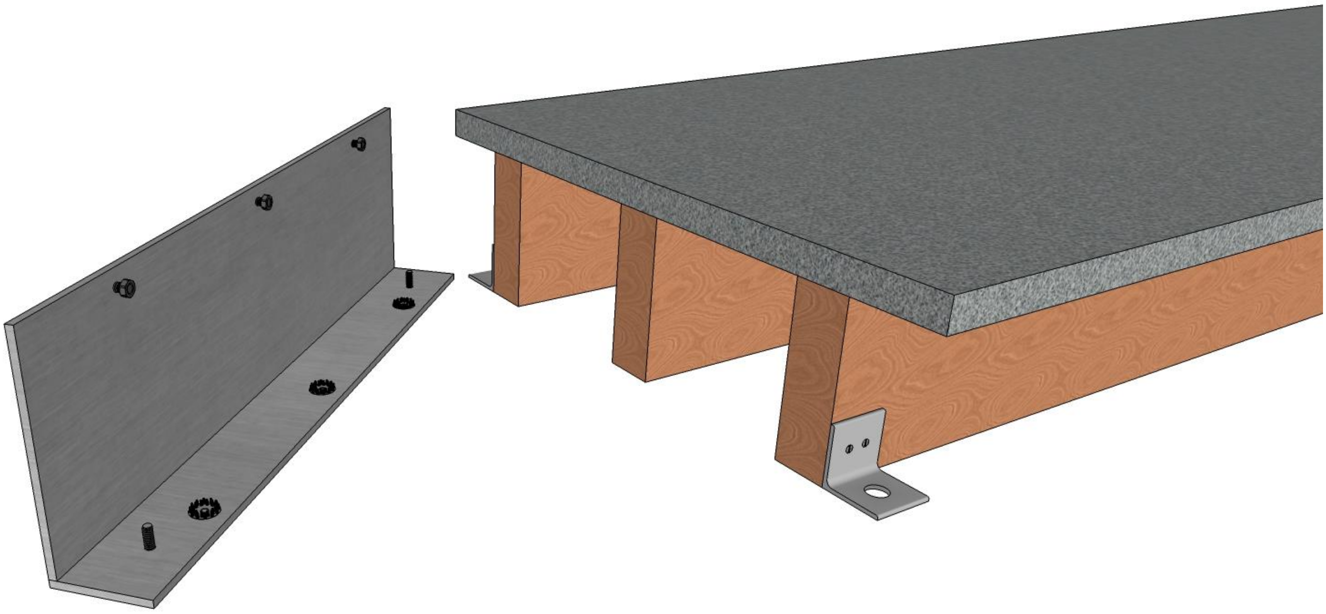
Development of an innovative demountable floor system

Structural design and verification



Master's thesis

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Development of an innovative demountable floor system

Structural design and verification

By

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in partial fulfilment of the requirements for the degree of
Master of Science
in Civil Engineering

Faculty of Civil Engineering and Geosciences
Delft University of Technology

To be defended publicly on Friday February 8, 2019 at 13:45.

Thesis committee

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An electronic version of this thesis is available at http://repository.tudelft.nl/.

Raadgevende
Ingenieurs

Preface

This thesis was written as the last step in obtaining a Master's degree in Civil Engineering at Delft University of Technology. The topic of this thesis is the development of a demountable floor system. An emphasis is set on the design of the connection between the head end of the floor slab and the edge beam.

This research was conducted at the Department of Structural Engineering at Delft University of Technology in collaboration with IMd Raadgevende Ingenieurs.

I would like to thank the members of my graduation committee; Prof. dr. M. Veljkovic, Dr. ir. R. Abspoel and Drs. W.F. Gard from Delft University of Technology, Ing. R.J. Stark from IMd Raadgevende Ingenieurs and Ir. J.P. den Hollander from Bouwen met Staal for their guidance. My special thanks goes to Roland for his tiresome support regarding the content of my thesis as well as his support during the entire process of graduating and to Rob for his support for my work and our productive and interesting discussions. Moreover, I would like to thank Ir. P. Lagendijk and Prof. dr. ir. J.W.G. van de Kuilen for their help regarding their expertise's. My thanks also goes to my colleagues at IMd for their time and effort in answering my questions and in providing a nice working environment.

Lastly, I would like to thank my family, friends and especially my boyfriend for their patience, encouragement, support and confidence in me during my entire studies and my graduation work.

Laura van Glabbeek
February 2019

Abstract

Today, society is becoming more and more aware of the environment and the (negative) influence we have on it. Especially the current workings of our economy have a detrimental effect on the environment. To reduce these effects, a transition is being made towards a circular economy. The construction sector plays a big role in the degradation: in the Netherlands the sector accounts for 50% of the raw material use, 40% of the energy consumption, 40% of the waste production and 35% of the CO_2 emission [24]. For the construction sector, one of the changes to establish a more circular economy, is the adoption of a circular design method. This method consists of the construction of buildings, building systems and building elements being designed for disassembly and reuse. This reduces raw material use and the expel of harmful emissions.

While we see some developments being made in the construction sector regarding circularity, not many developments can be found in the field of reusable floor systems. For the sizable use of raw materials and expel of harmful emissions during the manufacturing process of floors, a more circular design for floor systems can (eventually) contribute to the sectors transition to circularity. Therefore, this research is focussed on developing demountable connections which can be used in such a circular floor system. The result is a system comprising a prefabricated timber-concrete composite (hereafter: TCC) floor slab connected to steel edge beams with toothed-plate connectors. An important aspect taken into account in the development of this design is the efficiency of erecting, disassembling and reassembling the floor.

A multi-criteria analysis is conducted to determine floor slabs viable for use in a reusable floor system. A timber-concrete composite floor slab is chosen and designed. The slab is verified using analytical calculations.

Design variants are made for demountable connections at the slab-beam positions at the head end and the side of the slab and for the slab-slab position. A choice between design variants is made by reviewing the fabrication and assembly tolerances and by assuring a non-destructive disassembly procedure. Analytical calculations are performed to verify the chosen connections.

A case study building is used for the development of the floor system. The TCC floor slab spans 10.8m, has a total height of 510mm and a width of 1800mm. The used edge beam is an L-section. On the web a toothed-plate connector is adhesively bonded onto which the timber beams are placed to enable shear force transfer between the timber beams and the edge girder. At the top of the web a compression bolt is installed which is fastened after instalment of the floor slabs to ensure force transfer between the bolt and the concrete slab. Two angle sections are screwed to the outer sides of the outer timber beams and bolted to the edge girder to guide the slab to its intended position and to ensure structural soundness by vertically fixing the floor slab to the girder.

Lastly the limits of the developed floor system are determined by performing a parameter study of the floor slab and the connection. The floor slab can be designed to span 12.6m. The connection still meets the structural requirements when the system is used in a building of maximally 70m high. This is applicable on many combinations of the building length (10m to 70m) and width (6m to 12.6m), if the right cross-sectional dimensions are used.

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1 Introduction

As early as 1972, the environmental impact of industries and the growing world population were addressed in the United Nations Conference on the Human environment. After this, in 1987, the Report of the World Commission (the Brundtland commission) on Environment and Development; Our Common Future [25] was published, which stipulated the need for sustainable development. The Brundtland commission defined sustainable development as that, which “meets the needs and aspirations of the present without compromising the ability to meet those of the future”[26]. The term ‘needs’ can be defined as the availability of principal resources and the existence of a healthy and clean living environment. Unfortunately, our current economy doesn’t preserve these needs. Therefore, steps are being taken to transform our current linear economy into a circular economy. In a linear economy, principal resources are excavated and used to produce a product which is disposed of at its end-of-life. In a circular economy, the product is not disposed of but reused.

A big contributor to the degradation of the environment is the building industry. In the Netherlands the industry accounts for 50% of the raw material use, 40% of the energy consumption, 40% of the waste production and 35% of the CO_2 emission [24]. These percentages comprise all the life phases of a product in the building industry: production of sub-products and elements, transportation to the building site, construction of the final construction, use of the construction, and the end-of -life. In all these phases measurements can be taken to decrease the bad impact on the environment. To illustrate, production methods of building elements can be changed to be less harmful for the environment, green energy sources can be used during the use of structures and building products can be designed to be used as long as possible. These measures are all required for the transformation to a circular economy.

The stock management philosophy in a circular economy is to keep products, components and materials in use as long as possible due to which the use of principal resources, harmful emissions and waste production are reduced [27]. Increasing the use life can be achieved by adapting a circular product design: designing more durable products and/or reusing them. One measure to increase the use life is to design a building or building product to have a lifetime of more than the nowadays used design life of 50 years. Another possibility is to assure reuse of the building (product), by which the overall lifetime is increased. Only a few examples of the application of reuse of building (products) can be given. Therefore, developments have to be made to enable the implementation of reuse in the building industry.

Reuse of building products can be facilitated by designing them for reuse. In this so-called circular design procedure, several additional aspects must be regarded compared to a standard design procedure. One of the most important aspects is assuring a non-destructive and efficient assembly, disassembly and reassembly (hereafter: (dis)assembly) of the building products. This can be achieved by using easy demountable connections.

1.1 Problem definition

As of yet, no floor systems are available for utility buildings which are designed to be disassembled and reused. The connections found in common floor systems, used to join the floor slabs to the building structure and to join the floor slabs to each other, can usually not be disassembled without compromising the elements that are joined; they are not fully demountable.

1.2 Objectives

The main objective of this research is to develop demountable connections which can be implemented in a demountable floor system. To achieve this main objective, the following sub-objectives have to be completed:

- Determine the requirements for demountable connections in demountable floor systems and make a description of existing floor systems and demountable connections.
- Determine which type of floor slabs can be viable to use in a demountable floor system and choose and design a floor slab.
- Design demountable connections to join the floor slabs to the supporting beams (slab-beam positions) and to join the floor slabs (slab-to-slab position). Determine the structural performance of the connections.

1.3 Research questions

The main research question that will be answered in this thesis follows from the problem statement and the objectives:

What do the designs of demountable connections for a demountable floor system look like so that an efficient and non-destructive (dis)assembly procedure is assured?

To be able to answer the main research question, the following sub-questions are formulated:

- What are the main requirements for demountable connections in demountable floor systems?
- What type of floor slab can be used in a demountable floor system and what does the design of this floor slab look like?
- What do fully demountable connections between the floor slabs and the edge beams and between the floor slabs look like and are the elements comprising the joint structurally sound?

1.4 Framework

In this research, fully demountable connections are designed. In a fully demountable connection, the damaging of building elements when disassembling the connection doesn't occur, for example the breaking of a hardened wet joint or the cutting of welded plates.

Most of the floors in buildings are storey floors. Therefore, an emphasis is set on the structural design of a storey floor, and the connections required for these floor slabs.

1.5 Thesis structure

In the first part of this thesis, chapter 2, a literature research is conducted to determine the state of the art on demountability and reuse in the construction industry, to formulate the requirements for demountable connections and to determine the currently used floor systems and demountable connections.

In chapter 3, a multi-criteria analysis is conducted to determine which floor slab has the best expectation of functioning well in a reusable floor system. In chapter 4, a literature study is conducted into the chosen floor slab type. Subsequently, in chapter 5 the used case study building is introduced, and the structural design and verification of the floor slab is performed.

Chapter 6 states the design and verification of the demountable connections in the floor system.

The final design and limitation of the floor system are given in chapter 7.

Chapter 8 concludes this thesis with the research conclusions and recommendations.

2 Literature Research

This chapter contains a research out of which boundary conditions and starting points for the development of the demountable floor system can be stated. Paragraph 1 illustrates the requirements of the floor system. In paragraph 2, a state-of-the-art description is given of demountability and reuse in the construction industry. Paragraph 3 introduces the term environmental cost indicator and gives a short summary of available environmental impact calculation tools. In paragraph 4, currently available floor systems are described. In paragraph 5, available demountable connectors are illustrated. Paragraph 6 gives conclusions of this literature research.

2.1 Requirements for demountability and reuse

In this chapter an investigation is done to determine the requirements for demountable and reusable buildings and floor systems. The main objective of this thesis is to develop demountable connections for a demountable floor system, but the main reason for this objective is to contribute to the implementation of reusable floor systems. To increase the chance that the developed floor system can not only be demounted but also reused, the requirements for reuse are stated here as well.

If the requirements for a reusable floor system and for demountable connections are clear, the development of the design will be more efficiently.

2.1.1 Reusable buildings

To be able to reuse buildings and/or their components, they have to be designed for disassembly (DfD) and reuse. When the DfD approach is used, several extra aspects have to be regarded in comparison to the design of a permanent building.

Since buildings, and also floor systems, are made of different elements, the concept of Steward Brand is taken into regard; divide a building into separate layers which have different functions and lifespans [28], see Figure 2.1. Brand stated that a building can easily adapt if the layers are designed in such a way that they can be removed and replaced without having to change or demolish other layers in the structure. The same holds for deconstruction, if all the layers are not connected, it is easier to demount everything without damaging the separate parts.

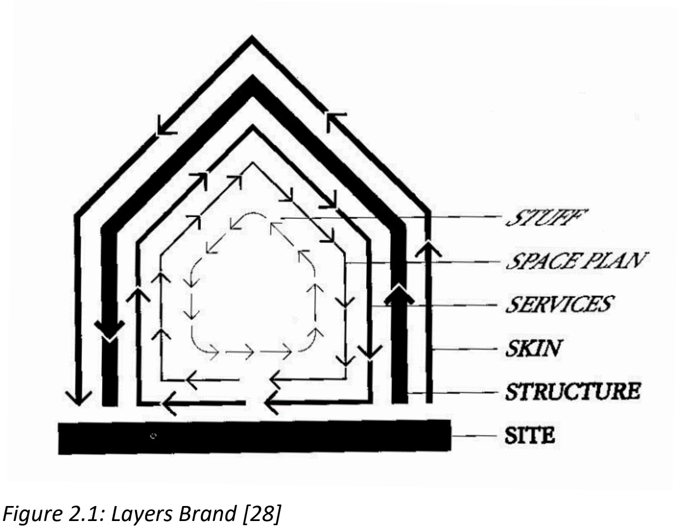

Figure 2.1: Layers Brand [28]

A reusable building will most likely be owned by several different companies/organisations which all have different functional requirements. To illustrate, take an empty office building that is acquired by a school with capacity problems. This results in a requirement for flexibility of the building in several different ways, like:

- The floorplan will change every time the ownership of the building switches, so it must be freely divisible. The layout can be changed handily if an open floor area is designed with a minimum number of columns and walls. This is achieved if long-span floor elements are used onto which light partition walls can be placed to create rooms. One of the difficulties of a changing floor plan is the position of recesses in the floor slabs for stairs, vertical transport of services, and so on. Another challenge is the changing of horizontal transport of services.
- The bearing capacity should be sufficient for all the possible categories of use which may be housed in the building. This also means that the building can be regarded as over dimensioned in some applications.

Furthermore, it is important to regard the extra steps which occur in the repetitive building process. Like a conventional permanent building, the building elements will first be produced, then transported to the site and the structure will there be erected. At the end-of-life, the traditional building will be demolished, and in some cases, partly deconstructed. A building which is designed for disassembly and reuse will be carefully deconstructed and then transported to a new location to be erected again. This means that a reusable building will be transported, erected and deconstructed several times before it reaches its overall lifetime. To ensure that the reuse of the building is efficient, this repetitive process should be optimised.

- To ensure an efficient (dis)assembly procedure, the structure should be designed in a simple manner. Using as few elements as possible and designing easy connections, reduces the erection and deconstruction time, and the possibility of mistakes, which both reduce costs. The erection and deconstruction time can also be decreased if the handling of elements is easy by using lightweight elements with moderate dimensions and if the same type of connector is used so that the construction workers don't have to switch between tools.
- The transportation should be made as efficiently as possible because the elements will be transported several times. If all the building elements can be transported using trucks which are allowed over the road at all times, costs and time are reduced. Therefore, the size of the building elements should be restricted to be able to fit in these types of trucks. The dimensions are: $L = 13600\ mm, W = 2550\ mm, H = 3000\ mm$ [29]. Moreover, the building elements should be dimensioned in such a way that they fit easily onto or besides each other. In this way the volume of the cargo hold of a truck can be used efficiently and no 'air' will be transported. In this case, less trucks will be required to transport all the elements and thus the harmful emissions and costs will be reduced. If the building elements are lightweight, transport of a loaded truck will require less fuel than for heavy elements which again reduces harmful emissions and costs.

Lightweight construction doesn't only benefit transportation and (dis)assembly. Using lightweight structural elements reduces the load on other elements in the building structure. When the acting load is reduced, the elements can be designed with a lower load bearing capacity which reduces material use and costs.

Besides the above stated requirements, the elements comprising the building should be designed in such a way that they can structurally be reused. Their load carrying capacity must remain sufficient during all the times it is used and reused.

2.1.2 Reusable floor system

All the important aspects for reusable buildings mentioned above can now be translated into specific requirements for a reusable floor system:

- A big free span length should be possible;
- Services must be easily adaptable;
- The load bearing capacity should ensure the housing of various categories of use;
- Easy demountable connections should to be used;
- (Dis)assembly should be straightforward;
- Element size and shape must be optimized to assist in an efficient (dis)assembly procedure and to optimize transportation.

These requirements all result in an added value for a regular floor system, making it demountable and possibly reusable. Besides these, a floor system must satisfy several other standard requirements regarding structural safety, building physics and fire safety.

The requirements for structural safety in ultimate limit state (ULS) and serviceability limit state (SLS) are found in the Eurocodes.

The requirements for building physical aspects and fire safety are found in the building decree [30]. In this decree, a differentiation is made between temporary constructions, new constructions and existing constructions. A temporary construction is a construction which is present in one place for a maximum of 15 years. It could finally be used as a permanent structure. For permanent structures, higher requirements are stated so these should be adopted. The requirements are also different for different categories of use, again the highest requirement should be adopted.

The structural functions of a floor in ULS are the transfer of vertical forces to the main load bearing structure and ensuring horizontal stability of the building by the transfer of horizontal forces to the stability system of the structure. Horizontal forces are transferred through diaphragm action of the total floor, see Figure 2.2.

Figure 2.2: Diaphragm action floor [10]

The floor functions in SLS are limiting deflections and assuring vibration comfort. A full dynamic vibration calculation is not required if the lowest natural frequency of the floor is not smaller than 3 Hz [31]. If the representative value of the self-weight and the quasi-static loading is bigger than 5 kN/m^2 or 150 kN per beam, the natural frequency may be higher [31].

Building physical aspects of a floor are thermal- and sound insulation between floor levels. Thermal insulation is expressed with the R_c-value which denotes the thermal resistance of a material or element. Sound insulation is split up in airborne sound and contact sound insulation.

Structural elements in a building must have a certain fire resistance. The fire resistance is expressed as the time in minutes during which the element must remain structurally sound, the fire safe time. The required fire resistance is dependent on the use function housed in the building and the height of the highest floor level, measured from ground level.

A building has to be equipped with several services, like piping for water supply and ventilation, and cables for electricity and internet. Generally, a requirement of the floor is to be able to integrate these services into the floor, which reduces the height of the floor package since the services don't have to be placed on top of or below the structural floor. Reduction of the floor package results in a reduction of the required height per floor and thus reducing the total height of the building. This reduces the material costs of e.g. the façade and might create the possibility to create an extra floor level in the available height. Integration of services can be conducted in several manners, for instance:

- For some concrete floor systems, it is possible to integrate services by casting them into the concrete. It is also possible to apply a service called concrete core activation (CCA). When CCA is applied, a piping system is cast into the concrete through which water is pumped. This water is hot in winter for heating and cold in summer for cooling.
- In some systems where cross girders are used, these girders can contain holes through which the services can be placed.

Casting in services and placing them through holes in girders goes against the principle of Brand and thus has a detrimental effect on the efficiency of deconstruction. So, in contrast to standard building practice, services should not be integrated in the above-mentioned fashions when designing reusable floor systems.

2.1.3 Demountable connections

Requirements for the connections between the floor slabs and the edge beams and between the floor slabs can be stated by reviewing the requirements set for reusable floor systems. Generally speaking all the connections:

- must be fully demountable;
- should be easy;
- should be made with the same type of connectors;
- must be protected from fire to assure the fire safe time of the floor is retained;
- must be insulated to assure the required thermal- and sound insulation between the floor levels.

The load carrying requirements for the connections are dependent on the manner in which the floor slab is supported on and connected to the edge beams. Therefore, the specific requirements are determined when the connections are designed. A few general statements about the load carrying requirements can be stated:

- In the connection located at the position where the floor slab is supported on the edge beams, usually at the head ends of the floor slab, all the forces acting on the floor and the self-weight of the floor slab are transferred. This connection must be designed in such a way that the floor surface can act as a diaphragm.
- The connections between the floor slabs must accommodate shear forces resulting from the diaphragm action of the floor. A bending moment could also occur due to the double bending of the floor slab.
- The loads on the connections between the side of the floor slab and the edge beam are the same as for the connections between the floor slabs. Depending on the manner in which the floor slab is supported, a part of the forces from the floor slab could be transferred here as well.

2.2 State of the art of demountability and reuse in the building industry

In this paragraph an overview is illustrated of some existing demountable building systems. Firstly, 'general' building systems which can be used for several purposes are introduced in paragraph 2.2.1. A distinction is made between the material used for the main structural elements. After this, specific examples of demountable building systems are given in paragraph 2.2.2. Finally, some information is given about current design guides for demountable building in paragraph 2.2.3.

This overview is given to emphasize the need for developments in the field of demountability and reuse in the building industry.

Building with modular units is explicitly disregarded in this thesis. Buildings constructed with these units can be deconstructed by unfastening the units, not the different building components like the walls and floors. Since in this thesis, the floor system will be designed to deconstruct separately, modular building units are not of interest.

2.2.1 Demountable building systems

Concrete building systems

In concrete building systems, almost all joints are 'wet joints' which means that cast in-situ concrete is used to connect elements. This is a big disadvantage when disassembly is required, the integrity of the building element is compromised and the systems are not fully demountable. To illustrate, a few concrete building systems are introduced and explained.

<u>MX-5 method</u>

In this building method, concrete rib floors and concrete columns are used [8]. Both ends of the columns have steel plates casted in, and the floor elements contain anchor holes. After the columns are placed and the floor element is laid on top, bolts are applied and fastened, see Figure 2.3. This system thus is fully demountable, only the bolts have to be loosened.

Figure 2.3: MX-5 method [8]

<u>Bestcon-30 system</u>

This system comprises concrete columns and a cassette floor [8]. In the top of the column a plate with four dowels and a pin is casted in and on the lower side a plate with a hole is casted in, see Figure 2.4. In the corners of the floor elements, a hole is present. After the installation of four columns, a floor element is placed in such a way that one of the dowels is placed inside the hole in the floor slab. Now the next layer of columns is placed after which the hole in which the pin is inserted is filled with mortar.

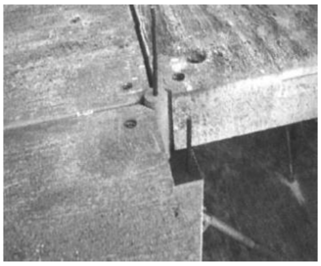

Figure 2.4: Bestcon-30 system [8]

<u>CD20 system</u>

The CD20 building system [3, 8] is depicted in Figure 2.5. Four columns are placed, which contain four cast-in steel pens on both ends. A floor slab with a hole in each corner is placed on top of the columns and the next layer of columns is placed. Now the grouting slots present in the floor elements are filled.

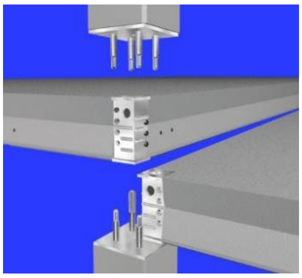

Figure 2.5: CD20 system [3]

Steel building systems

It is common practice to connect steel elements with welds or with demountable connectors, like bolts. When using bolted connections, steel building systems are a logical choice when designing for deconstruction. Several building systems exist which are not created for deconstruction but can easily be used for this purpose.

Steel framed construction[1]

Steel framed construction [32, 33] uses diaphragm elements for the floors and walls. These elements are constructed of frames made of cold formed, thin walled C- and U-profiles, covered with plates. The assembly of the frames can take place on site, but it is preferred that they are assembled in the factory. Forces are transferred through the diaphragms in a perpendicular and parallel direction, so they ensure vertical and horizontal load transport. Often extra wind bracings are applied. An example of a steel framed construction system is Star-Frame, see Figure 2.6.

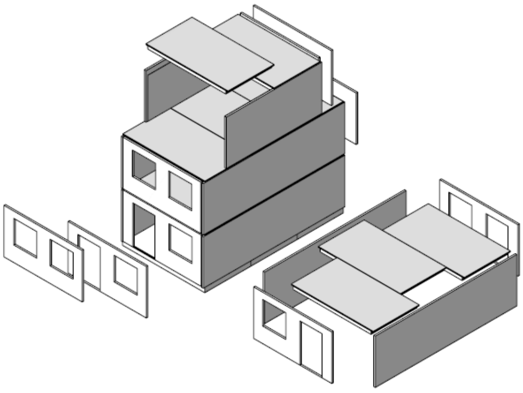

Figure 2.6: Star-Frame construction [22]

Steel frame construction[2]

Steel frame construction [32, 34], even though it sounds the same as the steel framed construction, uses hot formed, thick walled steel sections. A frame is formed by connecting steel columns, beams and stability elements with bolts or welding. The walls, floors and roof are subsequently attached to this frame.

An innovative modular steel frame construction system is the ConX system [35]. It uses plug-and-play type joints to increase erection speed.

Timber building systems

The connectors used to attach timber elements are always demountable, but some can damage the element. So, with the right choice of connector, timber building systems are a good choice when a deconstructable building is to be designed. An important aspect to consider when using timber, is durability. The timber elements can be deteriorated during the use phase of the structure by for instance the presence of water or micro-organisms. The elements can be protected by either chemicals or mechanical methods, but these preservative actions have some disadvantages. The use of chemicals is environmentally unfriendly, reduces the recycling potential and, for the use of some substances, the chemically treated elements cannot be handled by hand. Mechanical protection increases the required time for design and erection.

[1] In Dutch 'staalframebouw'
[2] In Dutch 'staalskeletbouw'

Timber frame construction[3]

Timber frame construction [36], is the same as steel framed construction, but the material used to construct the frames is wood instead of steel, see Figure 2.7.

Figure 2.7: Timber frame construction [37]

2.2.2 Structures designed for deconstruction

In this chapter, a few existing structures which are designed with the DfD methodology are analysed.

Demountable buildings

Temporary courthouse Amsterdam [17]

In order of Rijksvastgoedbedrijf, IMd Raadgevende Ingenieurs designed a temporary building to house the courthouse of Amsterdam. The main support structure is made of steel columns, beams and wind bracings jointed by bolts. The floor slabs are concrete hollow core slabs, supported by integrated girders. They are connected with a demountable connection, see Figure 2.8. The term 'reusable connection' is not used with reason. The floor slabs were not completely prefabricated: the bolt anchors were casted-in on site after fixing them to the integrated girder. This might cause tolerance problems when the building is to be reassembled because the chance is rather small that the steel skeleton will be erected in exactly the same manner as the first time.

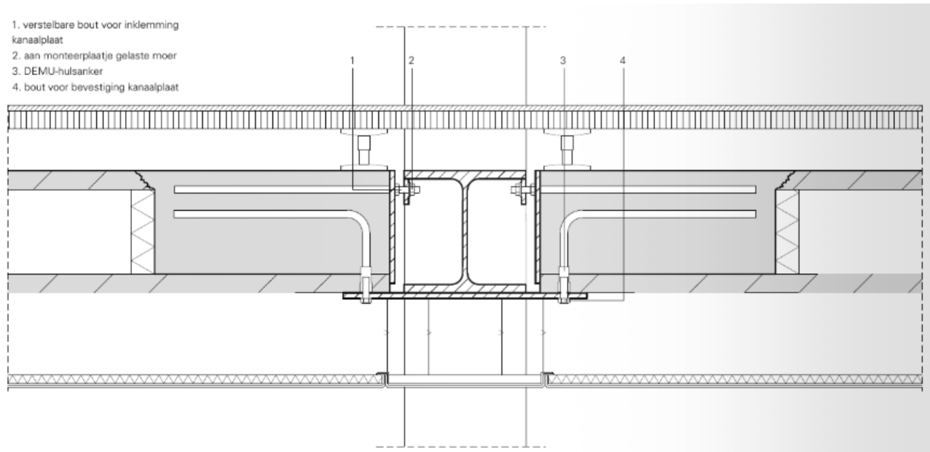

Figure 2.8: Connection HCS-girder temporary courthouse Amsterdam [17]

[3] In Dutch 'houtskeletbouw'

Neptunus demountable buildings [38]

The company Neptunus has designed several demountable construction systems. These can be used for the design of many different building types, like sport halls, residences and schools. The Flexolution and Flex2Home systems are made with extruded aluminium columns, laminated wooden girders, wind bracings and sandwich panels for the walls and roof, see Figure 2.9. The floor can either be made of concrete or isolated wooden panels placed on top of steel and wooden girders. In the Evolution building system, the roof is first constructed with a spaceframe and afterwards lifted with hydraulic jacks, see Figure 2.10. The columns, wind bracings and wall sandwich panels can now be placed. All these systems result in a rapid erection.

No detailed information is available on the connection method of the floors to the timber or steel girders.

Figure 2.9: Flex2Home construction [38]

Figure 2.10: Evolution construction [38]

Demountable parking garages

Below, a few existing demountable parking garages are introduced and explained. Because the objective of this research is to design a demountable floor system for utility buildings, it is important to remember that the requirements for floors in parking garages are different from the requirements for utility buildings.

PARK4ALL parking system

The PARK4ALL parking system [39] has been developed by the company PARK4ALL BV in collaboration with Royal HaskoningDHV. The system consists of a steel frame structure with bolted connections and floor elements made of a fibreglass plastic composite material. The load bearing structure consists of steel columns, main girders and secondary girders. The grid made by the girders ensures the load transfer, the fibreglass floor panels serve as non-load bearing elements. This results in a flexible, lightweight and quickly erected parking solution.

The fibreglass floor panels are damaging for the environment. Because they are non-load bearing, they can only have small dimensions to be able to stay structurally sound for the load distribution to the secondary girders. This means that a tight grid of steel girders is required which results in a lot of required elements to make the floor field, which is detrimental for the erection speed and the costs.

Parking garage ASML Veldhoven

The parking garage for the company ASML [40] is designed by Arcadis. It is constructed using steel columns and IPE beams bolted together and a TT slab floor system. The TT slabs contain holes in which doves fall which are welded to the IPE beams. The holes are filled with mortar and the seams between the TT slabs are filled with mortar and sealed with a silicone sealant. The instalment of the TT-slabs is shown in Figure 2.11. Due to the wet joint, diaphragm action is ensured. For disassembly, the doves are excised, and seams are cut after which the hardened mortar is removed. These actions might damage the jointed parts. Furthermore, the surfaces that are cut loose have to be cleaned before they can be reused again, which increases costs.

Figure 2.11: Instalment TT-slabs parking garage ASML [13]

EZ Park method

The company EZ Park has also designed a demountable parking structure [41, 42]. This system is made of a steel frame structure and a TT slab floor. The TT slab elements are linked with coupling plates which are bolted together. The seam between the slabs is sealed with a silicone sealant to ensure liquid-tightness of the floor. The TT slabs are supported by the steel beams onto which doves are welded and which fall into holes present in the concrete slabs. After placement of the slabs, the holes are filled with mortar. When the garage is demounted, the doves are excited, and the sealant is cut and removed. The doves might be damaged when the mortar is removed, and the surfaces have to be cleaned before they can be used again. An example is the parking garage is Gerstdijk, which has won the Dutch national steel award in 2014, see Figure 2.12.

Figure 2.12: Top deck parking garage Gerstdijk

Continental car parks

Continental car parks has created a demountable parking structure in collaboration with Tata Steel called the Flexideck [43]. A few parking garages are (being) built with this system, like the parking garage Morspoort in Leiden, parking garage Raadhuisplein Hoofddorp and parking garage Rotterdam Alexander. The parking garage is made of a steel structure with the Quantum floor system, connected with bolts.

2.2.3 Design guides on demountable construction

In the Netherlands no design guides to support design for deconstruction are available at present. However, in the United Kingdom, the Construction Industry Research and Information Association (CIRIA) has made a guide which discusses the possibilities for designing for reuse of components and recycling of materials, called the C607; Design for Deconstruction: Principles of design for deconstruction to facilitate reuse and recycling [44]. The Scottish Ecological Design Association (SEDA) has published the Design for Deconstruction, SEDA Design Guides for Scotland: No. 1. It addresses resource efficiency, approaches and principles involved in DfD and provides details which can be used for deconstruction.

2.3 Environmental impact calculation tools

The building decree states that for all office- and residential buildings, built after the first of January 2013, an environmental impact calculation must be executed. The environmental impact is expressed with the Environmental Cost Indicator (ECI). This monetary value illustrates the costs needed to compensate the damage which is done to the environment in terms of €/kg material. The calculation must be conducted using the document 'Estimation method for calculating environmental impact of buildings and civil engineering constructions[4]' [45]. This report states that it is mandatory to use the environmental impact information of basic materials and processes which is stated in the National Environmental Database and that a minimum of eleven environmental impact categories must be considered. Several tools are available to determine the ECI, which are illustrated below. The difference between the tools is found in the additional categories that are regarded.

- The Dutch Green Building Council has created the BREEAM-NL tool [46, 47]. It can be used for new and existing residential-, industrial- and utility buildings or renovations. It focusses on energy, materials, water use, land use & ecology, management, health, transport, waste and pollution. These topics include the eleven required categories and some extra criteria.

- The GPR Building tool [47, 48] determines the sustainability performance of new and existing buildings and renovations regarding residential- and utility buildings. A building is scored on five main topics; energy, environment, health, user quality and future value. These include the eleven categories and some additional criteria.

- GreenCalc+ [47, 49] is used for the assessment of new and existing offices, schools and houses. This method determines the ECI based on the eleven environmental impact categories. Further, the topics mobility and material-, energy- and water use are considered.

- MRPI [50] is a free tool which can be used to determine the environmental impact of buildings. For the eleven categories and additionally waste and energy- and water use, the environmental impact during production, waste transportation and waste processing is determined.

- DGMR, an engineering- and consultancy company, has made a free software called MPGcalc which is used to determine the environmental impact per material used in a building [51].

- The DuCo tool, created by IMd Raadgevende Ingenieurs can be used to calculate the ECI based on the eleven impact categories. The ECI due to the production of the material and the ECI resulting from the processing of the waste of the material after use are combined to obtain the total ECI for the material.

[4] In Dutch 'Bepalingsmethode Milieuprestatie gebouwen en GWW-werken'

2.4 Floor systems

In this chapter several floor systems are illustrated and a distinction is made with regards to the main material(s) used. Furthermore, the layup and erection of the systems, general used dimensions, the load transfer mechanism, and important characteristics are described. One of the illustrated floor systems can be chosen to be implemented in the developed demountable floor system.

If the manner in which the services are usually integrated have a detrimental effect on the ease of changing the services, the manner is not mentioned. For all floors, services can be placed on top of or hung from the floor. If the services are placed on top of the floor, a raised floor system (computer floor) can be used to cover the services. If they are hung from the floor, a suspended ceiling can be applied to conceal the services.

