CHALMERS
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Multi-storey timber buildings in practice

A study on how to practically use timber as a structural material in the early stage design

Master's thesis in Structural Engineering and Building Technology

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Master's thesis ACEX30
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Cover: Illustrations showing the concrete part and the timber part of the Brock Commons building. Images © Cadmakers Inc / www.cadmakers.com

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Abstract

During the latest years, the movement towards sustainable solutions have truly influenced the debate around the building industry. Many of the stakeholders have started to show an interest in reducing emissions. One way of doing this could be by using timber instead of the traditional steel and concrete in multi-storey buildings. However, as the previous use of timber in larger structures have been limited, there is a gap of knowledge and experience in the industry on how to practically implement timber in the design of multi-storey buildings. The purpose of this study is therefore to investigate, gather information and give example on how timber can successfully be used as a structural material. This to facilitate the implementation of timber early in the design process.

The study is done in three steps. First, a review is done on what timber systems that different manufacturers offers, together with the solutions used in some of the latest years new ground-breaking timber buildings. The second step is a case study where timber concepts are developed for two different projects, previously built in concrete/steel. In the last part, preliminary design tools are developed for some of the structural elements studied throughout the project.

The study shows that there are several interesting systems and products that already have been used in multi-storey timber building projects. Systems based on CLT and LVL have come very far and are available on the market in Sweden. The study also shows that there could be a large upside on using timber for buildings where a low structural self-weight is desired, for instance in areas with difficult soil conditions or when adding an extra storey on top of an existing structure. On the other hand, a too light-weight building could be problematic in terms of global stability against tilting. This call for solutions where combinations of materials are used in a wise manner. Here, the development of timber-concrete composites has a large potential and is already used in some projects abroad. But it is still associated with uncertainties regarding some of the technical questions on how to calculate and design, so it will probably be a few years before this technique will have its break-through on the commercial market in Sweden.

Keywords: Timber, multi-storey building, structural system, preliminary design, sizing diagrams, timber materials, TCC, hybrid structure

Flervåningshus i trä i praktiken
En studie av den praktiska tillämpningen för trästommar i det tidiga planeringsskedet av flervåningshus
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Sammanfattning

Under de senaste åren har intresset för hållbara lösningar fått inflytande i debatten kring byggindustrin. Flertalet intressenter har fått upp ögonen för att på olika sätt minska utsläppen. Ett sätt att uppnå detta kan vara genom att använda trä som stommaterial i flervåningshus, istället för de mer traditionella stål och betong alternativen. Emellertid har den tidigare användningen av trä för större konstruktioner varit begränsad, således finns en brist på kunskap och erfarenhet inom byggindustrin för den praktiska tillämpningen av trä som stommaterial i flervåningshus. Syften med denna studie är därför att samla och lyfta upp information samt ge exempel på hur trä på ett framgångsrikt sätt kan användas som stommaterial. Detta för att förenkla möjligheten att välja trä i det tidiga skedet av planeringsprocessen.

Studien utförs i tre huvudsakliga steg. Först görs en genomgång av de system som olika leverantörer erbjuder, tillsammans med ett urval av lösningar som använts i några av de senaste årens banbrytande träbyggnadsprojekt. Det andra steget är en fallstudie där stomkoncept i trä tas fram för två olika projekt som tidigare gjorts i betong/stål. I den sista delen utvecklas och presenteras verktyg för preliminärdimensionering för några av de byggelement som studerats under projektet.

Studien visar att det finns flertalet system och produkter som används i flervåningshus i trä. System av KL-trä och LVL har kommit väldigt långt och finns tillgängliga på den svenska marknaden. Studien visar också att det kan finnas en stor uppsida i att använda trä för byggnader där egentyngden bör begränsas, så som fall med komplicerad grundläggning eller påbyggnad av extra våningar till befintliga byggnader. Å andra sidan framkommer också att en alltför lätt byggnad kan få problem att uppfylla totalstabiliteten. Det senare visar på behovet att använda olika material på rätt sätt för att få fram en bra slutlösning. I det fallet har samverkanskonstruktioner med trä och betong en stor potential och har också använts vid ett antal projekt utomlands. Det är dock fortfarande förknippat med osäkerheter rörande vissa av de tekniska frågorna kring beräkningar och dimensionering, så troligtvis dröjer det ett antal år innan denna teknik får sitt genombrott kommersiellt på den svenska marknaden.

Nyckelord: Trä, flervåningshus, stomsystem, preliminär dimensionering, dimensioneringsdiagram, trämaterial, TCC, hybridkonstruktion

Contents

Preface

This thesis is written as the last part of the Master's programme *Structural Engineering and Building technology*. It also marks an end for 5 years of study at Chalmers University of Technology in Gothenburg, where we have experienced great times, grown both personally and professionally, and developed a hopefully lifelong friendship.

We would like to thank our supervisor and examiner at Chalmers, Assistant Professor Robert Jockwer for the help and feedback through the process of developing and writing the thesis. Also, we would like to thank Dennis Cederholm for the possibility to write the thesis in cooperation with Stomkon, and for the help in finding an interesting topic and the two projects used for the case studies. A big appreciation goes to Sebastian Forsberg at Stomkon, for the support with both technical questions, introduction to software's and feedback through the development of the whole thesis.

The work of this thesis was carried out during spring 2020, at Stomkons office in Gothenburg, where the required workspace and resources in terms of computers and software were provided. Therefore, we would like to thank the colleagues at Stomkon Gothenburg for this time. A special thanks also goes to Moelven Töreboda, for the possibility of a study trip through their glulam factory, which served as inspiration before the start of the work with the thesis.

Rasmus Kronberg, Martin Söderberg, Gothenburg, June 2020

1 Introduction

1.1 Background

Influenced by the ongoing public debate and movement to more sustainable solutions in all aspects of the society, the interest of widening the usage of timber in the building industry have increased in order to reduce the environmental impact during the construction stage and service life. This could be seen in projects where the municipality or other clients wants to push the industry in the direction of building timber buildings, by demanding that timber should be used for the structural system.

Traditionally in Sweden, timber have been widely used as a structural material for smaller one-family houses and terrace houses up to two stories. This while larger residential and office buildings usually have been made with concrete or steel. One of the reasons for this is that up until 1994 it where restricted to build more than two storeys in timber. A restriction that originates from the large city fires in earlier centuries. A reflection of this can be seen in the industry where the experience, solutions and way of working is well established when using steel and concrete for larger buildings. Even though the theory and research on how timber behaves have come very far, and is implemented in standards such as Eurocode, there is still a gap of knowledge and experience in the industry on how to practically use timber as the main structural material for mid-rise to high-rise buildings.

The lack of practical experience of planning and constructing timber buildings could lead to a certain resistance of choosing timber when deciding the structural system for a new building. Simply because it is easier and associated with less risks to go with a well-known concept. It could often be the case that in the system stage of projects, when many of the main decisions are made, choices as placing of structural elements are made with solutions adapted to concrete in mind. This could rule out the possibility to use timber for the project already at an early stage. In other cases, a timber system could be required by the client, but at the system stage still structural choices are done with solutions adapted to concrete or steel. This leads up to that later in the process, when construction documents should be prepared, previous work needs to be redone in order to make the structural system suitable for timber.

Also, there might still be a resistance in the public opinion about how higher timber buildings performs in important aspects such as fire resistance and acoustics. These are very important issues to handle when designing a timber building. As all materials, timber have both strengths and limitations. Therefore, it might be the best to use a combination of materials in the design of a building, to use the strengths of each material.

1.2 Purpose and aim

There is a lack of experience and knowledge when it comes to the design of larger timber buildings and therefore timber gets unfavoured in the process when deciding the structural system for a new building. The purpose with this study is to increase the knowledge on how to use timber in practice when it comes to design of multi-storey buildings. An increased knowledge and awareness about the practical applications of the material will increase the competitiveness when choosing the structural system in the early stage of the planning process.

The aim of the project is to find and use appropriate solutions to give examples on how to design high-performing multi-storey buildings with timber as one of the main structural materials. This will also include new ground-breaking techniques on how to combine materials in order to perform a hybrid structure with timber as one of the materials.

1.3 Limitations

The study is focused on how timber can be used structurally and for a wide application within the industry. Therefore, the calculations and developed tools will be focused for buildings of 4-10 storeys. However, as inspiration, reference projects of record class 15+ storeys will be included.

The economic part is indeed important when considering different options, hence only solutions that seems realistic to the authors will be studied, but no actual economical calculations will be done.

Timber is often considered to be a sustainable material. However, calculations or comparisons of the environmental impact, such as life cycle analyses, when choosing timber in favour of other materials is out of the scope for this study.

The study will focus on the structural behaviour of timber and timber hybrids. Important aspects such as acoustics, fire safety and installations will be considered in such way that it should be realistic for the proposed solution to fulfil the requirements, but no actual design calculations will be made for these aspects. Furthermore, the study is made from a Swedish perspective so the Swedish national annex has been used for preliminary design calculations, and seismic design is left out of the study.

The study will not include design of the foundation, but timber buildings generally have less weight, hence will it probably need less foundation works compared to a traditional concrete or steel building.

Moisture is of course a very important aspect for timber as a structural material. However, in this study a limitation is made where all timber members are assumed to be properly dried before assembly and protected during the whole service life of the building. Design will be done according to Eurocode, where moisture effects are included in the factors k_{def} and k_{mod}. Hence, no additional studies on how moisture affects the timber structure will be done.

1.4 Specific question formulation

The thesis addresses the following specific questions:

In the early stage of the planning process of a multi-story building, how can timber be used as a structural material by replacing concrete or steel?

How will the use of timber differ between a residential and an office building?

What preliminary sizes could be expected for different timber elements in different applications?

1.5 Method

The study was performed in three main stages, literature study, development of structural concepts including verification and development of preliminary sizing tools.

The project started with a literature study to gain a wider knowledge about how timber is used as a structural material today and possibly in the near future. This included study of different reference projects for timber or hybrid structures, both among ongoing projects and existing buildings. It also focused on what type of engineered wood products and system solutions that are available on the market already today.

In order to make good estimations and verifications in terms of hand calculations, literature on how to practically calculate the load-bearing capacity of different timber elements were studied, including, but not limited to, Eurocodes. Also, a study on the development and ongoing research in the field of timber-concrete composites were performed, to discover the areas of use where this might be suited.

In the next step, two different case studies were made, one for a residential building and one for the renovation of an office building. The cases were provided by Stomkon and are projects made in concrete and/or steel. In the studies, a structural system that uses timber were developed for each case. The structural systems were preliminary designed in ULS and SLS, to verify that the developed systems are realistic and applicable. The design and verifications were done by both hand calculations and by using FEM-models. The design in ULS includes static design and verification of how vertical loads and horizontal loads are transferred to the foundation, together with overall horizontal stability. The SLS design were done for deflections, springiness and vibrations. This is only a preliminary design, so detailed designing of connections, fire safety, acoustics, installations, insulation etc. were set as limitations. However, those aspects have been in mind when developing the concepts, and hence the proposed design should offer a good possibility to fulfil the requirements for these aspects.

The knowledge gained throughout the project were used to gather information on how to practically plan for a timber building already in the system stage of the planning process. This is presented in diagrams where rough estimations of dimensions needed for the applications of certain types of timber products. The information is directed for the use of architects and structural engineers when starting up the planning for a new building. These diagrams only serve to give an indication for preliminary sizes of individual members for cases with the given assumptions. Detailed design concerning several aspects is still needed.

2
Timber products

2.1 Timber materials

Today, the wooden industry effectively utilises most of the material from each log when producing wooden/timber materials, almost nothing is wasted. The pieces best suited for structural purposes is located closest to the core of the tree. Therefore, the number of top quality timber members that can be sawn directly from each log is restricted. However, through techniques and innovations that have been developed during the latest century, a large part of the log can today be used in different types of engineered wood products (EWP). Larger cross-sections for beams and columns as well as massive wall- and floor elements are created with gluing and jointing techniques. Less graded timber is used to build up volume in parts of elements and structures that are subjected to smaller stresses. Smaller wooden parts or remains such as chips, sawdust or even smaller particles are used in the manufacturing of several types of boards, which then could be used as sheeting material in wall- and floor constructions. In this following section, a short summary of the most common wooden products is presented.

2.1.1 Sawn timber

A member of sawn timber is referred to as a member which is directly cut out from the log. This means that the log itself sets the boundaries for the maximum length and cross-section. The nature of the logs makes timber an orthotropic material where the strength properties differs in different directions. Timber will also contain natural defects such as knots and deviations in the fibre orientation.

For timber, a distinction between softwood and hardwood is made. The coniferous trees are generally sorted as softwood, with species such as pine, spruce, larch and Douglas-fir, while hardwood comes from deciduous trees such as birch, beech and oak (Porteous and Kermani, 2009). In Sweden softwood is the most common of the standing volume in the forests; 39% is pine and 41% is spruce (Nilsson et al., 2019).

As timber is an organic material it could be sensitive to different type of deterioration processes such as mold, rot and fungi attack if not handled in a correct manner. Therefore, the drying process of the timber is of great importance to obtain a good moisture content in the timber before usage. There are also different ways to preserve and protect timber. Apart from ordinary painting, two of the most common techniques are pressure impregnating and thermally modified wood.

As structural members, sawn timber is mostly used for smaller beams and studs. In order to increase the efficiency in all stages, the sawmills produces the timber in certain standard dimensions.

2.1.2 Glued-laminated timber

Glued-laminated timber (glulam) is an EWP that consists of wood laminates that are stacked and glued together with all lamellas in the same fibre direction. The height of the laminates varies between 19mm to 50mm, where lamellas with heights in the upper span, 33mm-50mm, are mostly used to produce straight beams (Porteous and Kermani, 2009). Laminates with smaller heights are more flexible which makes it possible to create curved glulam structures. Glued-laminated timber can be connected with finger joints to obtain larger span lengths.

Usually the laminates have different strengths. Therefore, when manufacturing for instance glulam beams, a higher utilisation of the material is achieved by using high-quality timber for the outermost lamellas where the largest stresses occurs. Lower-quality timber can then be placed in the middle part of the cross-section where the stresses are less. In this way even low-quality timber can be used efficiently. This solution is called combined glulam. There are also glulams that have the same strength over the entire cross-section, which is called homogeneous glulam, see Figure 2.1.

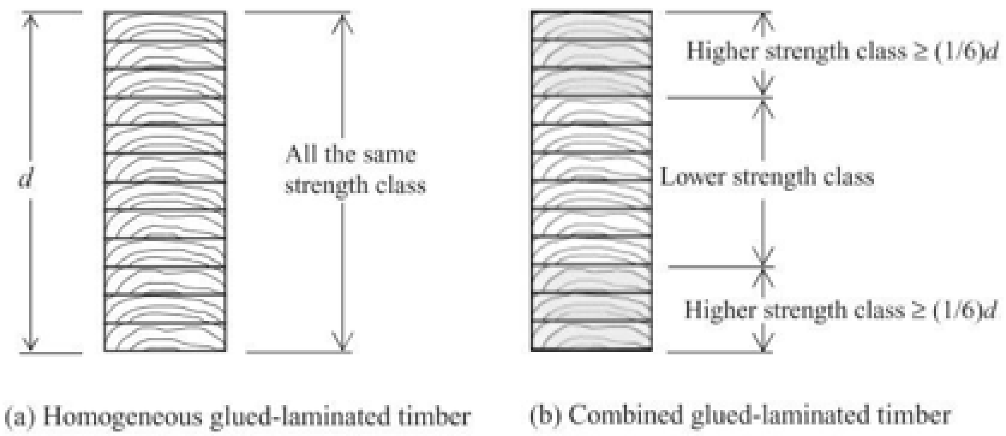

Figure 2.1: Illustrations of how the laminates are placed for homogeneous and combined glulam (Porteous and Kermani, 2009). Image © John Wiley and Sons.

Since timber is a natural material, defects such as knots, top rupture and slope of the grain can easily affect the structural properties (Thelandersson and Larsen, 2003). Glulam is less affected by defects compared to sawn timber as the glulam structure is built up by smaller wooden pieces. The glulam structure allows defects to be smoothed out and loads to be redistributed within the material in a better way. This also makes glulam a more even material that has less strength variability.

In Figure 2.2 it can be seen how knots has less impact on glulam as the knots only affect one lamella instead of the whole cross-section. The formability of glulam is flexible compared to sawn timber because of the build-up of smaller lamellas. This makes it useful in many applications such as tapered-, curved- and pitched cambered beams to mention some.

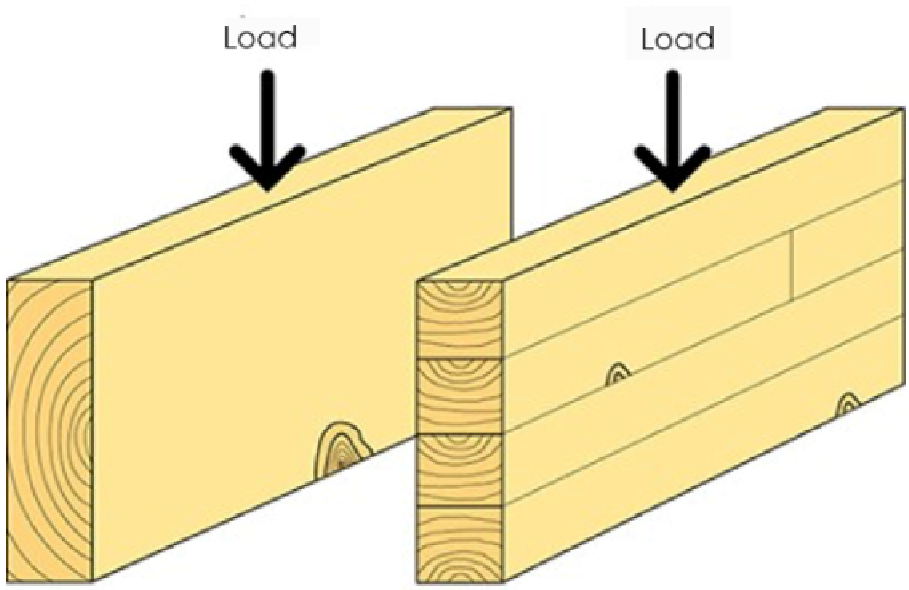

Figure 2.2: Illustration that shows how the glulam technique distributes defects in the material more evenly. Image © Swedish Wood / www.swedishwood.com

2.1.3 Cross laminated timber

Cross-laminated timber, or CLT, is a product category that consist of planed sawn timber lamellas that are glued together in perpendicular layers to form a massive panel. There should be an odd number of layers to make the top and the bottom layer run in the same direction. This creates one main load carrying direction of the panel that have a higher stiffness. To use the material efficiently it is common to have the layers that are perpendicular to the main direction made of lamellas of less strength grading as those layers will not influence the overall load bearing capacity as much. This is similar to combined glulam where the middle lamellas are also made of less strength graded timber.

CLT panels could be used for several structural purposes; floors, roofs and load bearing inner- and outer walls. The in-plane stiffness of the CLT panels makes it possible to achieve diaphragm action. Meaning that the product can be used for the stabilisation of a building both as shear-walls and as floor panels that distributes horizontal loads to the stabilising members. Usage of CLT elements is associated with a high degree of prefabrication where the wall or floor elements could be prepared already in the factory with holes for windows and doors as well as for electrical installations, reducing the need for site work (Swedish Wood, 2015).

2.1.4 Laminated veneer lumber, LVL

Laminated veneer lumber, LVL, is made of thin wooden veneer sheets that are glued together to form thick panels. The panels are then cut to fit the area of use. In the usual set up, all veneers are located with the fibres in the same direction, creating a material that is strong along its length. However, there are also some products available that have a few layers oriented perpendicular to the main direction in order to increase the strength in the orthogonal direction. This could help when performing connections (Thelandersson and Larsen, 2003). Figure 2.3 shows examples of both set ups. Typical areas of use are as beams or as flanges in I-joists.

Figure 2.3: Two smaller pieces of LVL. The left have three layers with the fibres perpendicular, while the right have all layers in the same direction.

2.1.5 Parallel strand lumber

Parallel strand lumber (PSL), or Parallam®, is a product which is uncommon in Sweden, but more frequently used in Canada and USA. In *Timber Engineering* (Thelandersson and Larsen, 2003), the production process of PSL is described thoroughly. First, small strands are cut out from veneers and defect strands are removed. Then an adhesive is applied before aligning and assembling the strands together to a large solid product. The curing is made under heat and large pressure, and afterwards the product is cut to the desired dimensions. Typical areas of use for the PSL technique are as beams and columns. An example of a small piece of a Parallam® is shown in Figure 2.4.

Figure 2.4: Small specimen showing the structure of PSL.

2.1.6 Panels (Plywood, OSB etc.)

There are several different types of EWP panels available on the market, such as plywood, oriented strand board (OSB) and particle board to mention some of the most common. One particular issue to be observant to when it comes to wooden based panel materials, is that they are often sensitive to moisture. When exposed to moisture there will be a risk of change in the shape of the boards. The moisture sensitivity however varies between the type of panel-categories and types.

2.1.6.1 Plywood

Plywood is a common panel type that is built up by an odd number of veneer sheets, which are placed perpendicular to each other (Swedish Wood, 2015). This makes it stiff in both directions. The odd number of veneers make the two outermost veneers run in the same direction (Porteous and Kermani, 2009) meaning that the panel is stronger in the fibre direction of the outermost veneers, similar to CLT. The boards are manufactured with different thicknesses which makes them useful in many areas, for instance as sheeting in both walls and floors.

2.1.6.2 Oriented Strand Board - OSB

Oriented strand board consists of thin wooden strands that are put together using an adhesive. To obtain a higher stiffness in both directions of the panel the wood strands are placed perpendicular to each other where the outermost strands are oriented in the long direction of the panel (Swedish Wood, 2015). The strands in the middle part of the panel are consequently placed perpendicular to the outermost strands. This panel type is the most common in Sweden according to Swedish Wood (2015), and the panel is often used as sheeting for walls.

2.1.6.3 Particleboards and fibreboards

Particle- and fibreboards are similar products which consists of small wooden particles, for instance sawdust or fibres. These are pressed together with adhesives and cured to form a panel that is cut to an appropriate size. There are several types of particleboards available for different purposes. *Structural Timber Design to Eurocode 5* (Porteous and Kermani, 2009) addresses seven different classes for particle boards based on the area of use, from non-structural use in furniture up to "*heavy-duty load-bearing in humid conditions*". According to Swedish Wood (2015), particleboards are most often used as a sheeting material in floors.

2.1.7 Built up cross sections

A way to create material-efficient structural members are by combining different wooden products to build up a specific cross section. Two of the more common built up cross sections are I-beams and box beams.

I-beams are usually built up with webs from some type of board material, which could be either plywood or OSB. The flanges are made of either sawn timber or LVL, and there are two typical ways to attach the flange to the web (Swedish Wood, 2015). The first is to use two separate flange parts, which are glued to each side of the web, both at the top and in the bottom. Another way is to cut a groove in each flange to provide more connection area. The web is then glued into the groove.

An alternative built up section is the box-beam which uses similar materials as an I-beam, but is made with two webs that are a bit separated, creating a hollow section between the webs (Porteous and Kermani, 2009). Figure 2.5 shows examples of both I-beams and box beams.

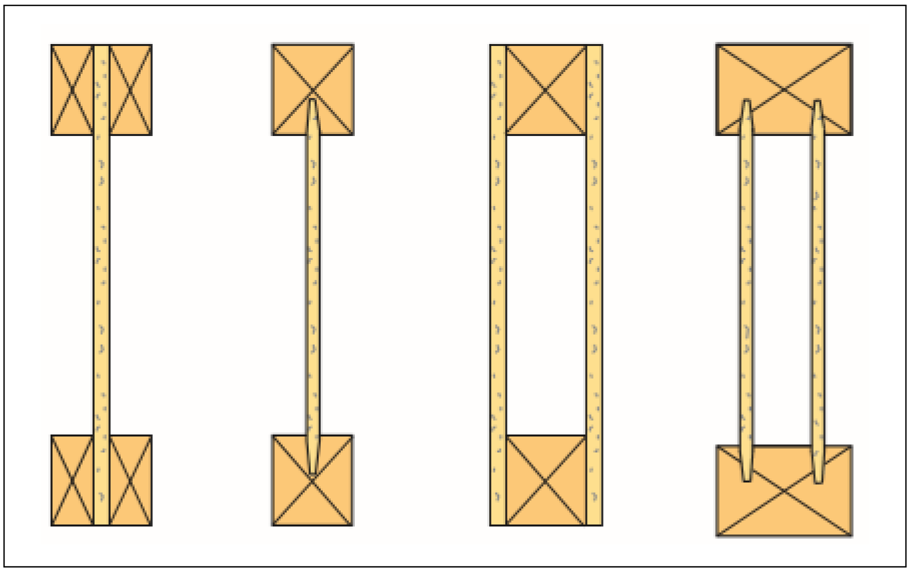

Figure 2.5: Example of different types of built up timber cross-sections. Image © Swedish Wood / www.swedishwood.com

2.2 Hybrid materials

As all materials, timber have both strengths and weaknesses. High strength to weight ratio and easy transportation and handling during construction thanks to the low weight are two of its main advantages (Al-Emrani et al., 2013). On the other hand, low weight could lead to issues concerning acoustics, dynamics and tilting. Also, the cross-section sizes will usually be larger compared to steel or concrete. By combining timber with other materials, hybrid elements can be created which takes the advantages of each material to create something better. The development and research within this area is ongoing, where new applications and methods for timber hybrids are being developed. This chapter will introduce some of the applications for timber-concrete composites and timber-steel composites.

2.2.1 TCC - Timber-concrete composite

The basic concept of timber-concrete composites, TCC, are built on three different parts; a timber element, a concrete section and some type of connection which makes the timber and concrete work together (Kupferlé, 2018). To utilise the strengths of each material, the concrete should work in compression and the timber in tension parallel to its fibres. The connection should create composite action between the two and will mainly take shear stresses.

Extra attention should be put on the connection, as it is of great importance for the behaviour of the TCC (Lukaszewska, 2009). If concrete and timber would be put together without any connector between the two materials, they would just slip freely and work independently of each other. In that case, *no* composite action would be achieved. An infinitely stiff connection would on the other hand keep the materials perfectly together to provide *full* composite action, which could be designed with an equivalent transformed cross-section. However, in practice the connection will give *partial* composite action corresponding to a behaviour somewhere between these two ideal cases, where a small slip will occur between the two materials, illustrated in Figure 2.6. As a consequence of this slip, the assumption in classic Euler-Bernoulli beam theory of "plane sections remain plane" does not hold for the case of partial composite action. Hence, there is need for other methods and models for the design and analysis of TCC's (Yeoh et al., 2011). There is no explicitly stated calculation method for this in Eurocode 5, however in appendix B, a method referred to as the γ-method is presented for design of timber beams with mechanical connectors and partial composite action.

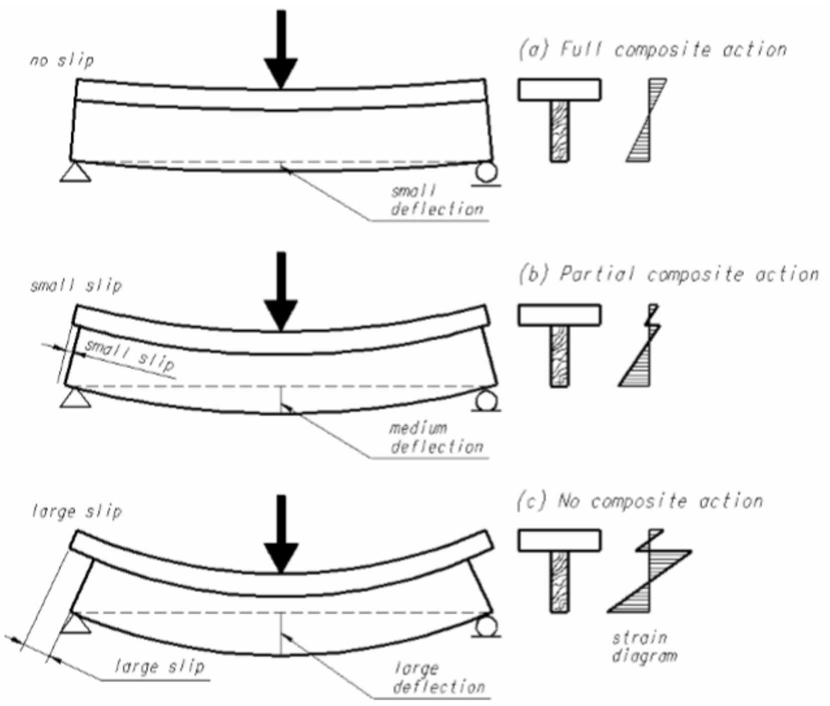

Figure 2.6: Illustrations of different levels of interaction between timber and concrete (Lukaszewska, 2009). Image © Elzbieta Lukaszewska

In addition of having a good connection between the two materials to achieve composite action, it is desirable to design the cross-section in such a way that the neutral axis is located close to the interface of the materials (Yeoh et al., 2011). This to reduce the risk of cracking in the concrete and have it work in compression, while the timber is in tension.

Yeoh et al. (2011) lists several advantages of a TCC floor construction in comparison with a pure timber or concrete floor. Increased stiffness and improved acoustic performance are among the mentioned compared to a timber floor. Less total weight, which also could ease handling and speed up construction, and less CO_2 emissions are the advantages compared to a concrete floor.

Two typical setups of TCC floors that easily comes to mind is either having timber beams to which some type of sheeting material is attached. The concrete is then cast on top of the sheeting. The sheeting could be either board-type such as plywood or panels of CLT. Another variant is to use a thicker massive CLT panel, without beams, on which the concrete is cast directly. Yeoh et al. (2011), states that a high degree of prefabrication which reduces production cost is necessary to make TCC's competitive enough to use in practice. Kupferlé (2018) presents both semi-prefabricated and full-prefabricated solutions. In the semi-prefab solution, the timber parts are manufactured and assembled in factory and then delivered to site where the concrete part is cast in situ with no need for extra formwork. For the fully prefabricated solution there are two alternatives presented. Either, the whole panels including both timber and concrete are done completely in factory so that only the connection to the rest of the system is done on site. Alternatively, the concrete parts are prefabricated separately and on site connected to both the timber part of the slab and to the adjacent panels.

The production methods seem to be not so far from what is used in practice in concrete structures already today. The semi-prefabricated solution has similarities with partly prefabricated floor plates, in the sense that no formwork needs to be built on site. However, it is important to ensure that the timber deck can take the dead-load from the wet concrete during casting, hence propping might be necessary. The fully prefabricated solution with whole TCC panels arriving finished to the building site have similarities with a full-prefab solution in concrete in terms of the need for good connections between the individual elements to achieve sufficient robustness and redundancy in the structure.

Yeoh et al. (2011), points out that the TCC has a really large potential but that it is still not used frequently because of a lack of awareness from the building industry. Some reference projects where TCC floors have been used is presented and described in Section 4.3; *HoHo building Vienna*, and Section 4.5; *J. W. Olver Design building*.

2.2.2 Timber-steel composite

Timber-steel composite (TSC) is another type of hybrid products. Steel is frequently used in connections between timber elements to achieve the desired boundary condition and solve complex connections, an example can be seen in Figure 2.7. However, a more uncommon application is to combine timber and steel to obtain composite action for beams, but there exist a few examples of this. Roof trusses with combination of timber and steel are on the other hand more commonly used.

Figure 2.7: A complex timber-steel connection to obtain a hinged boundary condition. Photo © Åke E:son Lindman

2.2.2.1 Beams

An example of a TSC beam is an I-beam with flanges of timber and the webs of corrugated steel. This type is not that common in Sweden but available on the market abroad. According to WST (2016) the beams can span up to 18 m. Furthermore, WST (2016) states that the beam can be used to produce prefabricate floor and roof elements with dimensions up to 2.5 m in width and 18 m in length. It is possible to increase the stiffness of the beams by adding an extra layer of steel in the web. Beam heights varies between 230-550 mm for a single steel web.

Other types of TSC beams have been tested in research but are today hard to find on the market. One test that has been made during a bachelor thesis in cooperation with Moelven was of a steel plate reinforced glulam beam (Kjellkvist and Lindahl, 2014). The steel plates were placed on different places to study where it had the most efficient utilisation. One of the tested beams can be seen Figure 2.8. The right part of the figure shows a glulam beam with the steel plate in the bottom and the left shows in more detail how it is connected to the glulam. In addition to the mechanical connectors visible in the figure, the steel plate is also glued to the glulam.

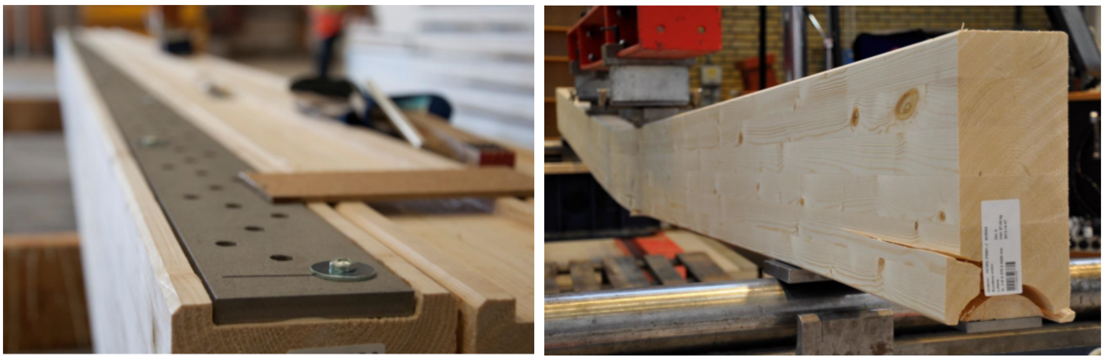

Figure 2.8: Glulam beam reinforced by a steel plate (Kjellkvist and Lindahl, 2014). Photo © Mateusz Kjellkvist and Fredrik Lindahl

2.2.2.2 TSC trusses

The most common TSC structural member is roof trusses where the steel is placed in the bottom to carry tensile stresses while the timber is taking compressive stresses. The steel parts are usually cables that have a parabolic shape or a V-shape, see Figure 2.9 and 2.10 for V-shaped TSC trusses. These types of structural members are advantageous for larger span lengths and usually they become relatively slim, which can be seen in the figures below.

Figure 2.9: V-shaped TSC trusses at the ticketing check in kiosk 02-PD at Raleigh Duram Airport. Photo © Paul Dingman, courtesy of Fentress Architects

Figure 2.10: V-shaped TSC trusses at one of the concourse at Raleigh Duram Airport. Photo © Brady Lambert, courtesy of Fentress Architects

Available on the market but not common are truss-beams which often are called "Space Joist" (Griggs, 2018). A Space Joist is built up by timber in top and bottom as flanges and a web which consists of special nail plates that are nailed between the flanges, see Figure 2.11. Due to its structure they become light-weight and HVAC installations can easier be performed. These truss-beams are advantageous for both floors and roofs.

Figure 2.11: Space joist truss (ITW, 2013).
Photo © ITW / www.flickr.com/photos/itw-industry

3

Timber systems on the market

The interest of building multi-storey timber buildings are increasing on a global level. Several of the timber product suppliers have developed interesting concepts for multi-storey timber buildings. The following chapter will introduce the types of structural systems that different manufacturers currently are producing. The companies are selected to give a variety of system types. Some examples that are brought up includes glulam beam-column systems, CLT-systems and timber-concrete hybrids. For each system, key data such as recommended span lengths, building heights and sizes given by the suppliers are presented. The section starts off with suppliers that are based in Sweden, followed by some of the largest foreign suppliers.

3.1 Moelven, Sweden/Norway

Moelven is a supplier and manufacturer of different type of wooden products, especially glulam. In year 2009 they launched a beam and column system for both smaller and larger buildings named Trä8 (Moelven, 2020). The structural elements consist of glulam beams and columns and the floor modules are built up by glulam, Kerto® LVL and insulation. All parts in the system are prefabricated to a high degree. The floor modules are claimed to be able to span 8 meters and are 470-590 mm thick to be able to fulfil sound class B (Moelven, 2019). The system has been used in two famous pilot projects in Norway, Treet, which is 14 stories and Mjöstornet, which is 18 stories. Figure 3.1 shows the system during production of another residential building project at Askims Torg.

Moelven does not produce any walls for the structural system which means that it is up to the contractor to decide which wall type that should be used. Even if they are not producing walls they have made a suggestion on a detail solution of how to connect walls and floors to fulfil sound class B, see Figure 3.1 (Moelven, 2019). In the suggested detail it can be seen how the wall and floor are connected to each other with as less attachment to the framework as possible. It can also be seen how the frameworks are separated from each other which leaves an air gap between the frameworks to minimise flanking transmission. Insulation is advantageous when it comes to improve noise reduction which also can be seen in the suggested detail.

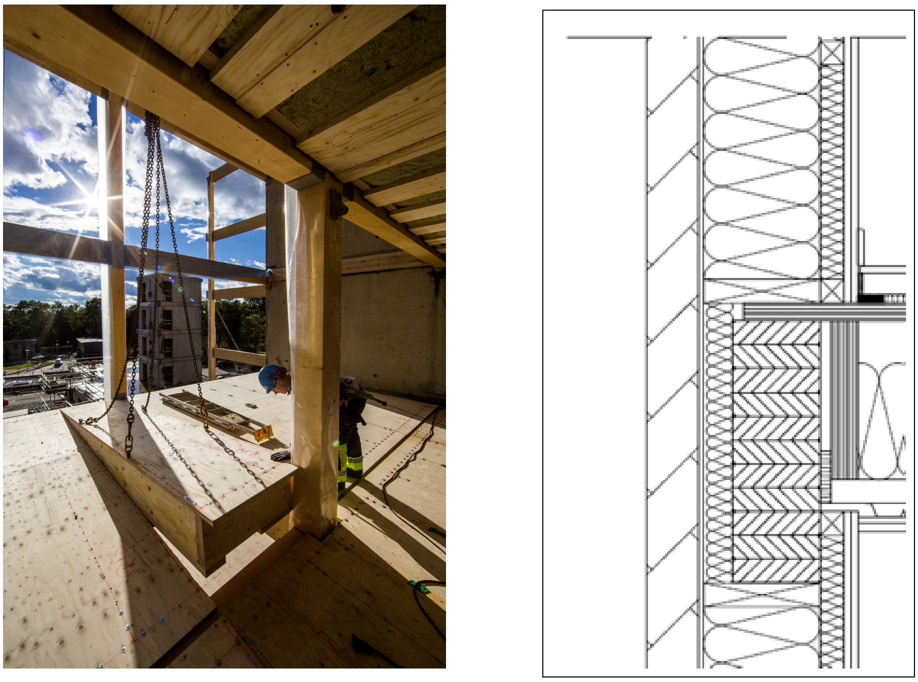

Figure 3.1: Left: Lifting of a Trä8 floor cassette into position. Photo © Moelven, Sören Håkanlind. Right: Illustration of a suggested connection between a Trä8 element and the wall, to fulfil sound class B. Image © Moelven

The beam and column system carries all vertical loads while the staircase and elevator shaft is commonly used for the stabilisation of the building (Moelven, 2019). This is certainly applicable in buildings where a framework made totally of timber would be too lightweight in terms of global stability (tilting), therefore extra weight might be necessary to put in. A heavy concrete core for the elevator shaft could in that case be used to solve both tilting and horizontal stability of the individual parts. Another way to stabilise the building is to have truss systems in the inner- and/or outer walls, the trusses could be either made in timber or steel (Moelven, 2019). A third possible solution would be to use CLT-panels as shear walls for the stabilisation. Shortly, Moelven has an idea of using the advantages of different materials and use the right material in the right place to create a better solution in the end.

Moelven also has another concept which consists of prefabricated modules/volumes. These modules are totally prefabricated, including electrical and plumbing works. This way of building with prefabricated modules is commonly called industrialized building and according to (Moevlen, 2020) both a faster production and higher quality can be achieved as most of the work is done under more controlled conditions. They also claim that it is better from a moisture damage perspective, as the whole modules are finalised indoors in the factory. Furthermore, the modules are, if necessary, movable in the future. According to the Norwegian part of Moelven Byggmodul, the apartment modules are produced in widths from 2.3 to 3.98 meters, in lengths of 12 meters, and it is possible to build 2-5 stories with the modules (Moelven Byggmodul, 2020).

3.2 Martinsons, Sweden

Martinsons is a manufacturer based in the north of Sweden. Apart from manufacturing wooden products such as sawn timber and glulam, they have also developed a structural system for residential buildings which uses CLT-panels as the main structural element. In a product information and guidance sheet they claim that the system can be used to build up to 8 stories (Martinsons, 2019b). The system is further described to be object-adapted, using CLT in load-bearing walls and floors. For partition walls, timber stud-walls are mentioned as an alternative to the CLT-panels. Martinsons provides only the timber parts of the CLT-system, which needs to be complemented with solutions to fulfil the requirements for fire and acoustics (Martinsons, 2019b). These solutions should be provided by the contractor and could differ in execution depending on the requirements set for the specific building.

The CLT-panels are produced with 3,5 or 7 layers up to a size of 3x16 meters, but in order to ease the handling during construction it is recommended to use a maximum length of 12 meters (Martinsons, 2019b). To give an idea on the capacity of the CLT-panels, a table of the maximum span lengths for different panels is presented on their website (Martinsons, 2019a). The table shows that the thickest panel with seven layers and a total thickness of 280mm can span 6.6 m to fulfil *recommended* eigenfrequency and springiness for residential buildings. The same panel can span 7.4 m to fulfil the corresponding requirements for an office building. However, it is important to keep in mind that the thickness is only for the CLT-plate itself which needs to be complemented with insulation and boards to fulfil acoustic and fire requirements, meaning that the actual floor thickness in the finished building will be clearly larger. As a complement, *usual* thicknesses for finished building elements is presented as (Martinsons, 2019b):

- CLT floor incl. topping: 400-500 mm
- Inner-wall, CLT: 250-380 mm
- Inner-wall, studs: 264-344 mm
- External wall, CLT: 350-450 mm

These thicknesses only serve as an indication, as it will be largely dependent on the building type, acoustic and fire requirements, as well as actual span lengths. Also, the geographical location will influence the thickness of the external wall.

Some advice that are given on how to perform good floor plans with the system is to use a similar plan for all floors and to not shift load bearing walls but keep them straight above each other. Shifting would result in higher costs both during planning and construction as quite complex solutions needs to be performed (Martinsons, 2019b). Stabilisation of the building is performed by diaphragm action in the CLT-panels in both floors and walls, which will bring horizontal loads down to the foundation. Therefore, symmetry in the plan and larger parts of the walls without openings are stated to be favourable for the stabilisation, especially in the lower floors where the load effects from horizontal forces are largest.

3.3 Setra, Sweden

One of Sweden's largest wood processing industries, Setra, are on their way to start up a new industry that will manufacture CLT panels. They claim that they will produce the largest CLT-panels on the market and that the production will start during the first half of 2020 (Setra, 2019b). The new industry is built to meet what they see as an increased demand of CLT-panels used as structural elements for residential buildings (Setra, 2018). It will be located close to one of their already existing glulam factories, and the idea is to be able to deliver structural systems where both CLT and glulam products are used.

Setra will produce CLT-panels in sizes up to 3.5 x 20 meters (Setra, 2019a), with two different setups. The first is the traditional one where each layer are perpendicular to the previous, in 3, 5 or 7 layers up to a thickness of 280 mm. The second type is manufactured with the two outer layers on each side put in the same direction, creating panels that have an even more defined main load carrying direction.

3.4 Masonite beams, Sweden

Masonite Beams AB is a company that are specialised in manufacturing lightweight I-beams with a web of OSB sheets and sawn timber as flanges. The I-beams are used in several types of building elements, including load-bearing walls, floors and roof structures. Using these elements, with I-beams as the main load-bearing part, they have developed a system called *Masonite Flexibla Byggsystem* (MFB). This could be translated to *Masonite Flexible Building Systems*. The system also covers typical connections between the elements. It is claimed to fulfil all requirements concerning fire, moisture and load-bearing capacity together with extra high set requirements for acoustics and energy consumption (Masonite Beams AB, 2010).

In the handbook *MFB Handbok* (Masonite Beams AB, 2010) a thorough description on how to build with the system in practice is given. The system is said to be suitable for both smaller houses and multi-story buildings up to 8 stories. A floor solution has been developed using insulation between the I-beams and placing different types of boards on both top and bottom of the beams. The floors are claimed to be able to span up to 7.1 m if simply supported and 8.1 m if placed continuous over two symmetrical spans. The total height of the floor is then 494 mm. The system uses primarily a hanging connection between floor elements and walls. This because the structural system is of balloon type, meaning that load-bearing walls are connected directly to each other. A principal illustration of a connection between the floor and wall element is shown in Figure 3.2.

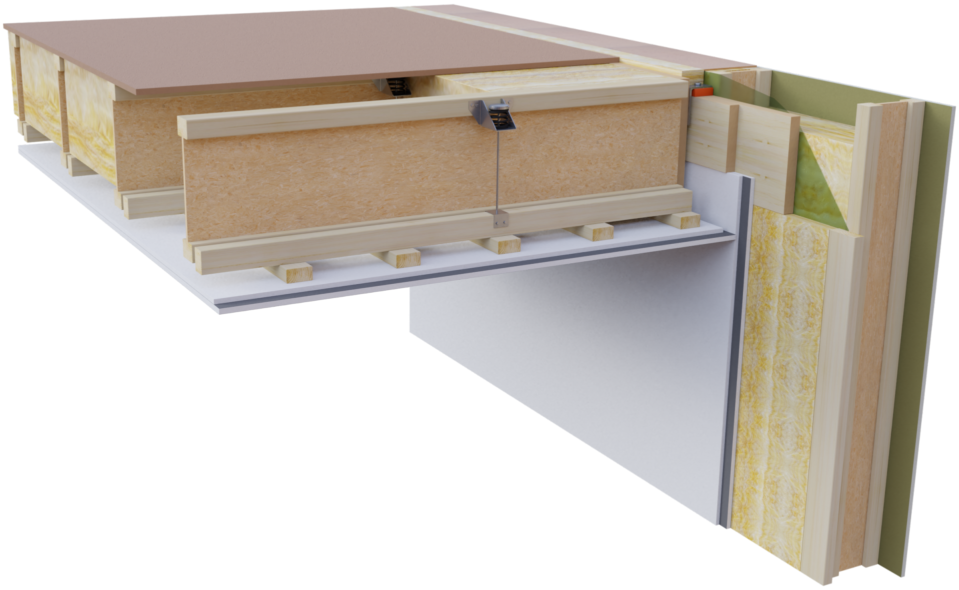

Figure 3.2: Illustration of a connection between a floor and wall element with the MFB system. Image © Masonite Beams AB

Wall elements are produced to a maximum height of 3 meters and length of 9 meters. However, to achieve a more efficient transportation it is recommended to divide 9 meter walls into sections of each 4.5 meters (Masonite Beams AB, 2010). The same sizes and recommendations applies for floor elements. Even though the system is mainly suited for solutions with load-bearing walls, adaption for a beam-column system using specific parts and developed details could also be made.

From the *MFB Handbok* (Masonite Beams AB, 2010), depending on the application, the thickness of different elements are mentioned to vary between:

- Floor system: 444-494 mm
- Outer-wall: 349-549 mm

3.5 KLH Massivholz GmbH, Austria

KLH Massivholz GmbH is an Austrian CLT producer who are at forefront in the field (KLH, 2019c). They are producing different structural elements of CLT, like walls and floors. Their products are both suitable for single-family houses and multi-storey buildings such as residential and office buildings. KLH offers a whole structural system in CLT including floors, walls and roofs and delivers the structural parts to the construction site where they are assembled together (KLH, 2019b).

KLH has created a whole series of manuals for different type of building solutions. Among these are for instance rib elements, multi-storey buildings, structural pre-analysis tables, timber-concrete composite floors, assembly and installation etc (KLH, 2019a). In these manuals a wide range of suggested detail solutions are presented for outer- and inner walls, roofs and terrace connections etc.

KLH have three different types of floor structures, standard CLT elements, rib elements and timber-concrete composite floors. Standard CLT panels are beneficial for spans between 3-7 meters and have a thickness between 120-280 mm (KLH, 2019e). Using only CLT as a floor is not advantageous when it comes to vibration, fire and sound requirements. Therefore, it is necessary to keep in mind that the total thickness of the floor will increase when additional solutions are applied to fulfil requirements of those aspects.

KLH also manufactures so called rib elements. These consists of a CLT panel that is reinforced by ribs (beams) underneath the panel, forming a T-section (KLH, 2019d). These are advantageous for larger spans between 6-10 m for floor structures and for roof structures up to 12 meters. Heights of the ribs varies between 240-600 mm depending on span lengths and applied loads. The CLT panel gives an additional height of approximately 110 mm which means that the total height will vary between 350-710 mm. In Figure 3.3 it can be seen how the rib elements are assembled and that a space between the ribs are obtained which can be used for HVAC and other installations. Furthermore, noise reducing elements and/or elements for an aesthetic appealing surface can be placed in these spaces (KLH, 2019d).

Figure 3.3: Two different ways of how to utilise the space between the ribs. Images © KLH / Rib Elements Version 09/2019

Two different solutions of how to connect two rib elements to obtain shear resistance so that the floor or roof structure can act as a diaphragm are presented in Figure 3.4 (KLH, 2019d). The left one is what KLH call a "stepped joint connection" where the two CLT panels are overlapping and screwed together. The solution to the right is what they call "top board connection" which means that the two CLT panels are connected with a wooden board which is placed in a recess between the elements.

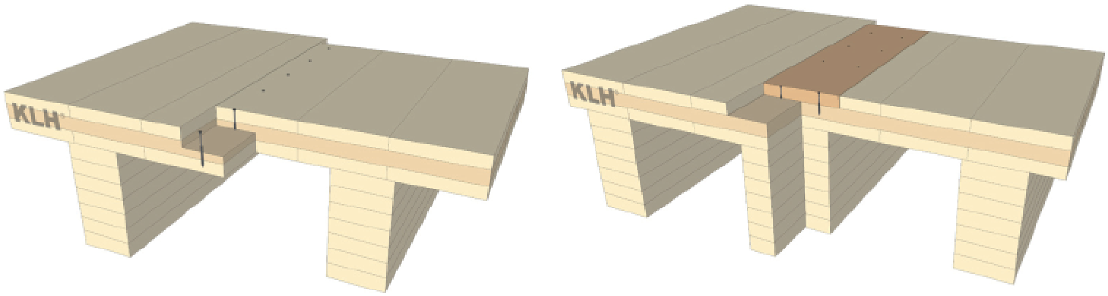

Figure 3.4: Two different connections to obtain shear resistance between two rib elements. Images © KLH / Rib Elements Version 09/2019

The floor to wall connections for rib elements have three possible solutions presented by KLH (KLH, 2019d). The three alternatives have the ribs placed either in, on top or against the support which is shown in Figure 3.5. In the solution to the left, a cut out is made in the wall to make the ribs fit into it. The middle solution shows a placing of the rib element on top of the support and additional cover of the unwanted holes that occurs. To the right, a solution is shown where the ribs are ending just before the support, with the CLT panel resting on the support while the ribs are connected to a joist hanger for instance.

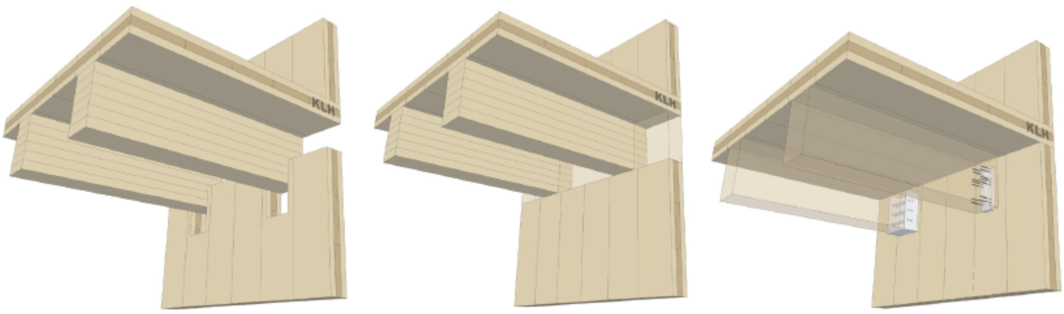

Figure 3.5: Three possible connections between support and rib element. Left: Cut out in the wall to fit in the rib-element. Middle: Rib-element is placed on top of the wall. Right: The ribs are connected against the wall while the CLT is laying on top of the wall. Images © KLH / Rib Elements Version 09/2019

The last floor type is a TCC floor. KLH does not have any standard solution for this type of floor which means that each system needs to be calculated individually (KLH, 2019f). KLH uses different approaches to obtain a good composite action such as glued perforated steel plates, flat steel strips or screw connectors which are placed in a specific angle. According to this manual a table is showing that the TCC elements have span lengths between 6.5-9 m and a total thickness between 240-300 mm. These sizes are only for preliminary design but gives a rough estimation of the thickness in relation to the span.

For residential buildings in Scandinavian countries, the noise requirements are among the highest (KLH, 2012). Therefore, a bit more attention is needed when designing connections to improve the noise reduction and it is recommended by KLH to bring in an expert within the field when the requirements are very high. The same recommendations are made when it comes to building physics and fire safety.

3.6 Stora Enso, Finland/Sweden,

Stora Enso is a large company within the wooden industry and has a wide range of wooden products. Together with Ramböll, they have developed a building concept that consist of combinations of CLT walls/floors, glulam beams/columns and ribbed floor panels. The building concept is suitable for both office and residential buildings, but the main load-bearing structure are usually chosen differently depending on the application. In the office building concept, glulam columns are dominant when it comes to vertical structural members to create more open spaces, which otherwise could be limited if using CLT-walls (Stora Enso, 2018a). The main structural system of the residential building concept consists instead of load-bearing CLT walls to separate apartments and rooms within an apartment (Stora Enso, 2016).

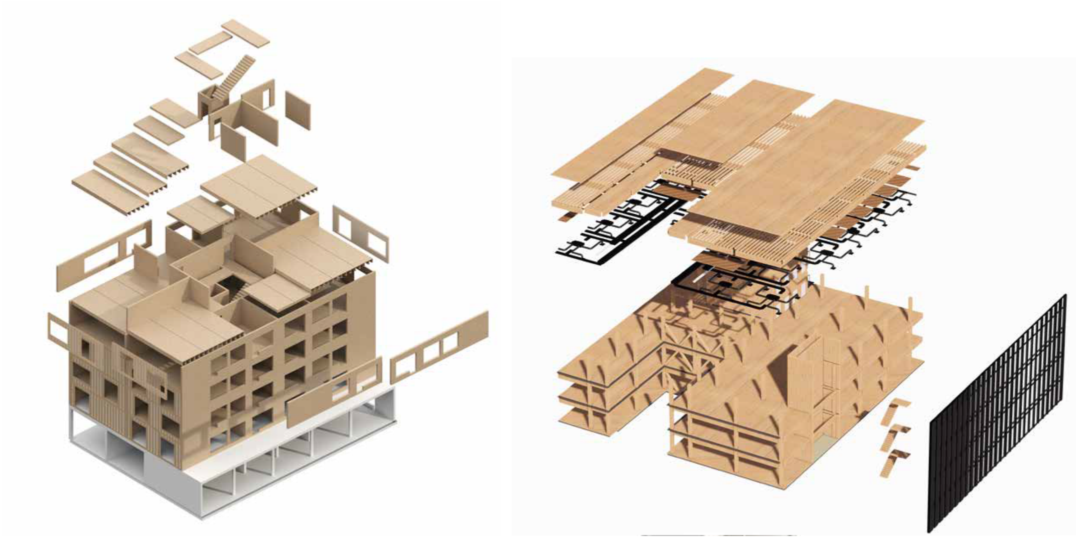

Figure 3.6: Stora Ensos building concepts. Left: Residential building (Stora Enso, 2016). Right: Office building (Stora Enso, 2018a). Images © Stora Enso / www.storaenso.com/en/products/wood-products/building-concepts

Stora Enso is providing a manual for their office building concept that contains guidelines for the preliminary design of a timber building. From an architectural perspective the building concept becomes a guide to help them fulfil the requirements of various types of office buildings (Stora Enso, 2018a). From the structural engineers perspective, the manual makes it easy to apply the building concept since it gives recommendation of grid planning, dimensions of floors, beams, columns and recommended bracing system for different number of floors. Acoustics and fire protection, two important aspects for timber buildings, is also brought up in the manual. Other aspects included in the manual are solutions for HVAC and some economic aspects.

They have also developed a concept manual for residential buildings. The basic idea behind the concept is to use massive CLT walls between the apartments together with ribbed floor elements with sufficient span lengths to keep the apartments free from additional load-bearing elements (Stora Enso, 2016). Depending on the chosen direction of the floor slab, some of the facades will be non load bearing, leaving room for more openings. In addition to floor and wall types, the manual also presents different types of fire protection, bracing systems and joints. The components and details are developed to optimise the performance and are carefully selected to fulfil the most demanding building regulations (Stora Enso, 2016).

Stora Enso have two main floor types in their concept manual: LVL rib slabs and massive CLT plates. Both are complemented with different boards and insulation etc. For a generic residential building of 4-7 storeys, building elements with the following varying thicknesses where proposed (Stora Enso, 2016).

- CLT external wall: 318-521 mm
- CLT partition wall: 318-511 mm
- CLT elevator shaft: 140-150 mm
- Rib slab, Apartment: 531-607 mm
- Rib slab, Bathroom: 726 mm
- CLT slab, Apartment: 230-345 mm
- CLT slab, Bathroom: 388-450 mm
- CLT slab, Balcony: 170 mm

Both the wall and floor thicknesses are varying depending on the function and different requirements. So, for a high performing building, the element thicknesses will probably be in the higher part of the scale. These two manuals are made to get a hint on a preliminary design of a timber building i.e. it is not representing a final design so calculations still needs to be done by a responsible designer.

Apart from the two floor types presented above, they have also two types of CLT rib panels. One with only a CLT panel on top of ribs (beams), and one with a CLT panel both on the top and bottom of ribs. The second element type is called a *closed rib element* (Stora Enso, 2018b). Stora Enso presents in their preliminary design table span lengths up to 12 meter for ordinary rib panels.

3.7 Cree by Rhomberg, Austria

Cree by Rhomberg is a subsidiary to the Austrian building company Rhomberg Group. They have in the project LifeCycle Tower, developed a hybrid beam-column structural system concept called LCT. This concept uses timber in combination with concrete and steel. The system is modular based and characterised by its high degree of prefabrication and standardisation. It is built up in modular grids which sets some boundaries for the structural parts, but non-structural walls can still be placed at different locations between the columns, so that creation of different plan layouts is possible (Cree by Rohmberg, 2018). This gives the possibility to specially adapt the system for offices, apartments or hotels.

The LCT system is a beam-column system stabilised by a concrete core and resting on a concrete foundation. It is described in the Planning Manual (Cree by Rohmberg, 2018). All the columns along the facades are made of glulam, while those inside the building are concrete-steel columns. The flooring system consists of fully prefabricated timber-concrete composite elements. These are made of glulam beams in the longitudinal direction, on which an approximately 100mm thick concrete layer is cast. Interaction between timber and concrete is created with notches in the glulam beams. Along the short edge of the element, an edge beam of reinforced concrete is cast, see Figure 3.7.

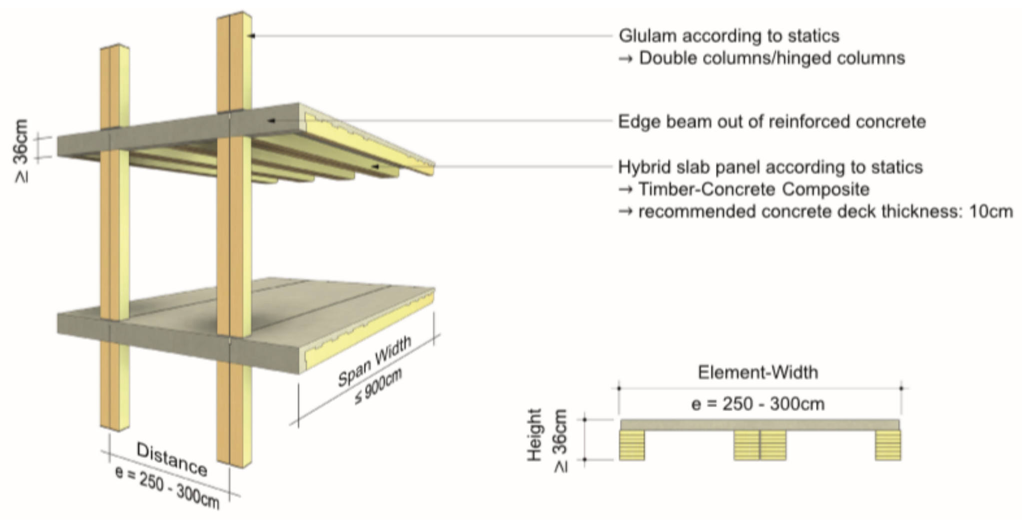

Figure 3.7: Illustration of the facade columns and hybrid floor construction for the LCT-system. (Cree by Rohmberg, 2018). Image © Cree by Rohmberg

In the largest variant, a maximum free span between columns of 9 meters in both directions is possible to achieve (Cree by Rohmberg, 2018). Furthermore, the standard floor height is 3.5 meters, with a free height of 3.0 meters, which indicates that the floors will have a total thickness of 500 mm. The facades are generally not load-bearing, but additional stiffening wall panels could be added if needed for horizontal stabilisation. The first building that was built with the system is the 27 meters high, 8 story *LCT One* building, located in Dornbirn, Austria.

