}:$

> $E_{2,\,q} := \dfrac{E_{CLT}}{1 + k_{def,\,t} \cdot \psi_2}$:

> $E_{2,\,fin} := \dfrac{E_{2,\,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + E_{2,\,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}}$:

4.1.3 Slip modulus

>

> $K_{ser,\,g} := \dfrac{K_{ser}}{1 + k_{def,\,t}}$:

> $K_{ser,\,q} := \dfrac{K_{ser}}{1 + k_{def,t} \cdot \psi_2}$:

> $K_{ser,\,2} := \dfrac{K_{ser,g} \cdot (g_{0,k} + g_{1,k}) \cdot \gamma_{G,1} + K_{ser,q} \cdot q_k \cdot \gamma_{Q,1}}{(g_{0,\,k} + g_{1,k}) \cdot \gamma_{G,\,1} + q_k \cdot \gamma_{Q,\,1}}$:

> $K_{u,\,fin} := K_{ser,\,2}$:

5. Long-term verifications

Annex B, EC5
B.2 Effective bending stiffenes

> $\gamma_{1,\,fin} := evalf\left(\dfrac{1}{1 + \dfrac{\pi\hat{}2 \cdot E_{1,fin} \cdot s \cdot A_1}{K_{u,\,fin} \cdot L\hat{}2}}\right)$;

$$\gamma_{1,fin} := 0.07225561973 \quad \textbf{(24)}$$

> $\gamma_{2,\,fin} := 0$; #*NO composite*

$$\gamma_{2,fin} := 0 \quad \textbf{(25)}$$

> $a_{2,fin} := \dfrac{\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot (h_1 + h_2)}{2 \cdot (\gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 + \gamma_{2,fin} \cdot E_{2,fin} \cdot A_2)}$; #*mm*

$$a_{2,fin} := 89.99999998 \quad \textbf{(26)}$$

>

> $a_{1,\,fin} := \dfrac{(h_1 + h_2)}{2} - a_{2,fin}$; #*mm*

$$a_{1,fin} := 2.\ 10^{-8} \quad \textbf{(27)}$$

> $EI_{eff,\,tot,\,fin} := E_{1,fin} \cdot I_1 + \gamma_{1,fin} \cdot E_{1,fin} \cdot A_1 \cdot a_{1,\,fin}\hat{}2 + E_{2,\,fin} \cdot I_2 + \gamma_{2,\,fin} \cdot E_{2,\,fin} \cdot A_2 \cdot a_{2,fin}\hat{}2$; #*Nmm^2*

$$EI_{eff,tot,fin} := 4.973776455\ 10^{11} \quad \textbf{(28)}$$

5.1 Normal stresses in the **CONCRETE** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_1 := \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,\,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

$$> \sigma_{m,\,1} := \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

Stresses at the **TOP** of the **CONCRETE** section

$$\#\sigma\, c,t \; = - \;\; \sigma 1 - \sigma\, m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{c,k}}{\gamma_c}$$

$$> M_1 := solve\left(M_{Ed,\,1} = \frac{f_{ck,\,c}}{\gamma_c \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_1 := 3.982271999\ 10^7 \qquad (29)$$

Stresses at the **BOTTOM** of the **CONCRETE** section

$$\#\sigma\, c,b \; = - \;\; \sigma\, 1 \; + \sigma\, m,1 = \frac{fck}{\gamma\, c}$$

$$\#M_{Ed,1} \cdot \left(\frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin} \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6; + \frac{(0.5 \cdot E_{1,fin} \cdot h_1 \cdot M_{Ed,1})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 \right) \leq \frac{f_{ctk,\,0.005,\,c}}{\gamma_c}$$

$$> M_2 := solve\left(M_{Ed,\,1} = \frac{f_{ctk,\,0.05,\,c}}{\gamma_c \cdot \left(- \frac{(\gamma_{1,fin} \cdot E_{1,fin} \cdot a_{1,\,fin})}{EI_{eff,tot,fin}} + \frac{(0.5 \cdot E_{1,fin} \cdot h_1)}{EI_{eff,tot,fin}} \right)}, M_{Ed,\,1} \right); \#Nmm$$

$$M_2 := 2.503142400\ 10^6 \qquad (30)$$

5.2 Normal stresses in the **TIMBER** section

MEd is unkown

Find Med to find the maximum loading for the timber-concrete-composite element

$$> \sigma_2 := \frac{(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,\,2})}{EI_{eff,tot,fin}} \cdot 10\hat{}6 : \#MPa$$

> $\sigma_{m,\,2} := \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,\,2}\right)}{EI_{eff,tot,fin}} \cdot 10\^6 : \#MPa$

Stresses at the **TOP** of the **TIMBER** section

$\#\sigma\, t,\, t\ = -\ \frac{\sigma_2}{f_{t,\,0,\,d}} - \frac{\sigma_{m,\,2}}{f_{m,\,d}} < 1.0$

$\#f_{m,\,d} := \frac{k_{modi,t} \cdot f_{m,\,k,\,t22}}{\gamma_M}$

$\#f_{t,\,d} := \frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}$

$$\#M_{Ed,2}\left(\frac{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin} \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\^6;\right)}{\frac{k_{modi,t} \cdot f_{t,\,0,\,k,\,t22}}{\gamma_M}} + \frac{\left(\frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2 \cdot M_{Ed,2}\right)}{EI_{eff,tot,fin}} \cdot 10\^6\,;\right)}{\frac{k_{modi,t} \cdot f_{m,k,t22}}{\gamma_M}}\right) \le 1.0$$

> $M_3 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_3 := 3.872055938\ 10^7 \qquad \textbf{(31)}$$

>

Stresses at the **BOTTOM** of the **TIMBER** section

$\#\sigma\, t,\, b\ = -\ \frac{\sigma_2}{f_{t,0,d}} + \frac{\sigma_{m,2}}{f_{m,d}} < 1.0$

> $M_4 := solve\left(M_{Ed,2} = \left(\frac{\frac{k_{modi,t}}{\gamma_M}}{\left(-\frac{\left(\gamma_{2,fin} \cdot E_{2,\,fin} \cdot a_{2,\,fin}\right)}{EI_{eff,tot,fin} \cdot f_{t,0,k,t22}} + \frac{\left(0.5 \cdot E_{2,\,fin} \cdot h_2\right)}{EI_{eff,tot,fin} \cdot f_{m,k,t22}}\right)}\right), M_{Ed,2}\right); \#Nmm$

$$M_4 := 3.872055938\ 10^7 \qquad \textbf{(32)}$$

Need to neglect the bending moment for the bottom of concrete (M2)

5.3 The maxiumum loading, Ped, Long-term

> $M_{Ed,\, new} := \dfrac{\min(M_1, M_3, M_4)}{10\hat{\ }6}$: #kNm

> $L_{out} := 0.75$: #m

> $L_{sup} := 2.0$: #m

> $P_{Ed,\, fin} := solve\left(\dfrac{P_{Ed,\, 1} \cdot L_{out}}{2} + \dfrac{1.5 \cdot g_{0,k} \cdot L_{sup}\hat{\ }2}{8} = M_{Ed,\, new}, P_{Ed,\, 1}\right)$; #kN

$$P_{Ed,fin} := 100.8193550 \quad (33)$$

5.4 Verification of the vertical deflection

Where creep is included.

> $w_{permanent} := \dfrac{5 \cdot \left(\dfrac{P_{Ed,\, fin}}{L_{sup}} + f_{d,SLS}\right) \cdot L\hat{\ }4}{384 \cdot EI_{eff,tot,fin}}$;

$$w_{permanent} := 21.62483785 \quad (34)$$

>

> $w_{\lim} := evalf\left(\dfrac{L}{150}\right)$;

$$w_{\lim} := 13.33333333 \quad (35)$$

Verification of the vertical deflection

> $Ver_{deflection} := \dfrac{w_{permanent}}{w_{\lim}}$; # <1.0 NOT OK

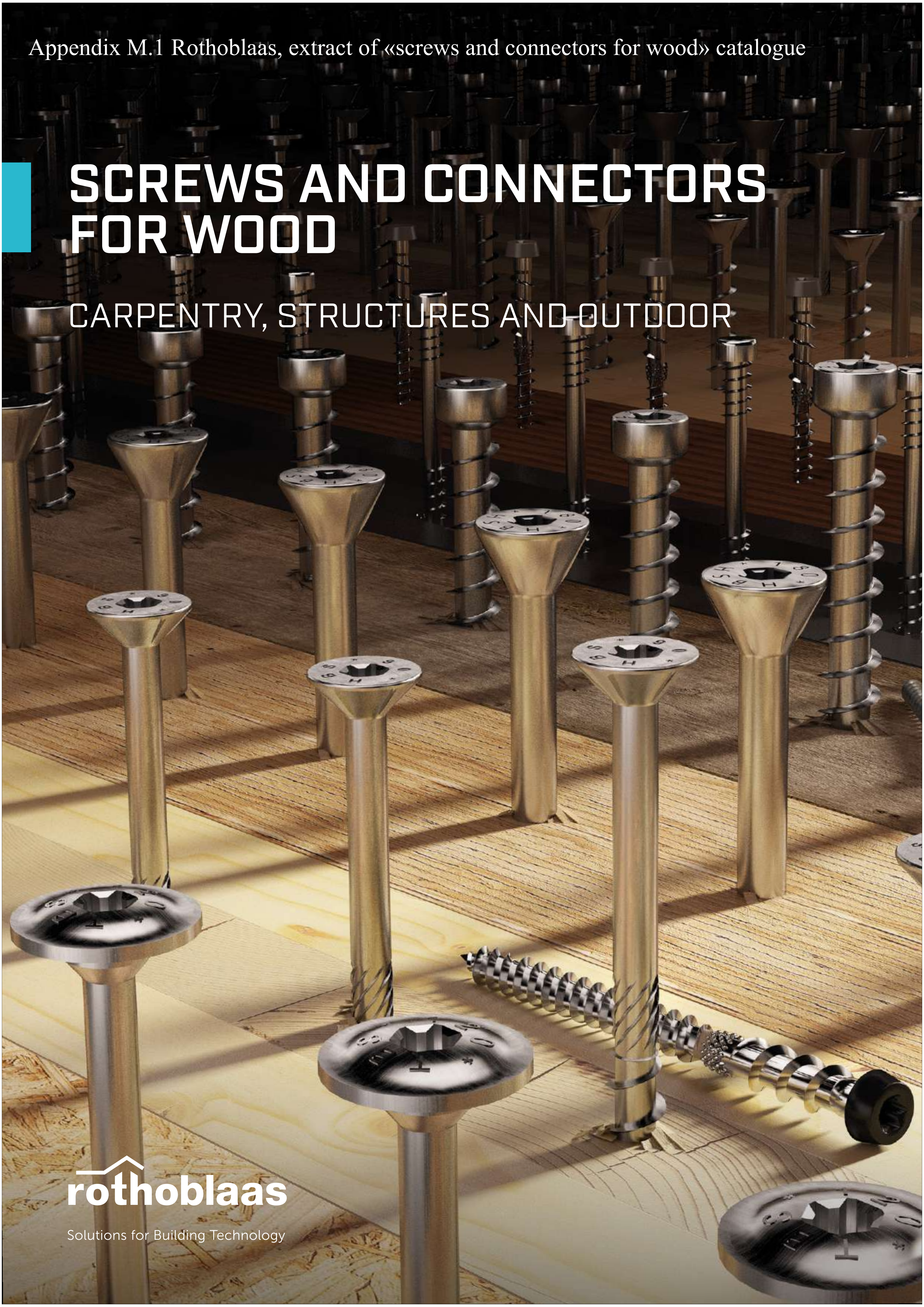
$$Ver_{deflection} := 1.621862839 \quad (36)$$

>

Appendix M. Rothoblaas, extract of “screws and connectors for wood” catalogue

M.1 Rothoblaas, extract of “screws and connectors for wood” catalogue

Appendix M.1 Rothoblaas, extract of «screws and connectors for wood» catalogue



SCREWS AND CONNECTORS FOR WOOD

CARPENTRY, STRUCTURES AND OUTDOOR

rothoblaas

Solutions for Building Technology

CONTENTS

CARPENTRY

STRUCTURES

OUTDOOR

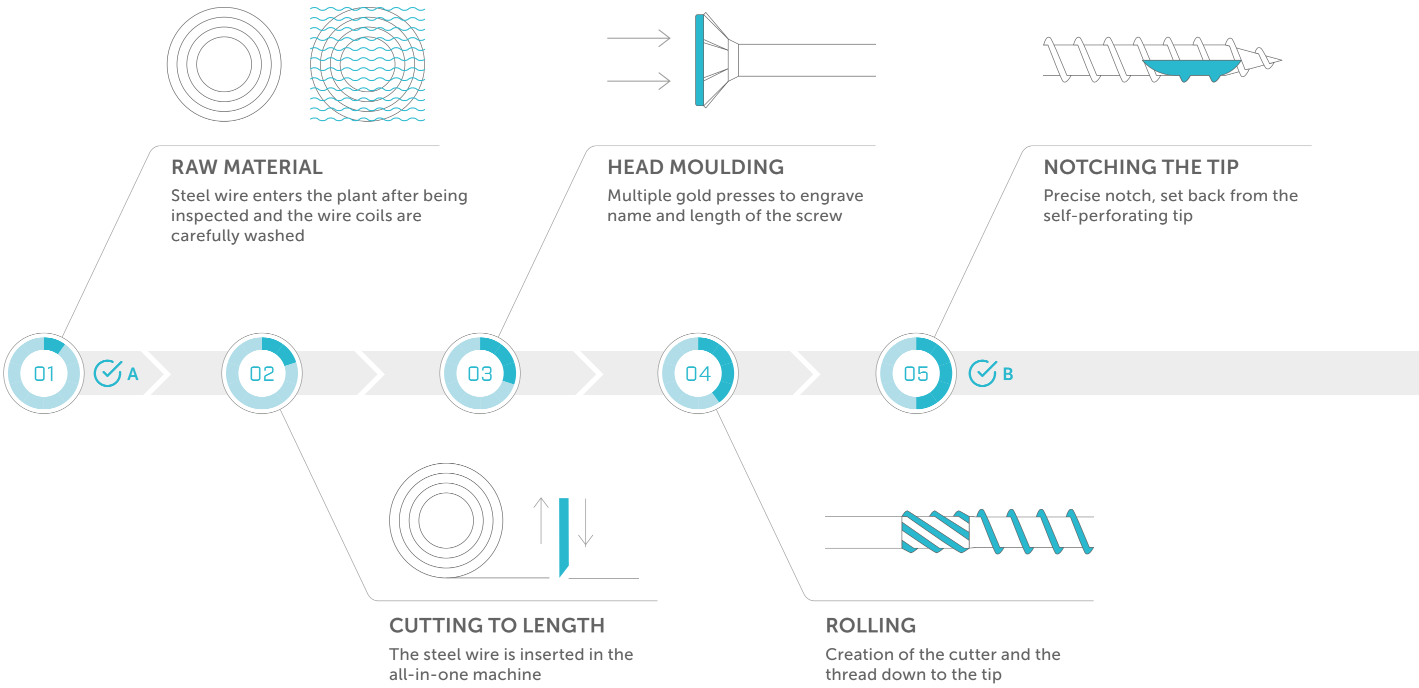
TIMBER-METAL

COMPLEMENTARY PRODUCTS

QUALITY CONTROL

PRODUCTION PHASES CONTROLS

Rothoblaas designs, tests, manufactures, certifies and markets its products under its own name and brand. The manufacturing process is systematically checked during each phase (FPC), the whole procedure strictly monitored and controlled to ensure compliance and quality at each stage.