2.4.1 Concrete floor systems

Concrete floor systems are made of prefabricated (prefab) elements and/or cast in-situ concrete. When prefab concrete slab elements are used, the elements have to be connected to each other and to the supporting beams to enable diaphragm action. This can be achieved by either filling the space between the elements with mortar to form a shear joint, casting a structural reinforced topping or welding or bolting two elements together with cast in steel plates. A topping is often used because it fills joints, increases the horizontal stability, strength and rigidity and improves the load distribution in transverse direction.

Completely cast in-situ floor systems are disregarded in this research since it is impossible to disconnect this type of floor without having to demolish the concrete or the use of extensive measures to create demountable slab elements.

A concrete floor can also be partially prefab and partially cast in-situ. If the in-situ concrete is only used to a small extend, the floor system is regarded as potentially demountable and is thus included in the research. When the concrete is used to fabricate the biggest part of the floor, the floor system is regarded as not suitable for disassembly and thus disregarded. The reinforced plank floor, poly slab floor and bubbledeck floor were considered but left out for this reason.

Hollow core slab floor

One of the most widely used floor systems in the Netherlands is the hollow core slab (HCS) [2, 52, 53]. It can be used in several types of buildings, like residential- and utility buildings. The system is made up of prefab eccentrically prestressed concrete slab elements with holes, or cores, in the length of the slab for weight reduction, see Figure 2.13. An HCS usually has a width of $1200\ mm$. The maximum span is $18\ m$ and the thickness ranges between $150 - 400\ mm$, depending on the span and the loading conditions.

Figure 2.13: Hollow core slab [21]

The slabs are placed side by side and the longitudinal joint is filled with mortar to form a shear joint. This shear joint transfers shear forces for horizontal stability and distributes loads in the transverse direction. Since only prestressing in longitudinal direction is applied and no distribution- and shear reinforcement is used, load transfer only occurs along the length of the slab.

The elements can be placed on a steel support. Usually, a rubber bearing is placed in between the slab and the supporting beam. The slabs could also be placed on top of the bottom flange of a steel profile, this type of beam is called an integrated girder. Using an integrated girder decreases the height of the floor package.

Prefabricated solid slab floor

The solid slab floor [2] is made of prefab prestressed solid concrete elements, see Figure 2.14. These elements are made with a span of up to $12\ m$, a width of $1200\ mm$ and a thickness of $150 - 300\ mm$ depending on the span and the loading conditions.

Figure 2.14: Solid slab floor [15]

The erection and load transfer are the same as for an HCS. Due to the increased weight, the load bearing capacity is lower than that of an HCS.

TT slab floor

A TT slab floor [2] is made of elements comprising a prefab, prestressed and reinforced top floor slab with two webs, also called ribs, see Figure 2.15. One element usually has a width of $2400\ mm$. The maximum span is $22\ m$ and the thickness is $400 - 1000\ mm$ depending on the span and the loading conditions. The load bearing capacity is quite big and therefore, it is mostly used for structures like parking garages.

Figure 2.15: TT slab floor [7]

The elements are placed side by side and the longitudinal joint is filled with mortar, resulting in a shear joint. The floor is supported by the main structure at the webs or at the narrowed end cross section. In between a rubber bearing is placed. Sometimes a structural topping is casted in-situ.

Services and insulation can be placed in between the ribs. This results in a height reduction while the services can still be changed in a simple manner.

2.4.2 Composite floor systems

A composite floor is a floor which is built up out of several parts which are connected in such a way that they display composite action. In engineering, composite action means that two or more separate elements, either made of the same or of different materials, work together to bear loads. The materials are connected to each other with glue or shear connectors. The simplest example is of timber beams loaded with a point load in the middle with either no connection or connected to each other with steel rods. When they aren't attached to each other, each beam will deflect independently resulting in tension at the bottom and compression at the top in each beam, see Figure 2.16. Every beam has to carry an equal amount of load. When the beams are connected, for instance closely to the supports, the beams will act together resulting in a continuous stress distribution over the height, thus tension the bottom beam and compression in the top beam, see Figure 2.17. When a component is comprised of a material which has good compressive properties on top, and a material which has good tensile properties below, the elements are loaded in their most favourable way.

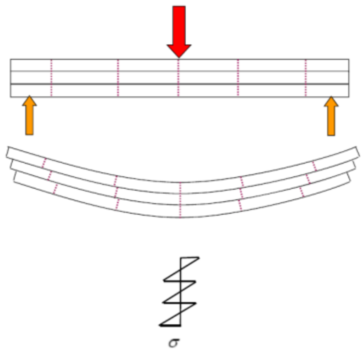

Figure 2.16: No composite action [11]

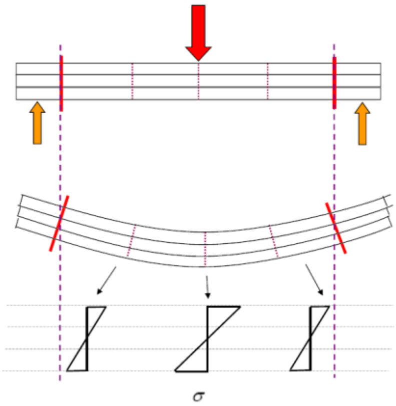

Figure 2.17: Composite action [11]

There are several advantages to using a composite floor system. The thickness of the floor can be reduced due to the composite action, which results in a material and weight reduction. Due to the weight reduction, the load bearing structure and foundation are loaded less heavily, which can result in a cutback in material use. Due to the smaller thickness of the floor package, the storey and thus building height can decrease. Now, the building height can be lower, which gives a reduction in the use of façade material or the reduced floor thickness could result in the addition of an extra floor level.

Composite steel deck-concrete floor

Another floor system that is common in the Netherlands, is the composite steel deck-concrete floor system [2, 52, 53]. It is comprised of a steel supporting beam, a thin steel sheeting and a concrete topping, see Figure 2.18. A span of maximally $6 - 9.5\ m$ can be obtained depending on the total thickness of the floor, the loading conditions and the use of propping. The thickness is usually no more than $130 - 350\ mm$, depending on the used steel deck.

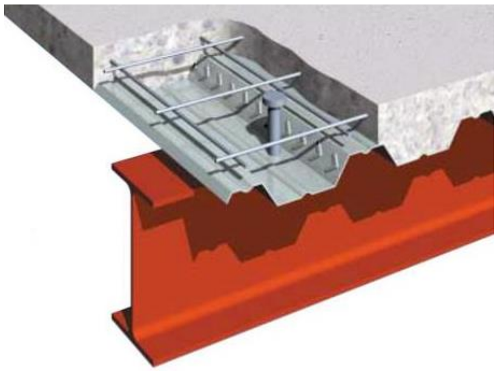

Figure 2.18: Steel deck-concrete floor [2]

The steel sheeting is connected to the supporting steel beam with shear connectors. After fastening the sheeting and placing of the reinforcement, concrete is poured on top. The concrete is activated in compression and the steel in tension, which are the most favourable properties of the materials. The composite action is created with the shear connectors and the dents in the steel sheeting. It is also possible to place the steel sheeting on an integrated girder.

Due to the ribs in the steel sheeting, the slab bears load in one direction. If bi-directional load transfer is required, continuous linear supports should be applied.

Research is being conducted into the use of demountable shear connectors for these types of floors. Using these novel connectors will create the possibility of reuse while still gaining the benefits of composite construction.

Cofradal

The Cofradal floor system [52] is made out of elements consisting of a profiled steel sheeting, mineral wool and a reinforced concrete topping, see Figure 2.19. The thickness of the floor is $200\ /\ 230\ /\ 260\ mm$ and the width is $600\ /\ 1200\ mm$. The span ranges form $2.5 - 7.5\ m$ depending on the required load bearing capacity and the applied thickness.

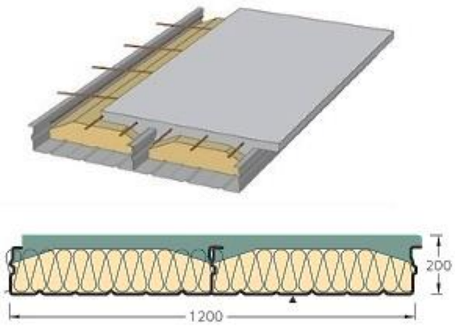

Figure 2.19: Cofradal floor [12]

The elements have a special profiling due to which they clasp each other when they are placed side by side. Now, the elements can transfer shear forces. Often a concrete topping is cast in-situ. The steel profile is activated in tension and the concrete top in compression which gives favourable composite action.

Slimline floor

The slimline floor system [2, 54, 55], formerly known as the Infra+ floor system, is made out of elements which consist of a prefab concrete lower shell, two or three steel profiles, a rubber bearing granulate and a top flooring, see Figure 2.20. The standard element widths are $2400\ /\ 2700\ /\ 3000\ mm$. A span of $4.5 - 16.2\ m$ can be achieved depending on the used steel profiles, their spacing and the loading conditions. The concrete shell thickness ranges from $70 - 80\ mm$ and the used profiles range from IPE180 to IPE600 resulting in a total thickness range of $213 - 633\ mm$.

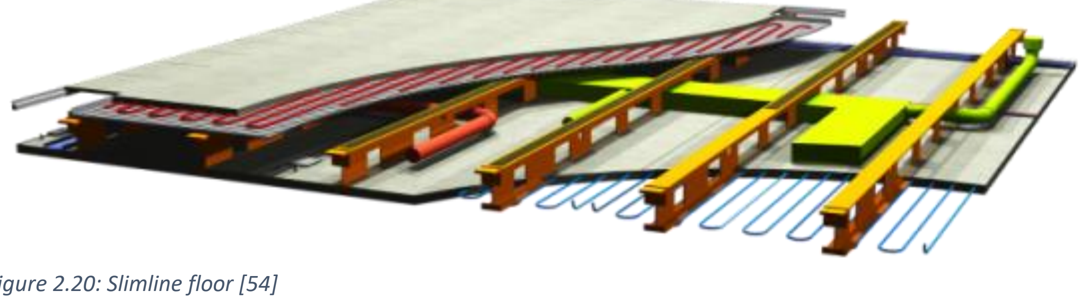

Figure 2.20: Slimline floor [54]

The bottom flanges of the steel profiles are casted into the concrete in the factory. The floor elements are placed side by side and the concrete shells are connected with casted-in steel plates which are welded together. The elements are hinged to the main support system via the steel beams. On top of the beams a rubber granulate is placed to ensure sound insulation. Lastly the top floor is placed on the granulate, which can be a thin steel-concrete composite floor or a wooden plank floor.

The main load transfer occurs in the direction of the steel beams. The connection between the concrete shells is capable of transferring small shear forces. The horizontal stability can be ensured by the concrete shell and/or the top floor.

The prefab elements, consisting of the concrete shell and steel beams, display composite action, but the materials are not placed in a manner in which they are loaded in their most favourable direction. The top floor is not connected to the steel beams, due to which no composite action between the top floor and the prefab element occurs.

Quantum floor

The quantum floor [20, 55, 56] consists of elements made up of cold formed C-profiles, angle sections and a reinforced concrete deck, see Figure 2.21. The most common used widths are 2400 / $3000\ mm$ but widths of $n * 600\ mm$ can be made as well. Spans of $5 - 11\ m$ are possible. The thickness of the deck is always equal for a specific use, for inside use in residential buildings it is $51\ mm$. Usually C220 is used but for large spans and large loads a bigger C-profile can be used. The range of the total thickness is $216 - 331\ mm$.

Figure 2.21: Quantum floor [20]

With back-to-back C-profiles and angle sections, a frame is formed onto which a reinforcement net is placed. The C-profiles can be made with a precamber to take up the deflection due to the self-weight of the floor element. In transverse direction the C-profiles could be linked with cross profiles. These cross profiles enable an even distribution of load in the floor and reduce vibrations. The frame is turned upside down and the concrete is poured. In the C-profiles, holes are present to be able to connect the profiles and to integrate services. At the head ends, where the elements are supported on the main structure, angle profiles are present. The angle profile and the steel supporting beam are usually joined with a bolted connection but can also be welded together. The concrete and steel parts are loaded in their strongest direction and thus composite action is activated in a good manner.

2.4.3 Steel floor systems

Steel floor systems are mostly used in steel frame(d) constructions. Systems with many separate loose elements decrease the ease of (dis)assembly and thus systems consisting of many single elements are disregarded. For this reason, the grate floor and beam floor were disregarded.

Star-Frame floor

The Star-Frame floor [22, 55] is almost the same as the quantum floor. The differences are that the C-profiles are single, and the concrete shell is replaced by either a thin composite steel deck-concrete floor or a wooden plate material, see Figure 2.22. The maximum span is $10\ m$ but the optimal span is $7.2\ m$.

Figure 2.22: Star-Frame floor [22]

Since no composite action exists between the steel frame and the top floor, the diaphragm action is smaller and thus the horizontal stability is reduced.

Ides floor system

Ides [4, 53] stands for Integrated Deck Extra Space. The system is made up of integrated steel I-profiles, cold formed U-shaped elements, rock wool and a top floor, see Figure 2.23. The U-profiles are $333 - 500\ mm$ wide so the total width of the floor can be any multiple of this width. The span can reach a length of $7.2\ m$. The thickness is about $300\ mm$.

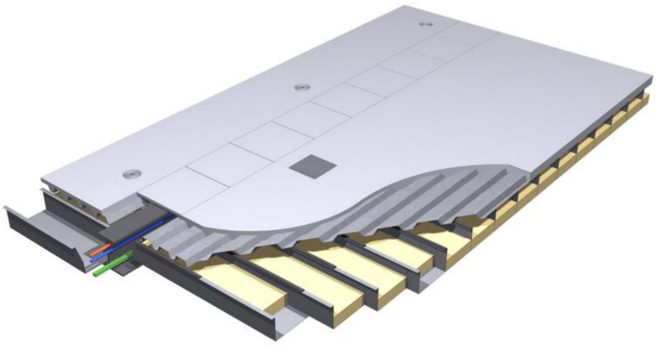

Figure 2.23: Ides floor [4]

The I-profiles are placed and on the lower flange the U-profiles are situated. On top of the U-profiles the top floor is placed, which can be a composite steel-concrete deck or wooden plate material. In the U-profiles, services and insulation can be placed. Furthermore, in the remaining area of the steel bottom flange services can be placed, although this space is limited.

2.4.4 Timber floor systems

Timber floor systems are mostly used in small residential buildings because of their limited load bearing capacity and low sound insulating properties. Due to the increased demand for sustainable building materials and the development of higher strength wooden products and profiles, the implementation of timber floors is becoming more interesting. An important aspect to consider is the durability of wood. Solid wood usually doesn't contain toxic agents, like adhesives, binders or preservatives, therefore it can be recycled or burned for energy release. When engineered timber products are used, these toxic resins are present, and the products can't be reused or burned. But in these wooden products, only fast-growing young trees are used, high tolerances are present and higher strength characteristics are obtained.

As for the steel floors, systems consisting of many single elements are disregarded. The beam floor was therefore left out.

Hollow core slab

Wooden hollow core slabs [6, 16] can be made with box- or surface elements.

Box elements (see Figure 2.24) have a standard width of $200\ mm$, a height ranging from $120 - 320\ mm$ and a maximum span of $12\ m$. The separate boxes are connected with a tongue-groove connection at the top and bottom and a horizontal screw connection every $1.5 - 2.0\ m$. The connection to the support can be made by either a diagonal screw on the tongue side or a vertical screw through every first web of every two boxes. To obtain diaphragm action, wooden plates can be screwed on top.

Figure 2.24: Wooden HCS, box elements [6]

Surface elements (see Figure 2.25) are made of a top and bottom plate with ribs in between. They have a standard width of $514\ mm$, $1000\ mm$ or $2400\ mm$ and a maximum span of $16 - 20\ m$ which depends on the used plates and ribs and thus on the thickness of the floor. The parts can be connected with diagonal screws and/or a tongue-groove click connection at the lower plate. The connection to the support is made with vertical screws through the middle webs. Extra horizontal stability can be obtained by using shear bolts between the elements.

Figure 2.25: Wooden HCS, surface element [6]

For both systems headboards are used at the ends. These increase the stability and can be used as support. Extra boards in the span can be used to increase the resistance against lateral torsional buckling.

The ends of the elements can be supported on a beam, an integrated beam, a steel plate section or a wooden console and then connected with screws.

Rib floor

The rib floor system [16] consists of elements made out of a top plate glued onto ribs, see Figure 2.26. The standard width used is $2400\ mm$ and the span ranges between $16 - 20\ m$ which depends on the used plate and ribs and the loading conditions.
At the ends, headboards are used, these increase the stability and can be used as support. Extra boards in the span can be used to increase the resistance against lateral torsional buckling.

Figure 2.26: Wooden rib floor [16]

After the elements are placed next to each other, diagonal screws are used to connect the parts. The elements are supported at both ends on a line-support, like a beam, an integrated beam, a steel plate section or a wooden console.

Services can be integrated in between the ribs.

CLT floor

A CLT floor [57, 58] is a plank floor made of cross laminated timber (CLT), see Figure 2.28. CLT is made of solid wooden planks glued crosswise together, see Figure 2.27. The maximum width is $2950\ mm$, the maximum span is $16\ m$ and the thickness depends on the required span and loading conditions. The elements are to be supported by a line support, for instance a steel angle.

Figure 2.28: CLT floor [5]

Figure 2.27: Schematic of CLT layer configuration [23]

2.5 Demountable connections

In this chapter an overview is given of available connectors which can be demounted. Knowledge of existing demountable connectors and connections can support the development of a new demountable connection.

A connector type is regarded demountable if the removal of the connector doesn't result in damage of the joined elements. The elements comprising the connection, and the type of loading which can be carried are described. Only a distinction between lateral and axial loads is made; a lateral load is in the direction perpendicular to the fastener, often a shear force, and an axial load is either a tensile or compressive load in the direction of the connector.

Bolted connections

Bolts can be used for connections between steel, timber and concrete. They can carry lateral and axial loads. A bolt [59] consist of a head, shank, thread, washer and nut, see Figure 2.29. Usually, the hole is an oversized hole, which means that it's diameter is bigger than the diameter of the bolt. This ensures that the holes in the joining parts will (partly) overlap, so the bolt can be inserted. The bolt can be either tightened 'normally' with a wrench, so slip can occur between the connecting parts, or tightened by preloading due to which no slip can occur. Preloading is used when the bolt is mainly loaded in tension and if cyclic loads occur. A washer is placed under the head and/or under the nut to ensure sufficient contact with the jointed members and, for preloaded bolts, to avoid damage of the joined parts.

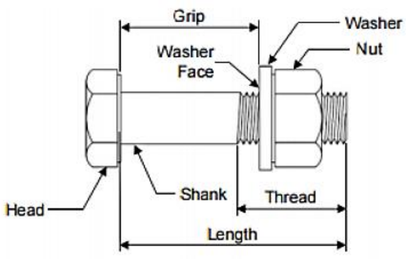

Figure 2.29: Bolt parts [14]

If, for instance, steel hollow sections must be joined, the bolt can't be reached on the inside of the section. In this situation blind bolts can be used, these can be installed in pre-drilled holes with a threaded surface or they expand like an umbrella after turning it.

Doweled connections

Dowels are used in timber-to-timber and steel-to-timber joints, used to withstand lateral loads, see Figure 2.30. They are inserted in a predrilled hole in the timber element(s) which is as big as the dowel diameter and a hole in the steel member(s) which has a clearance. A dowel is not fastened like a bolt and can therefore be pushed out, the axial loadbearing capacity is thus negligible.

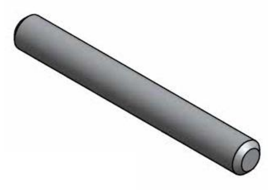

Figure 2.30: Dowel [19]

Screwed connections

Screws can be used for timber-to-timber and steel-to-timber joints. They can withstand lateral and axial loads. Several types are available, like the countersunk head screw, the coach screw (mostly used for steel-tot-timber connections), and the assy screw, used for high strength connections, see Figure 2.33, Figure 2.32 and Figure 2.31 respectively. Mostly self-tapping screws are used since this excludes the extra work of predrilling.

Figure 2.33: Countersunk head screw [19]

Figure 2.32: Coach screw [19]

Figure 2.31: Assy screw [19]

Carpentry joints

Carpentry joints, also called contact joints, are used in timber construction. They are made by cutting timber elements which have to be joined so that they fit into or onto each other. The load which can be transferred are dependent on the geometry of the joint. Examples of basic forms are the half-lap joint and the mortise and tenon joint, see Figure 2.34 and Figure 2.35 respectively.

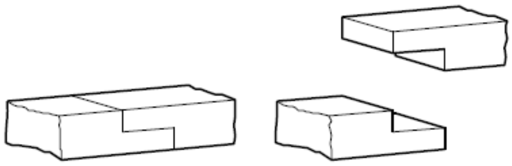

Figure 2.34: Half-lap joint

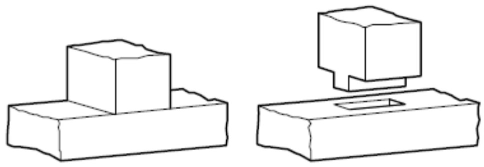

Figure 2.35: Mortise and tenon joint

Timber or metal fasteners can be added to the joints to ensure they stay in their intended location. These fasteners sometimes increase the load bearing capacity of the joint. An example is a mortise and tenon joint fastened with a wooden wedge, see Figure 2.36. An alternative on the basic carpentry joints is for instance a dovetail joint fastened with a wooden peg, see Figure 2.37.

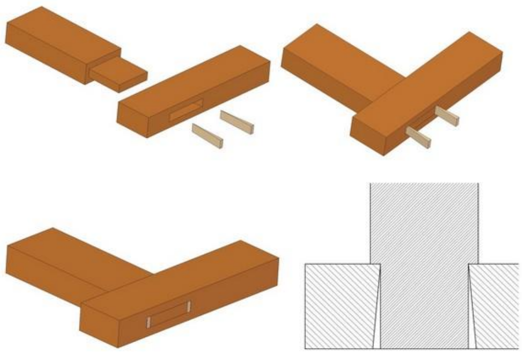

Figure 2.36: Mortise and tenon joint with wedge [60]

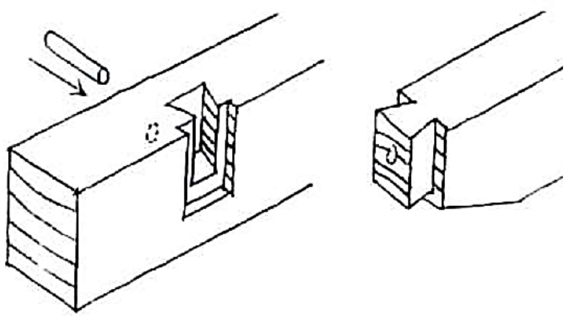

Figure 2.37: Dovetail joint with peg. Adapted from [61]

Surface connectors

Surface connectors are used for timber-to-timber and steel-to-timber connections loaded laterally and axially. The lateral load is carried by the plate or ring and the axial load is carried by the bolt or screws. Examples are the split-ring, shear-plate and toothed-plate connectors, see Figure 2.38, Figure 2.39 and Figure 2.40 respectively. Single-sided connectors are used for steel-to-timber and timber-to-timber connections and double-sided connectors are used for timber-to-timber connections.

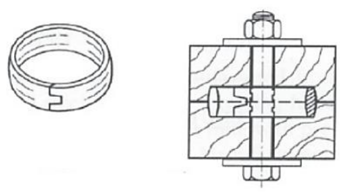

Figure 2.38: Split-ring connector [9]

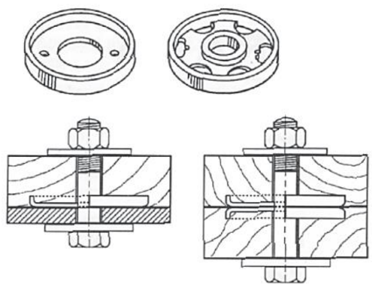

Figure 2.40: Shear-plate connector [9]

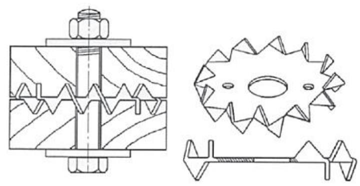

Figure 2.39: Toothed-plate connector [9]

Three-dimensional plate connections

3D plates are used for timber-to-timber connections. Examples are joist hangers and brace anchorages, see Figure 2.41 and Figure 2.42 respectively. The cold-formed steel plates are fastened to the timber with either nails or screws. Since there are several forms, not only one loading type can be stated.

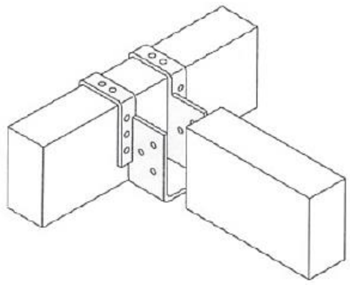

Figure 2.41: Joist hanger [9]

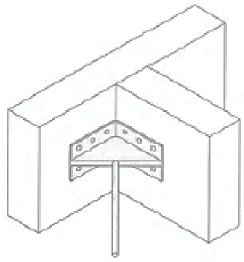

Figure 2.42: Anchorage bracing [9]

Tab and slot connections

Tab and slot connections are found in steel constructions, and they look a lot like carpentry joints. One of the connecting parts has a tab which is inserted into the slot present in the other element, see Figure 2.43. The tab can have different shapes, like a dovetail. It is possible to design the connection in such a way that the connected parts can be fastened with a wedge or a bolt, like with a t-nut, see Figure 2.44. Depending on the shape of the tab and whether the connection is fastened, it can carry only axial or lateral loads or both.

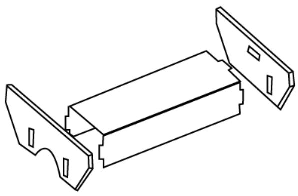

Figure 2.43: Tab and slot connection [62]

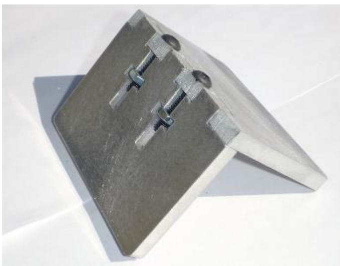

Figure 2.44: Tab and slot connection with T-nut [63]

Plug-and-play connections

Plug-and-play connections are those in which two elements, or their attached connectors or connector plates, are placed onto or into each other. Depending on the situation, the joined elements are in their final position or they will be fastened after or during the remaining erection time. A big advantage of this type of connection is a fast erection speed due to the smaller hanging time of the element on a crane and the possible elimination of fastening of bolts etcetera.

An example is the snap-fit connection [18, 64]. It looks a bit like a steel dovetail connection, see Figure 2.45. It is a relatively new type of connection designed by A. Verbossen. In the groove two holes and in the sled two press-able pins are present. The sled is slid into the groove and when the pins are aligned with the holes they snap into the holes, fixing the connection.
In the design of this connection not a lot of thought was put into the occurring tolerances. Therefore, fitting of the elements might become a problem.

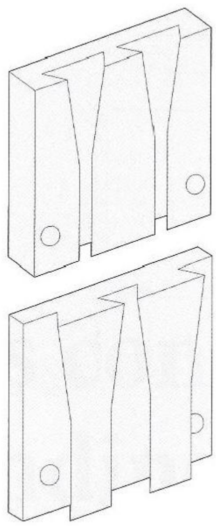

Figure 2.45: Snap-fit connection [18]

2.6 Conclusions

Aside from the standard requirements set for floor systems, additional requirements can be formulated for reusable floor systems. These requirements can assist in an efficient development of a reusable floor system. The requirements are: flexibility of the floor system, an efficient erection, deconstruction and reassembly procedure and the use of demountable connectors.

Concrete building systems are not suitable for designing buildings to be deconstructed. Standard practice in these type of building systems is the use of wet joints. Changing this manner in which elements are connected will most likely be elaborate and costly.

Steel building systems can be made with demountable connectors, like bolts. Therefore, many steel buildings are already deconstructable. This doesn't mean that the current steel building practice is prefect to be used for reusable buildings. One important aspect which is not regarded when designing for deconstruction and not for reuse is that the buildings, or building systems, have to be reassembled several times. To try and make an economically viable reusable system, this reassembly procedure has to be considered during the design. The erection, deconstruction and reassembly should be efficient. Many current used bolted connections are complex and require a lot of work on site, which doesn't make them suitable to use in reusable buildings, building systems or building elements.

Timber buildings are made with demountable connectors. Just like for the steel building systems, these connections are not designed to be reused.

Only one example of a floor systems designed for reuse and implemented in practice was found of which detailed information was available for an assessment of the design. The principle of the design is inspiring. The only adaptation that should be made to ensure that reuse is efficient is the implementation of space in the connecting elements for the accommodation of fabrication and assembly tolerances.

From all the illustrated parking systems, only the Flexideck provides an efficient and fully demountable parking garage. The other parking systems either require the handling of many elements on site or still use wet joints which have to be broken at disassembly.

Two documents containing design guidance on designing for deconstruction can be found. In the Netherlands such documents are not available. Moreover, no design guidance is found which can support designing for deconstruction and reuse.

The ECI of buildings and building elements can be calculated using a lot of different calculation tools. Every tool has a different calculation method and different topics on which the building or the element is assessed.

A lot of different floor types and demountable connection types exist. Many illustrated floors and connectors are widely used in practice. All the different types have situations in which they are best implemented and they have specific advantages and disadvantages. The choice of which floor slab and connector to use in the reusable floor system can't be made based on the general data given of these floor slabs and connectors. Therefore, in the next chapter a multi-criteria analysis is done to determine which floor slab should be used. The type of connector(s) used is determined in chapter 0, after the type of floor slab is chosen and designed.

3 Multi-criteria analysis

In this chapter a multi-criteria analysis (MCA) is conducted to determine the best choice of floor system to implement as a demountable floor system for a given situation. The manner in which the MCA is executed is first explained after which some assumptions are given. Hereafter, the criteria used in the analysis are clarified. Finally, a conclusion is formulated. In Appendix A, an overview of the input data, calculations and results of the MCA can be found.

3.1 Method

A multi-criteria analysis is a tool to determine the best choice of alternative for a specific situation. In this MCA this specifically means the best choice of existing floor system to be used as a demountable floor system in a given situation.

First, the situation is specified, which is done by determining a functional unit (FU). This FU defines the function and the required performance characteristics of the floor system. It serves as a reference by which the input data of each floor is normalised [65]. By determining and using one FU for all the floor systems, the comparison between the systems is impartial.

Hereafter, the requirements for demountable floor systems are transformed into specific criteria, which may be divided into sub-criteria. Each floor system is rated per sub-criterion on a scale of 1 to 5, 1 being the worst and 5 being the best score. This rating is performed by making a rating scale based on the input data. The scales are made to fit in the obtained input data to ensure that the floors will not all obtain an equal rating. Important to mention is that the rating scale is subjective and might be different if this MCA would be repeated by anyone else.

Some aspects which are deemed important for a demountable floor system cannot be used within the scope of the functional unit, for instance if the FU states a specific length of a floor slab the floor system can't be rated on the maximum possible span because this is already determined by the FU. The choice is made to include a bonus point system for these criteria. Again, a rating scale is established and depending on the score of the floor system, a bonus point is awarded.

Now, the criteria are given a weight. The weight reflects the importance of the criterion, a high weight means that the criterion is deemed very important. The weights of the criteria are multiplied by the ratings of the floor systems and added to obtain the final score. The floor system with the highest score is the best solution for the given situation.

Lastly the MCA is repeated for different functional units and the weights of the criteria are changed to examine the effect on the outcome.

Limitations and boundary conditions

Several assumptions are made before starting the MCA:

- The Slimline floor can have either a steel-concrete composite top floor or a raised floor system with wood-based boards. Since this research originates form the desire to create a more sustainable construction industry, the used top floor is a raised floor system;
- The Star-Frame floor can have either a steel deck-concrete composite top floor or a wooden top floor. For the same reason mentioned above, it is assumed that the top floor will be made of wood. The wooden top floor is assumed to be $40mm$ thick;
- The insulation material present in the Cofradal floor is rockwool;
- The construction wood used is softwood with strength class C24, Pine;
- The used concrete class is C30/37;
- The diameter of the prestressing strands used in the concrete elements is: $d = 12.5\ mm$.
- A standard reinforcement net is used [66].

Functional unit

The first MCA has the following functional unit (FU1):

' A floor area of 1000 m², having a fire safe time of 60 minutes and designed with floor slabs spanning 10.8 m dimensioned to withstand a load consisting of the self-weight of the slab, ceiling, top floor and services, and a variable loading comprising of a loading due to the use function and separation walls, simply supported on steel IPE profiles spanning 5.4 m dimensioned to withstand the loading of the slab.'

The second (FU2) and third (FU3) time the analysis is done the only difference is found in the span of the slab, 9.0 m and 7.8 m respectively.

A visual representation of the functional units is given in Figure 3.1. The rectangles with the green top and grey sides are the different floor slabs and the red profiles are the IPE beams. The green area is the total floor area, L_s is the varying length of the slab and L_b is the length of the beams.

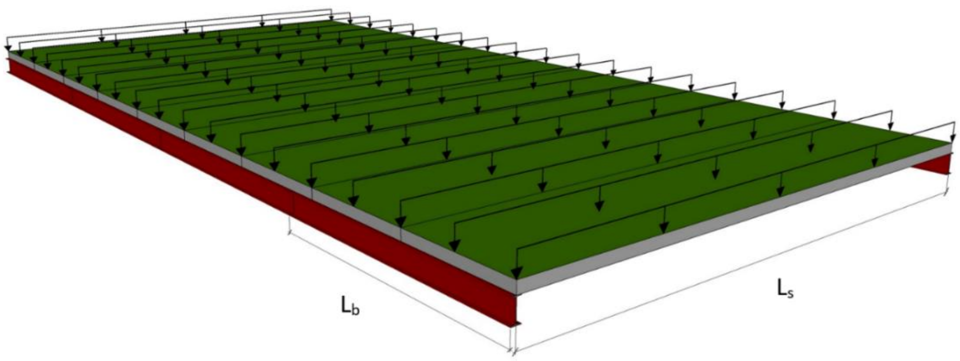

Figure 3.1: Functional unit

The influence of the different floor slabs is only determined for the used beams, the columns and foundation are left out of the analysis. The change in floor slab will influence the load on the columns and foundation, but only slightly. The difference between a timber HCS-box floor and a steel-concrete composite floor is regarded. The weights are $W_T = 0.7\ kN/m^2$ and $W_{SC} = 3.4\ kN/m^2$, the composite floor is about 5 times as heavy as the timber floor. Now the conclusion might be drawn that the influence is big but the opposite holds true. When regarding the difference in design loading in ULS, it can be seen that the load due to the composite floor is only about 1.5 times as high:

$$q_{d,T} = 1.2 * (0.7 + 0.5) + 1.5 * 3.5 = 6.7\ kN/m^2$$
$$q_{d,SC} = 1.2 * (3.4 + 0.5) + 1.5 * 3.5 = 9.9\ kN/m^2$$

Regarding the remainder of the building structure is obviously important but because the change in used columns and foundation when using different floor slabs is assumed small, they are left out of the MCA.