4

Ground breaking timber projects

During the latest years, several ground-breaking timber buildings have been brought up around the world. In Norway, the building Treet in Bergen set the new world record height with 51 meters in 2014, but the record was beaten only a few years later when Mjøstårnet were built as high as 85.4 meters. In Austria, a 24-storey timber-hybrid building are currently being finished, with the first shops opened in January 2020. The challenges of constructing higher and higher timber buildings drives innovation where new solutions are created when pushing the limits. The following chapter describes the structural systems for a selection of modern timber buildings. Emphasis have been put on gathering projects with a wide range of different systems, where innovation have been clearly a benchmark for the projects. Different types of solutions are presented together with some sizes for the structural members used in the projects. These state-of-the-art examples will serve as inspiration both when designing more regular timber buildings in the future, as well as when further pushing the limits in timber building design.

4.1 Mjøstårnet, Brumunddal, Norway

In the small Norwegian town of Brumunddal the 18-storey high timber building Mjøstårnet is located. The top of the building reaches 85.4 meters, which at the time when it was built, set the new world record height for a timber building. The building houses offices, hotel rooms, apartments, a cafeteria and a restaurant.

The structural system of Mjøstårnet, together with some interesting aspects such as dynamics and fire resistance is described in the conference paper *Mjøstårnet - Construction of an 81 m tall timber building* by Abrahamsen (2017). The structural system is based on Moelvens Trä8 system, with a glulam beam-column system as the primary vertical bearing on which the prefabricated Trä8 floor modules are placed. These floor elements were previously described in Section 3.1. For the seven top floors, 300mm thick partly prefabricated concrete floors were used instead of the Trä8 cassettes to increase the mass in the top of the building. This was needed to fulfil the comfort criteria for horizontal movements. The facades of the building envelope is covered with prefabricated sandwich elements which is not a part of the load-bearing structure. Figure 4.1 shows an illustration of the structural system.

Furthermore, Abrahamsen (2017) also describes how the facade columns are connected by diagonal struts to form glulam trusses along the whole height of the building. The trusses are used to increase the stiffness of the building and carry both the vertical and horizontal loads down to the thick pile supported concrete slab foundation. The elevator shafts and staircases are made with CLT walls, which only carry vertical loads and is not a part of the horizontal stabilisation system. The connection between glulam parts are performed with dowels and so called slot-in steel plates, see Figure 4.2. To protect the steel in the connections in case of fire, gaps and slots between elements are covered with a heat expanding fire strip.

Figure 4.1: Left: Structural system of Mjøstårnet (Abrahamsen, 2017). Image © Sweco AS. Right: Picture of the finished building (Abrahamsen, 2018).
Image © Moelven Limtre AS

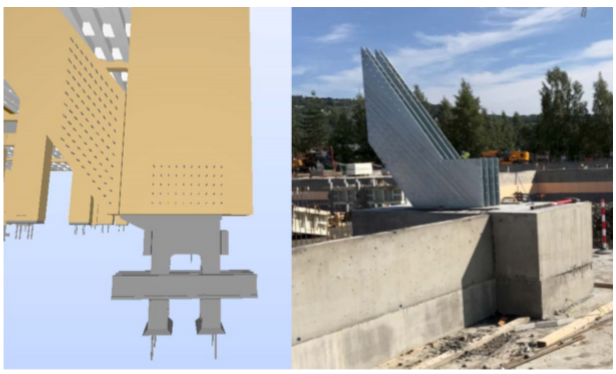

Figure 4.2: Connection between corner column and foundation, both in 3D model and during construction (Abrahamsen, 2017). Image © Moelven Limtre AS

Abrahamsen (2017) gives some examples of cross-sectional sizes for beams, columns and bracing:

• Corner columns:	1485mm x 625mm
• Internal columns:	725mm x 810mm
	625mm x 630mm
• Beams for timber floors:	395mm x 585mm
	395mm x 675mm
• Beams for concrete floors:	625mm x 990mm

One interesting aspect of the assembly of Mjøstårnet is that Moelven decided to go for a new construction strategy. On previous large projects such as Treet in Bergen a test assembly of complex parts such as the large trusses where first made in the factory to ensure that all parts were fitted to perfection. This was not done before the assembly of Mjøstårnet, but instead all parts where prepared individually into detail and assembled directly on site (Abrahamsen, 2018). This was successful and only one part of the several hundred did not fit and needed to be replaced by a new one.

4.2 Brock Commons, Vancouver, Canada

Brock Commons is an 18-storey hybrid mass timber building which is located in Vancouver, Canada. It is 53 m high and are built as a residential building for students. Of the 18 storeys, only the foundation and first floor are made in concrete; the other 17 are timber floors (Fast and Jackson, 2018).

The structural system is built up by CLT panels which are point-supported by timber columns (Fast et al., 2017). CLT is usually used as one-way slabs and the two-way action is often neglected, but for Brock Commons the two-way action is utilised which means that no beams between the columns were needed to support the CLT panels (Fast et al., 2017). The behaviour of the system could be comparable with a cast in-situ flat slab.

It is well known that it is more complex to fulfil requirements for stability when it comes to high-rise timber buildings compared to steel or concrete buildings. Even if the stabilisation could have worked with two cores of CLT panels it would be too time-consuming and costly to ascertain regulatory approvals which would delay the construction time (Fast et al., 2017). Instead, the solution became to build two concrete cores to transfer horizontal loads to the foundation. The two concrete cores and first floor can be seen to the left in Figure 4.3 and the timber floors can be seen to the right in the same Figure.

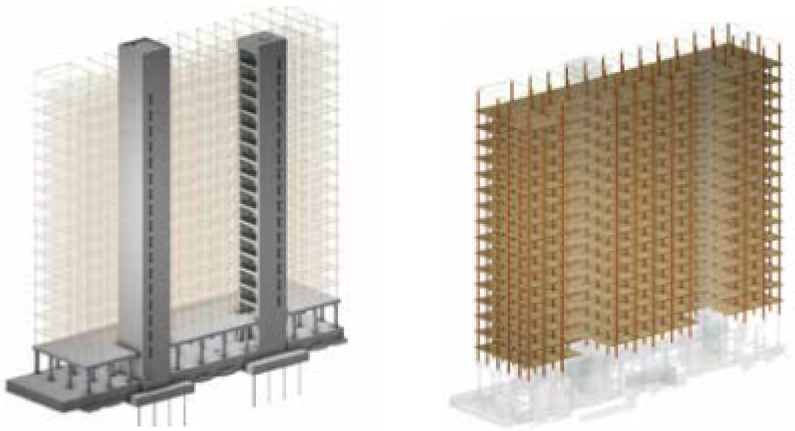

Figure 4.3: Structural parts of concrete and timber in Brock Commons. Images © Cadmakers Inc / www.cadmakers.com

According to Fast et al. (2017), the CLT panels are connected to each other by a plywood with the cross-section 25 mm x 140 mm, which is placed in a milled track between two panels. To achieve diaphragm action the plywood is then screwed into the CLT, which transfers horizontal loads to the two concrete cores. The connection between the cores and the CLT slab is important to make this work properly. This connection is solved by placing steel straps on top of the CLT and connect them to steel plates that are cast-in to the concrete cores (Fast et al., 2017). Kaufmann et al. (2018) presents the following dimensions of structural timber members used in the Brock Commons building:

- Timber columns: 260mm x 260mm
- CLT panels: 2.85m x 4.0m
- CLT thickness: 166mm

The production started by building the two cores in full height before assembling the timber parts (Fast et al., 2017). Making the cores first facilitated the use of only having one crane. At the same time as the concrete were cast on site, the timber manufacturer had plenty of time for the production of the prefabricated timber elements. After this stage, the built up of the timber structure were started. It took approximately nine weeks to raise the timber structure, which corresponds to two floors each week (Fast and Jackson, 2018).

The timber structure is only visible on the top floor, in all other spaces the timber is covered by plasterboards for fire protection (Kaufmann et al., 2018). The CLT panels themselves are not noise reductive enough. Therefore, to improve the acoustic performance of the building a 50mm thick layer of concrete were cast on top of the CLT panels (Fast and Jackson, 2018). This concrete does not have any structural connection to the CLT and is therefore considered non-structural. Furthermore, a dynamic wind load analysis were required as the building had a fundamental natural frequency less than 1 Hz (Fast et al., 2017). Other important aspects that is further described in the article by Fast et al. (2017) are rolling shear failure of the CLT and column shortening. Since CLT panels are built up by layers that are perpendicular to each other, rolling shear failure can govern the design. Therefore, have this been a typical important design control of the building. For column shortening, which needs to be considered during design, one main concern is the elevation tolerance between the timber structure and the two concrete cores (Fast et al., 2017). To handle the column shortening deformation, at nearly 45 mm for the whole building, steel plates were added to columns at three levels to offset the expected deformation.

Progressive collapse/robustness has been taken into account and have been solved by "column tie method" (Fast et al., 2017). This means that if a column fails at one floor, the floors above will contribute to keep the floor below in its place and spread the loads to adjacent columns. The vertical ties are created by using steel connectors with glued in bolts when connecting the columns. As illustrated in Figure 4.4, the CLT panels are placed onto and attached to bolts pointing up from the connector.

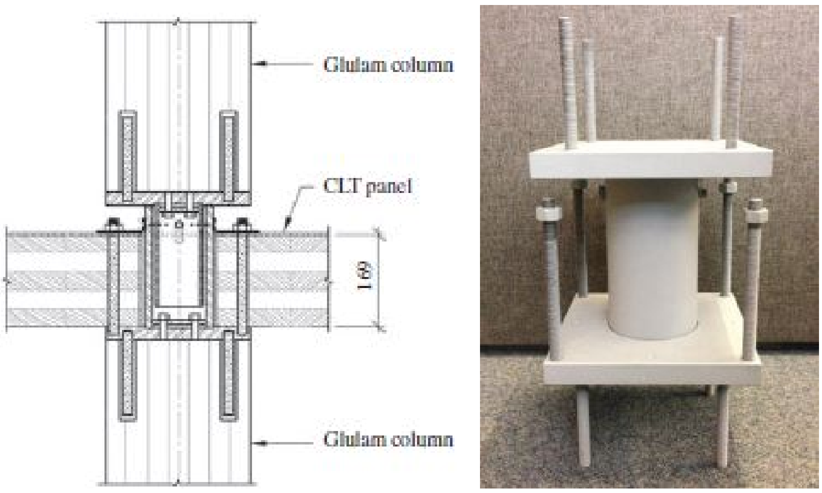

Figure 4.4: Column to column connection (Fast and Jackson, 2018). Images © Fast + Epp / www.fastepp.com

4.3 HoHo building, Vienna, Austria

The HoHo building in Vienna is a high-rise timber building that houses commercial spaces, offices, apartments and a hotel. The building consists of an 84 meter high main tower with 24 storeys, complemented with two side-buildings of each 57 and 40 meters height, with 16 and 11 storeys respectively.

The structural system has a rigid core of reinforced concrete, to which the rest of the structure is attached (Woschitz and Zotter, 2017). This is illustrated in Figure 4.5. The outer structure is characterized by a high degree of prefabrication and is built up with glulam columns, CLT wall elements and timber-concrete composite floor decks, which rests on precast reinforced concrete girders. Jockwer (2019) presents the sizes of the elements used in the system:

- TCC slabs: 2.7m x 7m
- CLT wall elements: 4.8m x 3.3m
- Concrete edge beam: 400mm x 600mm
- Glulam columns: 400mm x 400mm to 400m x 1240mm

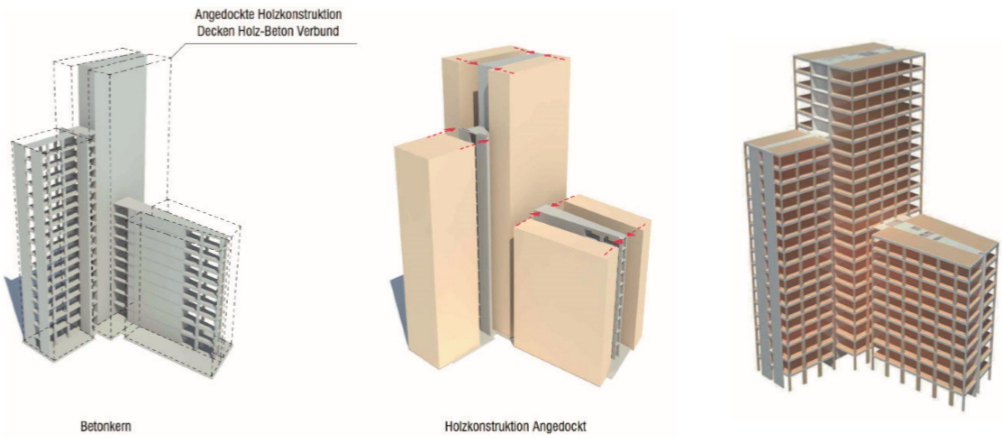

Figure 4.5: Illustration of the structural concept in HoHo Vienna. A timber hybrid structure that is attached to a concrete core (Woschitz and Zotter, 2017). Images © RWT plus ZT GmbH

Some of the key issues of the construction, including robustness and horizontal stabilisation, were described in the article by Woschitz and Zotter (2017). The solution presented for robustness consists of vertical reinforcement bars glued into the top and bottom of each column. In the edge beam a corrugated metal tube is cast and during the construction, the bars from the column are stacked into the tubes. The space inside the metal tube is then grouted to achieve interaction between the edge beam and the columns. This means that if a column on one floor fails, the column from the floor above will lift the edge beam through tension in the reinforcement bar. The tensile stresses will then develop inside the edge beam, which can be described by a strut and tie model where stirrups are placed to handle the tensile stresses. An illustration of such a connection is shown in Figure 4.6.

The edge beam will also work to tie the individual slab elements together all around, with connections described by Woschitz and Zotter (2017). The connection is achieved by saving a small gap between the concrete in the edge beam and the corner of the slabs. Reinforcement from both elements is pointing out into the gap, and when the elements are placed during construction, this gap is grouted to create interaction between the elements. The reinforcement in the gap is placed in both the direction along the beam and perpendicular to it, to be able to transfer stresses in both directions. A similar connection is also made in the middle of the span between the individual slabs and towards the concrete core. This tying system will make the elements work as one unit to distribute and carry the horizontal loads to the rigid concrete core, where the forces are taken down to the foundation. A detail of the connection between the slab and edge beam is illustrated in Figure 4.7. Figure 4.8 shows an illustration of the whole tying system of a floor plan.

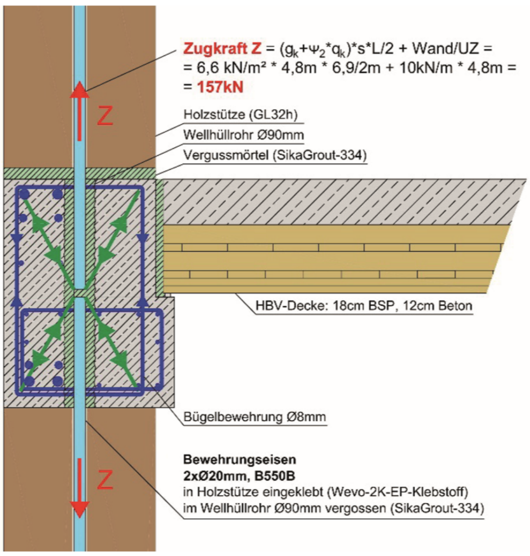

Figure 4.6: Illustration of vertical connection between the glulam columns and the edge beam in the HoHo Vienna building (Woschitz and Zotter, 2017). Image © RWT plus ZT GmbH

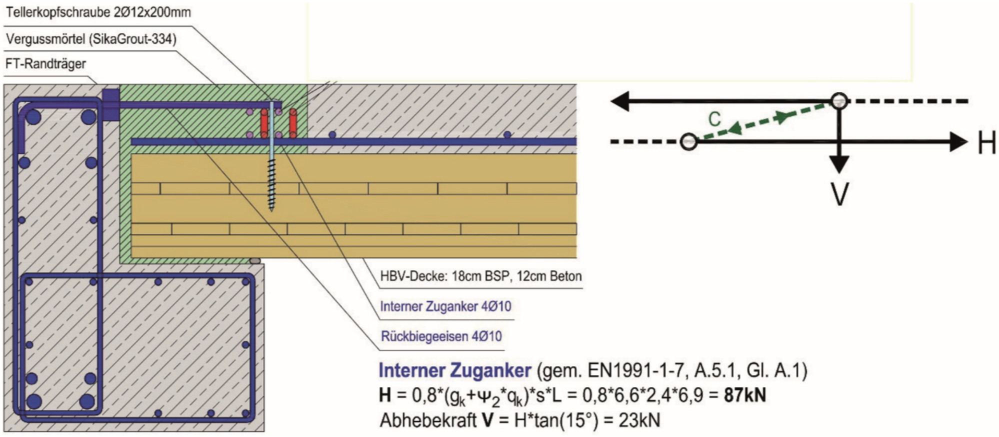

Figure 4.7: Illustration of a horisontal connection between a floor slab and the edge beam in the HoHo Vienna building (Woschitz and Zotter, 2017). Image © RWT plus ZT GmbH

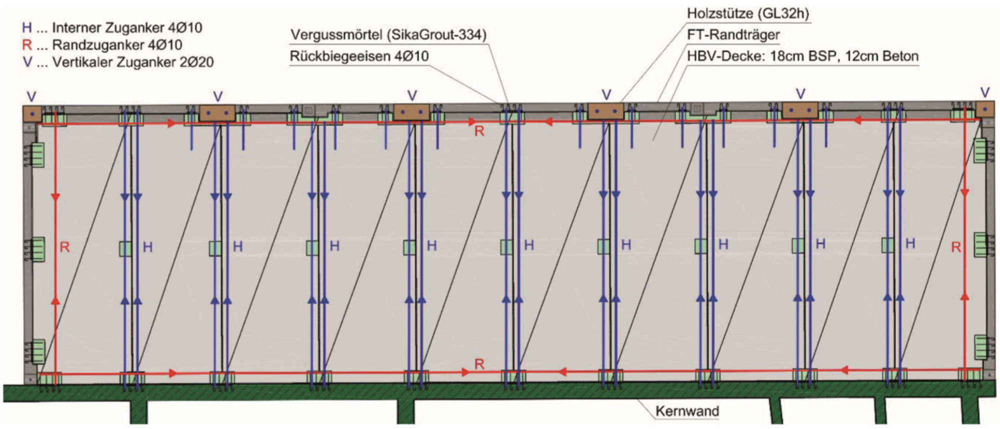

Figure 4.8: Illustration of the tying system for a floor plan in the HoHo Vienna building (Woschitz and Zotter, 2017). Image © RWT plus ZT GmbH

4.4 Origine condos, Quebec City, Canada

In Quebec city in eastern Canada, the 13 storey Origine building is located and stands out from the neighbourhood thanks to its height and striking appearance (Cecobois, 2018). With a height of 40.9 m Origine condos became, when completed in October 2017, the tallest building ever made with a structural system entirely made of timber according to the case study by Cecobois (2018). It is built as a residential building with 92 housing units with different sizes of the apartments.

The structural system is mainly made of CLT panels, complemented with beams and columns. The CLT walls are divided in two categories where some are constructed to resist lateral load and the others are designed to resist vertical loads together with the columns and beams (Cecobois, 2018). The different structural parts of the building can be seen in Figure 4.9. The shear walls are blue, vertical load bearing walls are purple, beams are green, and columns are yellow. Lateral loads are transferred to the shear walls by diaphragm action in the floor. The bottom of the shear walls is connected to steel beams that are anchored to the concrete foundation to avoid uplift. This connection is shown in Figure 4.10. According to Cecobois(2018) the foundation is a 1m thick concrete slab, to avoid making a pile foundation. They claim that, if the building would be built entirely in concrete it would only reach up to six storeys before it began to be too heavy for the soil. Building everything in wood made it possible to construct Origine Condos with 13 floors instead.

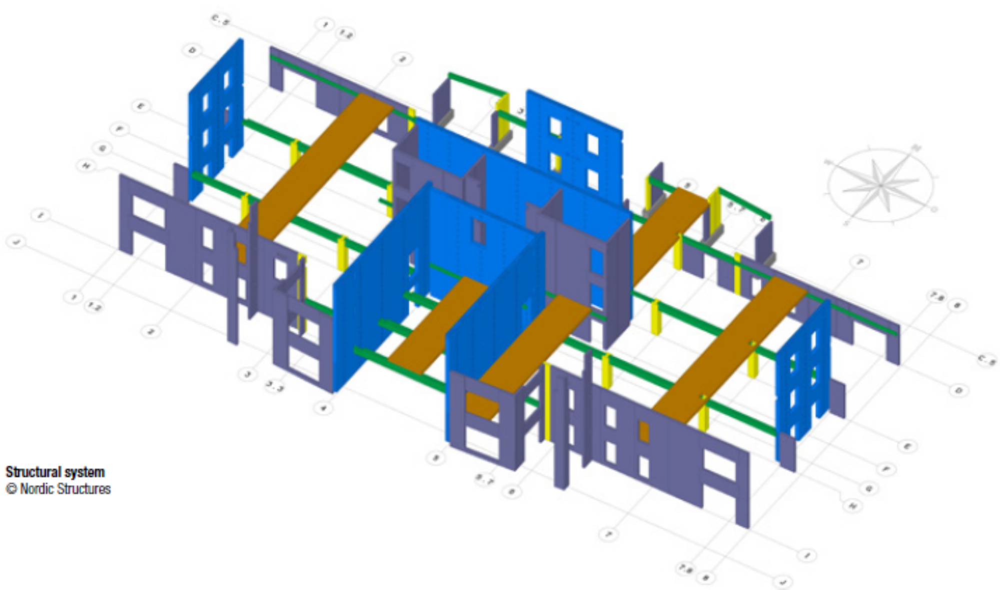

Figure 4.9: The structural system of Origine condos in different colours which represents different structural parts (Cecobois, 2018). Image © Nordic Structures / www.nordic.ca

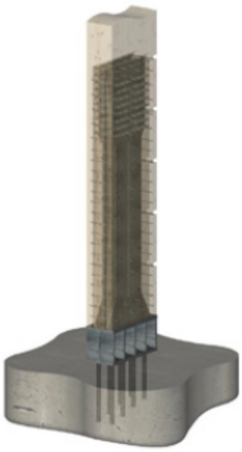

Figure 4.10: Detail of how the CLT shear walls are connected to the foundation (Cecobois, 2018). Image © Nordic Structures / www.nordic.ca

Both lateral and vertical loads are smallest in the top of the building and therefore the dimensions of the structural elements differ from floor to floor. The massive CLT shear walls are almost 2.5m wide and 9m high, which means that they cover three floors (Cecobois, 2018). Vertical load-bearing walls are a bit smaller, with dimensions 2.5m x 6m meaning that they reach two floors. The floor elements are 2.4m x 19.5m and spans over the whole building continuously over beams and walls. Beam spans varies between 4m - 7m, meaning that the beam cross-sections also varies. However, an approximated average beam cross-section is 280mm x 500mm (Cecobois, 2018).

Thicknesses of the shear walls varies in three steps. Between floor 7-12 they have the same thickness as all vertical load-bearing walls and floors. CLT thicknesses for these elements as given in the case study by Cecobois (2018) are summarised and listed below.

- Shear wall, floor 1-3: 291mm
- Shear wall, floor 4-6: 245mm
- Shear wall, floor 7-12: 175mm
- Vertical load-bearing wall: 175mm
- Floor: 175mm

Dimensions for floors and walls that are mentioned above is only the CLT thicknesses. To fulfil requirements concerning fire safety and acoustics the thicknesses increases a bit and are built up differently depending on the wall type, i.e. load-bearing, non load bearing, exterior wall or interior etc. The final largest thicknesses of walls and floors are listed below.

- Exterior wall: 379mm
- Stair/elevator shaft wall: 485mm
- Floor: 413mm

4.5 J. W. Olver Design building, Amherst, USA

The 4-storey high John W. Olver Design building was built in 2017 at the University of Massachusetts in Amherst and was the first of its kind in the US. The project started out to be designed as a steel structure, but in parallel the possibilities for a timber alternative was investigated. Constructing this building in timber were estimated to have a higher economic cost, so by the time funding was secured and the final decision to go for a wooden structures was taken, half of the design for a steel building were already made (WoodWorks, 2017). Luckily, the idea of a timber building had been taken into consideration already in early design so that the structural layout of the building had been chosen to fit either a steel or timber solution, to enable for a switch of material later on. Figure 4.11 shows pictures of the building both during construction and when finished.

Figure 4.11: Left: picture of the timber structure. Right: picture of the finished building. Photos © Alex Schreyer / UMass

The structural system is a beam-column system of glulam elements on which a timber-concrete composite floor is placed. The structural grid is approximately 7.6m x 7.6m (Schreyer and Clouston, 2019). Vertical loads are solely taken by the columns, meaning that the walls are not load-bearing. At locations where it was necessary, such as for longer or cantilevered spans, steel beams were used (Clouston, 2019). The floor consists of 175mm thick prefabricated CLT panels, with HBV® metal plate shear connectors, topped by a 100mm layer of cast in situ concrete (WoodWorks, 2017). On site, a layer of 25mm polystyrene were placed on top of the CLT panels to improve the acoustic performance, together with installations and a reinforcement grid, before the concrete were cast (Schreyer and Clouston, 2019).

The seismic loads were governing for the design of horizontal stability (WoodWorks, 2017). The horizontal stability of the building as described in the article by Schreyer and Clouston (2019) is achieved with glulam braces of the beam-column system, together with massive CLT shear cores in the shafts. The shear cores were made of continuous 7-layer CLT panels with the height corresponding to three storeys. As in all other prefabricated parts, openings where CNC cut in factory before delivered to the site. The connection to the concrete foundation were made with hold down steel brackets which were glued into the shear walls. Other connection techniques mentioned are self-tapping screws, glued-in rods and concealed beam hangers.

One of the most interesting features of the Olver Design building is the hybrid steel-timber structure referred to as *zipper truss* by (Schreyer and Clouston, 2019), which supports the roof over the interior atrium. The zipper truss is shown in Figure 4.12. It spans 16.7 meters at the largest side and consists of seven truss beams. The truss beams are built up with cylindrical glulam diagonals and steel rods connected with custom made steel connections. On the outside of the supported ceiling, an enclosed roof garden is located.

Figure 4.12: Left: The interior atrium with *zipper trusses* supporting the roof. Right: The roof garden which is supported by the trusses. Photos © Alex Schreyer / UMass

In the case study *Inspiration through Innovation*, WoodWorks (2017) gives some dimensions of the members used:

- Glulam floor beams: 360mm x 380mm
360mm x 420mm
- Glulam columns: 360mm x 570mm
360mm x 650mm
- CLT roof panel thickness: 245mm
- Glulam struts in *zipper truss*: ⌀230mm
- Steel rods in *zipper truss*: ⌀25mm - ⌀50mm

5
Requirements and important aspects

Apart from having enough resistance to handle the load effects in the ultimate limit state, the design of a well performing timber building must take several other aspects into account. This includes both structural aspects such as stabilisation and static deflections, but also requirements for acoustic performance, dynamic vibrations and fire safety. For instance, the deflections and vibrations are commonly governing for timber floor structures, hence larger dimensions are typically required than what the ULS design would give. In addition to the structural part of the timber, different types of solutions needs to be designed to fulfil the acoustic and fire requirements which might further increase the final size of the building elements. The following section will introduce some of the regulations and requirements that commonly influences the design of a timber structure.

5.1 Acoustics

Acoustics is a commonly discussed aspect within the timber building industry. Boverket claims for buildings in general that if the acoustic aspects are well planned in an early stage of the building process, a high-quality acoustic performance can be obtained at a relativity low cost (Boverket, 2008). Furthermore, they describe that lightweight structures need more considerations for this, especially when it comes to sound with low frequencies. These frequencies are important to consider because of the low density of the timber. In the formula of *mass law* according to Vigran (2008) it can be seen that a higher mass or a higher frequency gives a larger sound reduction which also gives a better sound damping. Since timber is a light-weight material and together with a low frequency will give a low sound reduction which means worse sound performance.

In Sweden, the requirements of acoustic environment are given in Boverkets byggregler (BBR). The general requirement described in BBR (BFS 2011:6) states that residential or commercial buildings shall be designed so that disturbing noise will be limited and not harm human health. Furthermore, they prescribe that noise from installations and elevators shall be damped so that people do not get disturbed. Even if these descriptions are subjective, there are maximum permissible values for noise level presented in BBR. For requirements expressed in difference in noise level, the lowest permissible value is presented.

In two swedish standards (SS), one for dwellings "*SS 25267*" and one for commercial buildings "*SS 25268*", the acoustics are requirements divided into four categories; sound class A,B,C and D, where "A" has the toughest requirements and "D" the lowest. Sound class "C" represents the minimum requirements from BBR (BFS 2011:6). The sound classes differs a bit in explanation between the standards but are in general very similar. Explanations of the sound classes are stated in the two lists below and summarises the explanations from (SS 25267:2015) and (SS 25268:2007).

SS 25267 - Dwellings

- Sound class A — Very good sound environment with few people that can be expected to be bothered in dwellings.
- Sound class B — Substantially better sound environment than required. It is suitable for dwellings where a good sound environment is desired.
- Sound class C — The sound class is the minimum requirement for dwellings and meets the regulation and general advice by the authorities.
- Sound class D — Bad sound environment and not suitable for dwellings. This sound class shall only be used if sound class C can't be fulfilled in a case where aesthetic, technical and economic aspects would be to complex or expensive.

SS 25268 - Offices, rooms for educations etc

- Sound class A — Very good sound environment and suitable for spaces and operations where high-quality sound environment is to prefer.
- Sound class B — Good sound environment and suitable for spaces and operations where a good sound environment is to prefer.
- Sound class C — The sound class meets the minimum requirements.
- Sound class D — This sound class shall only be used if sound class C can't be fulfilled in a case where aesthetic, technical and economic aspects would be to complex or expensive.

Noise can occur from different sources such as airborne sound, impact sound, installations and noise from outdoors. Requirements for these four sources and also reverberation time are presented in the two standards "*SS 25267*" and "*SS 25268*". The airborne sound requirement with indication $D_{nT,w,50}$ is a measure that describes the noise level difference between two spaces and therefore describes a building elements ability to resist airborne sound (SS 25267:2015).

Impact sound has the indication $L_{nT,w,50}$ and is noise that comes from an induced force, such as footsteps or an electric drill, which create vibrations into the structure. These vibrations pass through the partitioning structural member into other spaces where it creates a noise.

Acoustics concerning installation and elevator sound needs to fulfil two requirements. It is not allowed to exceed a certain equivalent sound level (average sound level) over a day, the value has indication $L_{A,eq,nT}$. What also needs to be fulfilled is a requirement for a maximum sound level during the day, $L_{A,Fmax,nT}$.

Noise from outdoors usually refers to traffic noise, industries, cooling systems or fans (Boverket, 2008). These noises also have two requirements to ensure that the inside environment will not be disturbed from outdoors. Equivalent sound level $L_{A,eq,24h}$ and maximum sound level during night time $L_{A,Fmax}$ are the two requirements to fulfil.

Reverberation time is another aspect to consider in spaces without furniture such as hallways and staircases (SS 25267:2015). According to BBR (BFS 2011:6) the definition of reverberation time is the time it takes for the sound level to drop with 60 dB from the moment when the sound source is switched off.

Another phenomenon that is important to consider is flanking transmission. This is when sound passes through other structural parts around the actual partitioning building element (Acoustical Surfaces, n.d.). In Figure 5.1 it can be seen how the sound passes around the actual partitioning element. To avoid this phenomenon the aim is to cut structural parts that passes through two apartments. Solution *a* in Figure 5.1 shows an illustration of how it can be solved.

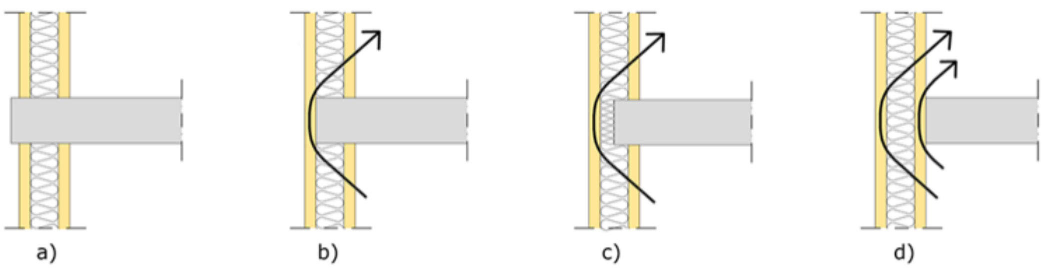

Figure 5.1: Illustration of how flanking transmission are avoided and how it passes around the partitioning wall (Swedish Wood, 2017). Image © Swedish Wood / www.swedishwood.com

The acoustic requirement also varies within a building, for instance in a bedroom the sound requirements are tougher compared to a kitchen. In Table 5.1 shows only some of the required sound levels given from BBR (BFS 2011:6) which refers to sound class C. The difference in requirement between sound classes is 4 dB which means that sound class B is 4 dB better than C and sound class A is 4 dB better than B.

Table 5.1: Summarised sound level requirements for sound level difference and impact sound level.

Requirements for sound level difference and impact sound level		
	Sound level difference $D_{nT,w,50}$ [dB]	Impact sound level $L_{nT,w,50}$ [dB] in spaces
From spaces outside dwellings to spaces inside dwellings	52	56

Table 5.2: Summarised sound level requirements for spaces for sleep, rest and daily social areas.

Sound level requirements for spaces for sleep, rest and daily social areas		
	Equivalent sound level $L_{A,eq,nT}$ [dB]	Maximum sound level $L_{A,Fmax,nT}$ [dB]
Noise from installations and elevators	25	35
Noise from outdoors, e.g. traffic	30	45

Table 5.3: Longest reverberation time allowed for residential buildings.

Requirements for reverberation time	
Spaces	Reverberation time [s]
Staircase	1.5
Hallway	1.0

5.2 Fire safety

As timber is a combustible material, the fire safety design is of great importance. The regulations of fire safety for buildings are in Europe set at the national level, leaving room for big differences between countries (SP Trätek, 2010). For instance, in Sweden timber buildings of more than two storeys where not allowed up until 1994. This was however changed when the Swedish regulations switched from a pure restriction to a performance requirement for fire safety. Regulations and general advice is given in chapter 5 of BBR (BFS 2011:6). Eurocode (SS-EN 1995-1-1:2004) gives principles and rules of application for fire design of timber buildings.

According to BBR (BFS 2011:6), a specific building should belong to one of four different building classes depending on the required safety level. The building classes ranges from class Br0, with the largest safety level, to class Br3 with the smallest. Buildings with more than 16 storeys will be categorised as Br0, and buildings with 3 storeys or more will generally be Br1. This with exception for some special purpose buildings and facilities for larger crowds. The regulation also states that the fire safety design for buildings of classes Br0 should be verified with analytical design, while buildings of other classes could be done with a simplified design.

BBR (BFS 2011:6) uses a classification system for building parts. The classification system uses combination of letters to describe different properties. The letters are combined with a number (between 15 and 360) to give information on how long time the given properties should last in a *standardised* fire. An example could be R90, EI60 or REI90. The letters for the main properties describe the following:

- R: Load bearing, for structural members.
- E: Integrity, to prevent spreading of smoke and gases to other fire cells.
- I: Insulation, to prevent high temperatures on the other side of a separating member.

Eurocode (SS-EN 1995-1-1:2004) describes methods on how to design the passive fire protection for a building. This includes design of the load bearing capacity (R) in a fire situation and the prevention of spreading between fire cells (E and I). The simplified design for the load bearing is based on the nature of how wood burns. During the fire process, charring occurs on the surface and gradually grows deeper into the timber, while the inner parts remains sound. In the design method with reduced cross-section, the charred part including an interface layer is considered to have no load bearing capacity. The cross-section is hence reduced to only consist of the remaining material. So, if the charring rate for different timber products is known, the reduced cross-section could simply be determined at the time for which the R-requirement needs to hold. The resistance of that cross-section must then be larger than the load effects in the load combination for fire.

For building parts with separation requirements, a simplified design method for the insulation performance is presented in Annex E of SS-EN 1995-1-1:2004. The basics for the design are to verify that the time for a certain temperature rise on the unexposed side of the member is larger than the time set for the I-requirement. The calculations of the insulation time for a building element consists of summing up the contribution for each individual layer of the element. This includes effects from joints and the positions of the layers. If the I-requirement is fulfilled and it is verified that the sheeting material on the unexposed side remains attached, then also the E-requirement could be considered to be fulfilled (SS-EN 1995-1-1:2004, Annex E).

Note that these simplified methods presented in Eurocode are not applicable for building class Br0, where an analytical design is required. Brandon, Just, and Östman (2018) also points out that if the fire design is made with calculations, products with documented properties must be used. As an alternative, the fire resistance could be shown by testing, for that case it is important that the same materials, products and attachment methods are used as during the test.

In the report by Brandon, Just, and Östman (2018) some examples of fire safety solutions are presented to prevent the spreading of fire. These are focused on the three main ways of spreading:

- Direct spread from compartment to compartment.
- Fire and smoke spread through cavities
- External spread along facades, through windows etc.

In timber structures, plaster boards are often used as cladding for fire protection. Brandon, Just, and Östman (2018) emphasises the importance to mount the boards in a correct manner to ensure a good protection, especially for sensitive connections between structural elements. This includes having continuous fire protective cladding along fire rated parts, that are not comprised by non-fire rated members. An example of this is shown in Figure 5.2. Furthermore, certain installations, such as vent channels, will probably penetrate through fire rated members and these openings needs extra attention. For ventilation channels it is important to prevent spreading through the channel, see example details in Figure 5.3.

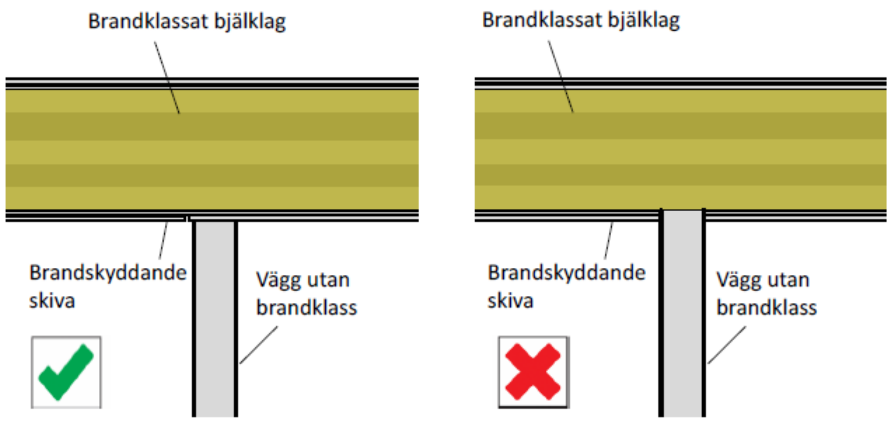

Figure 5.2: Left: Fire classed floor with uncomprised cladding on exposed side. Right: Fire classed floor with protective cladding comprised by non fire classed wall. (Brandon, Just, and Östman, 2018). Images © RISE

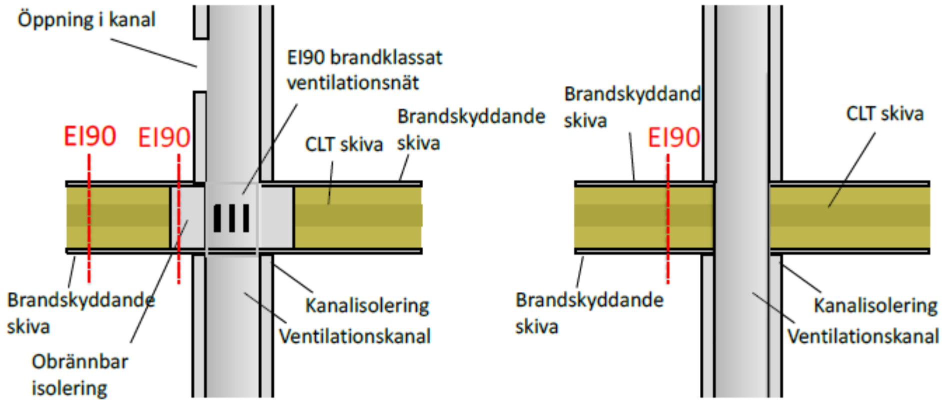

Figure 5.3: Left: Example detail of ventilation channel opening where spreading is prevented at the boarder of the fire cell. Right: Example detail of ventilation channel without opening where flames could enter. (Brandon, Just, and Östman, 2018). Images © RISE

There also exist other types of fire protective products on the market. Insulation and fire protective paint can be used to protect steel connectors in timber structures, while impregnation or surface cover of fire protective agents could be used on the timber itself (Isaksson et al., 2010). Brandon, Just, and Östman (2018) recommends to seal connections between massive timber elements with sealants tested according to EN 1364. This to avoid fire spreading through small gaps in the connections between the elements.

Some types of active fire protective measures includes sprinklers and active fire ventilation (Isaksson et al., 2010). Such methods could be used to fulfil certain requirements in BBR which otherwise would be difficult to reach. As timber structures are moisture sensitive, sprinklers could cause water damage to the structure. However, according to a case study by (Brandon, Just, Andersson, et al.) (2018) activation of sprinklers are associated with fewer cases of high water damage than intervention of the fire brigade.

5.3 Serviceability limit state - Deflection and Vibration

Serviceability limit state (SLS) represents how the structure behaves under normal use and seeks to ensure that a structure fulfils functional requirements during the whole service life without interfering with the experienced human comfort. For timber buildings, service limit state is often governing in design and two of the more important aspects are deflection and vibration.

5.3.1 Deflection

Check of deflection consists of three parts for vertical loads, these can be seen in Figure 5.4. Instantaneous deflection with indication w_{inst} is the deflection directly after an applied load plus the self-weight. This deflection is elastic and will after removal of the applied load go back to the initial deflection. The second parameter, creep deformation, w_{creep}, is a time-dependent deformation that increases with time. The third parameter is an initial deflection upwards, which can be used during design to counter-act and reduce the final deflection, it has the indication w_c. The requirements varies depending on where the deflection is located and are determined for each building project with regard to the specific business area (Al-emrani et al., 2011).

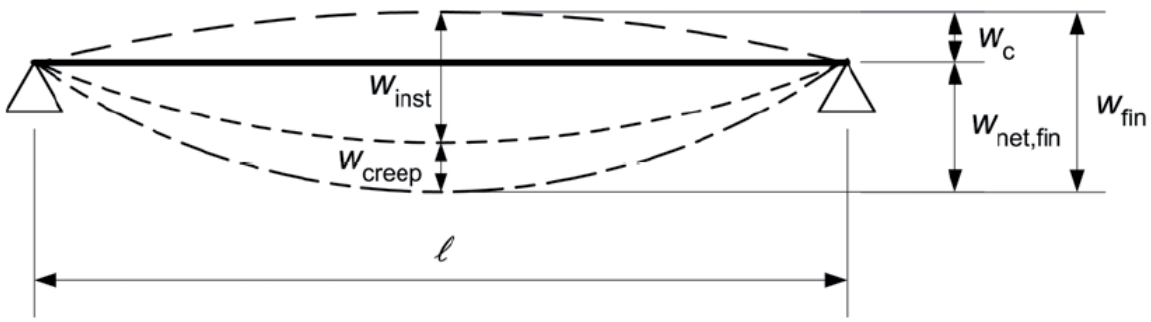

Figure 5.4: Components for calculating deflection. Image taken from SS-EN 1995-1-1:2004 with permission from Swedish Institute of Standards.

Tall timber buildings are sensitive for sway in the top of the building due to the light-weight structure. According to (SS-EN 1990) can a criteria regarding sway in the top of a building be expressed in terms of horizontal deflections. The criteria of how much a building is allowed to deflect is not stated explicitly in Eurocode (Näslund, 2015). As a recommendation of maximum horizontal deflection, the limit $H/500$ have been used by engineers in the industry, where H is the height of the building (Näslund, 2015; Ramage et al., 2017).

5.3.2 Vibration

The other important criteria, vibration, arise from dynamic forces such as movements from humans, rotating machinery or traffic. According to Al-emrani et al. (2011) vibrations can be divided into two categories, vibrations that disturb someone other than the one causing them and springiness where the deflections can be felt by a person that walks on the floor. *Dynamic forces*, *dynamic properties* of the floor structure and the *human tolerance* of vibrations in different situations are three areas of which there must be knowledge to be able to design for vibration and springiness (Al-emrani et al., 2011). Calculations of vibration is a complex area but a simplification for limiting the springiness of a floor structure is to study one joist and say that the deflection of this might not exceed 1.5mm for a short term point load of 1.0 kN (Al-emrani et al., 2011).

Sway acceleration of a building is relevant to keep low as it can cause nausea (Edskär, 2018). In the technical report by Näslund (2015) it is described that there are no guidelines or recommendations regarding acceleration and comfort criteria in Eurocode. However, recommendations for criteria of acceleration are stated in the three standards below.

- SS-ISO 10137:2008
- ISO 6897
- SS 460 48 61

The three standards are used for different frequency ranges, where *ISO 6897* is used for low frequencies between 0.063 - 1 Hz and *SS 460 48 61* can be used for frequencies between 1 - 80 Hz. Standard *SS-ISO 10137:2008* covers the frequency range between 0.06 - 5 Hz, and a diagram from that standard is shown in Figure 5.5. Edskär (2018) claims that it is common to reach natural frequencies at 1 Hz or below for concrete and steel buildings but for timber and hybrid buildings the typical natural frequency range is between 1 - 4 Hz. Therefore, it is important to choose a standard that is relevant for the specific building. Calculation of natural frequencies can be made with hand calculations according to Eurocode (SS-EN 1991-1-4). But the frequencies can also be numerically calculated using a FEM-model. The sway acceleration can be calculated by hand according to the Swedish construction regulation *Boverkets Konstruktionsregler* EKS 11 (2019). Estimations have been done by researchers of how humans perceive motions of a building. In a study by Mendis et al. (2007) humans response to different accelertion were found, these are presented in Table 5.4.

Table 5.4: Humans response to different acceleration (Mendis et al., 2007).

Acceleration [m/sec^2]	Effect
< 0.05	Humans cannot perceive motion
0.05 - 0.1	Sensitive people can perceive motion and hanging objects may move slightly
0.1 - 0.25	Most people will perceive motion
0.25 - 0.4	Desk work becomes difficult
0.4 - 0.5	People strongly perceive motion
0.5 - 0.6	Most people cannot tolerate motion, experience difficult to walk naturally and standing people may lose balance
0.6 - 0.7	People cannot walk
> 0.85	Objects begin to fall and people may be injured

An increased mass of the top floors lowers the natural frequency and the acceleration (Edskär, 2018) and this was applied to Mjöstornet, decribed in Section 4.1. Furthermore, Edskär (2018) means that this is not the only way to solve acceleration issues since it is depending on the building's natural frequency. Figure 5.5 shows recommendations for acceleration vs fundamental frequency to avoid vibration problems. The aim is to be under the two curves. Curve 1 is for office buildings with a slightly higher permissible vibration criteria and curve 2 is for residential buildings with a tougher criterion. Figure 5.5 shows that if the fundamental frequency of the building is between 0 - 1 Hz an increase in mass would be helpful, as it reduces both the eigenfrequency and the acceleration. For natural frequencies larger than 2 Hz it could be more efficient to increase the stiffness since that increases the fundamental frequency (Edskär, 2018).

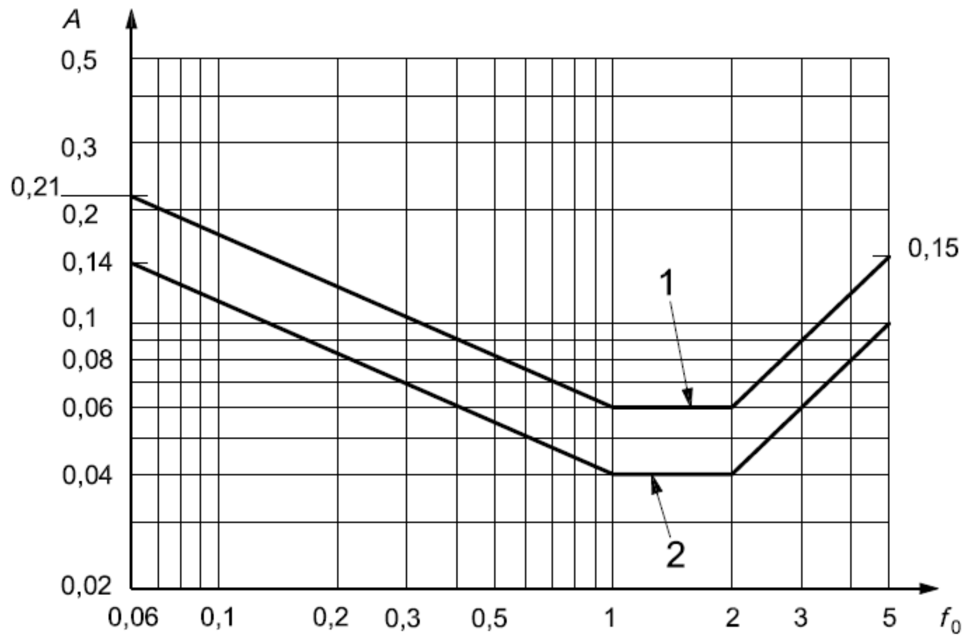

Figure 5.5: Acceleration vs first natural frequency. Image taken from SS-ISO 10137:2008 with permission from Swedish Institute of Standards

Two types of damping are described by Edskär (2018); passive and active damping. Passive damping systems relates to the chosen building layout and structural materials. Therefore, reducing the acceleration by increasing the mass is a passive damping system for instance. An active damping system could for instance be some sort of mechanical way to reduce the acceleration. Furthermore, Edskär (2018) mentions that it is hard to predict a damping ratio for a building. However, some damping ratio intervals are given for three structural systems and are presented in the list below.

- Post and beam structure : 1.9 %
- CLT structure : 2 - 2.5 %
- Hybrid structure : 2 - 3 %

5.4 Stability

As a typical structural system in timber is made with individual elements that are assembled, the horizontal stabilisation of the system is of great importance and needs to be carefully designed. Porteous and Kermani (2009) lists lateral bracing members, horizontal diaphragms (roof and floors) and vertical diaphragms (shear walls) as the most commonly used techniques for stabilisation of timber buildings. Furthermore, they also mention the main effects to which the stabilisation elements needs to be designed for according to Eurocode 5. For bracing members this includes axial stresses and deflections. Diaphragm elements needs to maintain static equilibrium in terms of sliding and uplift, as well as racking resistance for walls, and bending and shear stresses for floors/roofs.

The diaphragm action of floors and roofs are used to distribute horizontal loads to bracing units or shear walls which then carries the load vertically down to the foundation. In order to achieve this, the connections between elements needs to be well designed. The basic principle is that shear is transferred in the longitudinal joints between two floor elements and between the floor and shear walls. Edge beams working in tension or compression are used to tie the edges of the floors together. Swedish Wood (2015) shows an analogy between the whole floor diaphragm and the function of a large I girder. The floor plates are compared with the web, mainly working in shear while the edge beams are representing the flanges taking bending in compression (top flange) and tension (bottom flange).

Stora Enso (2018a) gives some guidelines for the applications of different stability systems. CLT/LVL shear walls are said to be useful for buildings up to 5 storeys, while timber bracing can be used up to 9 storeys. For even taller buildings, concrete cores could be used. However, it is clearly stated that those limits only give a hint on the capacity for different systems and that combinations of systems and good designing could of course increase the possible heights. For instance the project Mjøstårnet, described in Section 4.1, uses a large sized glulam truss system to reach as high as 18 storeys. Other projects described in this report, Brock Commons (Section 4.2) and HoHo Vienna (Section 4.3) uses instead a concrete core to stabilise 18 and 24 storeys respectively.

5.5 Robustness

In Eurocode (SS-EN 1991-1-7) robustness of a structure is defined as its ability to avoid disproportionate damage due to unforeseen events such as explosion, impact or consequences of human error. The research and practice on how to design robust steel or concrete structures have been well developed over the years. However, for timber structures the knowledge and guidelines are still limited, especially for CLT and post-beam timber systems (Huber et al., 2019).
On a general level, a deterministic design for the robustness of a structure could take two different approaches; scenario dependent or scenario independent (Huber et al., 2019). The first seeks to design robustness for a known specific exposure while the latter disregards the cause of the damage and focuses on limiting the spread of the damage for an unidentified exposure. There are several strategies for this where Eurocode (SS-EN 1991-1-7) recommends different strategies depending on the consequence class (CC) of the structure. For buildings in the lowest category, CC1, no extra considerations need to be taken for an unidentified accidental action, while for a building in the highest, CC3, a systematic risk assessment is required. CC2 is divided into group A (lower) and B (upper). Buildings that are classed in CC2a are recommended to be designed with a horizontal tying or anchored connections between floors and walls.

For buildings in group 2B, there are two alternatives that are recommended. Either could a horizontal tying system in floors and beams be used together with vertical ties in load bearing columns and walls. The second option is to perform a check that the local damage is kept below a limit when notionally removing supporting columns, beams and wall sections one at the time. This method is called *alternative load path design* (J Huber et al., 2018). The difference between the two strategies is that the tying system is designed to sustain a standard tensile load given by Eurocode, while the alternative load path design method seeks to verify that damage only spreads within the specific building up to a certain limit (J Huber et al., 2018). If removal of some elements would cause unacceptably large damage, the latter strategy could be complemented by designing "key elements". According to Swedish national annex EKS 11 (Boverket, 2019), "key elements" should be designed such that floors and walls withstands an accidental action of $34kN/m^2$. Beams and columns should be designed to resist at least 1.3 times of the design load effects. Eurocode also hints that this method is probable to be the most practical for structural systems with load-bearing walls.

The strategies described in Eurocode are general and not specifically adapted for timber structures. The major differences between timber and for instance steel is that timber have a brittle fracture in bending and tension while steel is ductile. This implies for a need of designing ductile connections for a timber building (Huber et al., 2019). However, J Huber et al. (2018) points out that some similarities can be seen when looking at the robustness for a steel frame compared to a post-beam timber system. The frame systems are said to often use the strategy of providing ties to fulfil the generic requirements, but for the timber structure additional checks for the ductility in connections should be done (Huber et al., 2019). For the stability of such a system, they suggest that the bracing should for instance be designed with moment resisting joints, shear walls or rigid cores to create redundancy.

Design with the standard tensile load strategy for CLT-systems is not practical or economic (Mpidi Bita and Tannert, 2019). Instead alternative load path design have mostly been used so far where similarities can also be seen between a precast concrete and a CLT-panel system (J Huber et al., 2018). This in the way that walls can behave as deep beams and floors as membranes. However, a new connection consisting of steel tubes connected with steel rods, have successfully been tested by (Mpidi Bita and Tannert, 2019) to see if CLT floors could develop catenary action, but further research is still needed on the subject.

5.6 Connections

Connecting timber elements to each other is easy when it comes to smaller dimensions, while for larger dimensions it becomes more complicated. As a comparison with steel elements, the same approach can be applied for both larger and smaller elements; they can still be welded or bolted. Connections between larger dimensions for timber can be very complex and also has a large impact of the overall economy of the building system, especially as they often is very time consuming to produce (Swedish Wood, 2015). For complex and highly loaded connections, timber elements are often connected with steel connectors, as for the example shown in Figure 5.6. Another important aspect is that wood and concrete are not allowed to have direct contact to each other for if the concrete is exposed for moisture which could be in, for instance, a foundation. This as wood can get rot damage due to the moisture coming from the concrete. In Figure 5.6 the steel connector also makes sure that there is no contact between the wood and concrete.

Figure 5.6: Timber-steel connection (Swedish Wood, 2016). Photo © Sören Håkanlind

Swedish wood (2015) lists three types of joints; traditional timber joint, dowelled joint and glued joint. A traditional timber joint uses a stronger wooden material such as oak to connect two timber parts. According to Swedish Wood (2015) traditional joints were more common in the past, while today the dowelled joints are the most frequently used. Furthermore, they describe that glued joints are very useful when prefabricating for instance glulam beams, while gluing joints at the construction site is not to prefer as it requires a controlled environment during production. It is also described that dowelled joints include many dowel-types such as dowels, nails, screws, nail plates and bolts.

In Figure 5.7 two different dowel joints are shown. The left one is more common for larger structures and needs to be prepared in the factory, while the one to the right is more suitable for smaller timber elements. The right dowel connection with a so-called nail plate can be performed both at the construction site and in the factory.

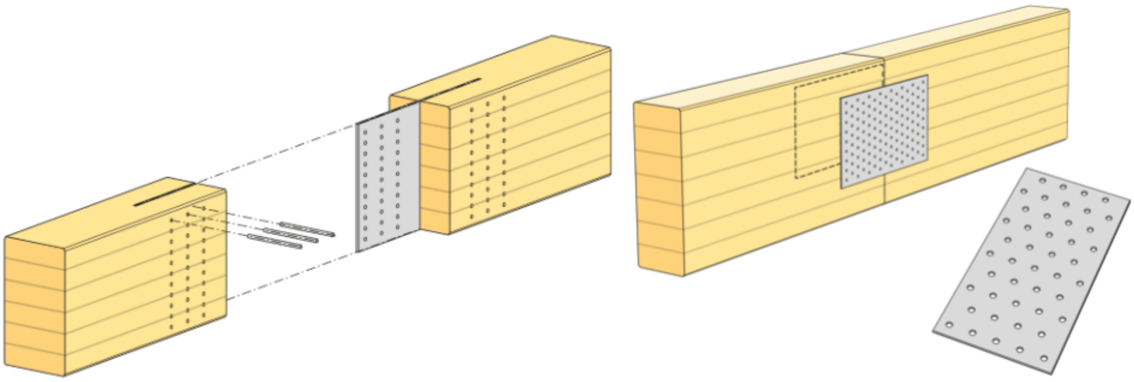

Figure 5.7: Left: Dowel connection with built-in steel plate. Right: Other type of dowel connection with visible a nail plate.
Images © Setra / www.setragroup.com/sv/limtra/konstruktionslosningar

Connections are necessary to keep the structural system together on a global level and should be well designed to achieve the desired behaviour in the connection. However, from different perspectives, such as fire safety, stability and acoustics, different connection designs are preferred to optimize their individual requirements and challenges. For instance, from an acoustic perspective as loose connections as possible would be favourable to avoid structure-borne sound and flanking transmission. This while from the stability point of view the elements should be stiffly connected to give an overall stable and stiff structure. Connections for a robustness or seismic design should preferably be ductile to absorb energy and avoid progressive collapse.

6

Case studies

This chapter describes the process of developing two alternative structural systems with timber as structural material. The systems proposed are fictional, but they are developed for real projects that previously have been structurally designed by Stomkon. One is a concrete residential building and the other is a renovation of an office building with addition of an extra storey in steel and concrete. The alternative concepts are developed to give a practical application and example of how timber can be used structurally in buildings.

Important to notice is that the following chapter only includes a preliminary design of the main structural members. This design is made to fulfil the most important criteria in ULS and SLS according to Eurocode. However, for a real case and in order to make the whole system fulfil functional requirements such as acoustics, a more detailed design including connections etc. is needed. This was set as a limitation for this thesis and no calculations were made for this, but general examples and requirements are found in the previous chapters.

6.1 Traneberg, Stockholm

The first project that is studied is of two similar buildings located in Stockholm. They are both 7 storey residential buildings with a total height of 21 meters. The footprint of each building is 12 times 23.5 meters. Six of the floors in each building have a similar layout with two large apartments of 5 rooms each plus kitchen. The bottom floor has a slightly differing floor plan layout in order to house some of the shared spaces such as entrance, bicycle storage etc.

However, in this study a simplification is made where all floor plans are modelled with the same layout. This except for openings in the walls which are modelled to match each floor in order to give as realistic conditions as possible for the stabilisation system. Also, as the two buildings are so similar, in this project only one of them is modelled.

6.1.1 Original design

This building was made as a traditional concrete residential building. The structural system consists of prefabricated massive concrete load bearing facades together with partly prefabricated inner walls and floors. In order to keep the flexibility in the apartments it is only the walls around the elevator shaft and staircase, together with one wall in each bathroom that is load bearing. The floors will act as one unit over each storey and distribute both horizontal and vertical loads to the walls that will transfer the loads down to the foundation.

6.1.2 Decision process

In the decision process several aspects have been considered. The starting point were to minimise the use of concrete and use timber instead, without compromising the function of the building. One of the main considerations were to find a concept that kept the total height of the building constant, i.e. keep the thickness of each floor constant. This is indeed one of the main challenges for a residential timber building, as the lightweight floor is likely to need some additional acoustic work. With this in mind, reducing the span lengths of the floors were a key to reduce the required height of the structural parts. Another aspect for the function of the building is to keep the flexibility of the floor plan, with as few vertical load bearing members as possible inside the apartments. Easy and fast production were also considered to be important to reduce the production cost and make the proposed solution realistic.

One idea to maintain the total height of the building is to make the roof flat instead of duo pitched. In this way an additional ≈ 1.5 meters would be available, meaning that the total thickness of each floor slab could increase by ≈ 0.2 meters without reducing the free height of the apartments. This would however come at the cost of extending the elevator up in a small extension that will be above the roof. How this could be performed is illustrated in the short-side section sketch in Figure 6.2.

As the structural system for a timber building get very lightweight compared to a concrete building, the global stability against tilting is important to verify. When this was done it was discovered that using only timber in the whole structural system would not give enough moment resistance against the applied wind loads. These calculations are show in the end of Appendix C. This calls for a solution where some self-weight is added for the structural systems. For this, four different options were considered that all of them would solve this:

1. Put the extra weight in the foundation and make sure to anchor the CLT-walls all the way down to it. This would need approximately a 45 cm thick concrete slab.
2. Make the first floor (including walls) with concrete and anchor the timber structure on top of it.
3. Make the elevator shaft in concrete.
4. Use TCC-floors to give extra weight at each floor level.

All of these alternatives have pros and cons which are further discussed in Chapter 8. However, option 2 were chosen, so it is proposed that the bottom floor (including walls) of the building is made in concrete and then the six following timber storeys is anchored down to the concrete structure. Performing the bottom floor in concrete could be beneficial and make it easier to fulfil further requirements from other points of view that are not included in this report: such as accidental action (car crash) and durability (snow against facade). This type of solution could also be relevant for a building with a basement which could be done in concrete with a timber structure added on top of it.