QUALITY OF THE STEEL

With the steel annealing and tempering process, Rothoblaas screws obtain the perfect balance between resistance (f_{yk} = 1000 N/mm^2) and ductility (excellent possibility of bending), thanks to high-level engineering know-how.

TRACEABILITY

During the production process each screw is assigned an identifying code (batch number) which guarantees the traceability of raw materials before the product is placed on the market.

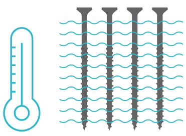
CE - ETA - DoP

As manufacturer, Rothoblaas is responsible for its products covered by ETA.
These products must be provided with CE marking, normally on the label, which ensures legal validity and must show the following information:

1. **Identification of the producer**
2. **ETA number**
3. **Declaration of performance**

CE
1 ----------------------Rotho Blaas
2 ----------------------ETA 11/0030
3 ----------------------DoP: HBS_DoP_110030
(www.rothoblaas.com)

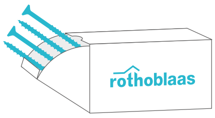
06 ✓ ✓ C D



HEAT TREATMENT/ GALVANIZING AND WAXING

Special furnace hardening process with controlled temperature evolution and zinc plated in an electrolytic tank followed by anti-friction waxing

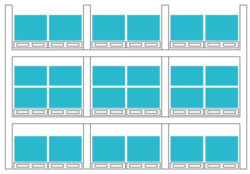
07 ✓ E



PACKAGING AND LABELLING

A mechanized line providing packaging and labelling

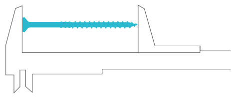
08



STORAGE

Acceptance of the incoming goods and sampling by the Quality Check Laboratory

09 ✓ F



QUALITY CHECK AT ROTHOBLAAS

The FPC procedure continues with a second stage of geometric and mechanical checks carried out at Rothoblaas

10

SELLING AND TRACEABILITY

With the batch number and the selling order it is possible to track all the manufacturing phases: the customer can be sure to obtain a certified quality product

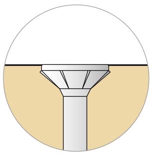
CONTROLS

- **A.** Verification, check and registration of the incoming raw materials
- **B.** Geometric inspection according to regulated tolerances and calibration
- **C.** Mechanical check: ultimate resistance to torsion, tension and bending angle
- **D.** Check on coating thickness and salt spray sample tests
- **E.** Inspection of package and label
- **F.** Application test

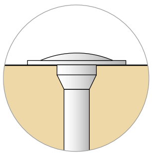
COMPLETE RANGE

"THE IDEAL COMBINATION"

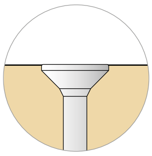
HEAD



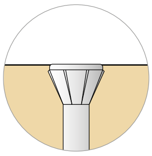
COUNTERSUNK WITH RIBS
HBS, HBS COIL, HBS EVO, HBS S, HBS S BULK, VGS, SCI A2/A4, SBS, SPP



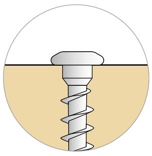
FLANGE
TBS , TBS MAX, TBS EVO



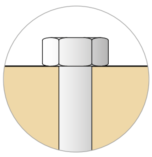
COUNTERSUNK SMOOTH
HTS, DRS, DRT, SKS, SCA A2, SBS A2, SBN, SBN A2



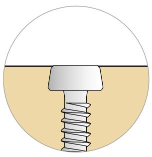
COUNTERSUNK 60°
SHS, SHS AISI410, HBS H



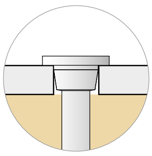
ROUND
LBS



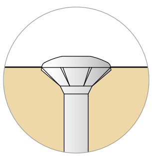
HEXAGONAL
KOP, SKR, VGS Ø13, MTS A2



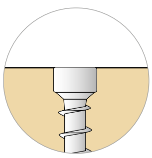
CONE-SHAPED
KKT A4 COLOR, KKT A4, KKT COLOR



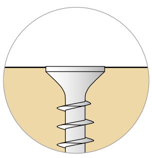
PAN HEAD
HBS P, HBS P EVO, KKF AISI410



CONVEX
EWS A2, EWS AISI410, MCS A2

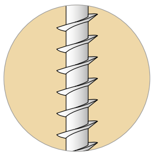


CYLINDRICAL
VGZ, VGZ EVO, VGZ H, DGZ, CTC, MBS, SBD, KKZ A2, KWP A2, KKA AISI410, KKA COLOR

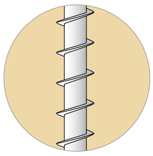


BUGLE
DWS, DWS COIL

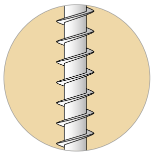
THREAD



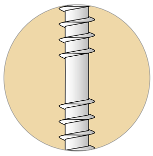
ASYMMETRIC "UMBRELLA"
HBS , HBS COIL , HBS EVO , HBS P, HBS P EVO, TBS, TBS EVO, SCI A2/A4



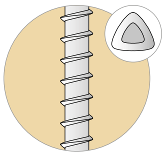
SYMMETRICAL COARSE THREAD
HBS S, HBS S BULK, VGZ, VGZ EVO, VGS, SCA A2



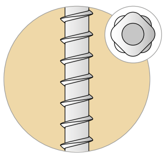
SYMMETRICAL FINE THREAD
HBS H, HTS, SHS, SHS AISI410, LBS, DWS, DWS COIL, KKF AISI410, MCS A2, VGZ H



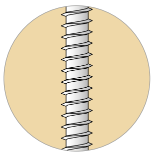
DOUBLE
DGZ, CTC, SBD, KKT A4 COLOR, KKT A4, KKT COLOR, KKZ A2, KWP A2, KKA AISI410



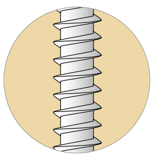
TRILOBULAR
KKT A4 COLOR, KKT A4, KKT COLOR



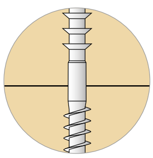
QUADLOBULAR
EWS A2, EWS AISI410



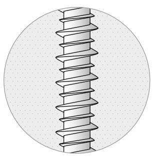
FINE, FOR METAL
KKA AISI 410, KKA COLOR, SBS, SPP, SBS A2, SBN, SBN A2



STANDARD FOR WOOD
KOP, RTR, MTS A2



SPACER
DRS, DRT



HI-LOW (CONCRETE)
MBS, SKR, SKS

TIP

SHARP
HBS (L ≤ 50 mm), HBS COIL (L ≤ 50 mm), HTS, LBS, DRS, DRT, DWS, DWS COIL, KWP A2, SCA A2, MCS A2

SHARP SAW
HBS S, HBS S BULK

SHARP SAW NIBS
VGS Ø13

SHARP 1 CUT
HBS (L > 50 mm), HBS COIL (L > 50 mm), HBS EVO, HBS P, HBS P EVO, TBS, TBS EVO, VGZ, VGZ EVO, VGS, DGZ, CTC, SHS, SHS AISI410, KKT A4 COLOR , KKT A4, EWS A2, EWS AISI410, KKF AISI410, SCI A2/A4

SHARP 2 CUT
KKT COLOR

HARD WOOD (DECKING)
KKZ A2

HARD WOOD (SOLID)
HBS H, VGZ H

ALUMINIUM (DECKING)
KKA AISI410, KKA COLOR

METAL (WITH FINS)
SBS, SBS A2, SPP

METAL (WITHOUT FINS)
SBD, SBN, SBN A2

STANDARD FOR WOOD
MBS, KOP, MTS A2

CONCRETE
SKR, SKS

MATERIALS AND COATINGS

carbon steel + zinc plated
HTS, SHS, HBS, HBS COIL, HBS S, HBS S BULK, TBS, HBS H, HBS P, LBS, KOP, DRS, DRT, MBS, VGZ, VGZ H, VGS, RTR, DGZ, SBD, CTC, SKR, SKS, SBS, SPP, SBN

carbon steel + color coating
KKT COLOR, KKA COLOR

carbon steel + C4 EVO coating
HBS EVO, TBS EVO, HBS P EVO, VGZ EVO, SKR EVO, SKS EVO

AISI410 martensitic stainless steel
KKF AISI410, EWS AISI410, KKA AISI410, SHS AISI410

A2 stainless steel (AISI304 | AISI305)
SCI A2, SCA A2, EWS A2, KKZ A2, KWP A2, SBS A2, SBN A2, MCS A2, MTS A2, WBAZ

A4 stainless steel (AISI316)
KKT A4 COLOR, KKT A4, SCI A4

bi-metal stainless steel + carbon steel
SBS A2

phosphate steel
DWS, DWS COIL

EPDM/PP/PU
XYLOFON WASHER, WBAZ, THERMOWASHER, ISULFIX

KOP

EN 14592

COACH SCREW DIN571

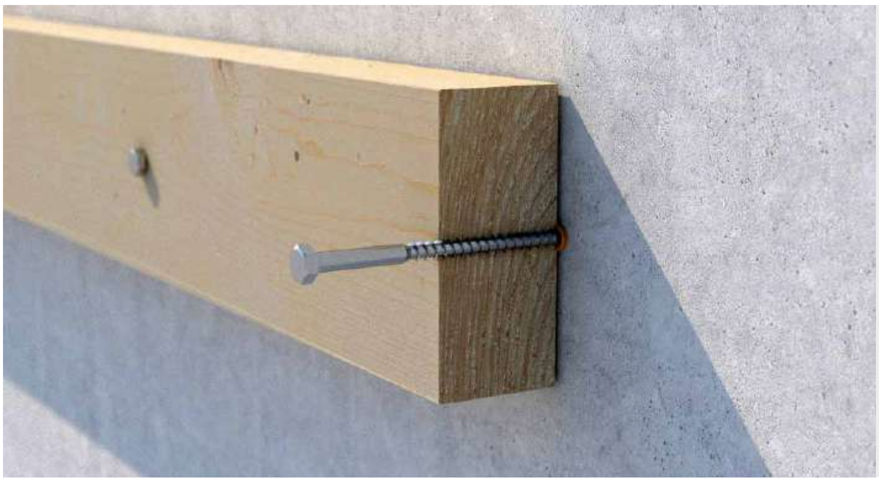
CE MARKING
Screws with the CE mark, in accordance with EN 14592.

HEXAGONAL HEAD
Appropriate for use on plates in steel-timber applications, thanks to its hexagonal head.

OUTDOOR VERSION
Also available in stainless steel A2 | AISI304 for outdoor use (service class 3).

CHARACTERISTICS

FOCUS	coach screw with CE marking
HEAD	hexagonal
DIAMETER	from 8,0 mm to 16,0 mm
LENGTH	from 50 mm to 400 mm

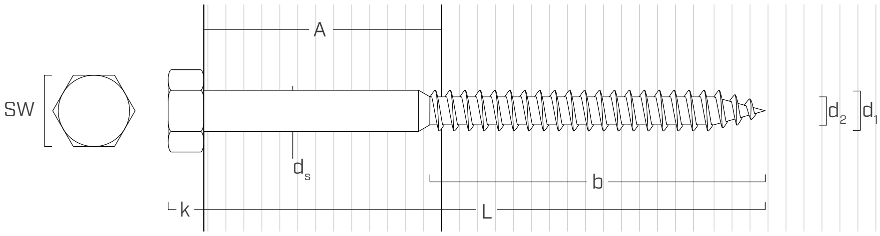


MATERIAL
Available in carbon steel with bright zinc plated and in stainless steel A2.

FIELDS OF USE
- wood based panels
- fibre board and MDF panels
- solid timber
- glulam (Glued Laminated Timber)
- CLT, LVL

Service classes 1 and 2.