In the following paragraphs, no criterion is presented regarding durability or costs. Durability is disregarded since the analysed floor systems are designed to have a satisfactory level of durability. The costs are left out of the MCA because they are difficult to estimate and because they are deemed less important for research purposes only. In all likelihood the costs will be higher than for a regular floor system since extra measures have to be taken to make the floor demountable. The extra costs and the reduction in costs (due to reuse) per floor system are very hard to determine.

3.2 Criteria

Environmental impact

Since the concept of demountable buildings originates from the aim to create a more sustainable construction industry, the environmental impact of the floor systems is of importance. The environmental impact is calculated with the use of the environmental cost indicator (ECI), which is explained in paragraph 2.3.

Transportation

In this criterion the amount of trucks required to transport the elements to construct the stated floor area is determined. The number of elements which fit in the area of the truck and the number which can be stacked on top of each other are determined, resulting in the number of trucks.

Connection possibility

Because the aim of this research is to design a demountable floor system, the effort required to design a dry connection is determined. Assessing something which does not yet exist is quite difficult, so a rough rating scale is established. The way in which the floor system is connected is completely independent of the functional unit and is therefore the same in any given situation, therefor it is included in the regular part of the MCA.

Lightweight

A lightweight floor system has several advantages: the material use for the rest of the structure can be decreased, the erection is safer and easier, and transportation results in less harmful emissions. Therefore, the weight of the systems is regarded in the analysis.

Building decree

The building decree imposes several requirements on storey floors which must be met. Here, three sub-criteria are regarded; sound insulating properties, vibration performance and fire resistance of the floor systems.

Flexibility

With this criterion, the ease of changing the user function of the building is regarded. The maximum free span length of the floor slab is important. Because the functional unit states a specific length of the floor slab, this criterion will be included in the bonus-point section. The ease of changing services is also important for the ease of changing the user function but since the services can be placed in such a way that they are easy to change for all systems, this is left out of the analysis.

Ease of (dis)assembly

Floor systems are made up of several different elements, like floor slabs, floor beams and fasteners. If the system consists of many parts, it takes a lot of time to (dis)assemble the system and mistakes are more likely to occur. Therefore, the number of required elements is important if the system has to be (dis)assembled many times. Because the span is fixed in the functional unit, the used number of elements can’t be optimized per floor system. Some can result in less elements if a different span is used, therefore, this criterion is included in the bonus-point section.

3.3 Results and conclusions

In this chapter, the influence of using a different functional unit and the influence of the separate criteria are determined. At the end some conclusions are given.

An investigation is conducted into the different ways in which the bonus points can be used. Either they are given a fixed value, or they can vary along with the other criteria. Two tests are done and the change in outcome of the MCA for functional unit 3 is regarded. The same test is carried out for functional unit 1 and 2. These results can be found in paragraph 9.1.5. In the tests the distribution of the weights is changed, while keeping the sum of the weights equal to 180.

For the first test, all the criteria are given an equal weight (1), the bonus points are given a weight of 10 and the rest is given an equal weight (2) and the bonus points are given a weight of 30 and the rest is given an equal weight (3).

In the second test, the criteria connection possibility, flexibility and ease of (dis)assembly are given the highest weight, the criteria ECI, transportation and lightweight are given a mediocre weight and the building decree criteria are given the lowest weight (4), the bonus points are given a weight of 10 and the rest of the weights are distributed in a manner equal to the former (5) and the bonus points are given a weight of 30 and the rest of the weights are distributed in a manner equal to the former (6).

In Figure 3.2 and Figure 3.3 the results of the tests can be found.

The first and second choices are summarised in Table 3-1. It is clear that for test 1 the outcome is completely the same and for test 2 only one choice differs.

	First choice	**Second choice**
(1)	HCS-box/ HCS-surface	HCS/ Slimline
(2)	HCS-box/ HCS-surface	HCS/ Slimline
(3)	HCS-box/ HCS-surface	HCS/ Slimline
(4)	HCS-box/ HCS-surface	Quantum
(5)	HCS-box/ HCS-surface	Quantum/ Slimline
(6)	HCS-box/ HCS-surface	Quantum

Table 3-1: Outcome change in usage bonus-points FU3

After regarding the outcome of the tests for all the functional units, the conclusion is drawn that the manner in which the bonus points are implemented doesn’t change the outcome of the MCA significantly. For this reason, the choice is made to vary the bonus points together with the rest of the criteria to keep the approach uniform.

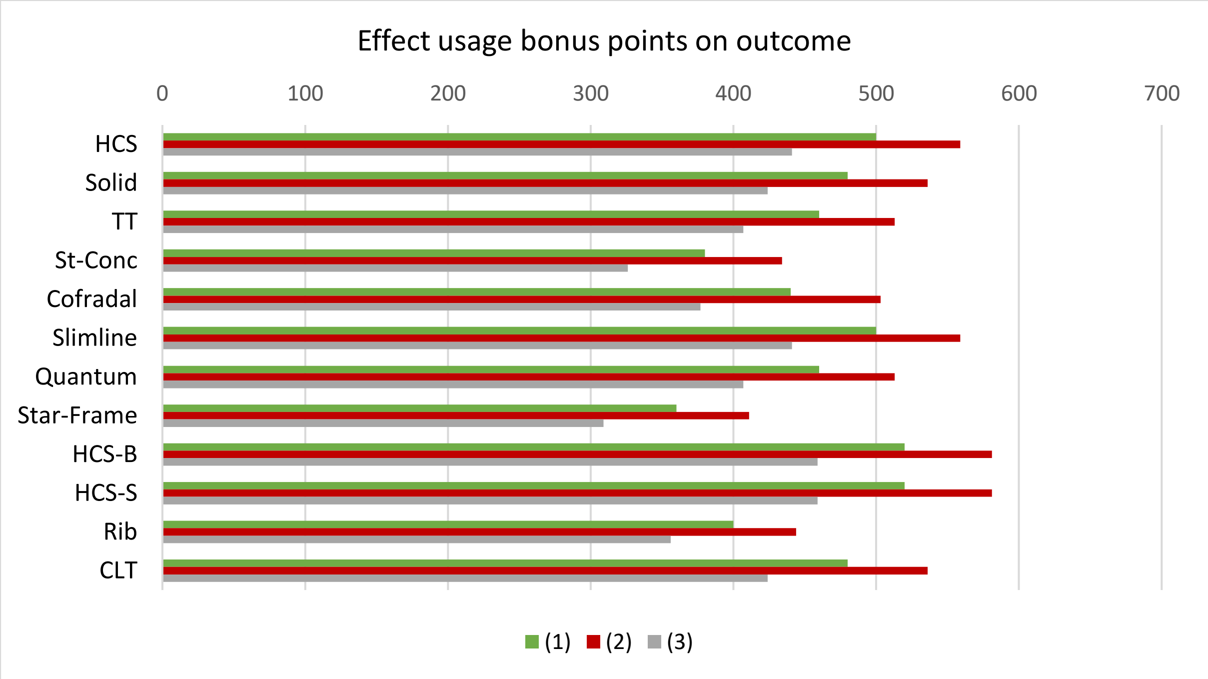

Figure 3.2: Effect of usage bonus points (1), (2) and (3), functional unit 3

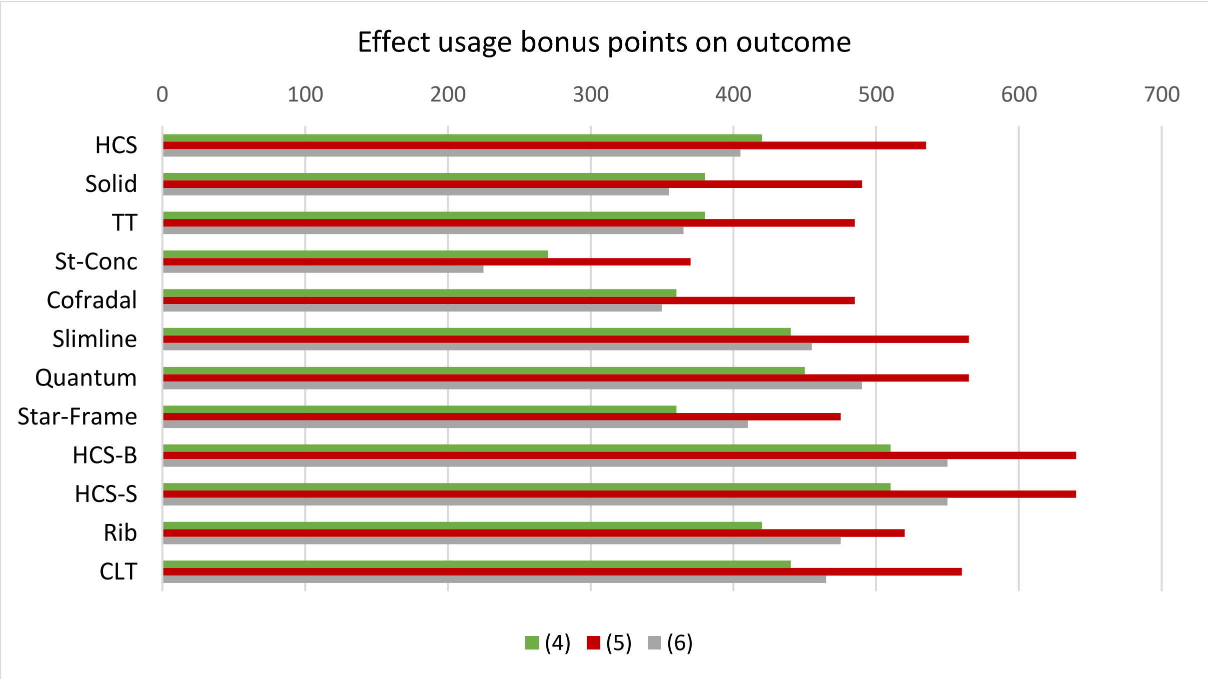

Figure 3.3: Effect of usage bonus points (4), (5) and (6), functional unit 3

Now the influence of the criteria is determined. Four weight divisions are used:

- Option 1: all weights are equal;
- Option 2: the criteria flexibility, connection possibility and ease of (dis)assembly are given an equally high weight and the rest is given a weight of zero;
- Option 3: the building decree criteria are given an equally high weight and the rest is given a weight of zero;
- Option 4: the criteria flexibility, connection possibility and ease of (dis)assembly are given the highest weight, the criteria ECI, transportation and lightweight are given a mediocre weight and the building decree criteria are given the lowest weight.

In the three tables below the floors with the highest three scores are given per functional unit and per option. Quite some floors have an equal final score, therefore the first, second and third choice can be the same floors and more than three floors can be given per option.

Functional unit 1	**First**	**Second**	**Third**
Option 1	CLT/ HCS	CLT/ HCS	Solid/ TT/ Slimline
Option 2	CLT/ Rib/ Slimline/ Quantum	CLT/ Rib/ Slimline/ Quantum	CLT/ Rib/ Slimline/ Quantum
Option 3	Solid	HCS	TT/ Slimline
Option 4	CLT	Rib/ Slimline/ Quantum	Rib/ Slimline/ Quantum

Figure 3.4: Highest scoring floors, functional unit 1

Functional unit 2	**First**	**Second**	**Third**
Option 1	HCS-surface	HCS-box/ Slimline	HCS-box/ Slimline
Option 2	CLT/ Rib/ HCS-surface/ Slimline/ Quantum	CLT/ Rib/ HCS-surface/ Slimline/ Quantum	CLT/ Rib/ HCS-surface/ Slimline/ Quantum
Option 3	Solid	Slimline	HCS/ TT
Option 4	HCS-surface	HCS-box	CLT/ Rib/ Slimline

Figure 3.5: Highest scoring floors, functional unit 2

Functional unit 3	**First**	**Second**	**Third**
Option 1	HCS-box/ HCS-surface	HCS-box/ HCS-surface	Slimline/ HCS
Option 2	CLT/ Rib/ HCS-box/ HCS-surface/ Slimline/ Quantum	CLT/ Rib/ HCS-box/ HCS-surface/ Slimline/ Quantum	CLT/ Rib/ HCS-box/ HCS-surface/ Slimline/ Quantum
Option 3	Solid	HCS/ TT/ Slimline/ Steel-concrete	HCS/ TT/ Slimline/ Steel-concrete
Option 4	HCS-box/ HCS-surface	HCS-box/ HCS-surface	Quantum

Figure 3.6: Highest scoring floors, functional unit 3

The highest scoring floors are mainly the timber floor systems. For the functional unit with the longest span, the choice is CLT and for the smaller spans the best options are the HCS-box and HCS-surface floors. Important to keep in mind is that for FU1 two composite, two timber and the steel floor systems and for FU2 the steel and two composite floor systems are unable to reach the required length and thus don't contribute to the MCA. This can give a tainted view when comparing the outcomes of different functional units. It could be the reason for the CLT floor scoring highest for FU1.

Option 1 results mostly in timber systems, followed by the concrete and composite types.

Weight option 2 consists of the criteria which give a regular floor system the added value of possibly becoming a demountable system. The results are timber and composite systems for all functional units. Here it is important to note that the bonus criteria result in a much lower score since they are given a rating of 0 or 1, instead of 1 through 5. The connection possibility mainly determines this outcome.

For option 3, the choices are concrete and composite systems. This outcome is logical since floors consisting of mainly concrete behave well when regarding the building decree criteria.

Weight option 4 reflects the division of weights deemed most suitable for finding the best choice of demountable floor system. Here the best choices for all functional units are mostly timber and a few composite systems.

The choices for FU1, in which the longest span of the floor slab is used, are less timber and more concrete floor systems. This would probably also be observed if even larger spans are regarded since concrete systems can span big distances with high loads.

The timber floors only score consistently low for the building decree criteria. If these would be higher, they might be the best choice for all the options. Since the concrete systems score highest for the building decree criteria, the addition of concrete to timber might solve the only disadvantage of the timber floor systems.

The results of the MCA are completely independent from the costs of the floor systems. For research purposes this is not a problem but when designing a demountable floor system for practice it should be designed to be economically viable. This means that the reduction in costs due to reuse, so the absent costs of making a new floor system every time one is required, should equate the extra costs required to make the floor system demountable, mainly eliminating wet connections. The best choice of floor is now a timber floor system, but when costs would be an important aspect the outcome could possibly lean toward the popular HCS or steel-concrete composite floors. The advantage of timber floor systems is that they are already made with dry connections so the additional costs will be minor. Floors made of (mainly) concrete are commonly made with wet connections, so a big increase in costs will occur when dry connections are to be designed. There is no telling which floor system will result in the best solution with the lowest costs. Therefore, the choice is made to neglect the costs and move forward with the outcome of the MCA as is.

4 Timber-Concrete Composite floor systems

The multi-criteria analysis performed in chapter 3 begs the question whether a floor slab made of timber and concrete would be an optimal solution for a demountable floor system. Therefore, in this chapter an investigation is done into timber-concrete composite (TCC) floor systems.

4.1 Introduction

A timber-concrete composite floor system consists of timber beams or plates and a concrete slab, connected to each other by shear connectors. See Figure 4.1 and Figure 4.2 for a plate- and rib floor respectively. Due to the shear connection, the two materials work together as a composite. The concrete is mainly loaded in compression and the timber is mainly loaded in tension and bending.

Figure 4.1: Timber-Concrete Composite plate floor [67]

Figure 4.2: Timber-Concrete Composite rib floor [67]

The first application of timber-concrete composites was the retrofitting of existing timber floor systems to increase the load carrying capacity and sound insulation. This was done by installing shear connectors in the existing timber floor, placing temporary supports and reinforcement, and casting concrete in-situ. The in-situ casting of concrete is mostly used as of yet, also in new constructions, but (semi-)prefabricated systems are also developed.

An example of a semi-prefabricated TCC floors is a M-panel [68], see Figure 4.3. When the panels are placed side by side, the outer timber joists are connected to each other with fully threaded screws. Now the shear connectors, reinforcement mesh and propping are installed and the concrete is poured on top. It is also possible to prefabricate the timber element and the concrete slab. A steel shear connector is casted into the concrete in the factory, see Figure 4.4 and Figure 4.5. On site the concrete slab is placed on the already installed timber elements. Nails or screws are used to connect the concrete plate through the casted-in fasteners to the timber. The screws can be pre-tensioned [69]. Prestressing the fastener results in a high contact force between the timber and concrete resulting in a more rigid and higher strength connection [70]. An advantage of a precast concrete slab is that the concrete shrinkage occurs before it is connected to the timber, so it doesn't exert any strain on the element.

Figure 4.3: Prefabricated LVL floor unit [1]

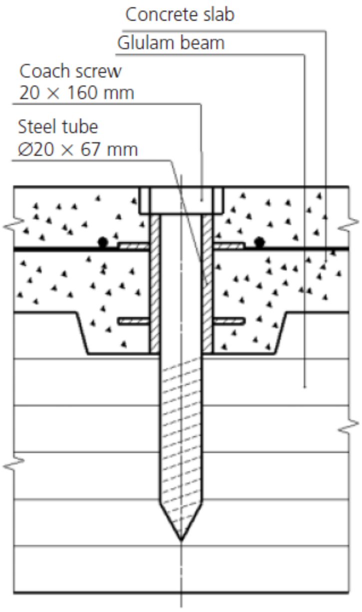

Figure 4.4: ST + S + N connector, adapted from [69]

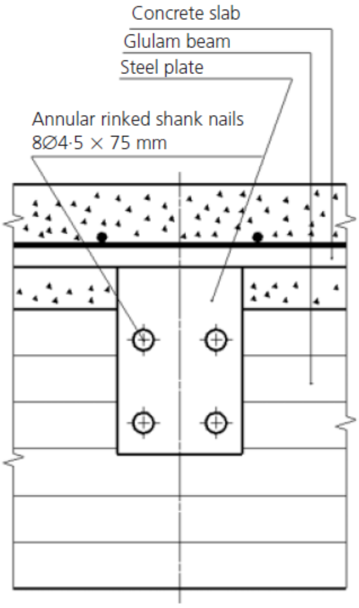

Figure 4.5: SP + N connector, adapted from [69]*

Fully prefabricated TCC floors have been developed as well, see Figure 4.6 and Figure 4.7. These prefabricated elements are connected on-site with wet concrete joints.

Figure 4.6: Prefabricated TCC floor slab [71]

Figure 4.7: Instalment prefabricated TCC floor slab [72]

The advantages of a TCC floor compared to a timber floor are:

- A higher load bearing capacity for an equally thin floor;
- An increased bending stiffness which results in reduced deflections;
- Higher damping ($\zeta_{tcc} = 2.5\%, \zeta_t = 1\%$) which results in smaller vibrations [73];
- Increased contact-noise insulation due to the better damping behaviour [73];
- Increased air-noise insulation due to a higher mass [73];
- Better fire resistance because of the protection of timber by concrete.

The advantages of a TCC floor compared to a concrete floor are:

- Smaller self-weight;
- Lower environmental cost.

4.2 Material properties

An important phenomenon of TCC's which should be regarded during the design is the difference in time-dependent behaviour of the materials: creep behaviour of timber and concrete, drying shrinkage in concrete, moisture related shrinkage and swelling of timber and temperature induced shrinkage and swelling of timber and concrete.

Several different types of wood (products) can be used; solid wood, glue laminated timber (glulam), cross laminated timber (CLT) and laminated veneer lumber (LVL). The advantages of using wood engineered products are the homogenised mechanical properties and the possibility of long span beams or plates.

The concrete that is used can be either normal concrete or a high performance concrete [74]. High performance concretes are for instance fibre reinforced-, high strength- or lightweight concrete. Fibre reinforced concrete (FRC) is concrete in which the regular bar reinforcement is replaced with fibres. The fibres can be made of steel, plastics or natural materials. The compressive strength and modulus of elasticity of FRC's are almost equal to regular reinforced concrete [75]. Some advantages are [75]: the absence of brittle failure due to the continuing load transfer through crack plains after initial cracking, a decreased thickness due to the lack of a required concrete cover over the rebars and a more uniform stress distribution.

High-strength concrete (HSC)[76] is concrete with a higher strength than regular concrete, but this is not always the advantage for which it is used. The higher strength results in an increased static modulus of elasticity, a decreased permeability and a decreased required thickness.

Lightweight concrete (LWC) is made with lightweight aggregates due to which its self-weight is smaller than that of regular concrete. Lightweight floor slabs have obvious advantages as mentioned in paragraph 2.1.1. LWC has an equal strength for a lower density. Some disadvantages [77, 78] are: a reduced modulus of elasticity and brittle behaviour (due to the stronger cement matrix than the aggregates). The results of tests on shrinkage and creep behaviour are contradictory. [78] reports an increase in shrinkage and creep while [79] reports no change in the shrinkage and creep behaviour.

4.3 Shear connectors

The mechanical behaviour of TCC's is strongly dependent on the strength and stiffness of the shear connection, therefore the choice of connector is critical. Many types of shear connectors are available and extensive research has been done into the differences between connectors [80-82].

Figure 4.8 shows several types of connectors between timber and concrete. The dowel-type fasteners in group A result in the connection with the lowest stiffness. Dowel-type fasteners are dowels, bolts, screws, nails and rods. Connections made with surface connectors, group B, have a higher stiffness than connections made with dowel-type fasteners. Furthermore, their ultimate strength and ductility is increased. Group C are connections made with notches, their stiffness and strength are a bit higher than group B. Connections with glued-in fasteners, group D, behave almost fully rigid.

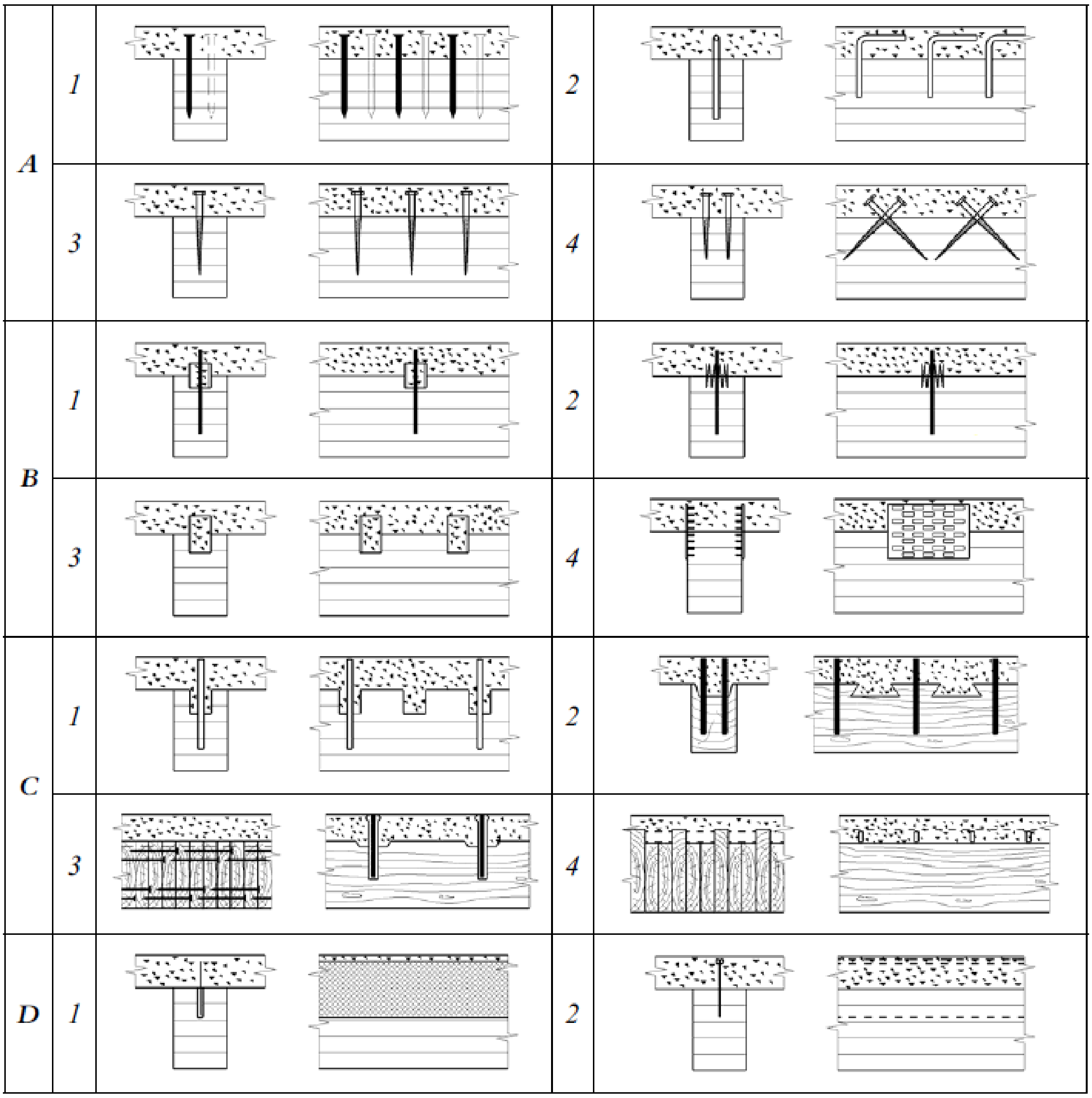

Figure 4.8: Examples of timber-concrete connections. (A1) Nails, (A2) Dowels, (A3) Screws, (A4) Inclined screws, (B1) Split ring, (B2) Toothed-plate, (B3) Steel tubes, (B4) Punched metal plate, (C1) Pre-bored notches with fasteners, (C2) Trapezoidal notch with fastener, (C3) cup indentations and prestressed steel bars, (C4) Nailed timber plank deck with steel shear plates slotted through the deeper planks, (D1) Glued-in steel lattice girder, (D2) Glued-in steel plate. [70]

4.4 Conclusion

A timber-concrete composite floor is a good option for the design of a demountable floor system due to the numerous advantages compared to full timber or full concrete floors. The systems can be completely prefabricated.

5 Design and verification floor slab

In this chapter a timber-concrete composite floor slab will be designed and verified. To be able to make a first design, the case study of an office building is used. The chosen floor slab will be implemented in this building and the structural integrity will be verified for the corresponding loads. First the building in which the floor system will be designed is introduced. Subsequently, the used design method will be described. Hereafter, the general verification method for the floor slab is explained. A worked-out design example can be found in Appendix B. Finally, a summary of the design of nine floor slabs is given and the slab which will be used in the floor system is chosen.

To avoid a text riddled with references, the used references are stated at the start of the calculation example in Appendix B.

5.1 Case study building

In this paragraph, the original structural building design of the case study building will be described. Hereafter, the changes that will be made to the structure will be explained.

The building in which the reusable floor system will be designed is an office building designed by CEPEZED architects. The building, named Bouwdeel D, has a transparent structure with an open floor plan. In Figure 5.1 a 3D view is given of the structural system. The building consists of a ground floor, three storey floors and a roof floor.

Figure 5.1: 3D structural view Bouwdeel D (IMd Raadgevende Ingenieurs)

Figure 5.2 shows the structural plan of the first floor of the building, which is equal to the plan of the second and third floor. The building's dimensions are $L_B = 21.2\ m$ and $W_B = 10.9\ m$ resulting in a floor area per storey of about $A_{floor} = 231\ m$. The storey heights, measured from top floor to top floor, are: $H_{s1} = 3.025\ m$, $H_{s2} = H_{s3} = 3.145\ m$ and $H_{s4} = 3.205\ m$. This gives a total building height of $H_B = 12.5\ m$ measured from the top of the ground floor.

The used columns are rectangular steel hollow core sections 160x80x6.3. The floor slabs used in the current design are timber Kerto Ripa rib floors. The floors are continuous slabs supported on edge beams in the facades at axes 1 and 8 and on an intermediate support at axis 5.

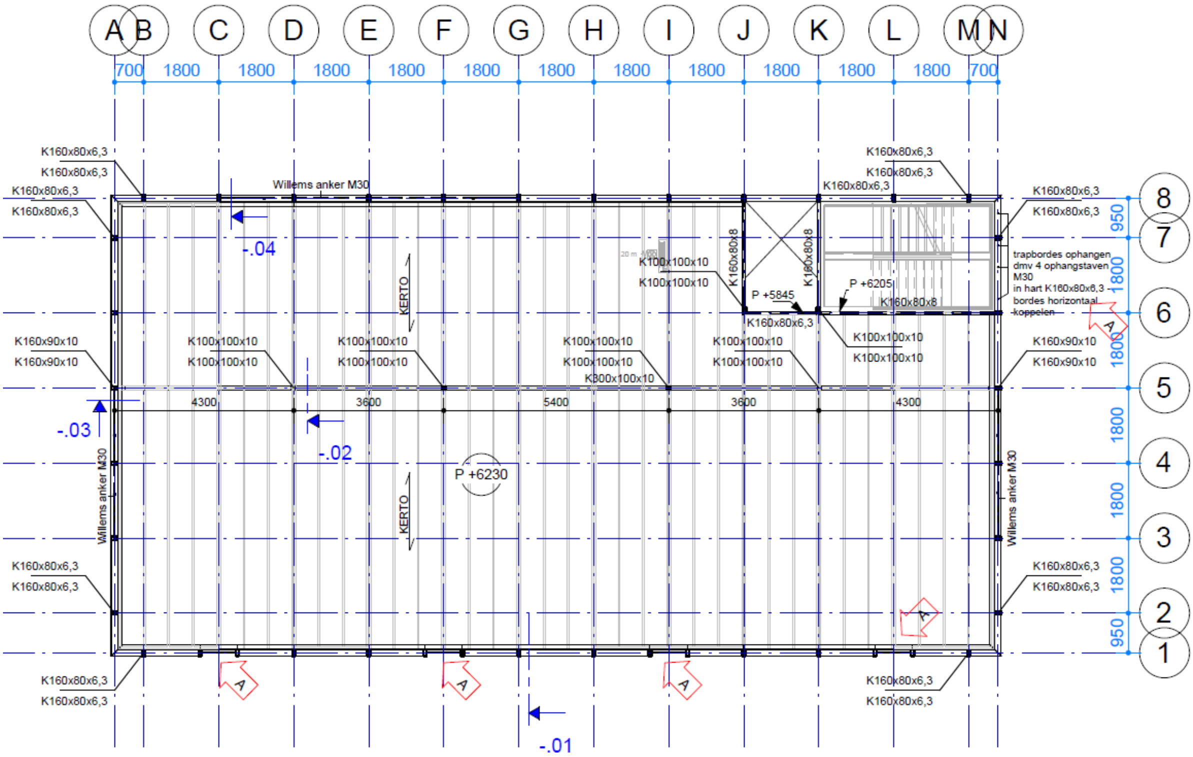

Figure 5.2: Plan first floor Bouwdeel D (IMd Raadgevende Ingenieurs)

Detail 1 is shown in Figure 5.3, here the steel double L-shaped edge beam can be seen. The vertical bottom L-section acts as a support for the floor slabs. The horizontal top L-section serves as a support for the top floor that will be implemented. The façade will be connected to the outer side of the steel columns, indicated in a simplified manner with the green line in Figure 5.3 through Figure 5.5. The horizontal L-section and the top floor assure that the floor levels are separated.

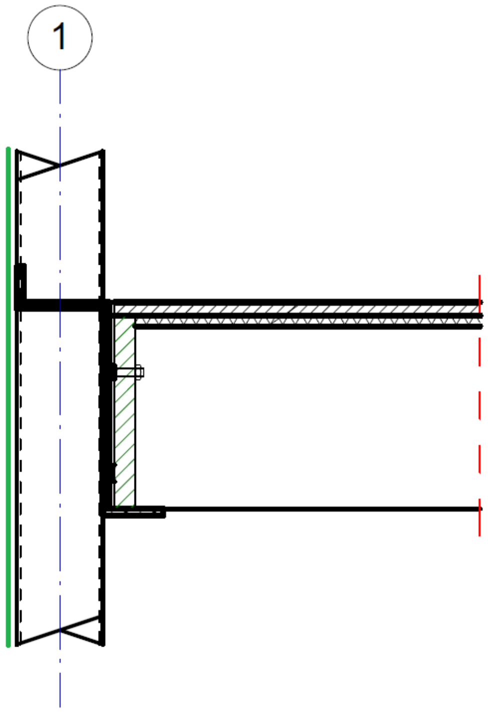

Figure 5.3: Detail 1, 1:10 (Adapted from: IMd Raadgevende Ingenieurs)

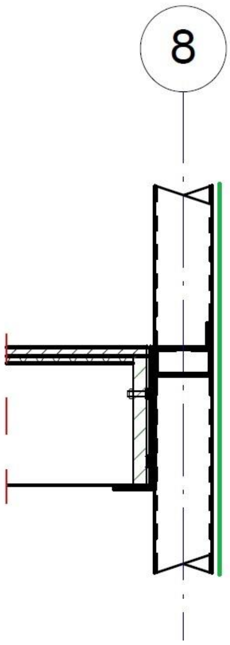

Figure 5.4: Detail 4, 1:20 (Adapted from: IMd Raadgevende Ingenieurs)

The wind bracings are 'Willems ankers', applied in three of the four facades. At the positions of the wind bracings, rectangular hollow core sections 160x80x6.3 are used as horizontal bracings at the positions where the wind bracings are applied. This is shown in Figure 5.4.

In Figure 5.5 detail 3 is shown. Here, the connection between the side of the floor slab and the edge beam is illustrated. The floor slab is connected to the L-section edge beam and supported on the intermediate support.

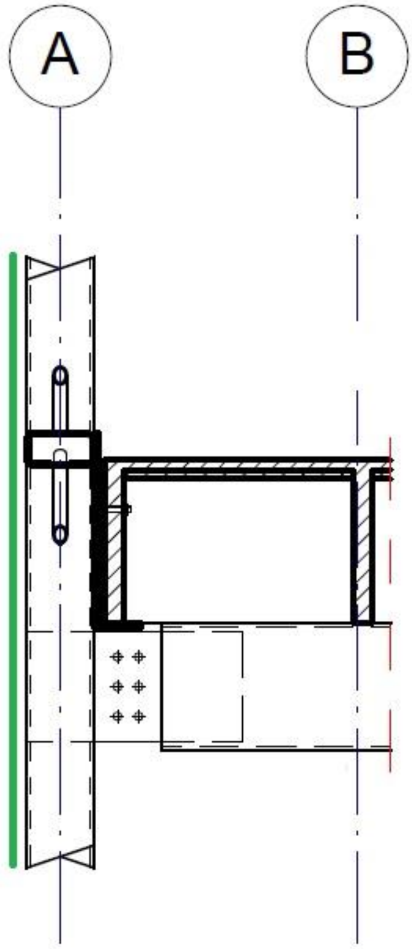

Figure 5.5: Detail 3, 1:20 (Adapted from: IMd Raadgevende Ingenieurs)

Changes building structure

One of the requirements for a reusable floor system is the possibility of a freely divisible floor plan. This can be achieved by reducing the number of walls and columns in the floor surface. Therefore, the column row used as the intermediate support for the floor slabs in the original design will not be implemented in the new design. This results in a required slab length of $10.8\ m$.

Depending on the connection that will be designed in chapter 6, the used edge beam might change as well.

5.2 Design method

The most common TCC floor slabs are the rib floor, beam floor and plate floor. Here the choice is made to design a rib floor, which is in design procedure equal to a beam floor. The reasoning for this decision mostly originates from the outcome of the multi-criteria analysis. Figure 5.6 shows an artist impression of a rib floor.