A rough comparison of the total self-weight between the concrete building and the proposed timber concept were done and is presented in Appendix A. It shows that the timber concept, with the whole building in timber would only have a weight of 11% compared to the full building in concrete. With the first storey in concrete and the rest in timber it would be approximately 25%. These calculations are only including the structural self-weight and does not include the weight of the foundation.

6.1.3 Proposed timber concept

The external walls of the proposed concept will be of CLT. Production-wise the CLT offers a very attractive solution where the wall elements are produced with the height of one story and lengths of up to 12 meters. This means that for this building, only six elements are required for the facades of each story. Openings will be CNC cut in the factory so that the elements arrive ready to be mounted with as less site work as possible. The inner walls will also be of CLT, with additional cladding to fulfil the EI60 fire-requirements for the partitioning walls.

To reduce the span length of the floors, two glulam beams are put into the construction for each apartment, supported on the walls. This gives slightly less than 4.5 meters in the longest floor span. The floors are of a 5 layer CLT and will be simply supported on the long side facade walls and the glulam beams. From the CLT plate a suspended ceiling is needed to fit extra acoustic works, and in this space also the installations will be placed. The suspended ceiling will also cover the glulam beams, so that those will not influence the architectural function of the apartments. Hence, the only added structural member inside the apartments which reduces the flexibility is the added CLT wall. However, this is the wall separating the bathroom and kitchen, so the probability that someone would like to remove that wall in the future is considered as very low. Figure 6.1 shows a plan view of floor two, including preliminary dimensions for the structural members.

The balconies are made of the same type of CLT as the floors. They are supported on the outside by a HEA steel beam which is held by a cable-stay connected to the floor above to minimize the local load effects on the wall. On the inside the balconies are hingely connected to the wall. The idea is illustrated in the long-side section view in Figure 6.3.

For the stability of a building of this height, CLT is a good choice as the in-plane stiffness of the plates will provide lateral stability without the need for extra bracing units. This is one of the main reasons that CLT were chosen in both walls and floors. As mentioned in the previous section, to ensure the global stability against tilting the bottom floor needs to be done with concrete to which the timber structure is connected. The specific design of the concrete structure is out of the scope for this thesis.

From a structural perspective a balloon type structure would be preferable as this avoids compression perpendicular to fibres in the CLT. However, this might be troublesome from a flanking transmission perspective. This calls for extra attention in the design of the connection. Also, the specific fire design is something that needs to be handled in the detailed design. The exact build-up of the external walls and roof including the need for thermal insulation is also things to be included in the detailed design. Other important connections that needs to be designed locally are the connections between the glulam beams and walls. At two places the beam should be connected to the wall where there is a balcony door, so here a lintel will be needed.

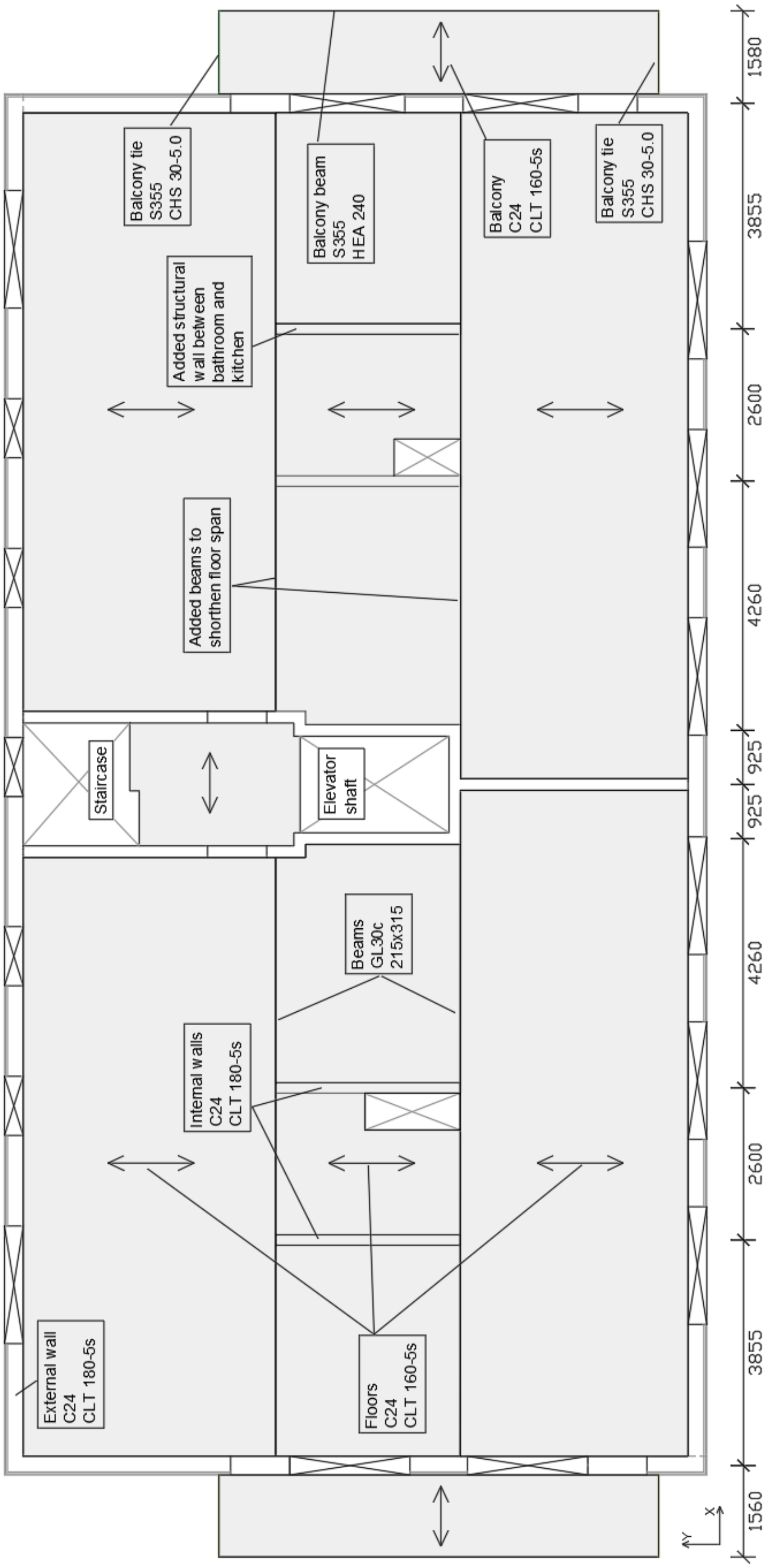

Figure 6.1: Plan view of the structural system in Traneberg building, including dimensions for the main structural members.

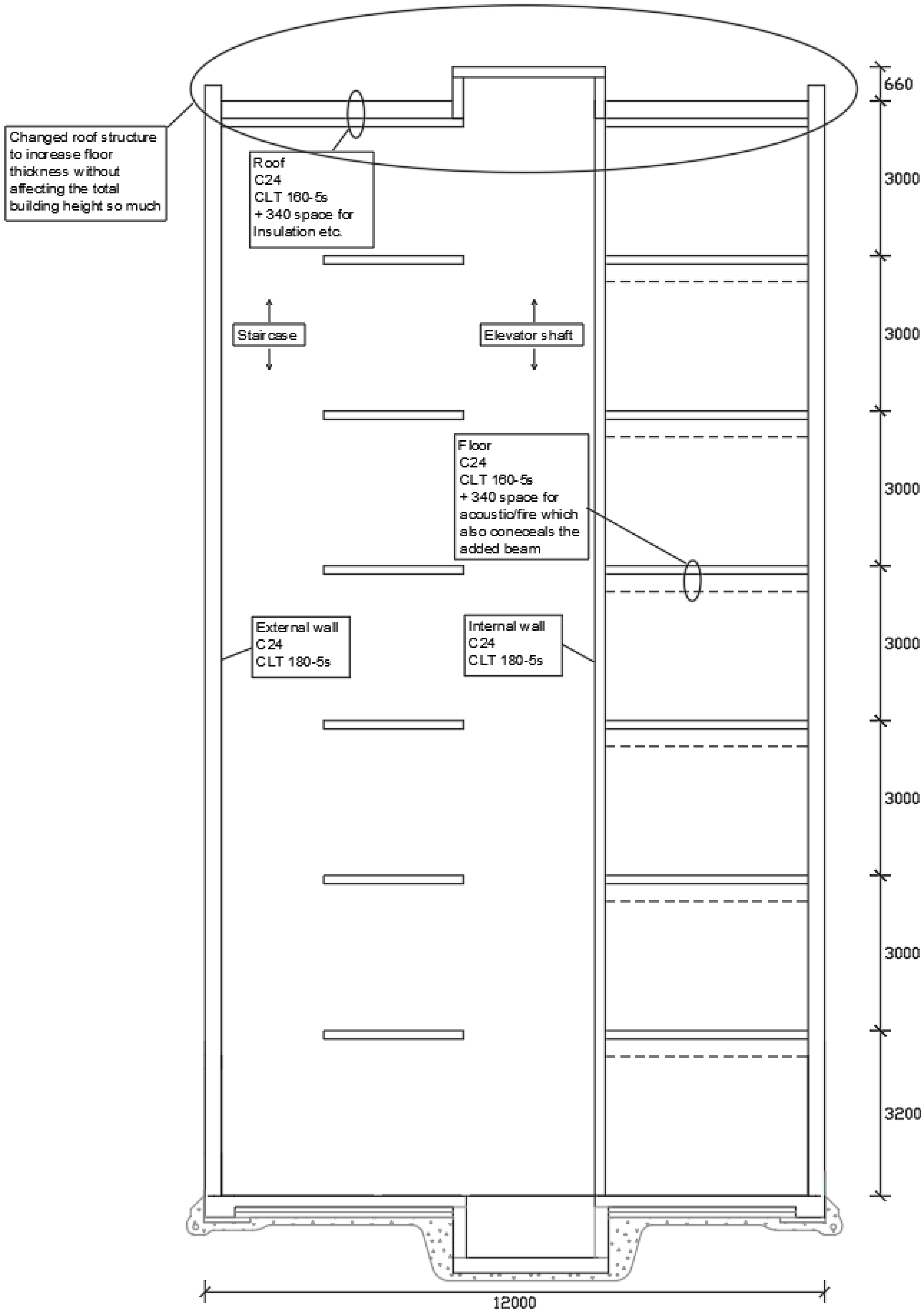

Figure 6.2: Short side section through Traneberg building.

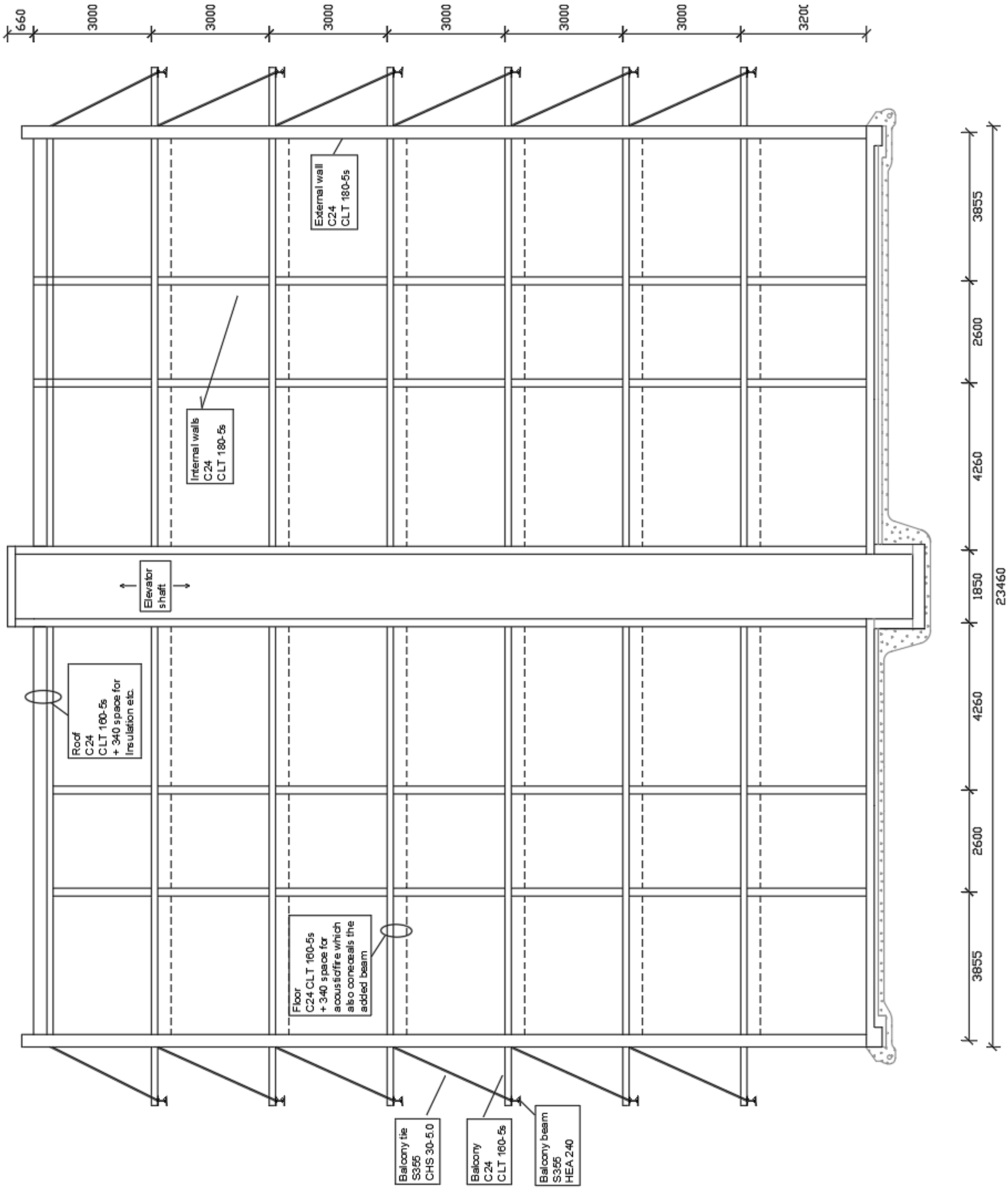

Figure 6.3: Long side section through Traneberg building.

6.1.4 FEM analysis of concept

An analysis of the structural system was made in the 3D structure module of Strusofts FEM-design software together with hand calculations for the main load bearing timber elements. This in order to verify that the proposed solution is possible to construct following the building regulations. Figure 6.4 shows the 3D model created in FEM-design. This model was made with the whole building in timber. When the analysis was done it was concluded that it was too lightweight to resist tilting. However, the individual parts studied in this section are not affected of how the first floor is constructed. So even though the model has CLT in the bottom floor, what is presented in this section is still valid and no new analysis were needed.

The utilisation ratios for the CLT panels are shown as a colour plot in Figure 6.5. This shows that the proposed sizes of the elements fulfil the ULS requirements. Full design calculations for different elements from FEM is attached in Appendix D. The deflections are later presented in a comparison between FEM-results and hand calculations. Further hand calculations including verification of SLS requirements such as springiness and vibrations of floor are shown in Appendix C.

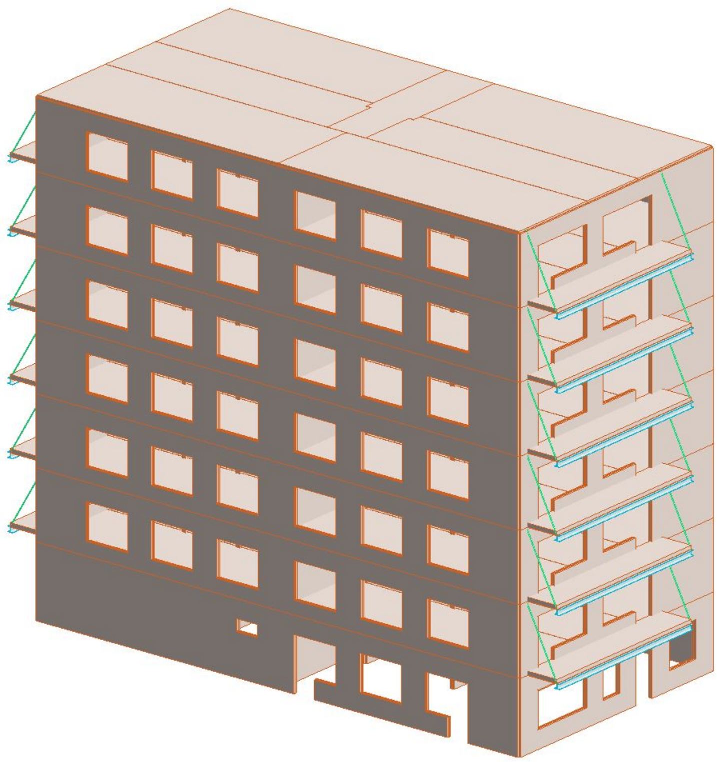

Figure 6.4: 3D view of the FEM model.

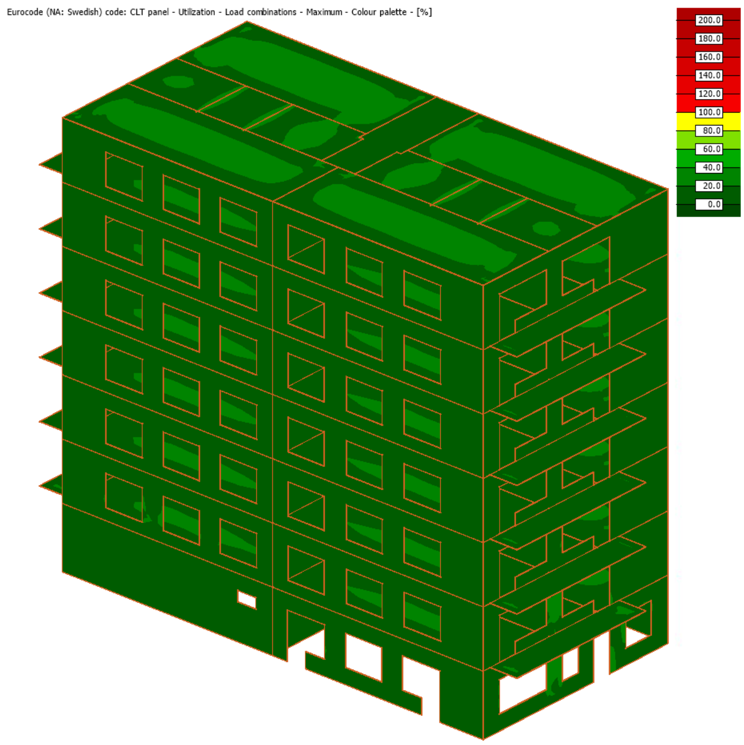

Figure 6.5: Utilisation ratios for the CLT panels in Traneberg.

6.1.4.1 Loads and load combinations

Loads that have been used in the model is the dead load of the building, two different imposed loads, one for the floors and one for the balconies. Snow load and wind loads have also been considered in the analysis. To represent dead weights from e.g. installations, insulation and other non-structural weights that are permanent the "extra dead weight" is added on the floors and roof. The characteristic loads that have been used are listed in the Table 6.1. Multiple load combinations were used to find the design load effects in different parts of the structure. All the load combinations used in the FEM-model can be seen in Appendix B.

Table 6.1: Vertical and horizontal characteristic loads on Traneberg residential building.

Characteristic loads		
Type of load	**Load** $[kN/m^2]$	**Standard**
Structural dead weight	Automatic in software	-
Extra dead weight (e.g. Installations)	0.5	Own assumption
Imposed load floors	2	Boverket EKS11
Imposed load balconies	3.5	Boverket EKS11
Snow load	1.6	EC SS-EN 1991-1-3
Wind load $[C_e(Z) = 2.2]$	0.8	EC SS-EN 1991-1-4

The characteristic wind load is multiplied by pressure coefficients depending on which direction the wind is acting. Used coefficients for the building are shown in Table 6.2 and comes from Eurocode (SS-EN 1991-1-4). Zone D is the coefficient used on the windward side and zone E is suction on the leeward side. Wind loads on the facades perpendicular to the wind directions are neglected as these would cancel out each other for stability calculations.

Table 6.2: Wind pressure coefficient.

Wind pressure coefficient $[C_{pe10}]$		
Load Direction	**D** (Pressure)	**E** (Suction)
Wind load X+	0.8	0.5
Wind load X-	0.8	0.5
Wind load Y+	0.8	0.55
Wind load Y-	0.8	0.55

6.1.4.2 Mesh and modelling

The FEM-model is built up with different FEM-elements which have different cross-sections, materials, boundary conditions, analytical axes and meshes. These elements and their properties are shown in Table 6.3.

Table 6.3: Presentation of how different elements were modelled in the FEM-design software.

FEM modelling of elements						
Part	**Material**	**Section [mm]**	**Element type**	**BC's**	**Analytical axis**	**Mesh [m]**
Floor CLT	C24/C14	160-5s	2D Shell	Hinged	Center	0.4x0.4
Roof CLT	C24/C14	160-5s	2D Shell	Hinged	Center	0.4x0.4
Balcony CLT	C24/C14	160-5s	2D Shell	Hinged	Center	0.4x0.4
Beams	GL30c	215x315	1D Beam	Continuous over support, Hinged at edges	Centric	2 parts
Diagonal bar	S275	CHS 30-5.0	1D Bar	Hinged	Centric	2 parts
Walls	C24/C14	180-5s	2D Shell	Hinged	Centric	0.4x0.4

An overview of the mesh for the FEM-model is used as a verification that the model is done without any larger errors. If it takes very long time to generate a mesh, usually there could be a modelling error. When the mesh is created a quick look at it can give a feeling if the model seems fine. If parts of the mesh look different to the rest of the model, there might be a modelling error. In Figure 6.6 the mesh of Traneberg residential building can be seen. The mesh looks good since it has an even mesh and around door/window corners the mesh changes its shape a bit, which is expected.

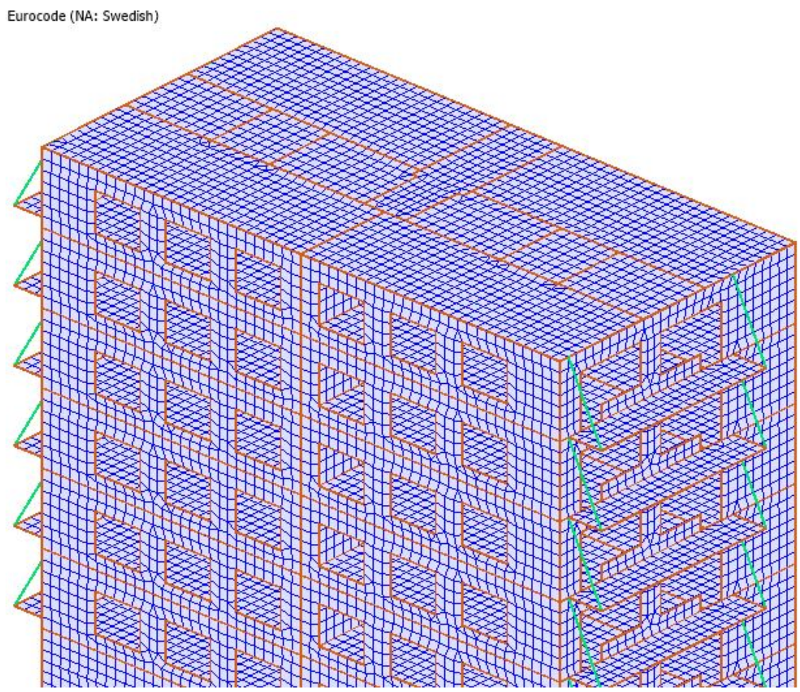

Figure 6.6: Mesh of Traneberg residential building

6.1.4.3 Verification

To verify the model several different small studies are done, such as check of deflection- moment- and shear behaviour of the structure. Figure 6.7 shows where the floors are supported and therefore where it can be expected with very small deflection. The beams are continuous over the two walls and therefore a negative support moment is to expect here. The moment around all plate edges can be expected to be zero since they are hinged. Over the two walls in the middle of each apartment only a small negative moment should occur since the primary load direction of the plate is in the same direction as the walls.

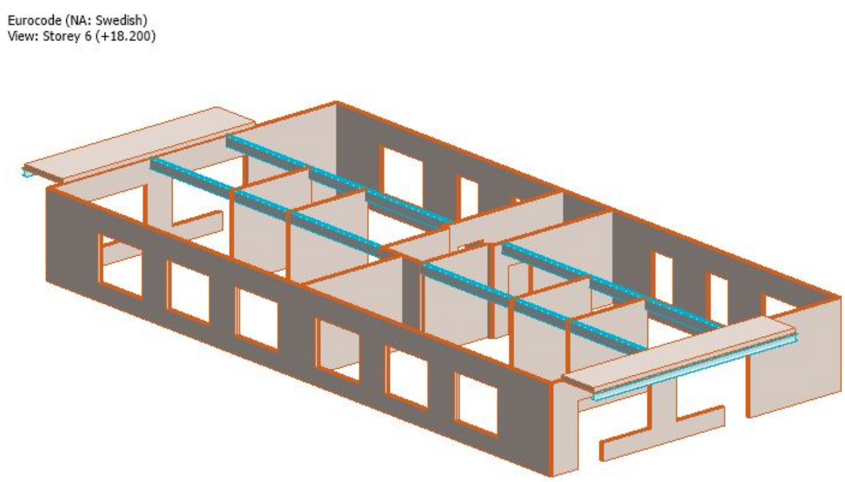

Figure 6.7: Walls and beams that supports the floor plates.

The first verification is a check of the translational displacements of the floor to see if the model behaves as desired. In Figure 6.8 the deflection of storey 6 in Traneberg building is shown under quasi-permanent load combination. It can be seen that the balconies has its largest deflections in the middle were both the *HEA 240* beam and the balcony plate are deflecting most. The *HEA 240* is hanging in two diagonal bars which act as supports, hence the beam is seen as simply supported. This only gives a small displacement in the edges which comes from the extension of the diagonal bars.

The floor consists of six individual plates where each one is simply supported around its edges. There are four continuous beams in total, two in each apartment. When the beams are deflecting the plate follows the beam but still it can be obtained that the plates have sharp edges towards each other. This means that they are hinged to each other and the simply supported behaviour is obtained. It is also expected that there are smaller deformations where the plates are supported by walls. This can be obtained around the building and the two squares in the middle of each apartment, where the displacements are close to zero.

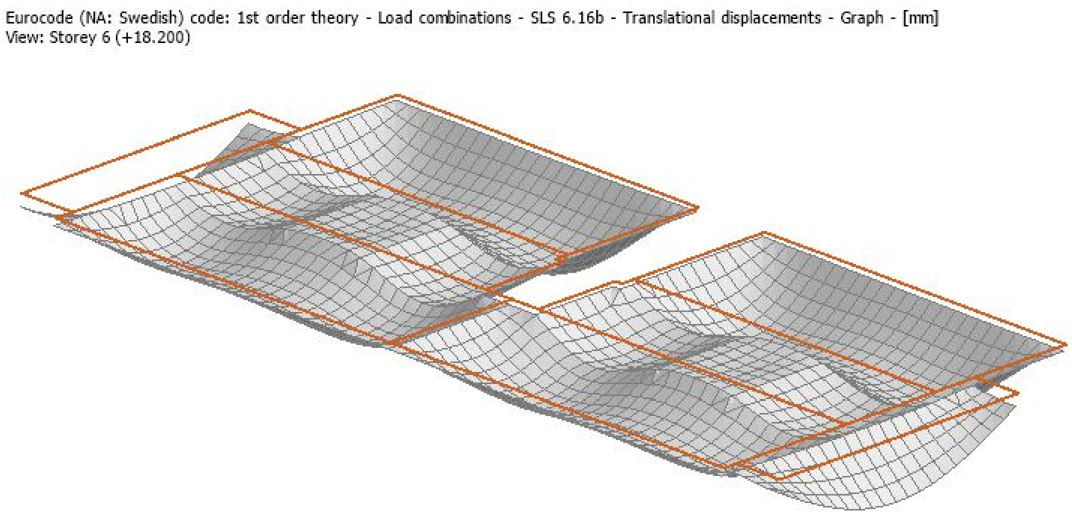

Figure 6.8: Deflection images (scale factor 200).

Moment verification is made on one of the most loaded beams. Figure 6.9 shows that the moment is as expected; zero moment in the edges and negative support moment over the two supports where the beam is continuous. The shear force diagram of the same beam is also looking as expected, with a linear variation and jumps corresponding to the reaction forces at the supports. Figure 6.10 shows the shear force along the beam and it can be obtained that over the support there is a bit larger shear force than at the edge. This comes from the continuity of the beam which attracts more load over the continuous supports.

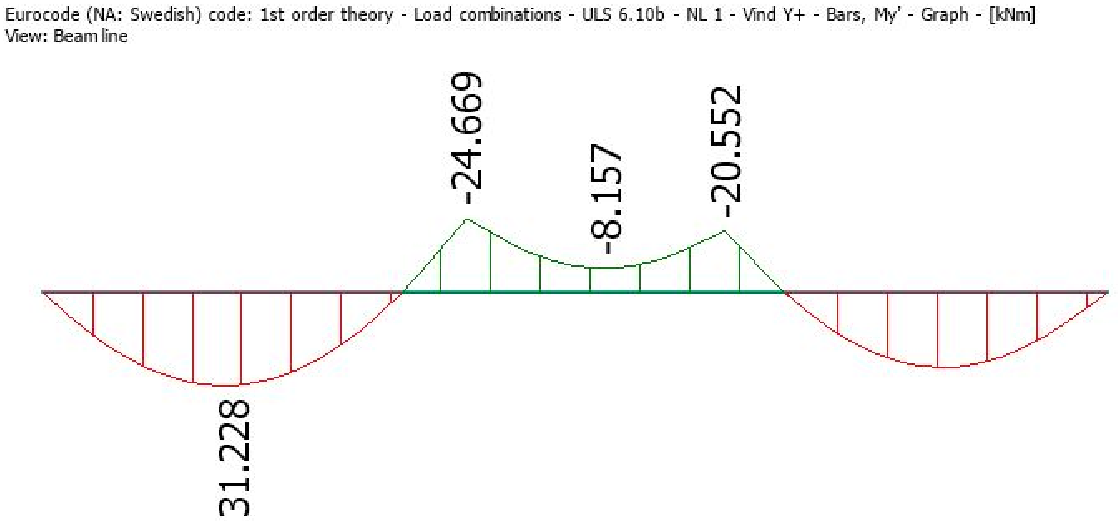

Figure 6.9: Diagram of the moment along one of the beams in Traneberg residential building.

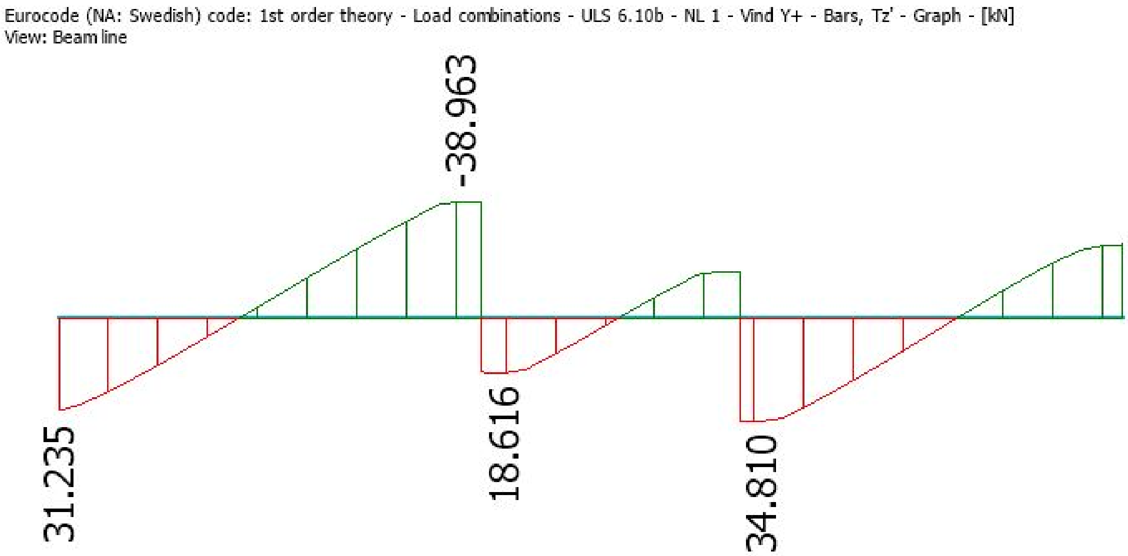

Figure 6.10: Diagram of the shear force along one of the beams in Traneberg residential building.

For the plate verification, a check of the normal stress distribution parallel to the grain in the bottom layer of the CLT plate has been made. This can be seen in Figure 6.11. The stresses are almost zero over the supports and around the edges, which was one of the expectations since the end connections are hinged. The largest tensile stresses occur in the middle of the span which also were to expect. The deflection has also been checked and will be compared to hand calculations.

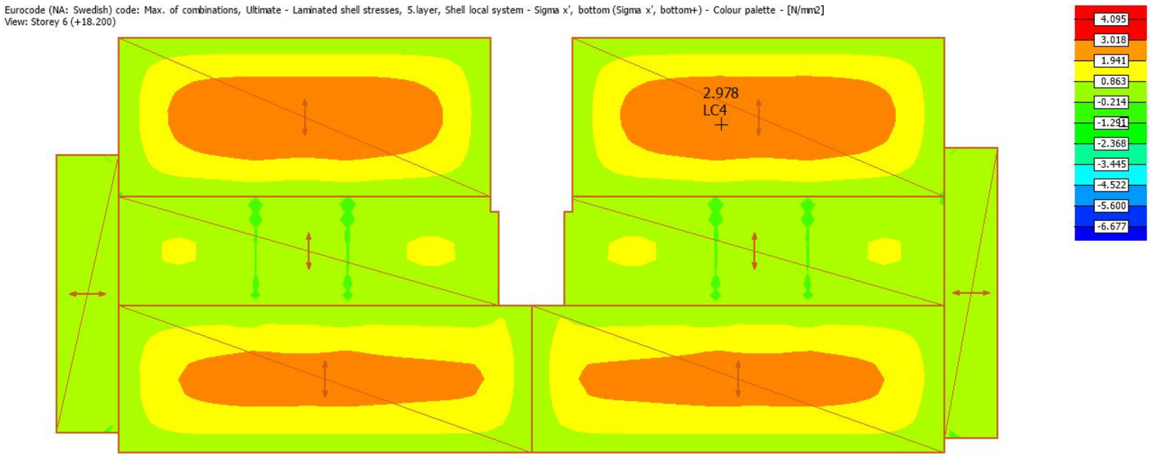

Figure 6.11: Colour plot of the stress distribution in the bottom of the CLT.

6.1.5 Hand calculations and Comparison

Calculations made by hand are done as a confirmation of that the FEM-model shows reasonable results that match the intended behaviour of the structure. In such way it can be seen if the modelling is made correctly. It also gives a better understanding of the FEM-model because when the hand- and FEM-calculation differ from each other it is important to understand why. Therefore, it is important to know what assumptions that are made to find a reasonable explanation to the difference. Hand calculations are made for one floor and one beam.

6.1.5.1 Floor

Hand calculations are made in Excel for the same floor element that was verified in the FEM-model, see calculation sheet in Appendix C. Bending stresses and deflections are the two checks that have been calculated and will be compared to the FEM-calculation. This is made with the assumption of neglecting the layers in the CLT that are perpendicular to the main load bearing direction. Therefore, higher stresses and deflections can be expected compared to the FEM-calculation. In Table 6.4 it can be seen that this is indeed the case when this comparison is done. Neglecting two layers in the CLT panel gives for instance a smaller moment of inertia which leads to both larger stresses and deflections. The results are therefore reasonable.

Table 6.4: Floor comparison

Floor comparison				
	FEM-design	**Hand calculation**	**Diff**	**Diff [%]**
Bending stresses	2.98 MPa	3.2 MPa	0.22 MPa	6.9%
Deflection	7.6 mm	9.5 mm	1.9 mm	20%

6.1.5.2 Beam

Calculation for the beam is made in Mathcad, see Appendix C. The beam is continuous over two walls which make the calculation a bit more complex. To solve the moment and shear distribution, the slope-deflection method has been used. There are uncertainties of how the loads are distributed in the CLT that are supported by two walls. It is assumed that a part of the load is transferred to the beam and some loads goes directly to the walls. It is also assumed that there is no interaction between the CLT and the beam which means that higher moments and reaction forces can be expected compared to FEM-calculations.

Several comparisons have been made for the beam to see if the results are reasonable. Since there are more uncertainties with this beam, it is good to have more checks. In Table 6.5 it can be seen that most of the results in the hand calculation are larger than the FEM-calculations. Two results stand out; the field moment in span 1 and the reaction force at support B.

The field moment in span 1 are a bit lower in the hand calculation but not very much, see Table 6.5. However, this could be explained by the uncertainties in how the vertical loads are distributed in this span. It has the largest span length, 4.26 m, and therefore the beam might be more loaded instead of the two supporting walls.

The reaction force at B differs quite much and here it is the hand calculation that gives the largest force. One explanation could be that in the hand calculation there is not any wall that support the CLT, so in the FEM-calculation some shear force is taken by the CLT directly to the supporting wall.

Table 6.5: Beam comparison

Beam comparison				
	FEM-design	**Hand calculation**	**Diff**	**Diff** [%]
Support Moment B	-24.67 kNm	-28.72 kNm	-4.05 kNm	14.1%
Support Moment C	-20.55 kNm	-21.6 kNm	-1.05 kNm	4.9%
Field Moment 1	31.23 kNm	29.78 kNm	1.45 kNm	4.6%
Field Moment 2	-8.16 kNm	-9.16 kNm	-1.0 kNm	10.9%
Reaction Force A	31.24 kN	33.59 kN	2.35 kN	7.0%
Reaction Force B	57.58 kN	74.43 kN	16.85 kN	22.6%

6.1.5.3 Shear walls

The shear wall check has been made by hand and the calculations are shown in Appendix C. It starts by calculating the wind load, the horizontal load due to unintended inclination and add them together. Calculation of unintended inclination is done according to Eurocode SS-EN 1992-1-1. When the loads are known an overview of the walls is made and each wall used as shear wall gets an indication with a number. There are many windows in the building and the simplification of neglecting the contribution of all wall parts with openings is done in the following calculation. This is made as the windows are at the same locations for all floor plans.

The load will be distributed depending on the location and stiffness of the walls. This comes from the assumption that the floor plates are assumed to be totally stiff in comparison to the walls. All walls are of the same material and cross section, hence only the length of each wall segment influences the stiffness ratios. With these assumptions the moment of inertia of each wall (depending of the wall length) is calculated and the location of each wall is noticed. The moments of inertia are used to find a stiffness ratio between the walls and in further calculation it is seen that stiffer walls attracts more load.

The known horizontal loads serves as input to the Excel calculation sheet for design of shear walls (see Appendix C), where the stresses in the walls are calculated. The most loaded wall of the building is checked here. The calculation consists of three checks which are made according to *The CLT Handbook* (Swedish Wood, 2019b). These are, shear at the lamellas, both the ones that are oriented vertical and those that are horizontal. The last check is shear between the layers.

If the building has not enough weight to resist horizontal loads from tilting, the walls need to be anchored in the foundation to resist the tension that occurs when the building tilts. The same applies locally for all the walls at each floor. When using the simplified method as used here when neglecting the wall parts with openings, there is a high risk that tension stresses would occur next to the openings implying for many tensile connections. In order to avoid this a more detailed calculation could be done to include the whole walls. These calculations have not been done, hence what is verified in this section is only that the shear capacity of the walls are sufficient.

6.2 Gasklockan, Västerås

This project is quite special, but it also illustrates a situation where the advantages of timber would come to real use. The project is a renovation of office spaces in an older building together with a vertical extension with the addition of one extra storey. The original building were four storeys with a height of 18 meters, a length of 147 meters and width of 25 meters. In the middle of the building a brand new 21 meters wide entrance with the whole building-height as free head space is replacing the old part of the building. However, this study will only focus on the new storey, which is built on top of the old slightly cambered concrete roof.

There are some natural limitations on how the new part can be constructed given by the layout of the original building. As the concrete roof is quite thin, there is no possibilities to increase the load on the roof. This means that all loads need to be taken to the columns in the existing building; this gives a fixed grid of support points of 7 meters x 9.1 meters. Also, the facade of the new floor will be 3.45 meters inside of the existing facade. This means that there will be no direct support beneath the new facade, and hence this load needs to be transferred to the adjacent columns below.

There is no possibility to strengthen the foundation, meaning that the loads from the extra story should not be of such magnitude that the total load on the foundation exceeds the capacity. This is possible as the original building were designed for traffic of heavy forklifts, now when the use is as office spaces the imposed load on each floor is reduced. But it still means that it is important to keep the self-weight of the new storey low. Here is where timber really could make an impact as it is possible to build lighter structures, compared to concrete or steel.

6.2.1 Original design

The extra storey was constructed with a steel and concrete beam column system. Concrete hollow core slabs where used for the floors, spanning 9.1 meters between steel I-beams. Square hollow steel columns were used to support two large steel roof trusses. TRP-profiled steel plates are used as roof structure. Diagonal steel braces is placed between the columns at a few locations to give horizontal stability to the structure. The columns in the facade are placed on I-beams that transfers the load to the adjacent columns beneath to ensure that no extra load is put on the existing concrete roof.

6.2.2 Decision process

For this project the self-weight of the structure has been considered important, therefore lightweight solutions have been preferred. As the new storey will be located on top of the existing building, but with the old concrete roof still in place, acoustic performance of the floor has been considered to not be a problem. The top of the roof should be at the same level as for the original design. However, the thickness of the floor panels is not strictly limited as the cambered shape of the old roof gives some extra room for the new floor panels, meaning that there is some extra space downwards which could be used. In this project, as the placement of the supporting columns on the floors below are fixed, the span lengths are also set. The spanning direction of slabs is however free to choose.

A comparison of the structural self-weight of the original concept shows that the proposed timber concept gets approximately one third of the weight of the original design. The biggest difference is in the floor, where the hollow-core concrete slabs are significantly heavier than the proposed CLT-rib elements. The steel parts only contribute to a smaller part weight, hence having the beams and columns in steel or timber does not give such a big impact on the total self-weight. The estimation is shown in Appendix A, and includes only the load-bearing structural parts.

6.2.3 Proposed timber concept

The proposed concept is a beam-column system of glulam. The beams that supports the floor runs in the long direction of the building and is simply supported in the 7 meter spans. Ribbed CLT elements is used as floors and is simply supported over the 9.1 meter between the beams. In order to keep the total thickness of the floor structure reasonable low, it is important that the ribs of the floor elements are connected to the side of the main beams. Only the CLT part should rest on top. The columns in the facades will be resting on transverse inclined side beams in a similar manner as for the steel structure, but these beams will be of glulam as well. Steel cross bracing is put at a few locations along the facade to give stability to the structure. A plan view of the new floor is illustrated in Figure 6.12, including the sizes of the main load bearing elements.

For the roof structure, a large double tapered glulam beam spans 18.2 meters between the two outer rows of the interior columns. This enables the removal of the middle column row, which gives even more freedom and flexibility in the plan layout for the future. It also reduces the load on the middle row of columns below, as these are heavier loaded with a larger influence area from the floor. The use of large roof beams will create sections that makes the drawing of installations a bit more complicated. However, the solution for this would be to use the space along the outside of the columns for longitudinal distribution and then take the transverse distribution in between each beam. The installations will be covered by a ceiling placed just below the bottom of the roof beams. TRP-profiled steel sheets is used for the roof as there is no timber alternative that really could match this option in terms of low weight. The roof structure is best shown in the section view in Figure 6.13.

The development of this concept only includes the preliminary design of the main load-bearing parts of the structure. Similar to the Traneberg project there is still need for a detailed design concerning fire, thermal insulation in facades and moisture. Some connections that needs extra consideration in the detailed design phase is the connection between the double tapered roof beam and side beams in the roof. This includes avoiding to much local pressure perpendicular to the grain of the roof beams. The connection of the columns down to the existing structure also needs some extra attention. The detailed design of such details is left without of the scope for this study.

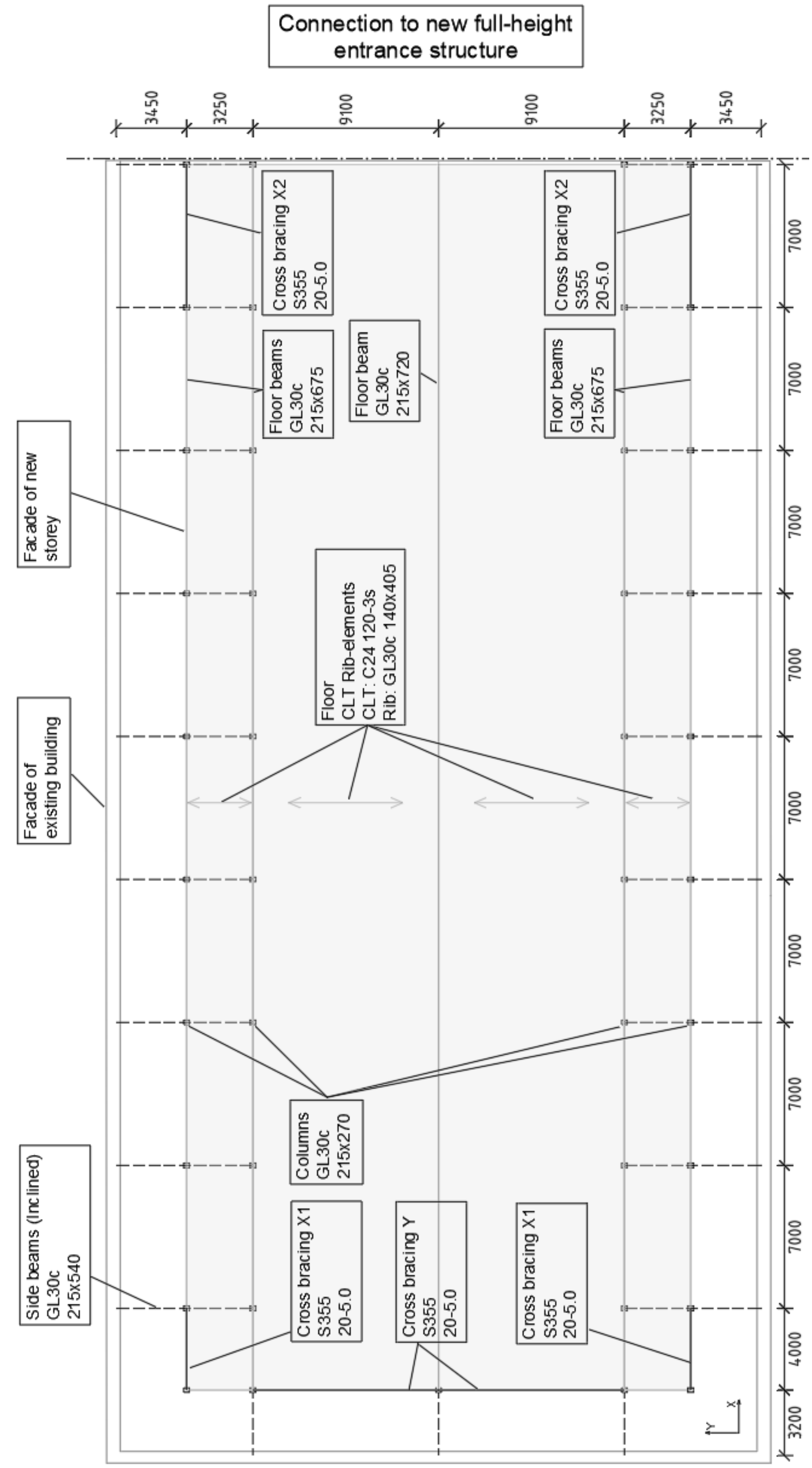

Figure 6.12: Plan view of the new floor built on top of the existing building, including sizes of structural elements. This only illustrates one side of the building, which is connected to the large main entrance. On the other side of the entrance a similar extra floor should be built.

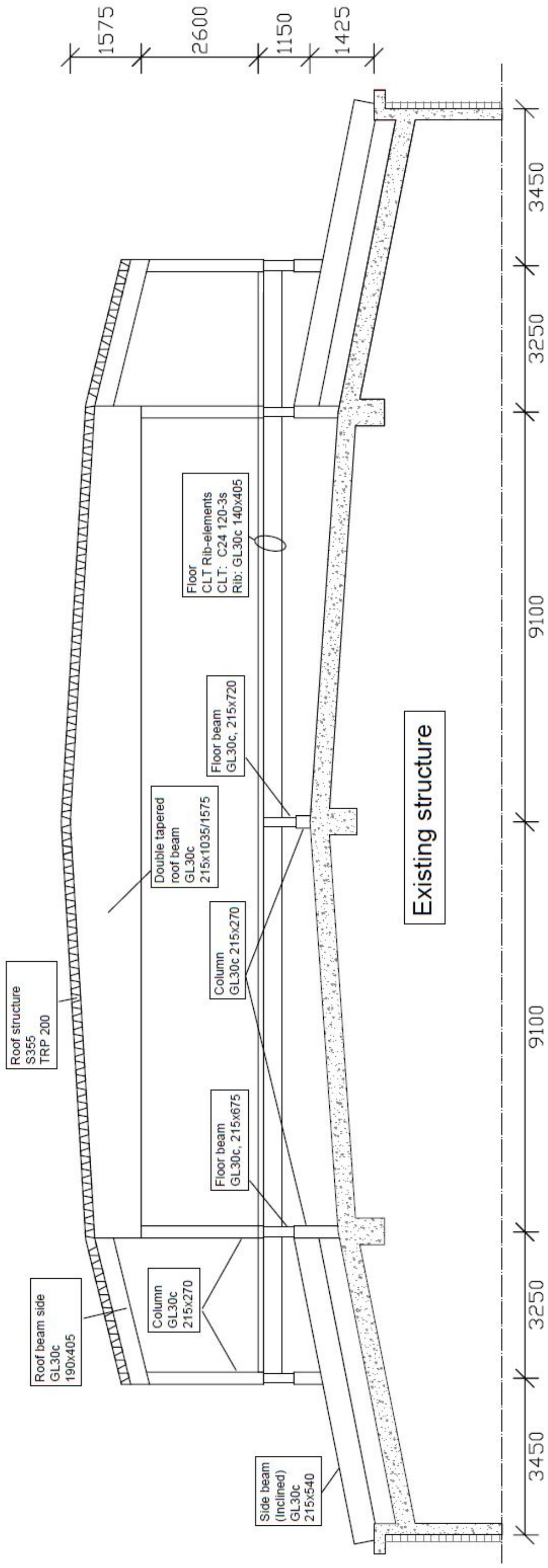

Figure 6.13: Section view on how the new floor is built on top of the existing building. Including sizes of main structural elements.

6.2.4 FEM analysis of concept

The proposed concept was modelled and analysed in Strusofts finite element software FEM-Design 3D structures. This was done in order to show that the proposed concept is a reasonable solution with dimensions that fulfils the requirements in ULS and SLS. A 3D view of the model is shown in Figure 6.14. The utilisation ratios in ULS for all the timber bars is illustrated with a colour scheme in Figure 6.15. As seen in the picture, the design has been made so that all elements fulfils the ULS requirements. Further analysis of the deflections are presented in the comparison between hand calculations and FEM results in Section 6.2.5. Other SLS criteria such as vibrations and springiness for floors where done with hand calculations and are attached in Appendix E. In the following section the model used in FEM is described together with the key verifications made to see that the model is reasonable.

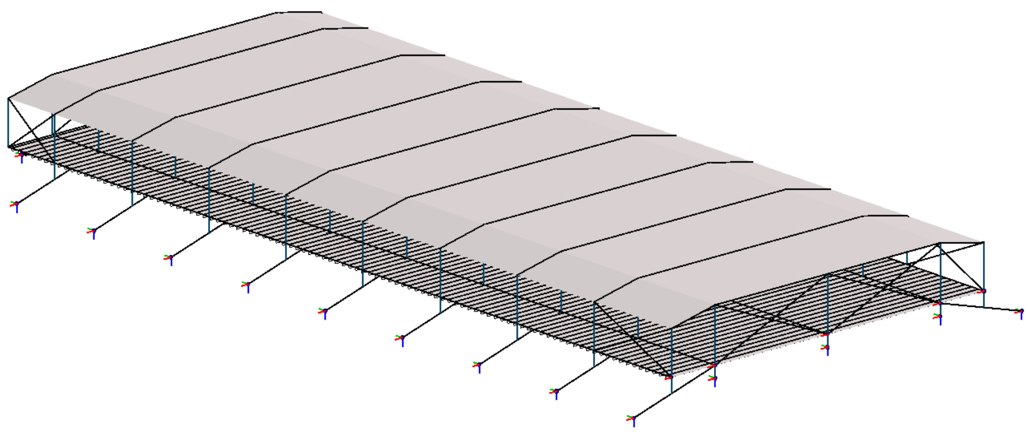

Figure 6.14: 3D view of the FEM model

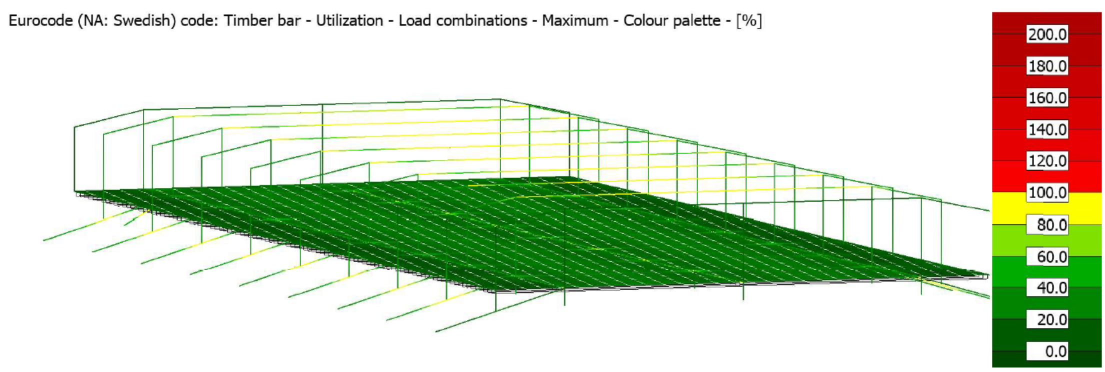

Figure 6.15: Utilisation ratios for timber bars in Gasklockan

6.2.4.1 Loads and load combinations

The applied characteristic loads are presented in Table 6.6 and the used wind-pressure coefficients are shown in Table 6.7. These are calculated according to Eurocode and used in both the FEM model and the hand calculations. All loads were applied as surface loads in the model. Covers spanning from roof to floor were used between columns in the facade to distribute the applied wind loads in the proper manner. Several load combinations were used to find the design stresses and deflections in the structure. These include both ultimate limit state load combinations and serviceability limit state characteristic and quasi-permanent load combination. All the load combinations used in the FEM-model are listed in Appendix B.

Table 6.6: Vertical and horizontal characteristic loads on Gasklockan new storey.

Characteristic loads		
Type of load	**Load** $[kN/m^2]$	**Standard**
Structural dead weight	Automatic in software	-
Extra dead weight roof (e.g. installation and insulation)	0.4	Own assumption
Imposed load floors	2.5	EKS 11
Snow load	1.6	EC SS-EN 1991-1-3
Wind load $[C_e(Z) = 2.5]$	0.9	EC SS-EN 1991-1-4

Table 6.7: Wind pressure coefficients.

Wind pressure coefficient $[C_{pe10}]$		
Load Direction	**D** (Pressure)	**E** (Suction)
Wind load X+	0.72	0.27
Wind load X-	0.72	0.27
Wind load Y+	0.72	0.45
Wind load Y-	0.72	0.45

6.2.4.2 Mesh and modelling

Table 6.8 gives an overview on how each of the elements were modelled, including boundary conditions, element type, mesh size etc. One main load carrying direction were used for both the TRP roof and the ribbed CLT floor. The double tapered roof beams were drawn with two beams with varying cross-section that were rigidly connected to each other in the middle of the span where they met, and simply supported at the edges on the columns. The TRP profile were drawn manually in FEM-designs section editor, with a plate thickness of 1 mm. As the model only includes the additional top-storey of the building, supports preventing translational displacements were put on the positions of the underlying columns.

Table 6.8: Presentation of how different elements where modelled in FEM.

FEM modelling of elements						
Part	**Material**	**Section [mm]**	**Element type**	**BC's**	**Analytical axis**	**Mesh [m]**
Floor CLT	C24/C14	120-3s	2D Shell	Hinged	Bottom	0.4x0.4
Floor Rib	GL30c	140x405	1D Beam	Hinged	Centric	2 parts
Floor beam (mid)	GL30c	215x720	1D Beam	Hinged	Centric	2 parts
Support beam	GL30c	215x540	1D Beam	Hinged	Centric	2 parts
Roof sheet	S355	TRP 130-1.0	2D Shell	Hinged	Centric	0.2x0.2
Roof beam (main)	GL30c	Varying	1D Beam	Hinged	Centric	20 parts
Column	GL30c	215x270	1D Column	Hinged	Centric	2 parts
Bracing tie	S355	20-5.0	1D Truss	Hinged	Centric	1 part

The autogenerated mesh in FEM-design seemed at first to be applicable without any changes, giving a nice and smooth mesh over larger surfaces and a denser mesh around singular point supports (i.e columns). However, after evaluating the results a change was done for the mesh of the TRP roof structure. A smaller mesh size were chosen here to give a smoother distribution of the snow loads as the TRP-profile is quite complex. Also, to give a more accurate calculation of the double tapered roof beams, these elements were divided into 20 parts each instead of two parts used for the ordinary beams. Figure 6.16 shows the final mesh for a part of the building.

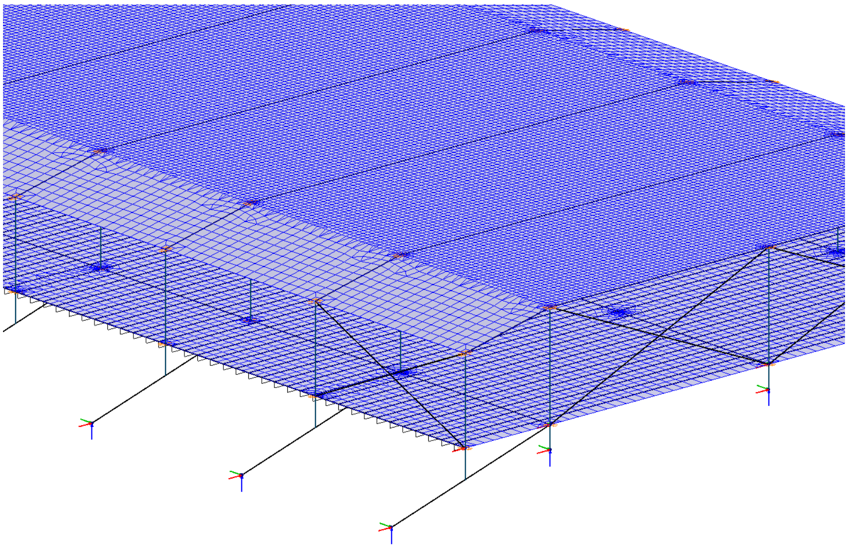

Figure 6.16: Close up picture showing the FEM mesh.

6.2.4.3 Verification of FEM-model

To verify that the model was created properly and behaves as expected the results of the analysis were thoroughly studied. Figure 6.17 shows the long-term deflections from quasi-permanent loads for the roof and floor structure. The results seem reasonable as the upper picture have zero displacements at the locations where the double tapered beams are supported by columns. The beams deflect across the building, and the TRP profile deflects between the beams to give this circular pattern with the largest deflections in the middle of the column grid. At the end spans of the TRP the deflection is largest, this comes as the plate is modelled continuous over the beams but simply supported at the outer edges. This gives zero moment in the plate at the gable, which will cause a larger deflection in that span. As the columns in the facades are not resting directly on a support but on an inclined beam, the deflections here will be given by that beam and is hence not zero. The moment distribution of the main beams also looks as expected as all of them are simply supported. This is illustrated in Figure 6.18.

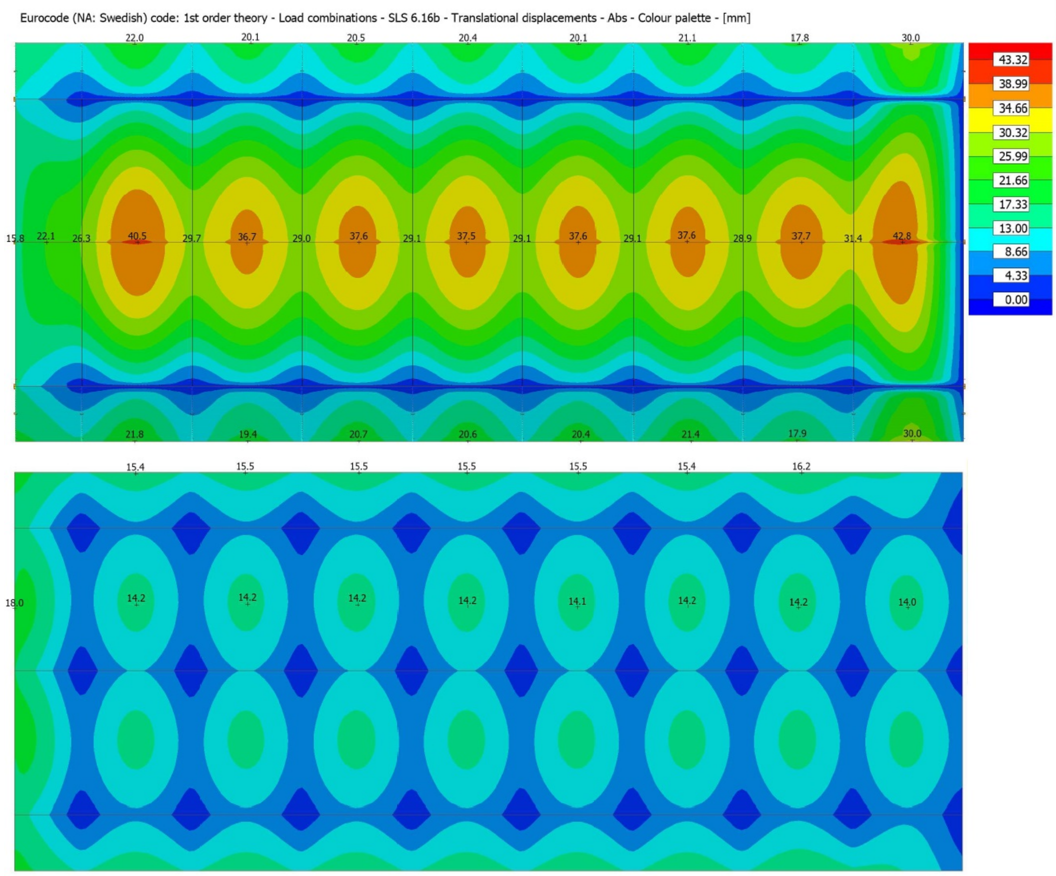

Figure 6.17: Deflections of the roof (top) and floor (bottom).