GEOMETRY AND MECHANICAL CHARACTERISTICS



Nominal diameter	d_1	[mm]	8	10	12	16
Wrench size	SW	[mm]	13	17	19	24
Tip diameter	d_2	[mm]	5,60	7,00	9,00	12,00
Shank diameter	d_S	[mm]	8,00	10,00	12,00	16,00
Diameter pre-drilling hole - smooth part	d_{V1}	[mm]	8,0	10,0	12,0	16,0
Diameter pre-drilling hole - threaded part	d_{V2}	[mm]	5,5	7,0	8,5	11,0
Thread length	b	[mm]	$\geq$ 0,6 L			
Characteristic yield moment	$M_{y,k}$	[Nmm]	16900	32200	65700	138000
Characteristic withdrawal-resistance parameter	$f_{ax,k}$	[N/mm^2]	12,9	10,6	10,2	10,0
Associated density	ρ_a	[kg/m^3]	400	400	440	360
Characteristic head-pull-through parameter	$f_{head,k}$	[N/mm^2]	22,8	19,8	16,4	16,5
Associated density	ρ_a	[kg/m^3]	440	420	430	430
Characteristic tensile strength	$f_{tens,k}$	[kN]	15,7	23,6	37,3	75,3

CODES AND DIMENSIONS

d_1 [mm]	CODE	L [mm]	pcs
8 SW 13	KOP850	50	100
	KOP860	60	100
	KOP870	70	100
	KOP880	80	100
	KOP8100	100	50
	KOP8120	120	50
	KOP8140	140	50
	KOP8160	160	50
	KOP8180	180	50
	KOP8200	200	50
10 SW 17	KOP1050	50	50
	KOP1060	60	50
	KOP1080	80	50
	KOP10100	100	50
	KOP10120	120	50
	KOP10140	140	50
	KOP10150	150	50
	KOP10160	160	50
	KOP10180	180	50
	KOP10200	200	50
	KOP10220	220	50
	KOP10240	240	50
	KOP10260	260	50
	KOP10280	280	50
	KOP10300	300	50
12 SW 19	KOP1250	50	50
	KOP1260	60	50
	KOP1270	70	50
	KOP1280	80	50
	KOP1290	90	25
	KOP12100	100	25
	KOP12120	120	25
	KOP12140	140	25

d_1 [mm]	CODE	L [mm]	pcs
12 SW 19	KOP12150	150	25
	KOP12160	160	25
	KOP12180	180	25
	KOP12200	200	25
	KOP12220	220	25
	KOP12240	240	25
	KOP12260	260	25
	KOP12280	280	25
	KOP12300	300	25
	KOP12320	320	25
	KOP12340	340	25
	KOP12360	360	25
	KOP12380	380	25
	KOP12400	400	25
16 SW 24	KOP1680	80	25
	KOP16100	100	25
	KOP16120	120	25
	KOP16140	140	25
	KOP16150	150	25
	KOP16160	160	25
	KOP16180	180	25
	KOP16200	200	25
	KOP16220	220	25
	KOP16240	240	25
	KOP16260	260	25
	KOP16280	280	25
	KOP16300	300	25
	KOP16320	320	25
	KOP16340	340	25
	KOP16360	360	25
	KOP16380	380	25
	KOP16400	400	25

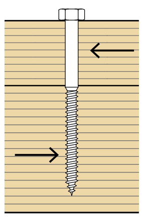
A2 | AISI304 VERSION CODES AND DIMENSIONS

d_1 [mm]	CODE	L [mm]	pcs
8 SW 13	AI571850	50	100
	AI571860	60	100
	AI571880	80	100
	AI5718100	100	50
	AI5718120	120	50
10 SW 17	AI5711050	50	50
	AI5711060	60	50
	AI5711080	80	50
	AI57110100	100	50
	AI57110120	120	50
	AI57110140	140	50
	AI57110160	160	50
	AI57110180	180	50
	AI57110200	200	50

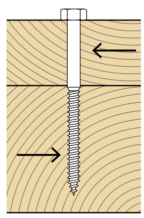
d_1 [mm]	CODE	L [mm]	pcs
12 SW 19	AI57112100	100	25
	AI57112120	120	25
	AI57112140	140	25
	AI57112160	160	25
	AI57112180	180	25

The stainless steel screws have not been granted the CE mark.

MINIMUM DISTANCES FOR SHEAR LOADS



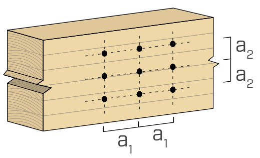
Load-to-grain angle α = 0°



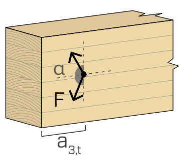
Load-to-grain angle α = 90°

		SCREWS INSERTED WITH PRE-DRILLING HOLES					SCREWS INSERTED WITH PRE-DRILLING HOLES				
			8	10	12	16		8	10	12	16
a_1	[mm]	5·d	40	50	60	80	4·d	32	40	48	64
a_2	[mm]	4·d	32	40	48	64	4·d	32	40	48	64
$a_{3,t}$	[mm]	7·d (min. 80 mm)	80	80	84	112	7·d (min. 80 mm)	80	80	84	112
$a_{3,c}$	[mm]	4·d	32	40	48	64	7·d	56	70	84	112
$a_{4,t}$	[mm]	3·d	24	30	36	48	4·d	32	40	48	64
$a_{4,c}$	[mm]	3·d	24	30	36	48	3·d	24	30	36	48

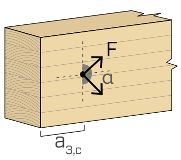
d = nominal nail diameter



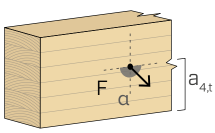
stressed end
-90° < α < 90°



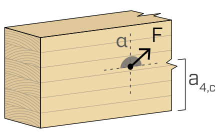
unloaded end
90° < α < 270°



stressed edge
0° < α < 180°



unload edge
180° < α < 360°

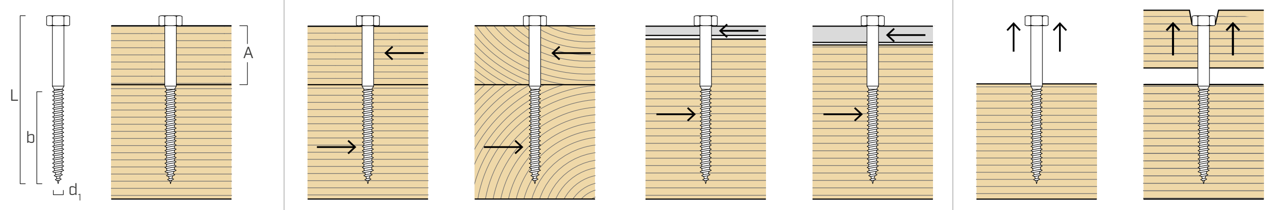


NOTES:

- Minimum distances in accordance with EN 1995:2014.
- For KOP screws with a diameter of d > 6 mm, a pre-drill is required as per EN 1995:2014:
  - pre-drill hole for smooth part of the shank, dimensions matching that of the shank itself, depth equal to the length of the shank.
  - pre-drill hole for the threaded portion, equal to approximately 70% of the shank diameter.

STATIC VALUES

CHARACTERISTIC VALUES
EN 1995:2014



geometry				SHEAR: timber-to-timber $\alpha = 0°$ [1]	SHEAR: timber-to-timber $\alpha = 90°$ [2]	SHEAR: thin steel-timber plate [3]		SHEAR: thick steel-timber plate [4]		TENSION: thread withdrawal[5]	TENSION: head pull-through [6]
d_1 [mm]	L [mm]	b[7] [mm]	A [mm]	$R_{V,k}$ [kN]	$R_{V,k}$ [kN]		$R_{V,k}$ [kN]		$R_{V,k}$ [kN]	$R_{ax,k}$ [kN]	$R_{head,k}$ [kN]
	50	30	20	2,96	2,23		2,64		3,75	2,78	3,54
	60	36	24	3,28	2,68		3,22		4,38	3,34	3,54
	70	42	28	3,55	2,87		3,51		4,56	3,90	3,54
	80	48	32	3,78	3,01		3,65		4,70	4,45	3,54
	100	60	40	3,96	3,32		3,93		4,98	5,56	3,54
8	120	72	48	3,96	3,42	$S_{PLATE} \le 4$ mm	4,20	$S_{PLATE} \ge 8$ mm	5,25	6,68	3,54
	140	84	56	3,96	3,42		4,48		5,53	7,79	3,54
	160	96	64	3,96	3,42		4,76		5,81	8,90	3,54
	180	108	72	3,96	3,42		5,04		6,09	10,02	3,54
	200	120	80	3,96	3,42		5,07		6,37	11,13	3,54
	50	30	20	3,48	2,56		3,10		4,65	2,86	5,45
	60	36	24	4,18	3,07		3,79		5,30	3,43	5,45
	80	48	32	5,01	4,01		4,97		6,56	4,57	5,45
	100	60	40	5,78	4,56		5,26		6,84	5,72	5,45
	120	72	48	6,05	4,92		5,54		7,13	6,86	5,45
	140	84	56	6,05	5,19		5,83		7,42	8,00	5,45
	150	90	60	6,05	5,19		5,97		7,56	8,57	5,45
10	160	96	64	6,05	5,19	$S_{PLATE} \le 5$ mm	6,12	$S_{PLATE} \ge 10$ mm	7,70	9,14	5,45
	180	108	72	6,05	5,19		6,40		7,99	10,29	5,45
	200	120	80	6,05	5,19		6,69		8,27	11,43	5,45
	220	132	88	6,05	5,19		6,97		8,56	12,57	5,45
	240	144	96	6,05	5,19		7,26		8,85	13,72	5,45
	260	156	104	6,05	5,19		7,54		9,13	14,86	5,45
	280	168	112	6,05	5,19		7,66		9,42	16,00	5,45
	300	180	120	6,05	5,19		7,66		9,70	17,15	5,45

NOTES:

[1] The characteristic shear resistance values are calculated using an angle α between the strength and the grain of 0°.

[2] The characteristic shear resistance values are calculated using an angle α between the strength and the grain of 90°.

[3] The shear resistance characteristics are calculated considering the case of a thin plate ($S_{PLATE} \le 0,5\ d_1$).

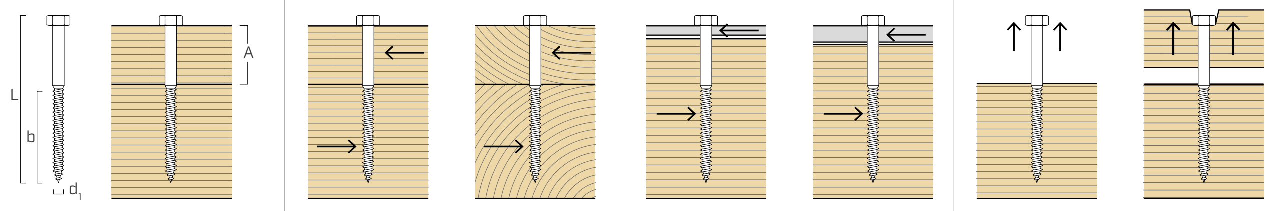
[4] The shear resistance characteristics are calculated considering the case of a thick plate ($S_{PLATE} \ge d_1$).

[5] The axial thread withdrawal resistance was calculated considering a 90° angle between the grain and the connector and for a fixing length of b.

[6] The axial resistance to head pull-through was calculated using wood elements. In the case of steel-timber connections, generally the steel tensile strength is binding with respect to head separation or pull-through.

[7] During calculation, a thread length of b = 0,6 L is used, with the exception of the measures (*).

STATIC VALUES



				SHEAR				TENSION	
geometry				timber-to-timber α = 0° [1]	timber-to-timber α = 90° [2]	thin steel-timber plate [3]	thick steel-timber plate [4]	thread withdrawal[5]	head pull-through [6]
d_1	L	b[7]	A	$R_{V,k}$	$R_{V,k}$	$R_{V,k}$	$R_{V,k}$	$R_{ax,k}$	$R_{head,k}$
[mm]	[mm]	[mm]	[mm]	[kN]	[kN]	[kN] ($S_{PLATE} \le 6$ mm)	[kN] ($S_{PLATE} \ge 12$ mm)	[kN]	[kN]
12	50	30	20	4,01	2,89	3,49	6,10	3,06	5,54
	60	36	24	4,81	3,46	4,28	6,67	3,67	5,54
	70	42	28	5,61	4,04	5,07	7,36	4,28	5,54
	80	48	32	6,42	4,62	5,86	8,12	4,89	5,54
	90	54	36	6,92	5,19	6,66	8,94	5,50	5,54
	100	60	40	7,20	5,63	7,40	9,78	6,12	5,54
	120	72	48	7,82	6,02	7,70	10,13	7,34	5,54
	140	84	56	8,50	6,41	8,01	10,44	8,56	5,54
	150	90	60	8,64	6,62	8,16	10,59	9,17	5,54
	160	96	64	8,64	6,84	8,31	10,74	9,78	5,54
	180	108	72	8,64	7,25	8,62	11,05	11,01	5,54
	200	120	80	8,64	7,25	8,92	11,36	12,23	5,54
	220	132	88	8,64	7,25	9,23	11,66	13,45	5,54
	240	144	96	8,64	7,25	9,54	11,97	14,68	5,54
	260	156	104	8,64	7,25	9,84	12,27	15,90	5,54
	280	168	112	8,64	7,25	10,15	12,58	17,12	5,54
	300	180	120	8,64	7,25	10,45	12,88	18,35	5,54
	320	192	128	8,64	7,25	10,76	13,19	19,57	5,54
	340	195 *	145	8,64	7,25	10,84	13,27	19,88	5,54
	360	195 *	165	8,64	7,25	10,84	13,27	19,88	5,54
	380	195 *	185	8,64	7,25	10,84	13,27	19,88	5,54
	400	195 *	205	8,64	7,25	10,84	13,27	19,88	5,54

NOTES:

(1) The characteristic shear resistance values are calculated using an angle α between the strength and the grain of 0°.

(2) The characteristic shear resistance values are calculated using an angle α between the strength and the grain of 90°.

(3) The shear resistance characteristics are calculated considering the case of a thin plate ($S_{PLATE} \le 0,5\ d_1$).

(4) The shear resistance characteristics are calculated considering the case of a thick plate ($S_{PLATE} \ge d_1$).

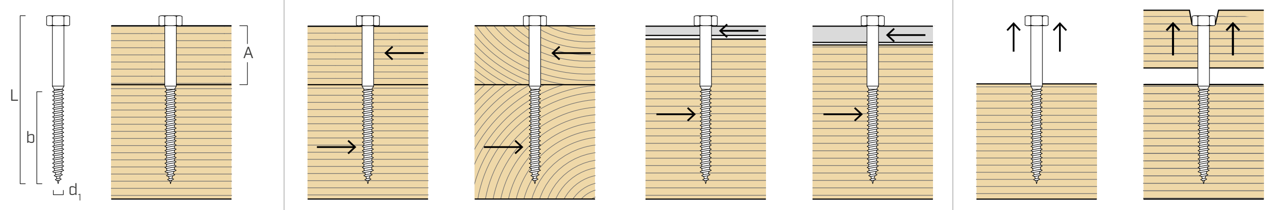
(5) The axial thread withdrawal resistance was calculated considering a 90° angle between the grain and the connector and for a fixing length of b.

(6) The axial resistance to head pull-through was calculated using wood elements. In the case of steel-timber connections, generally the steel tensile strength is binding with respect to head separation or pull-through.

(7) During calculation, a thread length of b = 0,6 L is used, with the exception of the measures (*).