The rib floor and CLT floor are the only timber floors able to reach a span of 10.8 meters. The rib floor is not the best choice due to its low score for the building-decree criteria and due to the required amount of trucks because of the big floor height. The CLT floor already scores high on these criteria so the addition of concrete would possibly not be a (big) advantage. For the rib floor, the building-decree requirements will be complied with much easier and the height will be reduced when concrete is added.

It is possible to place services and/or insulation between the ribs and form a type of box-section by adding wood based (possibly fire-resistant) boards on the bottom of the slab. Using this as a prefabricated element will further reduce the handlings needed on site and thus reduce the erection time and costs.

Figure 5.6: Rib floor, artist impression

At the present, no design guidance is available in the Netherlands for timber-concrete composite structural elements. Existing design methods, design guidance from the Eurocodes and information found in research papers are combined to verify TCC floor slabs.

A limit state design is done to verify the floor slab. The ultimate limit state (ULS) and serviceability limit state (SLS) are checked in the short and long term. The short term is the first time instance the element is loaded, t_0, and the long term is at the end of the service life, t_∞. The elements are verified at these two time instances because concrete and timber respond in different ways to long term actions, like creep. The different behaviour under these actions results in a change in stress distribution over the cross-section over time.

The verifications done in ULS are for the maximum normal and/or shear stresses in the concrete and the timber, bending and shear failure of the concrete and the maximum shear force, and possibly tensile force, in the connection. In SLS checks are done for deflections, vibrations and the cracking of concrete.

The gamma method for built-up timber beams is used. This method can be adopted for timber-concrete elements if some modifications are made. In Figure 5.7 a built-up beam is shown, including the stress distribution. In most of the TCC designs made here, the timber beams are built-up of two different glulam types: a low strength type at the top and a high strength type at the bottom. The stress distribution will be different than the one depicted below if two types of timer are used. The properties and dimensions of timber at the bottom will have subscript '3'.

The symbols in the figure are explained below:

- A_1, A_2 are the area of the concrete and timber respectively;
- E_1, E_2 are the moduli of elasticity of the concrete and timber respectively;
- I_1, I_2 are the moment of inertia of the concrete and timber part respectively;
- b_1, b_2 are the width of the concrete and timber respectively;
- h_1, h_2 are the height of the concrete and timber respectively;
- a_1, a_2 are the distances between the neutral axis of the combined element and the neutral axis of the concrete or timber respectively;
- σ_1, σ_2 are the normal stresses in the concrete and timber respectively;
- $\sigma_{m,1}, \sigma_{m,2}$ are the bending stresses the concrete and timber respectively;
- τ_{max} is the maximum shear stress in the element.

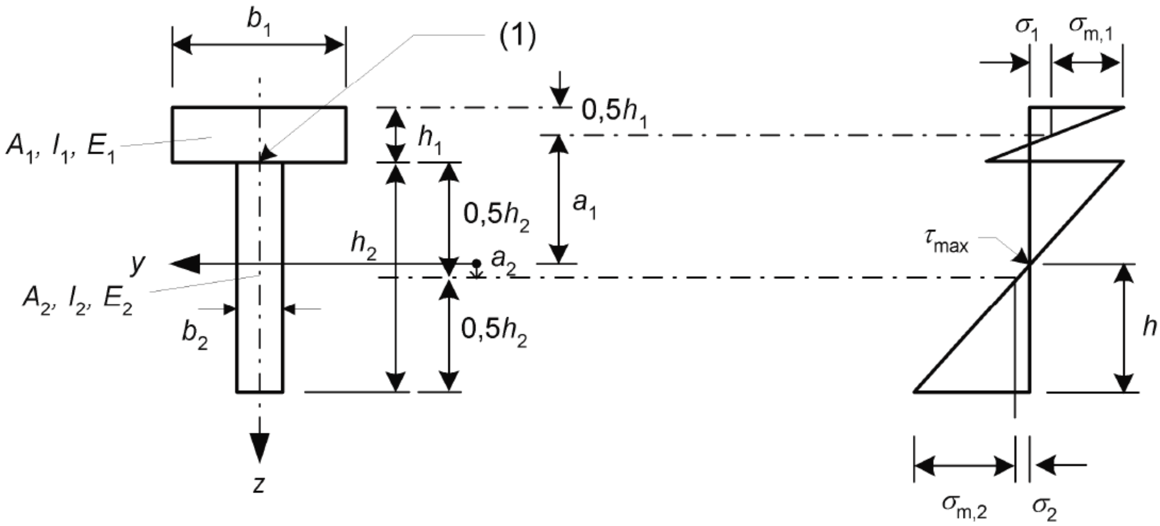

Figure 5.7: Cross-section and stress distribution built-up beam, gamma method [83]

In the gamma method, the connection efficiency coefficient, also called the composite factor, γ, is used to take into account shear deformations in the connection between the members. It depicts the connection as either rigid, $\gamma = 1$, not connected, $\gamma = 0$, or something in between.

The verification for the entire slab in the longitudinal (span) direction will be done for one T-shaped element with a width of $b_{ef,c}$, indicated in the figure above as b_1. In the transverse direction the entire cross section of the TCC slab is regarded.

5.2.1 Limitations and assumptions

In the floor area of every floor, slabs with deviating dimensions (fitting plates) or recesses for vertical transport of services and people are present. These deviating floor slabs could result in a different loading on the slabs, connections and edge beams. Here the floor slabs are designed as if the total floor area in the case study building consists of slabs without recesses.

The applied variable loading of the use category for which the floor system is verified is $2.5\ kN/m^2$.

The required fire protection, and sound- and thermal insulation are not regarded in the design. Only a structural design will be made of the floor slab.

Some assumptions that are made in the floor slab design are:

- The used concrete is regular concrete. Investigations into other types of concrete are too elaborate for this report.
- The type of timber used is either solid wood or glulam. LVL is left out since this material has a very high environmental impact.
- The total width of the designed floor slabs will be $W = 2400\ mm$.
- A slab with $\frac{L}{W} > 2$ can be simplified as a beam on two supports.
- The ECI values for materials used in the MCA are used here to determine the ECI of the designed floor slabs. Laminated tropical hardwood can't be found in the environmental database. To get an estimation for the ECI of tropical hardwood, the difference between solid and laminated pine is reviewed. The ECI for laminated pine is about 6x higher than for pine planks. This ratio is adopted for laminated hardwood.

5.2.2 Loading

Several loads are acting on the floor element, Table 5-1 gives a summary. The permanent load consists of structural and non-structural permanent loads: the self-weight of the slab, the top floor, the ceiling and the services. The variable load consists of the self-weight of the separation walls and the imposed load from the use category in the building. Use category B (office areas) is used. Shrinkage of concrete also results in a force acting on the concrete and timber parts and on the shear connection.

Loads might arise due to shrinkage and swelling of the timber beams when a change in moisture content occurs, concrete doesn't exhibit these changes. When temperature changes occur forces might arise due to the different thermal expansion coefficients of timber and concrete. Because the floor slab is used indoors, the effect of change in moisture content and temperature is assumed small and is disregarded.

Loading action	**Permanent [kN/m²]**	**Variable [kN/m²]**
Self-weight slab	Various	
Top floor and ceiling	0.25	
Services	0.25	
Imposed load		2.5
Separation walls		1
Concrete shrinkage	Various	

Table 5-1: Loads on floor slab

5.2.3 Concrete shrinkage

The shrinkage of concrete results in a strain, and thus a stress, acting in the concrete element. Because the concrete and the timber are connected with shear connectors, a force will also occur in the timber beams and on the shear connectors. The shrinkage stresses are calculated using the theory for two rigidly connected concrete elements having different properties of which one element is subjected to drying shrinkage [84]. The shear connection between the concrete and the timber is also assumed rigid.

First the shrinkage strain acting in the concrete is calculated. Shrinkage strain is a summation of drying shrinkage strain and autogenous shrinkage strain. Because the strain is subjected to relaxation, it may be lowered by multiplying it with a factor.

Secondly the section properties of all the layers and the properties of the total element are determined; the axial stiffness per layer, the axial stiffness of the element, the bending stiffness per layer, the bending stiffness of the total element and the location of the neutral axis. With 'layers' the different materials are meant; the concrete deck, the top timber part and the bottom timber part. The used total bending stiffness is calculated with the gamma-method. This choice is made after reviewing the occurring shrinkage stresses determined with a bending stiffness calculated in a regular way for composite elements and with a bending stiffness calculated with the gamma-method. Using the gamma-method results in more unfavourable stresses and is for this reason chosen to make the calculation as safe as possible.

The forces occurring in each layer can now be determined. The concrete slab wants to shrink but is kept in place due to the (assumed rigid) connection to the timber beams. This normal shrinkage force is calculated as

$$N^* = \varepsilon_{cs,\infty}(EA)_c$$

Because the concrete slab is kept in place, the normal shrinkage force results in a tensile force in the slab. The shrinkage force can be moved to the neutral axis which will result in a normal compressive force and a bending moment on the total cross section.

$$M^* = N^* * e$$

An illustration of these steps is given in Figure 5.8.

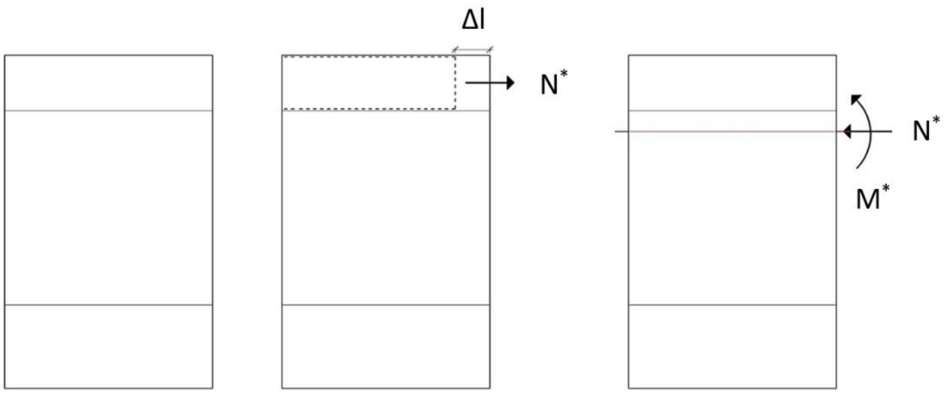

Figure 5.8: Steps concrete shrinkage

Every layer is subjected to a normal force due to the normal shrinkage force, N_{i,N^*}, a normal force due to the total bending moment, N_{i,M^*}, and a local bending moment due to the total bending moment, M_i. They are calculated with the following formulae

$$N_{i,N^*} = \frac{(EA)_i}{(EA)_{tot}} N^*$$

$$N_{i,M^*} = \frac{(EA)_i}{(EI)_{eff,fin}} N^*$$

$$M_i = \frac{(EI)_i}{(EI)_{eff,fin}} M^*$$

Finally the arising stresses can be determined. Due regard has to be taken of the direction of the total bending moment, and thus whether a tensile or a compressive stress occurs in an element. This is dependent on the position of the neutral axis.

After equating all the stresses, the concrete will be loaded in tension and the timber will be loaded in compression.

The stress values are quite small, for all the designed floor slabs with glulam beams the values in tension never exceeded 2.9 N/mm^2 and the values in compression are lower than 2.1 N/mm^2. The resulting unity checks (UC) for the stresses are below 0.7.

The assumption of a fully rigid connection might have a big, positive, influence on the outcome. The arising stresses will be lower when a flexible connection is taken into account because the concrete shrinkage deformation will not be fully restrained. A slip will occur in the connections between the timber and the concrete. So, it is safe to make this assumption.

No extra forces on the connections are taken into account. When designing the shear connections between the concrete and the timber it is aimed to keep the unity checks well below 1.0 to ensure that arising forces due to shrinkage can be restrained.

Because the UC's for the stresses are smaller than 0.7 it is concluded that concrete shrinkage doesn't influence the design of the floor slab significantly. So, the used, conservative, method is sufficient for this preliminary design procedure.

5.2.4 Reinforced concrete

The concrete slab has to be verified for two directions, the longitudinal and transverse direction. When the system in the longitudinal direction is regarded, the stresses in the outer fibres due to the permanent and variable load can be calculated. In the long term the stresses caused by concrete shrinkage have to be considered as well.

If the UC for concrete in tension is not satisfied, concrete is cracked and should be assumed to have no tensile strength and a reduced modulus of elasticity. The cracked cross section is then taken as a non-load bearing layer between the timber and compressive area in the concrete. This results in an increase in stresses in the concrete.

For design in the longitudinal direction an effective width of the concrete is used. The effective width of the concrete slab is calculated using the method for steel-concrete constructions shown in Figure 5.9. In the transverse direction a unit width of 1m is used.

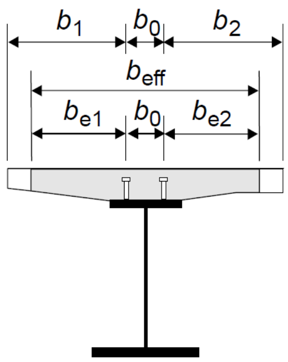

Figure 5.9: Effective width of a concrete flange. Adapted from [85]

Besides the verification for concrete stresses, the concrete reinforcement has to be designed. Rebars will be required in longitudinal direction due to the acting longitudinal bending moment and shear force, and in the transverse direction due to in-plane shear and transverse bending.

The maximum and minimum reinforcement and spacing between the bars must be verified to be between the maximum and minimum required values. These requirements are different for principal and secondary reinforcement. In a regular concrete slab, the principal reinforcement is in the longitudinal direction and the secondary reinforcement is in the transverse direction. These definitions are used because the principal is the biggest amount of reinforcement and the secondary is the least amount. Interesting in the case of a TCC slab is that in the longitudinal direction less reinforcement is required than in the transverse direction, so the definition of principal and secondary reinforcement doesn't hold. Here the assumption is made that the transverse reinforcement is the principal reinforcement since it is much more than the longitudinal reinforcement.

The material and mechanical properties of concrete and reinforcing steel are given in Eurocode 2. The effective modulus of elasticity of concrete is calculated with the following formula:

$$E_{conc,fin} = \frac{E_{conc}(t_0)}{1 + \psi_{conc} * \varphi(\infty, t_0)}$$

In which

$E_{conc}(t_0)$ is the modulus of elasticity of concrete at time t_0;
ψ_{conc} is the modification factor for the effective creep coefficient of concrete which accounts for the influence of the composite action;
$\varphi(\infty, t_0)$ is the final creep factor.

ψ_{conc} is calculated with the following assumptions: $t = \infty$, $k_{def} = 0.6$ and the used γ_1 factor is calculated for ULS initial verifications. The calculation procedure may only be used if the following conditions are met:

- $\frac{b_c}{b_t} \geq 5$;
- $1 < \frac{A_c}{A_t} < 5$.

Assumptions made to determine the final creep coefficient are a relative humidity of 50% and a cement class N.

5.2.5 Timber

In the timber beams, the UC's for the combination of bending- and tensile stresses and the shear stress should be verified. Since the timber beams contribute the most to the height of the cross section, is assumed that the total shear force could be acting in the beams.

The material and mechanical properties of timber are found in Eurocode 5 and NEN-EN14080. The effective modulus of elasticity of timber is calculated with:

$$E_{tim,fin} = E_{tim}\left[\frac{\%q_G}{1 + \psi_{tim} * k_{def,p}} + \frac{\%q_Q}{1 + \psi_{tim} * k_{def,m}}\right]$$

In which

E_{tim} is the mean modulus of elasticity of timber;
$\%q_i$ is the fraction of the total load consisting of either permanent or variable loading;
ψ_{tim} is the modification factor for the effective creep coefficient of timber which accounts for the influence of the composite action.

$$\psi_{tim} = \begin{cases} 1.0 \; for \; t = \infty \\ 0.5 \; for \; t = 3 \; to \; 7 \; years \end{cases}$$

5.2.6 Shear connection

The shear connection between the concrete and timber elements has to be verified for the acting shear force, and in some cases an acting tensile force. The type of loads acting and the manner in which the resistance to these forces is determined depends on the type of connection used. This is explained below for the three types of connections used in the different TCC designs.

The modification factor for the influence of load duration and moisture content for the connections can be calculated with $k'_{mod} = \sqrt{k_{tc}k_{mod}}$. The deformation modification factor for the connections can be calculated with $k'_{def} = 2k_{def}$. The partial safety factor for forces acting in the connection is $\gamma_v = 1.25$.

The effective slip modulus of the connection is calculated using the following equation:

$$K_{i,fin} = K_i \left[\frac{\%q_G}{1 + \psi_{conn} * k'_{def,p}} + \frac{\%q_Q}{1 + \psi_{conn} * k'_{def,m}}\right]$$

In which

K_i is the slip modulus for SLS or ULS;

ψ_{conn} is the modification factor for the effective creep coefficient of the connection which accounts for the influence of the composite action.

$$\psi_{conn} = \begin{cases} 1.0 \quad for\ t = \infty \\ 0.65\ for\ t = 3\ to\ 7\ years \end{cases}$$

The spacing of the connectors can be constant, s. It is also possible to use a non-continuous, more optimized spacing. In this case an effective spacing is used in the design, $s_{ef} = 0.75s_{min} + 0.25s_{max}$, where $s_{max} \leq 4 * s_{min}$.

Three different connections are regarded in the designs:

- Vertical screws in pairs;
- Inclined screws in pairs under an angle of 45°;
- Notches with a vertical screw.

Vertical screws

The calculation method for vertical screws can be found in Eurocode 5. Here also the slip modulus for dowel-type fasteners is presented. The only check which has to be performed is whether the acting shear force is lower than the shear resistance of the screws. The shear resistance is calculated by modifying the Johansen model for the shear resistance of timber-to-timber connections. For a timber-concrete connection, the embedment strength of one of the timber parts is changed to a fictitious embedment strength of the concrete. If the length of the fastener in the concrete part satisfies the following: $h_e \geq 3d$, the embedment strength of concrete can be taken as:

$$f_{h,1,k} = 3f_{c,k}$$

The influence of this assumption is determined to evaluate its validity. The concrete embedment strength is either halved or multiplied with two. The resulting shear resistance of the connection is increased or decreased with less than 5%. Hence the assumption is deemed valid.

Inclined screws

No equations are available to calculate the slip modulus of inclined screws. This modulus is usually determined with tests. Doing tests is too extensive for this research so the slip modulus has to be found in a different manner. To determine a value which can reasonably be used in this calculation procedure, a review of investigations into the slip modulus of inclined screws in timber-concrete joints [81, 86-89] has been done. All the experiments done in these articles have different boundary conditions. Therefore, an estimation has to be made for an appropriate value for the specific situation here.

The screws used in the design of the TCC floor slabs for the demountable floor system have an outer diameter of 12mm, a total length of 160 mm and an embedment length in the timber of 100mm. The experimental results in [87] and [88] are used because their conditions are most similar to the conditions in the designed TCC-slab. Linear dependencies are assumed for the variables which are presumed to influence the slip modulus. These variables are: the outer diameter, the embedment length in the timber and the number of screws.

In [87] the screws used in the tests have an inclination of 45° to the timber member. They have an outer diameter of 6mm, a total length of 100mm and an embedment length in the timber of 50mm. The used timber is glue laminated pine. It has a mean- and characteristic density of $\rho_m = 414\ kg/m^3$ and $\rho_k = 374\ kg/m^3$, therefore most likely GL22h was used. The push-out test set-up is shown in Figure 5.10. It can be seen that the specimen is loaded in double-shear.

Figure 5.11 shows the load-slip curve for one pair of screws obtained from one of the tests. The slip modulus for SLS can be determined by:

$$K_{ser} = \frac{0.4F_m}{u_{0.4}}$$

In which

$0.4F_m$ is 40% of the mean load;
$u_{0.4}$ is the slip at 40% of the mean load.

The SLS slip modulus calculated using the load-slip curve results in:

$$K_s = \frac{3.62}{0.33} = 10.97\ kN/mm$$

The value over 6 samples given in the paper is $K_s = 10.87\ kN/mm$, which is very close to the value calculated for one of the tests. The given values for the slip modulus are for one pair of screws. The assumption is made that one screw would result in halve the slip modulus: $K_s = 5.44\ kN/mm$.

In the situation here, the outer diameter and the embedment length in the timber are twice as big than in the experiment. Again, using a linear relation, the slip modulus for the used connection will result in:

$$K_s = 5.44 * 2 * 2 = 21.7\ kN/mm$$

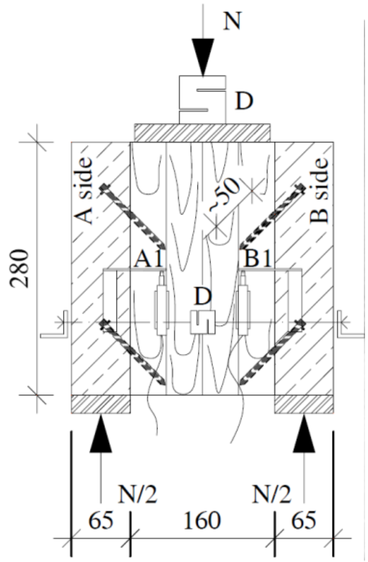

Figure 5.10: Specimen set-up [87]

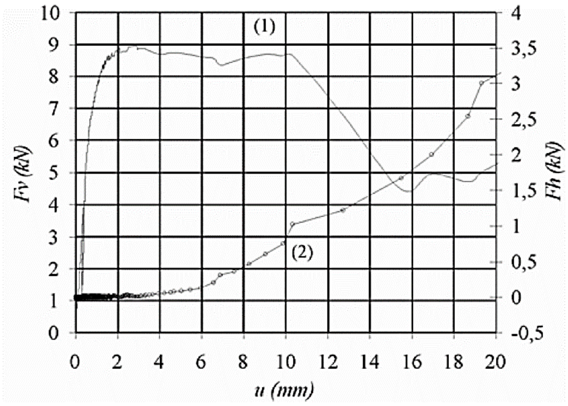

Figure 5.11: Load-slip curve for one pair of screws (1) and horizontal splitting load curve (2) [87]

In [88] the screws used in the tests have an inclination of 0° to 50°, changing in steps of 10°. The outer diameter is 16mm, the total length is 230mm and the embedment length in the timber is 120mm. A permanent formwork was used in these experiments made of a steel decking. The timber beams used are made of GL28h. The push-out test set-up is shown in Figure 5.12. It can be seen that the specimen is loaded in single-shear. Figure 5.13 shows the load-slip curve for one screw for one test and for different angles.

The mean slip modulus for 40° and 50° are respectively $K_{s,40} = 36.85\ kN/mm$ and $K_{s,50} = 44.5\ kN/mm$. Linear interpolation is used to obtain the mean slip modulus for a 45° inclination, resulting in $K_s = 39.18\ kN/mm$ for one screw.

In the situation here, the outer diameter is 3/4th and the embedment length in the timber is 5/6th of that of the tested screw. Using a linear relation between these variables and the slip modulus, for the given situation the value will become:

$$K_s = 39.18 * \frac{3}{4} * \frac{5}{6} = 24.5\ kN/mm$$

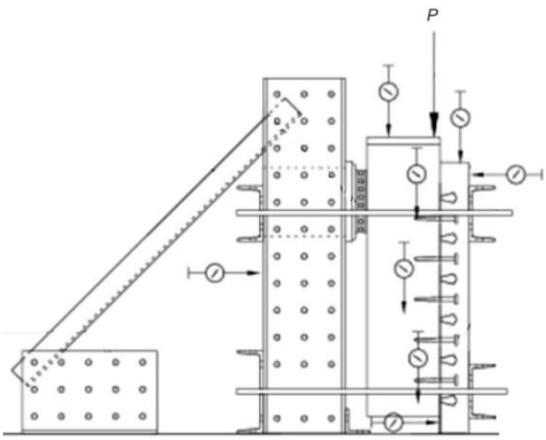

Figure 5.12: Push-out test rig, adapted from [88]

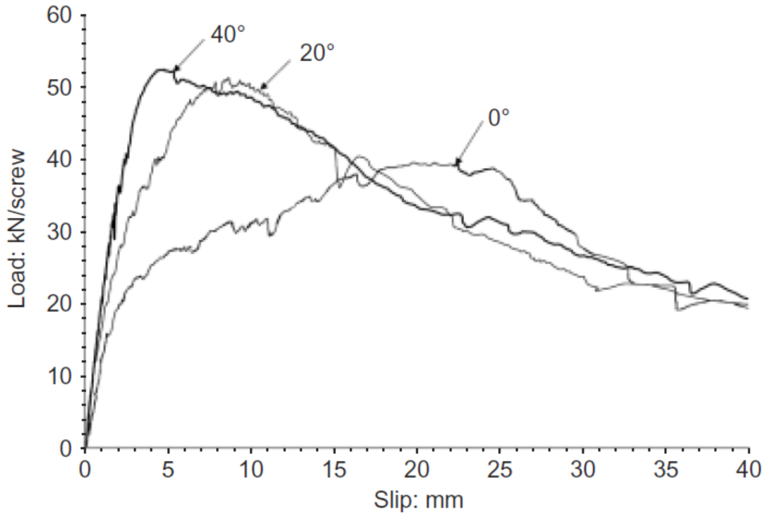

Figure 5.13: Load-slip response for one connector [88]

The second paper results in a higher transformed slip modulus for the design situation in this thesis compared to the one found when using the first paper. This might be a result of the used permanent formwork, the higher timber grade and the test set-up being in single-shear. For these reasons, and to be on the safe side, the slip modulus found by changing the modulus of the first paper is used.

The inclined screws are loaded laterally and axially. Therefore, they have to be verified for a combination of axial and lateral load. This is done with the following formula:

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leq 1$$

In Figure 5.14 the forces acting on the inclined screw can be seen. The above-mentioned equation can be modified to use for inclined screws:

$$\left(\frac{F_{ax,\alpha,Ed}}{F_{ax,\alpha,Rd}}\right)^2 + \left(\frac{F_{|,Ed}}{F_{|,Rd}}\right)^2 \leq 1$$

The acting shear force, $F_{v,Ed}$, can be calculated just like for the vertical screws. The shear force acting in a plane perpendicular to the screw can be calculated with: $F_{|,Ed} = \frac{1}{2}\sqrt{2} * F_{v,Ed}$, which is equal to the axial force acting parallel to the screw, $F_{ax,Ed} = \frac{1}{2}\sqrt{2} * F_{v,Ed}$.

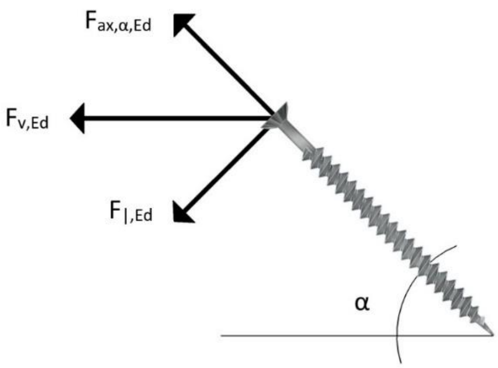

Figure 5.14: Acting forces on inclined screw

The axial resistance is equal to the withdrawal capacity of the screw. It the calculation the angle to the grain direction is already taken into account.

The resistance to shear is determined by calculating the shear resistance of a vertical screw. The differences are that the used embedment strength is determined at an angle of 45° to the grain, the rope effect is calculated with the withdrawal capacity of the screw at an angle of 45° and the used diameter is an effective diameter of

$$d_{ef} = 1.1 * d_1$$

In which

d_1 is the diameter of the screw measured from the root of the thread.

Notch

Figure 5.15 shows a schematisation of a rectangular notched connection. The symbols used are explained below:

- d is the fastener diameter;
- h_n Is the depth of the notch;
- h_c is the thickness of the concrete layer measured from the top of the notch;
- h_t is the thickness of the timber, measured until the top of the notch;
- l_n is the length of the notch;
- l_v is the length of the timber in front of the end notch;
- l_s is the distance between notches;
- α is the angle of the notch;
- F_t is the tensile force acting on the fastener.

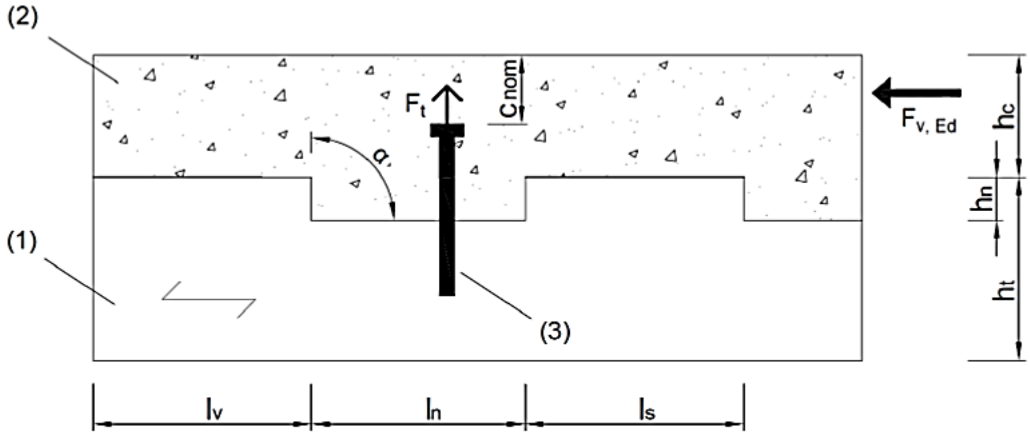

Figure 5.15: Dimensions of a notched connections [90]. (1) Timber, (2) Concrete, (3) Fastener.

In [91] experiments have been done on several connections, including notched connections. In Figure 5.16 the push-out test set-up is shown and in Figure 5.17 the load slip curve of four rectangular notched connections (A1, A2, A3 and B1) can be found. The examined notches have dimensions (l_n x h_n x b):

- A1: 150 x 50 x 63
- A2: 50 x 50 x 63
- A3: 150 x 25 x 63
- B1: 150 x 50 x 63

All the notches indicated with ‘A’ have a coach screw with a diameter of 16mm, notch ‘B’ doesn’t have a coach screw. It can clearly be seen that the use of a screw and a big notch length increases the maximum ultimate load and the slip modulus of the connection.

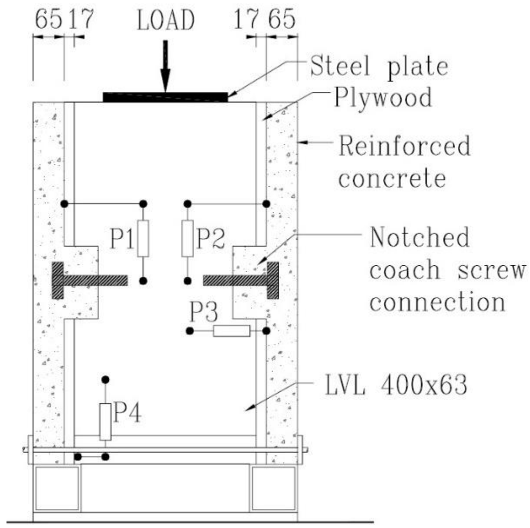

Figure 5.16: Test set-up, adapted from [91]

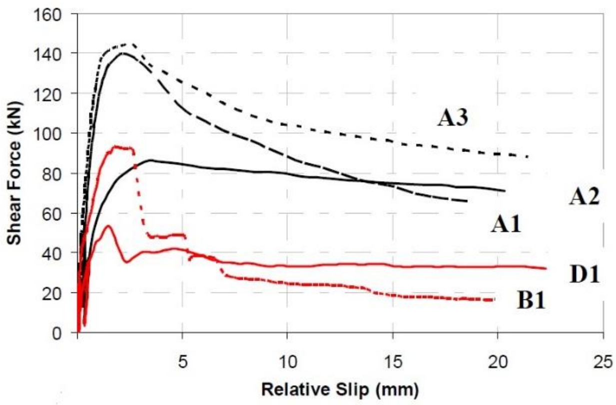

Figure 5.17: Load-slip curve [91]

The method of determination for the mean slip modulus used in SLS design for notched connections is dependent on the depth of the notch, h_n. One condition is that the length of the notch must be at least $150\ mm$.

$$K_{ser} = \begin{cases} 1000\ kN\ /mm\ /m & for \quad h_n = 20\ mm \\ 1500\ kN\ /mm\ /m & for \quad h_n \geq 30\ mm \end{cases}$$

If the depth of the notch is in between the limits, linear interpolation can be used. The mean slip modulus for ULS design is equal to the mean slip modulus for SLS design.

The design load carrying capacity of the notch is found by taking the smallest of: the resistance of timber and concrete to shear and the resistance of timber and concrete to crushing. One last check to verify the connection is the resistance of the fastener to an uplift force. This force occurs when the concrete deck and the timber beams want to separate; a tensile force in the fastener arises.

5.2.7 Serviceability limit state

Deformations

For a timber simply supported beam, the allowable initial deflection (short term) and the maximum final deflection (long term) are limited to respectively

$$w_{inst,max} \leq \frac{l}{300}$$

$$w_{fin,max} \leq \frac{l}{250}$$

There is also a limit for the additional deflection. This deflection results from the variable loading and the long term effects of creep and shrinkage:

$$w_{add,max} \leq \frac{3}{1000} l$$

It is possible that a precamber, w_c, should be added to ensure compliance with the deformation limit(s). The used precamber is taken equal to the instantaneous deflection caused by the self-weight of the concrete and the timber.

Vibrations

If the quasi-static loading is at least 5 kN/m^2, no calculation for vibrations is required. If the quasi-static load is smaller than the required value, either the fundamental frequency must be at least 3 Hz or the deflection in the short term under quasi-static loading may not be bigger than 34mm.

Cracking of concrete

Cracks in concrete in indoor environments should be limited to ensure a good appearance of the structure. The maximum crack width is $w_{max} = 0.4\ mm$. This value may be higher if specific limits are stated for an acceptable appearance. The cracking is assumed controlled if a minimum reinforcement is applied in mm2/m.

5.3 Final designs and design choice

In Table 5-2 a summary of the designed floor slabs is given. The slabs made with solid wooden beams have a length of 6000mm because solid wood is not available in bigger sizes, the width of the beams is 100mm. Due to the limited length, the floor slabs with the solid wood cannot be implemented in the case study building. The floors made with glulam beams have a span length of 10800mm, the width of the beams is 140mm. In all the designs the concrete deck is 80mm thick and the width of the floor slab is 2400mm.

Timber type(s)	Connection type	h_{tot} [mm]	h_{t1} [mm]	h_{t2} [mm]	ECI [€/m^2]	γ_1 ULS initial
Solid C24	Screw	360	280	0	1.80	0.24
Solid C24	Inclined	350	270	0	1.55	0.29
Solid C24	Notch	320	240	0	1.50	0.78
Glulam C24 and D50	Screw	570	370	120	5.54	0.25
Glulam GL24 and GL32	Screw	560	360	120	3.18	0.25
Glulam C24 and D50	Inclined	540	340	120	5.51	0.53
Glulam GL24 and GL32	Inclined	530	330	120	3.13	0.53
Glulam C24 and D50	Notch	510	310	120	5.24	0.92
Glulam GL24 and GL32	Notch	510	310	120	2.90	0.92

Table 5-2: Summary designed and verified TCC floor slabs

In Figure 5.18 the final design of the floor slab which will be used in the floor system is shown. The choice is made to use the floor slab made with glulam beams and notched connections because it has the lowest floor height and the lowest ECI for a floor span of 10.8m.