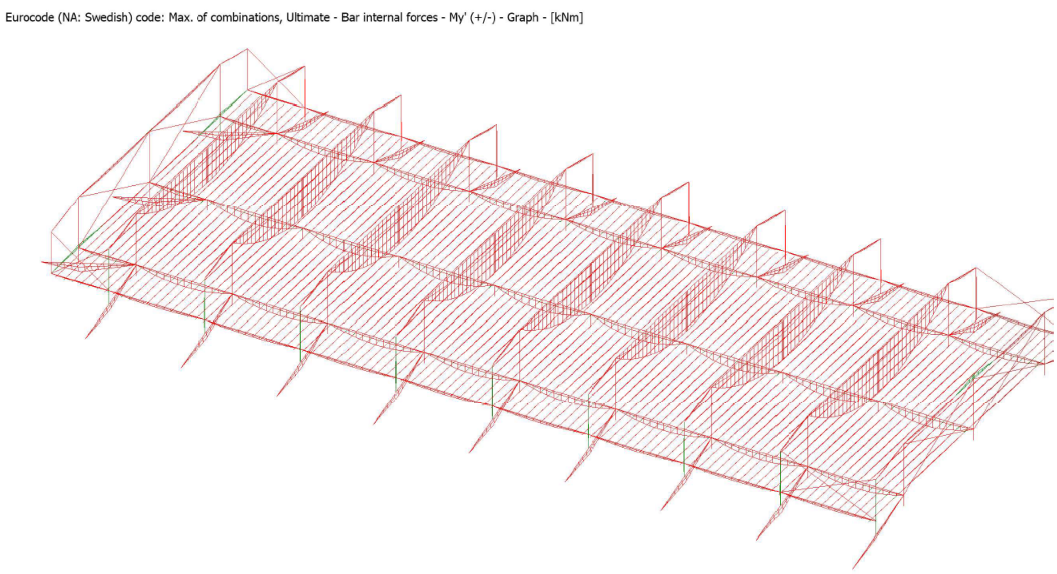

Figure 6.18: Moment distribution along beams in Gasklockan.

To ensure that the floor was modelled in a correct manner, the stresses in the ribbed CLT plate were also studied. As the floor is simply supported between the main floor beams a pure CLT plate would show tensile stresses in the bottom layer. However, as the CLT should form a composite cross-section with the rib beams, compression stresses are to be expected even in the bottom layer of the CLT. Figure 6.19 shows the difference in compression stresses between the top of the upper layer and the bottom of the lower layer under self-weight.

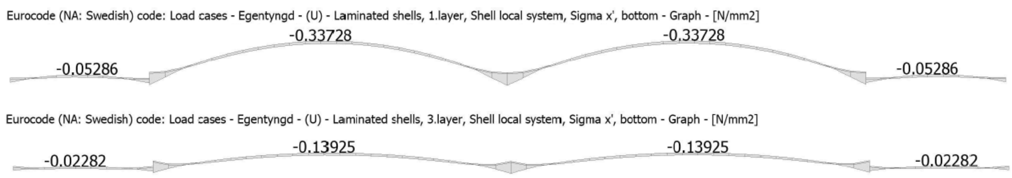

Figure 6.19: Normalstresses in top and bottom for a section of the CLT floor under self-weight. Notice that the composite action with the ribs makes the whole section act in compression in the spans.

6.2.5 Comparison between hand calculations and FEM results

In order to further verify that the FEM-model used is created in a reasonable manner, comparisons have been made to simplified hand calculations. The comparison as presented in this section includes mainly stresses and displacements as this was considered to be the most suitable units to use when presenting it. But simultaneously when calculating the stresses and displacements in hand calculation, checks were made in both ULS and SLS for the individual parts so that the elements fulfil the structural requirements. The hand calculations were made in Excel and the full calculation sheets are found in Appendix E. The Excel sheets are built up on the same basics as the ones which later are presented in Chapter 7, hence the further explanation on the theory behind those can be found in the next chapter. Design sheets on certain structural members exported from FEM model can be found in Appendix F.

The comparison is presented in Table 6.9. Here it can be seen that the stresses in ULS are matching well between the FEM model and the simplified hand calculations. The difference is only a few percent even for the quite complex double-tapered roof beam. This is a verification that the model shows the indented behaviour. For the displacements, which are made for characteristic load combination, the difference is larger, in the size between 12-20%.

Table 6.9: Comparison of stresses from FEM-design and hand calculations for different elements

Comparison of stresses from FEM-design and hand calculations								
	σ_{md} [MPa]		τ_{vd} [MPa]		σ_{c0d} [MPa]		Disp [mm]	
	FEM	Hand	FEM	Hand	FEM	Hand	FEM	Hand
Floor beam	14.2	15	2.5	2.3	-	-	18.2	15
Difference	5.5%		6.9%		-		17.6%	
Side beam	16.0	16.3	1.0	1.0	-	-	21.7	17.5
Difference	1.9%		0.0%		-		19.4%	
Roof beam	12.4	13.1	2.1	2.3	-	-	62.6	71.2
Difference	5.2%		8.6%		-		12.1%	
Roof column	-	-	-	-	4.4	4.2	-	-
Difference	-		-		4.3%		-	
Floor column	-	-	-	-	5.3	5.5	-	-
Difference	-		-		2.4%		-	

A special evaluation on the stresses in the ribbed floor elements is presented in Table 6.10. It is interesting to see that the stresses in the beam part of the structure seems to be underestimated with the hand-calculation compared to FEM, while the CLT part is overestimated. The match is however not very close with difference up to 38%. As this type of element with composite action between the CLT and the ribs is not covered by Eurocode there are no specific standard on how to calculate the stresses for it. Hence, there are several possible sources of error to why the stresses differ.

The first is the assumptions on how the cross-section is built up for the hand-calculation. A T-shaped effective cross section is created with a certain effective width of the CLT panel that is contributing to the stiffness. This is done based on a method for plywood described in (Swedish Wood, 2015). Also, this cross section only considers the layers in the CLT panel that are parallel to the beam. Here the rolling shear modulus of the cross layers will influence the stress distribution in the cross section, but this is not included in the hand calculations. It also assumes that all layers are rigidly connected.

The second is that the hand-calculation assumes that the floor is only carrying loads in one direction. This is also how it is modelled in FEM, but as the main glulam beams which the floor is resting on is deflecting, then also the CLT will deflect in the direction perpendicular to the load carrying direction. From this there will be stresses induced from which the effects are not easy to estimate without extensive study into this. So, both these two issues might contribute to the difference in stresses in the comparison.

Table 6.10: Comparison for ribbed floor element

Comparison for ribbed floor element								
	σ_{wcd} [MPa]		σ_{wtd} [MPa]		σ_{fcd} [MPa]		Disp [mm]	
	FEM	Hand	FEM	Hand	FEM	Hand	FEM	Hand
Floor beam	1.9	1.2	5.6	3.9	2.5	2.7	11.5	9.3
Difference	38%		30%		8%		19%	

In FEM-design a second, slightly modified FEM-model where created to find stresses in the tying system that were possible to verify against hand calculated values. This model considers fewer load combinations where wind is the dominant one. The verification is only done for wind in the positive global directions, and it is assumed that wind from the other directions will give a similar behaviour. As the bracing should work as ties in tension, only the diagonals working in tension are modelled. This differs from the full model where both diagonals in the crosses are modelled as the wind loads are applied for combinations of all directions.

The hand calculation is simplified and based on the assumption that each bracing unit takes the load from a certain influence area of the wind pressure on the facade. The loads acting on the facade are assumed to be divided equally up to the roof and down to the floor which distributes the loads to the bracing units. The horizontal loads going through the floor are assumed to be stabilised by the inclined beams under the facades. Hence, this calculation for the bracing units focuses on taking the horizontal loads in the roof down to the floor level.

For the ties working in global X-direction there is a quite large distance (56 meters) between the units. As the applied wind pressure on the windward side is larger than the suction on the leeward side it is assumed that more of the load will go straight down through the closest bracings and not evenly distributed through the system. In the hand calculations therefore the X1 bracing units are calculated for the windward side load, while the X2 is for the leeward. Labels of the bracing units comes from Figure 6.12.

When comparing FEM-results to the hand calculations the X1 side is less loaded in FEM and the X2 is more. This is a sign that in the FEM-model there clearly are some distribution. So as a further check, an extra comparison of the mean values of stresses in X1 and X2 are also made. When comparing the mean stresses, they show very similar results. For the Y-direction the bracing units are located next to each other, hence here only the mean values are compared as they are assumed to take equally much load. Table 6.11 presents the result of the comparison for the ties in the bracing units.

Table 6.11: Comparison for bracing steel ties

Comparison for bracing steel ties			
Tie	FEM [MPa]	Hand [MPa]	Diff [%]
X1	115	127.5	10.9%
X2	61.1	42.3	30.8%
X mean	88.1	84.9	3.6%
Y mean	181.3	187.9	3.5%

7

Preliminary design tools

In this chapter several design tools are presented in the form of preliminary sizing diagrams. These have been developed from the calculation sheets used when doing the preliminary design of the two reference projects. The purpose with the diagrams is that they could be used to get a hint of the sizes for the main structural elements in the building, early in the decision process. They do not represent a final design, and more detailed design including fire safety, acoustics, robustness and connections needs to be done. The diagrams include beams, columns and different types of floor structures. The main assumptions behind the calculations are described in each section. More precise details on coefficients and material properties used is shown in the excel calculation sheets in Appendix G.

7.1 Beam

There are many different types of timber beams available on the market, such as sawn timber, glulam and LVL. However, glulam is the most common for larger structures, so the beam diagrams are made for glulam beams with strength class GL30c. The stresses and deflections are calculated according to beam-theory for bending of a simply supported beam under distributed load. Bending- and shear capacity in ultimate limit state is calculated according to Section 6.1.6 and 6.1.7 in (SS-EN 1995-1-1). For the serviceability limit state, the calculation considers deflection of the beam under characteristic load combination including long-term effects for creep, with the limit set to $L/300$. The beam is assumed to be secured against tilting and lateral instability. Hence, these effects are not included in the calculation. Climate class 1 (indoor) for a medium-term loading (imposed load) is assumed to decide the design strength of the timber material.

In order to present clear and easy to use diagrams, they are divided into four separate ones, where the width of the beam is what varies between them. The widths vary from 90 mm to 215 mm. In each graph the different lines represent a span length. For a specific span length and a given influence width, the height of the beam can be obtained on the y-axis. The influence width corresponds to the total width from where the beam attracts load. This is dependent of the placement of the beams, span length of the floor and if the floor is continuous over the beams or simply supported. This layout of the diagram gives the possibility to quickly estimate the cross-section height for the glulam beam for different structural layouts. The diagrams are presented in Figure 7.1 to 7.4

The loading used includes the self-weight of the beam itself and a self-weight for a floor structure that would represent a 200 mm thick pure CLT floor, which is slightly less than $1.0kN/m^2$. An extra $0.5kN/m^2$ is added to the self-weight to represent insulation etc. The imposed load used as leading variable load is $2.5kN/m^2$, and an accompanying variable load of $0.5kN/m^2$, which could be for instance partition walls, is included in the load combination. For buildings where heavy installations are supposed to be in the floor, the cross-section height might need to be increased.

Load combination 6.10b from Swedish National Annex EKS11 is used in ULS:
$0.89 \cdot 1.35(G_{kfloor} + G_{kbeam} + G_{kextra}) + 1.5 \cdot Q_{k1} + \psi_0 \cdot 1.5 \cdot Q_{k2}$

Characteristic load combination 6.14b in SS-EN 1990 is used in SLS:
$G_k + G_{kbeam} + G_{kextra} + Q_{k1} + \psi_0 \cdot Q_{k2}$

Figure 7.1: Beam heights versus influence width for office building $G_k \approx 1.0kN/m^2$, $G_{kbeam} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$ and $Q_{k2} = 0.5kN/m^2$

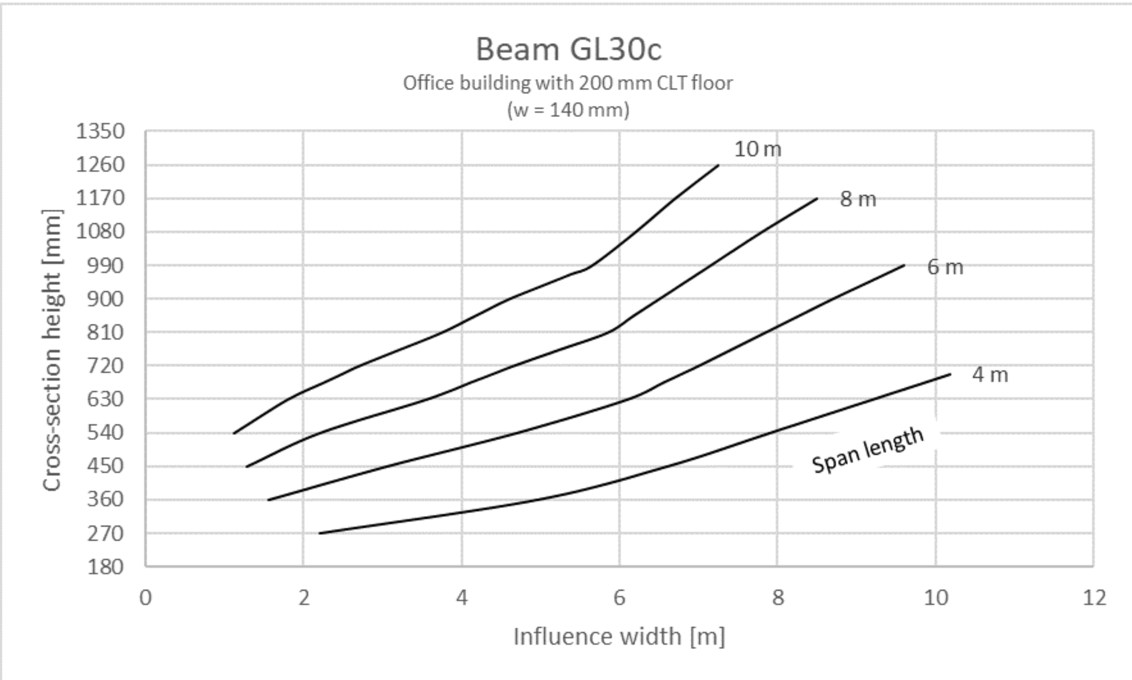

Figure 7.2: Beam heights versus influence width for office building $G_k \approx 1.0kN/m^2$, $G_{kbeam} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$ and $Q_{k2} = 0.5kN/m^2$

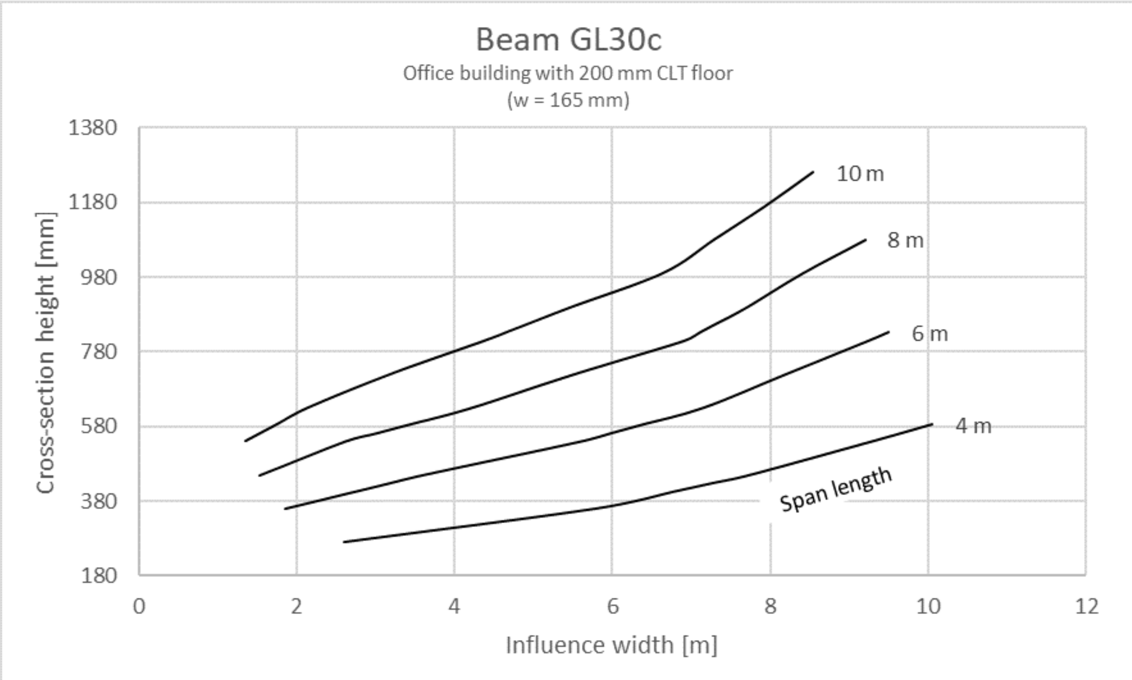

Figure 7.3: Beam heights versus influence width for office building
$G_k \approx 1.0kN/m^2$, $G_{kbeam} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$ and $Q_{k2} = 0.5kN/m^2$

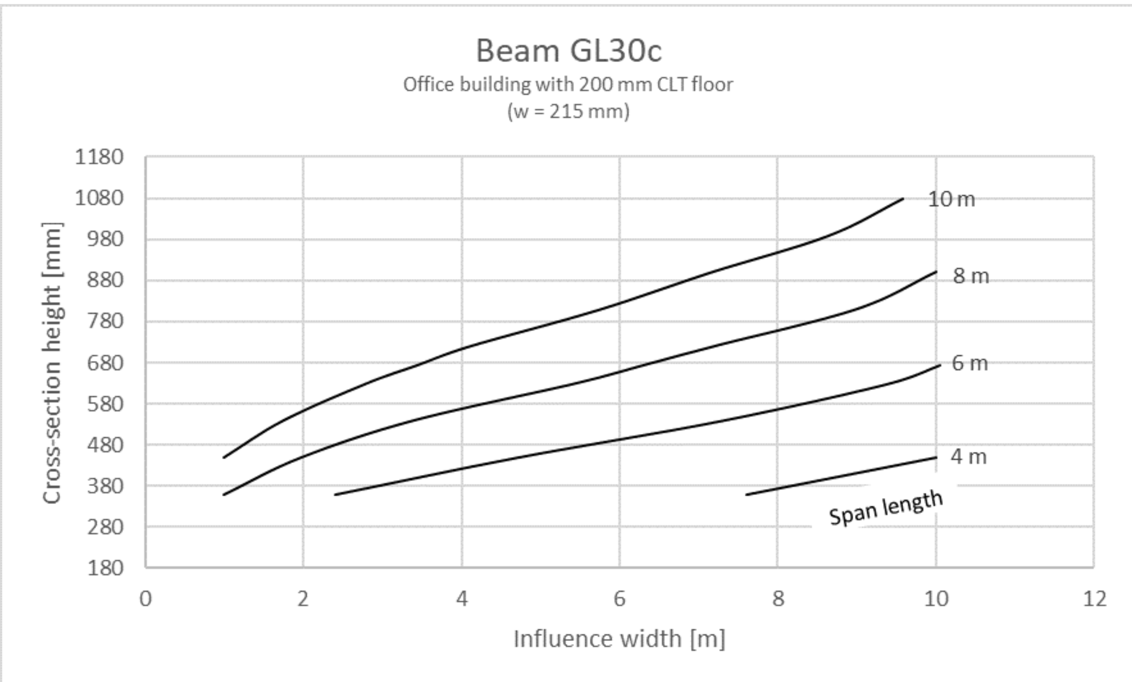

Figure 7.4: Beam heights versus influence width for office building
$G_k \approx 1.0kN/m^2$, $G_{kbeam} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$ and $Q_{k2} = 0.5kN/m^2$

7.2 Column

The column diagrams are done for glulam columns with quadratic cross-section. 3m is set as the height of the column. The total loading in the column depends on how large area (influence area) that the column attracts load from, and the number of storeys above the column. Only vertical loads are included. Verification of the resistance is done according to Section 6.3.2 in (SS-EN 1995-1-1), with columns in pure compression. It is modelled with hinged boundary conditions in both ends, hence the buckling length is equal to the column length. Climate class 1 and medium-term loading is assumed. GL30h is used as the strength class. This as the whole section is more or less equally loaded and therefore it is beneficial to use a homogeneous cross-section layup with lamellas of the same strength class throughout.

There are two diagrams presented for different characteristic imposed loads. One for residential buildings in Figure 7.5, and one for office buildings in Figure 7.6. The graph gives for a certain influence area a dimension of the column or vice versa. There are several lines in each diagram, which represents how many storeys that are above the column. For instance, if a building have 6 storeys and the column size at floor 2 is of interest, the column at floor 2 will have 4 storeys above. Calculate the influence area, go to the 4 storeys line and read the required dimension on the y-axis.

The loads that are included in the calculations are similar to those for the beams in the previous section. An approximated self-weight of slightly less than $1.0kN/m^2$ is to represent a floor structure of 200 mm thick pure CLT, together with an extra $0.5kN/m^2$ for insulation and the actual self-weight of the columns themselves. Imposed loads are set as $2.0kN/m^2$ for residential buildings and $2.5kN/m^2$ for office buildings. $0.5kN/m^2$ is again included as an accompanying variable load to represent for instance partition walls. Note that for floors where heavier installations are expected, the loading might increase above what is assumed here, and hence larger cross-sections will be needed.

ULS design is made with load combination 6.10b from Swedish National Annex EKS11:
$0.89 \cdot 1.35(G_{kfloor} + G_{kcolumn} + G_{kextra}) + 1.5 \cdot Q_{k1} + \psi_0 \cdot 1.5 \cdot Q_{k2}$

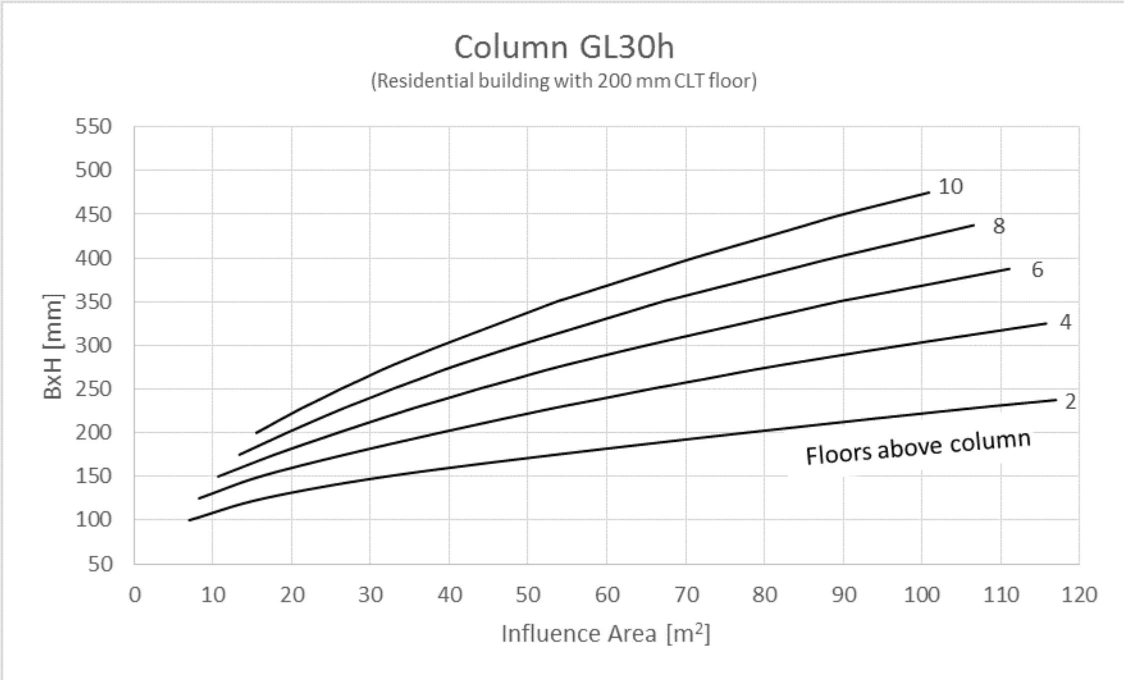

Figure 7.5: Column dimensions versus influence area for residential building $G_k \approx 1.0kN/m^2$, $G_{kcolumn} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.0kN/m^2$ and $Q_{k2} = 0.5kN/m^2$

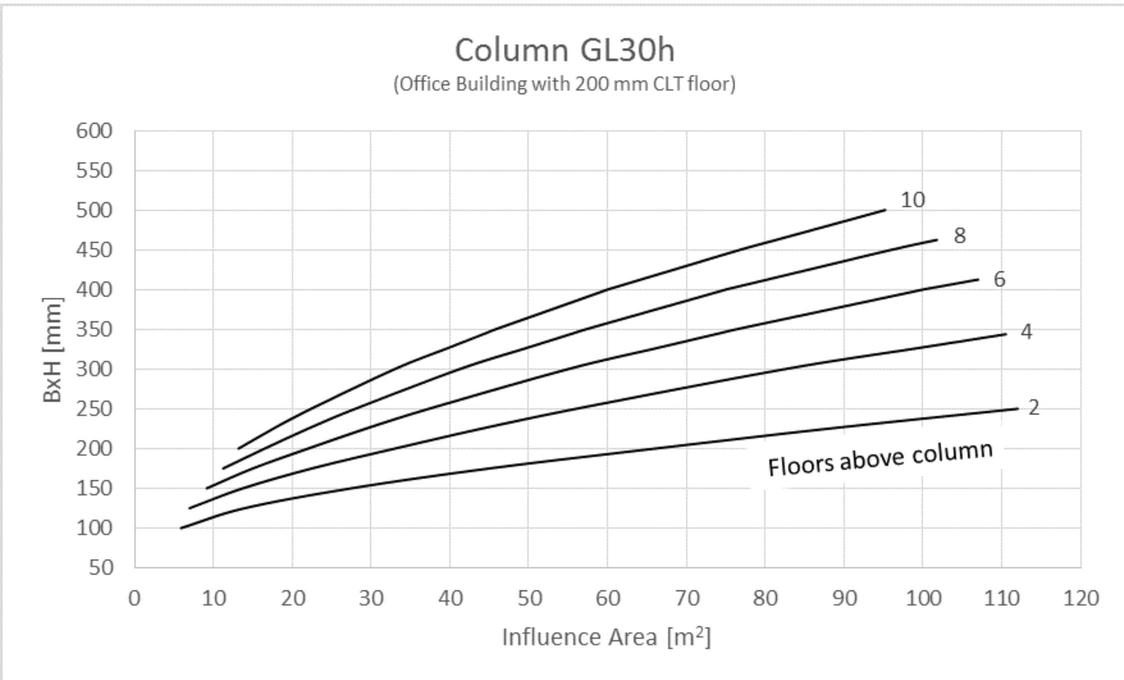

Figure 7.6: Column dimensions versus influence area for office building $G_k \approx 1.0kN/m^2$, $G_{kcolumn} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$ and $Q_{k2} = 0.5kN/m^2$

7.3 Floor

7.3.1 CLT plate

The calculations for the CLT panel are based on beam theory per unit width, simply supported on two edges (i.e. spanning in one direction). This is done by creating a net cross section according to *The CLT Handbook* Section 3.3 (Swedish Wood, 2019b), where the layers parallel to the load bearing direction are included and those perpendicular to it are neglected. It is assumed that the outer layers are running in the load bearing direction and that every other layer is perpendicular to that. The ULS design is made for bending and shear according to chapter 3.3.5 in *The CLT Handbook* (Swedish Wood, 2019b).

The deflections are calculated for SLS characteristic load combination after long time, with Kreuzingers theory as presented in *The CLT Handbook* (Swedish Wood, 2019b), Section 3.3.6. This method is similar to the Timoshenko beam theory as it includes the shear deformations in addition to the general bending deformations. The limit for deflection is set to L/300. Furthermore, a check of the vibrational performance of the floor is done by calculating fundamental frequency, unit impulse velocity and instantaneous deflection according to section 7.3.3 in (SS-EN 1995-1-1). The latter calculations are not developed specifically for a CLT floor and is hence only a rough estimation, further analyses are recommended to be done for those aspects. Also, in an upcoming revision of Eurocode these methods will be revised.

Calculations for CLT floors are made for several different built ups of the cross-section. In the diagram in Figure 7.7 it is illustrated in three different categories depending on the number of layers; 3,5 or 7. The exact thickness for each layer in the build ups used is shown in Appendix H. In the diagram, total thickness of the CLT panel can be obtained for different span lengths.

The calculations are made with calculated self-weight of the CLT plate itself plus an extra estimated $0.5kN/m^2$ self-weight for insulation and fire protectives. The imposed load used is that for an office building, $2.5kN/m^2$, which is a bit higher than those for a residential building. In addition to this, an accompanying variable load of $0.5kN/m^2$ is included, representing for instance partitioning walls.

ULS load combination used is 6.10b from Swedish National Annex EKS11:
$0.89 \cdot 1.35(G_{kfloor} + G_{kextra}) + 1.5 \cdot Q_{k1} + \psi_0 \cdot 1.5 \cdot Q_{k2}$

For SLS design of deflections characteristic load combination 6.14b in SS-EN 1990 is used:
$G_k + G_{kextra} + Q_{k1} + \psi_0 \cdot Q_{k2}$

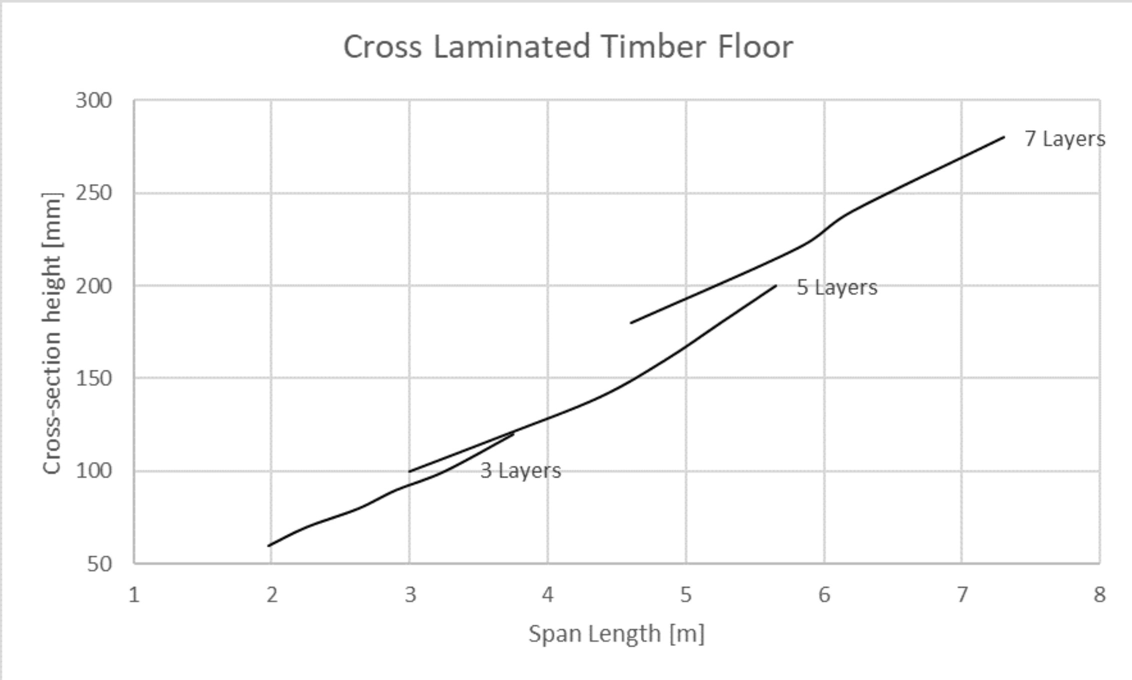

Figure 7.7: CLT floor for different layers versus cross-section height
$G_{kCLT} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$ and $Q_{k2} = 0.5kN/m^2$

7.3.2 CLT- and LVL rib-element

In the following section two diagrams are presented, one for CLT- rib elements in Figure 7.10, and one for LVL-rib elements in Figure 7.11. The diagrams illustrate the total height needed for the elements for different span lengths. The assumptions in the calculations behind the graphs are quite similar. Both are assumed to be simply supported, spanning in one direction and are calculated with the same limits as the pure CLT floor in the previous section. That is; bending stresses, shear stresses, deflections and vibrational performance. Climate class 1 (indoor) for a medium-term loading (imposed load) is assumed to calculate the design strength of the timber materials.

For the CLT-rib element, the cross section is built up according to Figure 7.8, with a constant height of 110 mm for the CLT plate and a constant width for the beams of 140 mm. Figure 7.9 shows similarly the cross section for the LVL rib panel. Here a 69 mm thick LVL-C plate is used, and again the beam width is constant. The cc distance between the ribs are varied with 600 mm and 400 mm for both element types. The material properties used correspond to C24 for the CLT according to *The CLT Handbook* (2019b), GL30c for the beams and a combined LVL-C with material properties according to Kerto-Q in *Design of timber structures Volume 2* (Swedish Wood, 2019a).

The cross-sectional constants for the ribbed CLT elements are calculated as a transformed equivalent T cross-section with an effective width of the CLT and full interaction between the plate and the beam. Yet there is no established way on how to calculate the effective width for a CLT plate, so the model for plywood described in Section 5.26 in *Design of timber structures Volume 1* (Swedish Wood, 2015) have been used. However, as the span lengths are quite long the effective width gets large, so this reduction does not have a very big impact. Similar as for the pure CLT floor, only the layers parallel to the beam is included in the equivalent cross section, the perpendicular layer is neglected. The calculation of deflections is simplified where the bending deflection is calculated for the whole cross-section while the shear deflection only are included for the beam.

In general, the same procedure is used when calculating the cross-sectional constants for the LVL-rib elements. What slightly differs is that the LVL already have a given strength and stiffness properties so that the whole thickness of the LVL plate is included in the equivalent T-section. Effective width is calculated according to section 4.4 in *LVL Handbook* (Finnish Woodworking Industries, 2019).

The same loads and load combinations are used for the two rib element types as was done for the pure CLT floor. This means; calculated self-weight of the floor plate itself plus an extra estimated $0.5kN/m^2$ self-weight for insulation or fire protectives. Imposed load for an office building: $2.5kN/m^2$. An extra variable load of $0.5kN/m^2$ is also included.

ULS load combination used is 6.10b from Swedish National Annex EKS11:
$0.89 \cdot 1.35(G_{kfloor} + G_{kextra}) + 1.5 \cdot Q_{k1} + \psi_0 \cdot 1.5 \cdot Q_{k2}$

For SLS design of deflections characteristic load combination 6.14b in SS-EN 1990 is used:
$G_k + G_{kextra} + Q_{k1} + \psi_0 \cdot Q_{k2}$

The SLS criteria is set to L/300 for deflections, including long term effects, and the vibration calculations are made according to (SS-EN 1995-1-1, 2009). For span lengths around 10 m or more where the beam heights become larger, the failure mode is the fundamental frequency check ($f_1 > 8Hz$). When having a fundamental frequency below 8 Hz, a special investigation of the vibration performance needs to be done according to Eurocode 5. So, for this purpose this is considered as "failure". For those longer span lengths, it can be seen that a decrease of the CC distance does not increase the first frequency. The reason that this happens is that when decreasing the CC distance, the mass of the floor increases slightly more than the stiffness, when using this calculation model. It can be seen in both Figure 7.10 and Figure 7.11 that the two curves approach each other. This means that the effect of decreasing the CC distance to get a lower cross section height becomes smaller and smaller with increasing span length. For the LVL rib-element the two curves in Figure 7.11 even crosses each other and lower heights can be obtained with *CC 600* instead of *CC 400* due to the effect on the fundamental frequency.

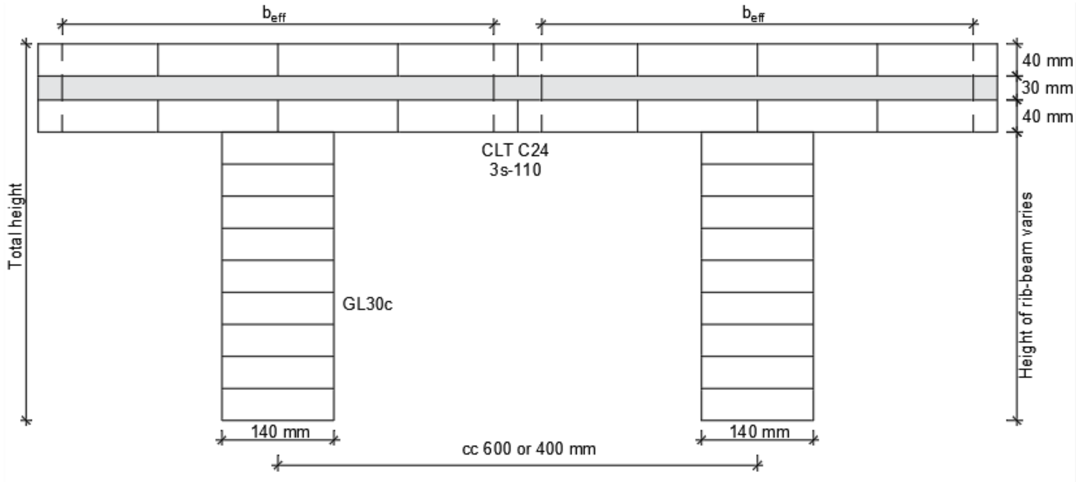

Figure 7.8

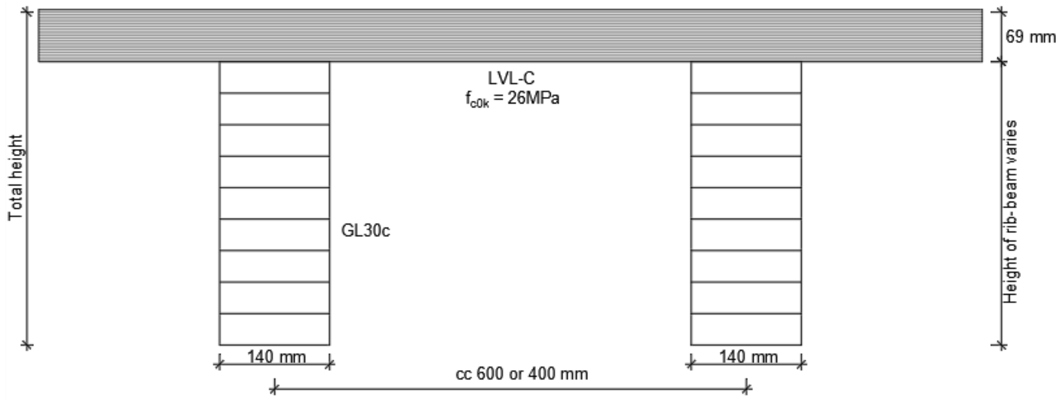

Figure 7.9

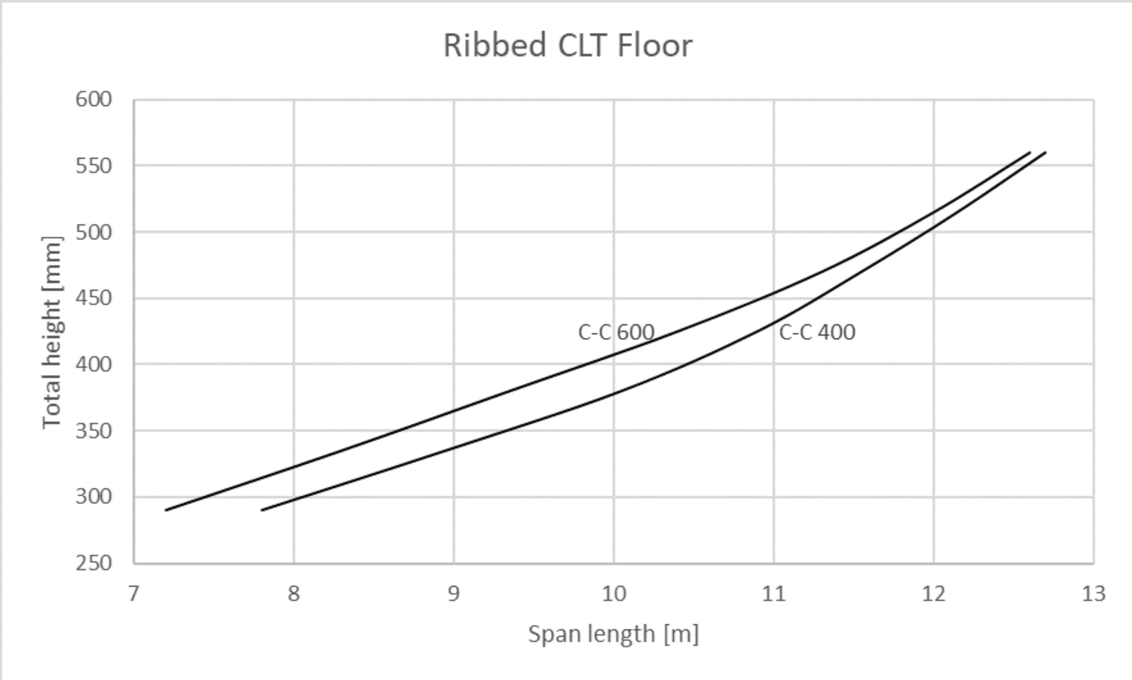

Figure 7.10: Total cross-section heights versus span lengths
$G_{kfloor} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$, $Q_{k2} = 0.5kN/m^2$

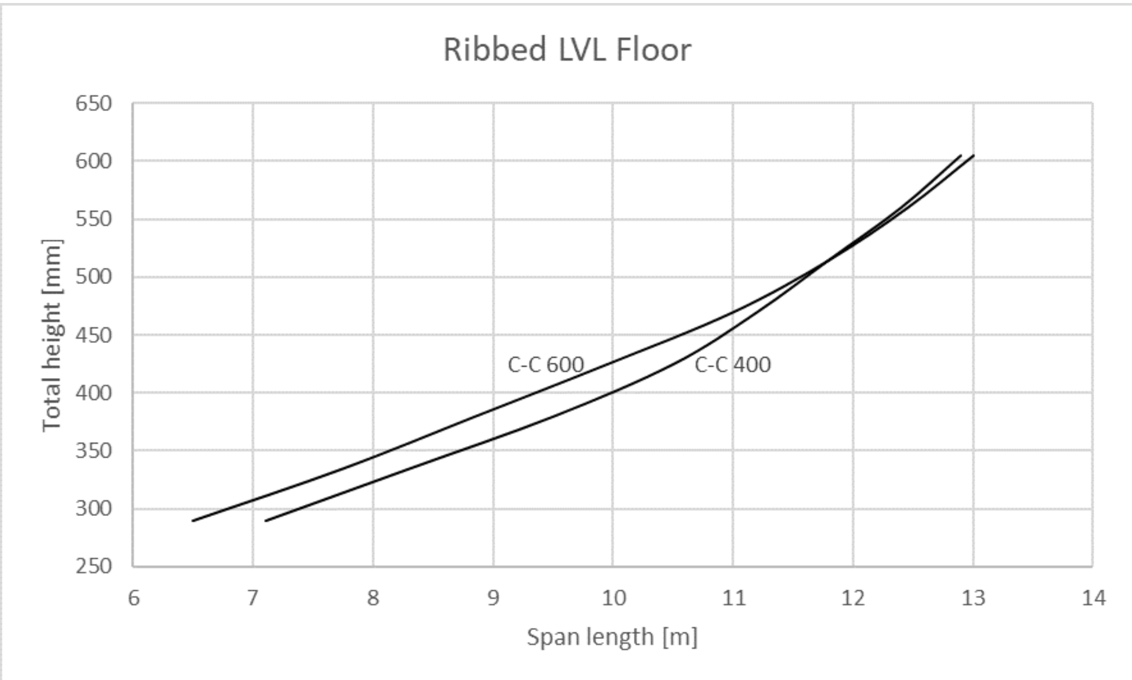

Figure 7.11: Total cross-section heights versus span lengths
$G_{kfloor} = Varies$, $G_{kextra} = 0.5kN/m^2$, $Q_{k1} = 2.5kN/m^2$, $Q_{k2} = 0.5kN/m^2$

7.3.3 TCC Floor

A very rough estimation of preliminary sizes of a timber-concrete composite floor is presented in this section, in Figure 7.12. The main idea behind the floor type is to use timber in tension and concrete in compression, with a shear connector between the two materials creating composite action. The built up of the floor is a CLT plate in the bottom, to which shear connectors of the HBV type is attached with epoxy. HBV is a steel mesh connector which is placed vertically between the two materials it connects. On top of this, a layer of concrete is cast, reinforced with a reinforcement mesh. The CLT plates used in the diagrams have either 5 layers where each layer is perpendicular, or seven layers where the two outer layers runs parallel and the layers between have every other located perpendicular to the main direction. Detailed layer setup of the sections used in calculations are found in Appendix H.

In Eurocode there are no specific methods on how to design timber-concrete composite floors. So in this estimation some general advices are taken from section 5.1.3 in *The CLT Handbook* (Swedish Wood). The first is the approximation that the shear connector type used could be assumed to give a level of interaction around 85%. Hence, the bending stiffness is calculated as a transformed cross section as if there were full interaction, but then reduced to 85% of this. The second assumption is on the long-term behaviour of the floor. Concrete and timber have different creep behaviour, with concrete more prone to creep, which will give the effect that the stresses in the concrete decreases as it loses comparatively more stiffness over time, and these stresses will be distributed to the timber instead. Hence, ULS design is made both for short term and long-term response. For the long-term calculations, the transformed cross-section based on the effective E-modules of the two materials, which takes the creep into account. Here again, 85% interaction level is assumed.

For the ULS verification, the compressive stresses in concrete, tensile stresses in timber, and shear stresses in the timber layers are checked. No reinforcement is included in the calculations, and detailed reinforcement design needs to be done for the concrete in a later stage. However, as it is mainly in compression, it is assumed for most applications there will not be a problem to fit required reinforcement within the section to take possible tensile stresses in the bottom of the section.

The same loads and load combinations are used as for the other floor types. SLS design for deflections are made for characteristic load after long time, with L/300 as requirement. Creep reduced effective cross-section properties are used for both permanent and variable loads. This is a safe-side simplification that probably will overestimate the deflections, as the variable loads are medium term and the creep for those could be calculated with the quasi-permanent ψ_2 factor. But it would be unnecessary complex to use different creep factors for different load types, as the materials also have different creep factors and the stress distribution is dependent on the effective E-modules and not necessarily proportional to the loads at different times. For the vibration criteria, the same method is used as for the pure CLT and ribbed floor elements. Here again the fundamental frequency, f1, is what often is most problematic. This as the concrete gives a large mass to the structure, which lowers the fundamental frequency. According to Eurocode, it might be tolerable with f1 less than 8 Hz, but only if a special investigation is made.

ULS load combination used is 6.10b from Swedish National Annex EKS11:
$0.89 \cdot 1.35(G_{kfloor} + G_{kextra}) + 1.5 \cdot Q_{k1} + \psi_0 \cdot 1.5 \cdot Q_{k2}$

For SLS design of deflections characteristic load combination 6.14b in SS-EN 1990 is used:
$G_k + G_{kextra} + Q_{k1} + \psi_0 \cdot Q_{k2}$

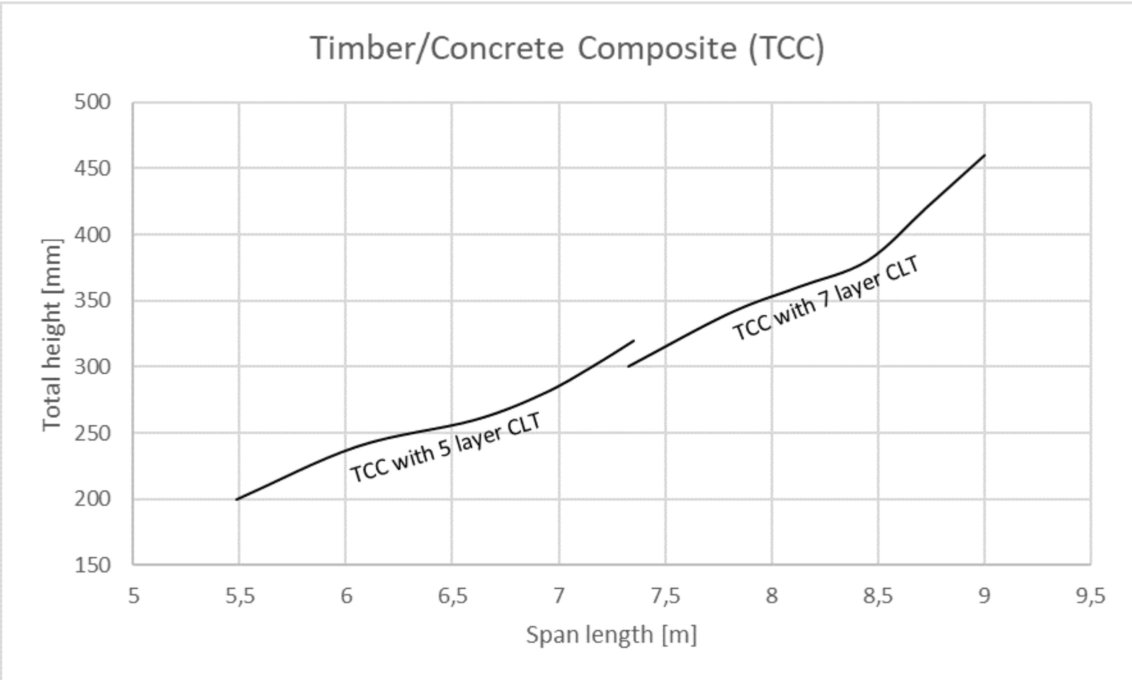

Figure 7.12: Possible cross-section heights for different span length and CLT layers. Key assumptions: Material ratio is close to 40% concrete and 60% CLT. 85% interaction assumed.

7.4 Tilting

Timber buildings are often very light structures and therefore it could be necessary to add weight in some way to prevent the building from tilting. In this section, two diagrams are presented to give a hint if the building geometry and weight resists the tilting moment from horizontal loads. The rough estimation is made by checking whether the load resultant of the vertical reaction is within one sixth of the width ($B/6$) from the centre of the building. The width is referred to as the short side of a building. The calculation is independent of the length, but this might not be true for specific building. This comes as the calculation is made for a general case as it assumes that the walls cover 5% of the floor area, the length can therefore be found in all loading terms and is therefore cancelled out in the final equation. The diagram is best suited for a residential building because there are usually less walls in an office building which means that the weight for an office building would be overestimated.

There are three different structure types in the diagram, timber, hybrid (timber-concrete composite floors) and concrete. The timber structure consists of a CLT floor plate which is $200mm$ thick and CLT walls that have a height of $3m$. Density for timber is assumed to be $480kg/m^3$. The concrete structure has the same dimensions as the timber concept, concrete plates with a thickness of $200mm$ and concrete walls that are $3m$. The density for concrete is assumed to be $2300kg/m^3$. The hybrid structure consists of composite timber/concrete floor plates (TCC) with a total height of $250mm$ where CLT is $150mm$ and concrete $100mm$. The same walls as in the timber structure is assumed for the hybrid structure.

The horizontal load is only wind load and it is calculated according to SS-EN 1991-1-4, (2005). Reference wind speed v_b is set to $25m/s$. The terrain category is assumed to be 2 which means that the area has low vegetation and is quite open. The terrain category gives the exposure factor $C_e(z)$ for different heights of the building. The corresponding velocity pressure profile is for simplification set to be evenly distributed along the height of the building. This is on the safe side since the reference height is set to the height of the building. The pressure coefficients $C_{pe,10}$ are set to be constant instead of varying depending on height to width ratio. The two coefficients used are 0.8 for pressure (zone D) and 0.7 for suction (zone E), this gives the total pressure coefficient equal to 1.5 $(0.8 + 0.7 = 1.5)$. Upward wind pressure is neglected in the calculation, which is a simplification.

There are two different diagrams, one where only the building is included (Figure 7.13) and one where a 200mm thick concrete slab foundation is included in the calculation (Figure 7.14). When using the diagram that includes a foundation it assumes that the whole building is anchored to it and that the connection is strong enough to keep the foundation and the building as one unit. In the first diagram the building still needs to be attached to the foundation to resist sliding. That connection would be of shear type and does not necessarily need to be designed for uplift as that is solved by enough weight of the building itself.

ULS load combination without any foundation 6.10 from EKS11:

$0.9 \cdot G_k,\ 1.5 \cdot Q_{k,wind}$

ULS load combination with foundation 6.10 from EKS11:

$0.9 \cdot (G_k + G_{foundation}),\ 1.5 \cdot Q_{k,wind}$

Figure 7.13: Possible building heights for different building widths and structure types. Key assumptions: Wall area: 5%. Storey height: 3 m. Floor slab thickness: 200 mm (timber and concrete), 250 mm (hybrid), No foundation.

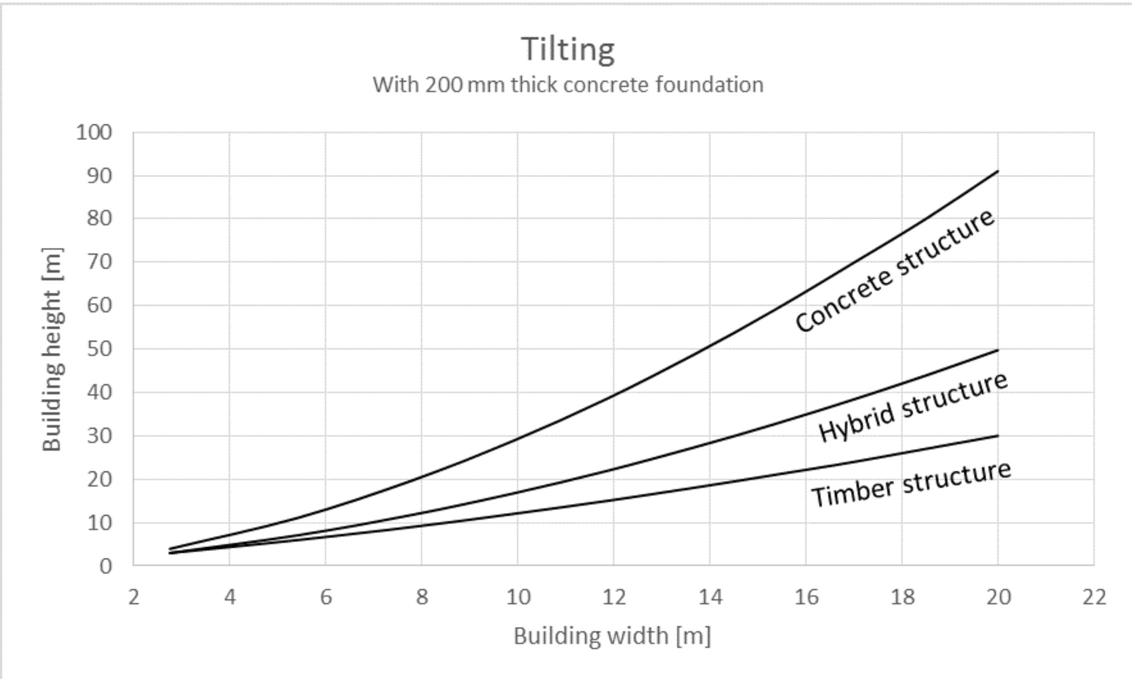

Figure 7.14: Possible building heights for different building widths and structure types. Key assumptions: Wall area: 5%. Storey height: 3 m. Floor slab thickness: 200 mm (timber and concrete), 250 mm (hybrid), Foundation: 200 mm thick concrete slab.

8
Discussion

Through the years many different engineered wood products have been developed which have increased the possibilities for building timber structures. Countries like Norway, Canada, Austria, Switzerland and Sweden goes in the front to continue pushing the development of timber construction forward. There is still much to do since the knowledge is limited of how to build mid-rise buildings in timber, nationally and globally. It has now been shown in different pilot projects that it is possible to perform efficient mid- and high-rise buildings in timber. Now it is time to spread the knowledge within the industry on how to practically implement the timber design to make it more competitive against concrete and steel.

The report has given preliminary design tools which are based on what have been presented throughout the report, from the systems available on the market, through the reference projects, important aspects and case studies. Here comes a discussion and summary where the authors give their view on where and how the different solutions are applicable in the building industry today.

8.1 Summary of the application for timber elements

8.1.1 CLT floor panels

The CLT panels are useful for shorter span lengths up to 7 meters. In standard dimensions, panels are available up to seven layers with 280 mm thickness in total thickness. The advantage with CLT is that there are several manufacturers that already today produces it and there are reference projects where the practical function can be evaluated for real buildings. Design methods and models are available and certain commercial FEM software's, such as Strusofts FEM Design, have included it as a standard element. This means that some of the uncertainties that comes with a new structure type have already been sorted out. The given drawback with CLT is that it needs to be complemented with solutions for acoustics, which means that the total thickness of the floor will usually end up somewhere between 400-500 mm. The quite short span lengths also increase the need for intermediate supports, which could affect the flexibility of the building and limit the possible areas of use.

8.1.2 CLT rib elements

To overcome some of the limitations for a pure CLT floor, composite action with timber beams (ribs) could increase the possible span lengths and create a more material efficient cross-section. KLH sets 6-10 meters as recommended span lengths, while Stora Enso includes up to 12 meters span lengths in their design tables. In the preliminary design diagram presented in Chapter 7, given the certain assumptions, up to 12 meters could be possible from a structural point of view. These extra meters in span length could for instance mean that in a residential building it might be possible to span the whole distance between the external walls without any supports in between. It could also give more flexibility in the design of office spaces where a larger structural grid would be possible. A rib element also allows for installations to be drawn in the space between the ribs. The given drawback is that the ribs automatically increases the total cross-section height and that the connection to the support will need to be carefully designed. Also, the exact way of calculating the stiffness and strength of the effective cross-section is not implemented in any standards yet, which also complicates the verification in both ULS and SLS. As far as the authors knows there is not yet any Swedish industries where these elements are in the standard assortment. So, there are some more uncertainties regarding the rib panels compared to the CLT alone. However, this might come to change in the near future as the ongoing development abroad is pushing the knowledge forward.

8.1.3 LVL rib elements

The LVL rib element is based on the same idea as the CLT-ribs but uses an LVL-panel instead of CLT as the flange. Moelvens Trä8 system is of this type and allows for up to 8 meters span length. The system is thoroughly developed and have already been used in several buildings. It is convenient as the floor modules arrive to the building site ready for mounting with a high degree of prefabrication. The drawback of the system is that it has a height of 500 mm and is limited to maximum 8 meter spans. However, in the preliminary design diagrams in Chapter 7, given the certain assumptions, it could, from a structural point of view, be possible to increase the span lengths. This is something that might be developed in the future.

8.1.4 TCC floors

The timber concrete composite floor is indeed interesting and seems to have a large potential to solve some of the main drawbacks of the other timber floor types. The concrete layer adds weight to the structure which could be favourable for both acoustics and global stability. The span lengths could reach up to 9 meters according to manufacturers and have been used in grids up to 7.6 meters in the projects studied in this report. This is in line with the calculations made for the preliminary design diagram. But the structural height gets larger in the calculations compared to what have been used in projects and what the manufacturers have come up with, which is around 300-360 mm. This could be explained by the fact that the calculations presented in this report is based on quite rough estimations and assumptions. Unfortunately, the concept is not yet implemented in a large scale but research on the behaviour of TCC is ongoing and several aspects needs to be further analysed. Effects from the difference in long term behaviour between the two materials and how to perform and design efficient shear connections are two examples that should be further analysed. Also, more standardised design models need to be developed together with clearer criteria in Eurocode for vibration and springiness. This as the mass of the concrete lowers the fundamental frequency which quickly makes the method in today's Eurocode criteria for timber not applicable. However, there are many companies and researchers that are putting effort into this and several reference projects have been built around the world, so things are moving forward. It will be interesting to see how much the system can be implemented and developed within the near future.

8.1.5 Timber-steel composites

Another type of hybrid element that has been interesting but has not been brought up so much during the report is the TSC (timber-steel composite) products. They are mostly used in roof trusses and many of the examples seems to be unique for each project, which could make them less competitive against ordinary steel trusses. However, TSC beams that has a web with holes, see Figure 2.11 in Chapter 2, could be interesting for buildings that require lots of installations. They should also be better in an acoustical perspective since the less solid structural parts in the floors would reduce structure-borne sound. These beams can according to ITW Construction Products(2020) span longer than a solid timber beams as well. The development for this type of hybrid structure is also going to be interesting to follow to see how it can be implemented in the future.

8.2 Service limit state

Referring to Chapter 7, preliminary design calculations were presented in diagrams. The CLT, Rib elements and TCC always failed in an SLS criteria with much capacity left in ULS. For the CLT plate it was deflection that governed, but for rib elements and TCC it differed depending on the span length. The deflection criterion was decisive for shorter spans while for longer spans the natural frequency got below $8Hz$ first. This is possibly restricting the rib-elements and CLT's, as the f1<8Hz criteria is not a hard criteria, as it according to the current Eurocode only states that a special investigation should be made. So, if that special investigation could verify the performance of the floor with frequencies less than 8Hz, longer spans could be achieved with less structural height.