STATIC VALUES

CHARACTERISTIC VALUES
EN 1995:2014



geometry				SHEAR				TENSION	
geometry				timber-to-timber α = 0° (1)	timber-to-timber α = 90° (2)	thin steel-timber plate (3)	thick steel-timber plate (4)	thread withdrawal(5)	head pull-through (6)
d_1 [mm]	L [mm]	b(7) [mm]	A [mm]	$R_{V,k}$ [kN]	$R_{V,k}$ [kN]	$R_{V,k}$ [kN] $S_{PLATE} \leq 8$ mm	$R_{V,k}$ [kN] $S_{PLATE} \geq 16$ mm	$R_{ax,k}$ [kN]	$R_{head,k}$ [kN]
16	80	48	32	8,49	6,03	6,99	11,17	7,51	8,89
	100	60	40	10,48	7,42	8,93	13,02	9,39	8,89
	120	72	48	11,43	8,46	10,87	15,10	11,26	8,89
	140	84	56	12,18	9,28	12,70	16,59	13,14	8,89
	150	90	60	12,58	9,50	12,93	16,83	14,08	8,89
	160	96	64	12,99	9,72	13,16	17,06	15,02	8,89
	180	108	72	13,86	10,20	13,63	17,53	16,89	8,89
	200	120	80	14,09	10,72	14,10	18,00	18,77	8,89
	220	132	88	14,09	11,26	14,57	18,47	20,65	8,89
	240	144	96	14,09	11,63	15,04	18,94	22,53	8,89
	260	156	104	14,09	11,63	15,51	19,41	24,40	8,89
	280	168	112	14,09	11,63	15,98	19,88	26,28	8,89
	300	180	120	14,09	11,63	16,45	20,35	28,16	8,89
	320	192	128	14,09	11,63	16,92	20,82	30,04	8,89
	340	204	136	14,09	11,63	17,39	21,29	31,91	8,89
	360	205 *	155	14,09	11,63	17,43	21,33	32,07	8,89
	380	205 *	175	14,09	11,63	17,43	21,33	32,07	8,89
	400	205 *	195	14,09	11,63	17,43	21,33	32,07	8,89

NOTES:

(1) The characteristic shear resistance values are calculated using an angle α between the strength and the grain of 0°.

(2) The characteristic shear resistance values are calculated using an angle α between the strength and the grain of 90°.

(3) The shear resistance characteristics are calculated considering the case of a thin plate ($S_{PLATE} \leq 0{,}5\ d_1$).

(4) The shear resistance characteristics are calculated considering the case of a thick plate ($S_{PLATE} \geq d_1$).

(5) The axial thread withdrawal resistance was calculated considering a 90° angle between the grain and the connector and for a fixing length of b.

(6) The axial resistance to head pull-through was calculated using wood elements. In the case of steel-timber connections, generally the steel tensile strength is binding with respect to head separation or pull-through.

(7) During calculation, a thread length of b = 0,6 L is used, with the exception of the measures (*).

GENERAL PRINCIPLES:

- Characteristic values according to EN 1995:2014.
- Design values can be obtained from characteristic values as follows:

$$R_d = \frac{R_k \cdot k_{mod}}{\gamma_m}$$

The coefficients γ_m and k_{mod} should be taken according to the current regulations used for the calculation.

- For the calculation process a timber density ρ_k = 350 kg/m³ has been considered.
- Values were calculated considering the minimum threaded part as being completely inserted into the wood.
- Dimensioning and verification of timber elements and steel plates must be carried out separately.
- The characteristic shear resistance values are calculated for screws inserted with pre-drilling holes.

BIT INCLUDED MY PROJECT SOFTWARE ETA 19/0244

CTC

CONNECTOR FOR TIMBER-CONCRETE FLOORS

CERTIFICATION
Timber-to-concrete fastener with specific CE certification according to ETA 19/0244, Tested and calculated with parallel and crossed arrangement of 45° and 30° connectors, with and without wooden planking.

RAPID DRY SYSTEM
Approved system, self-drilling, reversible, fast and minimally invasive. Optimum static and noise performances, both for new projects and structural restoration.

COMPLETE RANGE
Self-perforating tip with notch and countersunk cylindrical head. Available in two diameters (7 and 9 mm) and two lengths (160 and 240 mm) to optimize the number of fasteners.

INSTALLATION INDICATOR
During installation, the under head counter-thread serves as "correct installation" indicator and increases the fastener tightness inside the concrete.

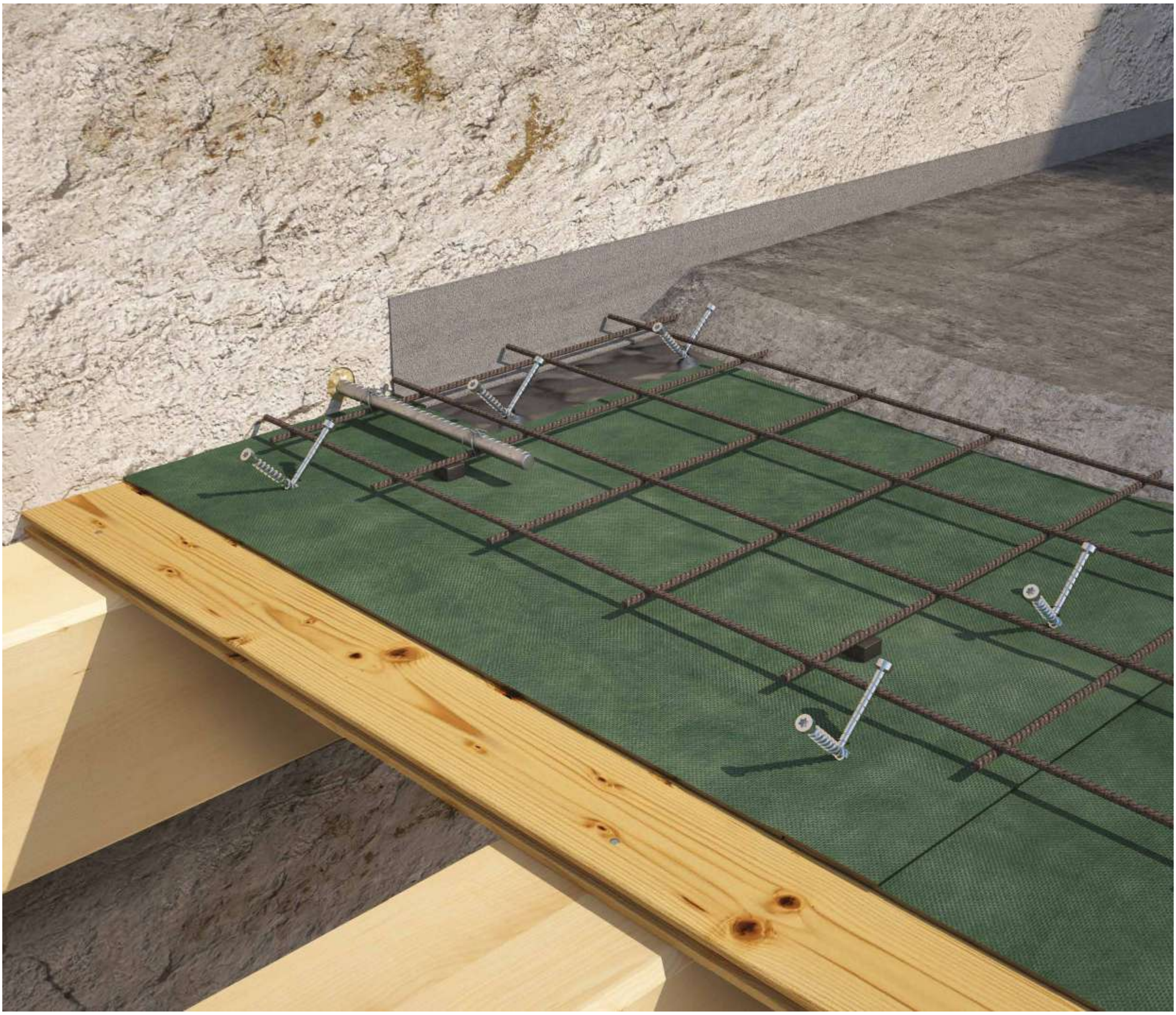
CHARACTERISTICS

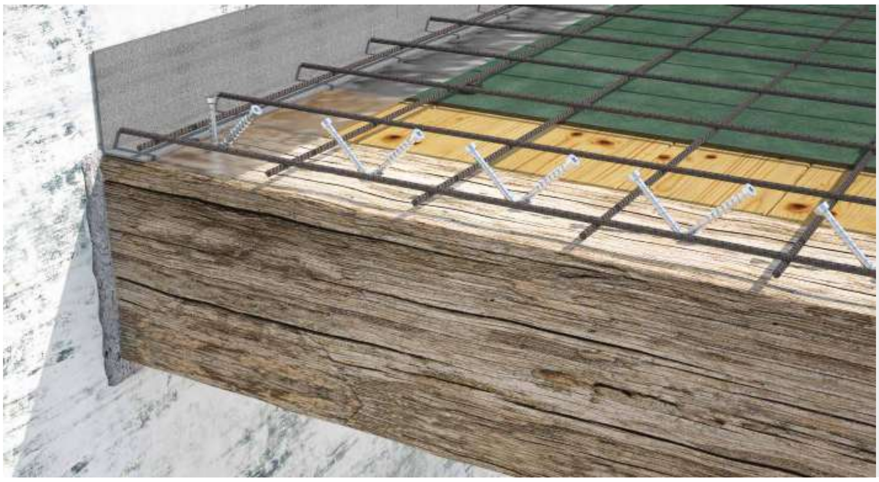
FOCUS	CE marking, timber-concrete
HEAD	cylindrical, countersunk
DIAMETER	7,0 | 9,0 mm
LENGTH	160 | 240 mm

MATERIAL
Carbon steel with bright zinc plated.

FIELDS OF USE
Screw connection system for timber-to-concrete floors, approved for:
- wood based panels
- solid timber and glulam
- CLT, LVL
- high density woods

Service classes 1 and 2.



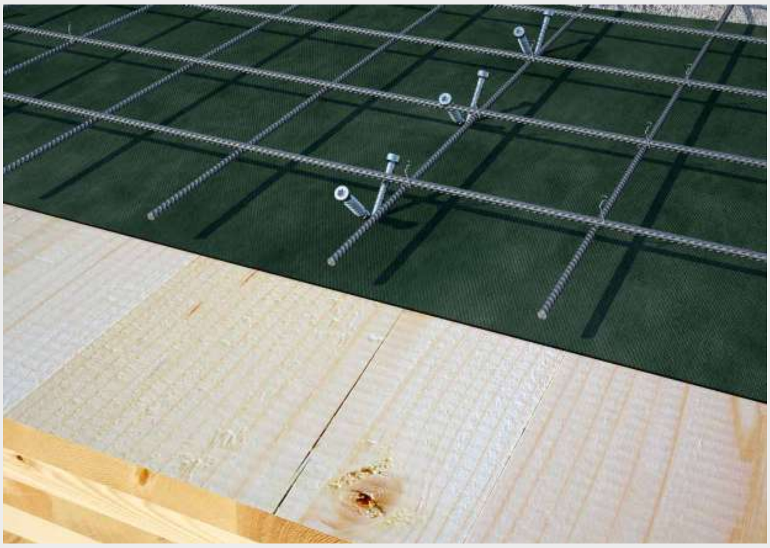


TIMBER-TO-CONCRETE

Ideal for composite floors and for renovation of existing floors. Stiffness values also calculated in the presence of vapour barrier sheet or soundproofing layer.

STRUCTURAL RESTORATION

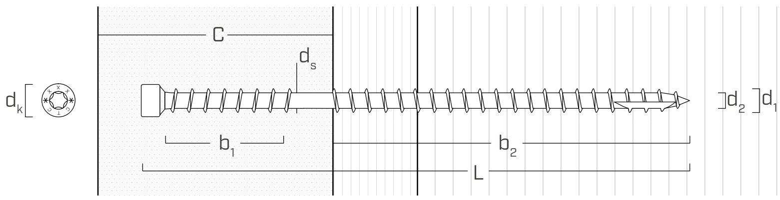
Values also tested, certified and calculated for high density woods. Certification specific for application in timber-concrete structures.



Composite timber-concrete floors on CLT panel with 45° connectors arranged in a single row.

Composite timber-concrete floors with 30° connectors arranged in a double row.

GEOMETRY AND MECHANICAL CHARACTERISTICS



Nominal diameter		d_1	[mm]	7	9
Head diameter		d_k	[mm]	9,50	11,50
Tip diameter		d_2	[mm]	4,60	5,90
Shank diameter		d_s	[mm]	5,00	6,50
Pre-drilling hole diameter		d_v	[mm]	4,0	5,0
Characteristic yield moment		$M_{y,k}$	[Nmm]	20000	38000
Characteristic withdrawal-resistance parameter		$f_{ax,k}$	[N/mm²]	11,3	11,3
Associated density		ρ_a	[kg/m³]	350	350
Characteristic tensile strength		$f_{tens,k}$	[kN]	20,0	30,0
Withdrawal-resistance characteristic - concrete	Crossed connectors at a 45° angle, with soundproofing layer(1)	$F_{ax,concrete,Rk}$	[kN]	10,0	10,0
	Parallel connectors at a 45° angle, with soundproofing layer(1)				
	Parallel connectors at a 30° angle, with soundproofing layer(1)				
	Parallel connectors at a 45° angle, without soundproofing layer	$F_{ax,concrete,Rk}$	[kN]	15,0	15,0
Coefficient of friction		μ	[-]	0,25	0,25

(1) Resilient underscreed foil, in bitumen and polyester felt, like SILENT FLOOR.

GENERAL PRINCIPLES:

- The design shear strength of each crossed connector is the minimum between the timber design shear strength ($R_{ax,d}$), the concrete design shear strength ($R_{ax,concrete,d}$) and the steel design shear strength ($R_{tens,d}$):

$$R_{v,Rd} = (\cos\alpha + \mu \cdot \sin\alpha) \cdot \min \begin{cases} F_{ax,\alpha,Rd} \\ f_{tens,d} \\ F_{ax,concrete,Rd} \end{cases}$$

The friction component μ can be considered only in arrangement with inclined screws (30° e 45°) and without the soundproofing layer.