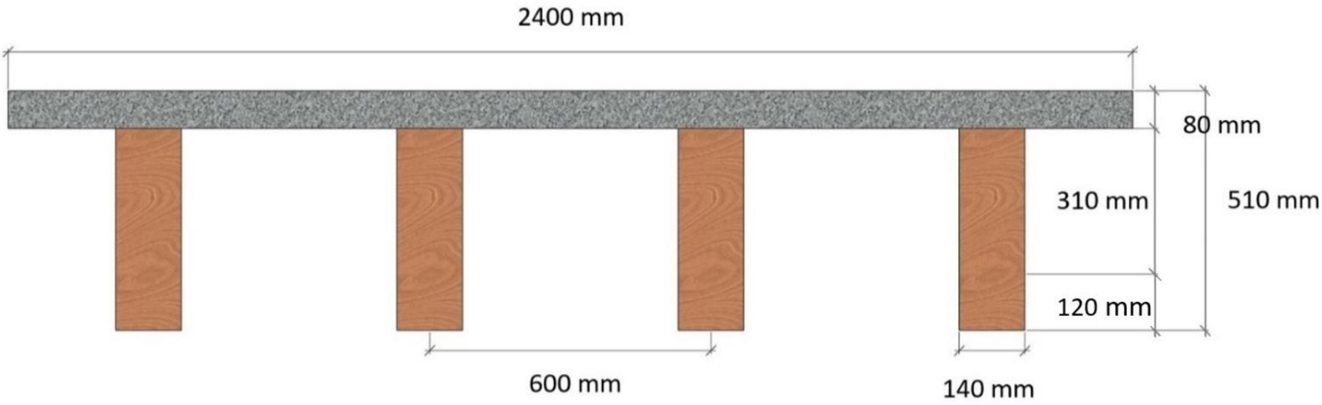

Figure 5.18: Final design TCC floor slab

The used concrete type is C30/37 and the timber strength classes are GL24h and GL32h. Since two different timber classes are used, extra thought has to be given to the choice of timber specie for each of the timber classes. Properties influencing the behaviour of timber under changing temperature or moisture content, for instance the thermal expansion coefficient, should be as close as possible to ensure that the behaviour is similar and no extra forces in the timber beams arise.

A comparison between the height and the bending stiffness of the TCC floor slab and a full timber element and a concrete hollow core slab floor are stated to point out the improvement or decline of these parameters.

The final total height of the floor slab is 510 mm. The required height of a full timber rib floor to be able to withstand the same loads at the same span length is 637mm. This is about 1.2x as high as the TCC floor slab. The HCS has a required height of 260mm, which is almost 2x smaller than the TCC floor slab.

This height is mainly required to ensure a big enough bending stiffness to decrease the deflection of the floor at mid-span. The bending stiffness of the TCC floor slab is $4.7 * 10^{13}\ Nmm^2$. The bending stiffness of a floor slab with the same glulam beams as the TCC floor but a deck made of LVL with a thickness of 37mm is $2.2 * 10^{13} Nmm^2$. This is 2.1x smaller than the bending stiffness of the TCC floor slab. The bending stiffness of the HCS is about $5.2 * 10^{13}\ Nmm^2$ which is only 1.1x higher than the bending stiffness of the TCC floor slab.

6 Design and verification connections

In this chapter the connections used to fasten the floor slabs to the edge beams and to fasten the floor slabs to each other are designed and verified. First a few design options for both of the slab-beam and the slab-slab connections are made. Hereafter, the tolerances that have to be accounted for are determined and a choice between the design options is made. Lastly the joints will be verified.

6.1 Design

In this paragraph, a few design variants for the demountable connections in the floor system are illustrated. As stated in paragraph 2.1.3, the connection between the head end of the floor slab and the edge beam has the most requirements. Therefore, this connection will be the most difficult to develop. An emphasis is set on the design of the slab-beam connection at the head end.

6.1.1 Slab-beam connection, head end

To be able to design the connection at the head end of the floor slab, a framework is first made. This framework consists of some starting points and the loads which will act in the connection.

- Since the floor slab is rather thick, the desire is to make an integrated system in which the edge beam and the floor slab are in one plane instead of the slab being supported on top of the edge beam. This will result in a height reduction;
- The floor slab was designed to be implemented in Bouwdeel D, so also the edge beams and the connections will be designed for this situation. This means that the edge beams will either be simply supported spanning 1.8m or they will be continuous beams with a span of n*1.8m. For simplification of the design procedure the choice is made to use simply supported beams spanning 1.8m on which a floor slab with a width of 1.8m will be placed;
- The timber-concrete composite slabs will be supported by the edge beams at the head ends. Here all the loads from the floor slab will be transferred to the beams, then the columns and finally the foundation. A vertical force and a moment will arise on the 'integrated' edge beam due to the loading of the floor slab. Horizontal forces in two directions will arise due to wind loading.

Two design options for the connection are made.

Connection 1 is illustrated in Figure 6.1, Figure 6.2 and Figure 6.3. Horizontal forces in two directions due to wind and the tensile component of the moment are taken up by the toothed-plate. This plate is adhesively bonded to the flange of the edge beam. Toothed-plate connectors are intended to be used with a bolt through the centre through which a part of the shear force is transferred. In this situation no bolt will be implemented and thus the only force transfer will be through the toothed-plate. When a toothed-plate connector is used in its originally intended manner, the plate can be mounted by using a hammer [92]. In this situation, the self-weight of the slab can produce the required push-in force. If this is not enough to fully push the teeth into the timber, a small pressure can be exerted on the top of the slab, comparable to the force exerted by a hammer.

The horizontal compressive force originating from the wind loading and the compressive component of the moment is taken up by the compression bolt at the top of the edge beam. This bolt will be tightened after the installation of the slab to ensure contact and thus force transfer between the bolt and the slab. To ensure structural soundness, the slab will be vertically fixed to the edge girder by fastening two small angle sections to the outer sides of the outer timber beams and the edge girder with screws and bolts (tension bolt).

Both the compression and tension bolts will be attached to the edge beam in the factory. This reduces time during the erection procedure. Moreover, the tension bolts can be used as a guide for the placement of the floor slabs. The bolts will be attached to the beam by welding the bolt or a nut in which a bolt is turned, to the beam. Other possibilities would be to cut a hole with a thread and inserting a bolt or to stud weld a thread end to the beam.

The edge beams that can be used in this connection are: integrated open SFB and IFB beams, closed integrated THQ, THQa and RHSFB beams, an L-profile and a profile built-up of steel plates or steel angle sections.

When designing the required dimensions of the connection, the required end and edge distances of the toothed-plate and the compression bolt have to be taken into account. The restrictions of the toothed-plate results in a normative size for the flange width of the edge beam.

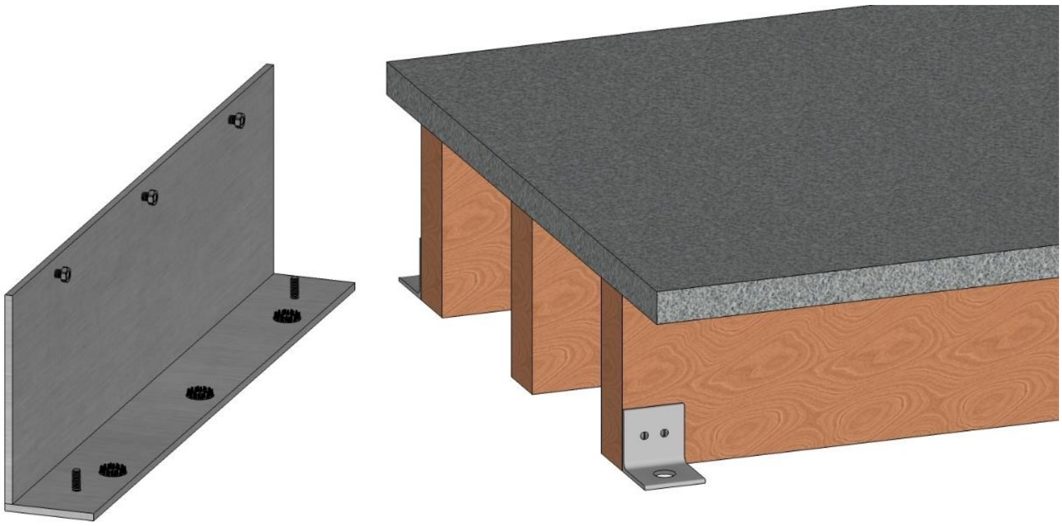

Figure 6.1: 3D view connection 1

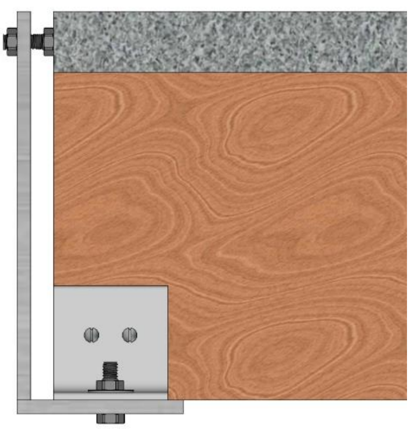

Figure 6.2: Side view connection 1

Figure 6.3: Section cut through connection 1

Connection 2 is illustrated in Figure 6.4, Figure 6.5 and Figure 6.6. A connection plate is mounted to the floor slab by screwing it onto the timber beams. The flange of this plate will be placed on top of the edge beam through which vertical forces can be transferred. Here structural soundness is assured by vertically fixing the floor slab to the beam by fastening bolts. In the web of the connection plate, cut-out V-shapes are made which are placed over bolts present on the edge beam. These bolts will transfer horizontal forces in two directions due to wind and the tensile component of the moment. This force transfer is guaranteed by fastening the bolt to the connection plate with two nuts, present at each side of the plate. Again, a compression bolt is used which takes up the compressive component of the moment and the horizontal compressive part of the wind loading.

All the vertical and horizontal bolts will be attached to the edge beam in the factory. This reduces time during the erection procedure. Moreover, the vertical bolts can be used as a guide for the placement of the floor slabs. The bolts will be attached to the beam by either welding the bolt or a nut to the beam. Other possibilities would be to cut a hole with a thread and inserting a bolt or to stud weld a thread end to the beam.

Edge beams which can be used for this connection are: a rectangular hollow section and open UNP, UAP and L-sections.

Figure 6.4: 3D view connection 2

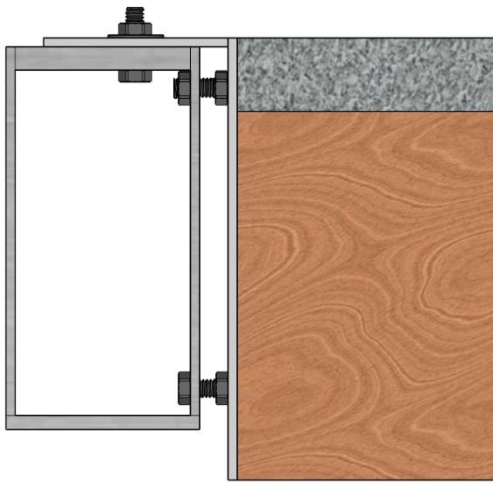

Figure 6.5: Side view connection 2

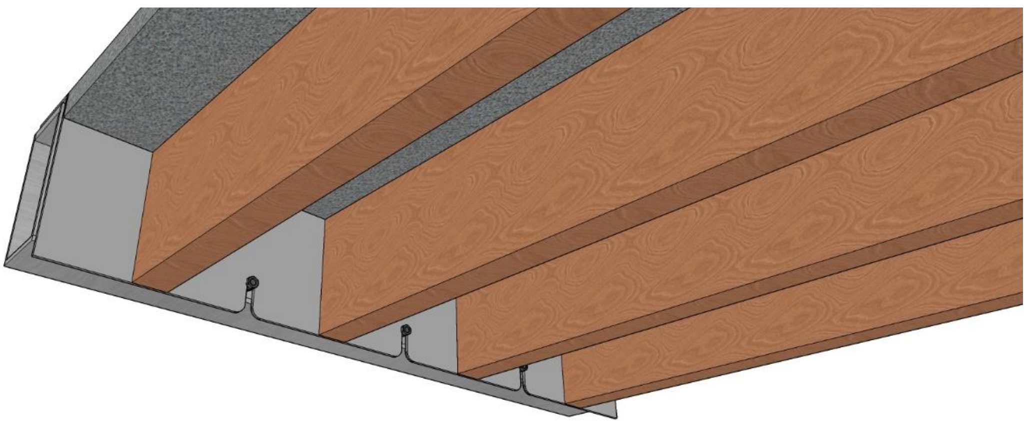

Figure 6.6: Bottom view TCC floor slab with connection plate

6.1.2 Slab-slab connection

In the connection between the slabs, only forces arising due to wind are present. These will act as shear forces in the longitudinal interface between the plates and as tensile or compressive forces in the transverse direction of the plates. Both these forces act on the slab-slab connection as a shear force. Two design options are made.

The first possibility is to cast-in concrete bolt-anchors in groups of two at the sides of the concrete slab. The bolt anchors are anchored to the concrete plate with reinforcement. When the floor slabs are placed side-by-side, a steel plate is placed over the anchors and four bolts are used to fasten the plate to the slabs, illustrated in Figure 6.7 and Figure 6.8. To ensure force transfer in all horizontal directions, Wyli tension discs are used instead of regular washers, see Figure 6.9. The thick part under the flange of the tension disc is inserted into the oversized hole. The nut is placed in the flange after which the nut is turned to fasten the bolt. When the nut is turned, the thick part of the flange is turned as well. The left-over space in the oversized hole is now filled and force transfer can occur between the bolt and the plate with very little slip.

The anchors are casted-in in a recess which is as deep as the thickness of the steel plate which will be installed in the concrete slab. The steel plate will fall into the recesses due to which the plate will be level with the top surface of the concrete and the nuts won't stick out too high above it. Because the concrete is only 80mm thick, the anchors can't be placed low enough to ensure a level concrete top surface.

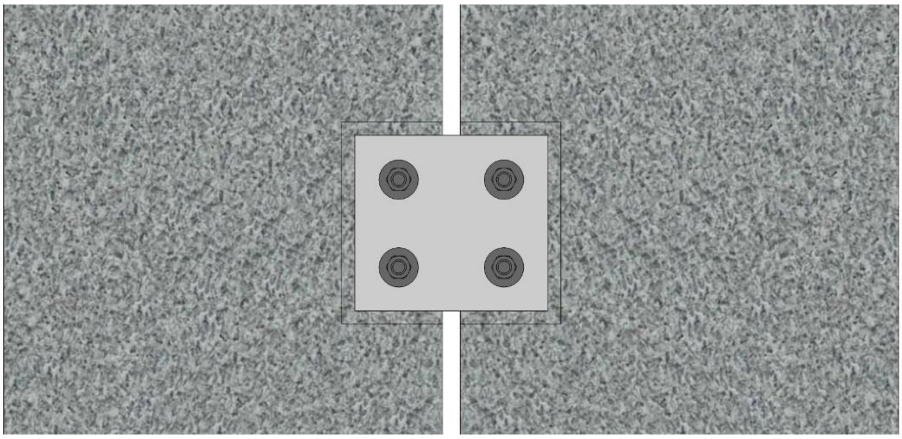

Figure 6.7: Slab-slab bolted connection, top view

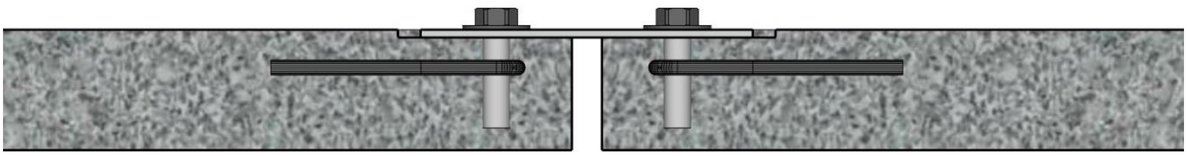

Figure 6.8: Section cut slab-slab bolted connection, side view

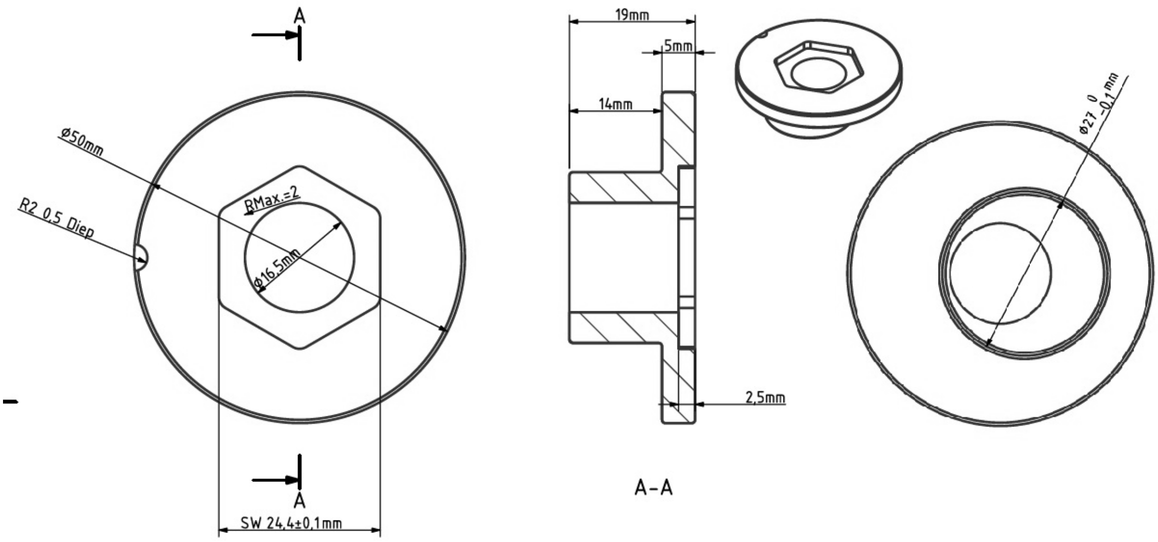

Figure 6.9: Wyli tension disc, adapted from [93]

For the second option a steel angle section is casted in at the sides of the concrete slab, illustrated in Figure 6.10 and Figure 6.11. The flange ensures that the section is anchored to the concrete slab. When the floor slabs are placed side-by-side, a steel plate is placed on top of two adjacent casted-in angle section and welded to the sections. Again, the casted-in elements are placed lower than the top surface of the concrete.

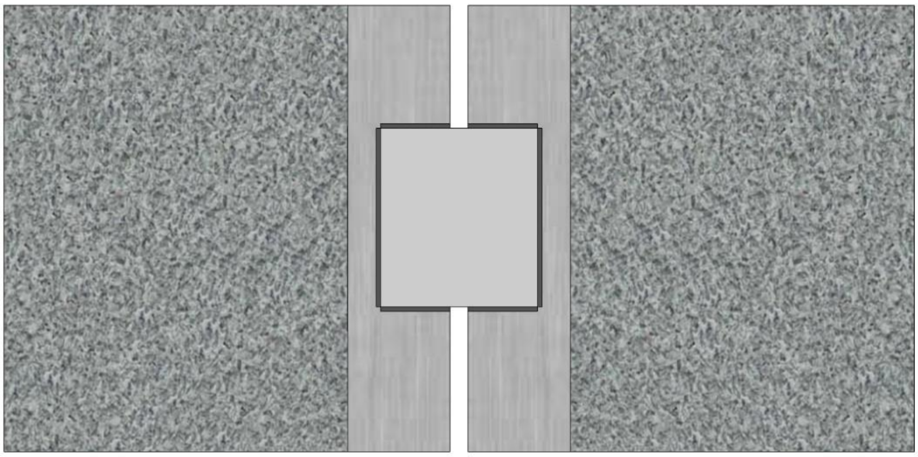

Figure 6.10: Slab-slab welded connection, top view

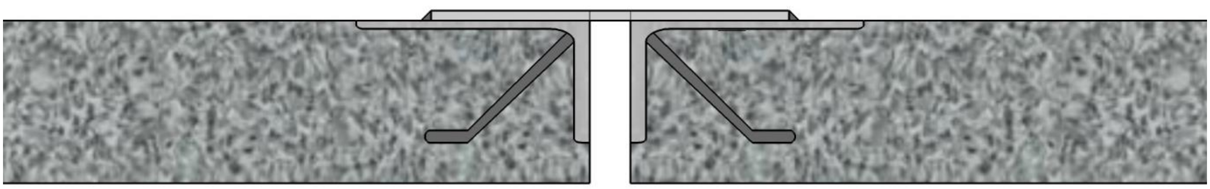

Figure 6.11: Section cut slab-slab welded connection, side view

6.1.3 Slab-beam connection, side

To simplify the erection of the floor system and the fabrication of the floor slabs it would be a big advantage if the slab-beam connection at the side of the floor slab is similar to the connection between the slabs. If the same connection is used, the floor slabs will all be the same. This will reduce the chance of mistakes being made during the fabrication of the floor slabs and during erection because the position of the slabs in the floor surface is not fixed. Using the same connection as the one used between the slabs is possible by using edge beams, for example an angle section, of which one of the flanges is installed level to the recess in the concrete deck or the top of the angle section casted into the concrete deck. Now a steel plate can be placed over the bolt anchors in the concrete slab and the already installed bolts in the angle section or over the casted-in angle section. The plate is fastened by tightening the bolts or by welding the plate. The bolts will be attached to the edge beam in the factory by welding the bolt to the beam. As mentioned before, other attachment possibilities exist.

See Figure 6.12 and Figure 6.13 for an illustration of the connection options.

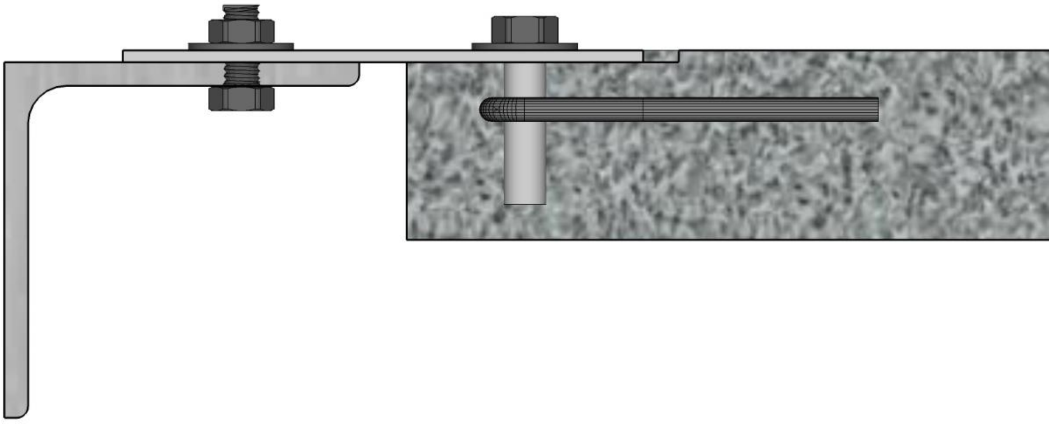

Figure 6.12: Section cut slab-beam bolted connection, side view

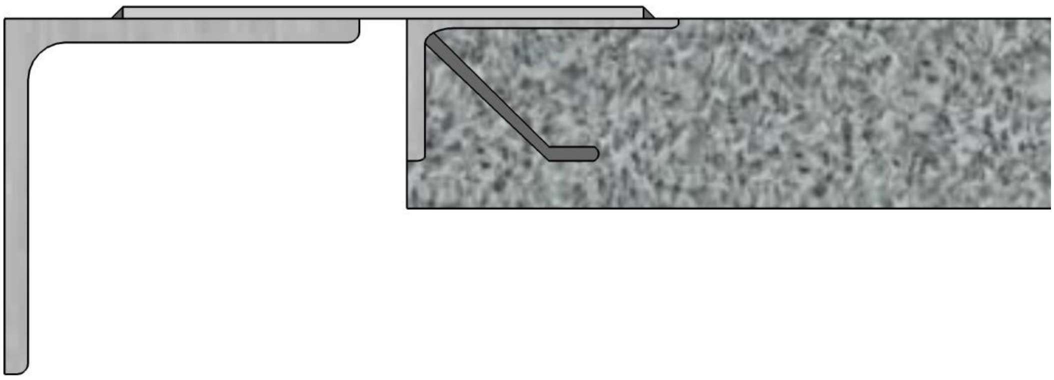

Figure 6.13: Section cut slab-beam welded connection, side view

6.2 Tolerances

Tolerances are of great importance when designing demountable and reusable connections. Like in 'regular' buildings, all elements have to fit into, next to or between each other during erection. In the case of a reusable system, the elements have to fit a number of times. To be able to determine the feasibility of the designed connections and also to choose between the different design options, the tolerances which have to be taken into account are ascertained.

Tolerances occur in vertical and both horizontal directions. They occur due to deviations in for example the size or position of an element during the fabrication and erection stage. The tolerances are determined by adding the maximum deviation in the fabrication stage to the maximum deviation in the erection stage.

Fabrication tolerances stated in the standards of prefabricated concrete elements are very big, 28mm for prefabricated non-prestressed floor slabs [94]. After consulting with Gerard Wittebol from VBI, the conclusion was drawn that concrete tolerances can be much reduced compared to the ones stated in the codes. The length of the slab can be produced with a tolerance of +- 5mm and the width of the slab can be accurately fabricated within a +-3mm reach.

Assembly tolerances can be reduced by measuring the deviations during erection with respect to the intended position. When a difference is found, the deviation can be adjusted by re-placing an element or pushing an element straight. These handlings increase the erection time and also the costs. Therefor it would be best if the connections can accommodate fabrication tolerances as well as assembly tolerances.

The work out of all the tolerances is given in paragraph 9.3 of appendix C. In Table 6-1 a summary is given of the required dimensions to ensure a good fit of the elements in the joints.

Connection	**Slab-beam head end 1**	**Slab-beam head end 2**		**Slab-slab**
	Hole angle section	**Slotted hole V-shape**	**Hole flange connection plate**	**Hole steel plate**
Bolt diameter	$d = 20$	$d = 20$	$d = 20$	$d = 16$
Start size hole	$d_H = 24$	$d_H = 22$ $l_H = 26$	$d_H = 24$	$d_H = 18$
Size hole fabrication tolerances	$d_H = 33$	$d_H = 26$ $l_H = 30$	$d_H = 33$	$d_H = 28$
Size hole all tolerances	$d_H = 52$	$d_H = 39$ $l_H = 30$	$d_H = 52$	

Table 6-1: Summary required dimensions

6.3 Choice of connections

Slab-beam connection, head end

In the design of connection 1, only the hole in the angle section through which the bolt for vertical fixation will go requires rather large tolerances.

When regarding connection 2, to ensure horizontal force transfer in a plane parallel to the head end of the slab, the distance between the bolt shank and the connection plate must be reduced to only about 4 mm at each side. When the erection tolerances are regarded in the design, this value is gravely exceeded. So, when using this connection the erection tolerances must be resolved during the erection stage, which increases erection time and costs.

Connection 1 will be used in the floor system design because the tolerances can easily be accommodated and because the design is more flexible since it can be used in combination with more types of edge beams.

Slab-slab connection

To ensure that the welded connection always fits, the casted-in plates should cover a bigger area than the cover plate. For the bolted connection, the holes in the cover plate should be oversized to ensure a good fit.

On the basis of the tolerances, the welded connection would be the best option. But to demount this connection, the welds have to be cut and the plates have to be cleaned before the next erection. For the bolted connection, the bolts are only loosened and then tightened again during the next erection. So, from a demountability and reassembly point of view, the bolted connection is preferred. Therefor this connection is chosen.

Slab-beam side

This connection is essentially the same as the slab-slab connection, so the same reasoning applies. The bolted connection will be used.

6.4 Verification

In this paragraph the results of the verifications of the components of the joints are given. In appendix C the complete verification can be found.
The dimensions used in the verifications are the final used dimensions which result from iterations during the calculation process.

6.4.1 Loads

Before the verifications can be done of all the elements in the joints, the acting forces are determined. Forces resulting from self-weight, variable loadings and wind are considered. For every verification in ULS the design load is determined by

$$F_{Ed} \; or \; q_{Ed} = \max \begin{cases} \gamma_G G_k + \sum_{i \geq 1} \gamma_Q \Psi_{0,i} Q_i \\ \gamma_G \xi G_k + \gamma_Q Q_{k,1} + \sum_{i > 1} \gamma_Q \Psi_{0,i} Q_i \end{cases}$$

6.4.2 Slab-beam head end

The elements that have to be structurally verified in the connection between the head end of the floor slab and the edge beam are: the edge beam, the toothed-plate connector and the timber-concrete composite floor slab. The tension bolt will not transfer forces in the regarded situation, it is only applied as a guide and for ensuring that the floor slab is vertically fixed to the edge beam.

Edge beam

The used edge beam will be an L-profile, shown in Figure 6.14. This choice is made to keep in line with the current structural system of the case study building and the architect's vision of the building. The required cross section of the profile is not available as a standard L-section.

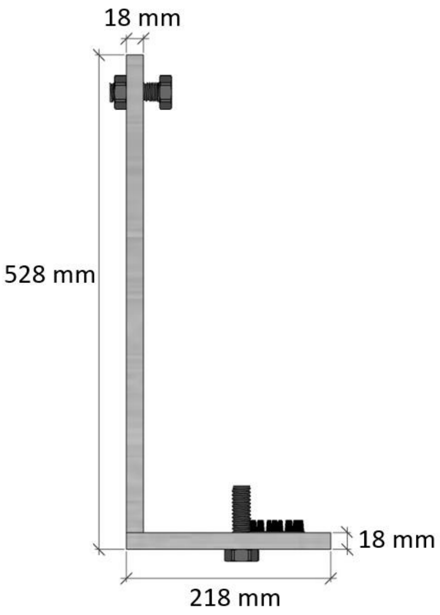

Figure 6.14: L-section beam

Checks in ULS:
- Transverse bending
- Longitudinal bending in combination with transverse bending
- Shear force
- Tensile and compressive stress due to wind

SLS check:
- Deformation

The acting transverse bending moment is $10.9\ kNm$ and the resistance of the beam is more than three times higher: $34.3\ kNm$. The resulting UC is 0.32.

The acting global bending moment in longitudinal direction is $21.8\ kNm$. The resistance of the beam is determined by first reducing the available area of the cross section with the area which is already used to resist the transverse bending moment and shear force. Even after the reduction the bending moment resistance is 20 times as high as the acting bending moment.

The acting shear force is 33.3 kN and the shear resistance of the beam is $1494.6\ kN$.

Tensile stresses arising in the beam due to the wind loading are maximally $22\ MPa$, which is less than 10% of the yield strength of the beam.

The maximum allowed initial and additional deflections of the edge beam are respectively $6\ mm$ and $5.4\ mm$. The acting initial and additional deflections are only $0.044\ mm$ and $0.023\ mm$ so the deflection is by no means normative.

All the unity checks done suffice. The normative check in ULS for the edge beam is transverse bending of the profile. The conclusion can be drawn that the choice for a simply supported edge beam is good because the use of a continuous beam will only reduce the deflections and the acting global bending moment, not the transverse bending moment.

Toothed plate

When determining the load bearing capacity of the toothed-plate connector, the angle of the force to the grain direction is not of importance. The capacity is dependent on the thickness of the timber, the height of the connector teeth and the density of the timber. Since these are independent of the direction to the grain, the resistance is equal in all directions. Toothed-plate type C11 is used, which has a shear resistance of $9.9\ kN$. The acting shear force is a combination of the tensile component of the acting bending moment and shear forces in two horizontal directions due to wind. The maximum possible acting force results in $9.0\ kN$ so the toothed plate has a sufficient load bearing capacity.

Timber-concrete composite floor slab

In chapter 5 the TCC floor slab has been verified, but the forces acting due to the manner in which the slab is supported were not yet considered. The slabs are supported on the edge girder by the timber beams due to which compressive stresses perpendicular to the grain of the timber occur. The acting stress is $1.2\ MPa$ and the resistance to compression perpendicular to the grain is $1.6\ MPa$ so the stress doesn't result in failure.

The compression bolt induces in a compressive force on the floor. This force is a combination of the horizontal component resulting from the bending moment and wind loading. The stress acts over the area of the nut pressing against the floor element. This results in a compressive stress of $10.9\ MPa$. The bolt can either press against the concrete or the timber. The resistance to compression parallel to the grain of the timber is $15.4\ MPa$ and the compression strength of concrete is $20\ MPa$. So, wherever the bolt is pressing, the stress can easily be taken. One important note is that the bolt can't be placed too close to the top of the concrete slab. In this case, the top part of the concrete could fail and break off.

6.4.3 Slab-slab

Four checks have to be done to ensure the proper working of the slab-slab connection. Because the tensile component of the global bending of the floor field due to wind acts in the floor field, a tensile force perpendicular to the plates occurs. This results in a shear force in the bolts, a bearing stress in the steel plate, a tensile force on the concrete and a tensile stress in the reinforcement all acting in the transverse direction of the floor slab. A shear force parallel to the floor slabs also occurs, which results in a moment acting on the bolt group. This moment results in a shear force in the bolts, a bearing stress in the steel plate parallel to the floor slab and a tensile force on the concrete perpendicular to the floor.

The shear resistance of the bolts and the bearing resistance of the plate are very high, $77.2\ kN$ and $64\ kN$ respectively. The resistance of the concrete to tensile breaking for pure tensile loading is only $12.4\ kN$ and the tensile resistance of the reinforcement is $18.5\ kN$. The maximum acting forces are: shear force = $9.2\ kN$, tension in concrete = $5.9\ kN$ and tension in the reinforcement = $2.2\ kN$.

All the elements have a sufficient load resistance. The normative UC is the breaking of concrete.

6.4.4 Slab-beam side

The slab-beam connection is the same as the slab-slab connection, but at one side a steel profile is present instead of a concrete slab. Resistance to tension of the steel profile is bigger than that of concrete, so no check is necessary. All the other resistances are equal to those for the slab-slab connection.

The maximum acing forces are a bit higher at the slab-beam position: shear force = $13.3\ kN$ and tension in concrete = $11.4\ kN$. Still all the all the elements have a sufficient load bearing resistance. The breaking of the concrete is again the normative UC for this connection.

7 Final design

In this chapter a summary of the developed demountable floor system is presented. Hereafter, this floor system is compared to a standard hollow core slab floor system based on costs and additional required measures. Lastly a parametric study is done of the floor system to determine its limitations.

7.1 Final design

In Figure 7.1, Figure 7.2 and Figure 7.3 the final design of the floor system is illustrated. The used dimensions of the edge girder and the floor slab can be found in chapter 5 and chapter 6.

Figure 7.1: L-girder final floor system design

Figure 7.2: Side view final floor system design

Figure 7.3: TCC floor slab final floor system design

7.2 Comparison hollow core slab floor system

To gain insight into the additional costs and handlings required for the demountable floor system, a comparison is done with a regular concrete hollow core slab floor. The HCS will also be supported on an integrated L- beam. The connection to the edge beam will be made with a casted in-situ concrete joint with reinforcement.