For an upcoming revision of Eurocode, the methods on how to calculate the vibrations in timber floors is being revised, meaning that the results from this study also might need a revision. For TCC floors the current method of calculating is setting tough demands on the structure, as the concrete adds weight to the structure, which quickly lowers the fundamental frequency down beneath 8Hz. Here, calculation methods that are more applied to TCC structures could possibly widen the areas of use for TCC floors. This is also seen in the comparison to manufacturers and built projects above, were the assumptions and methods used for the diagrams gives higher cross-sections needed than what have been used in the projects, implying that detailed calculations could reduce the cross-section heights. A bit of the same issue is seen for the calculation of rib-elements, where decreasing the cc distance does not give a large impact on reducing the required cross-section height for longer span lengths when the f1 requirement is governing. This comes as the extra ribs is increasing both the mass and the stiffness.

8.3 Differences between applications for residential and office buildings

The applications of timber products will differ depending on the function of the building. Here, the authors give their view on what differences that could be seen between residential and office buildings.

In residential buildings it is common to use load bearing external walls, for which CLT could be used. Depending on the building geometry it could be favourable to span the floors directly across the external walls. For some cases, a ribbed CLT-floor could be able to reach those extra meters in span length that is needed, compared to a pure CLT-panel floor. For other projects, load bearing inner-walls around for instance corridors acts as extra supports, and decreases the span lengths to make a CLT plates, or Trä8 better suited. Columns placed at a few locations complemented with beams could also be an alternative to complement the load-bearing external walls without affecting the flexibility so much. The thickness of the floors is a main issue for residential buildings, partly as the requirements regarding acoustics is higher than for office buildings. For this purpose, TCC could also serve well.

In offices it is common with open spaces and large window areas, hence a beam-column system is often a preferred solution. The span lengths could get quite large, making CLT-ribs or TCC a good choice. In the CLT-rib, installations could be drawn between the beams but is then limited to follow the direction of the beam. The TCC floors on the other hand gives freedom to go with the installations in all directions if a suspended ceiling is used. TCC would also for this application have good possibilities to create a robust structure with a tying system and edge beams like in HoHo Vienna, described in Chapter 4.3.

In an office building, people could find it appealing with visual structural members in timber. Hence, timber columns and beams placed in the office landscape will reduce the flexibility but could by certain clients thought to be accepted. This while structural members that interferes with the function of an apartment is less likely to be accepted. This might also influence on how the fire safety design is made.

In residential buildings it will probably come naturally to use cladding like plaster boards to protect the structural members. This could apply for offices as well, but for instance visible columns and beams would however probably be designed for the R criteria with the method of reduced cross-section dependent of the charring rate.

8.4 Proposed concepts vs original concepts

It has been a challenge to go from a building already made in concrete and steel to a timber building. This proves why it is so important to already in an early stage consider different critical parts of a timber structure. One such thing is the height of each floor, this needs to be considered already in the beginning of the project. In the Traneberg residential building, there was a need of increasing the total height of the building in order to practically use timber floors. A way to keep the height as low as possible, was to make the pitched roof to a flat roof by increasing the side of the roof to the same level as the ridge. This was a step out of the reality as such changes of the facade would probably need new planning permits etc and that would delay the construction.

To reduce the floor thickness, short span lengths was necessary. This was achieved by placing two glulam beams across the apartments, also one extra wall needed to be load bearing to support the beams. It was done in a manner that did not affect the function of the building, as the beams is hidden beneath the ceiling, and the wall is between the bathroom and kitchen. Also, the balconies required some extra features, with a steel beam that supports the outer part CLT. From a durability point of view, the balconies will need a well performed design to avoid problems with moisture as they are in an outdoor environment.

The total weight of the concept was at first only 11% of the concrete alternative. Depending on the ground conditions it could be very favourable to have such a light structure. However, for the given building geometry the weight needed to increase to have global stability against tilting. So, constructing the first floor with concrete to which the six timber storeys above is attached gives enough self-weight for this.

With Gasklockan there have been more space to work with vertically, however the roof height had to be slightly increased to fit the double tapered roof beam. However, this solution gives the possibility to remove the inner row in the columns and increase the flexibility in the office spaces. The placement of the other columns was set and could not be moved, but the grid spacing gave spans that were possible to achieve with a timber structure. The total weight of the proposed concept compared to the original concept is approximately 35% which shows that a timber structure is beneficial when it comes to adding floors to an existing building.

8.5 Stability

The low self-weight of timber structures makes it important to look at the global stability in an early stage of the planning process. The two diagrams in Section 7.4 could therefore be helpful in the beginning of a project. It gives a hint of whether a certain building type has enough weight or not for the given geometry. If more weight is needed, it should be added where it is most useful, and there are many possible alternatives for this, depending on the individual project. One solution that was used for Origine Condos in Canada, Section 4.4, was to make a thick slab foundation to avoid a pile foundation. By replacing the expensive pile foundation with a thicker concrete slab, the extra weight could solve two problems at once.

In other projects, where there is no need for a larger foundation, there are other better options to put in the extra weight. This could for instance be to make the first floor and/or the elevator shaft in concrete. This was done in the project Brock Commons, see Figure 4.3 in Chapter 4. In this way, the concrete comes to use in the horizontal stabilisation of the building. It also makes the bottom floor more robust, which helps in designing for accidental loads from cars and possible moisture issues from snow. This alternative could also be used for buildings with a basement or parking garage underneath.

Other examples of good utilisation of extra weight is for high-rise buildings where sway need to be considered. This is exemplified in Mjöstornet where the seven top floors were made in concrete to add weight that changes the frequency and reduces the dynamic acceleration of the building. TCC floors is another option that possibly could have been used in Mjöstornet instead of putting all concrete in the top. TCC floors are used in the building UMASS, and also have the nice feature of increasing the acoustic performance of a floor.

8.6 Application for mid-rise vs high-rise buildings

This thesis has included study of reference projects and systems for both mid-rise and high-rise buildings. Theoretically, at the static level of the individual elements the same methods can be applied for higher buildings as in the preliminary design tools that are presented in Chapter 7. However, when looking at the whole system there are several aspects that will need special investigations when performing an even higher structure. Apart from the heavier static loading, dynamic effects will truly have an impact and needs to be carefully considered in design. This includes dynamic effects such as sway and acceleration from lateral loading. The stabilisation system also needs to resist larger loads. The importance of designing connections that are stiff increases as well, as soft connections will have a large impact on the total stiffness of a higher building. These aspects are not included in the report but remains to be specially investigated for each project. For larger timber structures, the connections can get quite complex, and expensive, and needs to be carefully designed.

9
Conclusion

The starting point of this study was to investigate the practical applications for timber in structural systems for larger buildings. This with the purpose of increase the awareness of appropriate solutions, and to give useful tools for the preliminary design in the early stage of the planning process.

The study shows that there are several interesting timber concepts and solutions on the market already today. Manufacturers both in Sweden and internationally have developed practically applicable systems, using several different techniques. Also, a number of ground breaking projects have been built around the world demonstrating the possibilities for a variety of options for timber structures. This proves that the knowledge on how to structurally perform a timber structure is well established from a technical point of view, but that the large break-through for timber as a standard alternative for larger buildings is still waiting to happen.

The practical applications for different timber products is further exemplified in the case studies where an alternative timber structure is developed for two different projects. Here, the importance of considering the timber alternative early in the planning process is demonstrated. This particularly concerns:

- The structural layout, where timber type floors could be possible for span-lengths up to 12 meters.
- The total sizes of timber elements, which tends to be larger compared to steel or concrete alternatives.
- The total weight of the building, which is shown to be significantly less than the original design. This could for certain cases be very beneficial. For other cases, extra weight could be needed for global stability and then a wise placement of this weight could solve several issues at once.

The last part of the project presents a series of preliminary sizing diagrams for timber elements. These are made to be useful in the early stage of the design process to give a hint on what type of structural dimensions that could be expected for different structural layouts. It is the authors hope that the work of this thesis will contribute to increase the knowledge on practical solutions and ease the planning for timber structures in the future.

9.1 Further research suggestions

- The sizing diagrams in this report only includes the structural parts of different floor elements. Development of similar tools for the purpose of fulfilling acoustic and fire requirements for certain standard cases would be beneficial in the early stage design.

- Only the behaviour for individual members is presented in the sizing diagrams, further research is needed on different types of connections and how these influence the total system behaviour in terms of dynamic effects and sway for higher buildings.

- Further research is needed on how to efficiently design timber-concrete composite floors in a standardised manner and how the springiness and vibration criteria of such floors could be evaluated. This could increase the possibilities and potential of using such a floor type.

- For ribbed CLT and LVL panels, estimations of the effective width from the flange is not yet in the European standard. Also, development of this floor type to better fulfil SLS requirements would increase the possible span lengths. This in particular for methods of calculating vibrations for floors with eigenfrequencies less than 8Hz.

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A

Estimation and comparison of structural self-weight

Traneberg		
Indata		
Numer of storeys	7	
Floor area	312	m2
Wall lenght	105	m
Wall height	3	m
Floor thickness	0,27	m
Wall thickness	0,16	m
Density of concrete	2500	kg/m3

With a pure timber structure (Foundation not incl.)		
Est. Structural dead weight concrete concept	2356200	kg
Est. Structural dead weight timber concept (from FEM)	267000	kg
Difference	2089200	kg
Timber vs Concrete	11,33%	

With first storey in concrete (Foundation not incl.)		
Est. Structural dead weight concrete concept	2356200	kg
Est. Structural dead weight timber concept (from FEM)	594500	kg
Difference	1761700	kg
Timber vs Concrete	25,23%	

Gasklockan		
Indata		
Area	1500	m2
Self weight of floor 265 HDF 120/27	380	kg/m2
Total weight concrete	570000	kg

Steel profile	Total length [m]	Weight/m [kg/m]	Weight [kg]
HEA 260	120	68,2	8184
HEA 300	78	88,3	6887,4
HEA 340	140	105	14700
VKR 100x100x6.3	116	18,2	2111,2
VKR 150x150x6.3	58	28,1	1629,8
IPE 180	193,1	18,8	3630,28
IPE 200	25	22,4	560
Roof truss	145	48	6960
Total weight steel			44662,68

Est. Structural dead weight steel/concrete concept	614663	kg
Est. Structural dead weight timber concept (from FEM)	191000	kg
Difference	423663	kg
Timber vs concrete/steel	31,07%	

B

Load combinations used in FEM

Combination	Limit state	Factor	Load type
ULS 6.10a	U	1.35	Dead weight
		1.35	Extra dead weight
		0.00	Imposed load
		0.00	Snow load
		0.00	Wind X+
ULS 6.10b - Imposed load - Wind X+	U	1.20	Dead weight
		1.20	Extra dead weight
		1.50	Imposed load
		1.05	Snow load
		0.45	Wind X+
ULS 6.10b - Imposed load - Wind X-	U	1.20	Dead weight
		1.20	Extra dead weight
		1.50	Imposed load
		1.05	Snow load
		0.45	Wind X-
ULS 6.10b - Imposed load - Wind Y+	U	1.20	Dead weight
		1.20	Extra dead weight
		1.50	Imposed load
		1.05	Snow load
		0.45	Wind Y+
ULS 6.10b - Imposed load - Wind Y-	U	1.20	Dead weight
		1.20	Extra dead weight
		1.50	Imposed load
		1.05	Snow load
		0.45	Wind Y-
ULS 6.10b - Snow load - Wind X+	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.50	Snow load
		0.45	Wind X+
ULS 6.10b - Snow load - Wind X-	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.50	Snow load
		0.45	Wind X-
ULS 6.10b - Snow load - Wind Y+	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.50	Snow load
		0.45	Wind Y+
ULS 6.10b - Snow load - Wind Y-	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.50	Snow load
		0.45	Wind Y-
ULS 6.10b - Wind X+	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.05	Snow load

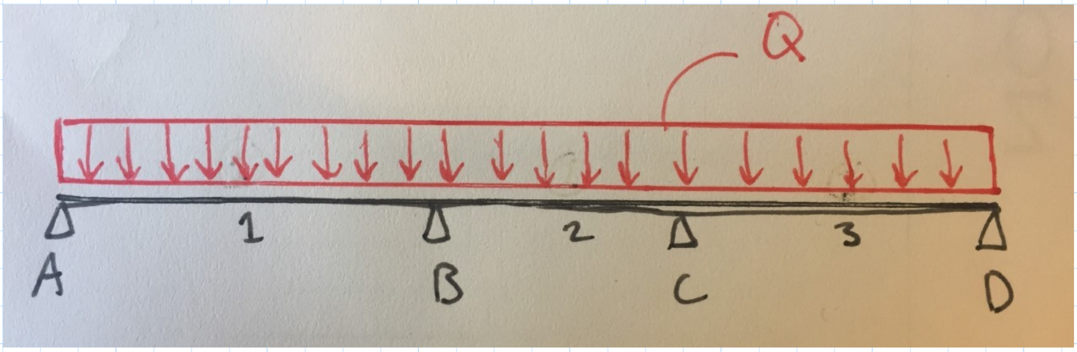
		1.50	Wind X+
ULS 6.10b - Wind X-	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.05	Snow load
		1.50	Wind X-
ULS 6.10b - Wind Y+	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.05	Snow load
		1.50	Wind Y+
ULS 6.10b - Wind Y-	U	1.20	Dead weight
		1.20	Extra dead weight
		1.05	Imposed load
		1.05	Snow load
		1.50	Wind Y-
SLS 6.14b - Imposed load - Wind X+	Sc	1.00	Dead weight
		1.00	Extra dead weight
		1.00	Imposed load
		0.70	Snow load
		0.30	Wind X+
SLS 6.14b - Imposed load - Wind X-	Sc	1.00	Dead weight
		1.00	Extra dead weight
		1.00	Imposed load
		0.70	Snow load
		0.30	Wind X-
SLS 6.14b - Imposed load - Wind Y+	Sc	1.00	Dead weight
		1.00	Extra dead weight
		1.00	Imposed load
		0.70	Snow load
		0.30	Wind Y+
SLS 6.14b - Imposed load - Wind Y-	Sc	1.00	Dead weight
		1.00	Extra dead weight
		1.00	Imposed load
		0.70	Snow load
		0.30	Wind Y-
SLS 6.14b - Snow load - Wind X+	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		1.00	Snow load
		0.30	Wind X+
SLS 6.14b - Snow load - Wind X-	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		1.00	Snow load
		0.30	Wind X-
SLS 6.14b - Snow load - Wind Y+	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		1.00	Snow load

		0.30	Wind Y+
SLS 6.14b - Snow load - Wind Y-	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		1.00	Snow load
		0.30	Wind Y-
SLS 6.14b - Wind X+	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		0.70	Snow load
		1.00	Wind X+
SLS 6.14b - Wind X-	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		0.70	Snow load
		1.00	Wind X-
SLS 6.14b - Wind Y+	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		0.70	Snow load
		1.00	Wind Y+
SLS 6.14b - Wind Y-	Sc	1.00	Dead weight
		1.00	Extra dead weight
		0.70	Imposed load
		0.70	Snow load
		1.00	Wind Y-
SLS 6.15b - Imposed load	Sf	1.00	Dead weight
		1.00	Extra dead weight
		0.50	Imposed load
		0.20	Snow load
SLS 6.15b - Snow load	Sf	1.00	Dead weight
		1.00	Extra dead weight
		0.30	Imposed load
		0.40	Snow load
SLS 6.15b - Wind X+	Sf	1.00	Dead weight
		1.00	Extra dead weight
		0.30	Imposed load
		0.30	Snow load
		0.20	Wind X+
SLS 6.15b - Wind X-	Sf	1.00	Dead weight
		1.00	Extra dead weight
		0.30	Imposed load
		0.30	Snow load
		0.20	Wind X-
SLS 6.15b - Wind Y+	Sf	1.00	Dead weight
		1.00	Extra dead weight
		0.30	Imposed load
		0.30	Snow load
		0.20	Wind Y+
SLS 6.15b - Wind Y-	Sf	1.00	Dead weight

		1.00	Extra dead weight
		0.30	Imposed load
		0.30	Snow load
		0.20	Wind Y-
SLS 6.16b	Sq	1.00	Dead weight
		1.00	Extra dead weight
		0.30	Imposed load
		0.20	Snow load

C
Hand calculations Traneberg

Hand calculations of roof beam



Loads

ULS 6.10b - Imposed load - Wind Y+

U		
	1.20	Dead weight
	1.20	Extra dead weight
	1.50	Imposed load
	1.05	Snow load
	0.45	Wind Y+

$\rho := 480 \, \frac{kg}{m^3}$ Density timber

$b := 215 \; mm \qquad h := 315 \; mm$ Beam dimesions

$Gbeam := b \cdot h \cdot \rho \cdot g = 0.32 \, \frac{kN}{m}$

$hclt := 160 \; mm$

$cc := 3 \; m$

$G.Clt := hclt \cdot cc \cdot \rho \cdot g = 2.26 \, \frac{kN}{m}$

$G.extra := 0.5 \, \frac{kN}{m^2} \cdot cc = 1.5 \, \frac{kN}{m}$

$G := Gbeam + G.Clt + G.extra = 4.08 \, \frac{kN}{m}$ Total dead load

$q := 2 \, \frac{kN}{m^2} \cdot cc = 6 \, \frac{kN}{m}$ Imposed load

$S := 1.6 \, \frac{kN}{m^2} \cdot cc = 4.8 \, \frac{kN}{m}$ Snow load

$Q := 1.2 \cdot G + 1.5 \cdot q + 1.05 \cdot S = 18.93 \, \frac{kN}{m}$ Load combination

$L1 := 4.26 \; m$ Span length 1

$L2 := 2.6 \; m$ Span length 2

$L3 := 3.85 \; m$ Span length 3

Equation to find the support moment over C, equation are derived from elementary case.

$$M := \left(\frac{Q}{24} \cdot \left(L2^3 + L3^3 \right) - \frac{\boxed{Mc}}{3} \cdot \left(L2 + L3 \right) \right) \cdot \frac{6 \cdot (L1 + L2)}{3 \cdot L2} + \frac{Mc \cdot L2}{6} - \frac{Q}{24} \cdot \left(L1^3 + L2^3 \right) = 0$$

$M \xrightarrow{solve, Mc} \frac{21.616805998246432461 \cdot m^2 \cdot kN}{m}$ Solving moment over support C

$Mc := 21.6\ kN \cdot m$ Support moment over C (Negative in FEM-design)

$Mb := \left(\frac{Q}{24} \cdot \left(L2^3 + L3^3\right) - \frac{Mc}{3} \cdot (L2 + L3)\right) \cdot \frac{6}{L2} = 28.72\ kN \cdot m$ Support moment over B (Negative in FEM-design)

$Ra := \frac{-Mb}{L1} + \frac{Q \cdot L1}{2} = 33.59\ kN$ Reaction force in A

$x := 1.8\ m$ Iteration to find maximum moment in span 1

$Mx := Q \cdot \frac{x^2}{2} - Ra \cdot x = -29.78\ kN \cdot m$ Field moment in part 1 (Positive in FEM)

$r1 := \frac{Mb}{L1} + Q \cdot \frac{L1}{2} = 47.07\ kN$ Reaction from part 1

$r2 := \frac{Mb}{L2} - \frac{Mc}{L2} + Q \cdot \frac{L2}{2} = 27.35\ kN$ Reaction from part 2

$Rb := r1 + r2 = 74.43\ kN$ Reaction force at B

$x2 := 5.56\ m$ Iteration to find maximum moment in span 2

$Mx2 := Q \cdot \frac{x2^2}{2} - Ra \cdot x2 - Rb \cdot (x2 - L1) = 9.16\ kN \cdot m$ Field moment in span 2, Note it is a negative moment

Indata

Laster		
q_k	2	kN/m^2
g_{kextra}	0,5	kN/m^2
g_{ktot}	1,579	kN/m^2

Rumsgeometri		
Upplag	Fritt upplagd	
l	4,474	m
l_{ref}	4,474	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	12,2	kNm/m
V_{Ed}	10,9	kN/m

Materialdata		
Materialtyp	Konstruktionsvirke	-
$f_{m,k,xlay}$	24	MPa
$f_{v,k,xlay}$	3	MPa
$f_{r,k,ylay}$	2	MPa
$E_{mean,xlay}$	11	GPa
$E_{mean,ylay}$	11	GPa
$G_{090,xlay}$	690	MPa
$G_{9090,ylay}$	50	MPa
ρ	687,5	kg/m^3
γ_m	1,25	-
Klimatklass	1	-
Lastvaraktighet	Medellång	-
k_{mod}	0,9	-
k_{def}	0,85	-
k_{sys}	1,0	-
ψ_2	0,3	-

Plattans uppbyggnad		
Antal lager		5
t1	40	mm
t2	20	mm
t3	40	mm
t4	20	mm
t5	40	mm

ULS Beräkningar

Momentkapacitet		
σ_{Myd}	3,2	MPa
$f_{m,xlay,d}$	17,28	MPa
M_{Rd}	65,7	kNm/m
Kontroll momentkapacitet		OK
Utnyttjandegrad		18,65%

Tvärkraftskapacitet		
τ_{vd}	0,094	MPa
τ_{rd}	0,086	MPa
f_{vd}	2,16	MPa
f_{rd}	1,44	MPa
V_{vRd}	252,6	kN/m
V_{rRd}	182,4	kN/m
Kontroll parallellskjuvning		OK
Utnyttjandegrad		4,34%
Kontroll rullskjuvning		OK
Utnyttjandegrad		6,00%

Tvärsnittsegenskaper		
t_{tot}	160	mm
I_{xnet}	30400	cm^4/m
I_{ynet}	3733	cm^4/m
S_{xnet}	2600	cm^3/m
S_{Rxnet}	2400	cm^3/m
W_{xnet}	3800	cm^3/m
I_{xef}	28100	cm^4/m
GA_{ef}	1,57E+07	N
k	1,2	-

SLS Beräkningar

Nedböjning krav		
Initiell krav: L/		300
Slutlig krav: L/		300
w_{inst}	14,9	mm
w_{fin}	14,9	mm

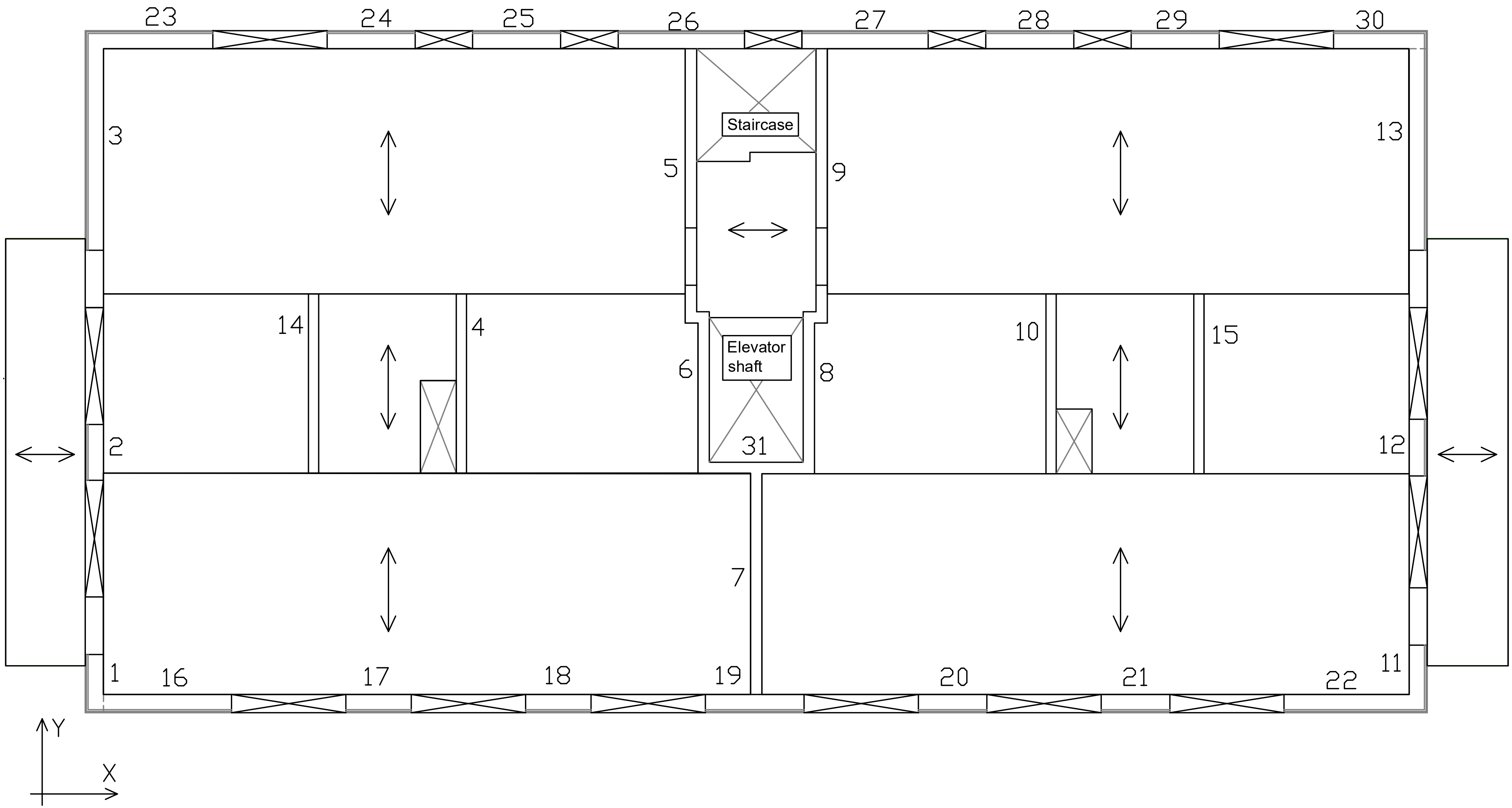
Beräknad nedböjning enligt SAV		
w_{ginst}	2,8	mm
w_{qinst}	3,5	mm
w_{gfin}	5,1	mm
w_{qfin}	4,4	mm
$w_{insttot}$	6,3	mm
w_{fintot}	9,5	mm
Kontroll initiell nedböjning		OK
Utnyttjandegrad		42,03%
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		63,77%

SLS Beräkningar

Vibration och svikt		
Bredd	4,5	m
m	110	kg/m2
f1	13,2	Hz
n40	2,88	st
ν	0,004	m/Ns2
b	100	m/Ns2
ξ	0,01	-
v_{max}	0,018	m/Ns2
a	1,5	mm/kN
w_{inst}	0,64	mm/kN
Kontroll f1		OK
Kontroll vibrationer		OK
Utnyttjandegrad		19,23%
Kontroll svikt		OK
Utnyttjandegrad		42,89%

Beräkningen är baserad på metoder från svenskt trä KL-trähandbok

Beräkning förutsätter samma virkeskvalitet för alla lager i huvudbärriktningen. Gäller endast för symmetriskt tvärsnitt.
Beräkningen görs enligt balkteori där endast bärförmåga räknas för lager i huvudbärriktningen, och övriga lager försummas.
Fullständig interaktion mellan lager förutsätts i ULS. Deformationsberäkning görs enligt gamma-metoden föreslagen i SS-EN 1995-1-1 bilaga B men gäller endast upp till 5 lager. Kompletterande beräkning görs också enl. SAV metoden


23
24
25
26
27
28
29
30
3
13
5
9
Staircase
14
4
Elevator
shaft
6
8
10
15
2
31
12
7
1
16
17
18
19
20
21
22
11
Y
X

Wind loads with unintended Inclination Traneberg

INDATA:

Unintended Inclination		
A_floor	240	m^2
A_balconies	9	m^2
Floor	368	kg/m^2
Gfloor	1000	N/m^2
Snowload	1500	N/m^2
Imposedres	2000	N/m^2
Imposedbalc	3500	N/m^3
Gd	1170,0	KN
Qresidental	1199,7	KN
phi0	0,005	
alphah	0,667	
alpham	0,726	
phii	0,002422	

Calculation with wind on longside:

Longside						
#	Hv [KN]	height	length	We	Qw [KN]	Qh [KN]
0	0	1,5	23	1,35	34,93	34,93
1	2,91	3	23	1,35	139,73	142,63
2	2,91	3	23	1,35	139,73	142,63
3	2,91	3	23	1,35	139,73	142,63
4	2,91	3	23	1,35	139,73	142,63
5	2,91	3	23	1,35	139,73	142,63
6	2,91	3	23	1,35	139,73	142,63
7	1,42	2	23	1,35	46,58	47,99
					Sum	903,77

Calculation with wind on shortside:

Shortside						
#	**Hv [KN]**	**height**	**length**	**We**	**Qw [KN]**	**Qh [KN]**
0	0	1,5	11,85	1,3	17,33	17,33
1	2,91	3	11,85	1,3	69,32	72,23
2	2,91	3	11,85	1,3	69,32	72,23
3	2,91	3	11,85	1,3	69,32	72,23
4	2,91	3	11,85	1,3	69,32	72,23
5	2,91	3	11,85	1,3	69,32	72,23
6	2,91	3	11,85	1,3	69,32	72,23
7	1,42	2	11,85	1,3	23,11	24,52
					Sum	457,89

Parameter explanation:

Hv = Horizontal load due to unintended inclination

We = External pressure coefficients (Pressure + Suction)

Qw = Wind load

Qh = Hv + Qw

Load distribution to each wall

Load distribution on each wall - Wind acting on longside

	#	b	h	Moment of inertia		Stiffness ratio	
Walls Y-direction	1	0,2	0,7	I1	0,01	S1	1,00
	2	0,2	0,95	I2	0,01	S2	2,50
	3	0,2	3,54	I3	0,74	S3	129,33
	4	0,2	3,1	I4	0,50	S4	86,85
	5	0,2	3,1	I5	0,50	S5	86,85
	6	0,2	2,7	I6	0,33	S6	57,38
	7	0,2	3,88	I7	0,97	S7	170,29
	8	0,2	3,1	I8	0,50	S8	86,85
	9	0,2	2,7	I9	0,33	S9	57,38
	10	0,2	3,1	I10	0,50	S10	86,85
	11	0,2	0,7	I11	0,01	S11	1,00
	12	0,2	0,95	I12	0,01	S12	2,50
	13	0,2	3,54	I13	0,74	S13	129,33
	14	0,2	3,1	I14	0,50	S14	86,85
	15	0,2	3,1	I15	0,50	S15	86,85

	#	b	h	Moment of inertia		Stiffness ratio	
Walls X-direction	16	0,2	2,2	I16	0,18	S16	31,04
	17	0,2	1,1	I17	0,02	S17	3,88
	18	0,2	1,1	I18	0,02	S18	3,88
	19	0,2	1,7	I19	0,08	S19	14,32
	20	0,2	1,3	I20	0,04	S20	6,41
	21	0,2	1,2	I21	0,03	S21	5,04
	22	0,2	2,2	I22	0,18	S22	31,04
	23	0,2	1,9	I23	0,11	S23	20,00
	24	0,2	1,5	I24	0,06	S24	9,84
	25	0,2	1,55	I25	0,06	S25	10,86
	26	0,2	2,2	I26	0,18	S26	31,04
	27	0,2	2,2	I27	0,18	S27	31,04
	28	0,2	1,55	I28	0,06	S28	10,86
	29	0,2	1,55	I29	0,06	S29	10,86
	30	0,2	1,3	I30	0,04	S30	6,41
	31	0,2	1,65	I31	0,07	S31	13,10

Rotation center		
xt	11,78	
yt	6,70	
ex	-0,28	
Hy	905,00	[KN]
T	-253,1	[KNm]
ST	58826,51	

Dimension of House	
x	23
y	11,85

Bracing units

WALLS in Y-direction

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
ai	0,1	0,1	0,1	6,6	10,7	10,8	11,8	12,9	12,7	17,0	23,4	23,4	23,4	4,1	19,5
bi	0,4	4,4	9,7	5,4	9,9	5,4	1,9	9,9	5,4	5,4	0,4	4,4	9,7	5,4	5,4
Sxi	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
Syi	1,0	2,5	129,3	86,9	86,9	57,4	170,3	86,9	57,4	86,9	1,0	2,5	129,3	86,9	86,9
ai*Syi	0,1	0,2	12,9	575,0	925,0	620,9	2009,5	1124,3	730,2	1476,5	23,4	58,4	3020,0	356,1	1693,7
bi*Sxi	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
xi	-11,7	-11,7	-11,7	-5,2	-1,1	-1,0	0,0	1,2	0,9	5,2	11,6	11,6	11,6	-7,7	7,7
yi	-6,3	-2,3	3,0	-1,3	3,2	-1,3	-4,8	3,2	-1,3	-1,3	-6,3	-2,3	3,0	-1,3	-1,3
Syi*xi^2	136,4	341,0	17643,2	2312,3	110,8	52,9	0,1	117,9	51,3	2366,9	133,9	334,6	17314,4	5122,4	5176,8
Sxi*yi^2	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
HiBy	0,8	2,1	109,2	73,3	73,3	48,5	143,8	73,3	48,5	73,3	0,8	2,1	109,2	73,3	73,3
HiCx	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
HiCy	0,1	0,1	6,5	1,9	0,4	0,2	0,0	-0,4	-0,2	-2,0	0,0	-0,1	-6,4	2,9	-2,9
Hix	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
Hiy	0,9	2,2	115,7	75,3	73,8	48,7	143,8	72,9	48,2	71,4	0,8	2,0	102,8	76,2	70,4

Bracing units

WALLS in X-direction

	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Sum
ai	1,3	5,1	8,3	11,8	15,3	18,5	22,2	1,2	5,0	7,6	10,5	13,7	16,6	19,2	22,6	11,7	
bi	0,1	0,1	0,1	0,1	0,1	0,1	0,1	11,8	11,8	11,8	11,8	11,8	11,8	11,8	11,8	4,2	
Sxi	31,0	3,9	3,9	14,3	6,4	5,0	31,0	20,0	9,8	10,9	31,0	31,0	10,9	10,9	6,4	13,1	239,61
Syi	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	1071,86
ai*Syi	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	12626,15
bi*Sxi	3,1	0,4	0,4	1,4	0,6	0,5	3,1	235,4	115,8	127,8	365,4	365,4	127,8	127,8	75,4	55,0	1605,26
xi	-10,5	-6,6	-3,5	0,0	3,5	6,7	10,4	-10,6	-6,8	-4,2	-1,3	1,9	4,8	7,4	10,8	-0,1	
yi	-6,6	-6,6	-6,6	-6,6	-6,6	-6,6	-6,6	5,1	5,1	5,1	5,1	5,1	5,1	5,1	5,1	-2,5	
Syi*xi^2	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	51214,90
Sxi*yi^2	1352,0	169,0	169,0	623,8	279,0	219,4	1352,0	514,1	253,0	279,1	798,2	798,2	279,1	279,1	164,7	81,8	7611,62
HiBy	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	905,00
HiCx	0,9	0,1	0,1	0,4	0,2	0,1	0,9	-0,4	-0,2	-0,2	-0,7	-0,7	-0,2	-0,2	-0,1	0,1	0,00
HiCy	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,00
Hix	0,9	0,1	0,1	0,4	0,2	0,1	0,9	-0,4	-0,2	-0,2	-0,7	-0,7	-0,2	-0,2	-0,1	0,1	0,00
Hiy	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	905,00

Dimensioning wall are number 7 at 143,8 kN

Load distribution to each wall

Load distribution on each wall - Wind acting on shortside

	#	b	h	Moment of inertia		Stiffness ratio	
Walls Y-direction	1	0,2	0,7	I1	0,01	S1	1,00
	2	0,2	0,95	I2	0,01	S2	2,50
	3	0,2	3,54	I3	0,74	S3	129,33
	4	0,2	3,1	I4	0,50	S4	86,85
	5	0,2	3,1	I5	0,50	S5	86,85
	6	0,2	2,7	I6	0,33	S6	57,38
	7	0,2	3,88	I7	0,97	S7	170,29
	8	0,2	3,1	I8	0,50	S8	86,85
	9	0,2	2,7	I9	0,33	S9	57,38
	10	0,2	3,1	I10	0,50	S10	86,85
	11	0,2	0,7	I11	0,01	S11	1,00
	12	0,2	0,95	I12	0,01	S12	2,50
	13	0,2	3,54	I13	0,74	S13	129,33
	14	0,2	3,1	I14	0,50	S14	86,85
	15	0,2	3,1	I15	0,50	S15	86,85

	#	b	h	Moment of inertia		Stiffness ratio	
Walls X-direction	16	0,2	2,2	I16	0,18	S16	31,04
	17	0,2	1,1	I17	0,02	S17	3,88
	18	0,2	1,1	I18	0,02	S18	3,88
	19	0,2	1,7	I19	0,08	S19	14,32
	20	0,2	1,3	I20	0,04	S20	6,41
	21	0,2	1,2	I21	0,03	S21	5,04
	22	0,2	2,2	I22	0,18	S22	31,04
	23	0,2	1,9	I23	0,11	S23	20,00
	24	0,2	1,5	I24	0,06	S24	9,84
	25	0,2	1,55	I25	0,06	S25	10,86
	26	0,2	2,2	I26	0,18	S26	31,04
	27	0,2	2,2	I27	0,18	S27	31,04
	28	0,2	1,55	I28	0,06	S28	10,86
	29	0,2	1,55	I29	0,06	S29	10,86
	30	0,2	1,3	I30	0,04	S30	6,41
	31	0,2	1,65	I31	0,07	S31	13,10

Rotation center		
xt	5,03	
yt	11,72	
ex	0,90	
Hy	460,00	[KN]
T	412,4	[KNm]
ST	58752,52	

Dimension of House	
x	11,85
y	23

Bracing units															
	Walls Y-direction														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
ai	11,5	7,4	1,8	6,2	1,6	6,1	9,7	1,6	6,1	6,2	11,5	7,4	1,8	6,2	6,2
bi	0,1	0,1	0,1	6,6	10,6	10,9	11,6	12,9	12,7	16,9	23,3	23,3	23,3	4,1	19,5
Sxi	1,0	2,5	129,3	86,9	86,9	57,4	170,3	86,9	57,4	86,9	1,0	2,5	129,3	86,9	86,9
Syi	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
ai*Syi	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
bi*Sxi	0,1	0,2	12,9	568,9	916,3	622,6	1975,4	1116,1	727,4	1471,3	23,3	58,3	3018,7	356,1	1693,7
xi	6,4	2,3	-3,3	1,2	-3,5	1,1	4,7	-3,5	1,1	1,2	6,4	2,3	-3,3	1,2	1,2
yi	-11,6	-11,6	-11,6	-5,2	-1,2	-0,9	-0,1	1,1	1,0	5,2	11,6	11,6	11,6	-7,6	7,8
Syi*xi^2	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
Sxi*yi^2	135,0	337,5	17461,1	2320,9	118,7	43,4	2,4	111,0	52,4	2367,3	135,0	337,6	17465,6	5042,2	5258,1
HiBy	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
HiCx	-0,1	-0,2	-10,5	-3,2	-0,7	-0,4	-0,1	0,7	0,4	3,2	0,1	0,2	10,6	-4,6	4,7
HiCy	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
Hix	-0,1	-0,2	-10,5	-3,2	-0,7	-0,4	-0,1	0,7	0,4	3,2	0,1	0,2	10,6	-4,6	4,7
Hiy	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0

Bracing units																	
	Walls X-direction																
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Sum
ai	11,8	11,8	11,8	11,8	11,8	11,8	11,8	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1	4,9	
bi	1,3	5,1	8,3	11,8	15,3	18,5	22,2	1,2	5,0	7,6	10,5	13,7	16,6	19,2	22,6	11,7	
Sxi	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1071,9
Syi	31,0	3,9	3,9	14,3	6,4	5,0	31,0	20,0	9,8	10,9	31,0	31,0	10,9	10,9	6,4	13,1	239,6
ai*Syi	366,3	45,8	45,8	169,0	75,6	59,4	366,3	2,0	1,0	1,1	3,1	3,1	1,1	1,1	0,6	63,5	1204,9
bi*Sxi	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	12561,4
xi	6,8	6,8	6,8	6,8	6,8	6,8	6,8	-4,9	-4,9	-4,9	-4,9	-4,9	-4,9	-4,9	-4,9	-0,2	
yi	-10,4	-6,6	-3,4	0,0	3,6	6,7	10,5	-10,6	-6,7	-4,1	-1,3	2,0	4,9	7,5	10,9	0,0	
Syi*xi^2	1423,5	177,9	177,9	656,8	293,7	231,0	1423,5	485,7	239,0	263,7	754,0	754,0	263,7	263,7	155,6	0,4	7564,2
Sxi*yi^2	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	51188,3
HiBy	59,6	7,4	7,4	27,5	12,3	9,7	59,6	38,4	18,9	20,8	59,6	59,6	20,8	20,8	12,3	25,1	460,0
HiCx	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,00
HiCy	1,5	0,2	0,2	0,7	0,3	0,2	1,5	-0,7	-0,3	-0,4	-1,1	-1,1	-0,4	-0,4	-0,2	0,0	0,00
Hix	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,00
Hiy	61,1	7,6	7,6	28,2	12,6	9,9	61,1	37,7	18,5	20,5	58,5	58,5	20,5	20,5	12,1	25,1	460,0

Dimensioning wall are number 16 and 22 at 61,1 kN

Check of CLT shear wall

Indata

Geometri		
h	3	m
b	3,88	m

Dimensionerande lasteffekter brottgräns		
V_{Ed}	145	kN
$V_{stående}$	145	kN
$V_{liggande}$	112,1134021	kN
M_{td}	217,5	kNm

Materialdata		
$f_{vk,090,stående}$	3,5	MPa
$f_{vk,090,liggande}$	3,5	MPa
$f_{mzk,9090}$	0,7	MPa
γ_m	1,25	-
Klimatklass	1	-
Lastvaraktighet	Medellång	-
k_{mod}	0,8	-

Väggens uppbyggnad		
Antal lager		5
$b_{xlamell}$	80	mm
$b_{ylamell}$	80	mm
t1	40	mm
t2	30	mm
t3	40	mm
t4	30	mm
t5	40	mm
	180	

ULS Beräkningar

Tvärsnittsegenskaper		
t_{tot}	180	mm
$A_{stående}$	465600	mm^2
$A_{liggande}$	180000	mm^2

Panelskjuvning (Kontroll av hela plattan)		
$\tau_{stående}$	0,31	MPa
$\tau_{liggande}$	0,62	MPa
$f_{vd,090,stående}$	2,24	MPa
$f_{vd,090,liggande}$	2,24	MPa
Kontroll skjuvning tvärs stående skikt		OK
Utnyttjandegrad		13,90%
Kontroll skjuvning tvärs liggande skikt		OK
Utnyttjandegrad		27,81%

Skjuvning mellan skikten		
I_p	6826666,667	mm^4
W_p	170666,6667	mm^3
n_t	7275	st
τ_{mzd}	0,175177191	MPa
$f_{mzd,9090}$	0,448	MPa
Kontroll av skjuvning mellan skikten		OK
Utnyttjandgrad		39,10%

Global stability against tilting

This method is a rough estimation to ensure the global stability against tilting of a structure. If the resultant of stresses in the bottom of the building is within 1/6 from the center of the building it means that no part is subjected to uplift and hence it is considered stable. The self-weight of the structure is favourable and counteracts the moment created by wind and unintended inclination.

Geometry

$H := 22\ m$	Height of building
$L := 24\ m$	Length of building
$B := 12\ m$	Width of building
$A := L \cdot B = 288\ m^2$	Area of foundation
$I := \frac{L \cdot B^3}{12} = \left(3.456 \cdot 10^3\right)\ m^4$	Moment of inertia foundation plate

Selfweight of structure (without including any foundation)

$W_{steel} := 6032\ kg$	Self-weight for steel parts (from FEM-model)
$W_{timber} := 238000\ kg$	Self-weight for timber parts (from FEM-model)

Horsontal loads

$q_{kwind} := 0.8 \cdot (0.8 + 0.55)\ \frac{kN}{m^2} = 1.08\ \frac{kN}{m^2}$	Characteristic windload
$\alpha_0 := 0.003$	Systematic part of inclination angle
$\alpha_d := 0.012$	Random part of inclination angle
$n := 2$	Number of walls perpendicular to wind load on each floor
$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt[2]{n}} = 0.011$	Unintended inclination angle
$F_{hk} := \alpha_{md} \cdot \left(W_{timber} \cdot g + W_{steel} \cdot g\right) = 27.486\ kN$	Horisontal force due to unintended inclination

Design loads

$G_d := 0.9 \cdot \left(W_{timber} \cdot g + W_{steel} \cdot g\right) = 2.154\ MN$	Design vertical selfweight (Favourable)

$Q_d := 1.5 \cdot (q_{kwind} \cdot H \cdot L + F_{hk}) = 896.589\ kN$ Design horisontal load (Unfavourable)

$Med := Q_d \cdot \frac{H}{2} = 9.862\ MN \cdot m$ Overturning moment from horisontal loads

$M_{rd} := G_d \cdot \frac{B}{6} = 4.308\ MN \cdot m$ Resisting moment from selfweight with a lever arm of B/6

$U := \frac{Med}{M_{rd}} = 2.29$ Utilisation, global stability not fulfilled. Structure is too lightweight!

Stresses in foundation

$\sigma := \frac{G_d}{A} - \frac{Med}{I} \cdot \frac{B}{2} = -9.644\ kPa$ Another way of checking the same thing, there is uplift in part of the building

So how much total weight is needed for the given building geometry and wind loads?

$$G_{needed} := \frac{Med}{\frac{B}{6}} \cdot \frac{1}{0.9 \cdot g} = \left(5.587 \cdot 10^5\right)\ kg$$

$$G_{extra} := G_{needed} - W_{timber} - W_{steel} = \left(3.147 \cdot 10^5\right)\ kg$$

4 alternatives on how to place this weight:

$$\rho_{concrete} := 2500\ \frac{kg}{m^3}$$

1. In the foundation, 45cm thick concrete slab

$$W_{slab} := 45\ cm \cdot A \cdot \rho_{concrete} = \left(3.24 \cdot 10^5\right)\ kg$$

2. In the first storey of the building, with a 150 mm foundation slab and 270mm 1st storey floor. 200 mm walls.

$L_{exwalls} := 2 \cdot B + 2 \cdot L = 72\ m$ $L_{intwalls} := 33\ m$ $H_{walls} := 3.2\ m$

$t_{slab} := 150\ mm$ $t_{floor} := 270\ mm$ $t_{walls} := 200\ mm$

$$W_{lost} := \frac{1}{7} \cdot \left(W_{timber} + W_{steel}\right) = \left(3.486 \cdot 10^{4}\right)\ kg$$

1/7 of the total timber weight assumed to be lost when replacing first storey with concrete

Concrete volume added

$$V_{concrete} := t_{walls} \cdot H_{walls} \cdot \left(L_{exwalls} + L_{intwalls}\right) + A \cdot \left(t_{slab} + t_{floor}\right) = 188.16\ m^{3}$$

$$W_{1ststorey} := V_{concrete} \cdot \rho_{concrete} - W_{lost} = \left(4.355 \cdot 10^{5}\right)\ kg$$

3. TCC floors - Use timber concrete composite as floors, assume 100 mm added concrete to each floor

$$W_{TCC} := 100\ mm \cdot A \cdot 6 \cdot \rho_{concrete} = \left(4.32 \cdot 10^{5}\right)\ kg$$

4. Make elevator shaft and dividing inner wall with concrete. 200 mm thick wall assumed. Plus a 150 mm foundation slab

$t_{elevator} := 220\ mm$ $\quad L_{elevator} := 21\ m$ $\quad t_{slab4} := 150\ mm$

$$W_{elevator} := \left(L_{elevator} \cdot t_{elevator} \cdot 3\ m \cdot 7 + t_{slab4} \cdot A\right) \cdot \rho_{concrete} = \left(3.506 \cdot 10^{5}\right)\ kg$$

D
FEM calculations Traneberg

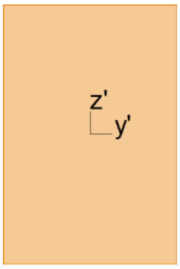
ROOF BEAM (Beam.27.1) Compared with hand calculation
Maximum of load combinations

GL 30c

(Glued laminated), Service class 1

$E_{0,05}$	=	10800	N/mm^2	$f_{t,90,k}$	=	0.50	N/mm^2
$G_{0,05}$	=	540	N/mm^2	$f_{c,0,k}$	=	24.50	N/mm^2
γ_M	=	1.25		$f_{c,90,k}$	=	2.50	N/mm^2
$\gamma_{M,acc./seis.}$	=	1.00		$f_{v,k}$	=	3.50	N/mm^2
k_{sys}	=	1.00					

Glulam 215x315



A	=	67725	mm^2	$f_{t,0,k}$	=	20.80	N/mm^2
W_1	=	3.556e+06	mm^3	$f_{m,1,k}$	=	32.00	N/mm^2
W_2	=	2.427e+06	mm^3	$f_{m,2,k}$	=	33.00	N/mm^2
i_1	=	91	mm				
i_2	=	62	mm				
I_2	=	2.609e+08	mm^4				
I_t	=	6.035e+08	mm^4				

Combined bending and axial tension - 6.2.3

LC: $'\text{ULS 6.10b} - \text{NL 1} - \text{Vind Y}+'$, $k_{mod} = 0.90$, $x = 1893.32$ mm

$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.02}{14.97}+\frac{8.78}{23.04}+0.70\frac{0.01}{23.76}=0.38\leq1.00$ (6.17) - OK

$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+k_m\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.02}{14.97}+0.70\frac{8.78}{23.04}+\frac{0.01}{23.76}=0.27\leq1.00$ (6.18) - OK

Combined bending and axial compression - 6.1.4, 6.2.4

LC: $'\text{ULS 6.10b} - \text{Vind Y}-'$, $k_{mod} = 0.90$, $x = 1893.32$ mm

$\sigma_{c,0,d}=0.01\ N/mm^2\leq f_{c,0,d}=17.64\ N/mm^2$ (6.2) - OK

$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{0.01}{17.64}\right)^2+\frac{7.48}{23.04}+0.70\frac{0.00}{23.76}=0.32\leq1.00$ (6.19) - OK

$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+k_m\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{0.01}{17.64}\right)^2+0.70\frac{7.48}{23.04}+\frac{0.00}{23.76}=0.23\leq1.00$ (6.20) - OK

Combined shear and torsion - 6.1.7, 6.1.8

LC: $'\text{ULS 6.10b} - \text{NL 1} - \text{Vind Y}-'$, $k_{mod} = 0.90$, $x = 4259.98$ mm

$\tau_d=1.28\ N/mm^2\leq f_{v,d}=2.52\ N/mm^2$ (6.13) - OK

Flexural buckling around axis 1 - 6.3.2

LC: $'\text{ULS 6.10b} - \text{Vind X}-'$, $k_{mod} = 0.90$, $x = 1893.32$ mm

$\beta_c = 0.1$ (6.29)

$$\lambda_1 = \frac{l_0}{i_1} = \frac{4260}{91} = 46.85$$

$$\lambda_{rel,1} = \frac{\lambda_1}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{46.85}{\pi}\sqrt{\frac{24.50}{10800}} = 0.710 \qquad (6.21)$$

$$k_1 = 0.5\left(1 + \beta_c\left(\lambda_{rel,1} - 0.3\right) + \lambda_{rel,1}{}^2\right) = 0.5\left(1 + 0.1\,(0.710 - 0.3) + 0.710^2\right) = 0.773 \qquad (6.27)$$

$$k_{c,1} = \frac{1}{k_1+\sqrt{k_1{}^2-\lambda_{rel,1}{}^2}} = \frac{1}{0.773+\sqrt{0.773^2-0.710^2}} = 0.928 \qquad (6.25)$$

$$\frac{\sigma_{c,0,d}}{k_{c,1}\cdot f_{c,0,d}}+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\cdot\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.01}{0.928\cdot 17.64}+\frac{7.48}{23.04}+0.70\cdot\frac{0.01}{23.76}=0.33\leq 1.00 \qquad (6.23)\text{ - OK}$$

Flexural buckling around axis 2 - 6.3.2

LC: $'\mathrm{ULS\ 6.10b\ -\ Vind\ X-}'$, $k_{mod} = 0.90$, $x = 1893.32$ mm

$$\beta_c = 0.1 \qquad (6.29)$$

$$\lambda_2 = \frac{l_0}{i_2} = \frac{4260}{62} = 68.64$$

$$\lambda_{rel,2} = \frac{\lambda_2}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{68.64}{\pi}\sqrt{\frac{24.50}{10800}} = 1.041 \qquad (6.22)$$

$$k_2 = 0.5\left(1 + \beta_c\left(\lambda_{rel,2} - 0.3\right) + \lambda_{rel,2}{}^2\right) = 0.5\left(1 + 0.1\,(1.041 - 0.3) + 1.041^2\right) = 1.078 \qquad (6.28)$$

$$k_{c,2} = \frac{1}{k_2+\sqrt{k_2{}^2-\lambda_{rel,2}{}^2}} = \frac{1}{1.078+\sqrt{1.078^2-1.041^2}} = 0.734 \qquad (6.26)$$

$$\frac{\sigma_{c,0,d}}{k_{c,2}\cdot f_{c,0,d}}+k_m\cdot\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.01}{0.734\cdot 17.64}+0.70\cdot\frac{7.48}{23.04}+\frac{0.01}{23.76}=0.23\leq 1.00 \qquad (6.24)\text{ - OK}$$

Lateral torsional buckling - 6.3.3

LC: $'\mathrm{ULS\ 6.10b\ -\ NL\ 1\ -\ Vind\ Y-}'$, $k_{mod} = 0.90$, $x = 1893.32$ mm

$$l_{ef} = l/\frac{12.5\cdot M_{max}}{2.5\cdot M_{max} + 3\cdot M_2 + 4\cdot M_3 + 3\cdot M_4} + 2\cdot h = 4260/\frac{12.5\cdot 31.21}{2.5\cdot 31.21 + 3\cdot 25.12 + 4\cdot 29.79 + 3\cdot 12.27} + 2\cdot 315 = 4008\text{ mm}$$

$$\sigma_{m,crit} = \frac{\pi\sqrt{E_{0,05}\cdot I_2\cdot G_{0,05}\cdot I_t}}{l_{ef}\cdot W_1} = \frac{\pi\sqrt{10800\cdot 2.609e+08\cdot 540\cdot 6.035e+08}}{4008\cdot 3.556e+06} = 211.26\ \mathrm{N/mm^2} \qquad (6.31)$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,1,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30.00}{211.26}} = 0.377 \qquad (6.30)$$

$$\lambda_{rel,m} = 0.377 \leq 0.75 \quad \rightarrow \quad k_{crit} = 1.000 \qquad (6.34)$$

$$\frac{\sigma_{m,1,d}}{k_{crit}\cdot f_{m,1,d}}=\frac{8.78}{1.000\cdot 23.04}=0.38\leq 1.00 \qquad (6.33)\text{ - OK}$$

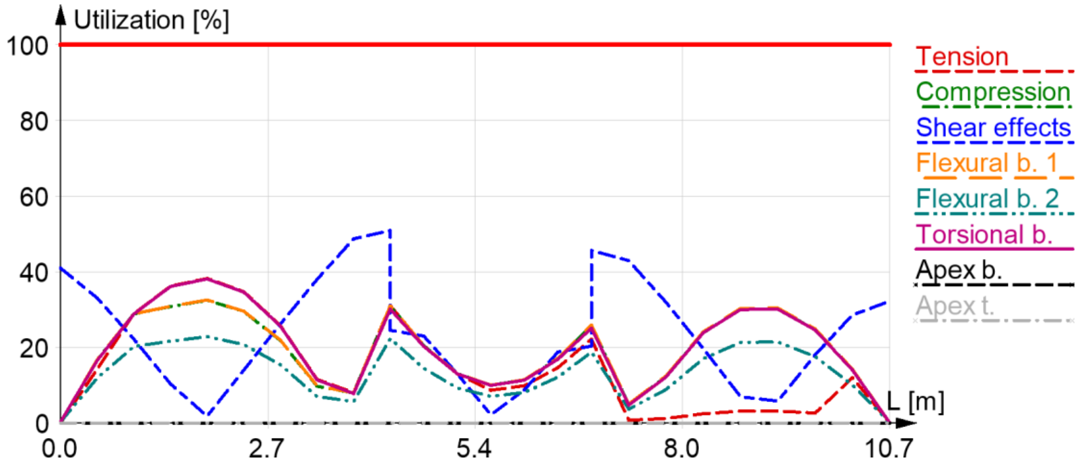
Bending at apex - 6.4.3

Not relevant

Tension at apex - 6.4.3

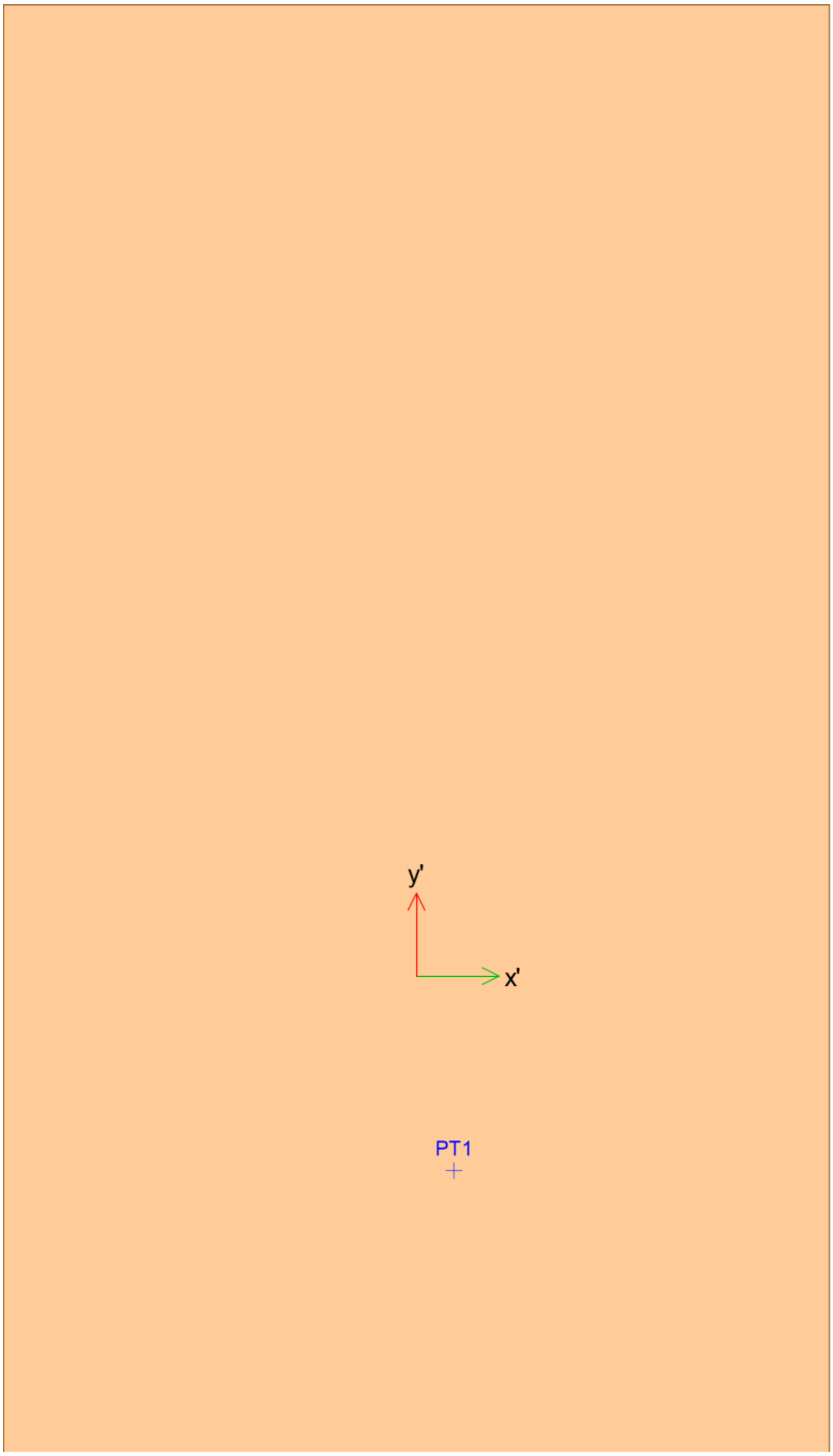
Not relevant

Summary


Utilization [%]
100
80
60
40
20
0
0.0
2.7
5.4
8.0
10.7
L [m]
Tension
Compression
Shear effects
Flexural b. 1
Flexural b. 2
Torsional b.
Apex b.
Apex t.