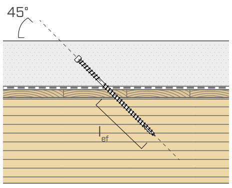
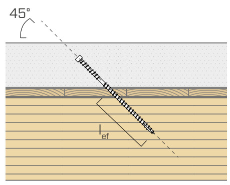
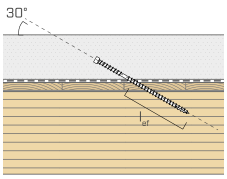
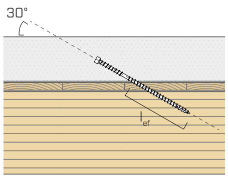
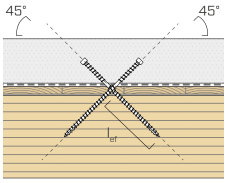
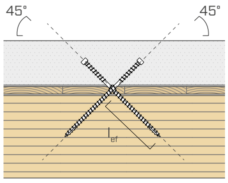
- For the mechanical strength values and the geometry of the screws, reference was made to ETA-19/0244,

CODES AND DIMENSIONS

d_1 [mm]	CODE	L [mm]	b_1 [mm]	b_2 [mm]	pcs
7 TX 30	**CTC7160**	160	40	110	100
	CTC7240	240	40	190	100

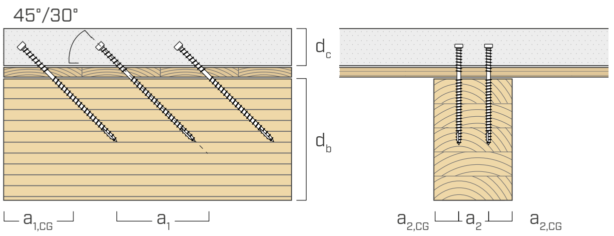
d_1 [mm]	CODE	L [mm]	b_1 [mm]	b_2 [mm]	pcs
9 TX 40	**CTC9160**	160	40	110	100
	CTC9240	240	40	190	100

SLIP MODULUS K_{ser}

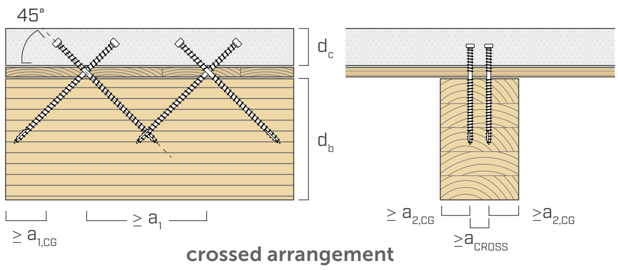
connector arrangement with soundproofing layer (1)	K_{ser} [N/mm]		connector arrangement without soundproofing layer (1)	K_{ser} [N/mm]	
	CTC Ø7	CTC Ø9		CTC Ø7	CTC Ø9
 **45° parallels**	16 l_{ef}	22 l_{ef}	 **45° parallels**	48 l_{ef}	60 l_{ef}
 **30° parallels**	48 l_{ef}	48 l_{ef}	 **30° parallels**	80 l_{ef}	80 l_{ef}
 **45° crossed**	70 l_{ef}	100 l_{ef}	 **45° crossed**	70 l_{ef}	100 l_{ef}

(1) Resilient underscreed foil, in bitumen and polyester felt, like SILENT FLOOR.
The K_{ser} slip modulus is to be considered as relating to a single inclined connector or a pair of crossed connectors subject to a parallel force at the slip surface.
l_{ef} = depth of CTC connector penetration into timber element, in millimetres.

MINIMUM DISTANCES FOR AXIAL STRESSES [1]



parallel arrangement



crossed arrangement

		7	9
a_1	[mm]	130·sin(α)	130·sin(α)
a_2	[mm]	35	45
$a_{1,CG}$	[mm]	85	85
$a_{2,CG}$	[mm]	32	37
a_{CROSS}	[mm]	11	14

d_c = thickness of concrete slab (50 mm ≤ d_c ≤ 0,7 d_b)
d_b = height of wooden beam (d_b ≥ 100 mm)

NOTES:

(1) The minimum distances for connectors stressed axially are compliant with ETA-19/0244.

STATIC VALUES

CALCULATION STANDARD
NTC 2018 - UNI EN 1995:2014

PRELIMINARY SIZING OF CTC CONNECTORS FOR TIMBER- CONCRETE FLOORS

CALCULATION EXAMPLE

distance between joists = 660 mm

concrete slab thickness C20/25 = 50 mm

strain limit w_{ist} = l/400

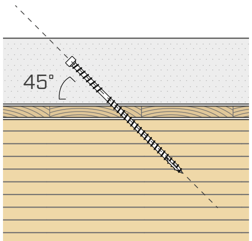
$w_{net,fin}$ = l/250

LOADS

own weight (g_{k1}) = timber beam + wooden planking + concrete slab

permanent non-structural load (g_{k2}) = 2 kN/m²

variable overload (q_k) = 2 kN/m²

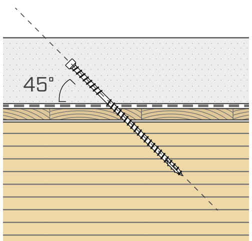


Installation at a 45° angle, without soundproofing layer.

CONNECTOR CTC Ø7 - Glulam GL 24h (EN 14080:2013)

Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	3,5	4	4,5	5	5.5	6
120 x 160	no. pairs per beam	8	10	20	30			
	CTC	7x160	7x240	7x240	7x240	-	-	-
	pitch [mm]	500/500	250/500	160/400	220/440			
	no. connectors/m²	4,0	4,3	7,6	10,1			
120 x 200	no. pairs per beam		10	12	20	30		
	CTC	-	7x160	7x240	7x240	7x240	-	-
	pitch [mm]		300/500	250/500	160/320	130/260		
	no. connectors/m²		4,3	4,5	6,7	9,1		
140 x 200	no. pairs per beam			10	20	30	34	
	CTC	-	-	7x240	7x240	7x240	7x240	-
	pitch [mm]			300/500	180/360	130/260	110/220	
	no. connectors/m²			3,8	6,7	9,1	9,4	
140 x 240	no. pairs per beam				12	20	30	36
	CTC	-	-	-	7x240	7x240	7x240	7x240
	pitch [mm]				300/500	200/400	150/300	120/240
	no. connectors/m²				4,0	6,1	8,3	9,1

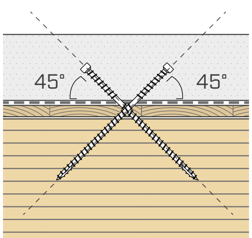


Installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø7 - Glulam GL 24h (EN 14080:2013)

Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	3,5	4	4,5	5	5.5	6
120 x 160	no. pairs per beam	10	14	38	30			
	CTC	7x160	7x240	7x240	7x240	-	-	-
	pitch [mm]	300/500	180/500	100/100	220/440			
	no. connectors/m²	5,1	6,1	14,4	10,1			
120 x 200	no. pairs per beam		8	12	24	56		
	CTC	-	7x160	7x240	7x240	7x240	-	-
	pitch [mm]		500/500	250/500	120/240	160/160 (1)		
	no. connectors/m²		3,5	4,5	8,1	17,0		
140 x 200	no. pairs per beam			10	22	54	90	
	CTC	-	-	7x240	7x240	7x240	7x240	-
	pitch [mm]			300/500	200/200	150/200 (1)	150/200 (2)	
	no. connectors/m²			3,8	7,4	16,4	24,8	
140 x 240	no. pairs per beam				8	16	34	64
	CTC	-	-	-	7x240	7x240	7x240	7x240
	pitch [mm]				500/500	300/500	140/200	150/200 (1)
	no. connectors/m²				2,7	4,8	9,4	16,2



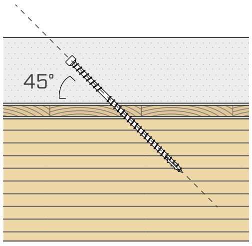
Crossed installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø7 - Glulam GL 24h (EN 14080:2013)

Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	3,5	4	4,5	5	5.5	6
120 x 160	no. pairs per beam	16	20	40	48			
	CTC	7x160	7x240	7x240	7x240	-	-	-
	pitch [mm]	500/500	250/500	160/320	120/400			
	no. connectors/m²	8,1	8,7	15,2	16,2			
120 x 200	no. pairs per beam		16	24	40	48	60	
	CTC	-	7x240	7x240	7x240	7x240	7x240	-
	pitch [mm]		400/500	250/500	180/400	150/400	120/400	
	no. connectors/m²		6,9	9,1	13,5	14,5	16,5	
140 x 200	no. pairs per beam			20	28	48	60	88
	CTC	-	-	7x240	7x240	7x240	7x240	7x240
	pitch [mm]			280/500	200/500	150/400	120/400	100/200
	no. connectors/m²			7,6	9,4	14,5	16,5	22,2
140 x 240	no. pairs per beam				24	40	52	64
	CTC	-	-	-	7x240	7x240	7x240	7x240
	pitch [mm]				300/500	200/500	150/400	120/400
	no. connectors/m²				8,1	12,1	14,3	16,2

STATIC VALUES

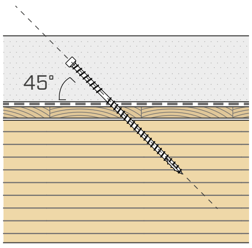


Installation at a 45° angle, without soundproofing layer.

CONNECTOR CTC Ø9 - Glulam GL 24h (EN 14080:2013)

Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	8	10	16	24			
	CTC	9x160	9x240	9x240	9x240	-	-	-
	pitch [mm]	450/500	250/500	150/500	120/300			
	no. connectors/m²	4,0	4,3	6,1	8,1			
120 x 200	no. pairs per beam		8	12	20	24	34	
	CTC	-	9x240	9x240	9x240	9x240	9x240	-
	pitch [mm]		450/500	250/500	180/400	140/400	110/250	
	no. connectors/m²		3,5	4,5	6,7	7,3	9,4	
140 x 200	no. pairs per beam			10	14	22	34	46
	CTC	-	-	9x240	9x240	9x240	9x240	9x240
	pitch [mm]			300/500	200/500	160/500	120/300	180/350
	no. connectors/m²			3,8	4,7	6,7	9,4	11,6
140 x 240	no. pairs per beam				12	20	24	32
	CTC	-	-	-	9x240	9x240	9x240	9x240
	pitch [mm]				300/500	200/500	160/500	120/400
	no. connectors/m²				4,0	6,1	6,6	8,1

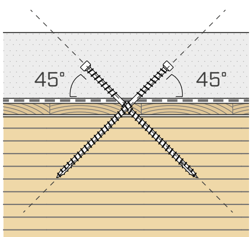


Installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø9 - Glulam GL 24h (EN 14080:2013)

Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	8	10	30				
	CTC	9x160	9x240	9x240	-	-	-	-
	pitch [mm]	500/500	250/500	100/200				
	no. connectors/m²	4,0	4,3	11,4				
120 x 200	no. pairs per beam		8	10	24	60		
	CTC	-	9x240	9x240	9x240	9x240	-	-
	pitch [mm]		400/500	280/500	130/300	140/160		
	no. connectors/m²		3,5	3,8	8,1	18,2		
140 x 200	no. pairs per beam			10	40	52	66	
	CTC	-	-	9x240	9x240	9x240	9x240	-
	pitch [mm]			300/500	200/200	160/200	200/300	
	no. connectors/m²			3,8	13,5	15,8	18,2	
140 x 240	no. pairs per beam				12	22	36	68
	CTC	-	-	-	9x240	9x240	9x240	9x240
	pitch [mm]				300/500	180/400	210/420	140/200
	no. connectors/m²				4,0	6,7	9,9	17,2



Crossed installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø9 - Glulam GL 24h (EN 14080:2013)

Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	16	24	40				
	CTC	9x160	9x160	9x160	-	-	-	-
	pitch [mm]	500/500	250/500	150/300				
	no. connectors/m²	8,1	10,4	15,2				
120 x 200	no. pairs per beam		16	24	40	52		
	CTC	-	9x160	9x160	9x160	9x240	-	-
	pitch [mm]		400/400	250/500	180/360	130/300		
	no. connectors/m²		6,9	9,1	13,5	15,8		
140 x 200	no. pairs per beam			24	40	60	68	
	CTC	-	-	9x160	9x160	9x240	9x240	-
	pitch [mm]			250/500	180/360	130/260	120/240	
	no. connectors/m²			9,1	13,5	18,2	18,7	
140 x 240	no. pairs per beam				32	48	60	72
	CTC	-	-	-	9x160	9x240	9x240	9x240
	pitch [mm]				300/500	150/300	140/280	120/240
	no. connectors/m²				10,8	14,5	16,5	18,2

NOTES:

(1) Connectors placed in two rows.

(2) Connectors placed in three rows.

For different calculation configurations, the MyProject software is available (www.rothoblaas.com).

STATIC VALUES

CALCULATION STANDARD
EN 1995:2014

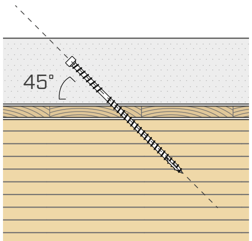
PRELIMINARY SIZING OF VB CONNECTORS FOR TIMBER- CONCRETE FLOORS

CALCULATION EXAMPLE

distance between joists = 660 mm
concrete slab thickness C20/25 = 50 mm
strain limit w_{ist} = l/400
$w_{net,fin}$ = l/250

LOADS

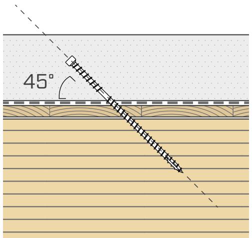
own weight (g_{k1}) = timber beam + wooden planking + concrete slab
permanent non-structural load (g_{k2}) = 2 kN/m²
variable overload (q_k) = 2 kN/m²



Installation at a 45° angle, without soundproofing layer.