A cost inquiry was sent to a professional building costs advisor. The received cost estimate only comprises the material costs of the examined floor systems. Many other factors that are not included in the cost estimate influence the costs of a floor system, for instance transportation costs and labour costs.

- A HCS of 260mm width, placed on top of L- beams and connected with reinforced cast in-situ joints
  - Hollow core slab: €60 /m^2
  - L-section: €220 /m^1

    → €101 /m^2
- TCC plate
  - TCC slab: €225 /m^2
  - L-section: €500 /m^1

    → €318 /m^2

The TCC floor slab is about 3x as expensive as the HCS. A few reasons can be given for the big difference in costs:

- The HCS is one of the most widely used floor slabs in the Netherlands. Due to its regular use, the fabrication costs are very low. It is the cheapest floor system solution used in the Netherlands;
- Prefabricated TCC floor slabs are almost never used in the Netherlands. This results in higher fabrication costs of the floor slab;
- Glue laminated timber beams are quite expensive;
- The L-section used for the HCS is almost twice as small as the one used for the TCC floor system, which explains the lower costs for the L-section.

Besides the additional material costs of the structural floor system, extra measures and handlings are required when adopting the demountable floor system:

- To meet fire safety requirements, fire resistant measures might have to be taken.
- Thermal insulation could be required, depending on the separating function of the floor slab.
- Sound insulation is practically always required in floor slabs, the amount is dependent on the functions found in the spaces separated by the floor slab.
- A top floor must be used because the top surface of the structural floor is not level nor completely closed.

The hollow core slab floor system also requires all the extra measures stated above except from the fire safety measures. A HCS can be designed in such a way that it satisfies the fire safety requirement. There is a limit, very high fire safety requirements can't be met with a proper design alone and additional measures do have to be implemented.

The required sound insulation and the necessity of a top floor will decrease if a reinforced structural topping is used.

7.3 Parametric study

A parametric study is done of the floor system to determine the application possibilities. First an assessment is done of the floor slab. From this assessment input data is found for the study of the slab-beam connection at the head end of the floor slab. The connection between the slabs and at the sides of the slabs is not assessed because these are easily adapted to ensure a sufficient resistance to the acting loads.

The assumptions made in the study of each parameter are stated in the text below.

7.3.1 Floor slab

A study is done to determine the influence of several parameters on the maximum possible span length of the floor slab. The design of the floor slab is dependent on a lot of different variables. Five parameters are chosen which are assumed to have a noticeable influence on the possible span length. These parameters are: the type of timber used for the upper and the lower part of the beams, the type of concrete used, the total height of the timber beams and the centre-to-centre distance of the beams. These parameters are changed together with the slab length. An investigation is done whether the design of the floor slab suffices when the changed parameters are adopted.

The normative unity check (UC) in the floor slab design is the additional deflection. This UC is chosen as the indicator to show whether the input data results in a slab able to withstand the applied loading. This UC is depicted on all the vertical axis in the figures. When the line exceeds the limit value of 1.0, the slab doesn't suffice.

For the study of the influence of the timber and concrete types, the dimensions of cross-section of the floor slab are fixed. The material types are regarded since they define the material properties and also the material's resistance to stresses.

Timber type 2 is taken as GL32 in the study of timber type 2. As can clearly be seen in Figure 7.4 the type of timber used at the top of the timber beams doesn't change the resistance of the floor slab when regarding the deflection. When GL20 is used, combined bending and tension in the timber also results in a UC higher than 1 but the UC for deflection is still normative.

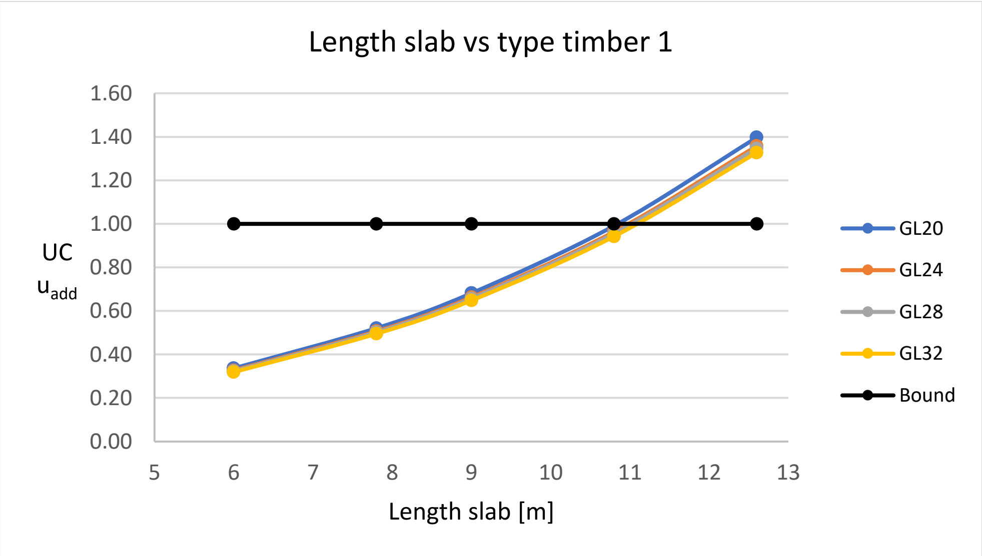

Figure 7.4: Length floor slab vs timber type 1

The choice for timber type 2 is also not very definable for the resistance of the floor slab as can be seen in Figure 7.5. In this check, timber type 1 is taken equal to GL24. Obvious is that only small span lengths can be implemented when using GL20. The other timber types don't result in big differences for the changing slab lengths. GL24 and GL28 are almost equal and GL32 results in a bit lower UC. This result can be explained by the increasing modulus of elasticity for a higher timber type. This results in a higher bending stiffness and thus a lower deflection.

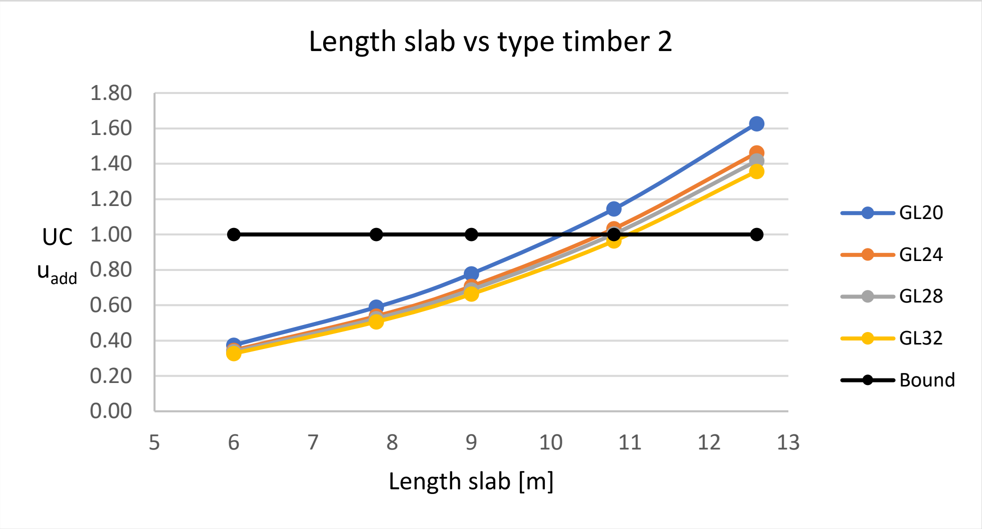

Figure 7.5: Length floor slab vs timber type 2

In the study of the influence of the concrete type, timber type 1 and timber type 2 are respectively GL24 and GL32. In Figure 7.6 can be seen that changing the span length when using concrete type C25/30 result in a slightly bigger change than when the other types of concrete are used. The creep factor increases for lower strength concrete which results in more creep deflection and thus an increased additional deflection. The difference between the creep factor for C25/30 and C30/37 is twice as big as the difference between the other concrete types. Therefor the additional deflection increases more and the influence is bigger.

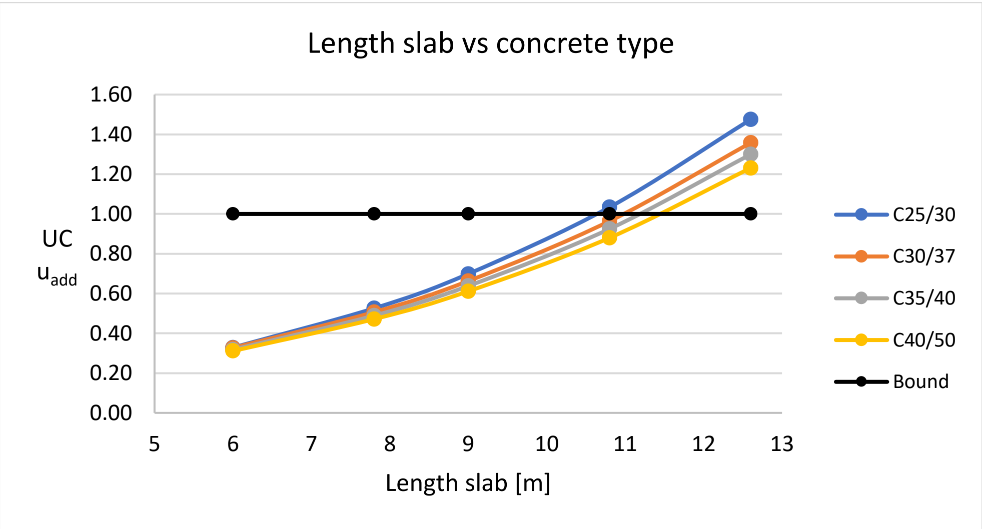

Figure 7.6: Length floor slab vs concrete type

When determining the influence of the height of the timber beams, the used timber and concrete types are the same as the ones used in the final design. The height of timber 2 is kept constant and the height of timber type 1 is changed. This is done because it is preferable to use the least amount of timber with a higher strength class. Higher strength timber is more expensive and unnecessary for a satisfactory floor slab design.

It can be seen in Figure 7.7 that a higher timber beam result in bigger possible span lengths. This is easily explained because the deflection is dependent on the bending stiffness and the bending stiffness is dependent on the height of the timber to the power three: bigger beam height → bigger bending stiffness → lower deflection.

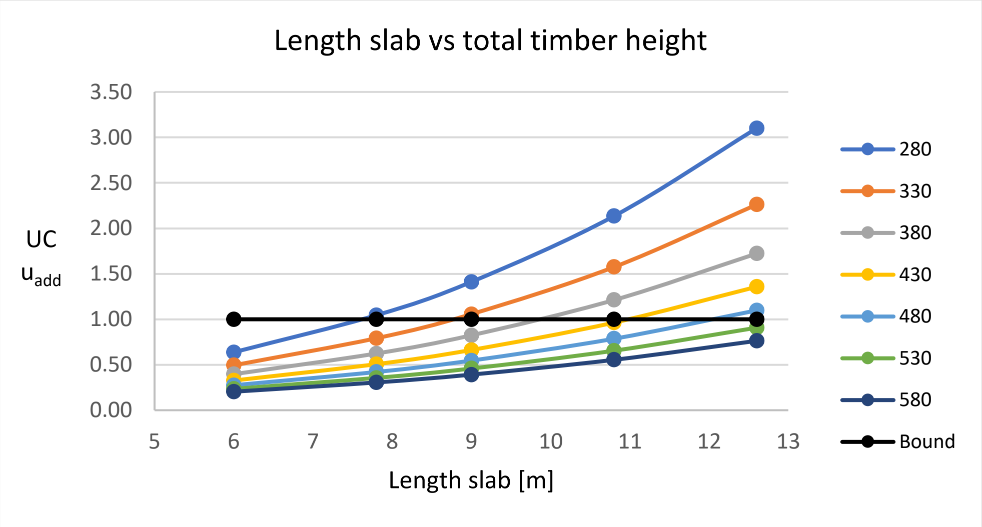

Figure 7.7: Length floor slab vs total timber height

Lastly the influence of the centre-to-centre distance of the timber beams is determined. The height concrete and timber types and dimensions are equal to the ones used in the final design. As shown in Figure 7.8, for a higher span length, a lower c-t-c distance can be used. This is easily explained by the following. When a bigger span length is used, the dead weight of the floor slab increases resulting in a bigger loading per m^2. The floor slab is verified by regarding one T-element of the floor slab. When the c-t-c distance is increased, the loading per m^1 on one T-element increases. So, increasing the c-t-c distance of the beam and the span length increase the load on the slab and inherently increase the unity check.

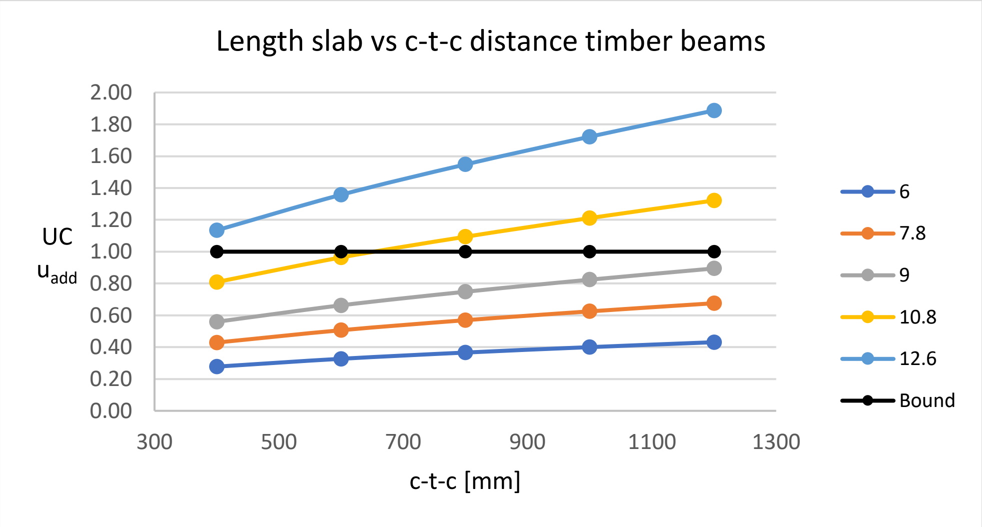

Figure 7.8: Length floor slab vs c-t-c distance timber beams

7.3.2 Slab-beam connection

The resistance of the slab-beam connection is dependent on many parameters, just like the resistance of the TCC floor slab. To make a clear determination of the influence of a specific parameter, the change in a unity check or an occurring force is determined by taking a look at different loading types. In the final design, the highest UC's are found for the shear strength of the toothed-plate and the compressive stress perpendicular to the grain of the timber. These two checks are used to determine whether the connection suffices under the changed parameters.

The acting shear force in the toothed-plate is a combination of the floor slab loading and the wind loading. Both these loading types are regarded separately. The change in the contribution of the floor slab loading to the shear force is determined by changing the span length of the slab. The change in contribution to the shear force of the wind loading is determined by changing the building height, the building length, and the wind area and terrain category in which the building is placed. The acting compressive stress perpendicular to the grain of the timber beams is dependent on the floor loading. The limit of the connection with regards to the compressive stress is determined by changing the span of the floor slab.

The study of the floor slab showed that a floor span of 12.6m can be adopted with reasonable cross-sectional dimensions. Therefor the spans used in the study of the connection are the spans used in the floor slab study. The building decree states requirements for buildings up to 70m height. Therefor this height is taken as a maximum building height in which the floor system is implemented. For all the checks, the dimensions of the building, the terrain category, the wind area and the dimensions and materials of the TCC floor slab are taken equal to the values used in the final design unless given otherwise.

First an assessment is done to determine if and when the compression perpendicular to the grain in the timber beams becomes normative. The acting stress is dependent on the width of the timber beam, the bearing length on the steel edge beam and the slab loading. The slab loading depends on the slab length, so the effect of a varying slab length is determined. The width and bearing length of the timber on the edge beam are kept constant. In Figure 7.9 it can be seen that for an increasing span length, the acting stress increases and thus the UC increases. This is logical since the bearing area remains constant but the acting load increases.

For none of the spans, the compressive stress becomes normative. If it would it could easily be avoided by increasing the width of the timber beams or by increasing the length of the bottom flange of the edge beam.

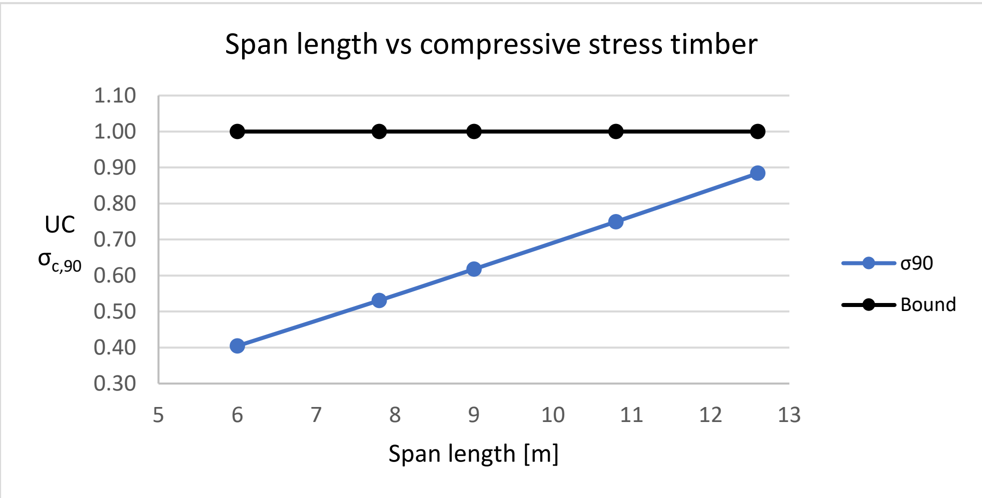

Figure 7.9: Span length vs compressive stress perpendicular to the timber beam

The change in the part of the shear force originating from the floor loading due to a change in floor span is now determined. Every unique floor length has a different optimal floor height. The height of the edge beam is taken equal to the height of the floor slab to ensure that the compression bolt introduces the compressive force in the concrete slab. For this reason, the height of the beam is taken equal to the height of the floor slab. This changing beam height results in a different lever arm when calculating the shear force in the toothed-plate due to the acting bending moment.

The acting forces given in Figure 7.10 are design loads of the floor loading. Both are calculated by using load combination 6.10b in Eurocode 0, $F_{v,M(b2)}$ is calculated by taking the variable load on the floor slab as the main variable load and $F_{v,M(b1)}$ is the part of the shear force due to the floor loading when taking the wind load as the main variable load. From the graph it is clear that the lever arm changes in such a proportion to the change in load that the acting shear force is almost equal for the different slab lengths. The biggest difference is found for a span of 7.8 m, but the increase compared to the 6 m span or 12.6 m span is only 0.3 kN for $F_{v,M(b2)}$ and 0.2 kN for $F_{v,M(b1)}$. The resistance of the toothed-plate is 9.9 kN so the increase in load is only maximally 3% of the resistance of the plate.

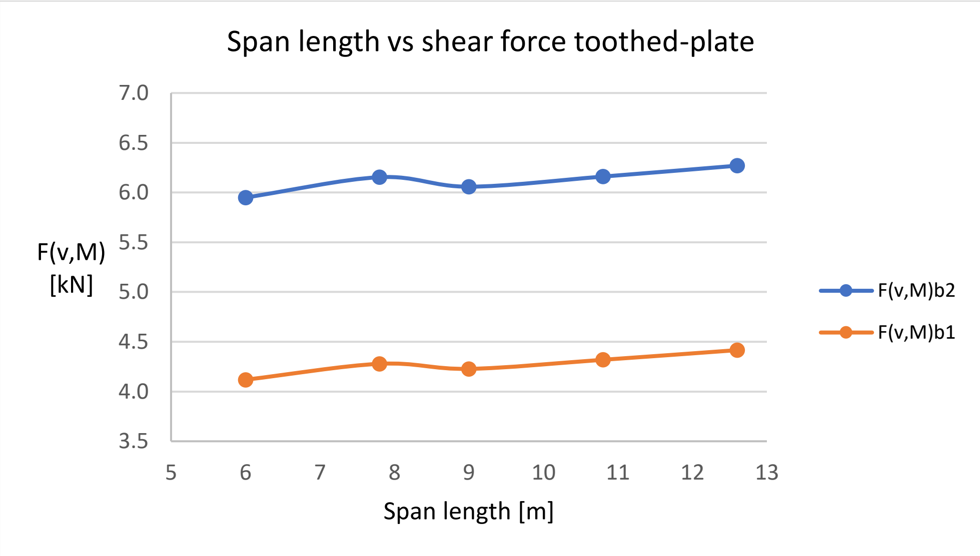

Figure 7.10: Span length vs shear force toothed-plate due to floor loading

The influence of the height and the length of the building on the acting shear force in the toothed-plate is determined. For these two parameters the change in total shear force is given for both wind on the length of the building and wind on the width of the building. The total shear force is regarded because above it is shown that the shear force due to the floor slab loading is just about constant. The line called 'Bound 1' is the resistance when using one toothed-plate.

While regarding both parameters the width of the building is taken as the length of the floor slab. Below four changes are stated which might occur due to the changing building width.

- Using a variable building width results in a change in $\frac{h}{d}$ ratio for wind on the length of the building. This ratio determines the external pressure coefficient on the leeward side. The extreme pressure coefficients on this surface are -0.5 and -0.7, so only a small difference is found, but it does influence the acting wind pressures on the building and thus the wind force on the toothed-plate slightly.

- For wind on the length of the building, the width of the building is taken as the height of the deep beam used to determine the tensile and compressive components of the bending moment in the floor surface. For a constant length of the building this results in a decreasing shear force due to the wind loading.
- When regarding wind on the width of the building, an increasing width results in an increase in bending moment acting on the floor surface and thus an increase in the shear force on the toothed-plate.
- The change in building width can result in a different number of pressure planes when regarding wind on the width of the building. This doesn't influence the shear force on the toothed-plate because the statement was made that all the floor slabs and connections have to be able to withstand the same load. To assure this, the peak velocity pressure at the top of the building was assumed to act over the entire height of the building.

To determine the influence of the building height, the length of the building is taken as 20m. In Figure 7.11 the influence of the building height on the shear force due to wind on the length of the building is shown for different widths of the building. For all the different building widths, the increase in shear force over the different building heights is almost equal. The increase in shear force reduces for bigger building heights because the peak velocity pressure does the same, see Table 7-1. The acting shear forces remain well below the resistance of the toothed-plate connector.

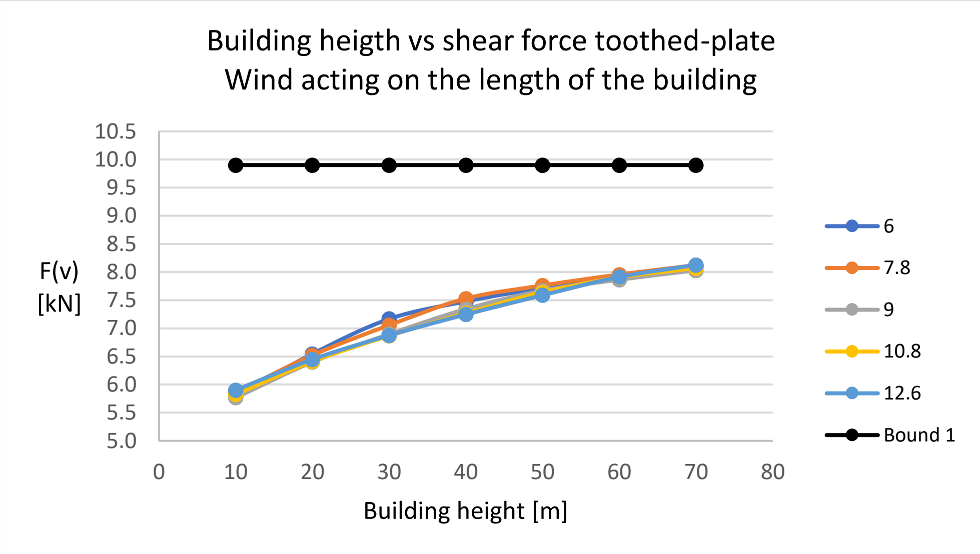

Figure 7.11: Building height vs shear force toothed-plate, wind acting on the length of the building

Building height [m]	10	30	50	70
Peak velocity pressure [kN/m^2]	0.68	1.03	1.21	1.34

Table 7-1: Peak velocity pressure per building height

In Figure 7.12 the influence of the building height on the shear force due to wind on the width of the building is shown for different widths of the building. The same observations can be made as for wind on the length of the building. One big difference can be noted; the shear force for the separate widths of the building differ more. The bigger the width, the bigger the shear force. This is consistent with the remark made above which stated that for an increase in building width, the shear force increases significantly due to the increase in global bending of the floor surface.

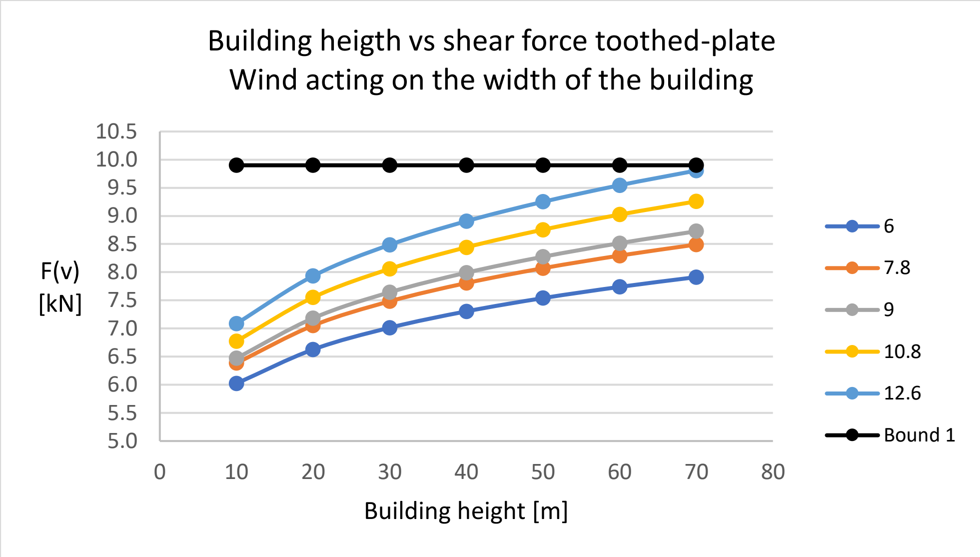

Figure 7.12: Building height vs shear force toothed-plate, wind acting on the width of the building

To determine the influence of the length of the building, the building height is taken as 20m. In Figure 7.13 the influence of the building length on the shear force due to wind on the length of the building is shown for different widths of the building. For the different building widths, the increase in shear force over the different building lengths is similar. The increase in shear force increases for bigger building lengths and the increments are bigger for smaller building widths. An increase for bigger building lengths is observed because the acting bending moment in the floor area and thus the tensile and compressive forces increase in the same manner. In Table 7-2 this increase is given for a building length of 6 m and 10.8 m. The increase is smaller for bigger building widths due to the increase in height of the deep beam.

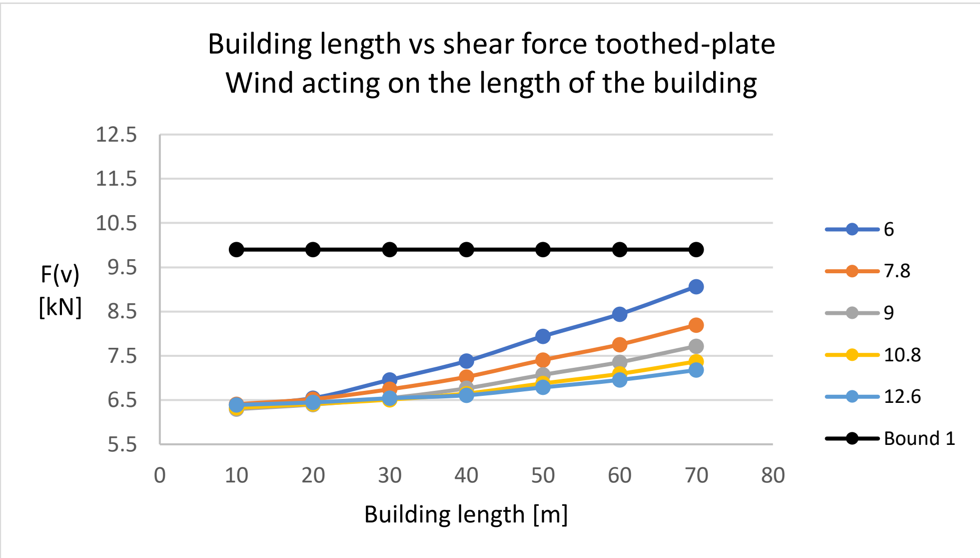

Figure 7.13: Building length vs shear force toothed-plate, wind acting on the length of the building

Building length [m]	10	30	50	70
Bending moment, W=6m [kNm]	43.1	387.9	1077.6	2112.1
Bending moment, W=10.8m [kNm]	40.9	368.1	1022.6	2004.3

Table 7-2: Acting bending moment per building length

In Figure 7.14 the influence of the building length on the shear force due to wind on the width of the building is shown for different widths of the building. For all the different building widths, the decrease in shear force over the different building lengths looks similar. The total force on the length of the building increases with $100\ kN$ for a difference in length of the building from 10m to 70m. At the same time the amount of floor slabs used and thus the amount of toothed-plates available to take up this force increases with 3 for every 1.8m. For this reason, a decrease in the shear force occurs.

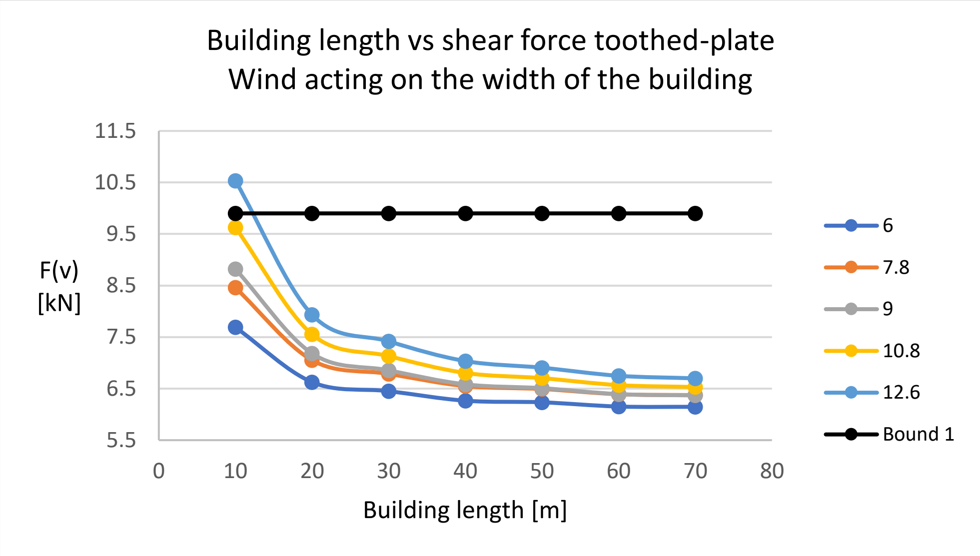

Figure 7.14: Building length vs shear force toothed-plate, wind acting on the width of the building

Now the influence of changing the wind area and the terrain category is determined. The dimensions of the building are taken equal to the case study building. Figure 7.15 shows the increase in the UC of the shear force in the toothed-plate for different building widths and different combinations of the wind area and the terrain category. The only combination which results in an exceedance of the resistance is wind area 1 and terrain category 0 which is logical since at this location the wind can reach the highest speeds. In this case the choice can be made to use two toothed-plates.

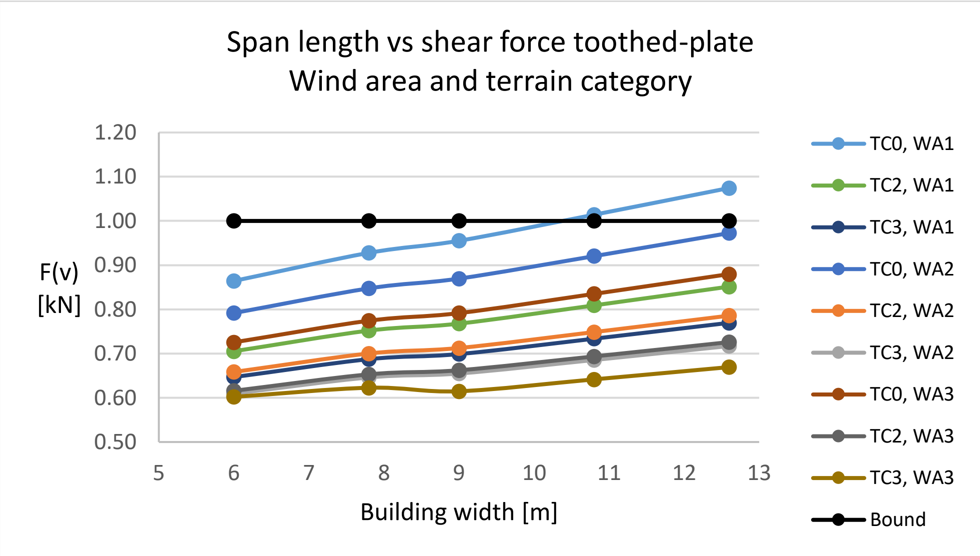

Figure 7.15: Span length vs shear force toothed-plate for changing wind area and terrain category

Lastly the limit for the combination of a certain building length and building width is determined. The building height is set to $70\ m$. Figure 7.16 shows the UC for shear force in the toothed-plate connector. The bigger the building width, the longer the building can be while still having a sufficient shear resistance of the toothed-plate. At a building length of $10\ m$ only one of the building options suffices. The buildings with a width of $10.8\ m$ and $12.6\ m$ only don't suffice when a building with a length of $10\ m$ is constructed.

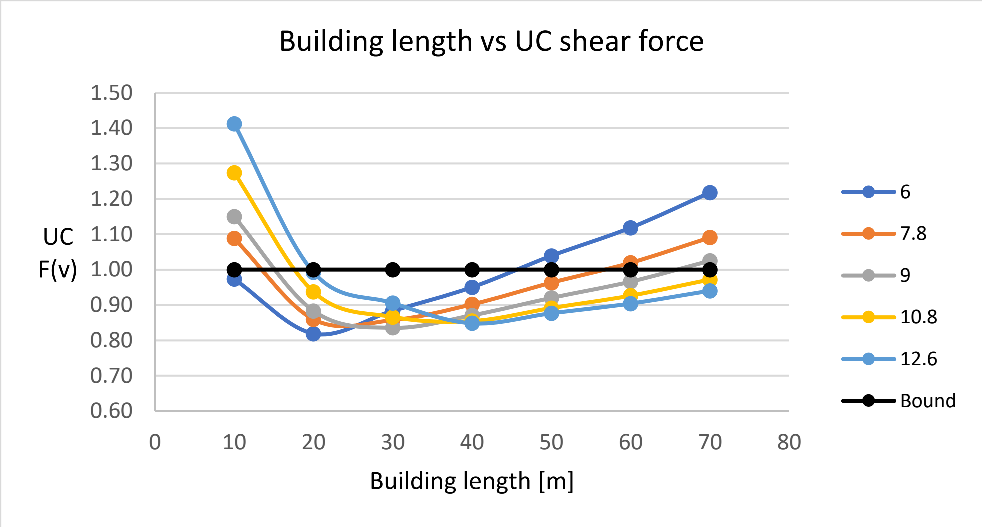

Figure 7.16: Combinations building length and width vs UC shear force

7.4 Conclusions

Comparison between floor systems

The rough cost estimate indicates that the reusable floor system has a 3x higher cost than the hollow core floor slab system. This big difference can be explained by the fact that:

- A hollow core slab floor is the cheapest floor option found in the Netherlands.
- Glue laminated timber beams are very expensive.
- Timber-concrete composite floors are a very new type of floor. This partly means that there is not a big demand for the floor slabs, which results in higher fabrication costs. If the floor slabs become a more utilized type, the costs will decrease.