FLOOR AT LEVEL 6 (Floor2.28) Compared with hand calculation
Maximum of load combinations

Geometry



Maximum nodes:
PT1 (16.99, 9.24, 18.20) [m]
PT2 (13.21, 11.27, 18.20) [m]
PT3 (13.21, 11.68, 18.20) [m]

Node numbers:
PT1: 77123
PT2: 63933
PT3: 64025

Panel type:
160-5s

Total thickness:
t = 160.00 mm

Panel properties

Service class: 1, $\gamma_{M,ult.} = 1.25$, $\gamma_{M,acc./seis.} = 1.00$, $k_{sys} = 1.00$

No	Material	Thickness [mm]	Theta [°]	Rho [kg/m^3]
1	C24	40	0	420
2	C14	20	90	350
3	C24	40	0	420
4	C14	20	90	350
5	C24	40	0	420

Mechanical properties

No	$E_{0,mean}$ [N/mm^2]	$E_{90,mean}$ [N/mm^2]	ν_{xy} [-]	$G_{xy,mean}$ [N/mm^2]	$G_{xz,mean}$ [N/mm^2]	$G_{yz,mean}$ [N/mm^2]
1	11000	0	0.00	690	690	69
2	7000	0	0.00	440	440	44
3	11000	0	0.00	690	690	69
4	7000	0	0.00	440	440	44
5	11000	0	0.00	690	690	69

Limit stresses

No	$f_{m,0,k}$ [N/mm	$f_{m,90,k}$ [N/mm	$f_{t,0,k}$ [N/mm	$f_{t,90,k}$ [N/mm	$f_{c,0,k}$ [N/mm	$f_{c,90,k}$ [N/mm	$f_{xy,k}$ [N/mm	$f_{v,k}$ [N/mm	$f_{vR,k}$ [N/mm
1	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
2	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
3	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
4	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
5	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00

Tension and bending, x - 6.2.3

Panel: 'Floor2.28.1', Layer: '5', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT1

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{|\sigma_{m,0,d}|}{f_{m,0,d}} = \frac{2.24}{10.44} + \frac{|0.74|}{17.28} = 0.26 \leq 1.00 \qquad (6.17) \text{ - OK}$$

Compression and bending, x - 6.1.4, 6.2.4

Panel: 'Floor2.28.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y – ', $k_{mod} = 0.90$, PT1

$$\frac{|\sigma_{c,0,d}|}{f_{c,0,d}} = \frac{|-2.23|}{15.12} = 0.15 \leq 1.00 \qquad (6.2) \text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{|\sigma_{m,0,d}|}{f_{m,0,d}} = \left(\frac{-2.23}{15.12}\right)^2 + \frac{|0.74|}{17.28} = 0.06 \leq 1.00 \qquad (6.19) \text{ - OK}$$

Shear, xy - 6.1.7

Panel: 'Floor2.28.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT2

$$\frac{|\tau_{xy,d}|}{f_{xy,d}} = \frac{|0.50|}{2.88} = 0.17 \leq 1.00 \qquad (6.13) \text{ - OK}$$

Shear, xz - 6.1.7

Panel: 'Floor2.28.1', Layer: '4', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT3

$$\frac{|\tau_{xz,d}|}{f_{v,d}} = \frac{|0.14|}{2.16} = 0.06 \leq 1.00 \qquad (6.13) \text{ - OK}$$

Shear, yz - 6.1.7

Panel: 'Floor2.28.1', Layer: '3', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT3

$$\frac{|\tau_{yz,d}|}{f_{R,d}} = \frac{|0.14|}{1.44} = 0.09 \leq 1.00 \qquad (6.13) \text{ - OK}$$

Shear interaction

Panel: 'Floor2.28.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT2

$$\left(\frac{\tau_{xy,d}}{f_{xy,d}}\right)^2 + \left(\frac{\tau_{xz,d}}{f_{v,d}}\right)^2 = \left(\frac{0.50}{2.88}\right)^2 + \left(\frac{0.00}{2.88}\right)^2 = 0.03 \leq 1.00 \text{ - OK}$$

Tension and shear

Panel: 'Floor2.28.1', Layer: '3', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT3

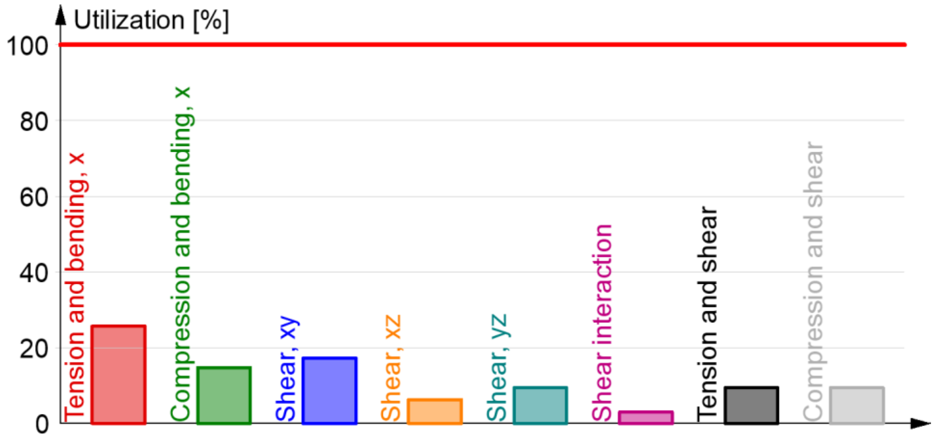
$$\frac{\sigma_{t,90,d}}{f_{t,90,d}} + \frac{|\tau_{yz,d}|}{f_{R,d}} = \frac{0.00}{0.29} + \frac{|0.14|}{1.44} = 0.09 \leq 1.00 \text{ - OK}$$

Compression and shear

Panel: 'Floor2.28.1', Layer: '3', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT3

$$\frac{|\sigma_{c,90,d}|}{f_{c,90,d}} + \frac{|\tau_{yz,d}|}{f_{R,d}} = \frac{|0.00|}{1.80} + \frac{|0.14|}{1.44} = 0.09 \leq 1.00 \text{ - OK}$$

Summary

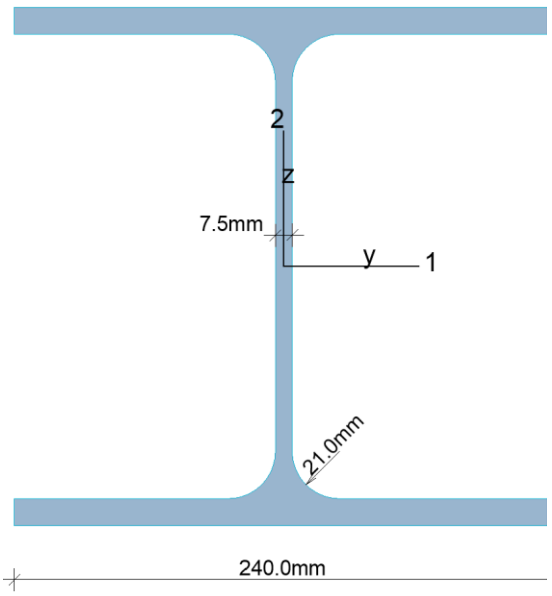

Utilization [%]
100
80
60
40
20
0
Tension and bending, x
Compression and bending, x
Shear, xy
Shear, xz
Shear, yz
Shear interaction
Tension and shear
Compression and shear

BALCONY BEAM (B.3.1)
Maximum of load combinations

S 355

E	=	210000	N/mm^2			
G	=	80769	N/mm^2			
$\gamma_{M0,ult}$	=	1.00		$\gamma_{M0,acc/seis}$	=	1.00
$\gamma_{M1,ult}$	=	1.00		$\gamma_{M1,acc/seis}$	=	1.00
$\gamma_{M2,ult}$	=	1.20		$\gamma_{M2,acc/seis}$	=	1.00

HE-A 240



P	=	1369	mm	f_y	=	355	N/mm^2
A	=	7684	mm^2	ε	=	0.81	
I_y	=	7.763e+0	mm^4	λ_1	=	76.40	
I_z	=	2.769e+0	mm^4				
I_1	=	7.763e+0	mm^4				
I_2	=	2.769e+0	mm^4				
$W_{pl,1}$	=	7.446e+0	mm^3				
$W_{pl,2}$	=	3.517e+0	mm^3				
$W_{el,min,1}$	=	6.751e+0	mm^3				
$W_{el,min,2}$	=	2.307e+0	mm^3				
i_1	=	101	mm				
i_2	=	60	mm				
I_t	=	4.104e+0	mm^4				
I_w	=	3.216e+1	mm^6				

Shear resistance, 1-1 - Part 1-1: 6.2.6, 6.2.8

LC: 'ULS 6.10b - NL 1 - Vind Y+', x = 0 mm

$\text{Class}_N = 2,\ \text{Class}_{M1} = 2,\ \text{Class}_{M2} = 2$

$$V_{1,pl,Rd} = \frac{A_{1,v} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{6139 \cdot 355}{\sqrt{3} \cdot 1.00} = 1258.15\ \text{kN} \qquad (6.18)$$

$$V_{1,pl,T,Rd} = \sqrt{1 - \frac{\tau_{t,Ed}}{1.25\,(f_y/\sqrt{3})/\gamma_{M0}}} \cdot V_{1,pl,Rd} = \sqrt{1 - \frac{0.00}{1.25\,(355/\sqrt{3})/1.00}} \cdot 1258.15 = 1258.15\ \text{kN} \qquad (6.26)$$

$$\frac{V_{1,Ed}}{V_{1,pl,T,Rd}} = \frac{9.31}{1258.15} = 0.01 \leq 1.00 \qquad (6.25)\ \text{- OK}$$

Shear resistance, 2-2 - Part 1-1: 6.2.6, 6.2.8

LC: 'ULS 6.10b - NL 1 - Vind Y-', x = 0 mm

$\text{Class}_N = 2,\ \text{Class}_{M1} = 2,\ \text{Class}_{M2} = 2$

$$V_{2,pl,Rd} = \frac{A_{2,v} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{2518 \cdot 355}{\sqrt{3} \cdot 1.00} = 516.00\ \text{kN} \qquad (6.18)$$

$$V_{2,pl,T,Rd} = \sqrt{1 - \frac{\tau_{t,Ed}}{1.25\,(f_y/\sqrt{3})/\gamma_{M0}}} \cdot V_{2,pl,Rd} = \sqrt{1 - \frac{0.00}{1.25\,(355/\sqrt{3})/1.00}} \cdot 516.00 = 516.00\ \text{kN} \qquad (6.26)$$

$$\frac{V_{2,Ed}}{V_{2,pl,T,Rd}} = \frac{27.01}{516.00} = 0.05 \leq 1.00 \qquad (6.25)\text{ - OK}$$

Torsional resistance - Part 1-1: 6.2.7

LC: 'ULS 6.10a', x = 0 mm

$\text{Class}_N = 2,\ \text{Class}_{M1} = 2,\ \text{Class}_{M2} = 2$

$\tau_{max,unit} = 50.87\ \frac{\text{N/mm}^2}{\text{kNm}}$ is calculated by FEM analysis.

$$T_{Rd} = \frac{f_y}{\sqrt{3} \cdot \tau_{max,unit} \cdot \gamma_{M0}} = \frac{355}{\sqrt{3} \cdot 50.87 \cdot 1.00} = 4.03\ \text{kNm}$$

$$\frac{T_{Ed}}{T_{Rd}} = \frac{0.00}{4.03} = 0.00 \leq 1.00 \qquad (6.23)\text{ - OK}$$

Shear stress - Part 1-1: 6.2.6

Not relevant

Normal stress - Part 1-1: 6.2.1

Not relevant

Normal capacity - Part 1-1: 6.2

LC: 'ULS 6.10b - NL 1 - Vind X-', x = 3906 mm

$\text{Class}_N = 2,\ \text{Class}_{M1} = 2,\ \text{Class}_{M2} = 2$

$V_{1,Ed} = 0.01\ \text{kN} \leq 0.5 \cdot V_{1,pl,T,Rd} = 0.5 \cdot 1258.15 = 629.08\ \text{kN} \quad \rightarrow \quad \rho_1 = 0.00$

$V_{2,Ed} = 0.00\ \text{kN} \leq 0.5 \cdot V_{2,pl,T,Rd} = 0.5 \cdot 516.00 = 258.00\ \text{kN} \quad \rightarrow \quad \rho_1 = 0.00$

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{1,Ed}}{M_{1,Rd}} + \frac{M_{2,Ed}}{M_{2,Rd}} = \frac{0.87}{2727.66} + \frac{49.33}{264.34} + \frac{0.00}{124.86} = 0.19 \leq 1.00 \qquad (6.2)\text{ - OK}$$

Flexural buckling, 1-1 - Part 1-1: 6.3.1

LC: 'Stability Vind X+', x = 3906 mm

$\text{Class}_N = 2,\ \text{Class}_{M1} = 2,\ \text{Class}_{M2} = 2$

$$\bar{\lambda}_1 = \frac{L_{cr,1}}{i_1 \cdot \lambda_1} = \frac{7813}{101 \cdot 76.40} = 1.02 \qquad (6.50)$$

$\alpha_1 = 0.34$ (Buckling curve: b)

$$\varphi_1 = 0.5\left[1 + \alpha_1 \cdot (\bar{\lambda}_1 - 0.2) + \bar{\lambda}_1^{\,2}\right] = 0.5\left[1 + 0.34 \cdot (1.02 - 0.2) + 1.02^2\right] = 1.16$$

$$\chi_1 = \min\left(\frac{1}{\varphi_1 + \sqrt{\varphi_1^{\,2} - \bar{\lambda}_1^{\,2}}}, 1.0\right) = \min\left(\frac{1}{1.16 + \sqrt{1.16^2 - 1.02^2}}, 1.0\right) = 0.59 \qquad (6.49)$$

$$N_{b,Rd,1} = \frac{\chi_1 \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.59 \cdot 7684 \cdot 355}{1.00} = 1598.45\ \text{kN} \qquad (6.47)$$

$$\frac{N_{Ed}}{N_{b,Rd,1}} = \frac{1.32}{1598.45} = 0.00 \leq 1.00 \qquad (6.46)\text{ - OK}$$

Flexural buckling, 2-2 - Part 1-1: 6.3.1

LC: 'Stability Vind X+', x = 3906 mm

$Class_N = 2,\ Class_{M1} = 2,\ Class_{M2} = 2$

$\bar{\lambda}_2 = \frac{L_{cr,2}}{i_2 \cdot \lambda_1} = \frac{7813}{60 \cdot 76.40} = 1.70$ (6.50)

$\alpha_2 = 0.49$ (Buckling curve: c)

$\varphi_2 = 0.5\left[1 + \alpha_2 \cdot (\bar{\lambda}_2 - 0.2) + \bar{\lambda}_2^{\,2}\right] = 0.5\left[1 + 0.49 \cdot (1.70 - 0.2) + 1.70^2\right] = 2.32$

$\chi_2 = \min\left(\frac{1}{\varphi_2 + \sqrt{\varphi_2^2 - \bar{\lambda}_2^{\,2}}}, 1.0\right) = \min\left(\frac{1}{2.32 + \sqrt{2.32^2 - 1.70^2}}, 1.0\right) = 0.26$ (6.49)

$N_{b,Rd,2} = \frac{\chi_2 \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.26 \cdot 7684 \cdot 355}{1.00} = 700.55\ kN$ (6.47)

$\frac{N_{Ed}}{N_{b,Rd,2}} = \frac{1.32}{700.55} = 0.00 \le 1.00$ (6.46) - OK

Torsional-flexural buckling - Part 1-1: 6.3.1

LC: 'Stability Vind X+', x = 3906 mm

$Class_N = 2,\ Class_{M1} = 2,\ Class_{M2} = 2$

$i_0 = \sqrt{i_1^2 + i_2^2 + y_0^2 + z_0^2} = \sqrt{101^2 + 60^2 + 0^2 + 0^2} = 117\ mm$

$N_{cr,1} = \frac{\pi^2 \cdot E \cdot I_1}{L_{cr,1}^2} = \frac{\pi^2 \cdot 210000 \cdot 77631853}{7813^2} = 2635.87\ kN$

$N_{cr,2} = \frac{\pi^2 \cdot E \cdot I_2}{L_{cr,2}^2} = \frac{\pi^2 \cdot 210000 \cdot 27688082}{7813^2} = 940.10\ kN$

$N_{cr,T} = \frac{1}{i_0^2}\left(G \cdot I_t + \frac{\pi^2 \cdot E \cdot I_w}{L_t^2}\right) = \frac{1}{117^2}\left(80769 \cdot 4.104e+05 + \frac{\pi^2 \cdot 210000 \cdot 3.216e+11}{7813^2}\right) = 3214.75\ kN$

$$i_0^2\,(N - N_{cr,1})\,(N - N_{cr,2})\,(N - N_{cr,T}) - N^2\,y_0^2\,(N - N_{cr,2}) - N^2\,z_0^2\,(N - N_{cr,1})$$
$$= 117^2\,(N - 2635.87)\,(N - 940.10)\,(N - 3214.75) - N^2\,0^2\,(N - 940.10) - N^2\,0^2\,(N - 2635.87)$$
$$= 0$$

Smallest root of the above equation related to the torsional-flexural buckling:

$N_{cr,TF} = 3214.75\ kN$

$N_{cr} = \min(N_{cr,T}, N_{cr,TF}) = \min(3214.75, 3214.75) = 3214.75\ kN$

$\bar{\lambda}_T = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{7684 \cdot 355}{3214.75}} = 0.92$ (6.53)

$\alpha_T = 0.49$ (Buckling curve: c)

$\varphi_T = 0.5\left[1 + \alpha_T \cdot (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^{\,2}\right] = 0.5\left[1 + 0.49 \cdot (0.92 - 0.2) + 0.92^2\right] = 1.10$

$\chi_T = \min\left(\frac{1}{\varphi_T + \sqrt{\varphi_T^2 - \bar{\lambda}_T^{\,2}}}, 1.0\right) = \min\left(\frac{1}{1.10 + \sqrt{1.10^2 - 0.92^2}}, 1.0\right) = 0.59$ (6.49)

$N_{b,Rd,T} = \frac{\chi_T \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.59 \cdot 7684 \cdot 355}{1.00} = 1600.87\ kN$ (6.47)

$\frac{N_{Ed}}{N_{b,Rd,T}} = \frac{1.32}{1600.87} = 0.00 \le 1.00$ - OK

Lateral torsional buckling, top flange - Part 1-1: 6.3.2.2

LC: 'ULS 6.10b - NL 1 - Vind Y+', x = 3906 mm

$Class_N = 2,\ Class_{M1} = 2,\ Class_{M2} = 2$

$N_{cr,LT} = \frac{\pi^2 \cdot E \cdot I_z}{(k_z \cdot L_{cr})^2} = \frac{\pi^2 \cdot 2.100e+05 \cdot 2.769e+07}{(1.00 \cdot 7813)^2} = 940.10\ kN$

Loaded on top edge.

$$Z = (C_2 \cdot z_g - C_3 \cdot z_j) = (0.45 \cdot 115 - 0.52 \cdot 0) = 51.75\ \text{mm}$$

$$M_{cr} = C_1 \cdot N_{cr,LT} \cdot \left(\left[\left(\frac{k_z}{k_w}\right)^2 \cdot \frac{I_w}{I_z} + \frac{G \cdot I_t}{N_{cr,LT}} + Z^2\right]^{0.5} - Z\right)$$

$$= 1.13 \cdot 9.401e+05 \cdot \left(\left[\left(\frac{1.00}{1.00}\right)^2 \cdot \frac{3.216e+11}{2.769e+07} + \frac{8.077e+04 \cdot 4.104e+05}{9.401e+05} + 51.75^2\right]^{0.5} - 51.75\right)$$

$$= 181.50\ \text{kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}} = \sqrt{\frac{744624 \cdot 355}{1.815e+08}} = 1.21$$

$\alpha_{LT} = 0.21$ (Buckling curve: a)

$$\varphi_{LT} = 0.5\left[1 + \alpha_{LT} \cdot (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2\right] = 0.5\left[1 + 0.21 \cdot (1.21 - 0.2) + 1.21^2\right] = 1.33$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \bar{\lambda}_{LT}^2}}, 1.0\right) = \min\left(\frac{1}{1.33 + \sqrt{1.33^2 - 1.21^2}}, 1.0\right) = 0.53 \qquad (6.56)$$

$$M_{y,b,Rd} = \frac{\chi_{LT} \cdot W_y \cdot f_y}{\gamma_{M1}} = \frac{0.53 \cdot 744624 \cdot 355}{1.00} = 138.96\ \text{kNm} \qquad (6.55)$$

$$\frac{M_{1,Ed}}{M_{y,b,Rd}} = \frac{49.35}{138.96} = 0.36 \le 1.00 \qquad \text{(6.54) - OK}$$

Lateral torsional buckling, bottom flange - Part 1-1: 6.3.2.2

LC: 'Stability Vind X+', x = 3906 mm

$Class_N = 2$, $Class_{M1} = 2$, $Class_{M2} = 2$

$$N_{cr,LT} = \frac{\pi^2 \cdot E \cdot I_z}{(k_z \cdot L_{cr})^2} = \frac{\pi^2 \cdot 2.100e+05 \cdot 2.769e+07}{(1.00 \cdot 7813)^2} = 940.10\ \text{kN}$$

Loaded on top edge.

$$Z = (C_2 \cdot z_g - C_3 \cdot z_j) = (0.45 \cdot -115 - 0.52 \cdot 0) = -51.75\ \text{mm}$$

$$M_{cr} = C_1 \cdot N_{cr,LT} \cdot \left(\left[\left(\frac{k_z}{k_w}\right)^2 \cdot \frac{I_w}{I_z} + \frac{G \cdot I_t}{N_{cr,LT}} + Z^2\right]^{0.5} - Z\right)$$

$$= 1.13 \cdot 9.401e+05$$

$$\cdot \left(\left[\left(\frac{1.00}{1.00}\right)^2 \cdot \frac{3.216e+11}{2.769e+07} + \frac{8.077e+04 \cdot 4.104e+05}{9.401e+05} + -51.75^2\right]^{0.5} - -51.75\right) = 291.45\ \text{kNm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}} = \sqrt{\frac{744624 \cdot 355}{2.914e+08}} = 0.95$$

$\alpha_{LT} = 0.21$ (Buckling curve: a)

$$\varphi_{LT} = 0.5\left[1 + \alpha_{LT} \cdot (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2\right] = 0.5\left[1 + 0.21 \cdot (0.95 - 0.2) + 0.95^2\right] = 1.03$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \bar{\lambda}_{LT}^2}}, 1.0\right) = \min\left(\frac{1}{1.03 + \sqrt{1.03^2 - 0.95^2}}, 1.0\right) = 0.70 \qquad (6.56)$$

$$M_{y,b,Rd} = \frac{\chi_{LT} \cdot W_y \cdot f_y}{\gamma_{M1}} = \frac{0.70 \cdot 744624 \cdot 355}{1.00} = 184.68\ \text{kNm} \qquad (6.55)$$

$$\frac{M_{1,Ed}}{M_{y,b,Rd}} = \frac{5.67}{184.68} = 0.03 \le 1.00 \qquad \text{(6.54) - OK}$$

Interaction between normal force and bending 1. - Part 1-1: 6.3.3

LC: 'ULS 6.10b - NL 1 - Vind Y+', x = 3906 mm

$Class_N = 2$, $Class_{M1} = 2$, $Class_{M2} = 2$

k_{ij} factors are calculated according to Method 1

C_{my}	=	1.00	C_{yy}	=	1.00
C_{mz}	=	1.00	C_{yz}	=	0.83
C_{mLT}	=	1.00	C_{zy}	=	1.00

$M_{2,Rk} = f_y \cdot W_{pl,2} = 355 \cdot 351708 = 124.86$ kNm

$$\frac{N_{Ed}^{comp}}{N_{b,Rd,1}} + k_{11} \cdot \frac{M_{1,Ed}}{M_{y,b,Rd}} + k_{12} \cdot \frac{M_{2,Ed}}{\frac{M_{2,Rk}}{\gamma_{M1}}} = \frac{0.00}{1598.45} + 1.00 \cdot \frac{49.35}{138.96} + 0.84 \cdot \frac{0.01}{\frac{124.86}{1.00}} = 0.36 \leq 1.00 \qquad \text{(6.61) - OK}$$

Interaction between normal force and bending 2. - Part 1-1: 6.3.3

LC: 'ULS 6.10b - NL 1 - Vind Y+', x = 3906 mm

$Class_N = 2$, $Class_{M1} = 2$, $Class_{M2} = 2$

k_{ij} factors are calculated according to Method 1

C_{my}	=	1.00	C_{yy}	=	1.00
C_{mz}	=	1.00	C_{yz}	=	0.83
C_{mLT}	=	1.00	C_{zy}	=	1.00

$M_{2,Rk} = f_y \cdot W_{pl,2} = 355 \cdot 351708 = 124.86$ kNm

$$\frac{N_{Ed}^{comp}}{N_{b,Rd,2}} + k_{21} \cdot \frac{M_{1,Ed}}{M_{y,b,Rd}} + k_{22} \cdot \frac{M_{2,Ed}}{\frac{M_{2,Rk}}{\gamma_{M1}}} = \frac{0.00}{700.55} + 0.51 \cdot \frac{49.35}{138.96} + 1.04 \cdot \frac{0.01}{\frac{124.86}{1.00}} = 0.18 \leq 1.00 \qquad \text{(6.62) - OK}$$

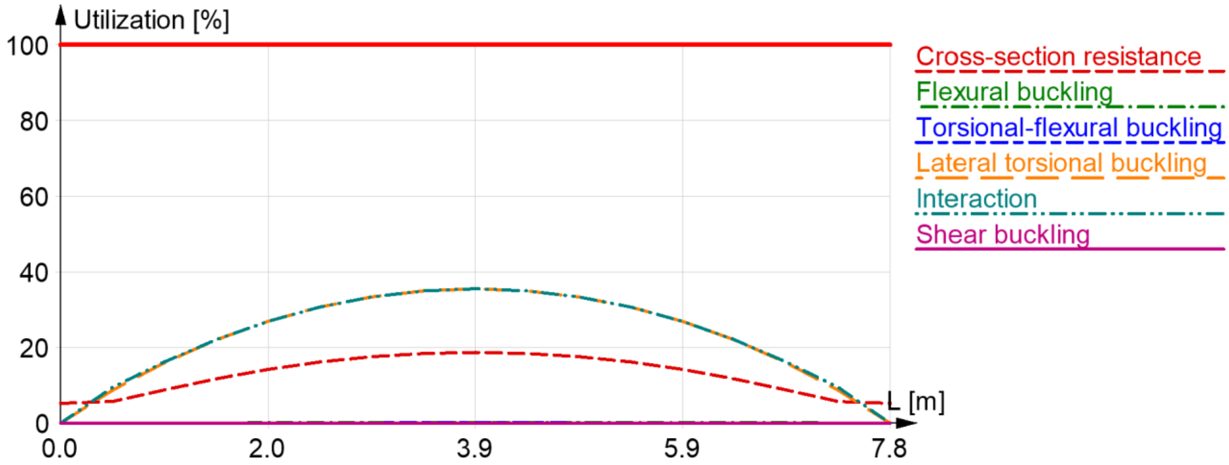
Interaction between normal force and bending, 2nd order - Part 1-1: 6.3.3

Not relevant

Shear buckling - Part 1-5: 5

$\frac{h_w}{t} = \frac{206}{7} = 27.5 \leq \frac{72}{\eta} \cdot \varepsilon = \frac{72}{1.20} \cdot 0.81 = 48.8 \quad \rightarrow \quad$ Not relevant

Summary



Most loaded BALCONY DIAGONAL BAR (T.13.1)
Maximum of load combinations

S 275

E =210000 N/mm^2

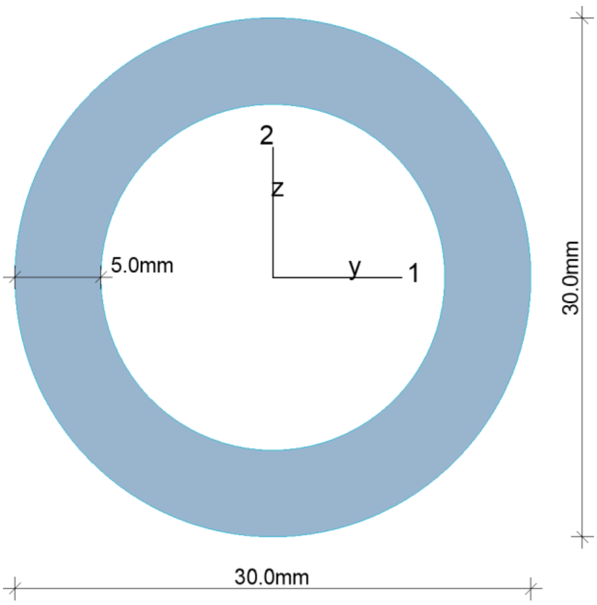
G = 80769 N/mm^2

$\gamma_{M0,ult}$= 1.00 $\gamma_{M0,acc/seis}$=1.00

$\gamma_{M1,ult}$= 1.00 $\gamma_{M1,acc/seis}$=1.00

$\gamma_{M2,ult}$= 1.20 $\gamma_{M2,acc/seis}$=1.00

CHS 30-5.0



P	= 94 mm	f_y = 275 N/mm^2
A	= 393 mm^2	ε = 0.92
I_y	=3.191e+04 mm^4	λ_1=86.80
I_z	=3.191e+04 mm^4	
I_1	=3.191e+04 mm^4	
I_2	=3.191e+04 mm^4	
$W_{pl,1}$	=3.167e+03 mm^3	
$W_{pl,2}$	=3.167e+03 mm^3	
$W_{el,min,1}$	=2.127e+03 mm^3	
$W_{el,min,2}$	=2.127e+03 mm^3	
i_1	= 9 mm	
i_2	= 9 mm	
I_t	=6.381e+04 mm^4	
I_w	=0.000e+00 mm^6	

Shear resistance, 1-1 - Part 1-1: 6.2.6, 6.2.8

LC: 'ULS 6.10a', x = 0 mm

$\text{Class}_N = 1,\ \text{Class}_{M1} = 1,\ \text{Class}_{M2} = 1$

$$V_{1,pl,Rd} = \frac{A_{1,v} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{250 \cdot 275}{\sqrt{3} \cdot 1.00} = 39.69\ \text{kN} \qquad (6.18)$$

$$V_{1,pl,T,Rd} = 1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}} \cdot V_{1,pl,Rd} = 1 - \frac{0.00}{(275/\sqrt{3})/1.00} \cdot 39.69 = 39.69\ \text{kN} \qquad (6.28)$$

$$\frac{V_{1,Ed}}{V_{1,pl,T,Rd}} = \frac{0.00}{39.69} = 0.00 \leq 1.00 \qquad (6.25)\text{ - OK}$$

Shear resistance, 2-2 - Part 1-1: 6.2.6, 6.2.8

LC: 'ULS 6.10a', x = 0 mm

$\text{Class}_N = 1,\ \text{Class}_{M1} = 1,\ \text{Class}_{M2} = 1$

$$V_{2,pl,Rd} = \frac{A_{2,v} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{250 \cdot 275}{\sqrt{3} \cdot 1.00} = 39.69\ \text{kN} \qquad (6.18)$$

$$V_{2,pl,T,Rd} = 1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}} \cdot V_{2,pl,Rd} = 1 - \frac{0.00}{(275/\sqrt{3})/1.00} \cdot 39.69 = 39.69\ \text{kN} \qquad (6.28)$$

$$\frac{V_{2,Ed}}{V_{2,pl,T,Rd}} = \frac{0.00}{39.69} = 0.00 \leq 1.00 \qquad (6.25)\text{ - OK}$$

Torsional resistance - Part 1-1: 6.2.7

LC: 'ULS 6.10a', x = 0 mm

$\text{Class}_N = 1, \ \text{Class}_{M1} = 1, \ \text{Class}_{M2} = 1$

$\tau_{max,unit} = 235.06 \ \frac{N/mm^2}{kNm}$ is calculated by FEM analysis.

$$T_{Rd} = \frac{f_y}{\sqrt{3} \cdot \tau_{max,unit} \cdot \gamma_{M0}} = \frac{275}{\sqrt{3} \cdot 235.06 \cdot 1.00} = 0.68 \text{ kNm}$$

$\frac{T_{Ed}}{T_{Rd}} = \frac{0.00}{0.68} = 0.00 \leq 1.00$ (6.23) - OK

Shear stress - Part 1-1: 6.2.6

Not relevant

Normal stress - Part 1-1: 6.2.1

Not relevant

Normal capacity - Part 1-1: 6.2

LC: 'ULS 6.10b - NL 1 - Vind X-', x = 0 mm

$\text{Class}_N = 1, \ \text{Class}_{M1} = 1, \ \text{Class}_{M2} = 1$

$V_{1,Ed} = 0.00 \text{ kN} \leq 0.5 \cdot V_{1,pl,T,Rd} = 0.5 \cdot 39.69 = 19.85 \text{ kN} \rightarrow \rho_1 = 0.00$

$V_{2,Ed} = 0.00 \text{ kN} \leq 0.5 \cdot V_{2,pl,T,Rd} = 0.5 \cdot 39.69 = 19.85 \text{ kN} \rightarrow \rho_1 = 0.00$

$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{1,Ed}}{M_{1,Rd}} + \frac{M_{2,Ed}}{M_{2,Rd}} = \frac{33.90}{107.99} + \frac{0.00}{0.87} + \frac{0.00}{0.87} = 0.31 \leq 1.00$ (6.2) - OK

Flexural buckling, 1-1 - Part 1-1: 6.3.1

LC: 'Stability Vind X+', x = 0 mm

$\text{Class}_N = 1, \ \text{Class}_{M1} = 1, \ \text{Class}_{M2} = 1$

$\bar{\lambda}_1 = \frac{L_{cr,1}}{i_1 \cdot \lambda_1} = \frac{3478}{9 \cdot 86.80} = 4.45$ (6.50)

$\alpha_1 = 0.49$ (Buckling curve: c)

$$\varphi_1 = 0.5 \left[1 + \alpha_1 \cdot (\bar{\lambda}_1 - 0.2) + \bar{\lambda}_1^{\,2}\right] = 0.5 \left[1 + 0.49 \cdot (4.45 - 0.2) + 4.45^2\right] = 11.42$$

$$\chi_1 = \min\left(\frac{1}{\varphi_1 + \sqrt{\varphi_1^{\,2} - \bar{\lambda}_1^{\,2}}}, 1.0\right) = \min\left(\frac{1}{11.42 + \sqrt{11.42^2 - 4.45^2}}, 1.0\right) = 0.05 \qquad (6.49)$$

$N_{b,Rd,1} = \frac{\chi_1 \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.05 \cdot 393 \cdot 275}{1.00} = 4.92 \text{ kN}$ (6.47)

$\frac{N_{Ed}}{N_{b,Rd,1}} = \frac{3.91}{4.92} = 0.80 \leq 1.00$ (6.46) - OK

Flexural buckling, 2-2 - Part 1-1: 6.3.1

LC: 'Stability Vind X+', x = 0 mm

$\text{Class}_N = 1, \ \text{Class}_{M1} = 1, \ \text{Class}_{M2} = 1$

$\bar{\lambda}_2 = \frac{L_{cr,2}}{i_2 \cdot \lambda_1} = \frac{3478}{9 \cdot 86.80} = 4.45$ (6.50)

$\alpha_2 = 0.49$ (Buckling curve: c)

$$\varphi_2 = 0.5 \left[1 + \alpha_2 \cdot (\bar{\lambda}_2 - 0.2) + \bar{\lambda}_2^{\,2}\right] = 0.5 \left[1 + 0.49 \cdot (4.45 - 0.2) + 4.45^2\right] = 11.42$$

$$\chi_2 = \min\left(\frac{1}{\varphi_2 + \sqrt{\varphi_2^{\,2} - \bar{\lambda}_2^{\,2}}}, 1.0\right) = \min\left(\frac{1}{11.42 + \sqrt{11.42^2 - 4.45^2}}, 1.0\right) = 0.05 \qquad (6.49)$$

$N_{b,Rd,2} = \frac{\chi_2 \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.05 \cdot 393 \cdot 275}{1.00} = 4.92\ \text{kN}$ (6.47)

$\frac{N_{Ed}}{N_{b,Rd,2}} = \frac{3.91}{4.92} = 0.80 \leq 1.00$ (6.46) - OK

Torsional-flexural buckling - Part 1-1: 6.3.1

LC: 'Stability Vind X+', x = 0 mm

$Class_N = 1,\ Class_{M1} = 1,\ Class_{M2} = 1$

$$i_0 = \sqrt{i_1^2 + i_2^2 + y_0^2 + z_0^2} = \sqrt{9^2 + 9^2 + 0^2 + 0^2} = 13\ \text{mm}$$

$$N_{cr,1} = \frac{\pi^2 \cdot E \cdot I_1}{L_{cr,1}^2} = \frac{\pi^2 \cdot 210000 \cdot 31907}{3478^2} = 5.47\ \text{kN}$$

$$N_{cr,2} = \frac{\pi^2 \cdot E \cdot I_2}{L_{cr,2}^2} = \frac{\pi^2 \cdot 210000 \cdot 31907}{3478^2} = 5.47\ \text{kN}$$

$$N_{cr,T} = \frac{1}{i_0^2}\left(G \cdot I_t + \frac{\pi^2 \cdot E \cdot I_w}{L_t^2}\right) = \frac{1}{13^2}\left(80769 \cdot 6.381e+04 + \frac{\pi^2 \cdot 210000 \cdot 0.000e+00}{3478^2}\right) = 31718.00\ \text{kN}$$

$$i_0^2 (N - N_{cr,1})(N - N_{cr,2})(N - N_{cr,T}) - N^2 y_0^2 (N - N_{cr,2}) - N^2 z_0^2 (N - N_{cr,1})$$
$$= 13^2 (N - 5.47)(N - 5.47)(N - 31718.00) - N^2 0^2 (N - 5.47) - N^2 0^2 (N - 5.47) = 0$$

Smallest root of the above equation related to the torsional-flexural buckling:

$N_{cr,TF} = 31718.00\ \text{kN}$

$N_{cr} = \min(N_{cr,T}, N_{cr,TF}) = \min(31718.00, 31718.00) = 31718.00\ \text{kN}$

$\bar{\lambda}_T = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{393 \cdot 275}{31718.00}} = 0.06$ (6.53)

$\alpha_T = 0.49$ (Buckling curve: c)

$\varphi_T = 0.5\left[1 + \alpha_T \cdot (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2\right] = 0.5\left[1 + 0.49 \cdot (0.06 - 0.2) + 0.06^2\right] = 0.47$

$\chi_T = \min\left(\frac{1}{\varphi_T + \sqrt{\varphi_T^2 - \bar{\lambda}_T^2}}, 1.0\right) = \min\left(\frac{1}{0.47 + \sqrt{0.47^2 - 0.06^2}}, 1.0\right) = 1.00$ (6.49)

$N_{b,Rd,T} = \frac{\chi_T \cdot A \cdot f_y}{\gamma_{M1}} = \frac{1.00 \cdot 393 \cdot 275}{1.00} = 107.99\ \text{kN}$ (6.47)

$\frac{N_{Ed}}{N_{b,Rd,T}} = \frac{3.91}{107.99} = 0.04 \leq 1.00$ - OK

Lateral torsional buckling, top flange - Part 1-1: 6.3.2.4

Not relevant

Lateral torsional buckling, bottom flange - Part 1-1: 6.3.2.4

Not relevant

Interaction between normal force and bending 1. - Part 1-1: 6.3.3

Not relevant

Interaction between normal force and bending 2. - Part 1-1: 6.3.3

Not relevant

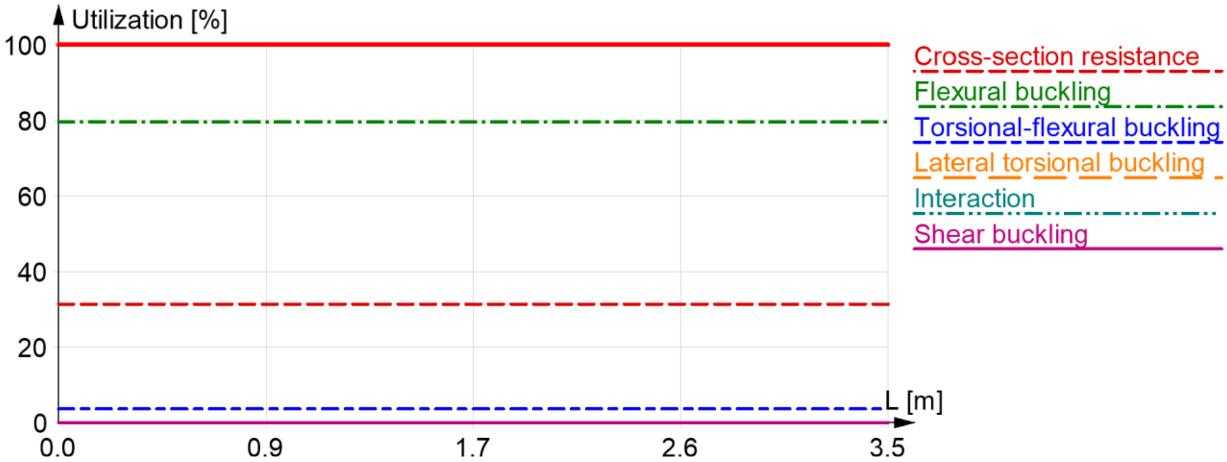
Interaction between normal force and bending, 2nd order - Part 1-1: 6.3.3

Not relevant

Shear buckling - Part 1-5: 5

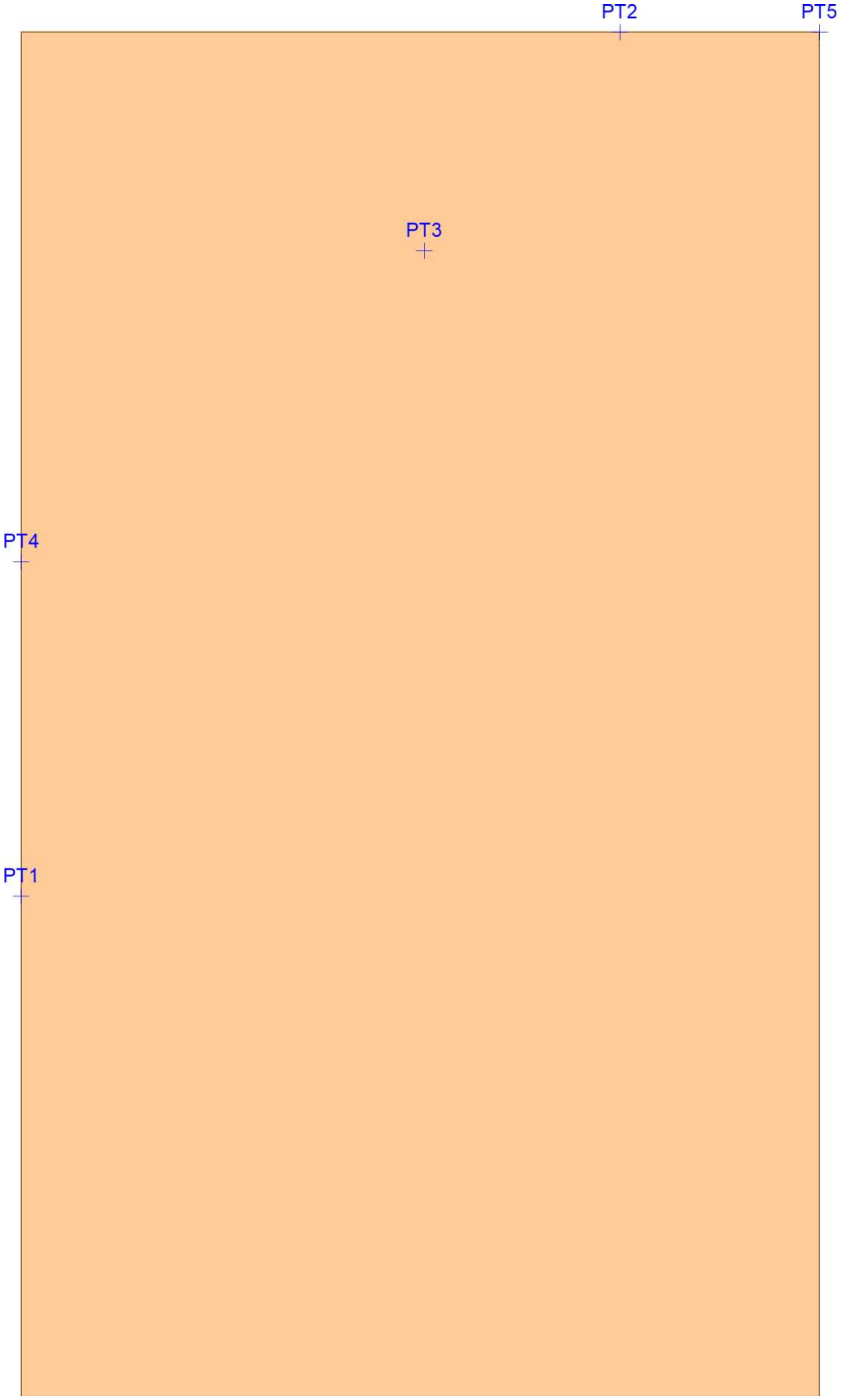
Not relevant

Summary



Most Loaded BALCONY PLATE (TP.9)
Maximum of load combinations

Geometry



Maximum nodes:
- PT1 (23.28, 6.97, 3.20) [m]
- PT2 (24.40, 8.58, 3.20) [m]
- PT3 (24.03, 8.17, 3.20) [m]
- PT4 (23.28, 7.59, 3.20) [m]
- PT5 (24.77, 8.58, 3.20) [m]

Node numbers:
- PT1: 98887
- PT2: 105973
- PT3: 105626
- PT4: 99004
- PT5: 106890

Panel type:
- 160-5s

Total thickness:
- $t = 160.00$ mm

Panel properties

Service class: 2, $\gamma_{M,ult.} = 1.30$, $\gamma_{M,acc./seis.} = 1.00$, $k_{sys} = 1.00$

No	Material	Thickness [mm]	Theta [°]	Rho [kg/m^3]
1	C24	40	0	420
2	C14	20	90	350
3	C24	40	0	420
4	C14	20	90	350
5	C24	40	0	420

Mechanical properties

No	$E_{0,mean}$ [N/mm^2]	$E_{90,mean}$ [N/mm^2]	ν_{xy} [-]	$G_{xy,mean}$ [N/mm^2]	$G_{xz,mean}$ [N/mm^2]	$G_{yz,mean}$ [N/mm^2]
1	11000	0	0.00	690	690	69
2	7000	0	0.00	440	440	44
3	11000	0	0.00	690	690	69
4	7000	0	0.00	440	440	44
5	11000	0	0.00	690	690	69

Limit stresses

No	$f_{m,0,k}$ [N/mm	$f_{m,90,k}$ [N/mm	$f_{t,0,k}$ [N/mm	$f_{t,90,k}$ [N/mm	$f_{c,0,k}$ [N/mm	$f_{c,90,k}$ [N/mm	$f_{xy,k}$ [N/mm	$f_{v,k}$ [N/mm	$f_{vR,k}$ [N/mm
1	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
2	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
3	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
4	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
5	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00

Tension and bending, x - 6.2.3

Panel: 'TP. 9.1', Layer: '2', LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, PT1

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{|\sigma_{m,0,d}|}{f_{m,0,d}} = \frac{0.57}{4.98} + \frac{|-0.05|}{9.69} = 0.12 \leq 1.00 \qquad (6.17) \text{ - OK}$$

Compression and bending, x - 6.1.4, 6.2.4

Panel: 'TP. 9.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT2

$$\frac{|\sigma_{c,0,d}|}{f_{c,0,d}} = \frac{|-1.23|}{14.54} = 0.08 \leq 1.00 \qquad (6.2) \text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{|\sigma_{m,0,d}|}{f_{m,0,d}} = \left(\frac{-1.23}{14.54}\right)^2 + \frac{|0.25|}{16.62} = 0.02 \leq 1.00 \qquad (6.19) \text{ - OK}$$

Shear, xy - 6.1.7

Panel: 'TP. 9.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT3

$$\frac{|\tau_{xy,d}|}{f_{xy,d}} = \frac{|-0.63|}{2.77} = 0.23 \leq 1.00 \qquad (6.13) \text{ - OK}$$

Shear, xz - 6.1.7

Panel: 'TP. 9.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', $k_{mod} = 0.90$, PT4

$$\frac{|\tau_{xz,d}|}{f_{v,d}} = \frac{|-0.20|}{2.08} = 0.10 \leq 1.00 \qquad (6.13) \text{ - OK}$$

Shear, yz - 6.1.7

Panel: 'TP. 9.1', Layer: '4', LC: 'ULS 6.10b – NL 1 – Vind Y – ', $k_{mod} = 0.90$, PT5

$$\frac{|\tau_{yz,d}|}{f_{R,d}} = \frac{|-0.21|}{1.04} = 0.20 \leq 1.00 \qquad (6.13) \text{ - OK}$$

Shear interaction

Panel: 'TP. 9.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, PT3

$$\left(\frac{\tau_{xy,d}}{f_{xy,d}}\right)^2 + \left(\frac{\tau_{xz,d}}{f_{v,d}}\right)^2 = \left(\frac{-0.63}{2.77}\right)^2 + \left(\frac{0.00}{2.77}\right)^2 = 0.05 \leq 1.00 \text{ - OK}$$

Tension and shear

Panel: 'TP. 9.1', Layer: '4', LC: 'ULS 6.10b – NL 1 – Vind Y – ', $k_{mod} = 0.90$, PT5

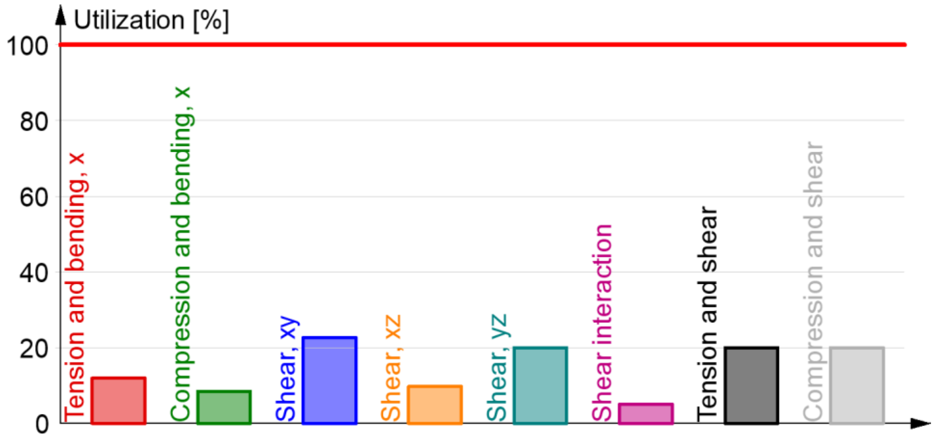
$$\frac{\sigma_{t,90,d}}{f_{t,90,d}} + \frac{|\tau_{yz,d}|}{f_{R,d}} = \frac{0.00}{0.28} + \frac{|-0.21|}{1.04} = 0.20 \leq 1.00 \text{ - OK}$$

Compression and shear

Panel: 'TP. 9.1', Layer: '4', LC: 'ULS 6.10b – NL 1 – Vind Y – ', $k_{mod} = 0.90$, PT5

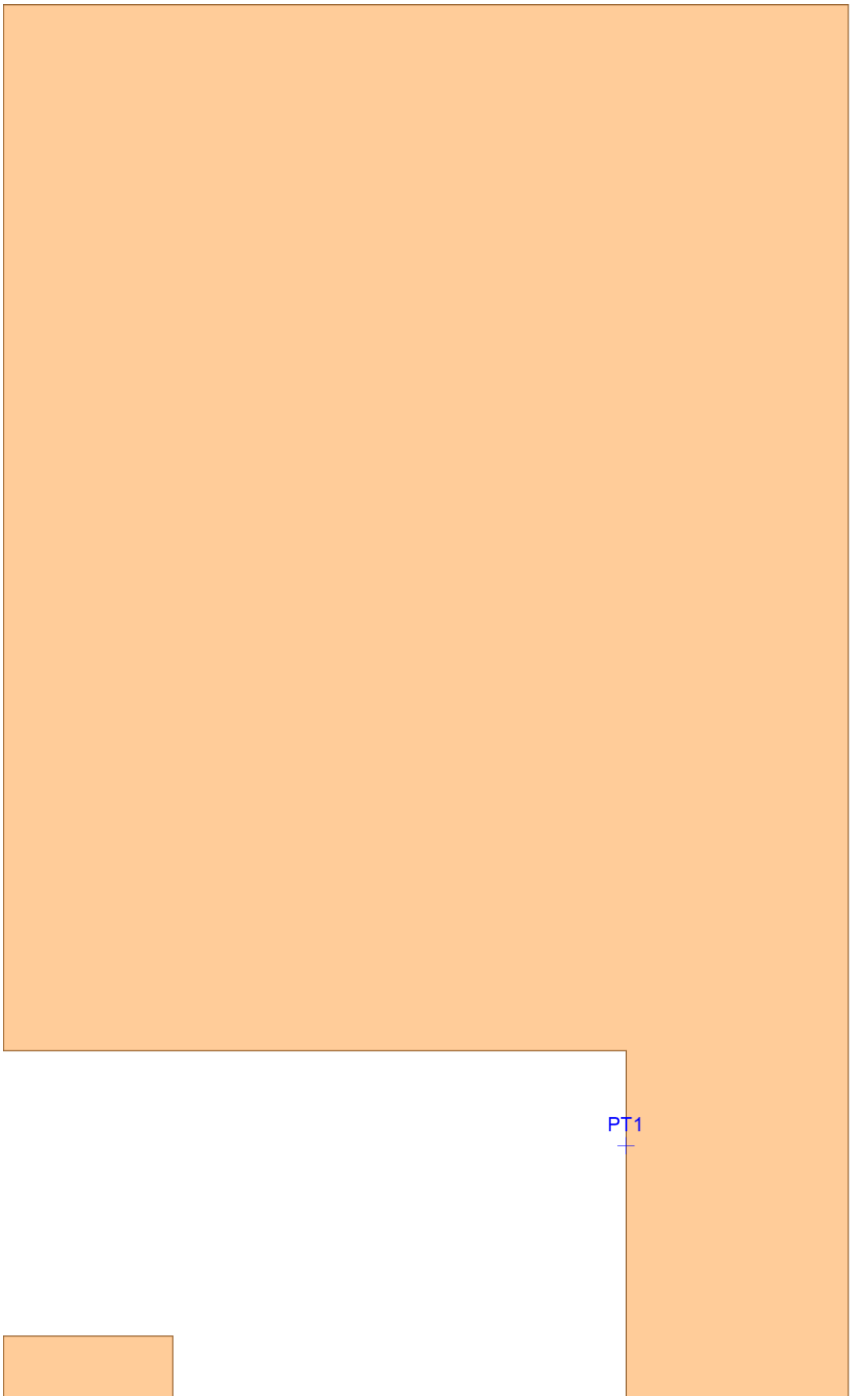
$$\frac{|\sigma_{c,90,d}|}{f_{c,90,d}} + \frac{|\tau_{yz,d}|}{f_{R,d}} = \frac{|0.00|}{1.38} + \frac{|-0.21|}{1.04} = 0.20 \leq 1.00 \text{ - OK}$$

Summary


Utilization [%]
100
80
60
40
20
0
Tension and bending, x
Compression and bending, x
Shear, xy
Shear, xz
Shear, yz
Shear interaction
Tension and shear
Compression and shear

Most loaded OUTER WALL (YV2.1)
Maximum of load combinations

Geometry



Maximum nodes:
PT1 (0.00, 7.64, 5.41) [m]
PT2 (0.00, 3.93, 4.20) [m]
PT3 (0.00, 3.93, 3.80) [m]
PT4 (0.00, 3.52, 3.80) [m]

Node numbers:
PT1: 6660
PT2: 5507
PT3: 5520
PT4: 5374

Panel type:
180-5s

Total thickness:
t = 180.00 mm

Panel properties

Service class: 1, $\gamma_{M,ult.} = 1.30$, $\gamma_{M,acc./seis.} = 1.00$, $k_{sys} = 1.00$

No	Material	Thickness [mm]	Theta [°]	Rho [kg/m^3]
1	C24	30	0	420
2	C14	45	90	350
3	C24	30	0	420
4	C14	45	90	350
5	C24	30	0	420

Mechanical properties

No	$E_{0,mean}$ [N/mm^2]	$E_{90,mean}$ [N/mm^2]	ν_{xy} [-]	$G_{xy,mean}$ [N/mm^2]	$G_{xz,mean}$ [N/mm^2]	$G_{yz,mean}$ [N/mm^2]
1	11000	0	0.00	690	690	69
2	7000	0	0.00	440	440	44
3	11000	0	0.00	690	690	69
4	7000	0	0.00	440	440	44
5	11000	0	0.00	690	690	69

Limit stresses

No	$f_{m,0,k}$ [N/mm	$f_{m,90,k}$ [N/mm	$f_{t,0,k}$ [N/mm	$f_{t,90,k}$ [N/mm	$f_{c,0,k}$ [N/mm	$f_{c,90,k}$ [N/mm	$f_{xy,k}$ [N/mm	$f_{v,k}$ [N/mm	$f_{vR,k}$ [N/mm
1	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
2	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
3	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
4	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
5	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00

Tension and bending, x - 6.2.3

Panel: 'YV2.1.1', Layer: '4', LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, PT1

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\frac{1.23}{4.98}+\frac{|0.00|}{9.69}=0.25\leq 1.00 \qquad (6.17)\text{ - OK}$$

Compression and bending, x - 6.1.4, 6.2.4

Panel: 'YV2.1.1', Layer: '5', LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, PT2

$$\frac{|\sigma_{c,0,d}|}{f_{c,0,d}}=\frac{|-6.62|}{14.54}=0.46\leq 1.00 \qquad (6.2)\text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\left(\frac{-6.62}{14.54}\right)^2+\frac{|-0.00|}{16.62}=0.21\leq 1.00 \qquad (6.19)\text{ - OK}$$

Shear, xy - 6.1.7

Panel: 'YV2.1.1', Layer: '5', LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, PT3

$$\frac{|\tau_{xy,d}|}{f_{xy,d}}=\frac{|-0.54|}{2.77}=0.20\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear, xz - 6.1.7

Panel: 'YV2.1.1', Layer: '2', LC: 'Stability Vind X + ', $k_{mod} = 0.90$, PT4

$$\frac{|\tau_{xz,d}|}{f_{v,d}}=\frac{|0.14|}{2.08}=0.07\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear, yz - 6.1.7

Panel: 'YV2.1.1', Layer: '3', LC: 'Stability Vind X + ', $k_{mod} = 0.90$, PT4

$$\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|0.14|}{1.38}=0.10\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear interaction

Panel: 'YV2.1.1', Layer: '5', LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, PT3

$$\left(\frac{\tau_{xy,d}}{f_{xy,d}}\right)^2+\left(\frac{\tau_{xz,d}}{f_{v,d}}\right)^2=\left(\frac{-0.54}{2.77}\right)^2+\left(\frac{0.00}{2.77}\right)^2=0.04\leq 1.00\text{ - OK}$$

Tension and shear

Panel: 'YV2.1.1', Layer: '3', LC: 'Stability Vind X + ', $k_{mod} = 0.90$, PT4

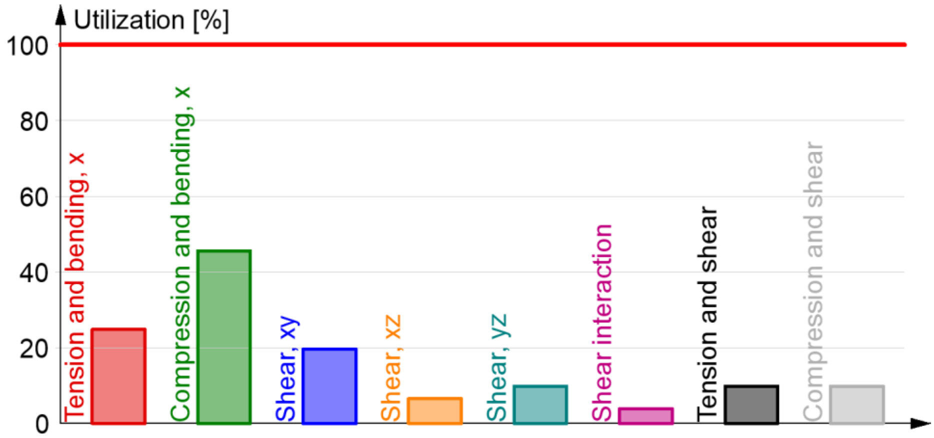
$$\frac{\sigma_{t,90,d}}{f_{t,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{0.00}{0.28}+\frac{|0.14|}{1.38}=0.10\leq 1.00\text{ - OK}$$

Compression and shear

Panel: 'YV2.1.1', Layer: '3', LC: 'Stability Vind X + ', $k_{mod} = 0.90$, PT4

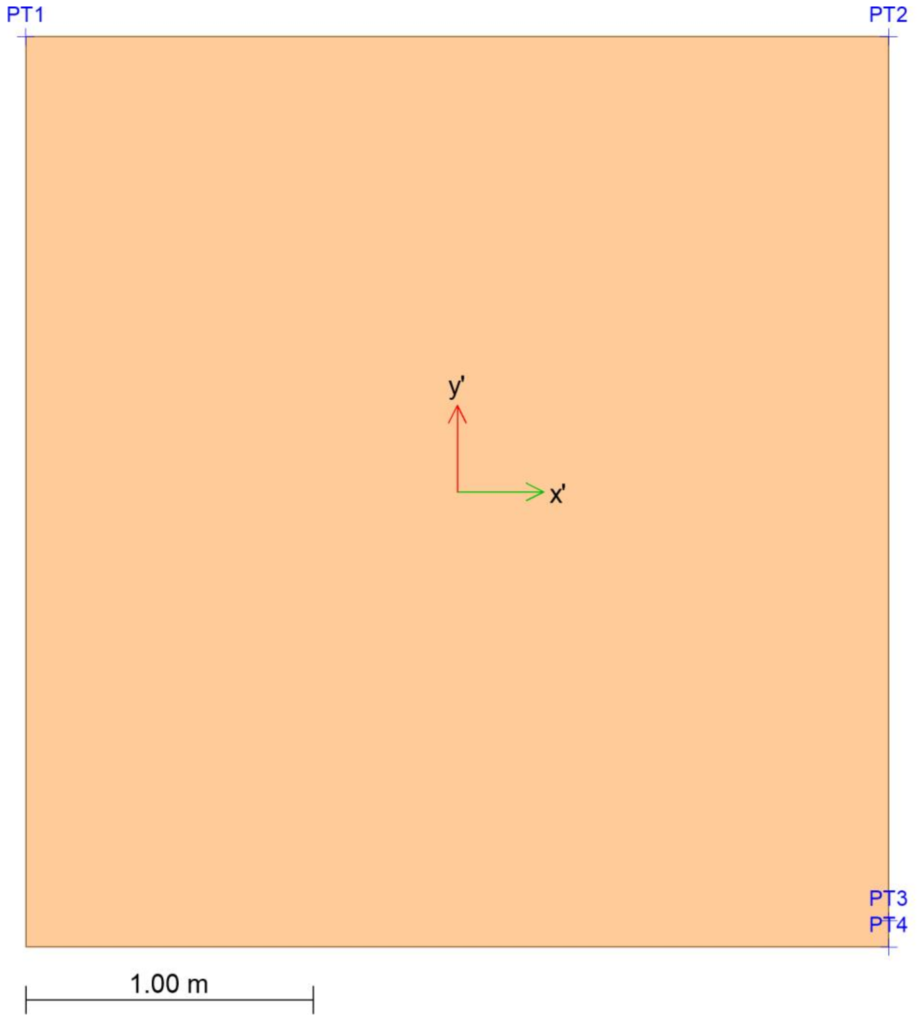
$$\frac{|\sigma_{c,90,d}|}{f_{c,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|0.00|}{1.73}+\frac{|0.14|}{1.38}=0.10\leq 1.00\text{ - OK}$$