CONNECTOR CTC Ø7 - Glulam GL 24h (EN 14080:2013)
Plank thickness t_s = 21 mm

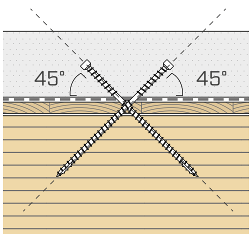
beam section BxH [mm]		**span** [m]						
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	8	10	18	24	-	-	-
	CTC	7x160	7x240	7x240	7x240			
	pitch [mm]	500/500	250/500	200/400	120/240			
	no. connectors/m²	4,0	4,3	6,8	8,1			
120 x 200	no. pairs per beam	-	8	10	18	24	-	-
	CTC		7x160	7x240	7x240	7x240		
	pitch [mm]		500/500	300/500	200/400	140/280		
	no. connectors/m²		3,5	3,8	6,1	7,3		
140 x 200	no. pairs per beam	-	-	10	12	22	32	-
	CTC			7x240	7x240	7x240	7x240	
	pitch [mm]			400/500	250/500	180/360	130/260	
	no. connectors/m²			3,8	4,0	6,7	8,8	
140 x 240	no. pairs per beam	-	-	-	10	16	22	30
	CTC				7x240	7x240	7x240	7x240
	pitch [mm]				400/500	300/500	200/400	150/300
	no. connectors/m²				3,4	4,8	6,1	7,6



Installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø7 - Glulam GL 24h (EN 14080:2013)
Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	8	10	38	-	-	-	-
	CTC	7x160	7x240	7x240				
	pitch [mm]	500/500	250/500	100/100				
	no. connectors/m²	4,0	4,3	14,4				
120 x 200	no. pairs per beam	-	8	10	24	54	-	-
	CTC		7x160	7x240	7x240	7x240		
	pitch [mm]		500/500	300/500	120/240	150/200 (1)		
	no. connectors/m²		3,5	3,8	8,1	16,4		
140 x 200	no. pairs per beam	-	-	8	22	46	90	-
	CTC			7x240	7x240	7x240	7x240	
	pitch [mm]			500/500	150/300	150/300 (1)	150/200 (2)	
	no. connectors/m²			3,0	7,4	13,9	24,8	
140 x 240	no. pairs per beam	-	-	-	8	14	34	60
	CTC				7x240	7x240	7x240	7x240
	pitch [mm]				500/500	400/500	140/200	150/250 (1)
	no. connectors/m²				2,7	4,2	9,4	15,2



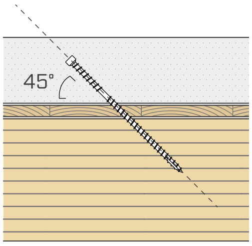
Crossed installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø7 - Glulam GL 24h (EN 14080:2013)
Plank thickness t_s = 21 mm

beam section BxH [mm]		**span** [m]						
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	16	20	36	44	-	-	-
	CTC	7x160	7x240	7x240	7x240			
	pitch [mm]	500/500	250/500	200/400	150/300			
	no. connectors/m²	8,1	8,7	13,6	14,8			
120 x 200	no. pairs per beam	-	16	20	36	48	52	-
	CTC		7x240	7x240	7x240	7x240	7x240	
	pitch [mm]		500/500	300/500	200/400	150/300	150/350	
	no. connectors/m²		6,9	7,6	12,1	14,5	14,3	
140 x 200	no. pairs per beam	-	-	20	24	44	52	84
	CTC			7x240	7x240	7x240	7x240	7x240
	pitch [mm]			280/500	250/500	180/360	150/400	110/200
	no. connectors/m²			7,6	8,1	13,3	14,3	21,2
140 x 240	no. pairs per beam	-	-	-	20	36	44	60
	CTC				7x240	7x240	7x240	7x240
	pitch [mm]				400/500	250/500	200/400	150/300
	no. connectors/m²				6,7	10,9	12,1	15,2

STATIC VALUES

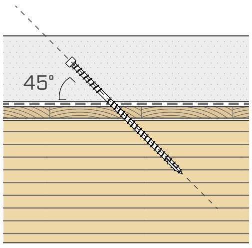
CALCULATION STANDARD
EN 1995:2014



Installation at a 45° angle, without soundproofing layer.

CONNECTOR CTC Ø9 - Glulam GL 24h (EN 14080:2013)
Plank thickness t_s = 21 mm

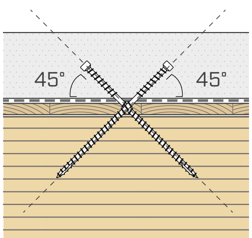
beam section BxH [mm]					**span** [m]			
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	8	10	14	22			
	CTC	9x160	9x240	9x240	9x240	-	-	-
	pitch [mm]	500/500	250/500	200/500	150/300			
	no. connectors/m²	4,0	4,3	5,3	7,4			
120 x 200	no. pairs per beam		8	10	18	22	30	
	CTC	-	9x240	9x240	9x240	9x240	9x240	-
	pitch [mm]		500/500	300/500	200/400	160/400	130/300	
	no. connectors/m²		3,5	3,8	6,1	6,7	8,3	
140 x 200	no. pairs per beam			10	12	22	30	46
	CTC	-	-	9x240	9x240	9x240	9x240	9x240
	pitch [mm]			400/500	250/500	180/400	150/300	180/350 (2)
	no. connectors/m²			3,8	4,0	6,7	8,3	11,6
140 x 240	no. pairs per beam				10	16	22	30
	CTC	-	-	-	9x240	9x240	9x240	9x240
	pitch [mm]				400/500	300/500	200/400	150/300
	no. connectors/m²				3,4	4,8	6,1	7,6



Installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø9 - Glulam GL 24h (EN 14080:2013)
Plank thickness t_s = 21 mm

beam section BxH [mm]					**span** [m]			
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	8	10	26				
	CTC	9x160	9x240	9x240	-	-	-	-
	pitch [mm]	500/500	300/500	120/200				
	no. connectors/m²	4,0	4,3	9,8				
120 x 200	no. pairs per beam		8	10	22	38		
	CTC	-	9x240	9x240	9x240	9x240	-	-
	pitch [mm]		500/500	300/500	150/300	100/140		
	no. connectors/m²		3,5	3,8	7,4	11,5		
140 x 200	no. pairs per beam			10	18	34	64	
	CTC	-	-	9x240	9x240	9x240	9x240	-
	pitch [mm]			300/500	200/400	200/400 (2)	210/300 (2)	
	no. connectors/m²			3,8	6,1	10,3	17,6	
140 x 240	no. pairs per beam				8	20	30	48
	CTC	-	-	-	9x240	9x240	9x240	9x240
	pitch [mm]				500/500	200/400	150/300	100/150
	no. connectors/m²				2,7	6,1	8,3	12,1



Crossed installation at a 45° angle, with soundproofing layer.

CONNECTOR CTC Ø9 - Glulam GL 24h (EN 14080:2013)
Plank thickness t_s = 21 mm

beam section BxH [mm]					**span** [m]			
		3	**3,5**	**4**	**4,5**	**5**	**5.5**	**6**
120 x 160	no. pairs per beam	16	24	36				
	CTC	9x160	9x160	9x160	-	-	-	-
	pitch [mm]	500/500	250/500	200/300				
	no. connectors/m²	8,1	10,4	13,6				
120 x 200	no. pairs per beam		16	20	36	48		
	CTC	-	9x160	9x160	9x160	9x160	-	-
	pitch [mm]		500/500	300/500	250/500	150/500		
	no. connectors/m²		6,9	7,6	12,1	14,5		
140 x 200	no. pairs per beam			20	36	48	60	
	CTC	-	-	9x160	9x240	9x240	9x240	-
	pitch [mm]			300/500	200/400	150/300	140/300	
	no. connectors/m²			7,6	12,1	14,5	16,5	
140 x 240	no. pairs per beam				24	40	52	60
	CTC	-	-	-	9x240	9x240	9x240	9x240
	pitch [mm]				500/500	200/400	150/400	150/300
	no. connectors/m²				8,1	12,1	14,3	15,2

NOTES:

(1) Connectors placed in two rows.

(2) Connectors placed in three rows.

For different calculation configurations, the MyProject software is available (www.rothoblaas.com).

MY PROJECT SOFTWARE

PROJECT DATA

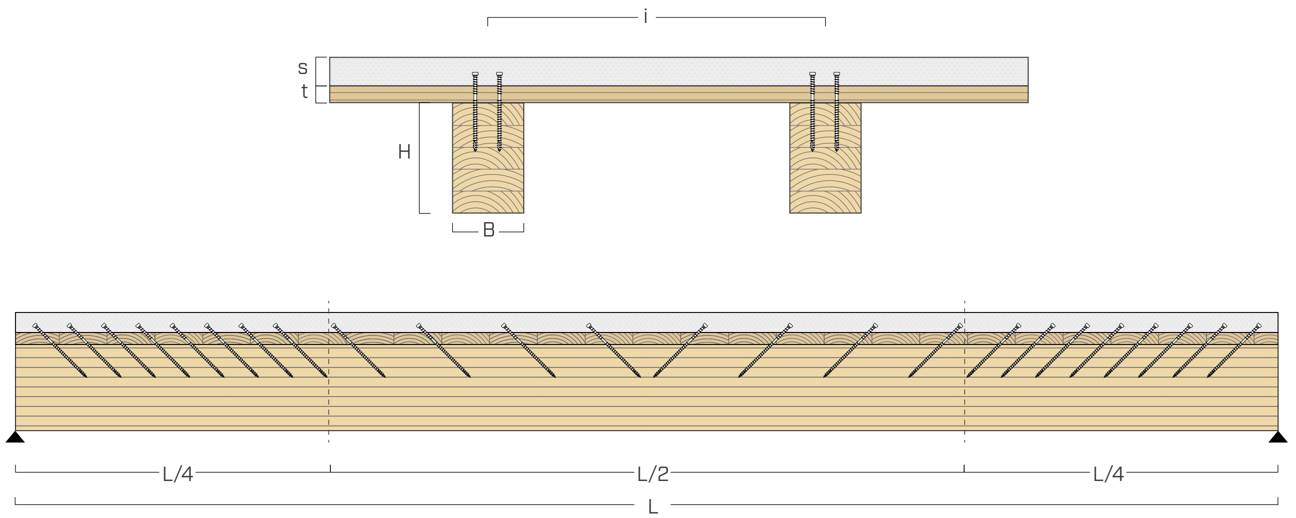
BEAMS
B = 120 mm
H = 160 mm
i = 650 mm
L = 4,0 m
Wood GL24h (EN 14081:2013)

CONNECTORS - CTC Ø9 x 240	
Diameter	9 mm
Length	240 mm
Connector arrangement	inclined at 45°
Distribution	L/4-L/2

COMPOSITE FLOOR
s = 50 mm
Concrete C25/30

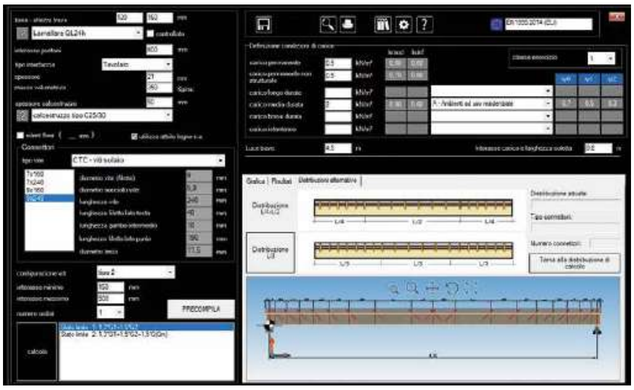
INTERMEDIATE LAYER
t = 21 mm
C20 Plank (EN 14081:2013)

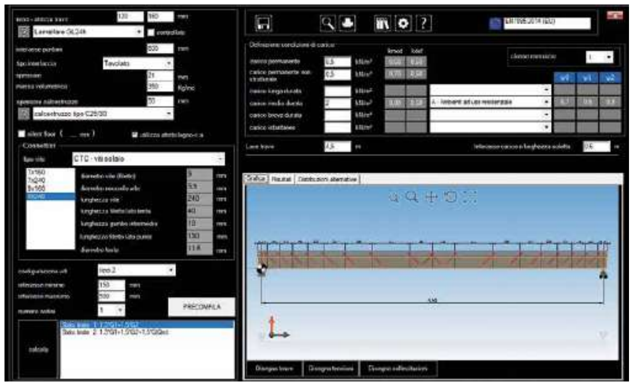
LOAD CONDITIONS	
Permanent structural load (G_1)	1,50 kN/m²
Permanent non-structural load (G_2)	2,50 kN/m²
Variable load (Q) Category A: residential environment	2,00 kN/m²
Variable load duration	medium

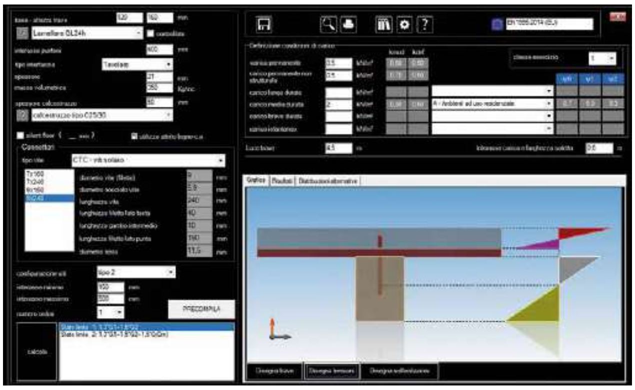


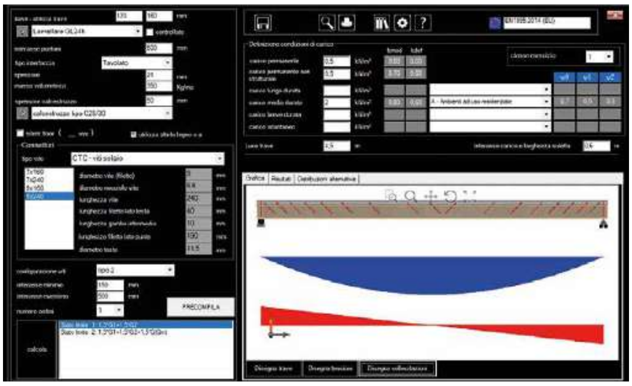
CALCULATION WITH MYPROJECT SOFTWARE (EN 1995:2014 and ETA-19/0244)

RESULTS			
Number of connectors	22 CTC Ø9x240	Min spacing (L/4)	180 mm
Incidence of connectors	8 connectors/m²	Max spacing (L/2)	370 mm



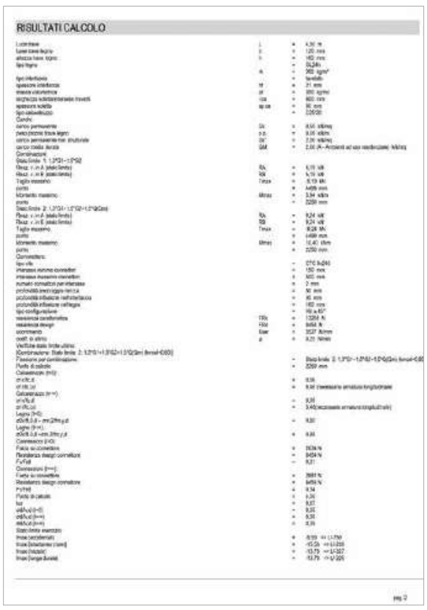






For different calculation configurations, the MyProject software is available (www.rothoblaas.com).