The costs of the reusable floor system will probably increase when the costs of the extra measures are added. However, they will decrease if the reduction in cost due to the reuse is taken into consideration.

Parametric study

The types of timber and concrete used almost don't influence the span possibilities of the floor slab. Two important parameters though are the height and the centre-to-centre distance of the timber beams. When choosing the right values for these parameters, a total floor height of 590mm and a c-t-c distance of 600mm, a floor slab with a span length of 12.6m can be realized. A bigger span might be possible for a bigger floor height but since the required 590mm is already quite high for use in a utility building, the choice is made to not further investigate this. Also, if the c-t-c distance is decreased even further, the possible span length might be increased. When doing so, the distance between the beams becomes very small compared to the width of the beams. Now the structural system of the floor slab might change and the use of another type of slab might be advantageous.

In the joint, the compression perpendicular to the grain of the timber doesn't become normative for the investigated slab lengths.

Because the height of the edge beam changes when a different floor slab length with a different optimal height is used, the acting shear force on the toothed-plate only increases slightly. This increase is maximally 3% of the resistance of the toothed-plate, therefor the contribution of the floor slab loading to the shear force on the toothed-plate is assumed constant.

Increasing the building height results in bigger acting shear forces on the toothed-plate for both wind on the length and wind on the width of the building. When the building length is changed, the shear force increases when wind on the length of the building is observed and the shear force decreases when wind on the width of the building is regarded. Almost all used values for the length and height of the building result in satisfactory connection designs.

The only placement of a building with a width of 10.8m and 12.6m due to which the toothed-plate doesn't suffice is in wind area 1 and terrain category 0.

Many combinations of the building length and width result in a satisfactory connection design when the floor system is implemented in a building situated in any wind area and any terrain category with a height of up to 70m.

The addition of an extra toothed-plate can increase the application possibilities even further.

8 Conclusions and recommendations

8.1 Conclusions

The initial costs for implementing a demountable floor system are higher than the costs for a regular floor system due to the extra required design work, the use of non-standard building elements and the deviation from regular building practice. However, the final costs of the floor system will be lower than the initial costs if the floor system is reused. Moreover, when demountable floors become more regularly used, the material and fabrication costs will decrease. In addition, by adopting a well-considered design, the additional costs can partly be reduced. This can be achieved by assuring a simple and straightforward building procedure due to which the time required for and the possibility of mistakes during the assembly, disassembly and reassembly are reduced.

The requirements that can be set for demountable connections which are to be implemented in demountable floor systems are:

- The connections must be fully demountable;
- The connections should be easy;
- The connections should be made with the same type of connectors;
- The connections must be able to resist the applied loads;
- The connections must accommodate diaphragm action of the floor surface;
- The connections must be protected from fire to assure the fire safe time of the floor is retained;
- The connections must be insulated to assure the required thermal- and sound insulation between the floor levels.

A timber-concrete composite floor slab can be used in a demountable floor system. The TCC floor slab spanning 10.8m having a concrete slab of 80mm thick connected to timber glulam beams with a height of 430mm having a centre-to-centre distance of 600mm using 16 notched connections per beam, which have a length of 150mm, a depth of 40mm and a width of 140mm including a screw with a diameter of 12mm, can withstand an imposed variable load of 3.5 kN/m^2, including light separation walls, and a permanent load comprising the self-weight of the slab, the services, the top floor and the ceiling. This TCC floor element has a 2x higher bending stiffness compared to a full timber rib floor designed for the same span and load. The bending stiffness is only 1.1x smaller than the bending stiffness of a concrete hollow core slab floor designed for the same span and load.

The connection from the TCC floor slab to the L-section edge beam is made of one toothed-plate connector type C11 per timber beam and a compression bolt M20 tightened to press against the centre of the concrete slab at the position of the timber beam. In this configuration the wind load and floor slab loading can be transferred through shear forces in all directions at the toothed-plate with a maximum of $9.0\ kN$ and a compressive force of $7.7\ kN$ at the compression bolt. For the instalment of the floor slab, only 4 nuts have to be installed and tightened and 6 nuts have to be turned to press against the concrete slab, which results in a fast erection of the floor slabs.

The connections between the TCC floor slabs and between the sides of the floor slabs to the steel angle section are fully demountable and able to withstand an acting shear force of $13.3\ kN$ and a tensile force on the side of the concrete of $11.3\ kN$. Every side of the floor slab has 6 connections each comprising of 4 bolts, 2 per floor slab. This results in the installation of 24 bolts per floor slab, which takes up quite some time. Therefore, the total erection time of the floor system is increased.

The steel L-section edge beam spans 1800mm, has a total height of 528mm, a total width of 218mm and a web and flange thickness of 18mm. This beam can withstand an eccentric load caused by the floor slab described above, induced as three point loads of each $28.4\ kN$.

The dimensions of the floor system can be chosen in such a way that the floor system can be implemented in a building with a height of up to 70m. This is applicable on many combinations of the building length (10m to 70m) and width (6m to 12.6m).

The found differences when using a timber-concrete composite floor slab compared to a hollow core slab floor and a full timber rib floor with a span of 10.8m are shown in Table 8-1.

	TCC	**Timber**	**HCS**
ECI [€/m²]	2.90	2.53	3.66
Floor height [mm]	510	637	260
Weight [kg/m²]	243.8	59	371.7
Material costs [€/m²]	318	-	101
Floor slabs per trucks	5	4	22
Fire resistance	Medium	Low	High
Sound insulation	Medium	Low	High
Vibration comfort	Medium	Low	High

Table 8-1: Differences TCC compared to Timber rib floor and HCS

8.2 Recommendations

The developed floor system in this thesis is demountable and possibly also reusable. Because the reuse of a floor system decreases raw material use and reduces energy consumption and harmful emissions required for the fabrication of the floor system, it is assumed that the floor system is less damaging for the environment. To truly determine the difference in environmental performance compared to regular floor systems, a life cycle assessment should be performed. To make this life cycle assessment as realistic as possible, research should first be conducted to determine if the system can be reused and the number of times it can be reused.

In this thesis, only a general material cost comparison was made between the developed reusable floor system and a standard hollow core slab floor. To thoroughly determine the difference in costs, an extensive cost analysis should be performed. This analysis should include much more than only the material costs. For instance, transportation costs, labour costs and the costs made to set smaller concrete fabrication tolerances can be included. Moreover, the reduction in costs originating from the reuse of the floor system should be included.

The connection between the head end of the floor slab and the edge beam is designed to be (dis)assembled efficiently. The design of the connection between the floor slabs and between the side of the floor slab and the edge beams is fully demountable but also rather labour intensive. Fastening and loosening all the required bolts takes up quite some time which will increase the labour costs for erecting the floor system. Efficient demountable connections should be designed for the slab-slab and side slab-beam positions to increase the efficiency of the total (dis)assembly procedure of the floor system.

The toothed-plate connector used in the connection is adhesively bonded to the steel edge beam. The teeth are pressed into the timber by the self-weight of the floor slab and, if required, by pressing onto the floor slab. Shear force transfer occurs only through the toothed-plate, no bolt is present like for regular use of toothed-plate connectors. The shear force travels through the adhesive to the edge beam. Experiments should be performed to confirm the behaviour of the connection assumed here and to determine the adhesive type to be used.

For determination of the feasibility of the connections, the used hand calculations for structural verification are sufficient. To make a further optimization of the connections, a finite element analysis should be performed. This finite element model can also be used to make a more comprehensive parametric study. The structural verifications done in the research are performed only for the use phase of the building. For the construction phase, similar verifications should be performed.

At the present, a lot of effort is required to design a well-functioning demountable floor system. This is partly the case because in the Netherlands, only a few projects and developments are made in this field. Moreover, no design guidance regarding demountability is available as of yet. To promote the implementation of a design approach with a bigger emphasis on reuse, design guidance should be developed. Moreover, different design options should be available for the designer to choose from or to be inspired by. Therefore, different types of demountable and reusable floor systems should be made. The design of these different floor systems should be made for different loading conditions to increase the applicability of the systems.

In the multi-criteria analysis, the choice was made to use a TCC floor slab for the development of the demountable floor system. This resulted in a floor slab with a rather big system height, $510\ mm$. The big height results in an increase in the required building height, and thus also an increase in material costs for the remainder of the structural components and, most importantly, the façade. Therefore, research should be done into the development of different demountable floor systems with different floor slab types resulting in a smaller system height.

The performed MCA is not completely objective. During the procedure of the MCA, general uncomprehensive calculation and determination methods were used to determine the input data of each floor system. Moreover, the rating scale of the criteria was based on the input of the floor systems. Several parts in a MCA are always dependent on the person performing the MCA. An attempt is made to reduce the subjectivity of the MCA as much as possible by using a fixed rating scale and by determining the influence of different criteria weights. If the assumptions made in the MCA are followed, the same outcome will be generated. But if different assumptions are made, the outcome might differ to the one found here. A different type of MCA could be performed to determine different floor slab types to be used in the design of a demountable floor system.

Many variables in the TCC floor slab design were determined by making assumptions and substantiated choices. This was done to not further extend the already elaborate design and verification of the floor slab. Design guidance for timber-concrete composite floor slabs should be developed so that the design will become less elaborate and to assure that correct values for properties are used in the verifications.

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9 Appendices

9.1 Appendix A - MCA

9.1.1 Input MCA

Floor type	Length max [mm]	Width max [mm]	Thickness 1 [mm]	Weight 1 [kg/m²]	Thickness 2 [mm]	Weight 2 [kg/m²]	Thickness 3 [mm]	Weight 3 [kg/m²]
HCS	18000	1200	260	371.1	200	297.3	200	297.3
Solid	12000	1200	300	739.3	260	640.1	200	492.8
TT	22000	2400	400	317.9	400	317.9	400	317.9
Steel-concrete	95000	915	-	-	-	-	300	335.4
Cofradal	7500	1200	-	-	-	-	260	249.7
Slimline	16200	2400	433	299.1	363	305.2	333	296.6
Quantum	11000	2400	341	207.4	301	190.3	291	184.2
Star-Frame	7200	2400	-	-	-	-	340	71
HCS-box	12000	200	-	-	320	68	280	63
HCS-surface	20000	2400	-	-	320	48	280	45
Rib	20000	2400	637	59	487	41	437	36
CLT	16000	2400	320	144	260	117	240	108

Table 9-1: Input floor slabs MCA

Material	Density [kg/m³]	Modulus of elasticity [kN/m²]
Steel	7850	$2.1*10^8$
Concrete	2400	$3.3*10^7$
Reinforced concrete	2500	$3.3*10^7$
Rockwool	40	
OSB	650	$3.5*10^6$
Softwood, Spruce, grain direction	420	$1.1*10^7$
Softwood, Spruce, perpendicular	420	$3.7*10^5$
LVL, Spruce	460	
CLT	450	

Table 9-2: Density and modulus of elasticity used materials

9.1.2 Rating MCA

Criterion	Sub-criterion	HCS	Solid	TT	Steel-concrete	Cofradal	Slimline	Quantum	Star-frame	HCS-box	HCS-surface	Rib	CLT
Environmental impact	**ECI**	2	1	3	-	-	1	2	-	-	-	3	1
Transportation	**Amount of trucks**	4	4	3	-	-	2	3	-	-	-	1	3
Connection possibility	**Connection possibility**	3	3	3	-	-	5	5	-	-	-	5	5
Lightweight	**Lightweight**	2	1	2	-	-	3	3	-	-	-	5	4
Building decree	**Sound insulation**	4	5	3	-	-	3	2	-	-	-	1	3
	Vibration performance	5	5	4	-	-	4	3	-	-	-	2	2
	Fire performance	4	4	4	-	-	4	2	-	-	-	2	5
Flexibility	**Free span length**	1	1	1	-	-	1	1	-	-	-	1	1
Ease of (Dis)assembly	**Amount of elements**	0	0	1	-	-	1	1	-	-	-	1	1

Table 9-3: Rating functional unit 1

Criterion	Sub-criterion	HCS	Solid	TT	Steel-concrete	Cofradal	Slimline	Quantum	Star-frame	HCS-box	HCS-surface	Rib	CLT
Environmental impact	ECI	3	1	3	-	-	1	3	-	5	5	4	1
Transportation	Amount of trucks	4	3	2	-	-	3	3	-	3	3	2	3
Connection possibility	Connection possibility	3	3	3	-	-	5	5	-	5	5	5	5
Lightweight	Lightweight	2	1	2	-	-	3	3	-	5	5	5	4
Building decree	Sound insulation	3	5	3	-	-	4	2	-	2	2	1	3
	Vibration performance	4	5	4	-	-	4	3	-	2	2	1	2
	Fire performance	4	4	4	-	-	4	1	-	3	3	2	5
Flexibility	Free span length	1	1	1	-	-	1	1	-	1	1	1	1
Ease of (Dis)assembly	Number of elements	0	0	1	-	-	1	1	-	0	1	1	1

Table 9-4: Rating functional unit 2

Criterion	Sub-criterion	HCS	Solid	TT	Steel-concrete	Cofradal	Slimline	Quantum	Star-frame	HCS-box	HCS-surface	Rib	CLT
Environmental impact	ECI	3	1	3	2	2	1	3	2	5	5	4	1
Transportation	Amount of trucks	4	4	2	2	3	3	3	2	3	3	1	3
Connection possibility	Connection possibility	3	3	3	1	3	5	5	5	5	5	5	5
Lightweight	Lightweight	2	1	2	2	3	3	4	4	5	5	5	4
Building decree	Sound insulation	3	5	3	4	4	3	2	1	2	2	1	3
	Vibration performance	4	5	4	4	4	4	3	2	2	2	1	2
	Fire performance	5	4	5	4	3	5	2	2	3	3	2	5
Flexibility	Free span length	1	1	1	0	0	1	1	0	1	1	1	1
Ease of (Dis)assembly	Number of elements	0	0	0	0	0	0	0	0	0	0	0	0

Table 9-5: Rating functional unit 3

9.1.3 Dimensioning floor systems for functional unit

The floor systems have to be designed to satisfy the functional unit (FU). In Table 9-6, Table 9-7 and Table 9-8 the specification are given for FU1, FU2 and FU3 respectively. The required dimensions of the slabs are preferably found in load-span tables provided by manufacturers. When no tables are available the dimensions are calculated by determining whether the deflection is smaller than or equal to the SLS deflection criterion, $w_{max} = \frac{l}{300}$. The deflection is determined by assuming a simply supported beam loaded by a uniformly distributed load: $w = \frac{5*ql^4}{384*EI}$. Iterations are done if the deflection is much smaller than the limit value. The loading used for obtaining the dimensions is the self-weight of the slab and an additional load of 4 kN/m^2. The safe fire time for the obtained dimensions is looked up in data from manufacturers.

Floor type	Thickness slab [mm]	Weight slab [kg/m^2]	IPE profile	Fire safe time [min]	Indication
HCS	260	371.1	400	120	HCS260
Solid	300	739.3	450	120	S300
TT-slab	400	317.9	360	60	TT40/14
Steel-concrete	-	-	-	-	-
Cofradal	-	-	-	-	-
Slimline	433	299.1	360	120	SL433
Quantum	341	207.4	360	15	Q341
Star-Frame	-	-	-	-	-
HCS-box	-	-	-	-	-
HCS-surface	-	-	-	-	-
Rib	637	59	330	15	KRT2400x37-5x75x600
CLT	320	144	330	90	L8s-2-320

Table 9-6: Specifications floor system functional unit 1

Floor type	Thickness slab [mm]	Weight slab [kg/m^2]	IPE profile	Fire safe time [min]	Indication
HCS	200	297.3	360	60	HCS200
Solid	260	640.1	400	90	S260
TT-slab	400	317.9	360	60	TT40/14
Steel-concrete	-	-	-	-	-
Cofradal	-	-	-	-	-
Slimline	363	305.2	360	120	SL363
Quantum	301	190.3	330	15	Q301
Star-Frame	-	-	-	-	-
HCS-box	320	68	300	30	HCS-B320
HCS-surface	320	48	300	30	HCS-S320
Rib	487	41	300	15	KRT2400x37-5x57x450
CLT	260	117	300	90	L7s-260

Table 9-7: Specifications floor system functional unit 2

Floor type	Thickness slab [mm]	Weight slab [kg/m^2]	IPE profile	Fire safe time [min]	Indication
HCS	200	297.3	330	60	HCS200
Solid	200	492.8	360	60	S200
TT-slab	400	317.9	330	60	TT40/14
Steel-concrete	300	335.4	330	30	ComFlor210
Cofradal	260	249.7	330	30	Cofradal260
Slimline	333	296.6	330	120	SL333
Quantum	291	184.2	330	15	Q291
Star-Frame	340	71	300	15	SF340
HCS-box	280	63	300	30	HCS-B280
HCS-surface	280	45	300	30	HCS-S280
Rib	437	36	300	15	KRT2400x37-5x51x400
CLT	240	108	300	90	L7s-2-240

Table 9-8: Specifications floor system functional unit 3

The used concrete hollow core slabs are determined from [95-97]. For HCS260, 20 prestressing strands are used and for the HCS200, 12 strands are used. Since the same HCS is used for FU2 and FU3, it can be stated that the HCS in FU3 is over-dimensioned.

For the solid slab floor, the required height is determined by using the deflection limit. For S300, 24 prestressing strands are used, for S260, 20 strands and for S200, 16 strands are used [97]. The smallest possible TT-slab is used, TT40/14 [98]. At a span of 10.8m this slab can withstand an additional load of 6 kN/m^2 so it is quite over-dimensioned. Per rib 14 prestressing strands are used.

The steel-concrete floor used is the ComFlor210 [99], which has a deep deck steel plate with a thickness of 1mm. It is assumed that propping is used.

The Cofradal260 [100] is used with a steel plate thickness of 1.25mm.

The required IPE profile for the Slimline floor is determined with [54, 101]. The centre-to-centre distance between the profiles, and thus the number of used profiles, differs. The concrete slab always has a thickness of 75mm.

The required C-profile for the Quantum floor is determined with [102]. The concrete layer always has a thickness of 61mm.

For the Star-Frame floor, no load-span tables are available. Therefore load-span tables for steel frame floors, but not specifically for the Star-Frame floor, found in [103] are used.

The satisfactory HCS-box and HCS-surface elements are found in [6]. They are dimensioned to satisfy a fire safe time of 30 min.

The required dimensions of the Rib floor are determined with an online programme called ripaschuif [104].

The built-up of CLT floors is found in [58, 105]. To determine the required layup, the deflection criterion is used.

The required IPE profile for the supporting beams is determined by finding the required profile to satisfy the moment capacity, shear capacity and deflection limit and taking the biggest required profile. For IPE profiles I300 to I750, the loading at which the deflection criterion is exactly met is determined, see Table 9-9. Since the weight of the slab and the beam, and the additional load are known, the required profile can be determined.

To verify the profiles for the moment and shear capacity, first the cross-section class is determined which gives the allowed analysis method. A plastic analysis is done for class 1 and 2, an elastic analysis is adopted for class 3. The moment resistance is calculated with:

$$M_{Rd} = \frac{W_i f_y}{\gamma_{M0}}$$

In which

W_i	is the section modulus;
$f_y = 235\ MPa$	is the yield strength of the profile;
$\gamma_{M0} = 1.0$	is the partial safety factor.

The elastic section modulus is taken from tables. The plastic section modulus is calculated in a simplified manner, neglecting the extra material in the corner radius from the web to the flange:

$$W_{y,pl} = 2 * A_{fl} a_{NC-NC(fl)} + 2 * \frac{1}{2} A_w a_{NC-NC(w)}$$

In which

A_i	is the area of one flange or the web;
$a_{NC-NC(i)}$	is the distance from the neutral axis of the profile to the neutral axis of the flange or to the neutral axis of the halve of the web;

The plastic shear resistance is calculated with:

$$V_{pl,Rd} = \frac{A_v (f_y/\sqrt{3})}{\gamma_{M0}}$$

In which

A_v is the shear area of the profile.

The elastic shear resistance is calculated with:

$$V_{el,Rd} = \frac{f_y}{\sqrt{3}\gamma_{M0}}$$

The acting moment and shear force are calculated for a simply supported beam and the required profile can be determined.

IPE	q(max) [kN/m]	Cross-section class	W_{pl} x10^6 [m3]	W_{el} x10^6 [m3]	A_v x10^4 [m2]	M_{Rd} [kNm]	V_{Rd} [kN]
300	21.4	1	602.1	557	25.7	141.5	348.3
330	30.1	1	762.8	713	30.8	179.2	417.9
360	41.7	1	973.7	904	35.1	228.8	476.3
400	59.2	1	1238.3	1156	42.7	291.0	579.8
450	86.4	1	1623.9	1500	50.8	381.6	689.6
500	123.4	1	2107.3	1628	60.4	495.2	818.8
550	171.9	1	2662.2	2441	71.9	625.6	975.9
600	235.8	1	3376.1	3069	83.8	793.4	1137.0
750	409.4	1	4969.0	4246	97.0	1167.7	1316.1

Table 9-9: Specifications IPE profiles

9.1.4 Criteria

Environmental impact

To calculate the environmental impact, firstly the ECI of the used materials is calculated using the DuCo tool, these can be found in Table 9-10. Multiplex is used as LVL because LVL was not available in the national environmental database and both products are made of thin veneers glued together to form a plate material. Hereafter, the kilograms of the used materials are determined, multiplied with the ECI of the material and added to obtain the ECI for the floor system. Only the slabs and beams, not for instance the floor finish, are regarded. The floor finish will most likely be equal, regardless of the used structural floor. The rating scale used can be seen in Table 9-11 and in Table 9-12 the ECI per floor system and the corresponding rating can be found.

It can be seen that the ECI reduces and the rating increases when the span of the slab decreases. This occurs because for a lower span, less material is required for the slabs. This results in a lower load on the beams and thus possibly a smaller IPE profile. The ECI of the TT-slab increase because the same slab is used in all three cases.

Used materials	ECI [€/kg]
Reinforcing steel	0.0788
Prestressing steel	0.0788
Profiles steel	0.0327
Concrete C30/37	0.0056
Pine laminated	0.0379
Pine planks	0.0059
OSB	0.0310
Multiplex	0.1090
Rockwool	0.0960

Table 9-10: ECI per material

ECI [€/m^2]	Rating
$\mathbf{ECI \geq 4}$	1
$\mathbf{3 \leq ECI < 4}$	2
$\mathbf{2 \leq ECI < 3}$	3
$\mathbf{1 \leq ECI < 2}$	4
$\mathbf{0 \leq ECI < 1}$	5

Table 9-11: Rating scale ECI

Floor type	ECI 1 [€/m2]	Rating 1	ECI 2 [€/m2]	Rating 2	ECI 3 [€/m2]	Rating 3
HCS	3.66	2	2.78	3	2.78	3
Solid	6.02	1	5.24	1	4.18	1
TT	2.88	3	2.95	3	2.95	3
Steel-concrete	-	-	-	-	3.26	2
Cofradal	-	-	-	-	3.26	2
Slimline, raised	5.59	1	5.86	1	5.58	1
Quantum	3.27	2	2.72	3	2.57	3
Star-Frame, OSB	-	-	-	-	3.48	2
HCS-box	-	-	0.71	5	0.73	5
HCS-surface	-	-	0.59	5	0.62	5
Rib, LVL	6.85	1	4.84	1	4.32	1
Rib, laminated	2.53	3	1.85	4	1.71	4
Rib, solid	0.62	5	0.53	5	0.55	5
CLT	5.76	1	4.79	1	4.45	1

Table 9-12: ECI floor systems and rating

The HCS and TT-slab score mediocre because their self-weight is quite high. Since the solid slab weighs a lot more, the ECI is also much higher.

The Steel-Concrete and Cofradal floor systems are optimal for smaller spans, therefore, to achieve the used span, a lot of material is required. Also, rockwool is integrated in the Cofradal floor. This results in a rather high ECI. The Slimline scores low because of the high weight of the steel profiles and the addition of the raised floor system. If a steel-concrete deck was used, the ECI would increase. The Quantum floor uses a small concrete deck but quite some cold formed steel. Unfortunately, no ECI could be obtained for cold-formed steel so the ECI for hot rolled steel is used. Cold formed steel has a lower environmental impact and thus the actual ECI for this floor is lower than determined here. This floor system is also optimal for smaller spans.

The Star-Frame floor uses quite some cold formed steel. The same argument holds as for the Quantum floor, the actual ECI is lower. Also, this floor is optimal for smaller spans.

The HCS-box and HCS-surface floors have a very low ECI because solid wood has a very low weight and a low ECI. The ECI of the Rib floor is given for several possible timber (products); LVL, CLT and solid wood. It can clearly be seen that the use of LVL results in a higher ECI. This is due to the fact that LVL is made with a lot of glue, which is bad for the environment. The laminated Rib floor is used in the MCA. CLT also has a higher ECI due to the use of the glue in the production process, but it is much less than that of LVL.

To verify the computed ECI values of the floor systems, they are compared to some ECI values found in the MPGcalc tool. Table 9-13 shows for some floor systems both values.

Floor system	Own ECI	Floor MPGcalc	MPGcalc ECI
HCS200	€ 2.37	**HCS200 incl. joint filling**	€ 3.00
Slimline IPE300 c-t-c 800	€ 3.40	**Slimline IPE300 c-t-c 1200**	€ 4.13
Comflor210	€ 2.85	**SBV 210**	€ 3.60
HCS-surface 280, glulam	€ 1.71	**HCS 280, glulam + rockwool + 2 plasterboards**	€ 2.74
CLT L7s-240	€ 4.09	**CLT L5s-240 + rockwool + plasterboard**	€ 6.28

Table 9-13: Comparison self-computed ECI and MPGcalc ECI

The MPGcalc values for all the floor systems are higher partly because transport is included in the calculation of the ECI. All floors except the Slimline are category 3. This means they were not tested according to the SBK-review protocol which could indicate that errors were made during the calculation. In the calculation of the ECI values of the materials using the DUCO tool, the categories for the product information are not used. Therefor it is possible that the ECI would result in a higher value if they were used.

The values of the HCS200 are quite close, the MPGcalc value is higher which is partly originating from the addition of joint filling. The higher values for the composite systems don’t have a clear explanation. There is no elaboration in the program regarding the included materials, e.g. insulation, so it’s possible that a difference can be found here. Also, since several products are used, the inclusion of transportation could have a bigger influence than for single-product floors. Both timber floors include rockwool and plasterboard in the MPGcalc software which increases the ECI value. Also, the transportation has a bigger influence due to the multiple materials used.

The ECI values are accepted because the differences are explainable and transportation is regarded separately in the MCA.

Transportation

For the transportation criterion, the amount of trucks that are required to transport all the floor slabs and beams is determined. The assumption is made that the floor slabs and the steel beams can be transported in the same truck if space is available. First the required number of slabs and the amount that fit in one truck are calculated. The space left in the height is determined. If the left over space is negative, the rest of the slabs must be transported in an extra truck. Now the assessment is made whether the beams fit in the left-over height. Following, it is determined whether the rest of the slabs fit in the second truck, what space is left over and whether or not the beams fit in this left-over space. This is repeated until all the elements are 'placed' in a truck. The total number of required trucks and the corresponding rating can be found in Table 9-15. The rating scale can be found in Table 9-14.

Beams	Rating
$\# \boldsymbol{trucks} > \mathbf{8}$	1
$\mathbf{6} < \# \leq \mathbf{8}$	2
$\mathbf{4} < \# \leq \mathbf{6}$	3
$\mathbf{2} < \# \leq \mathbf{4}$	4
$\# \boldsymbol{trucks} \leq \mathbf{2}$	5

Table 9-14: Rating scale trucks

Floor type	# trucks 1	Rating 1	# trucks 2	Rating 2	# trucks 3	Rating 3
HCS	4	4	4	4	4	4
Solid	4	4	5	3	4	4
TT	6	3	7	2	8	2
Steel-concrete	-	-	-	-	8	2
Cofradal	-	-	-	-	5	3
Slimline	7	2	6	3	6	3
Quantum	5	3	6	3	6	3
Star-Frame	-	-	-	-	7	2
HCS- box	-	-	6	3	6	3
HCS-surface	-	-	6	3	6	3
Rib	10	1	8	2	9	1
CLT	5	3	5	3	5	3

Table 9-15: Amount of trucks required and rating

Connection possibility

In Table 9-16 the rating scale of the connection possibility is found. Floor systems which are already made with demountable connectors score highest, systems in which wet connection are used score mediocre. When, during the construction process, almost the entire floor area is made with cast in-situ concrete, big changes are required to change the system into a demountable one. Therefore, this type of floor scores lowest. The rating per floor system is given in Table 9-17.

Connection possibility	Rating
In-situ	1
Change possible	3
Made for disassembly	5

Table 9-16: Rating scale connection possibility

Floor type	Rating connection
HCS	3
Solid	3
TT	3
Steel-concrete	1
Cofradal	3
Slimline	5
Quantum	5
Star-Frame	5
HCS- box	5
HCS-surface	5
Rib	5
CLT	5

Table 9-17: Rating connection possibility per floor

Lightweight

The slabs and beams are observed here, just like for the calculation of the ECI. Their weights are combined and rated. For the steel-concrete composite floor, the weight of the steel and concrete is combined because a composite element is be present after the first instalment. In Table 9-18 the rating scale can be found and in Table 9-19 the weights and the corresponding ratings are documented.

Weight [kg/m^2]	Rating
$W \geq 400$	1
$300 \leq W < 400$	2
$200 \leq W < 300$	3
$100 \leq W < 200$	4
$0 \leq W < 100$	5

Table 9-18: Rating scale weight

Floor type	Weight 1 [kg/m^2]	Rating lightweight 1	Weight 1 [kg/m^2]	Rating lightweight 2	Weight 3 [kg/m^2]	Rating lightweight 3
HCS	384.6	2	310.2	2	310.0	2
Solid	754.3	1	655.1	1	507.6	1
TT	316.7	2	318.6	2	318.4	2
Steel-concrete	-	-	-	-	348.2	2
Cofradal	-	-	-	-	262.5	3
Slimline	254.9	3	256.7	3	256.5	3
Quantum	218.4	3	201.5	3	196.9	4
Star-Frame	-	-	-	-	108.0	4
HCS- box	-	-	77.6	5	73.9	5
HCS-surface	-	-	57.6	5	55.9	5
Rib	68.4	5	50.3	5	46.7	5
CLT	153.6	4	128.1	4	118.9	4

Table 9-19: Weight floor systems and rating

Building decree

Sound insulation

The only floor type which, known from practice, can suffice the condition stated in the building decree without extra measures is the cast in-situ solid concrete slab floor. Therefore, all the analysed floor systems will require the use of extra means to satisfy the requirement. It is assumed that the sound insulating performance and the required measures are linearly related to each other, therefore the amount of required measures is not considered separately.

The calculation of the sound insulation of a floor slab is very complex, it is not only dependent on the weight of the slab but also on the connections. The connections (including support detailing) can be designed in such a way, for instance using an intermediate layer, that little or no sound transfer occurs. The assumption is made that connections are not the weakest link for sound transfer so the sound insulating properties of a slab is equal to that of the entire floor area.

Because calculating the sound insulation is too extensive, an estimation of the sound insulation of the slabs is made by regarding the weight and the layup of the slab. The higher the weight, the bigger the sound insulating properties of the floor. A floor with a high weight will get a high rating, see Table 9-20. With layup the possible presence of cavities or open spaces in the longitudinal direction of the floor is meant. For instance, a hollow core slab has cavities and a TT-slab has open spaces between the ribs. If these are present, sound can travel through these voids which can result in sound disturbance between rooms in a building. The rating scale for the layup can be found in Table 9-21.

The total score per floor = 0.7 * rating weight + 0.3 * rating layup.

Weight [kg/m²]	**Rating**
$\mathbf{0 \leq W < 100}$	1
$\mathbf{100 \leq W < 200}$	2
$\mathbf{200 \leq W < 300}$	3
$\mathbf{300 \leq W < 400}$	4
$\mathbf{W \geq 400}$	5

Table 9-20: Rating sound insulation, weight

Layup	**Rating**
Open	1
Cavities	3
Solid	5

Table 9-21: Rating scale sound insulation, layup

Vibration performance

The vibration performance is determined by calculating the fundamental frequency of and the quasi-static load on the floor slab, see Table 9-23 for the results. Simple rules are available to calculate these values for a simply supported floor slab. In these rules the supporting conditions are included, therefore the beams are not regarded separately. Making a connection between the floor slabs will only increase the vibration performance due to the decreased freedom of deformation. This is difficult to take into account and therefore a simple and lower bound assumption is made that the vibration performance of one floor slab is equal to that of the entire floor area.

For an eigenfrequency lower than 3, the rating is 1, if it is higher than 3, the rating is 5. In the calculation of the eigenfrequency, the mass is not regarded separately, therefore it should be included in a different fashion. If the quasi-static load in [kN/m²] in which the mass of the floor is included is low, the easier vibrations will occur. Therefore, a reduction of the score of the eigenfrequency is used, see Table 9-22.

Load [kN/m²]	Rating
$\mathbf{4 \le q_{Q-S} < 5}$	- 1
$\mathbf{3 \le q_{Q-S} < 4}$	- 2
$\mathbf{2 \le q_{Q-S} < 3}$	- 3
$\mathbf{1 \le q_{Q-S} < 2}$	- 4

Table 9-22: Reduction rating scale floor mass

The eigenfrequency is calculated according to [106]

$$f_e = \sqrt{\frac{a}{\delta}}$$

In which

a is the vibration acceleration;

δ is the maximal deflection.

The vibration acceleration is determined from figure A.17 in [106] assuming the floor is simply supported and the mass is evenly distributed over the system:

$$a = 0.315\ m/s^2$$

The deflection is calculated by using a forget-me-not for a simply supported beam loaded by a distributed load:

$$\delta = \frac{5}{384}\frac{q_{Q-S} * l^4}{\sum E * I}$$

In which:

$l = 10.8\ /\ 9.0\ /\ 7.8\ \ m$;

$\sum E * I$ is the bending stiffness of the cross section;

q_{Q-S} is the quasi-static load on the floor slab in $[kN/m]$ (see below).

$$q_{Q-S} = \sum_{j\geq 1} G_{k,j} + \sum_{i \geq 1} \Psi_k * \Psi_{2,i} * Q_{k,i}$$

In which

$G_{k,j}$ is the permanent load;

Ψ_k is the correction factor for the instantaneous loading, equal to 1.0 for all loading combinations which are not used for creep calculations;

$\Psi_{2,i}$ is the correction factor for variable loadings, equal to 0.3 for office areas;

$Q_{k,i}$ is the variable load.