Summary


Utilization [%]
100
80
60
40
20
0
Tension and bending, x
Compression and bending, x
Shear, xy
Shear, xz
Shear, yz
Shear interaction
Tension and shear
Compression and shear

Most loaded INNER WALL (IV2.61)
Maximum of load combinations

Geometry



x', y': Shell local system

Maximum nodes:
PT1 (6.45, 7.21, 18.20) [m]
PT2 (6.45, 7.21, 21.20) [m]
PT3 (6.45, 4.14, 21.20) [m]
PT4 (6.45, 4.05, 21.20) [m]

Node numbers:
PT1: 31966
PT2: 31986
PT3: 30249
PT4: 30115

Panel type:
180-5s

Total thickness:
$t = 180.00$ mm

Panel properties

Service class: 1, $\gamma_{M,ult.} = 1.30$, $\gamma_{M,acc./seis.} = 1.00$, $k_{sys} = 1.00$

No	Material	Thickness [mm]	Theta [°]	Rho [kg/m^3]
1	C24	30	0	420
2	C14	45	90	350
3	C24	30	0	420
4	C14	45	90	350
5	C24	30	0	420

Mechanical properties

No	$E_{0,mean}$ [N/mm^2]	$E_{90,mean}$ [N/mm^2]	ν_{xy} [-]	$G_{xy,mean}$ [N/mm^2]	$G_{xz,mean}$ [N/mm^2]	$G_{yz,mean}$ [N/mm^2]
1	11000	0	0.00	690	690	69
2	7000	0	0.00	440	440	44
3	11000	0	0.00	690	690	69
4	7000	0	0.00	440	440	44
5	11000	0	0.00	690	690	69

Limit stresses

No	$f_{m,0,k}$ [N/mm	$f_{m,90,k}$ [N/mm	$f_{t,0,k}$ [N/mm	$f_{t,90,k}$ [N/mm	$f_{c,0,k}$ [N/mm	$f_{c,90,k}$ [N/mm	$f_{xy,k}$ [N/mm	$f_{v,k}$ [N/mm	$f_{vR,k}$ [N/mm
1	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
2	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
3	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
4	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
5	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00

Tension and bending, x - 6.2.3

Panel: 'IV2.61.1', Layer: '5', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT1

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\frac{1.69}{10.04}+\frac{|0.00|}{16.62}=0.17\leq 1.00 \qquad (6.17)\text{ - OK}$$

Compression and bending, x - 6.1.4, 6.2.4

Panel: 'IV2.61.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT2

$$\frac{|\sigma_{c,0,d}|}{f_{c,0,d}}=\frac{|-6.68|}{14.54}=0.46\leq 1.00 \qquad (6.2)\text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\left(\frac{-6.68}{14.54}\right)^2+\frac{|0.00|}{16.62}=0.21\leq 1.00 \qquad (6.19)\text{ - OK}$$

Shear, xy - 6.1.7

Panel: 'IV2.61.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT2

$$\frac{|\tau_{xy,d}|}{f_{xy,d}}=\frac{|-1.09|}{2.77}=0.39\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear, xz - 6.1.7

Panel: 'IV2.61.1', Layer: '4', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT3

$$\frac{|\tau_{xz,d}|}{f_{v,d}}=\frac{|-0.00|}{2.08}=0.00\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear, yz - 6.1.7

Panel: 'IV2.61.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT4

$$\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|-0.00|}{1.04}=0.00\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear interaction

Panel: 'IV2.61.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT2

$$\left(\frac{\tau_{xy,d}}{f_{xy,d}}\right)^2+\left(\frac{\tau_{xz,d}}{f_{v,d}}\right)^2=\left(\frac{-1.09}{2.77}\right)^2+\left(\frac{0.00}{2.77}\right)^2=0.16\leq 1.00\text{ - OK}$$

Tension and shear

Panel: 'IV2.61.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT4

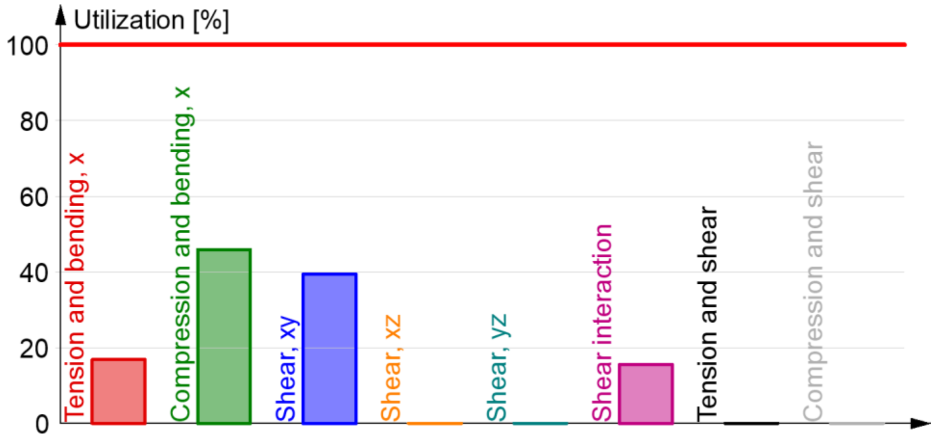
$$\frac{\sigma_{t,90,d}}{f_{t,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{0.00}{0.28}+\frac{|-0.00|}{1.04}=0.00\leq 1.00\text{ - OK}$$

Compression and shear

Panel: 'IV2.61.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT4

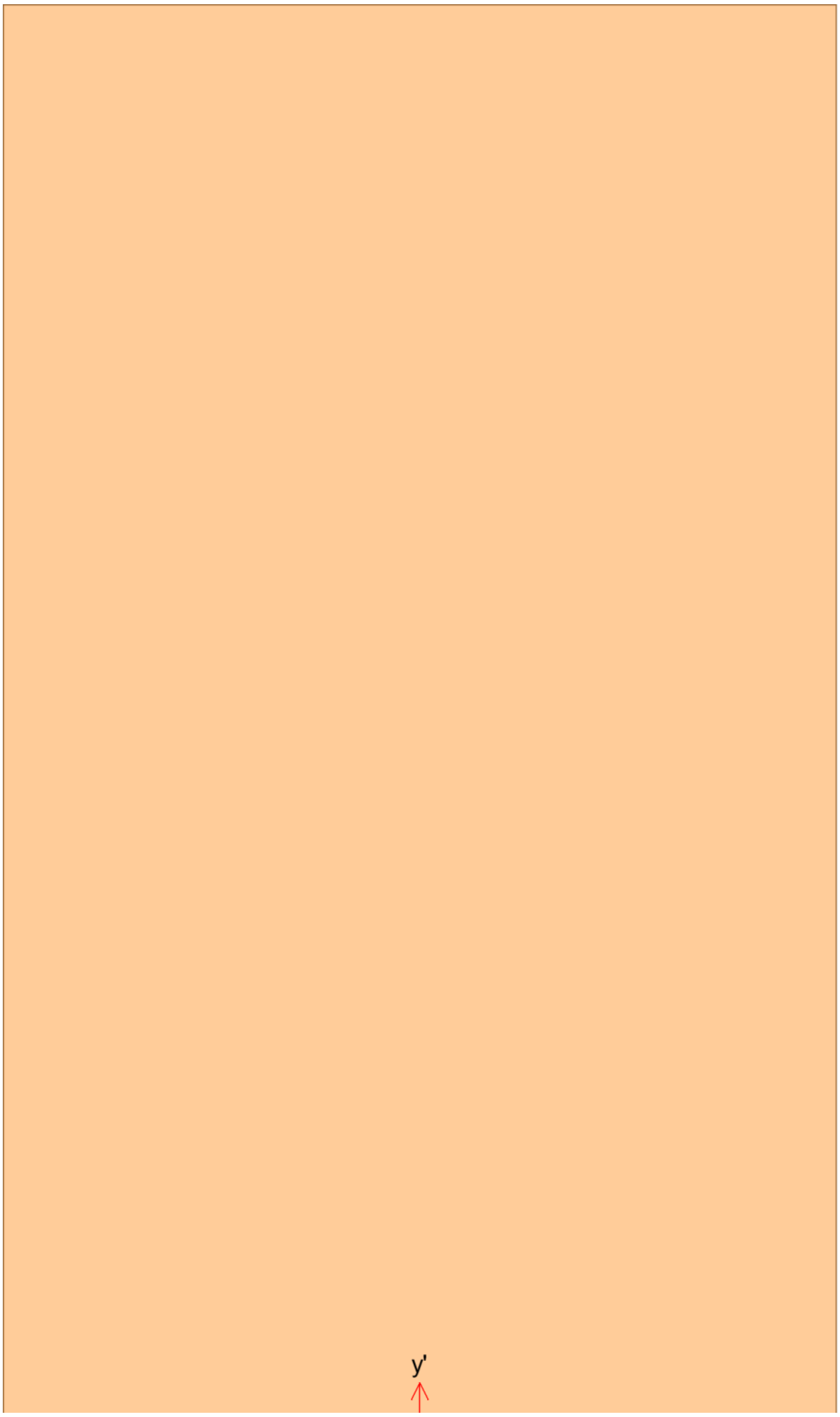
$$\frac{|\sigma_{c,90,d}|}{f_{c,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|0.00|}{1.38}+\frac{|-0.00|}{1.04}=0.00\leq 1.00\text{ - OK}$$

Summary


Utilization [%]
100
80
60
40
20
0
Tension and bending, x
Compression and bending, x
Shear, xy
Shear, xz
Shear, yz
Shear interaction
Tension and shear
Compression and shear

Most loaded FLOOR (Floor2.36)
Maximum of load combinations

Geometry



Maximum nodes:
PT1 (16.82, 5.89, 21.20) [m]
PT2 (12.56, 6.35, 21.20) [m]
PT3 (12.56, 6.79, 21.20) [m]

Node numbers:
PT1: 75938
PT2: 57490
PT3: 57185

Panel type:
160-5s

Total thickness:
t = 160.00 mm

Panel properties

Service class: 1, $\gamma_{M,ult.} = 1.25$, $\gamma_{M,acc./seis.} = 1.00$, $k_{sys} = 1.00$

No	Material	Thickness [mm]	Theta [°]	Rho [kg/m^3]
1	C24	40	0	420
2	C14	20	90	350
3	C24	40	0	420
4	C14	20	90	350
5	C24	40	0	420

Mechanical properties

No	$E_{0,mean}$ [N/mm^2]	$E_{90,mean}$ [N/mm^2]	ν_{xy} [-]	$G_{xy,mean}$ [N/mm^2]	$G_{xz,mean}$ [N/mm^2]	$G_{yz,mean}$ [N/mm^2]
1	11000	0	0.00	690	690	69
2	7000	0	0.00	440	440	44
3	11000	0	0.00	690	690	69
4	7000	0	0.00	440	440	44
5	11000	0	0.00	690	690	69

Limit stresses

No	$f_{m,0,k}$ [N/mm	$f_{m,90,k}$ [N/mm	$f_{t,0,k}$ [N/mm	$f_{t,90,k}$ [N/mm	$f_{c,0,k}$ [N/mm	$f_{c,90,k}$ [N/mm	$f_{xy,k}$ [N/mm	$f_{v,k}$ [N/mm	$f_{vR,k}$ [N/mm
1	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
2	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
3	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
4	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
5	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00

Tension and bending, x - 6.2.3

Panel: 'Floor2.36.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT1

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\frac{2.04}{5.18}+\frac{|-0.69|}{10.08}=0.46\leq1.00 \qquad (6.17)\text{ - OK}$$

Compression and bending, x - 6.1.4, 6.2.4

Panel: 'Floor2.36.1', Layer: '4', LC: 'ULS 6.10b – NL 1 – Vind X – ', k_{mod} = 0.90, PT1

$$\frac{|\sigma_{c,0,d}|}{f_{c,0,d}}=\frac{|-2.09|}{11.52}=0.18\leq1.00 \qquad (6.2)\text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\left(\frac{-2.09}{11.52}\right)^2+\frac{|0.69|}{10.08}=0.10\leq1.00 \qquad (6.19)\text{ - OK}$$

Shear, xy - 6.1.7

Panel: 'Floor2.36.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT2

$$\frac{|\tau_{xy,d}|}{f_{xy,d}}=\frac{|0.30|}{2.88}=0.10\leq1.00 \qquad (6.13)\text{ - OK}$$

Shear, xz - 6.1.7

Panel: 'Floor2.36.1', Layer: '4', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT3

$$\frac{|\tau_{xz,d}|}{f_{v,d}}=\frac{|0.41|}{2.16}=0.19\leq1.00 \qquad (6.13)\text{ - OK}$$

Shear, yz - 6.1.7

Panel: 'Floor2.36.1', Layer: '3', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT3

$$\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|0.41|}{1.44}=0.28\leq1.00 \qquad (6.13)\text{ - OK}$$

Shear interaction

Panel: 'Floor2.36.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT3

$$\left(\frac{\tau_{xy,d}}{f_{xy,d}}\right)^2+\left(\frac{\tau_{xz,d}}{f_{v,d}}\right)^2=\left(\frac{-0.05}{2.16}\right)^2+\left(\frac{0.41}{2.16}\right)^2=0.04\leq1.00\text{ - OK}$$

Tension and shear

Panel: 'Floor2.36.1', Layer: '3', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT3

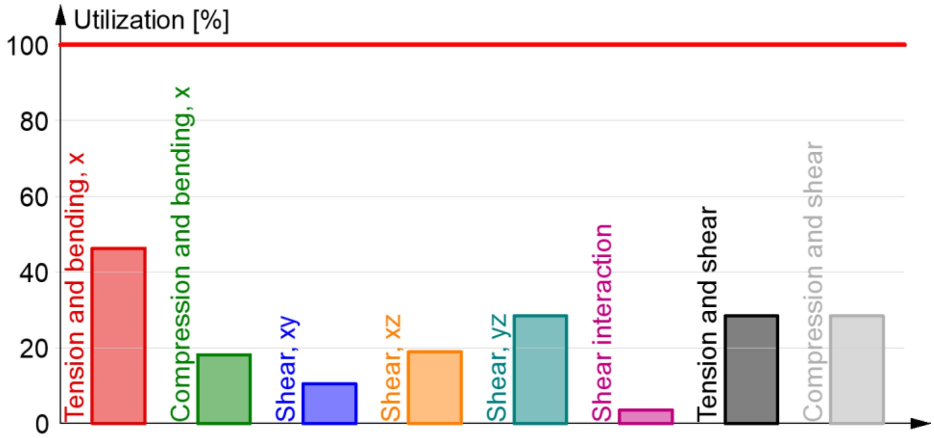
$$\frac{\sigma_{t,90,d}}{f_{t,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{0.00}{0.29}+\frac{|0.41|}{1.44}=0.28\leq1.00\text{ - OK}$$

Compression and shear

Panel: 'Floor2.36.1', Layer: '3', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT3

$$\frac{|\sigma_{c,90,d}|}{f_{c,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|0.00|}{1.80}+\frac{|0.41|}{1.44}=0.28\leq1.00\text{ - OK}$$

Summary


Utilization [%]
100
80
60
40
20
0
Tension and bending, x
Compression and bending, x
Shear, xy
Shear, xz
Shear, yz
Shear interaction
Tension and shear
Compression and shear

E

Hand calculations Gasklockan

Floorbeam middle

Indata

Laster		
g_k	1,03	kN/m^2
q_k	2,5	kN/m^2

Rumsgeometri		
"Influensbredd"	9,1	m
l	7	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	277,9	kNm
V_{Ed}	158,8	kN

Balktvärsnitt		
h	720	mm
b	215	mm
Föreligger risk för vippning?		Nej

Materialdata		
Materialtyp	Limträ	-
f_{mk}	30	MPa
f_{vk}	3,5	MPa
E_{mean}	13	GPa

Koefficienter och faktorer		
Exponerat för nederbörd/solstrålning?		Nej
Klimatklass	1	-
Lastvaraktighet	Korttid	-
γ_m	1,25	-
k_{mod}	0,9	-
k_h	1,0	-
k_{cr}	0,67	-
k_{def}	0,6	-
ψ_2	0,3	-

ULS Beräkningar

Momentkapacitet		
f_{md}	21,6	MPa
W	18576000	mm^3
M_{Rd}	401,2	kNm
σ_{Md}	15,0	MPa
Kontroll momentkapacitet		OK
Utnyttjandegrad		69,26%

Tvärkraftskapacitet		
f_{vd}	2,52	MPa
b_{ef}	144,1	mm
A_{ef}	103716,0	mm^2
V_{Rd}	174,2	kN
τ_d	2,3	MPa
Kontroll tvärkraftskapacitet		OK
Utnyttjandegrad		91,14%

SLS Beräkningar

Nedböjning krav		
Slutlig krav: L/		300
w_{fin}	23,3	mm

Beräknad nedböjning		
w_{ginst}	3,4	mm
w_{qinst}	8,2	mm
w_{gfin}	5,4	mm
w_{qfin}	9,7	mm
$w_{insttot}$	11,6	mm
w_{fintot}	15,0	mm
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		64,49%

Inclined side beam

Indata

Laster		
P_{gk}	26,8	kN
P_{snow}	19,8	kN
P_{imp}	31,0	kN
P_{Ed}	99,4	kN

Rumsgeometri		
Balkens längd	6,874	m
Pelarens placering, a	3,187	m
b	3,687	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	169,9	kNm
V_{Ed}	53,3	kN

Balktvärsnitt		
h	540	mm
b	215	mm
Föreligger risk för vippning?		Nej

Materialdata		
Materialtyp	Limträ	-
f_{mk}	30	MPa
f_{vk}	3,5	MPa
E_{mean}	13	GPa

Koefficienter och faktorer		
Exponerat för nederbörd/solstrålning?		Nej
Klimatklass	1	-
Lastvaraktighet	Korttid	-
γ_m	1,25	-
k_{mod}	0,9	-
k_h	1,0	-
k_{cr}	0,67	-
k_{def}	0,6	-
ψ_2	0,3	-

ULS Beräkningar

Momentkapacitet		
f_{md}	21,8	MPa
W	10449000	mm^3
M_{Rd}	228,1	kNm
σ_{Md}	16,3	MPa
Kontroll momentkapacitet		OK
Utnyttjandegrad		74%

Tvärkraftskapacitet		
f_{vd}	2,52	MPa
b_{ef}	144,1	mm
A_{ef}	77787,0	mm^2
V_{Rd}	130,7	kN
τ_d	1,03	MPa
Kontroll tvärkraftskapacitet		OK
Utnyttjandegrad		40,8%

SLS Beräkningar

Nedböjning krav		
Slutlig krav: L/		300
w_{fin}	22,9	mm

Beräknad nedböjning		
w_{ginst}	4,9	mm
w_{qinst}	8,2	mm
w_{gfin}	7,8	mm
w_{qfin}	9,7	mm
$w_{insttot}$	13,1	mm
w_{fintot}	17,5	mm
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		76,27%

Double tapered roof beam

Indata

Laster		
g_k	1,03	kN/m^2
g_k	7,21	kN/m
q_k	1,6	kN/m^2
q_k	11,2	kN/m

Rumsgeometri		
"Influensbredd"	7	m
l	18,2	m

Dimensionerande lasteffekter brottgräns		
$M_{ap,d}$	1053,84006	kNm
V_{Ed}	231,6132	kN

Balktvärsnitt		
hap	1575	mm
h_i	1035	mm
b	215	mm
Iy	19868997778	mm4
Föreligger risk för vippning?		Nej

Materialdata		
Materialtyp	Limträ	-
f_{mk}	30	MPa
$f_{t,90,k}$	0,5	MPa
f_{vk}	3,5	MPa
$f_{c,90,k}$	2,5	MPa
E_{mean}	13000	GPa
$E_{0,mean}$	13000	GPa
G_{mean}	650	GPa

Koefficienter och faktorer		
Exponerat för nederbörd/solstrålning?		Nej
Klimatklass	1	-
Lastvaraktighet	Korttid	-
γ_m	1,25	-
k_{mod}	0,9	-
k_h	1,0	-
k_{cr}	0,67	-
k_{def}	0,6	-
ψ_2	0,2	-
α_{ap}	3,4	-

ULS Beräkningar

Korrektionsfaktorer moment		
kr	1,0	-
k1	1,1	-
kl	1,1	-

orrektionsfaktorer (dragspänning vinkelrät mot fibern		
kdis	1,4	-
v0	0,01	m3
V	0,5333	m3
kvol	0,45	-
k5	0,0119	-
kp	0,01	-

Momentkapacitet		
f_{md}	21,6	MPa
kr * fmd	21,6	MPa
sMd	13,07	MPa
Kontroll momentkapacitet		OK
Utnyttjandegrad		60,5%

Kapacitet dragning vinkelrätt mot fiberna		
$f_{t,90,d}$	0,4	MPa
ft,90,rd	0,2	mm
st,90,d	0,14	MPa
Kontroll tvärkraftskapacitet		OK
Utnyttjandegrad		39,08%

Skjuvkapacitet		
f_{vd}	2,5	MPa
b_{ef}	144,1	mm
A_{ef}	149102,9	mm^2
V_{Rd}	250,5	kN
τ_d	2,33	MPa
Kontroll tvärkraftskapacitet		OK
Utnyttjandegrad		92,46%

SLS Beräkningar

Nedböjning krav		
Slutlig krav: L/		300
w_{fin}	60,7	mm

Beräknad nedböjning		
X	1,2	-
km	0,40	-
kv	0,86	mm
$w_{g,inst}$	21,3	mm
$w_{q,inst}$	33,1	mm
w_{fin}	71,2	mm
Kontroll slutlig nedböjning		EJ OK
Utnyttjandegrad		117,38%

Note: deflection criteria not passed in handcalculation, but passed in FEM model

Roof supporting column

Indata

Laster		
Vertikal last	3,222	kN/m^2

Geometri och materialdata		
Materialtyp	Limträ	-
Antal våningar	1	st
Inf. Area	76,09	m^2
h	270	mm
b	215	mm
$f_{c,0,k}$	24	MPa
$f_{m,k}$	30	MPa
Lcr	3,83	m
E0,05	10800	MPa

Koefficienter och faktorer		
Exponerat för nederbörd/solstrålning?		Nej
Klimatklass	1	-
Lastvaraktighet	Korttid	-
γ_m	1,25	-
k_{mod}	0,9	-
β	0,1	-

Beräkningar

Knäckning		
i	0,06	mm
λ	61,71	-
λ_{rel}	0,93	-
k	0,96	
kc	0,82	
$f_{c,0,d}$	17,28	Mpa
Ned	245,16	kN
$\sigma_{c,0,d}$	4,2233	Mpa
Behövs det ta hänsyn till knäckning?		Ja
Kontroll med hänsyn till knäckning		30%

Floor supporting short column

Indata

Laster		
Vertikal last	4,986	kN/m^2

Geometri och materialdata		
Materialtyp	Limträ	-
Antal våningar	1	st
Inf. Area	63,7	m^2
h	270	mm
b	215	mm
$f_{c,0,k}$	24	MPa
$f_{m,k}$	30	MPa
Lcr	0,82	m
E0,05	10800	MPa

Koefficienter och faktorer		
Exponerat för nederbörd/solstrålning?		Nej
Klimatklass	1	-
Lastvaraktighet	Korttid	-
γ_m	1,25	-
k_{mod}	0,9	-
β	0,1	-

Beräkningar

Knäckning		
i	0,06	mm
λ	13,21	-
λ_{rel}	0,20	-
k	0,51	
kc	1,00	
$f_{c,0,d}$	17,28	Mpa
Ned	317,61	kN
$\sigma_{c,0,d}$	5,4713	Mpa
Behövs det ta hänsyn till knäckning?		Nej
Kontroll utan hänsyn till knäckning		32%

Floor CLT rib panel

Indata

Laster		
q_k	2,5	kN/m^2
g_{kextra}	0	kN/m^2
g_{ktot}	0,96	kN/m^2

Rumsgeometri		
Upplag	Fritt upplagd	
l	9,1	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	30,5	kNm/balk
V_{Ed}	13,4	kN/balk

Materialdata CLT platta		
Materialtyp	Konstruktionsvirke	-
$f_{c,k,xlay}$	21	MPa
$f_{v,k,xlay}$	4	MPa
$f_{r,k,ylay}$	2	MPa
E_{mean}	11	GPa
ρ_{platta}	480	kg/m^3

Materialdata balk		
Materialtyp	Limträ	-
f_{ck}	24,5	MPa
f_{tk}	19,5	MPa
f_{vk}	3,5	MPa
E_{mean}	13	GPa
G_{mean}	0,65	GPa
ρ_{balk}	430	kg/m^3

Tvärsnittets uppbyggnad		
t1	40	mm
t2	40	mm
t3	40	mm
h_{balk}	405	mm
b_{balk}	140	mm
cc_{balk}	600	mm

Tvärsnittsberäkningar

Gemensamma data		
γ_m	1,25	-
Klimatklass	1	-
Lastvaraktighet	Medellång	-
k_{mod}	0,9	-
k_{def}	0,85	-
k_{sys}	1,0	-
ψ_2	0,3	-
k_{cr}	0,67	-

Beräkning av effektiv flänsbredd enl. 5.26 dot1		
b_f	460	mm
β_0	1,06	-
β_1	1,8	-
b_{eff}	445,7	mm

Tvärsnittsegenskaper		
h_{platta}	120	mm
b_{efftot}	585,7	mm
h_{tot}	525	mm
z_{tp}	214,5	mm
I_{xnet}	245252	cm^4/balk
S_{max}	6750	cm^3/balk
S_{rx}	3856	cm^3/balk
I_{ynet}	533,3	cm^4/m

ULS Beräkningar

Tryck och drag från böjning		
σ_{wcd}	1,2	MPa
σ_{wtd}	3,9	MPa
σ_{fcdmax}	2,7	MPa
f_{wcd}	17,6	MPa
f_{wtd}	14,0	MPa
f_{fcd}	15,1	MPa
Kontroll tryckspänningar liv		OK
Utnyttjandegrad		6,65%
Kontroll dragspänningar liv		OK
Utnyttjandegrad		27,48%
Kontroll tryckspänningar fläns		OK
Utnyttjandegrad		17,62%

Tvärkraftskapacitet		
$b_{ef,web}$	93,8	mm
$b_{ef,flange}$	220	mm
τ_{vd}	0,39	MPa
τ_{rd}	0,10	MPa
f_{vd}	2,5	MPa
f_{rd}	1,44	MPa
Kontroll parallellskjuvning		OK
Utnyttjandegrad		15,60%
Kontroll rullskjuvning		OK
Utnyttjandegrad		6,65%

SLS Beräkningar

Nedböjning krav		
Slutlig krav: L/		300
w_{fin}	30,3	mm

Beräknad nedböjning		
w_{ginst}	1,8	mm
w_{qinst}	4,7	mm
w_{gfin}	3,4	mm
w_{qfin}	5,9	mm
$w_{insttot}$	6,5	mm
w_{fintot}	9,3	mm
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		30,54%

Vibration och svikt		
Bredd	10	m
$m_{längd}$	58,9	kg/m
m_{yta}	98,2	kg/m2
f1	14,0	Hz
n40	9,88	st
ν	0,003	m/Ns2
b	100	m/Ns2
ξ	0,01	-
ν_{max}	0,019	m/Ns2
a	1,5	mm/kN
w_{inst}	0,30	mm/kN
Kontroll f1		OK
Kontroll vibrationer		OK
Utnyttjandegrad		14,57%
Kontroll svikt		OK
Utnyttjandegrad		19,70%

Stresses in steel bracings wind X+		
Total height	3,6	m
Total length	25,2	m
Tie inclination 1	35,5	deg
Tie inclination 2	23,2	deg
Cross-section area	236,0	mm^2
Charateristic load on windward side	0,7	kN/m^2
Characteristic load on leeward side	0,3	kN/m^2
Influencearea for one supporting unit	22,7	m^2
Design load on supporting unit 1	24,5	kN
Design load on supporting unit 2	9,2	kN
Tie force 1	30,1	kN
Tie force 2	10,0	kN
Tensile stress tie 1	127,5	MPa
Tensile stress tie 2	42,3	MPa

Stresses in steel bracings wind Y+		
Total height	3	m
Total length	60,2	m
Tie inclination	18,2	deg
Cross-section area	236,0	mm^2
Characteristic load (windward+leeward)	1,2	kN/m^2
Influencearea for one supporting unit	45,2	m^2
Design load on one supporting unit	40,6	kN
Tie force	42,8	kN
Tensile stress tie	181,3	MPa

F

FEM calculations Gasklockan

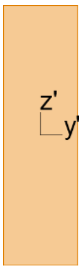
B.8.1 Floor beam (middle)
Maximum of load combinations

GL 30c

(Glued laminated), Service class 1

$E_{0,05}$	$=10800\ \mathrm{N/mm^2}$	$f_{t,90,k}$	$= 0.50\ \mathrm{N/mm^2}$
$G_{0,05}$	$= 540\ \mathrm{N/mm^2}$	$f_{c,0,k}$	$=24.50\ \mathrm{N/mm^2}$
γ_M	$= 1.25$	$f_{c,90,k}$	$= 2.50\ \mathrm{N/mm^2}$
$\gamma_{M,acc./seis.}$	$= 1.00$	$f_{v,k}$	$= 3.50\ \mathrm{N/mm^2}$
k_{sys}	$= 1.00$		

Glulam 215x720



A	$= 154800\ \mathrm{mm^2}$	$f_{t,0,k}$	$=19.50\ \mathrm{N/mm^2}$
W_1	$=1.858e+07\ \mathrm{mm^3}$	$f_{m,1,k}$	$=30.00\ \mathrm{N/mm^2}$
W_2	$=5.547e+06\ \mathrm{mm^3}$	$f_{m,2,k}$	$=33.00\ \mathrm{N/mm^2}$
i_1	$= 208\ \mathrm{mm}$		
i_2	$= 62\ \mathrm{mm}$		
I_2	$=5.963e+08\ \mathrm{mm^4}$		
I_t	$=1.936e+09\ \mathrm{mm^4}$		

Combined bending and axial tension - 6.2.3

LC: 'ULS 6.10b – NL 1 – Vind X – ', $k_{mod} = 0.90$, $x = 3591.00$ mm

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.00}{14.04}+\frac{14.17}{21.60}+0.70\frac{0.00}{23.76}=0.66\le 1.00 \qquad (6.17)\text{ - OK}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+k_m\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.00}{14.04}+0.70\frac{14.17}{21.60}+\frac{0.00}{23.76}=0.46\le 1.00 \qquad (6.18)\text{ - OK}$$

Combined bending and axial compression - 6.1.4, 6.2.4

LC: 'ULS 6.10b – NL 1 – Vind Y – ', $k_{mod} = 0.90$, $x = 3290.98$ mm

$\sigma_{c,0,d}=0.01\ \mathrm{N/mm^2}\le f_{c,0,d}=17.64\ \mathrm{N/mm^2}$ (6.2) - OK

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{0.01}{17.64}\right)^2+\frac{14.23}{21.60}+0.70\frac{0.00}{23.76}=0.66\le 1.00 \qquad (6.19)\text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+k_m\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{0.01}{17.64}\right)^2+0.70\frac{14.23}{21.60}+\frac{0.00}{23.76}=0.46\le 1.00 \qquad (6.20)\text{ - OK}$$

Combined shear and torsion - 6.1.7, 6.1.8

LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, $x = 7000.00$ mm

$\tau_d=2.47\ \mathrm{N/mm^2}\le f_{v,d}=2.52\ \mathrm{N/mm^2}$ (6.13) - OK

Flexural buckling around axis 1 - 6.3.2

LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, $x = 3290.98$ mm

$\beta_c = 0.1$ (6.29)

$$\lambda_1 = \frac{l_0}{i_1} = \frac{600}{208} = 2.89$$

$$\lambda_{rel,1} = \frac{\lambda_1}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{2.89}{\pi}\sqrt{\frac{24.50}{10800}} = 0.300 \qquad (6.21)$$

$$k_1 = 0.5\left(1 + \beta_c\left(\lambda_{rel,1} - 0.3\right) + \lambda_{rel,1}{}^2\right) = 0.5\left(1 + 0.1\,(0.300 - 0.3) + 0.300^2\right) = 0.545 \qquad (6.27)$$

$$k_{c,1} = \frac{1}{k_1+\sqrt{k_1{}^2-\lambda_{rel,1}{}^2}} = \frac{1}{0.545+\sqrt{0.545^2-0.300^2}} = 1.000 \qquad (6.25)$$

$$\frac{\sigma_{c,0,d}}{k_{c,1}\cdot f_{c,0,d}}+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\cdot\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.01}{1.000\cdot 17.64}+\frac{14.23}{21.60}+0.70\cdot\frac{0.00}{23.76}=0.66\leq 1.00 \qquad (6.23)\text{ - OK}$$

Flexural buckling around axis 2 - 6.3.2

LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, $x = 3290.98$ mm

$$\beta_c = 0.1 \qquad (6.29)$$

$$\lambda_2 = \frac{l_0}{i_2} = \frac{600}{62} = 9.67$$

$$\lambda_{rel,2} = \frac{\lambda_2}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{9.67}{\pi}\sqrt{\frac{24.50}{10800}} = 0.300 \qquad (6.22)$$

$$k_2 = 0.5\left(1 + \beta_c\left(\lambda_{rel,2} - 0.3\right) + \lambda_{rel,2}{}^2\right) = 0.5\left(1 + 0.1\,(0.300 - 0.3) + 0.300^2\right) = 0.545 \qquad (6.28)$$

$$k_{c,2} = \frac{1}{k_2+\sqrt{k_2{}^2-\lambda_{rel,2}{}^2}} = \frac{1}{0.545+\sqrt{0.545^2-0.300^2}} = 1.000 \qquad (6.26)$$

$$\frac{\sigma_{c,0,d}}{k_{c,2}\cdot f_{c,0,d}}+k_m\cdot\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.01}{1.000\cdot 17.64}+0.70\cdot\frac{14.23}{21.60}+\frac{0.00}{23.76}=0.46\leq 1.00 \qquad (6.24)\text{ - OK}$$

Lateral torsional buckling - 6.3.3

LC: 'ULS 6.10b – NL 1 – Vind X – ', $k_{mod} = 0.90$, $x = 3290.98$ mm

$$l_{ef} = l/\frac{12.5\cdot M_{max}}{2.5\cdot M_{max} + 3\cdot M_2 + 4\cdot M_3 + 3\cdot M_4} + 2\cdot h = 600/\frac{12.5\cdot 264.34}{2.5\cdot 264.34 + 3\cdot 254.49 + 4\cdot 257.88 + 3\cdot 261.11} + 2\cdot 720$$
$$= 2028\text{ mm}$$

$$\sigma_{m,crit} = \frac{\pi\sqrt{E_{0,05}\cdot I_2\cdot G_{0,05}\cdot I_t}}{l_{ef}\cdot W_1} = \frac{\pi\sqrt{10800\cdot 5.963e{+}08\cdot 540\cdot 1.936e{+}09}}{2028\cdot 1.858e{+}07} = 216.38\text{ N/mm}^2 \qquad (6.31)$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,1,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30.00}{216.38}} = 0.372 \qquad (6.30)$$

$$\lambda_{rel,m} = 0.372 \leq 0.75 \quad \rightarrow \quad k_{crit} = 1.000 \qquad (6.34)$$

$$\frac{\sigma_{m,1,d}}{k_{crit}\cdot f_{m,1,d}}=\frac{14.23}{1.000\cdot 21.60}=0.66\leq 1.00 \qquad (6.33)\text{ - OK}$$

$$\left(\frac{\sigma_{m,1,d}}{k_{crit}\cdot f_{m,1,d}}\right)^2+\frac{\sigma_{c,0,d}}{k_{c,2}\cdot f_{c,0,d}}=\left(\frac{14.23}{1.000\cdot 21.60}\right)^2+\frac{0.00}{1.00\cdot 17.64}=0.43\leq 1.00 \qquad (6.35)\text{ - OK}$$

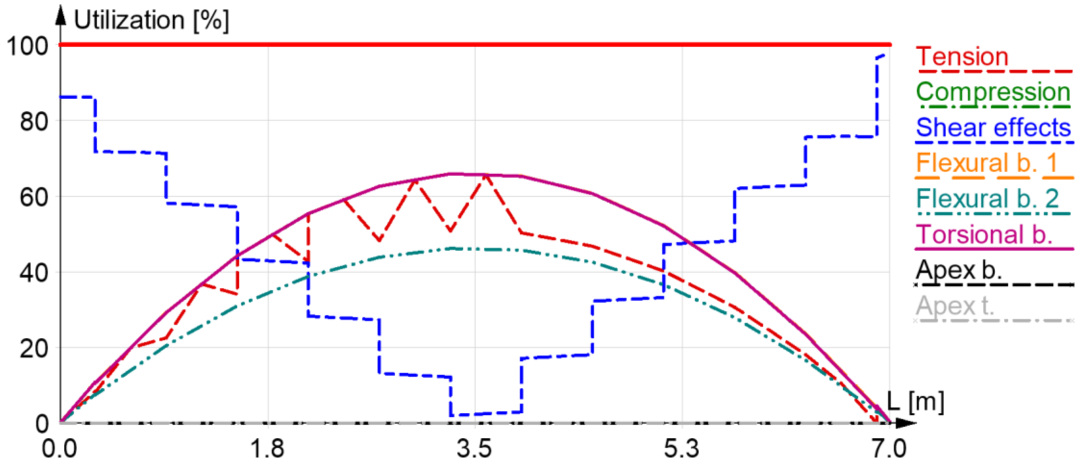
Bending at apex - 6.4.3

Not relevant

Tension at apex - 6.4.3

Not relevant

Summary



Utilization [%]
100
80
60
40
20
0
L [m]
0.0
1.8
3.5
5.3
7.0
Tension
Compression
Shear effects
Flexural b. 1
Flexural b. 2
Torsional b.
Apex b.
Apex t.

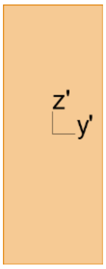
Basebeam.17.1 – Inclined side beam (floor)
Maximum of load combinations

GL 30c

(Glued laminated), Service class 1

$E_{0,05}$	$=10800\ N/mm^2$	$f_{t,90,k}$	$=0.50\ N/mm^2$
$G_{0,05}$	$=540\ N/mm^2$	$f_{c,0,k}$	$=24.50\ N/mm^2$
γ_M	$=1.25$	$f_{c,90,k}$	$=2.50\ N/mm^2$
$\gamma_{M,acc./seis.}$	$=1.00$	$f_{v,k}$	$=3.50\ N/mm^2$
k_{sys}	$=1.00$		

Glulam 215x540



A	$=116100\ mm^2$	$f_{t,0,k}$	$=19.71\ N/mm^2$
W_1	$=1.045e+07\ mm^3$	$f_{m,1,k}$	$=30.32\ N/mm^2$
W_2	$=4.160e+06\ mm^3$	$f_{m,2,k}$	$=33.00\ N/mm^2$
i_1	$=156\ mm$		
i_2	$=62\ mm$		
I_2	$=4.472e+08\ mm^4$		
I_t	$=1.340e+09\ mm^4$		

Combined bending and axial tension - 6.2.3

LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, $x = 3686.73$ mm

$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{1.11}{14.19}+\frac{15.99}{21.83}+0.70\frac{0.60}{23.76}=0.83\leq1.00$ (6.17) - OK

$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+k_m\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{1.11}{14.19}+0.70\frac{15.99}{21.83}+\frac{0.60}{23.76}=0.62\leq1.00$ (6.18) - OK

Combined bending and axial compression - 6.1.4, 6.2.4

Not relevant

Combined shear and torsion - 6.1.7, 6.1.8

LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, $x = 6873.60$ mm

$\tau_d=1.03\ N/mm^2\leq f_{v,d}=2.52\ N/mm^2$ (6.13) - OK

Flexural buckling around axis 1 - 6.3.2

Not relevant

Flexural buckling around axis 2 - 6.3.2

Not relevant

Lateral torsional buckling - 6.3.3

LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, $x = 3686.73$ mm

$$l_{ef} = l / \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_2 + 4 \cdot M_3 + 3 \cdot M_4} + 2 \cdot h = 3687 / \frac{12.5 \cdot 167.10}{2.5 \cdot 167.10 + 3 \cdot 42.49 + 4 \cdot 84.51 + 3 \cdot 126.04} + 2 \cdot 540 = 3306 \text{ mm}$$

$$\sigma_{m,crit} = \frac{\pi \sqrt{E_{0,05} \cdot I_2 \cdot G_{0,05} \cdot I_t}}{l_{ef} \cdot W_1} = \frac{\pi \sqrt{10800 \cdot 4.472e{+}08 \cdot 540 \cdot 1.340e{+}09}}{3306 \cdot 1.045e{+}07} = 170.02 \text{ N/mm}^2 \qquad (6.31)$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,1,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30.00}{170.02}} = 0.420 \qquad (6.30)$$

$$\lambda_{rel,m} = 0.420 \leq 0.75 \quad \rightarrow \quad k_{crit} = 1.000 \qquad (6.34)$$

$$\frac{\sigma_{m,1,d}}{k_{crit} \cdot f_{m,1,d}} = \frac{15.99}{1.000 \cdot 21.83} = 0.73 \leq 1.00 \qquad (6.33) \text{ - OK}$$

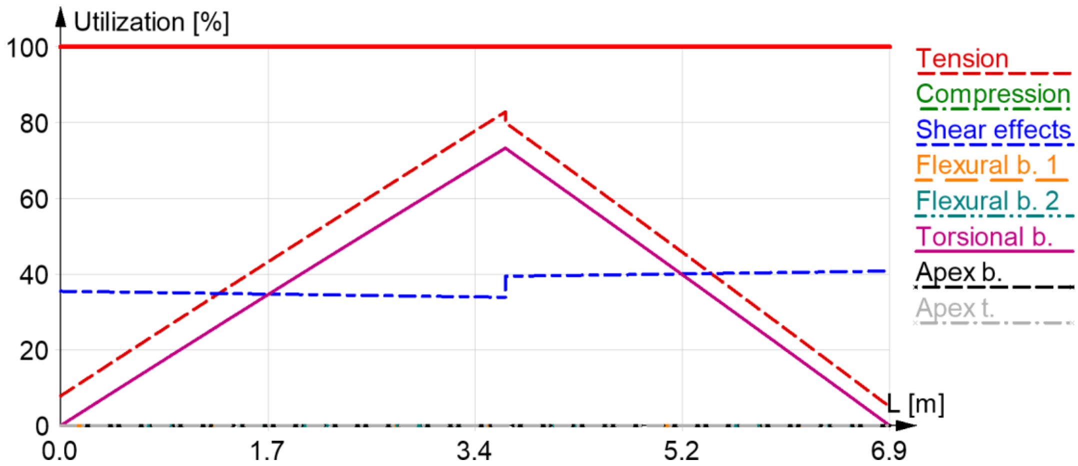
Bending at apex - 6.4.3

Not relevant

Tension at apex - 6.4.3

Not relevant

Summary



B.21.1 – Double tapered roof beam
Maximum of load combinations

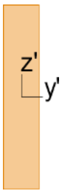
GL 30c

(Glued laminated), Service class 1

$E_{0,05}$	$=10800\ N/mm^2$	$f_{t,90,k}$	$= 0.50\ N/mm^2$
$G_{0,05}$	$= 540\ N/mm^2$	$f_{c,0,k}$	$=24.50\ N/mm^2$
γ_M	$= 1.25$	$f_{c,90,k}$	$= 2.50\ N/mm^2$
$\gamma_{M,acc./seis.}$	$= 1.00$	$f_{v,k}$	$= 3.50\ N/mm^2$
k_{sys}	$= 1.00$		

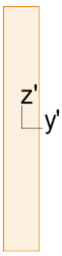
Sections of relevant verifications

x = 0 mm, Glulam 215x1035



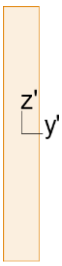
A	$= 222525\ mm^2$	$f_{t,0,k}$	$=19.50\ N/mm^2$
W_1	$=3.839e+07\ mm^3$	$f_{m,1,k}$	$=30.00\ N/mm^2$
W_2	$=7.974e+06\ mm^3$	$f_{m,2,k}$	$=33.00\ N/mm^2$
i_1	$= 299\ mm$		
i_2	$= 62\ mm$		
I_2	$=8.572e+08\ mm^4$		
I_t	$=2.980e+09\ mm^4$		

x = 5747 mm, Intermediate 1



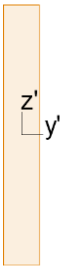
A	$= 295851\ mm^2$	$f_{t,0,k}$	$=19.50\ N/mm^2$
W_1	$=6.785e+07\ mm^3$	$f_{m,1,k}$	$=30.00\ N/mm^2$
W_2	$=1.060e+07\ mm^3$	$f_{m,2,k}$	$=33.00\ N/mm^2$
i_1	$= 397\ mm$		
i_2	$= 62\ mm$		
I_2	$=1.140e+09\ mm^4$		
I_t	$=4.110e+09\ mm^4$		

x = 6705 mm, Intermediate 1



A	$= 308072\ mm^2$	$f_{t,0,k}$	$=19.50\ N/mm^2$
W_1	$=7.357e+07\ mm^3$	$f_{m,1,k}$	$=30.00\ N/mm^2$
W_2	$=1.104e+07\ mm^3$	$f_{m,2,k}$	$=33.00\ N/mm^2$
i_1	$= 414\ mm$		
i_2	$= 62\ mm$		
I_2	$=1.187e+09\ mm^4$		
I_t	$=4.298e+09\ mm^4$		

x = 7184 mm, Intermediate 1



$A = 314183\ \text{mm}^2$ $\quad f_{t,0,k} = 19.50\ \text{N/mm}^2$

$W_1 = 7.652e{+}07\ \text{mm}^3$ $\quad f_{m,1,k} = 30.00\ \text{N/mm}^2$

$W_2 = 1.126e{+}07\ \text{mm}^3$ $\quad f_{m,2,k} = 33.00\ \text{N/mm}^2$

$i_1 = 422\ \text{mm}$

$i_2 = 62\ \text{mm}$

$I_2 = 1.210e{+}09\ \text{mm}^4$

$I_t = 4.392e{+}09\ \text{mm}^4$

Combined bending and axial tension - 6.2.3

LC: 'ULS 6.10b – NL 2 – Vind X + ', $k_{mod} = 0.90$, $x = 5747.37$ mm

$\alpha = 0.03$ rad

$$k_{m,\alpha} = 1/\sqrt{1 + \left(\frac{f_{m,1,d}}{0.75 \cdot f_{v,d}} \tan(\alpha)\right)^2 + \left(\frac{f_{m,1,d}}{f_{t,90,d}} \tan^2(\alpha)\right)^2} = 1/\sqrt{1 + \left(\frac{21.60}{0.75 \cdot 2.52} \tan(0.03)\right)^2 + \left(\frac{21.60}{0.36} \tan^2(0.03)\right)^2} = 0.95$$

(6.39)

$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}} + k_m \frac{\sigma_{m,2,d}}{f_{m,2,d}} = \frac{0.01}{14.04} + \frac{12.39}{0.95 \cdot 21.60} + 0.70 \frac{0.00}{23.76} = 0.61 \le 1.00$ (6.17) - OK

$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}} + \frac{\sigma_{m,2,d}}{f_{m,2,d}} = \frac{0.01}{14.04} + 0.70 \frac{12.39}{0.95 \cdot 21.60} + \frac{0.00}{23.76} = 0.43 \le 1.00$ (6.18) - OK

Combined bending and axial compression - 6.1.4, 6.2.4

LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, $x = 6705.26$ mm

$\alpha = 0.03$ rad

$$k_{m,\alpha} = 1/\sqrt{1 + \left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}} \tan(\alpha)\right)^2 + \left(\frac{f_{m,1,d}}{f_{c,90,d}} \tan^2(\alpha)\right)^2} = 1/\sqrt{1 + \left(\frac{21.60}{1.50 \cdot 2.52} \tan(0.03)\right)^2 + \left(\frac{21.60}{1.80} \tan^2(0.03)\right)^2} = 0.99$$

(6.40)

$\sigma_{c,0,d} = 0.00\ \text{N/mm}^2 \le f_{c,0,d} = 17.64\ \text{N/mm}^2$ (6.2) - OK

$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}} + k_m \frac{\sigma_{m,2,d}}{f_{m,2,d}} = \left(\frac{0.00}{17.64}\right)^2 + \frac{9.67}{0.99 \cdot 21.60} + 0.70 \frac{0.00}{23.76} = 0.45 \le 1.00$ (6.19) - OK

$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}} + \frac{\sigma_{m,2,d}}{f_{m,2,d}} = \left(\frac{0.00}{17.64}\right)^2 + 0.70 \frac{9.67}{0.99 \cdot 21.60} + \frac{0.00}{23.76} = 0.32 \le 1.00$ (6.20) - OK

Combined shear and torsion - 6.1.7, 6.1.8

LC: 'ULS 6.10b – NL 2 – Vind Y – ', $k_{mod} = 0.90$, $x = 0.00$ mm

$\tau_d = 2.13\ \text{N/mm}^2 \le f_{v,d} = 2.52\ \text{N/mm}^2$ (6.13) - OK

Flexural buckling around axis 1 - 6.3.2

LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, $x = 6705.26$ mm

$\beta_c = 0.1$ (6.29)

$$\lambda_1 = \frac{l_0}{i*_1} = \frac{9100}{377} = 24.16$$

$$\lambda_{rel,1} = \frac{\lambda_1}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{24.16}{\pi} \sqrt{\frac{24.50}{10800}} = 0.366 \qquad (6.21)$$

$$k_1 = 0.5\left(1 + \beta_c\left(\lambda_{rel,1} - 0.3\right) + \lambda_{rel,1}{}^2\right) = 0.5\left(1 + 0.1\,(0.366 - 0.3) + 0.366^2\right) = 0.570 \qquad (6.27)$$

$$k_{c,1} = \frac{1}{k_1 + \sqrt{k_1{}^2 - \lambda_{rel,1}{}^2}} = \frac{1}{0.570 + \sqrt{0.570^2 - 0.366^2}} = 0.992 \qquad (6.25)$$

$\alpha = 0.03$ rad

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{c,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{1.50 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{1.80}\tan^2(0.03)\right)^2} = 0.99$$

(6.40)

$$\frac{\sigma_{c,0,d}}{k_{c,1} \cdot f_{c,0,d}}+\frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+k_m \cdot \frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.00}{0.992 \cdot 17.64}+\frac{9.67}{0.99 \cdot 21.60}+0.70 \cdot \frac{0.00}{23.76}=0.45 \leq 1.00 \qquad (6.23) \text{ - OK}$$

Flexural buckling around axis 2 - 6.3.2

LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, $x = 6705.26$ mm

$\beta_c = 0.1$ (6.29)

$$\lambda_2 = \frac{l_0}{i*_2} = \frac{9100}{62} = 146.62$$

$$\lambda_{rel,2} = \frac{\lambda_2}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{146.62}{\pi}\sqrt{\frac{24.50}{10800}} = 2.223 \qquad (6.22)$$

$$k_2 = 0.5\left(1+\beta_c\left(\lambda_{rel,2}-0.3\right)+\lambda_{rel,2}{}^2\right) = 0.5\left(1+0.1\,(2.223-0.3)+2.223^2\right) = 3.067 \qquad (6.28)$$

$$k_{c,2} = \frac{1}{k_2+\sqrt{k_2{}^2-\lambda_{rel,2}{}^2}} = \frac{1}{3.067+\sqrt{3.067^2-2.223^2}} = 0.193 \qquad (6.26)$$

$\alpha = 0.03$ rad

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{c,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{1.50 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{1.80}\tan^2(0.03)\right)^2} = 0.99$$

(6.40)

$$\frac{\sigma_{c,0,d}}{k_{c,2} \cdot f_{c,0,d}}+k_m \cdot \frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.00}{0.193 \cdot 17.64}+0.70 \cdot \frac{9.67}{0.99 \cdot 21.60}+\frac{0.00}{23.76}=0.32 \leq 1.00 \qquad (6.24) \text{ - OK}$$

Lateral torsional buckling - 6.3.3

LC: 'ULS 6.10b – NL 2 – Vind X + ', $k_{mod} = 0.90$, $x = 7184.21$ mm

$$l_{ef} = l/\frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max}+3 \cdot M_2+4 \cdot M_3+3 \cdot M_4}+2 \cdot h$$

$$= 9100/\frac{12.5 \cdot 973.53}{2.5 \cdot 973.53+3 \cdot 424.27+4 \cdot 728.45+3 \cdot 911.79}+2 \cdot 1461 = 9919 \text{ mm}$$

$$\sigma_{m,crit} = \frac{\pi\sqrt{E_{0,05} \cdot I_2 \cdot G_{0,05} \cdot I_t}}{l_{ef} \cdot W_1} = \frac{\pi\sqrt{10800 \cdot 1.210e{+}09 \cdot 540 \cdot 4.392e{+}09}}{9919 \cdot 7.652e{+}07} = 23.05 \text{ N/mm}^2 \qquad (6.31)$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,1,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30.00}{23.05}} = 1.141 \qquad (6.30)$$

$$0.75 \leq \lambda_{rel,m} = 1.141 < 1.40 \quad \rightarrow \quad k_{crit} = 1.56-0.75 \cdot \lambda_{rel,m} = 1.56-0.75 \cdot 1.141 = 0.704 \qquad (6.34)$$

$\alpha = 0.03$ rad

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{c,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{1.50 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{1.80}\tan^2(0.03)\right)^2} = 0.95$$

(6.40)

$$\frac{\sigma_{m,1,d}}{k_{crit} \cdot k_{m,\alpha} \cdot f_{m,1,d}}=\frac{12.15}{0.704 \cdot 0.95 \cdot 21.60}=0.84 \leq 1.00 \qquad (6.33) \text{ - OK}$$

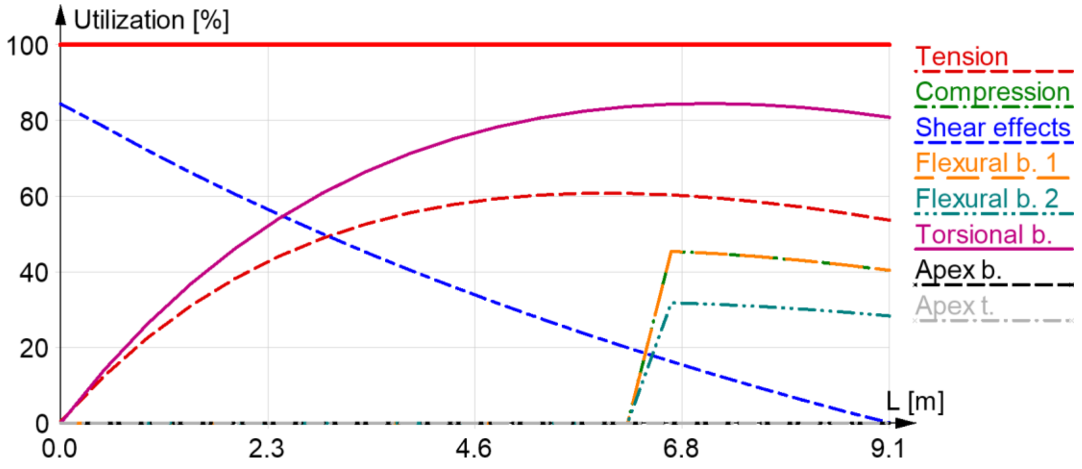
Bending at apex - 6.4.3

Not relevant

Tension at apex - 6.4.3

Not relevant

Summary


Utilization [%]
100
80
60
40
20
0
L [m]
0.0
2.3
4.6
6.8
9.1
Tension
Compression
Shear effects
Flexural b. 1
Flexural b. 2
Torsional b.
Apex b.
Apex t.