CALCULATION REPORT




rothoblaas
CTC 7x160
CTC7160


CTC
7x160
d x L [mm]
TX30
CE 19
ETA-19/0244
EAD 130090-00-0303
1034
CTC7 | self-tapping screws
for wood-concrete slab
(www.rothoblaas.com)
Essential characteristics:
refer to ETA-19/0244
100-10A2001802
100/
CTC7160
MADE IN TAIWAN, R.O.C.
8 058776 179132
rothoblaas
0819


rothoblaas
Solutions for Building Technology
rothoblaas
rothoblaas
KOP 10x140
KOP 10x140


KOP
10x140
EN 14592:2008+A1:2012
KOP10
screw for structural timber
(www.rothoblaas.com)
DIN
571
17
1OA2001896
50
8 033224 485850
rothoblaas
1016

Appendix N. Splitkon, CLT catalogue

N.1 Splitkon, CLT catalogue

SINTEF

Teknisk Godkjenning

SINTEF Certification

Nr. 20712

Utstedt første gang: 07.10.2019
Revidert:
Korrigert:
Gyldig til: 01.11.2024
Forutsatt publisert på www.sintefcertification.no

SINTEF bekrefter at

Splitkon krysslimt tre

er vurdert å være egnet i bruk og tilfredsstiller krav til produktdokumentasjon i henhold til forskrift om omsetning og dokumentasjon av produkter til byggverk (DOK) og forskrift om tekniske krav til byggverk (TEK), for de egenskaper, bruksområder og betingelser for bruk som er angitt i dette dokumentet

1. Innehaver av godkjenningen

Splitkon AS
Industriveien 3
3340 Åmot
https://splitkon.no

2. Produktbeskrivelse

Splitkon krysslimt tre er treelementer sammenlimt i sjikt av krysslagte, fingerskjøtte trelameller, se fig. 1. Lamellene limes sammen med lim av mellamin urea formaldehyd (MUF). Lamellene kantlimes ikke.

Elementene produseres med lameller av gran, sortert til fasthetsklasse T15 og T22 i henhold til EN 338.

Elementene leveres i tykkelser fra 60 mm til 300 mm, og med minst 3 og inntil 9 lamellsjikt. Elementoppbygningen er symmetrisk om midtsnittet. Oppbygning av standard elementer er vist i tabell 1.

Maksimal elementbredde er 3,5 m og maksimal elementlengde er 16 m. Elementene leveres forøvrig med lengder og bredder, og eventuelt med hull, innsnitt e.l., etter spesifikasjon for det enkelte byggeprosjekt. Elementer kan også settes sammen til større formater med mekaniske forbindelser, noe som må prosjekteres spesifikt i hvert enkelt tilfelle.

Måltoleranser for ferdige elementer:
- Lengde ±5 mm
- Bredde: ±2 mm
- Tykkelse ±2 mm
- Kantretthet: ±2 mm
- Vinkelretthet: ±1°
- Diagonalmål: ±5 mm

Ved produksjon er fuktinnholdet i lamellene 8 – 18 vekt %, med maksimalt 5 vekt % variasjon mellom lamellene. Forøvrig tilpasses fuktinnholdet til bruksområdet for den enkelte leveranse.

Midlere densitet av elementene regnes som 420 kg/m^3.

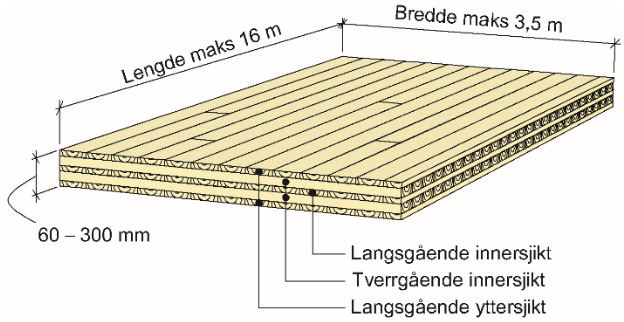


Fig. 1
Prinsipiell oppbygning av Splitkon krysslimt tre. Eksempel på fem-sjikts element.

3. Bruksområder

Elementene kan brukes som bærende konstruksjonselementer til etasjeskillere, tak og vegger i klimaklasse 1 og 2, innendørs og under tak, i henhold til NS-EN 1995-1-1.

Elementene kan benyttes i bygninger i risikoklasse 1 - 6 i brannklasse 1 og 2. Ved bruk i brannklasse 3 må brannsikkerheten dokumenteres ved analytisk brannteknisk prosjektering. Se forøvrig pkt. 6 vedr. betingelser for bruk.

4. Egenskaper

4.1 Bæreevne

Tabell 2 viser materialfastheter, stivhetsmoduler og densitet til lamellene for bruk ved dimensjonering av Splitkon krysslimt tre. Tabell 3 - 6 i pkt. 6 viser karakteristiske konstruksjonsdata for standard elementoppbygninger.

4.2 Egenskaper ved brannpåvirkning

Elementene har brannteknisk klasse D-s2,d0 i henhold til EN 13501-1. Brukt som gulv er den branntekniske klassen D_{FL}-s1. Klassifiseringen gjelder bruk på alle underlag og mot hulrom.

4.3 Brannmotstand

Brannmotstand bestemmes for komplette bygningsdeler i hvert enkelt byggeprosjekt som elementene benyttes i, og bestemmes ved standardisert prøving eller ved beregning i henhold til aktuelle standarder og håndbøker.

Tabell 1
Standard elementoppbygninger for Splitkon krysslimt tre

Elementbetegnelse Tykkelse i mm og antall sjikt	Tykkelse i mm til hvert lamellsjikt [1]						
	L	T	L	T	L	T	L
Standardelementer							
60 (3s)	20	20	20				
80 (3s)	30	20	30				
100 (3s)	33	33	33				
120 (3s)	40	40	40				
100 (5s)	20	20	20	20	20		
120 (5s)	30	20	20	20	30		
140 (5s)	33	20	33	20	33		
160 (5s)	32	32	32	32	32		
180 (5s)	40	30	40	30	40		
200 (5s)	45	32	45	32	45		
220 (5s)	44	44	44	44	44		
240 (7s)	45	20	45	20	45	20	45
260 (7s)	45	34	34	34	34	34	45
280 (7s)	45	33	45	33	45	33	45
300 (7s)	45	40	45	40	45	40	45
Elementer med doble langsgående yttersjikt [2]							
	L	L	T	L	L/T	L	L
160 (5ss)	32	32	32	32	32		
200 (5ss)	45	32	45	32	45		
220 (7ss)	33	33	33	20	33	33	33
240 (7ss)	32	32	40	32	40	32	32
260 (7ss)	45	34	34	34	34	34	45
280 (7ss)	45	45	33	33	33	45	45
300 (7ss)	45	45	40	40	40	45	45

[1] L = lameller lagt i elementets lengderetning
T = lameller lagt på tvers av elementets lengderetning
Elementer leveres også med tverrgående lameller i yttersjikt
Ytterlameller er i kvalitet T22, innerlameller i T8.
[2] Doble langsgående ytterlameller er begge i kvalitet T22

Tabell 2
Materialfastheter, stivhetsmoduler og densitet for lameller til Splitkon krysslimt tre [1]

Egenskap		Fasthetsklasse T15	Fasthetsklasse T22
Karakteristiske fastheter		*N/mm²*	*N/mm²*
Bøyefasthet	$f_{m,k}$	22,0	30,5
Strekkfasthet	$f_{t,0,k}$	15,0	22,0
Trykkfasthet	$f_{c,0,k}$	21,0,0	26,0
Skjærfasthet	$f_{v,k}$	4,0	4,0
Rulleskjærfasthet	$f_{v,90,k}$	0,7	0,7

Egenskap		Fasthetsklasse T8	Fasthetsklasse T22
Stivheter for deformasjons-beregninger		*N/mm²*	*N/mm²*
Elastisitetsmodul strekk	$E_{0,mean}$	11500	13000
	$E_{90,mean}$	230	430
Skjærmodul	$G_{0,mean}$	720	810
	$G_{90,mean}$	72	81
Densitet		*kg/m³*	*kg/m³*
Midlere densitet	ρ_m	430	470
Karakteristisk densitet	ρ_k	360	390

[1] I henhold til EN 338, NS EN 14080 og EN 16351

4.4 Varmeisolering

Dimensjonerende varmekonduktivitet for trevirket i elementene er λ_d = 0,12 W/(m·K) i henhold til NS-EN ISO 10456. Spesifikk varmekapasitet er 1600 J/kg·K.

4.5 Vanndampmotstand

Trevirket i elementene har en vanndampmotstandsfaktor μ = 50 ved tørre forhold og μ = 20 ved fuktige forhold i henhold til NS-EN ISO 10456.

4.6 Fuktbevegelser

Følgende endringer av elementenes dimensjoner pr. % endring i trevirkets fuktinnhold bør forventes:

- Lengderetning 0,01 %
- Bredderetning 0,03 %
- Tykkelsesendring 0,20 %

5. Miljømessige forhold

5.1 Helse– og miljøfarlige kjemikalier

Splitkon Krysslimt tre inneholder ingen prioriterte miljøgifter, eller andre relevante stoffer i en mengde som vurderes som helse- og miljøfarlige. Prioriterte miljøgifter omfatter CMR, PBT og vPvB stoffer.

5.2 Inneklimapåvirkning

Elementene er bedømt å ikke avgi partikler, gasser eller stråling som gir negativ påvirkning på inneklimaet, eller som har helsemessig betydning.

5.3 Avfallshåndtering / Gjenbruksmuligheter

Elementene sorteres som trematerialer ved avhending, og leveres til godkjent avfallsmottak der de kan energigjenvinnes.

5.5 Miljødeklarasjon

Det er ikke utarbeidet miljødeklarasjon (EPD) for Splitkon krysslimt tre.

6. Betingelser for bruk

6.1 Beregning av bæreevne

Beregning av elementenes bæreevne, inkludert oppleggskapasitet og effekt av hulltaking, innsnitt etc., skal gjøres for hver enkelt leveranse. Den statiske dimensjoneringen skal være tilpasset det enkelte byggeprosjekt, og være basert på NS-EN 1995-1-1 og relevante laster i henhold til NS-EN 1991 med nasjonale tillegg NA. Karakteristiske fastheter og stivheter som angitt i tabell 2 skal legges til grunn.

Dersom det ikke gjøres andre spesifikke beregninger kan det for dimensjonering av standard elementoppbygninger som vist i tabell 1 og 2 anvendes karakteristiske fastheter og stivhetsmoduler for den enkelte elementoppbygning som angitt i tabell 3 – 5.

Alternativt kan det også anvendes karakteristiske kapasiteter for standard elementoppbygninger som vist i tabell 6.

6.2 Spennvidder for etasjeskillere

I tabell 7 er det vist anbefalte spennvidder for Splitkon krysslimt tre elementer med standard elementbredde, brukt som dekkeelementer i bolighus og lignende bygg der det er viktig å unngå sjenerende svingninger og rystelser som følge av normal gangtrafikk. Spennviddene er basert på SINTEFs anbefalte komfortkriterium for dynamisk og statisk stivhet.

For bygninger med mange gående personer, rytmiske aktiviteter eller sensitivt utstyr bør krav til stivhet utredes spesifikt.

Anbefalte spennvidder for bolighus ol. i tabell 7 gjelder for elementer uten hensyn til eventuell avstivende effekt av overgolv eller himling. Dersom det monteres ikke bærende vegger på tvers av elementene, tilnærmet midt i spennet på over- eller undersiden av elementene, kan det benyttes spennvidder basert på dimensjonering med jevnt fordelt nyttelast alene (dvs. uten kontroll av komfortkriterium). Dette forutsetter at veggene festes til elementene.

6.3 Sikkerhet ved brann

For hvert enkelt prosjekt må nødvendig brannmotstand i henhold til TEK være bestemt for bygningsdeler som skal ha bærende og/eller branncellebegrensende egenskap ved brann, og dimensjonerende lastkapasitet ved ulykkesgrensetilstand brann må kontrolleres. Valg av oppbygning gjøres blant annet ut fra behovet for brannmotstand.

6.4 Lydisolering og akustikk

Ved bruk i konstruksjoner med krav til lydisolasjon og/eller akustisk regulering skal de lydtekniske egenskapene til den ferdige konstruksjonen være forhåndsprosjektert, og eventuelle supplerende golvkonstruksjoner og kledninger være bestemt. Dette inkluderer også oppleggsdetaljer.

I etasjeskillere med krav til lydisolasjon må elementene kompletteres med et oppbygd golv og / eller en nedsenket himling for å kunne tilfredsstille lydisolasjon klasse C eller bedre i henhold til NS 8175 med hensyn til luft- og trinnlydisolasjon. Også elementer som skal benyttes til lydskillevegger må i praksis kompletteres med en tilleggskonstruksjon i form av utlektet veggkledning på én eller to sider, eller bruk av to uavhengige veggskall.

6.5 Fukttekniske hensyn

Det må tas hensyn til hvilke klimavariasjoner med tilhørende fuktbevegelser som elementene kan bli utsatt for, se pkt. 4.6. Spesielt gjelder dette for store flater sammensatt av mange elementer.

Ved bruk i varmeisolerte konstruksjoner må eventuell bruk av dampsperre som supplement til elementenes dampmotstand vurderes spesielt, se pkt. 4.5.

6.6 Montasje

Elementene skal monteres i henhold til en montasjeplan med tilhørende konstruksjonsdetaljer som er utarbeidet spesifikt for hvert enkelt byggeprosjekt. Krav til understøttelser og nødvendige toleranser på tilstøtende konstruksjoner skal være klarlagt.

6.7 Transport og lagring

Under transport og lagring skal elementene være plassert på et tilstrekkelig plant og stivt underlag som hindrer permanente deformasjoner, og være beskyttet mot nedbør og kontakt med fritt vann.

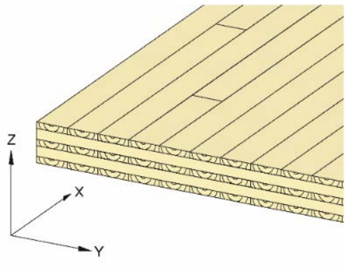


Fig. 2
Akseretninger angitt i tabell 3 – 6

6.8 Bestandighet

Trevirket i standard elementer er ubehandlet, og har i praksis samme bestandighet som vanlig konstruksjonsvirke ved anvendelse som angitt i pkt. 3.

Tabell 3
Beregnede karakteriske fastheter i N/mm^2 til standard elementoppbygninger av Splitkon krysslimt tre (5%-fraktilen) [1)]

Element-betegnelse	Bøyefasthet $f_{m,k}$				Strekkfasthet $f_{t,k}$			Trykkfasthet $f_{c,k}$			Skjærfasthet $f_{v,k}$		
	Bøyning om akse				Last i retning			Last i retning			Bøyning om akse		
Tykkelse i mm og antall sjikt	Y (sterk)	X (svak)	Z_1 skive-virkn. (sterk)	Z_2 skive-virkn. (svak)	X (sterk)	Y (svak)	Z (tverr-strekk)	X (sterk)	Y (svak)	Z vinkel-rett fiber-retn.	XZ (sterk)	YZ (svak)	XY skive-virkn.
Standardelementer													
60 (3s)	29,4	0,7	20,3	7,3	14,7	2,7	0,4	17,3	7,0	3,0	1,1	4,0	1,3
80 (3s)	30,0	0,3	22,9	5,5	16,5	2,0	0,4	19,5	5,3	3,0	0,7	4,0	1,0
100 (3s)	29,4	0,7	20,3	7,3	14,7	2,7	0,4	17,3	7,0	3,0	0,7	4,0	1,3
120 (3s)	29,4	0,7	20,3	7,3	14,7	2,7	0,4	17,3	7,0	3,0	0,7	4,0	1,3
100 (5s)	24,1	4,0	16,6	8,8	11,8	3,2	0,4	14,6	8,4	3,0	1,1	1,1	1,6
120 (5s)	26,8	2,3	18,9	7,3	13,5	2,7	0,4	16,5	7,0	3,0	0,7	0,7	1,3
140 (5s)	26,4	2,6	19,6	6,3	14,0	2,3	0,4	17,3	6,0	3,0	0,7	0,7	1,2
160 (5s)	24,1	4,0	16,6	8,8	11,8	6,0	0,4	14,6	8,4	3,0	0,7	0,7	1,6
180 (5s)	25,6	3,1	18,4	7,3	13,1	2,7	0,4	16,2	7,0	3,0	0,7	0,7	1,3
200 (5s)	25,8	3,0	18,7	7,0	13,3	2,6	0,4	16,5	6,8	3,0	0,7	0,7	1,3
220 (5s)	24,1	4,0	16,6	8,8	11,8	3,2	0,4	14,6	8,4	3,0	0,7	0,7	1,6
240 (7s)	25,6	2,9	19,7	5,5	13,9	2,0	0,4	17,6	5,3	3,0	0,7	0,7	1,0
260 (7s)	23,5	4,3	16,3	8,6	11,5	3,1	0,4	14,5	8,2	3,0	0,7	0,7	1,6
280 (7s)	23,3	4,4	16,9	7,8	11,9	2,8	0,4	15,2	7,5	3,0	0,7	0,7	1,4
300 (7s)	22,2	5,1	15,8	8,8	11,1	3,2	0,4	14,1	8,4	3,0	0,7	0,7	1,6
Elementer med doble langsgående yttersjikt													
160 (5ss)	30,3	0,2	24,4	4,4	17,6	1,6	0,4	20,8	4,2	3,0	0,7	0,7	0,8
200 (5ss)	30,1	0,2	23,5	5,0	17,0	1,8	0,4	20,1	4,7	3,0	0,7	0,7	0,9
220 (7ss)	28,6	1,2	20,3	6,6	14,7	2,4	0,4	17,7	6,4	3,0	0,7	0,7	1,2
240 (7ss)	27,5	1,9	19,2	7,3	13,7	2,7	0,4	16,7	7,0	3,0	0,7	0,7	1,3
260 (7ss)	28,7	1,1	21,4	5,8	15,3	2,1	0,4	18,5	5,5	3,0	0,7	0,7	1,0
280 (7ss)	29,2	0,8	22,2	5,2	16,0	1,9	0,4	19,3	5,0	3,0	0,7	0,7	0,9
300 (7ss)	28,6	1,2	21,2	5,9	15,2	2,1	0,4	18,4	5,6	3,0	0,7	0,7	1,1

[1)] Akseretninger Z, X og Y er angitt i fig. 2.
- Sterk betegner belastning i elementets lengderetning
- Svak betegner belastning i elementets tverretning

Tabell 4
Beregnede karakteriske stivhetsverdier i N/mm^2 til standard elementoppbygninger av Splitkon krysslimt tre for **deformasjonsberegninger** (50%-fraktilen) [1)]

Element-betegnelse	E-modul ved bøyning $E_{m.50}$		E-modul ved aksielt strekk og trykk $E_{t,50}$ / $E_{c,50}$			Skjærmodul ved bøyning G_{50}		
	Bøyning om akse		Last i retning			Bøyning om akse		
Tykkelse i mm og antall sjikt	Y (sterk)	X (svak)	X (sterk)	Y (svak)	Z vinkelrett fiberretn.	XZ bøyning (sterk)	YZ bøyning (svak)	XY bøyning skivevirk.
Standardelementer								
60 (3s)	12519	426	8667	3833	413	105	200	780
80 (3s)	12797	180	9750	2875	418	126	150	788
100 (3s)	12519	426	8667	3833	413	105	200	780
120 (3s)	12519	426	8667	3833	413	105	200	780
100 (5s)	10284	2392	7500	4600	400	112	62	756
120 (5s)	11428	1384	8417	3833	405	119	52	765
140 (5s)	11271	1512	8903	3309	404	139	46	763
160 (5s)	10284	2392	7500	4600	400	112	62	756
180 (5s)	10897	1846	8333	3833	402	126	52	760
200 (5s)	10997	1757	8480	3698	403	129	50	761
220 (5s)	10284	2392	7500	4600	400	112	62	756
240 (7s)	10927	1707	9188	2875	399	164	55	754
260 (7s)	10035	2546	7508	4512	397	117	73	751
280 (7s)	9925	2609	7903	4081	396	130	68	749
300 (7s)	9450	3036	7350	4600	395	120	75	747
Elementer med doble langsgående yttersjikt								
160 (5ss)	12896	92	10400	2300	420	147	120	792
200 (5ss)	12850	133	10060	2601	419	135	136	790
220 (7ss)	12211	697	8927	3482	410	119	52	774
240 (7ss)	11706	1141	8467	3833	407	116	53	768
260 (7ss)	12241	669	9404	3008	410	134	41	775
280 (7ss)	12438	495	9747	2720	412	141	37	778
300 (7ss)	12195	709	9333	3067	410	132	42	774

[1)] Akseretninger Z, X og Y er angitt i fig. 2.
- Sterk betegner belastning i elementets lengderetning
- Svak betegner belastning i elementets tverretning

Tabell 5
Beregnede karakteriske stivhetsverdier i N/mm^2 til standard elementoppbygninger av Splitkon krysslimt tre for **styrkeberegninger** (5%-fraktilen) [1) 2)]

Elementbetegnelse Tykkelse i mm og antall sjikt	E-modul ved bøyning $E_{m.5}$		E-modul ved aksielt strekk og trykk $E_{t,5}$ / $E_{c,5}$	
	Bøyning om akse		Last i retning	
	Y (sterk)	X (svak)	X (sterk)	Y (svak)
Standardelementer				
60 (3s)	8378	285	5800	2567
80 (3s)	8564	120	6525	1925
100 (3s)	8378	285	5800	2567
120 (3s)	8378	285	5800	2567
100 (5s)	6882	1602	5020	3080
120 (5s)	7648	927	5633	2567
140 (5s)	7543	1012	5959	2216
160 (5s)	6882	1602	5020	3080
180 (5s)	7293	1236	5578	2567
200 (5s)	7359	1176	5676	2476
220 (5s)	6882	1602	5020	3080
240 (7s)	7313	1143	6150	1925
260 (7s)	6716	1705	5025	3021
280 (7s)	6642	1747	5290	2732
300 (7s)	6324	2033	4920	3080
Elementer med doble langsgående yttersjikt				
160 (5ss)	8630	62	6960	1540
200 (5ss)	8599	89	6733	1741
220 (7ss)	8172	467	5974	2331
240 (7ss)	7834	764	5667	2567
260 (7ss)	8192	448	6294	2014
280 (7ss)	8324	331	6524	1822
300 (7ss)	8161	475	6247	2053

[1)] Akseretninger Z, X og Y er angitt i fig. 2.
- Sterk betegner belastning i elementets lengderetning
- Svak betegner belastning i elementets tverretning

[2)] Tabellen er basert på Timoshenkos bjelketeori

Tabell 6
Beregnede karakteriske kapasiteter til standard elementoppbygninger av Splitkon krysslimt tre.
Elementbredde 1 m [1)]

Element-betegnelse Tykkelse i mm og antall sjikt	Bøyemomentkapasitet i kNm/m Last vinkelrett på plateplanet M_k		Skjærkraftkapasitet i kN/m Last vinkelrett på plateplanet V_k		Trykkapasitet i kN/m i plateplanet $F_{c,d}$		Strekkapasitet i kN/m i plateplanet $F_{t,d}$	
	Bøyning om akse		Bøyning om akse		Last i retning		Last i retning	
	Y (sterk)	X (svak)	XZ (sterk)	YZ (svak)	X (sterk)	Y (svak)	X (sterk)	Y (svak)
Standardelementer								
60 (3s)	17,6	0,4	47,7	1,6	1040	420	880	300
80 (3s)	32,0	0,3	61,6	0,6	1560	420	1320	300
100 (3s)	48,0	1,2	50,1	1,7	1716	693	1452	495
120 (3s)	70,5	1,7	60,7	2,1	2080	840	1760	600
100 (5s)	40,2	6,7	90,6	20,0	1460	840	1180	600
120 (5s)	64,3	5,6	103,1	8,0	1980	840	1620	600
140 (5s)	85,2	8,2	122,0	9,7	2409	840	1947	600
160 (5s)	102,9	17,3	92,3	20,3	2336	1344	1888	960
180 (5s)	138,1	16,9	101,8	16,2	2920	1260	2360	900
200 (5s)	170,3	19,6	112,2	16,8	3285	1344	2655	960
220 (5s)	194,6	32,7	126,9	28,0	3212	1848	2596	1320
240 (7s)	246,1	27,7	187,5	18,6	4230	1260	3330	900
260 (7s)	265,3	48,5	135,0	34,3	3768	2142	3000	1530
280 (7s)	302,1	57,3	141,9	37,3	4230	2079	3330	1485
300 (7s)	332,5	77,1	154,1	49,5	4230	2520	3330	1800
Elementer med doble langsgående yttersjikt								
160 (5ss)	129,1	0,7	77,2	0,6	3328	672	2816	480
200 (5ss)	199,0	1,5	96,7	1,0	4004	945	3388	675
220 (7ss)	226,9	9,3	113,2	6,5	3852	1386	3204	990
240 (7ss)	263,7	18,5	128,9	12,3	4000	1680	3296	1200
260 (7ss)	323,6	12,7	135,0	7,2	4822	1428	3986	1020
280 (7ss)	378,6	10,9	142,5	5,6	5373	1386	4455	990
300 (7ss)	429,2	18,0	156,3	8,9	5520	1680	4560	1200

[1)] Akseretninger Z, X og Y er angitt i fig. 2.
- Sterk betegner belastning i elementets lengderetning.
- Svak betegner belastning i elementets tverretning

Tabell 7
Anbefalte maksimale spennvidder for Splitkon Krysslimt tre i bolighus o.l. [1)]

Elementbetegnelse Tykkelse i mm og antall sjikt	Spennvidde i meter
Standardelementer	
60 (3s)	2,20
80 (3s)	2,85
100 (3s)	3,40
120 (3s)	3,95
100 (5s)	3,20
120 (5s)	3,85
140 (5s)	4,35
160 (5s)	4,75
180 (5s)	5,30
200 (5s)	5,80
220 (5s)	6,00
240 (7s)	6,40
260 (7s)	6,60
280 (7s)	6,80
300 (7s)	7,00
Elementer med doble langsgående yttersjikt	
160 (5ss)	5,05
200 (5ss)	6,00
220 (7ss)	6,20
240 (7ss)	6,50
260 (7ss)	6,90
280 (7ss)	7,20
300 (7ss)	7,50

1) Tabellen gjelder for elementer montert fritt opplagt over ett spenn og klimaklasse 1 i henhold til NS–EN 1995-1-1, og for en jevnt fordelt nyttelast på 2,0 kN/m^2 pluss 0,5 kN/m^2 egenlast i tillegg til selve massivtreelementet

7. Produkt- og produksjonskontroll

Splitkon Krysslimt tre produseres av Splitkon AS, Industriveien 3, 3340 Åmot, Norge.

Innehaver av godkjenningen er ansvarlig for produksjons-kontrollen for å sikre at elementene blir produsert i henhold til de forutsetninger som er lagt til grunn for godkjenningen.

Fabrikkfremstillingen av elementene er underlagt over-våkende produkt- og produksjonskontroll i henhold til kontrakt om SINTEF Teknisk Godkjenning.

8. Grunnlag for godkjenningen

Godkjenningen er primært basert på produktegenskaper som er dokumentert i følgende rapporter:

- Norsk Treknisk Institutt. Mekanisk testing av krysslimte treelementer produsert av Splitkon AS. Rapport nr. 310069-1 av 13.06.2019
- Norsk Treknisk Institutt. Bøyetesting av fingerskjøtte limtrelameller til bruk i krysslimt treelement produsert av Splitkon AS. Rapport nr. 310069-3 av 19.06.2019
- Splitkon AS. Splitkon – teknisk godkjenning - materialtabeller. Beregningsrapport juli 2019 v/Ole Edvard Bakken
- Norsk Treknisk Institutt. Kvalitetssikring av Splitkon beregningsrapport til TG 20712. Rapport nr. 365489 av 26.09.2019
- SINTEF Byggforsk. Splitkon krysslimt tre, komfortegenskaper. Notat av 05.09.2019 prosj. 102020834-3/99

9. Merking

Hvert element skal være merket med relevant nummerering, kode eller lignende som angir spesifikk plassering i det enkelte byggeprosjekt. Produsentnavn og produksjons-tidspunkt skal også fremgå av merkingen. Det kan også merkes med godkjenningsmerket for Teknisk Godkjenning; TG 20712.

TEKNISK GODKJENNING TG 20712 SINTEF Certification

Godkjenningsmerke

10. Ansvar

Innehaver/produsent har det selvstendige produktansvar i henhold til gjeldende rett. Krav kan ikke fremmes overfor SINTEF utover det som er nevnt i NS 8402.

for SINTEF

Hans Boye Skogstad
Godkjenningsleder