B.21.1 – roof supporting column (inside)
Maximum of load combinations

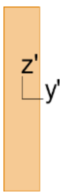
GL 30c

(Glued laminated), Service class 1

$E_{0,05}$	$=10800\ N/mm^2$	$f_{t,90,k}$	$= 0.50\ N/mm^2$
$G_{0,05}$	$= 540\ N/mm^2$	$f_{c,0,k}$	$=24.50\ N/mm^2$
γ_M	$= 1.25$	$f_{c,90,k}$	$= 2.50\ N/mm^2$
$\gamma_{M,acc./seis.}$	$= 1.00$	$f_{v,k}$	$= 3.50\ N/mm^2$
k_{sys}	$= 1.00$		

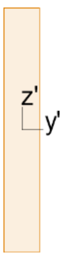
Sections of relevant verifications

x = 0 mm, Glulam 215x1035



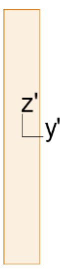
A	$= 222525\ mm^2$	$f_{t,0,k}$	$=19.50\ N/mm^2$
W_1	$=3.839e+07\ mm^3$	$f_{m,1,k}$	$=30.00\ N/mm^2$
W_2	$=7.974e+06\ mm^3$	$f_{m,2,k}$	$=33.00\ N/mm^2$
i_1	$= 299\ mm$		
i_2	$= 62\ mm$		
I_2	$=8.572e+08\ mm^4$		
I_t	$=2.980e+09\ mm^4$		

x = 5747 mm, Intermediate 1



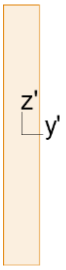
A	$= 295851\ mm^2$	$f_{t,0,k}$	$=19.50\ N/mm^2$
W_1	$=6.785e+07\ mm^3$	$f_{m,1,k}$	$=30.00\ N/mm^2$
W_2	$=1.060e+07\ mm^3$	$f_{m,2,k}$	$=33.00\ N/mm^2$
i_1	$= 397\ mm$		
i_2	$= 62\ mm$		
I_2	$=1.140e+09\ mm^4$		
I_t	$=4.110e+09\ mm^4$		

x = 6705 mm, Intermediate 1



A	$= 308072\ mm^2$	$f_{t,0,k}$	$=19.50\ N/mm^2$
W_1	$=7.357e+07\ mm^3$	$f_{m,1,k}$	$=30.00\ N/mm^2$
W_2	$=1.104e+07\ mm^3$	$f_{m,2,k}$	$=33.00\ N/mm^2$
i_1	$= 414\ mm$		
i_2	$= 62\ mm$		
I_2	$=1.187e+09\ mm^4$		
I_t	$=4.298e+09\ mm^4$		

x = 7184 mm, Intermediate 1



$A = 314183\ mm^2$	$f_{t,0,k} = 19.50\ N/mm^2$
$W_1 = 7.652e+07\ mm^3$	$f_{m,1,k} = 30.00\ N/mm^2$
$W_2 = 1.126e+07\ mm^3$	$f_{m,2,k} = 33.00\ N/mm^2$
$i_1 = 422\ mm$	
$i_2 = 62\ mm$	
$I_2 = 1.210e+09\ mm^4$	
$I_t = 4.392e+09\ mm^4$	

Combined bending and axial tension - 6.2.3

LC: $'ULS\ 6.10b\ -\ NL\ 2\ -\ Vind\ X+'$, $k_{mod} = 0.90$, $x = 5747.37\ mm$

$\alpha = 0.03\ rad$

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{0.75 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{t,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{0.75 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{0.36}\tan^2(0.03)\right)^2} = 0.95$$

(6.39)

$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.01}{14.04}+\frac{12.39}{0.95 \cdot 21.60}+0.70\frac{0.00}{23.76}=0.61 \leq 1.00$ (6.17) - OK

$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+k_m\frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.01}{14.04}+0.70\frac{12.39}{0.95 \cdot 21.60}+\frac{0.00}{23.76}=0.43 \leq 1.00$ (6.18) - OK

Combined bending and axial compression - 6.1.4, 6.2.4

LC: $'ULS\ 6.10b\ -\ Vind\ Y-'$, $k_{mod} = 0.90$, $x = 6705.26\ mm$

$\alpha = 0.03\ rad$

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{c,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{1.50 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{1.80}\tan^2(0.03)\right)^2} = 0.99$$

(6.40)

$\sigma_{c,0,d}=0.00\ N/mm^2 \leq f_{c,0,d}=17.64\ N/mm^2$ (6.2) - OK

$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{0.00}{17.64}\right)^2+\frac{9.67}{0.99 \cdot 21.60}+0.70\frac{0.00}{23.76}=0.45 \leq 1.00$ (6.19) - OK

$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+k_m\frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{0.00}{17.64}\right)^2+0.70\frac{9.67}{0.99 \cdot 21.60}+\frac{0.00}{23.76}=0.32 \leq 1.00$ (6.20) - OK

Combined shear and torsion - 6.1.7, 6.1.8

LC: $'ULS\ 6.10b\ -\ NL\ 2\ -\ Vind\ Y-'$, $k_{mod} = 0.90$, $x = 0.00\ mm$

$\tau_d=2.13\ N/mm^2 \leq f_{v,d}=2.52\ N/mm^2$ (6.13) - OK

Flexural buckling around axis 1 - 6.3.2

LC: $'ULS\ 6.10b\ -\ Vind\ Y-'$, $k_{mod} = 0.90$, $x = 6705.26\ mm$

$\beta_c = 0.1$ (6.29)

$$\lambda_1 = \frac{l_0}{i*_1} = \frac{9100}{377} = 24.16$$

$$\lambda_{rel,1} = \frac{\lambda_1}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{24.16}{\pi}\sqrt{\frac{24.50}{10800}} = 0.366 \quad (6.21)$$

$$k_1 = 0.5\left(1+\beta_c\left(\lambda_{rel,1}-0.3\right)+\lambda_{rel,1}{}^2\right) = 0.5\left(1+0.1\left(0.366-0.3\right)+0.366^2\right) = 0.570 \quad (6.27)$$

$$k_{c,1} = \frac{1}{k_1+\sqrt{k_1{}^2-\lambda_{rel,1}{}^2}} = \frac{1}{0.570+\sqrt{0.570^2-0.366^2}} = 0.992 \quad (6.25)$$

$\alpha = 0.03\ rad$

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{c,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{1.50 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{1.80}\tan^2(0.03)\right)^2} = 0.99$$

(6.40)

$$\frac{\sigma_{c,0,d}}{k_{c,1} \cdot f_{c,0,d}}+\frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+k_m \cdot \frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.00}{0.992 \cdot 17.64}+\frac{9.67}{0.99 \cdot 21.60}+0.70 \cdot \frac{0.00}{23.76}=0.45 \leq 1.00 \qquad (6.23) \text{ - OK}$$

Flexural buckling around axis 2 - 6.3.2

LC: 'ULS 6.10b – Vind Y – ', $k_{mod} = 0.90$, $x = 6705.26$ mm

$\beta_c = 0.1$ (6.29)

$$\lambda_2 = \frac{l_0}{i*_2} = \frac{9100}{62} = 146.62$$

$$\lambda_{rel,2} = \frac{\lambda_2}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{146.62}{\pi}\sqrt{\frac{24.50}{10800}} = 2.223 \qquad (6.22)$$

$$k_2 = 0.5\left(1+\beta_c\left(\lambda_{rel,2}-0.3\right)+\lambda_{rel,2}{}^2\right) = 0.5\left(1+0.1\,(2.223-0.3)+2.223^2\right) = 3.067 \qquad (6.28)$$

$$k_{c,2} = \frac{1}{k_2+\sqrt{k_2{}^2-\lambda_{rel,2}{}^2}} = \frac{1}{3.067+\sqrt{3.067^2-2.223^2}} = 0.193 \qquad (6.26)$$

$\alpha = 0.03$ rad

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{c,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{1.50 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{1.80}\tan^2(0.03)\right)^2} = 0.99$$

(6.40)

$$\frac{\sigma_{c,0,d}}{k_{c,2} \cdot f_{c,0,d}}+k_m \cdot \frac{\sigma_{m,1,d}}{k_{m,\alpha} \cdot f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{0.00}{0.193 \cdot 17.64}+0.70 \cdot \frac{9.67}{0.99 \cdot 21.60}+\frac{0.00}{23.76}=0.32 \leq 1.00 \qquad (6.24) \text{ - OK}$$

Lateral torsional buckling - 6.3.3

LC: 'ULS 6.10b – NL 2 – Vind X + ', $k_{mod} = 0.90$, $x = 7184.21$ mm

$$l_{ef} = l/\frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max}+3 \cdot M_2+4 \cdot M_3+3 \cdot M_4}+2 \cdot h$$

$$= 9100/\frac{12.5 \cdot 973.53}{2.5 \cdot 973.53+3 \cdot 424.27+4 \cdot 728.45+3 \cdot 911.79}+2 \cdot 1461 = 9919 \text{ mm}$$

$$\sigma_{m,crit} = \frac{\pi\sqrt{E_{0,05} \cdot I_2 \cdot G_{0,05} \cdot I_t}}{l_{ef} \cdot W_1} = \frac{\pi\sqrt{10800 \cdot 1.210e{+}09 \cdot 540 \cdot 4.392e{+}09}}{9919 \cdot 7.652e{+}07} = 23.05 \text{ N/mm}^2 \qquad (6.31)$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,1,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30.00}{23.05}} = 1.141 \qquad (6.30)$$

$$0.75 \leq \lambda_{rel,m} = 1.141 < 1.40 \quad \rightarrow \quad k_{crit} = 1.56-0.75 \cdot \lambda_{rel,m} = 1.56-0.75 \cdot 1.141 = 0.704 \qquad (6.34)$$

$\alpha = 0.03$ rad

$$k_{m,\alpha} = 1/\sqrt{1+\left(\frac{f_{m,1,d}}{1.50 \cdot f_{v,d}}\tan(\alpha)\right)^2+\left(\frac{f_{m,1,d}}{f_{c,90,d}}\tan^2(\alpha)\right)^2} = 1/\sqrt{1+\left(\frac{21.60}{1.50 \cdot 2.52}\tan(0.03)\right)^2+\left(\frac{21.60}{1.80}\tan^2(0.03)\right)^2} = 0.95$$

(6.40)

$$\frac{\sigma_{m,1,d}}{k_{crit} \cdot k_{m,\alpha} \cdot f_{m,1,d}}=\frac{12.15}{0.704 \cdot 0.95 \cdot 21.60}=0.84 \leq 1.00 \qquad (6.33) \text{ - OK}$$

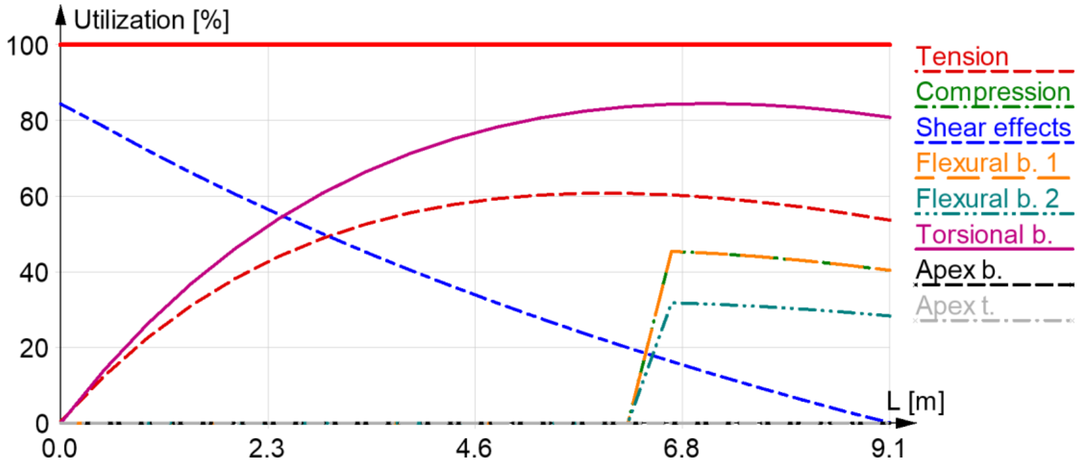
Bending at apex - 6.4.3

Not relevant

Tension at apex - 6.4.3

Not relevant

Summary


Utilization [%]
100
80
60
40
20
0
0.0
2.3
4.6
6.8
9.1
L [m]
Tension
Compression
Shear effects
Flexural b. 1
Flexural b. 2
Torsional b.
Apex b.
Apex t.

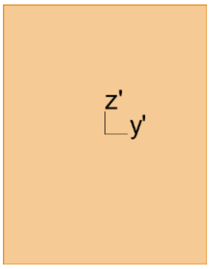
Basecolumn.13.1 – Floor supporting short column Maximum of load combinations

GL 30c

(Glued laminated), Service class 1

$E_{0,05}$	$=10800$ N/mm^2	$f_{t,90,k}$	$= 0.50$ N/mm^2
$G_{0,05}$	$= 540$ N/mm^2	$f_{c,0,k}$	$=24.50$ N/mm^2
γ_M	$= 1.25$	$f_{c,90,k}$	$= 2.50$ N/mm^2
$\gamma_{M,acc./seis.}$	$= 1.00$	$f_{v,k}$	$= 3.50$ N/mm^2
k_{sys}	$= 1.00$		

Glulam 215x270



$A =$	58050 mm^2	$f_{t,0,k}$	$=21.12$ N/mm^2
W_1	$=2.612e+06$ mm^3	$f_{m,1,k}$	$=32.49$ N/mm^2
W_2	$=2.080e+06$ mm^3	$f_{m,2,k}$	$=33.00$ N/mm^2
$i_1 =$	78 mm		
$i_2 =$	62 mm		
I_2	$=2.236e+08$ mm^4		
I_t	$=4.625e+08$ mm^4		

Combined bending and axial tension - 6.2.3

Not relevant

Combined bending and axial compression - 6.1.4, 6.2.4

LC: $'\mathrm{ULS\ 6.10b\ -\ NL\ 1\ -\ Vind\ X+}'$, $k_{mod} = 0.90$, $x = 0.00$ mm

$\sigma_{c,0,d}=5.34$ N/mm$^2 \leq f_{c,0,d}=17.64$ N/mm^2 (6.2) - OK

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{5.34}{17.64}\right)^2+\frac{0.00}{23.40}+0.70\frac{0.00}{23.76}=0.09\leq1.00 \qquad (6.19)\text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+k_m\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\left(\frac{5.34}{17.64}\right)^2+0.70\frac{0.00}{23.40}+\frac{0.00}{23.76}=0.09\leq1.00 \qquad (6.20)\text{ - OK}$$

Combined shear and torsion - 6.1.7, 6.1.8

LC: $'\mathrm{ULS\ 6.10b\ -\ Vind\ Y-}'$, $k_{mod} = 0.90$, $x = 0.00$ mm

$\tau_d=0.00$ N/mm$^2 \leq f_{v,d}=2.52$ N/mm^2 (6.13) - OK

Flexural buckling around axis 1 - 6.3.2

LC: $'\mathrm{ULS\ 6.10b\ -\ NL\ 1\ -\ Vind\ X+}'$, $k_{mod} = 0.90$, $x = 0.00$ mm

$\beta_c = 0.1$ (6.29)

$$\lambda_1 = \frac{l_0}{i_1} = \frac{820}{78} = 10.52$$

$$\lambda_{rel,1} = \frac{\lambda_1}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{10.52}{\pi}\sqrt{\frac{24.50}{10800}} = 0.300 \qquad (6.21)$$

$$k_1 = 0.5\left(1+\beta_c\left(\lambda_{rel,1}-0.3\right)+\lambda_{rel,1}{}^2\right) = 0.5\left(1+0.1\left(0.300-0.3\right)+0.300^2\right) = 0.545 \qquad (6.27)$$

$$k_{c,1} = \frac{1}{k_1+\sqrt{k_1^2-\lambda_{rel,1}^2}} = \frac{1}{0.545+\sqrt{0.545^2-0.300^2}} = 1.000 \qquad (6.25)$$

$$\frac{\sigma_{c,0,d}}{k_{c,1}\cdot f_{c,0,d}}+\frac{\sigma_{m,1,d}}{f_{m,1,d}}+k_m\cdot\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{5.34}{1.000\cdot 17.64}+\frac{0.00}{23.40}+0.70\cdot\frac{0.00}{23.76}=0.30\leq 1.00 \qquad (6.23)\text{ - OK}$$

Flexural buckling around axis 2 - 6.3.2

LC: 'ULS 6.10b – NL 1 – Vind X + ', $k_{mod} = 0.90$, $x = 0.00$ mm

$$\beta_c = 0.1 \qquad (6.29)$$

$$\lambda_2 = \frac{l_0}{i_2} = \frac{820}{62} = 13.21$$

$$\lambda_{rel,2} = \frac{\lambda_2}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{13.21}{\pi}\sqrt{\frac{24.50}{10800}} = 0.300 \qquad (6.22)$$

$$k_2 = 0.5\left(1+\beta_c\left(\lambda_{rel,2}-0.3\right)+\lambda_{rel,2}^2\right) = 0.5\left(1+0.1\,(0.300-0.3)+0.300^2\right) = 0.545 \qquad (6.28)$$

$$k_{c,2} = \frac{1}{k_2+\sqrt{k_2^2-\lambda_{rel,2}^2}} = \frac{1}{0.545+\sqrt{0.545^2-0.300^2}} = 1.000 \qquad (6.26)$$

$$\frac{\sigma_{c,0,d}}{k_{c,2}\cdot f_{c,0,d}}+k_m\cdot\frac{\sigma_{m,1,d}}{f_{m,1,d}}+\frac{\sigma_{m,2,d}}{f_{m,2,d}}=\frac{5.34}{1.000\cdot 17.64}+0.70\cdot\frac{0.00}{23.40}+\frac{0.00}{23.76}=0.30\leq 1.00 \qquad (6.24)\text{ - OK}$$

Lateral torsional buckling - 6.3.3

LC: 'ULS 6.10b – NL 1 – Vind Y + ', $k_{mod} = 0.90$, $x = 0.00$ mm

$$l_{ef} = l/\frac{12.5\cdot M_{max}}{2.5\cdot M_{max}+3\cdot M_2+4\cdot M_3+3\cdot M_4} - 0.5\cdot h = 820/\frac{12.5\cdot 0.00}{2.5\cdot 0.00+3\cdot 0.00+4\cdot 0.00+3\cdot 0.00} - 0.5\cdot 270 = 357\text{ mm}$$

$$\sigma_{m,crit} = \frac{\pi\sqrt{E_{0,05}\cdot I_2\cdot G_{0,05}\cdot I_t}}{l_{ef}\cdot W_1} = \frac{\pi\sqrt{10800\cdot 2.236e{+}08\cdot 540\cdot 4.625e{+}08}}{357\cdot 2.612e{+}06} = 2616.32\text{ N/mm}^2 \qquad (6.31)$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,1,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30.00}{2616.32}} = 0.107 \qquad (6.30)$$

$$\lambda_{rel,m} = 0.107 \leq 0.75 \quad\rightarrow\quad k_{crit} = 1.000 \qquad (6.34)$$

$$\frac{\sigma_{m,1,d}}{k_{crit}\cdot f_{m,1,d}}=\frac{0.00}{1.000\cdot 23.40}=0.00\leq 1.00 \qquad (6.33)\text{ - OK}$$

$$\left(\frac{\sigma_{m,1,d}}{k_{crit}\cdot f_{m,1,d}}\right)^2+\frac{\sigma_{c,0,d}}{k_{c,2}\cdot f_{c,0,d}}=\left(\frac{0.00}{1.000\cdot 23.40}\right)^2+\frac{5.34}{1.00\cdot 17.64}=0.30\leq 1.00 \qquad (6.35)\text{ - OK}$$

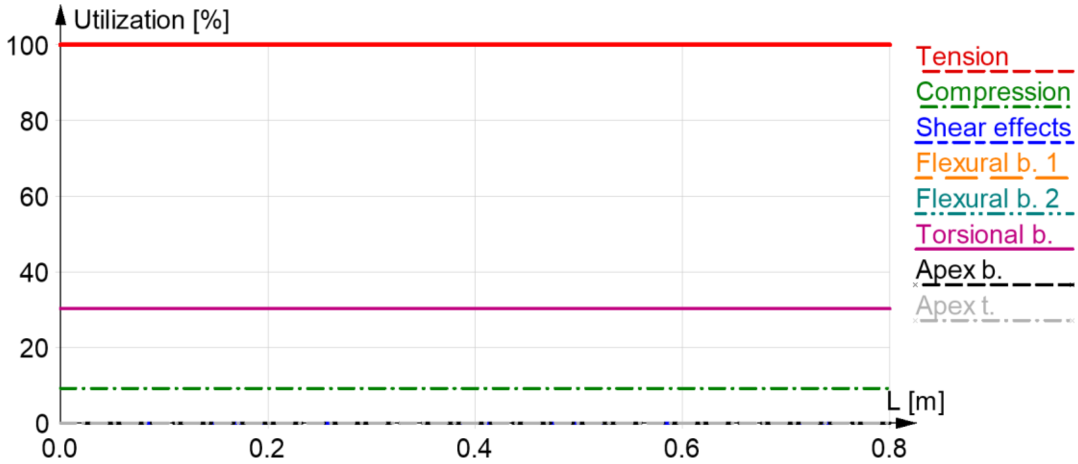
Bending at apex - 6.4.3

Not relevant

Tension at apex - 6.4.3

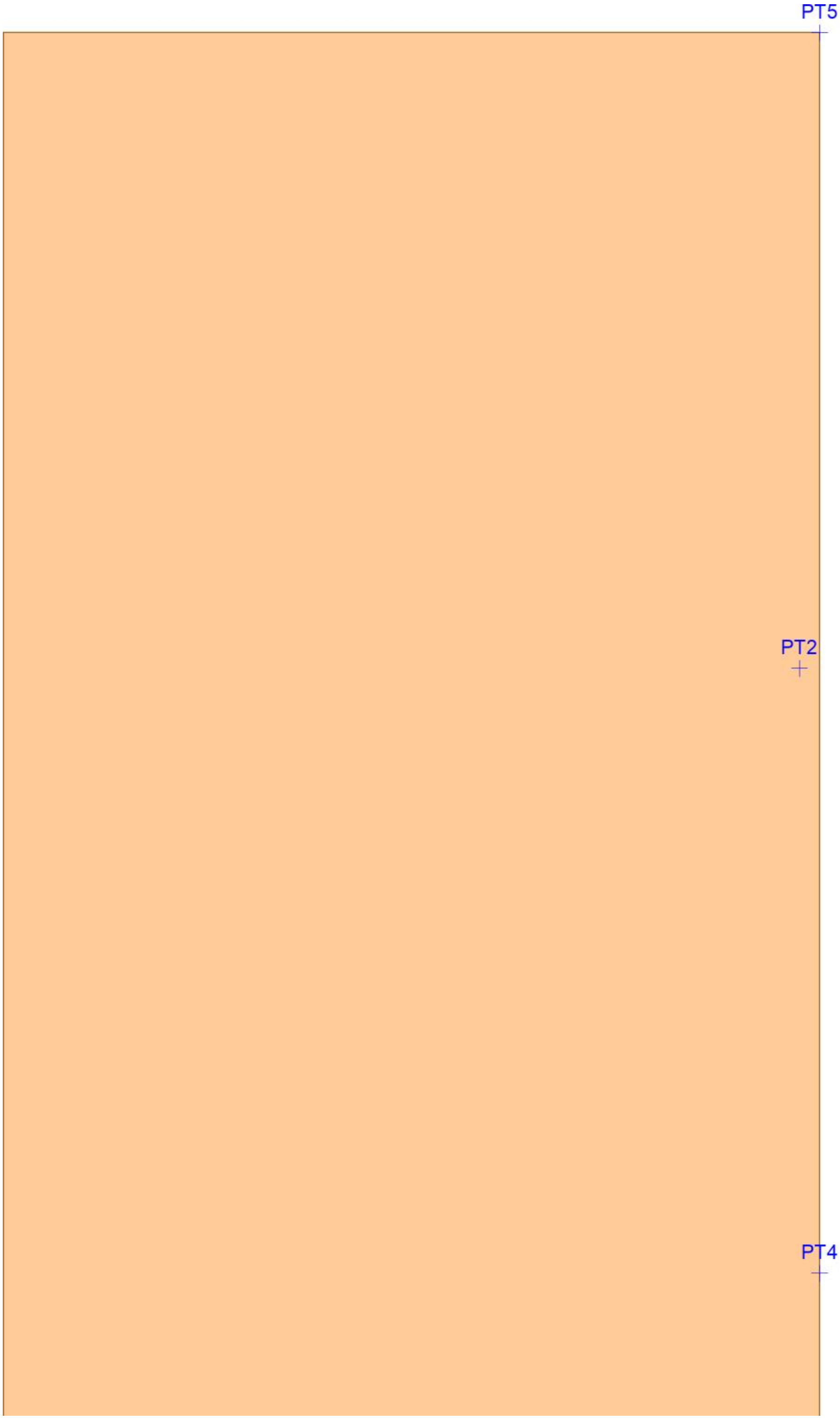
Not relevant

Summary


Utilization [%]
100
80
60
40
20
0
0.0
0.2
0.4
0.6
0.8
L [m]
Tension
Compression
Shear effects
Flexural b. 1
Flexural b. 2
Torsional b.
Apex b.
Apex t.

Ribelement CLT.2 – CLT part of floor
Maximum of load combinations

Geometry



Maximum nodes:
PT1 (27.94, 15.70, 1.92) [m]
PT2 (55.88, 15.92, 1.92) [m]
PT3 (3.03, 24.40, 1.92) [m]
PT4 (49.16, 15.70, 1.92) [m]
PT5 (62.94, 15.70, 1.92) [m]

Node numbers:
PT1: 80725
PT2: 169184
PT3: 1771
PT4: 148430
PT5: 191600

Panel type:
120-3s

Total thickness:
t = 120.00 mm

Panel properties

Service class: 1, $\gamma_{M,ult.} = 1.25$, $\gamma_{M,acc./seis.} = 1.00$, $k_{sys} = 1.00$

No	Material	Thickness [mm]	Theta [°]	Rho [kg/m^3]
1	C24	40	0	420
2	C14	40	90	350
3	C24	40	0	420

Mechanical properties

No	$E_{0,mean}$ [N/mm^2]	$E_{90,mean}$ [N/mm^2]	ν_{xy} [-]	$G_{xy,mean}$ [N/mm^2]	$G_{xz,mean}$ [N/mm^2]	$G_{yz,mean}$ [N/mm^2]
1	11000	0	0.00	690	690	69
2	7000	0	0.00	440	440	44
3	11000	0	0.00	690	690	69

Limit stresses

No	$f_{m,0,k}$ [N/mm	$f_{m,90,k}$ [N/mm	$f_{t,0,k}$ [N/mm	$f_{t,90,k}$ [N/mm	$f_{c,0,k}$ [N/mm	$f_{c,90,k}$ [N/mm	$f_{xy,k}$ [N/mm	$f_{v,k}$ [N/mm	$f_{vR,k}$ [N/mm
1	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00
2	14.0	14.0	7.20	0.400	16.0	2.00	3.00	3.00	1.50
3	24.0	24.0	14.5	0.400	21.0	2.50	4.00	4.00	2.00

Tension and bending, x - 6.2.3

Panel: 'Ribelement CLT.2.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind X – ', k_{mod} = 0.90, PT1

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}}+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\frac{0.01}{5.18}+\frac{|-5.07|}{10.08}=0.50\leq 1.00 \qquad (6.17)\text{ - OK}$$

Compression and bending, x - 6.1.4, 6.2.4

Panel: 'Ribelement CLT.2.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT2

$$\frac{|\sigma_{c,0,d}|}{f_{c,0,d}}=\frac{|-0.14|}{11.52}=0.01\leq 1.00 \qquad (6.2)\text{ - OK}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2+\frac{|\sigma_{m,0,d}|}{f_{m,0,d}}=\left(\frac{-0.14}{11.52}\right)^2+\frac{|-7.84|}{10.08}=0.78\leq 1.00 \qquad (6.19)\text{ - OK}$$

Shear, xy - 6.1.7

Panel: 'Ribelement CLT.2.1', Layer: '1', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT3

$$\frac{|\tau_{xy,d}|}{f_{xy,d}}=\frac{|0.57|}{2.88}=0.20\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear, xz - 6.1.7

Panel: 'Ribelement CLT.2.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT4

$$\frac{|\tau_{xz,d}|}{f_{v,d}}=\frac{|-0.61|}{2.16}=0.28\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear, yz - 6.1.7

Panel: 'Ribelement CLT.2.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT5

$$\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|-0.12|}{1.08}=0.11\leq 1.00 \qquad (6.13)\text{ - OK}$$

Shear interaction

Panel: 'Ribelement CLT.2.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y + ', k_{mod} = 0.90, PT4

$$\left(\frac{\tau_{xy,d}}{f_{xy,d}}\right)^2+\left(\frac{\tau_{xz,d}}{f_{v,d}}\right)^2=\left(\frac{0.02}{2.16}\right)^2+\left(\frac{-0.61}{2.16}\right)^2=0.08\leq 1.00\text{ - OK}$$

Tension and shear

Panel: 'Ribelement CLT.2.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT5

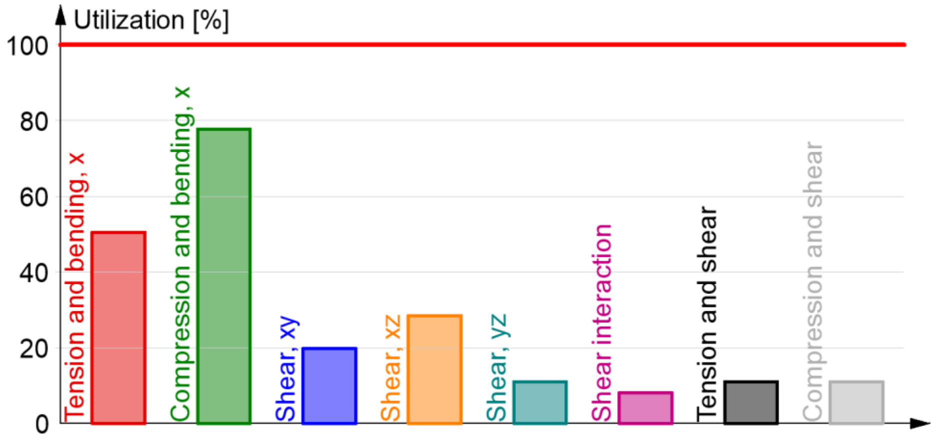
$$\frac{\sigma_{t,90,d}}{f_{t,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{0.00}{0.29}+\frac{|-0.12|}{1.08}=0.11\leq 1.00\text{ - OK}$$

Compression and shear

Panel: 'Ribelement CLT.2.1', Layer: '2', LC: 'ULS 6.10b – NL 1 – Vind Y – ', k_{mod} = 0.90, PT5

$$\frac{|\sigma_{c,90,d}|}{f_{c,90,d}}+\frac{|\tau_{yz,d}|}{f_{R,d}}=\frac{|0.00|}{1.44}+\frac{|-0.12|}{1.08}=0.11\leq 1.00\text{ - OK}$$

Summary


Utilization [%]
100
80
60
40
20
0
Tension and bending, x
Compression and bending, x
Shear, xy
Shear, xz
Shear, yz
Shear interaction
Tension and shear
Compression and shear

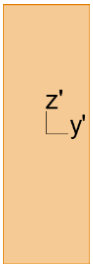
Ribelement beam.399.1 – Rib-beam for floor
Maximum of load combinations

GL 30c

(Glued laminated), Service class 1

$E_{0,05}$	$=10800\ \mathrm{N/mm^2}$	$f_{t,90,k}$	$= 0.50\ \mathrm{N/mm^2}$
$G_{0,05}$	$= 540\ \mathrm{N/mm^2}$	$f_{c,0,k}$	$=24.50\ \mathrm{N/mm^2}$
γ_M	$= 1.25$	$f_{c,90,k}$	$= 2.50\ \mathrm{N/mm^2}$
$\gamma_{M,acc./seis.}$	$= 1.00$	$f_{v,k}$	$= 3.50\ \mathrm{N/mm^2}$
k_{sys}	$= 1.00$		

Glulam 140x405



A	$= 56700\ \mathrm{mm^2}$	$f_{t,0,k}$	$=20.28\ \mathrm{N/mm^2}$
W_1	$=3.827e+06\ \mathrm{mm^3}$	$f_{m,1,k}$	$=31.20\ \mathrm{N/mm^2}$
W_2	$=1.323e+06\ \mathrm{mm^3}$	$f_{m,2,k}$	$=33.00\ \mathrm{N/mm^2}$
i_1	$= 117\ \mathrm{mm}$		
i_2	$= 40\ \mathrm{mm}$		
I_2	$=9.261e+07\ \mathrm{mm^4}$		
I_t	$=2.898e+08\ \mathrm{mm^4}$		

Combined bending and axial tension - 6.2.3

LC: 'ULS 6.10b – NL 1 – Vind X – ', $k_{mod} = 0.90$, $x = 4789.47$ mm

$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,1,d}}{f_{m,1,d}} + k_m \frac{\sigma_{m,2,d}}{f_{m,2,d}} = \frac{2.19}{14.60} + \frac{3.56}{22.47} + 0.70 \frac{0.08}{23.76} = 0.31 \le 1.00$ (6.17) - OK

$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,1,d}}{f_{m,1,d}} + \frac{\sigma_{m,2,d}}{f_{m,2,d}} = \frac{2.19}{14.60} + 0.70 \frac{3.56}{22.47} + \frac{0.08}{23.76} = 0.26 \le 1.00$ (6.18) - OK

Combined bending and axial compression - 6.1.4, 6.2.4

Not relevant

Combined shear and torsion - 6.1.7, 6.1.8

LC: 'ULS 6.10b – NL 1 – Vind Y – ', $k_{mod} = 0.90$, $x = 9100.00$ mm

$\tau_d = 0.95\ \mathrm{N/mm^2} \le f_{v,d} = 2.52\ \mathrm{N/mm^2}$ (6.13) - OK

Flexural buckling around axis 1 - 6.3.2

Not relevant

Flexural buckling around axis 2 - 6.3.2

Not relevant

Lateral torsional buckling - 6.3.3

LC: 'ULS 6.10b – NL 1 – Vind X – ', $k_{mod} = 0.90$, $x = 4789.47$ mm

$$l_{ef} = l/\frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_2 + 4 \cdot M_3 + 3 \cdot M_4} + 2 \cdot h = 9100/\frac{12.5 \cdot 13.64}{2.5 \cdot 13.64 + 3 \cdot 9.35 + 4 \cdot 13.61 + 3 \cdot 10.01} + 2 \cdot 405 = 8635 \text{ mm}$$

$$\sigma_{m,crit} = \frac{\pi \sqrt{E_{0,05} \cdot I_z \cdot G_{0,05} \cdot I_t}}{l_{ef} \cdot W_1} = \frac{\pi \sqrt{10800 \cdot 9.261e{+}07 \cdot 540 \cdot 2.898e{+}08}}{8635 \cdot 3.827e{+}06} = 37.61 \text{ N/mm}^2 \qquad (6.31)$$

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,1,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30.00}{37.61}} = 0.893 \qquad (6.30)$$

$$0.75 \leq \lambda_{rel,m} = 0.893 < 1.40 \quad \rightarrow \quad k_{crit} = 1.56 - 0.75 \cdot \lambda_{rel,m} = 1.56 - 0.75 \cdot 0.893 = 0.890 \qquad (6.34)$$

$$\frac{\sigma_{m,1,d}}{k_{crit} \cdot f_{m,1,d}} = \frac{3.56}{0.890 \cdot 22.47} = 0.18 \leq 1.00 \qquad (6.33) \text{ - OK}$$

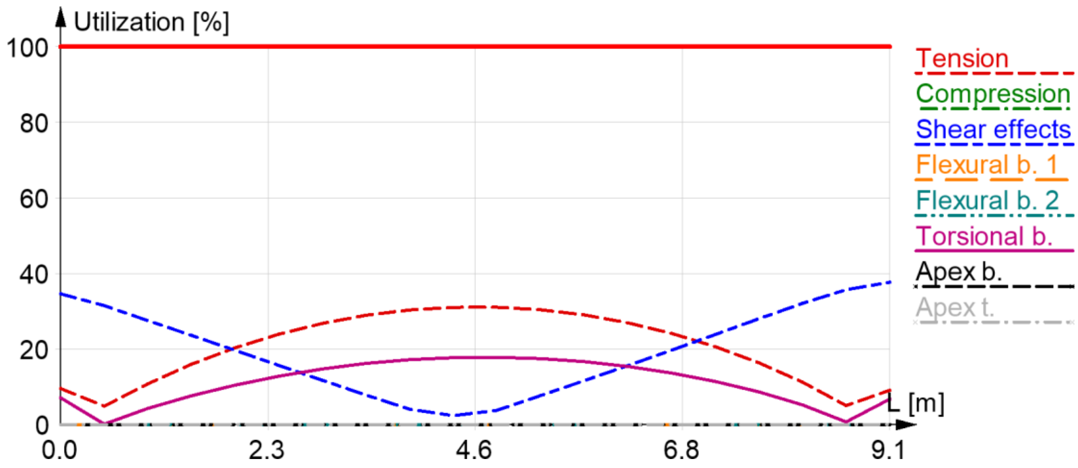
Bending at apex - 6.4.3

Not relevant

Tension at apex - 6.4.3

Not relevant

Summary



T.2.1 – Bracing tie
Maximum of load combinations

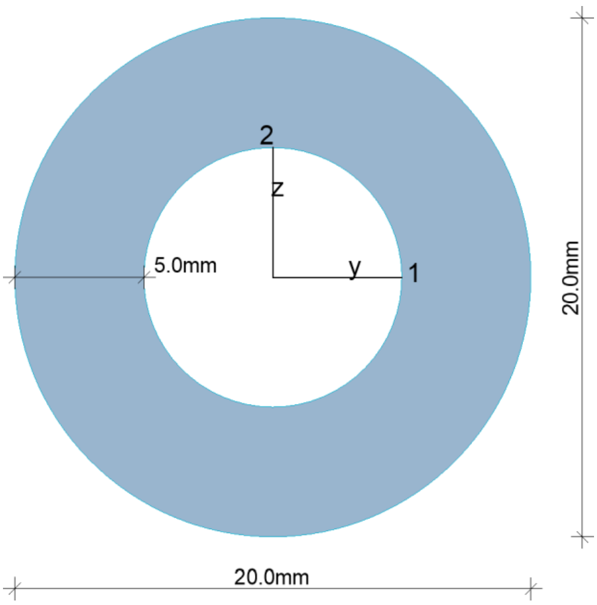
S 355

E =210000 N/mm^2

G = 80769 N/mm^2

$\gamma_{M0,ult}$=	1.00	$\gamma_{M0,acc/seis}$=	1.00
$\gamma_{M1,ult}$=	1.00	$\gamma_{M1,acc/seis}$=	1.00
$\gamma_{M2,ult}$=	1.20	$\gamma_{M2,acc/seis}$=	1.00

CHS 20-5.0



P	= 63 mm	f_y =	355 N/mm^2
A	= 236 mm^2	ε =	0.81
I_y	=7.363e+03 mm^4	λ_1=	76.40
I_z	=7.363e+03 mm^4		
I_1	=7.363e+03 mm^4		
I_2	=7.363e+03 mm^4		
$W_{pl,1}$	=1.167e+03 mm^3		
$W_{pl,2}$	=1.167e+03 mm^3		
$W_{el,min,1}$	=7.363e+02 mm^3		
$W_{el,min,2}$	=7.363e+02 mm^3		
i_1	= 6 mm		
i_2	= 6 mm		
I_t	=1.473e+04 mm^4		
I_w	=0.000e+00 mm^6		

Shear resistance, 1-1 - Part 1-1: 6.2.6, 6.2.8

LC: 'ULS 6.10a',   x = 0 mm

$\text{Class}_N = 1,\ \text{Class}_{M1} = 1,\ \text{Class}_{M2} = 1$

$$V_{1,pl,Rd} = \frac{A_{1,v}\cdot f_y}{\sqrt{3}\cdot\gamma_{M0}} = \frac{150\cdot 355}{\sqrt{3}\cdot 1.00} = 30.74\ \text{kN} \qquad (6.18)$$

$$V_{1,pl,T,Rd} = 1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}}\cdot V_{1,pl,Rd} = 1 - \frac{0.00}{(355/\sqrt{3})/1.00}\cdot 30.74 = 30.74\ \text{kN} \qquad (6.28)$$

$$\frac{V_{1,Ed}}{V_{1,pl,T,Rd}} = \frac{0.00}{30.74} = 0.00 \le 1.00 \qquad (6.25)\text{ - OK}$$

Shear resistance, 2-2 - Part 1-1: 6.2.6, 6.2.8

LC: 'ULS 6.10a',   x = 0 mm

$\text{Class}_N = 1,\ \text{Class}_{M1} = 1,\ \text{Class}_{M2} = 1$

$$V_{2,pl,Rd} = \frac{A_{2,v}\cdot f_y}{\sqrt{3}\cdot\gamma_{M0}} = \frac{150\cdot 355}{\sqrt{3}\cdot 1.00} = 30.74\ \text{kN} \qquad (6.18)$$

$$V_{2,pl,T,Rd} = 1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}}\cdot V_{2,pl,Rd} = 1 - \frac{0.00}{(355/\sqrt{3})/1.00}\cdot 30.74 = 30.74\ \text{kN} \qquad (6.28)$$

$$\frac{V_{2,Ed}}{V_{2,pl,T,Rd}} = \frac{0.00}{30.74} = 0.00 \le 1.00 \qquad (6.25)\text{ - OK}$$

Torsional resistance - Part 1-1: 6.2.7

LC: 'ULS 6.10a',   x = 0 mm

$Class_N = 1,\ Class_{M1} = 1,\ Class_{M2} = 1$

$\tau_{max,unit} = 679.08\ \frac{N/mm^2}{kNm}$ is calculated by FEM analysis.

$$T_{Rd} = \frac{f_y}{\sqrt{3}\cdot\tau_{max,unit}\cdot\gamma_{M0}} = \frac{355}{\sqrt{3}\cdot 679.08\cdot 1.00} = 0.30\ kNm$$

$\frac{T_{Ed}}{T_{Rd}} = \frac{0.00}{0.30} = 0.00 \le 1.00$ (6.23) - OK

Shear stress - Part 1-1: 6.2.6

Not relevant

Normal stress - Part 1-1: 6.2.1

Not relevant

Normal capacity - Part 1-1: 6.2

LC: 'ULS 6.10b - Vind Y+',   x = 0 mm

$Class_N = 1,\ Class_{M1} = 1,\ Class_{M2} = 1$

$V_{1,Ed} = 0.00\ kN \le 0.5\cdot V_{1,pl,T,Rd} = 0.5\cdot 30.74 = 15.37\ kN \rightarrow \rho_1 = 0.00$

$V_{2,Ed} = 0.00\ kN \le 0.5\cdot V_{2,pl,T,Rd} = 0.5\cdot 30.74 = 15.37\ kN \rightarrow \rho_1 = 0.00$

$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{1,Ed}}{M_{1,Rd}} + \frac{M_{2,Ed}}{M_{2,Rd}} = \frac{46.03}{83.64} + \frac{0.00}{0.41} + \frac{0.00}{0.41} = 0.55 \le 1.00$ (6.2) - OK

Flexural buckling, 1-1 - Part 1-1: 6.3.1

Not relevant

Flexural buckling, 2-2 - Part 1-1: 6.3.1

Not relevant

Torsional-flexural buckling - Part 1-1: 6.3.1

Not relevant

Lateral torsional buckling, top flange - Part 1-1: 6.3.2.4

Not relevant

Lateral torsional buckling, bottom flange - Part 1-1: 6.3.2.4

Not relevant

Interaction between normal force and bending 1. - Part 1-1: 6.3.3

Not relevant

Interaction between normal force and bending 2. - Part 1-1: 6.3.3

Not relevant

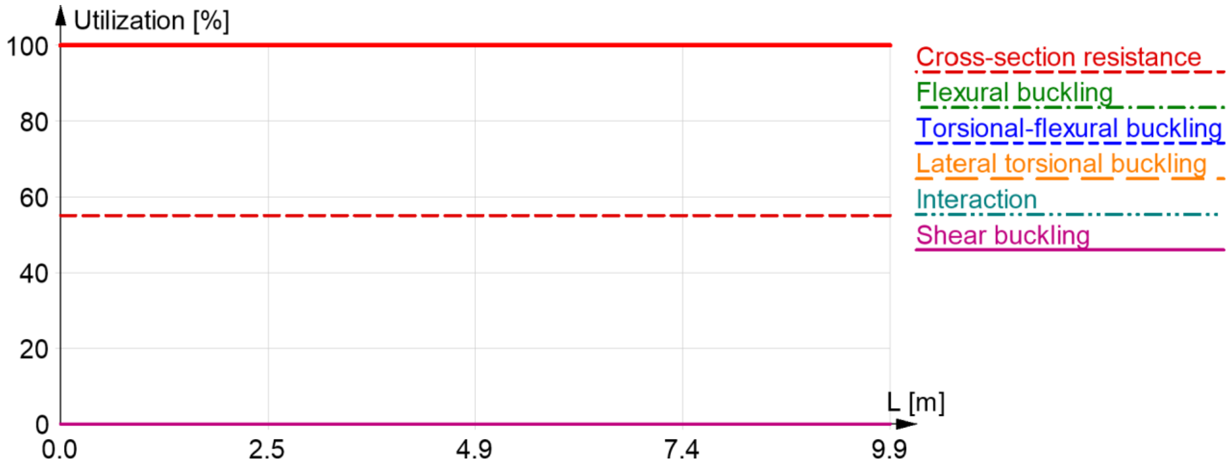
Interaction between normal force and bending, 2nd order - Part 1-1: 6.3.3

Not relevant

Shear buckling - Part 1-5: 5

Not relevant

Summary



G

Hand calculation for diagrams

Limträbalk

Indata

Laster		
$g_k$extra	0,5	kN/m^2
g_kCLT	0,981	kN/m^2
g_k	1,481	kN/m^2
q_k1	2,5	kN/m^2
q_k2	0,5	kN/m^2

Rumsgeometri		
"Influensbredd"	4,75	m
l	6	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	131,37	kNm
V_{Ed}	87,58	kN

Balktvärsnitt		
h	540	mm
b	140	mm
Föreligger risk för vippning?		Nej

Materialdata		
Materialtyp	Limträ	-
f_{mk}	30	MPa
f_{vk}	3,5	MPa
E_{mean}	13	GPa
Densitet	500	kg/m3

Koefficienter och faktorer		
Exponerat för nederbörd/solstrålning?		Nej
Klimatklass	1	-
Lastvaraktighet	Medellång	-
γ_m	1,25	-
k_{mod}	0,8	-
k_h	1,0	-
k_{cr}	0,86	-
k_{def}	0,6	-
ψ_2	0,3	-

ULS Beräkningar

Momentkapacitet		
f_{md}	19,4	MPa
W	6804000	mm^3
M_{Rd}	132,0	kNm
σ_{Md}	19,3	MPa
Kontroll momentkapacitet		OK
Utnyttjandegrad		99,51%

Tvärkraftskapacitet		
f_{vd}	2,2	MPa
b_{ef}	120,0	mm
A_{ef}	64800,0	mm^2
V_{Rd}	96,8	kN
τ_d	2,0	MPa
Kontroll tvärkraftskapacitet		OK
Utnyttjandegrad		90,50%

SLS Beräkningar

Nedböjning krav		
Initiell krav: L/		300
Slutlig krav: L/		300
w_{inst}	20,0	mm
w_{fin}	20,0	mm

Beräknad nedböjning		
w_{ginst}	5,0	mm
w_{qinst}	8,4	mm
w_{gfin}	8,0	mm
w_{qfin}	9,9	mm
$w_{insttot}$	13,4	mm
w_{fintot}	17,9	mm
Kontroll initiell nedböjning		OK
Utnyttjandegrad		66,82%
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		89,28%

Beräkningen är baserad på Eurokod 5, samt nationell bilaga EKS 11

Column

Indata

Laster		
q_k1	2,5	kN/m^2
q_k2	0,5	kN/m^2
g_{kextra}	0,5	kN/m^2
g_{ktot}	91	kN
Vertikal kraft	739	kN

Geometri och materialdata		
Materialtyp	Limträ	-
Antal våningar	4	st
Inf. Area	36,8	m^2
h	225	mm
b	225	mm
$f_{c,0,k}$	24,5	MPa
$f_{m,k}$	30	MPa
Lcr	3	m
E0,05	11000	MPa
Densitet	500	kg/m3

Koefficienter och faktorer		
Exponerat för nederbörd/solstrålning?		Nej
Klimatklass	1	-
Lastvaraktighet	Medellång	-
γ_m	1,25	-
k_{mod}	0,8	-
β	0,1	-

Beräkningar

i	0,064951905	mm
λ	46,18802154	-
λ_{rel}	0,693850949	-
k	0,760407117	
kc	0,933255629	
$f_{c,0,d}$	15,68	Mpa
Ned	738,50	kN
$\sigma_{c,0,d}$	14,58773588	Mpa
Behövs det ta hänsyn till knäckning?		Ja
Kontroll utan hänsyn till knäckning		-
Kontroll med hänsyn till knäckning		99,69%

CLT platta

Indata

Laster		
q_k1	2,5	kN/m^2
q_k2	0,5	kN/m^2
g_{kextra}	0,5	kN/m^2
g_{ktot}	1,714	kN/m^2

Rumsgeometri		
Upplag	Fritt upplagd	
l	5,25	m
l_{ref}	5,25	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	21,8	kNm/m
V_{Ed}	16,6	kN/m

Materialdata		
Materialtyp	Konstruktionsvirke	-
$f_{m,k,xlay}$	24	MPa
$f_{v,k,xlay}$	4	MPa
$f_{r,k,ylay}$	0,9	MPa
$E_{mean,xlay}$	11	GPa
$E_{mean,ylay}$	11	GPa
$G_{090,xlay}$	690	MPa
$G_{9090,ylay}$	50	MPa
ρ	687,5	kg/m^3
γ_m	1,25	-
Klimatklass	1	-
Lastvaraktighet	Medellång	-
k_{mod}	0,8	-
k_{def}	0,85	-
k_{sys}	1,0	-
ψ_2	0,3	-

Plattans uppbyggnad		
Antal lager		5
t1	40	mm
t2	30	mm
t3	40	mm
t4	30	mm
t5	40	mm

ULS Beräkningar

Momentkapacitet		
σ_{Myd}	4,8	MPa
$f_{m,xlay,d}$	15,4	MPa
M_{Rd}	69,6	kNm/m
Kontroll momentkapacitet		OK
Utnyttjandegrad		31,33%

Tvärkraftskapacitet		
τ_{vd}	0,122	MPa
τ_{rd}	0,114	MPa
f_{vd}	2,6	MPa
f_{rd}	0,58	MPa
V_{vRd}	348,2	kN/m
V_{rRd}	83,9	kN/m
Kontroll parallellskjuvning		OK
Utnyttjandegrad		4,77%
Kontroll rullskjuvning		OK
Utnyttjandegrad		19,80%

Tvärsnittsegenskaper		
t_{tot}	180	mm
I_{xnet}	40800	cm^4/m
I_{ynet}	7800	cm^4/m
S_{xnet}	3000	cm^3/m
S_{Rxnet}	2800	cm^3/m
W_{xnet}	4533	cm^3/m
I_{xef}	37414	cm^4/m
GA_{ef}	1,49E+07	N
k	1,2	-

SLS Beräkningar

Nedböjning krav		
Initiell krav: L/		300
Slutlig krav: L/		300
w_{inst}	17,5	mm
w_{fin}	17,5	mm

Beräknad nedböjning enligt SAV		
w_{ginst}	4,3	mm
w_{qinst}	7,4	mm
w_{gfin}	7,9	mm
w_{qfin}	9,3	mm
$w_{insttot}$	11,7	mm
w_{fintot}	17,2	mm
Kontroll initiell nedböjning		OK
Utnyttjandegrad		66,85%
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		98,36%

SLS Beräkningar

Vibration och svikt		
Bredd	1	m
m	124	kg/m2
f1	10,4	Hz
n40	0,56	st
ν	0,003	m/Ns2
b	100	m/Ns2
ξ	0,01	-
v_{max}	0,016	m/Ns2
a	1,5	mm/kN
w_{inst}	0,78	mm/kN
Kontroll f1		OK
Kontroll vibrationer		OK
Utnyttjandegrad		21,39%
Kontroll svikt		OK
Utnyttjandegrad		51,83%

Beräkningen är baserad på metoder från svenskt trä KL-trähandbok

Beräkning förutsätter samma virkeskvalitet för alla lager i huvudbärriktningen. Gäller endast för symmetriskt tvärsnitt.
Beräkningen görs enligt balkteori där endast bärförmåga räknas för lager i huvudbärriktningen, och övriga lager försummas.
Fullständig interaktion mellan lager förutsätts i ULS. Deformationsberäkning görs enligt gamma-metoden föreslagen i SS-EN 1995-1-1 bilaga B men gäller endast upp till 5 lager. Kompletterande beräkning görs också enl. SAV metoden.

CLT rib panel

Indata

Laster		
q_k1	2,5	kN/m^2
q_k2	0,5	kN/m^2
g_{kextra}	0,5	kN/m^2
g_{ktot}	1,37	kN/m^2

Rumsgeometri		
Upplag	Fritt upplagd	
l	10,4	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	48,0	kNm/balk
V_{Ed}	18,5	kN/balk

Materialdata CLT platta		
Materialtyp	Konstruktionsvirke	-
$f_{c,k,xlay}$	21	MPa
$f_{v,k,xlay}$	4	MPa
$f_{r,k,ylay}$	0,9	MPa
E_{mean}	11	GPa
ρ_{platta}	500	kg/m^3

Materialdata balk		
Materialtyp	Limträ	-
f_{ck}	24,5	MPa
f_{tk}	19,5	MPa
f_{vk}	3,5	MPa
E_{mean}	13	GPa
G_{mean}	0,65	GPa
ρ_{balk}	430	kg/m^3

Tvärsnittets uppbyggnad		
t1	40	mm
t2	30	mm
t3	40	mm
h_{balk}	315	mm
b_{balk}	140	mm
cc_{balk}	600	mm

Tvärsnittsberäkningar

Gemensamma data		
γ_m	1,25	-
Klimatklass	1	-
Lastvaraktighet	Medellång	-
k_{mod}	0,8	-
k_{def}	0,85	-
k_{sys}	1,0	-
ψ_2	0,3	-
k_{cr}	0,86	-

Beräkning av effektiv flänsbredd enl. 5.26 dot1		
b_f	460	mm
β_0	1,06	-
β_1	1,8	-
b_{eff}	451,0	mm

Tvärsnittsegenskaper		
h_{platta}	110	mm
b_{efftot}	591,0	mm
h_{tot}	425	mm
z_{tp}	166,4	mm
I_{xnet}	136717	cm^4/balk
S_{max}	9361	cm^3/balk
S_{rx}	2929	cm^3/balk
I_{ynet}	225,0	cm^4/m

ULS Beräkningar

Tryck och drag från böjning		
σ_{wcd}	1,98	MPa
σ_{wtd}	9,1	MPa
σ_{fcdmax}	5,84	MPa
f_{wcd}	15,7	Mpa
f_{wtd}	12,5	MPa
f_{fcd}	13,4	MPa
Kontroll tryckspänningar liv		OK
Utnyttjandegrad		12,63%
Kontroll dragspänningar liv		OK
Utnyttjandegrad		72,71%
Kontroll tryckspänningar fläns		OK
Utnyttjandegrad		43,45%

Tvärkraftskapacitet		
$b_{ef,web}$	120,0	mm
$b_{ef,flange}$	220	mm
τ_{vd}	1,05	MPa
τ_{rd}	0,18	MPa
f_{vd}	2,2	MPa
f_{rd}	0,58	MPa
Kontroll parallellskjuvning		OK
Utnyttjandegrad		47,00%
Kontroll rullskjuvning		OK
Utnyttjandegrad		31,19%

SLS Beräkningar

Nedböjning krav		
Initiell krav: L/		300
Slutlig krav: L/		300
w_{inst}	34,7	mm
w_{fin}	34,7	mm

Beräknad nedböjning		
w_{ginst}	7,5	mm
w_{qinst}	16,4	mm
w_{gfin}	13,9	mm
w_{qfin}	20,6	mm
$w_{insttot}$	23,9	mm
w_{fintot}	34,5	mm
Kontroll initiell nedböjning		OK
Utnyttjandegrad		69,04%
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		99,50%

Vibration och svikt		
Bredd	1	m
$m_{längd}$	52,0	kg/m
m_{yta}	86,6	kg/m2
f1	8,5	Hz
n40	1,21	st
ν	0,004	m/Ns2
b	100	m/Ns2
ξ	0,01	-
ν_{max}	0,015	m/Ns2
a	1,5	mm/kN
w_{inst}	0,79	mm/kN
Kontroll f1		OK
Utnyttjandegrad		94,19%
Kontroll vibrationer		OK
Utnyttjandegrad		27,72%
Kontroll svikt		OK
Utnyttjandegrad		52,74%

LVL-rib-panel

Indata

Laster		
q_k1	2,5	kN/m^2
q_k2	0,5	kN/m^2
g_{kextra}	0,5	kN/m^2
g_{ktot}	1,13	kN/m^2

Rumsgeometri		
Upplag	Fritt upplagd	
l	8,85	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	33,1	kNm/balk
V_{Ed}	15,0	kN/balk

Materialdata LVL-platta		
Materialtyp	LVL-C	-
$f_{c,0,k}$	26	MPa
$f_{v,0,flat,k}$	1,3	MPa
$E_{0,mean}$	10,5	GPa
$E_{90,mean}$	2	GPa
ρ_{mean}	500	kg/m^3
γ_m	1,2	-

Materialdata balk		
Materialtyp	Limträ	-
f_{c0k}	30	MPa
f_{t0k}	19,5	MPa
f_{vk}	3,5	MPa
$E_{0,mean}$	13	GPa
G_{mean}	0,65	GPa
ρ_{mean}	500	kg/m^3
γ_m	1,25	-

Gemensamma data		
Klimatklass	1	-
Lastvaraktighet	Medellång	-
k_{mod}	0,8	-
k_{def}	0,60	-
ψ_2	0,3	-
k_{cr}	0,86	-

Tvärsnittsberäkningar

Tvärsnittets uppbyggnad		
t_{LVL}	63	mm
h_{balk}	270	mm
b_{balk}	140	mm
cc_{balk}	600	mm

Tvärsnittsegenskaper		
h_{tot}	333	mm
b_{eff}	600,0	mm
z_{tp}	123,6	mm
I_{xnet}	71035	cm^4/balk
S_{max}	6138	cm^3/balk
S_{rx}	2812	cm^3/balk
I_{ynet}	2083,7	cm^4/m

ULS Beräkningar

Tryck och drag från böjning		
σ_{wcd}	2,82	MPa
σ_{wtd}	9,8	MPa
σ_{fcd}	4,29	MPa
f_{wcd}	19,2	Mpa
f_{wtd}	12,5	MPa
f_{fcd}	17,3	MPa
Kontroll tryckspänningar liv		OK
Utnyttjandegrad		14,70%
Kontroll dragspänningar liv		OK
Utnyttjandegrad		78,13%
Kontroll tryckspänningar fläns		OK
Utnyttjandegrad		24,74%

Tvärkraftskapacitet		
$b_{ef,web}$	120,0	mm
$b_{con,flange}$	140	mm
τ_{vd}	1,08	MPa
τ_{rd}	0,42	MPa
f_{vd}	2,2	MPa
f_{rd}	0,87	MPa
Kontroll parallellskjuvning		OK
Utnyttjandegrad		48,06%
Kontroll rullskjuvning		OK
Utnyttjandegrad		48,78%

SLS Beräkningar

Nedböjning krav		
Initiell krav: L/		300
Slutlig krav: L/		300
w_{inst}	29,5	mm
w_{fin}	29,5	mm

Beräknad nedböjning		
w_{ginst}	6,2	mm
w_{qinst}	16,4	mm
w_{gfin}	9,9	mm
w_{qfin}	19,4	mm
$w_{insttot}$	22,6	mm
w_{fintot}	29,3	mm
Kontroll initiell nedböjning		OK
Utnyttjandegrad		76,67%
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		99,29%

Vibration och svikt		
Bredd	1	m
$m_{längd}$	37,8	kg/m
m_{yta}	63,0	kg/m2
f1	9,9	Hz
n40	0,98	st
ν	0,005	m/Ns2
b	100	m/Ns2
ξ	0,01	-
ν_{max}	0,016	m/Ns2
a	1,5	mm/kN
w_{inst}	0,94	mm/kN
Kontroll f1		OK
Utnyttjandegrad		80,70%
Kontroll vibrationer		OK
Utnyttjandegrad		33,04%
Kontroll svikt		OK
Utnyttjandegrad		62,55%

TCC floor

Indata

Laster		
q_k1	2,5	kN/m^2
q_k2	0,5	kN/m^2
g_{kextra}	0,5	kN/m^2
g_{ktot}	4,2	kN/m^2

Rumsgeometri		
Upplag	Fritt upplagd	
l	6,925	m

Dimensionerande lasteffekter brottgräns		
M_{Ed}	56,2	kNm/m
V_{Ed}	32,4	kN/m

Materialdata trä		
Materialtyp	C24	-
$f_{t,k,xlay}$	14	MPa
$f_{v,k,xlay}$	4	MPa
$f_{r,k,ylay}$	0,9	MPa
$E_{mean,xlay}$	11	GPa
$E_{mean,ylay}$	11	GPa
$G_{090,xlay}$	690	MPa
$G_{9090,ylay}$	50	MPa
ρ	510	kg/m^3
γ_m	1,25	-
Klimatklass	1	-
Lastvaraktighet	Medellång	-
k_{mod}	0,8	-
k_{def}	0,85	-
ψ_2	0,3	-
E_{ef}	5,9	GPa
G_{efxlay}	373,0	MPa
G_{efylay}	27,0	MPa

Materialdata betong		
Materialtyp	C30/37	-
f_{ck}	30	MPa
E_{cm}	33	GPa
γ_m	1,5	-
ρ	2500	kg/m^3
$\varphi(\infty,t0)$	2,15	-
G_{cm}	13,2	GPa
$E_{c,ef}$	10,5	GPa
$G_{c,ef}$	4,2	GPa

ULS Beräkningar

Golvets uppbyggnad		
Betongens tjocklek	120	mm
Antal lager i CLT	5	st
t1	40	mm
t2	20	mm
t3	40	mm
t4	20	mm
t5	40	mm

Tvärsnittsegenskaper korttid		
Samverkansgrad	85%	
t_{tot}	280	mm
α	3	-
z_{tp}	95,0	mm
I_{xnet}	250000	cm^4/m
I_{xef}	212500	cm^4/m
S_{xnet}	12600	cm^3/m
S_{Rxnet}	14400	cm^3/m
GA_{ef}	36	MN
I_{ynet}	46933	cm^4/m

Momentkapacitet korttid		
σ_{cbtg}	7,5	MPa
f_{cd}	20	MPa
σ_{tCLT}	4,9	MPa
$f_{t,xlay,d}$	9,0	MPa
Kontroll tryck i betong		OK
Utnyttjandegrad		37,65%
Kontroll drag i trä		OK
Utnyttjandegrad		54,56%

Tvärkraftskapacitet korttid		
τ_{vd}	0,192	MPa
τ_{rd}	0,220	MPa
f_{vd}	2,6	MPa
f_{rd}	0,58	MPa
Kontroll parallellskjuvning		OK
Utnyttjandegrad		7,51%
Kontroll rullskjuvning		OK
Utnyttjandegrad		38,16%

43%

ULS Beräkningar

Tvärsnittsegenskaper långtid		
Samverkansgrad	85%	
α_{ef}	1,76	-
z_{tp}	110,7	mm
I_{xnet}	205813	cm^4/m
I_{xef}	174941	cm^4/m
S_{xnet}	10717	cm^3/m
S_{Rxnet}	11890	cm^3/m
GA_{ef}	19	MN

Momentkapacitet långtid		
σ_{cbtg}	6,3	MPa
f_{cd}	20	MPa
σ_{tCLT}	5,4	MPa
$f_{t,xlay,d}$	9,0	MPa
Kontroll tryck i betong		OK
Utnyttjandegrad		31,30%
Kontroll drag i trä		OK
Utnyttjandegrad		60,65%

Tvärkraftskapacitet långtid		
τ_{vd}	0,199	MPa
τ_{rd}	0,220	MPa
f_{vd}	2,6	MPa
f_{rd}	0,58	MPa
Kontroll parallellskjuvning		OK
Utnyttjandegrad		7,76%
Kontroll rullskjuvning		OK
Utnyttjandegrad		38,27%

SLS Beräkningar

Nedböjning krav		
Slutlig krav: L/		300
w_{fin}	23,1	mm

Beräknad nedböjning långtid		
k	1,2	-
w_{ginst}	13,8	mm
w_{qinst}	9,3	mm
w_{tot}	23,1	mm
Kontroll slutlig nedböjning		OK
Utnyttjandegrad		99,92%

Vibration och svikt		
Bredd	1	m
m	382	kg/m2
f1	8,11	Hz
n40	0,46	st
ν	0,001	m/Ns2
b	100	m/Ns2
ξ	0,01	-
ν_{max}	0,015	m/Ns2
a	1,5	mm/kN
w_{inst}	0,35	mm/kN
Kontroll f1		OK
Utnyttjandegrad		98,68%
Kontroll vibrationer		OK
Utnyttjandegrad		6,57%
Kontroll svikt		OK
Utnyttjandegrad		23,60%

H

Used cross-sections for CLT and TCC diagrams

CLT Cross-section build-up

3 Layers

H [mm]	*|| [mm]*	*T [mm]*	*|| [mm]*
60	20	20	20
70	20	30	20
80	30	20	30
90	30	30	30
100	40	20	40
120	40	40	40

5 Layers

H [mm]	*|| [mm]*	*T [mm]*	*|| [mm]*	*T [mm]*	*|| [mm]*
100	20	20	20	20	20
120	30	20	20	20	30
140	40	20	20	20	40
160	40	20	40	20	40
180	40	30	40	30	40
200	40	40	40	40	40

7 Layers

H [mm]	*||[mm]*	*T [mm]*	*||[mm]*	*T [mm]*	*||[mm]*	*T [mm]*	*||[mm]*
180	20	40	20	20	20	40	20
220	30	40	30	20	30	40	30
240	30	40	30	40	30	40	30
280	40	40	40	40	40	40	40

CLT Cross-section build-up for TCC floors

5 Layers

H [mm]	*|| [mm]*	*T [mm]*	*|| [mm]*	*T [mm]*	*|| [mm]*
120	30	20	20	20	30
140	40	20	20	20	40
160	40	20	40	20	40
180	40	30	40	30	40
200	40	40	40	40	40

7 Layers (Note that the two outer most lamellas goes in the same direction)

H [mm]	*||[mm]*	*|| [mm]*	*T[mm]*	*|| [mm]*	*T[mm]*	*||[mm]*	*||[mm]*
180	30	30	20	20	20	30	30
200	30	30	20	40	20	30	30
220	40	40	20	20	20	40	40
240	40	40	20	40	20	40	40
260	40	40	30	40	30	40	40
280	40	40	40	40	40	40	40