Substituting all these variables gives the general formula for all systems:

$$q_{Q-S} = \sum_{j\geq 1} G_{k,j} + \sum_{i \geq 1} 0.3 * Q_{k,i}$$

To ensure that the deflection calculations are not too extensive, several assumptions are done:

- For the Cofradal floor, it is assumed that the rockwool doesn’t contribute to the load bearing capacity of the cross section;
- The top floor of the Slimline is regarded as not structurally connected to the IPE profiles;
- The OSB top floor of the Star-Frame floor is assumed structurally connected to the steel frame;
- One HCS-box element with a width of 200mm is regarded in the calculation. The transverse boards are disregarded;
- The transverse boards in the HCS-surface elements are disregarded;
- In the Rib floor calculation, the transverse boards are disregarded.

Floor type	f_e 1 [Hz]	$q_{Q\text{-}S}$ 1 [kN/m²]	f_e 2 [Hz]	$q_{Q\text{-}S}$ 2 [kN/m²]	f_e 3 [Hz]	$q_{Q\text{-}S}$ 3 [kN/m²]
HCS	3.6	5.3	5.7	4.5	7.6	4.5
Solid	3.8	8.9	4.7	8.0	4.7	6.5
TT	5.6	4.7	8.1	4.7	10.7	4.7
Steel-concrete	-	-	-	-	5.9	4.9
Cofradal	-	-	-	-	5.1	4.0
Slimline	3.6	4.5	4.1	4.6	5.1	4.5
Quantum	4.6	3.6	5.3	3.5	6.4	3.4
Star-Frame	-	-	-	-	8.5	2.5
HCS- box	-	-	5.9	2.2	6.7	2.2
HCS-surface	-	-	3.6	2.0	4.1	2.0
Rib	7.1	2.1	6.8	2.0	7.6	1.9
CLT	4.0	3.0	4.6	2.7	5.5	2.6

Table 9-23: Results vibration calculations

Fire resistance

The rating is based on the required fire measures and the effort needed to implement these measures to obtain the fire safe time stated in the functional unit. The safe fire time can be determined for one slab. If all slabs have the same safe fire time, and the connections are designed in such a way that they are not the weakest parts, the entire floor area has the same safe fire time. The rating scale for the floor slabs is given in Table 9-24. If the standard safe fire time of the slab is at least 60 minutes, no measures are required and a rating of 5 is awarded. If the standard safe fire time is below 60 minutes, the required type of measures is regarded. Small internal measures are e.g. adding reinforcement, small external measures are e.g. adding a small amount of fire proof paint, big internal measures are e.g. increasing the thickness of a timber plate and big external measures are e.g. applying a big amount of fire proof cladding.

External safety measures are always required for steel beams. A distinction between beams is made by regarding the area of beam which has to be protected and thus the required amount of protection material. The rating scale for the beams is found in Table 9-25.

The final rating is determined by: total rating = 0.7 * rating measures + 0.3 * rating beam perimeter.

Measures	Rating
Big external	1
Big internal	2
Small external	3
Small internal	4
None	5

Table 9-24: Rating scale fire measures

Beam perimeter [m]	Rating
$P > 1.05$	1
$0.95 < P \leq 1.05$	2
$0.85 < P \leq 0.95$	3
$0.75 < P \leq 0.85$	4
$P \leq 0.75$	5

Table 9-25: Rating scale beam perimeter

Summary results building decree

Floor type	Rating weight 1	Rating layup 1	Rating sound 1	Rating weight 2	Rating layup 2	Rating sound 2
HCS	4	3	4	3	3	3
Solid	5	5	5	5	5	5
TT	4	1	3	4	1	3
Steel-concrete	-	-	-	-	-	-
Cofradal	-	-	-	-	-	-
Slimline	3	3	3	4	3	4
Quantum	3	1	2	2	1	2
Star-Frame	-	-	-	-	-	-
HCS- box	-	-	-	1	3	2
HCS-surface	-	-	-	1	3	2
Rib	1	1	1	1	1	1
CLT	2	5	3	2	5	3

Table 9-26: Rating sound insulation 1&2

Floor type	Rating weight 3	Rating layup 3	Rating sound 3
HCS	3	3	3
Solid	5	5	5
TT	4	1	3
Steel-concrete	4	5	4
Cofradal	3	5	4
Slimline	3	3	3
Quantum	2	1	2
Star-Frame	1	1	1
HCS- box	1	3	2
HCS-surface	1	3	2
Rib	1	1	1
CLT	2	5	3

Table 9-27: Rating sound insulation 3

Floor type	Rating f_e 1	Reduction $q_{Q\text{-}S}$ 1	Rating vibration 1	Rating f_e 2	Reduction $q_{Q\text{-}S}$ 2	Rating vibration 2
HCS	5	0	5	5	1	4
Solid	5	0	5	5	0	5
TT	5	1	4	5	1	4
Steel-concrete	-	-	-	-	-	-
Cofradal	-	-	-	-	-	-
Slimline	5	1	4	5	1	4
Quantum	5	2	3	5	2	3
Star-Frame	-	-	-	-	-	-
HCS- box	-	-	-	5	3	2
HCS-surface	-	-	-	5	3	2
Rib	5	3	2	5	4	1
CLT	5	3	2	5	3	2

Table 9-28: Rating vibration performance 1&2

Floor type	Rating f_e 3	Reduction $q_{Q\text{-}S}$ 3	Rating vibration 3
HCS	5	1	4
Solid	5	0	5
TT	5	1	4
Steel-concrete	5	1	4
Cofradal	5	1	4
Slimline	5	1	4
Quantum	5	2	3
Star-Frame	5	3	2
HCS- box	5	3	2
HCS-surface	5	3	2
Rib	5	4	1
CLT	5	3	2

Table 9-29: Rating vibration performance 3

Floor type	Rating measures 1	Rating beams 1	Rating fire 1	Rating measures 2	Rating beams 2	Rating fire 2
HCS	5	2	4	5	3	4
Solid	5	1	4	5	2	4
TT	5	3	4	5	3	4
Steel-concrete	-	-	-	-	-	-
Cofradal	-	-	-	-	-	-
Slimline	5	3	4	5	3	4
Quantum	1	3	2	1	4	2
Star-Frame	-	-	-	-	-	-
HCS- box	-	-	-	2	5	3
HCS-surface	-	-	-	2	5	3
Rib	1	4	2	1	5	2
CLT	5	4	5	5	4	5

Table 9-30: Rating fire performance 1&2

Floor type	Rating measures 3	Rating beams 3	Rating fire 3
HCS	5	4	5
Solid	5	3	4
TT	5	4	5
Steel-concrete	4	4	4
Cofradal	3	4	3
Slimline	5	4	5
Quantum	1	4	2
Star-Frame	1	5	2
HCS- box	2	5	3
HCS-surface	2	5	3
Rib	1	5	2
CLT	5	5	5

Table 9-31: Rating fire performance 3

Flexibility

The floor systems will get a bonus point if the maximum span length is bigger than or equal to 10.8m. In Table 9-32 the bonus points per floor system can be found.

Floor type	Bonus 'Span length' 1	Bonus 'Span length' 2	Bonus 'Span length' 3
HCS	1	1	1
Solid	1	1	1
TT	1	1	1
Steel-concrete	-	-	0
Cofradal	-	-	0
Slimline	1	1	1
Quantum	1	1	1
Star-Frame	-	-	0
HCS- box	-	1	1
HCS-surface	-	1	1
Rib	1	1	1
CLT	1	1	1

Table 9-32: Bonus 'free span length'

Ease of (dis)assembly

The floor system will receive a bonus point if the required number of elements is less than 100. In Table 9-33 the bonus points per floor system can be found.

Floor type	Bonus 'Elements' 1	Bonus 'Elements' 2	Bonus 'Elements' 3
HCS	0	0	0
Solid	0	0	0
TT	1	1	0
Steel-concrete	-	-	0
Cofradal	-	-	0
Slimline	1	1	0
Quantum	1	1	0
Star-frame	-	-	0
HCS- box	-	0	0
HCS-surface	-	1	0
Rib	1	1	0
CLT	1	1	0

Table 9-33: Bonus 'required elements'

9.1.5 Changing weights

The two tests regarding the manner in which the weights of the bonus points are used performed in paragraph 3.3 for functional unit 1 are shown here for functional unit 2 and 3.

Functional unit 1

Figure 9.1 and Figure 9.2 give the results of the tests, which are summarized in Table 9-34.

	First choice	Second choice
(1)	CLT/ HCS	Solid/ TT/ Slimline
(2)	HCS	CLT
(3)	CLT	HCS
(4)	CLT	Rib/ Slimline/ Quantum
(5)	CLT	Slimline
(6)	CLT	Rib

Table 9-34: Outcome change in usage bonus-points FU1

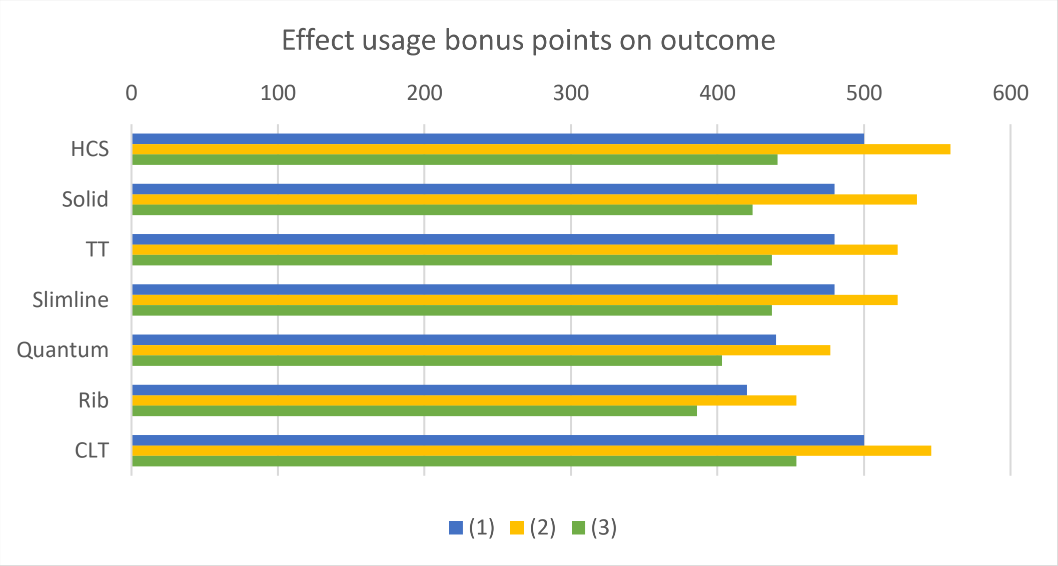

Figure 9.1: Effect of usage bonus points (1), (2) and (3), functional unit 1

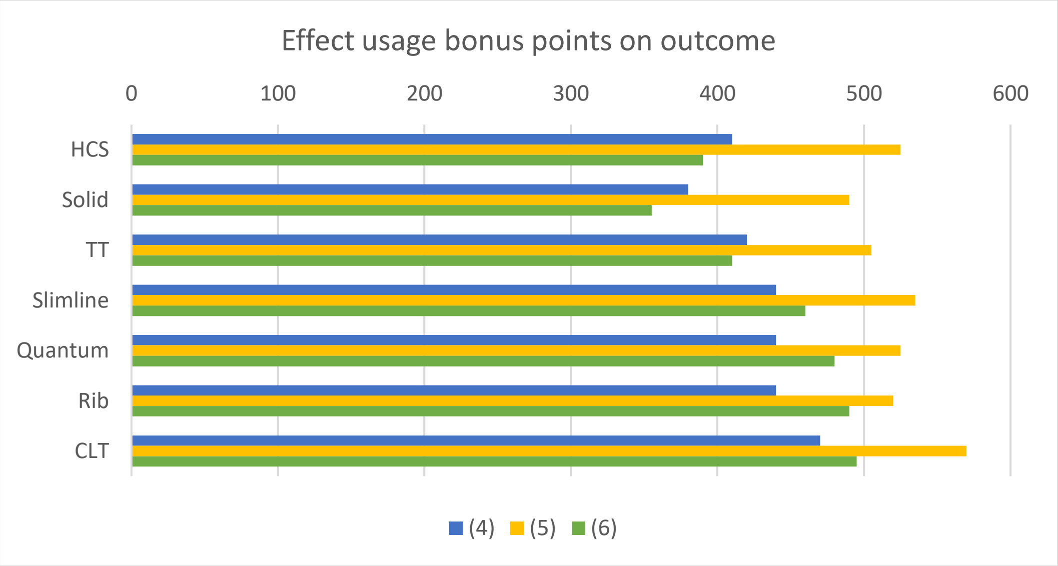

Figure 9.2: Effect of usage bonus points (4), (5) and (6), functional unit 1

Functional unit 2

Figure 9.3 and Figure 9.4 give the results of the tests, which are summarized in Table 9-35.

	First choice	Second choice
(1)	HCS-surface	HCS-box/ Slimline
(2)	HCS-surface	HCS-box
(3)	HCs-surface	Slimline
(4)	HCS-surface	HCS-box
(5)	HCS-surface	HCS-box
(6)	HCS-surface	HCS-box

Table 9-35: Outcome change in usage bonus points FU2

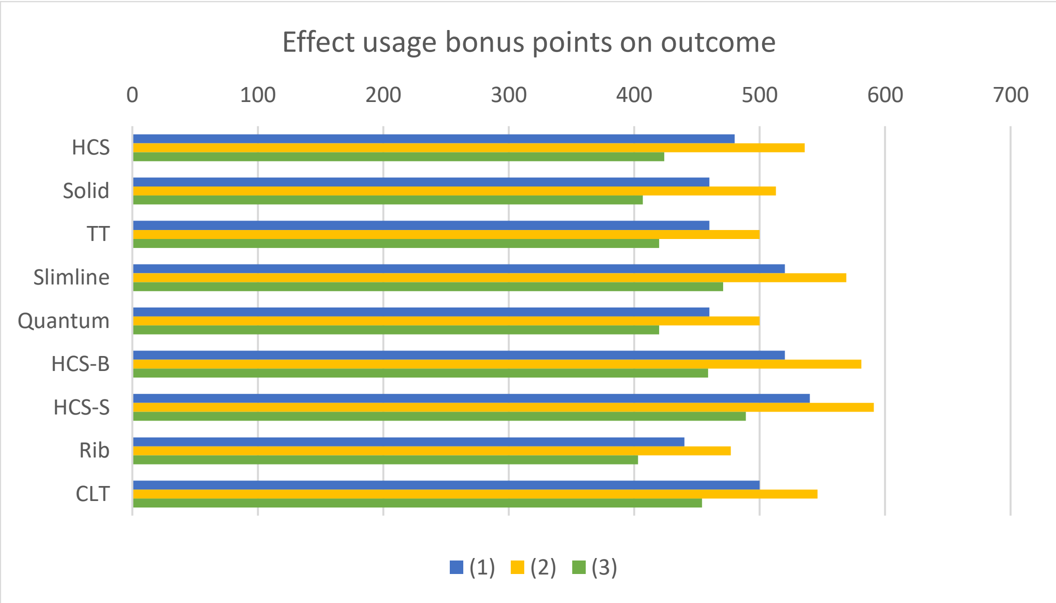

Figure 9.3: Effect of usage bonus points (1), (2) and (3), functional unit 2

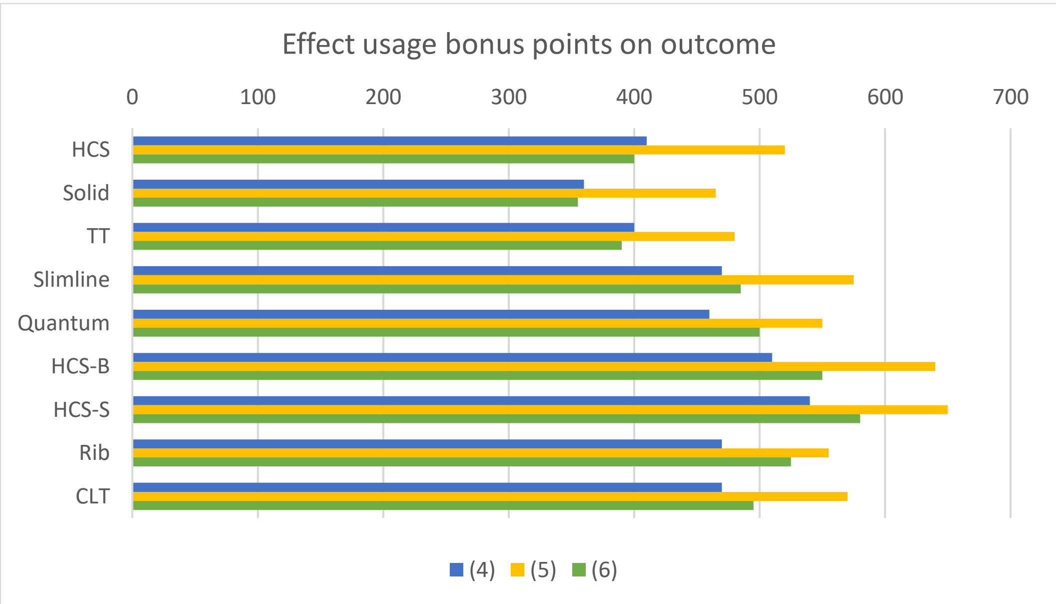

Figure 9.4: Effect of usage bonus points (4), (5) and (6), functional unit 2

Below four tables are shown which give the total score per floor system per functional unit for the different divisions of the weights given to the criteria. Score 1 = functional unit 1: L = 10.8m, Score 2 = functional unit 2: L = 9.0m and Score 3 = functional unit 3: L = 7.8m.
Four divisions of the weights are used to determine their influence:

- Option 1: all weights are equal;
- Option 2: the criteria flexibility, connection possibility and ease of (dis)assembly are given an equally high weight and the rest is given a weight of zero;
- Option 3: the building decree criteria are given an equally high weight and the rest is given a weight of zero;
- Option 4: the criteria flexibility, connection possibility and ease of (dis)assembly are given the highest weight, the criteria ECI, transportation and lightweight are given a mediocre weight and the building decree criteria are given the lowest weight.

Floor type	Score 1	Score 2	Score 3
HCS	500	480	500
Solid	480	460	480
TT	480	460	460
Steel-concrete	-	-	380
Cofradal 200	-	-	440
Slimline	480	520	500
Quantum	440	460	460
Star-Frame	-	-	360
HCS- box	-	520	520
HCS-surface	-	540	520
Rib	420	440	400
CLT	500	500	480

Table 9-36: Score for weight option 1

Floor type	Score 1	Score 2	Score 3
HCS	240	240	240
Solid	240	240	240
TT	300	300	240
Steel-concrete	-	-	60
Cofradal	-	-	180
Slimline	420	420	360
Quantum	420	420	360
Star-Frame	-	-	300
HCS- box	-	360	360
HCS-surface	-	420	360
Rib	420	420	360
CLT	420	420	360

Table 9-37: Score for weight option 2

Floor type	Score 1	Score 2	Score 3
HCS	780	660	720
Solid	840	840	840
TT	660	660	720
Steel-concrete	-	-	720
Cofradal	-	-	660
Slimline	660	720	720
Quantum	420	420	420
Star-Frame	-	-	300
HCS- box	-	420	420
HCS-surface	-	420	420
Rib	300	240	240
CLT	600	600	600

Table 9-38: Score for weight option 3

Floor type	Score 1	Score 2	Score 3
HCS	410	410	420
Solid	380	360	380
TT	420	400	380
Steel-concrete	-	-	270
Cofradal	-	-	360
Slimline	440	470	440
Quantum	440	460	450
Star-Frame	-	-	360
HCS- box	-	510	510
HCS-surface	-	540	510
Rib	440	470	420
CLT	470	470	440

Table 9-39: Score for weight option 4

9.2 Appendix B – Design example TCC floor slab

In this appendix a calculation example of a timber-concrete composite rib floor is given.

As of yet no design guidance is available in the Netherlands for timber-concrete composite structural elements. In the following calculations existing design methods, design guidance from the Eurocodes and information found in research papers are combined to verify a TCC floor slab. To prevent a text riddled with references, the mainly used documents are stated below:

- Eurocode 1990-1-1 [107];
- Eurocode 1991-1-1 [108];
- Eurocode 1992-1-1 [109];
- Eurocode 1994-1-1 [85];
- Eurocode 1995-1-1 [83];
- Draft for Eurocode 1995 – Structural design of timber-concrete composite structures – common rules and rules for buildings [90];
- Timber Engineering – Principles for design [110];
- NEN-EN 14080 [111].

When using the gamma method, the following values can be determined.
The connection efficiency coefficient

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1 s_1}{K_i l^2}}$$

$$\gamma_2 = 1$$

In which

s_1 is the spacing between the fasteners;
K_i is the slip modulus of the connection.

The bending stiffness of the built-up element

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

The stresses

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

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

The shear force acting in the plane of the connection

$$\tau_{conn} = \frac{\gamma_1 E_1 a_1 A_1}{b_2 (EI)_{ef}} V$$

The force acting in the connection

$$F_{conn} = \frac{\gamma_1 E_1 a_1 A_1 s_1}{(EI)_{ef}} V$$

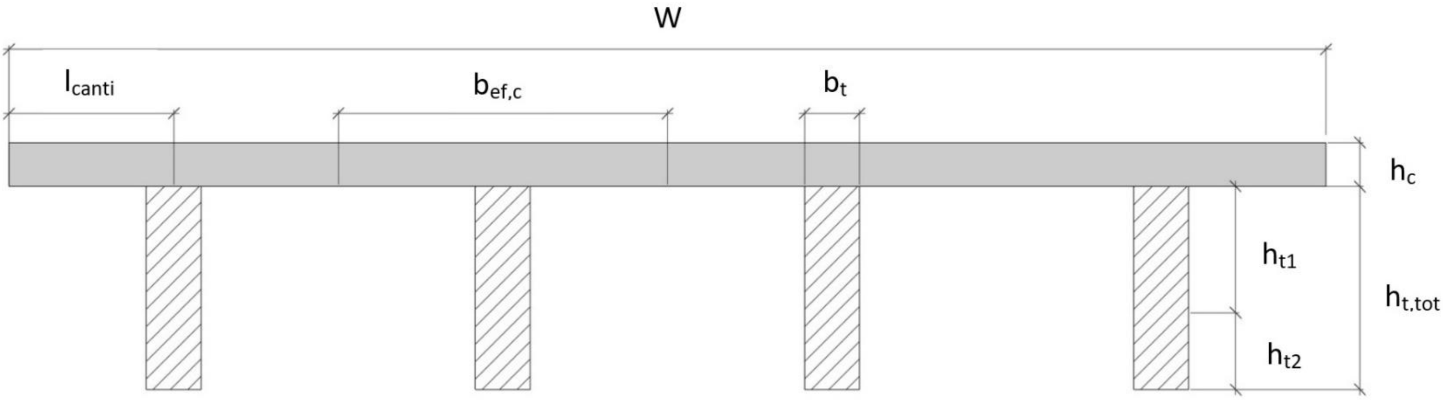

Figure 9.5: Rib floor, symbols

In Figure 9.5 a rib floor is shown including the used symbols to indicate the dimensions of the slab and its elements. The verification for the entire slab in the longitudinal (span) direction will be done on one T-element with a width of $b_{ef,c}$. In the transverse direction the entire cross section shown is regarded.

9.2.1 Input parameters

General

Because the floor is used indoors, the service class is 1 and the exposure class is XC1. The load duration class depends on the type of loading regarded in the design step.
Use category B (office areas) is used in the design of the floor system, this results in a consequence class 2 and reduction factors for the combination of loads: $\Psi_0 = 0.5, \Psi_1 = 0.5, \Psi_2 = 0.3$.

Dimensions timber, concrete and connection

The dimensions of the timber beams, shown in Figure 9.6 are:

- $h_{t,tot} = 430\ mm$
- $h_{t1} = h_2 = 310\ mm$
- $h_{t2} = h_3 = 120\ mm$
- $b_{t1} = b_{t2} = b_t = 140\ mm$
- $A_{t1} = A_2 = 43400\ mm^2$
- $A_{t2} = A_3 = 16800\ mm^2$
- $A_{t,tot} = 60200\ mm^2$

The c-t-c distance of the beams is 600 mm. This results in a cantilevering part at the sides of the slab of $l_{cant} = 300\ mm$.

The used dimensions of the concrete, shown in Figure 9.6, are:

$$h_c = h_1 = 80\ mm$$
$$w_{conc} = W_{slab} = 2400\ mm$$

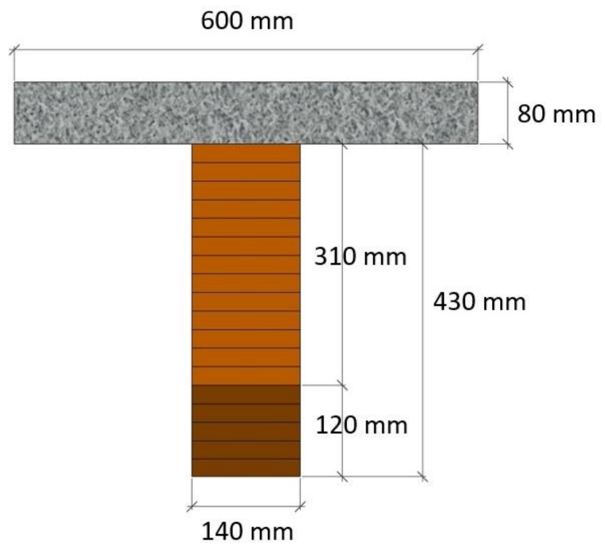

Figure 9.6: Dimensions timber and concrete

The effective width of the concrete slab

$$b_{ef,c} = b_0 + \sum b_{e,i}$$

In which

b_0 is the centre-to-centre distance between the outer fasteners, equal to 0 for calculations of building structures;

$b_{e,i} = \frac{L_e}{8} \leq b_i$ is the effective width of one halve of the flange.

In which

L_e is the distance between points of zero moment;

b_i is the geometrical width equal to the distance between the outer fastener to the centre between two webs. For calculations of building structures, it is equal to the distance between the centre of the web to the mid-point between two webs.

$$b_{ef,c} = \sum b_{e,i} = 2 * \frac{L_e}{8} = 2 * \frac{10900}{8} = 2725 \leq \sum b_i = 300 + 300 = 600\ mm$$

$$\rightarrow b_{ef,c} = b_c = b_1 = 600\ mm$$

$$A_c = A_1 = 48000\ mm^2$$

The dimensions of the notched connection and the screw, shown in Figure 9.7 are:

- $d_s = 12\ mm$
- $d_1 = 10.5\ mm$
- $h_{in\ c} = 60\ mm$
- $h_{in\ t} = 100\ mm$
- $h_n = 40\ mm$
- $b_n = 140\ mm$
- $l_n = 150\ mm$

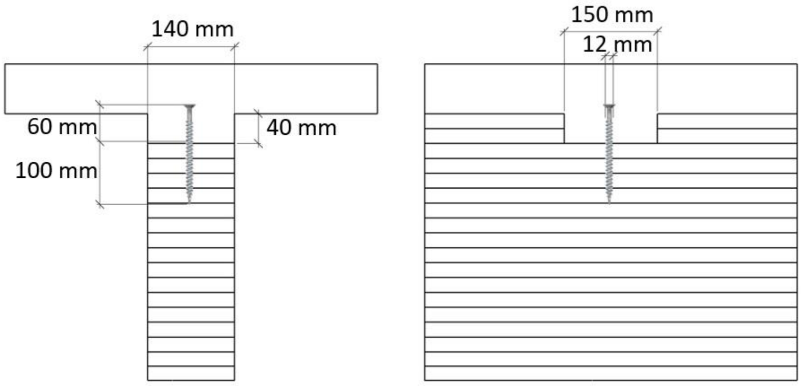

Figure 9.7: Dimensions of the notch

Loads and loading combinations

Several loads are acting on the floor element. For determination of loads in ULS, partial safety factors are used to ensure a safe design. For permanent loads $\gamma_G = 1.35$ and for variable loading $\gamma_Q = 1.5$.

The load combination used for ULS:

$$q_{ULS} = \max\left(\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \ ; \ \sum_{j\geq 1} \gamma_{G,j} \xi G_{k,j} + \gamma_{Q,i} Q_{k,i} \right) * b$$

Part 1 between the brackets will be prescriptive if the permanent loading is bigger than the variable loading and part 2 will be normative if the reverse is true.

To obtain the design loads in [kN/m^2], the above given equation has to be multiplied with the width of the regarded part. For design in the longitudinal direction, q_{ULS}, this load will be multiplied with the effective width of the concrete and for design in the transverse direction, $q_{ULS,T}$, it is multiplied with a unit width of 1m.

$$q_{ULS} = \max[1.35(0.5 + 2.4) + 1.5 * 0.5 * 3.5; (1.32 * 0.89)(0.5 + 2.3) + 1.5 * 3.5] * b_{ef,c}$$
$$= 5.2\ kN/m$$
$$q_{ULS,T} = \max[1.35(0.5 + 2.4) + 1.5 * 0.5 * 3.5; (1.32 * 0.89)(0.5 + 2.3) + 1.5 * 3.5] * 1$$
$$= 8.8\ kN/m$$

The acting bending moment and the shear force at its maximum and at L/4 in longitudinal direction can be determined by:

$$M_{Ed} = \frac{1}{8} * q_{ULS} * L^2 = 77.4\ kNm$$
$$V_{Ed} = \frac{1}{2} * q_{ULS} * L = 28.4\ kN$$
$$V_{\frac{1}{4L}} = \frac{1}{2} * V_{Ed} = 14.2\ kN$$

The load combination used for SLS short term:

$$q_{SLS} = \sum_{j\geq 1} G_{k,j} + Q_{k,i} = 1.7 + 2.1 = 3.8\ kN/m$$

The load combination used for SLS long term:

$$q_{SLS} = \sum_{j\geq 1} G_{k,j} + \Psi_{2,i} Q_{k,i}$$

Reinforced concrete

The concrete used is C30/37, class N. The partial safety factor for concrete is given by $\gamma_c = 1.5$. The material- and strength properties are:

$$\rho_{m,c} = 2400\ kg/m^3$$
$$E_{cm} = E_{conc} = E_1 = 33000\ N/mm^2$$
$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{30}{1.5} = 20\ N/mm^2$$
$$f_{ctmd} = \frac{f_{ctm}}{\gamma_c} = \frac{2.9}{1.5} = 1.9\ N/mm^2$$
$$f_{cm} = 38\ N/mm^2$$
$$\upsilon = 0.6\left(1 - \frac{f_{ck}}{250}\right) = 0.53$$
$$f_{v,c,d} = \frac{\upsilon f_{c,d}}{(\cot\theta + \tan\theta)} = \frac{0.53 * 20}{(\cot 30 + \tan 30)} = 4.6\ N/mm^2$$

The creep coefficient is determined with the use of the nomogram in Figure 9.8. A relative humidity of 50% is used and the age of the concrete at the time of loading is taken as 24 days. The fictive thickness is calculated with the circumference of the concrete exposed to drying, u :

$$h_0 = \frac{2A_c}{u} = \frac{2A_c}{2b_{ef,c} + h_c - h_t} = 84.2mm$$

Following the nomogram results in a creep coefficient of $\varphi(\infty, t_0) = 2.7$.

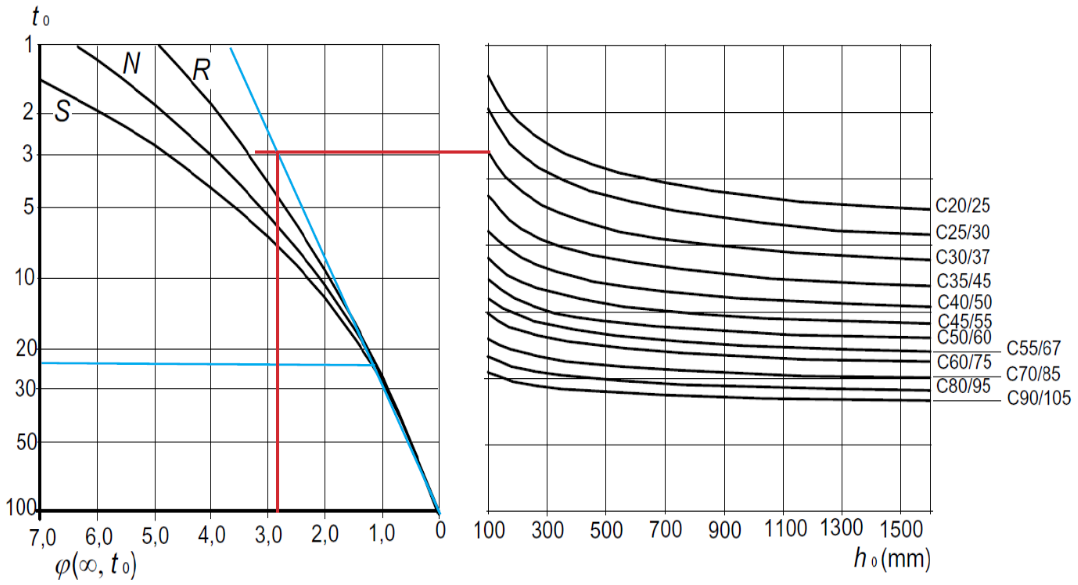

Figure 9.8: Used nomogram for creep coefficient, adapted from [109]

The modification factor for the effective creep coefficient is determined by using linear interpolation. This method may be used if the following conditions are met:

- $\frac{b_c}{b_t} \geq 5 \rightarrow \frac{600}{140} = 4.3 = 5\ \ Satisfied;$
- $1 \leq \frac{A_c}{A_t} \leq 5 \rightarrow \frac{48000}{60200} = 1, 1 \leq 0.8 \leq 5\ \ Satisfied.$

$$\psi_{conc}(\varphi = 3.5) = 2.6 - 0.8\gamma_1^2$$
$$\psi_{conc}(\varphi = 2.5) = 2.3 - 0.5\gamma_1^{2.6}$$
$$\psi_{conc}(\varphi = 2.8) = \psi_{conc}(\varphi = 2.5) + \big(\psi_{conc}(\varphi = 3.5) - \psi_{conc}(\varphi = 2.5)\big) * (2.8 - 2.5)$$
$$= 1.6$$

The effective modulus of elasticity

$$E_{conc,fin} = E_{1,fin} = \frac{E_{conc}(t_0)}{1 + \psi_{conc} * \varphi(\infty, t_0)} = \frac{33000}{1 + 1.7 * 2.8} = 6140\ N/mm^2$$

A minimal concrete cover has to be applied to the reinforcing steel, which is calculated with:

$$c_{nom} = c_{min} + \Delta c_{dev}$$

In which

Δc_{dev} is an allowance in design for deviation, taken as 5mm;

$c_{min} = \max[c_{min,b}; c_{mindur}; +\Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10\ mm]$ is the minimum concrete cover.

The minimum concrete cover with regards to bond for separated bars is the diameter of the bar, $c_{min,b} = \emptyset$.

The minimal concrete cover with respect to durability is found assuming structural class S4 without reduction: $c_{min,dur} = 15\ mm$. A reduction can be applied if the element has a slab geometry and/or if the concrete class used is ≥ C30/37, then $c_{min,dur} = 10\ mm$. All the delta terms in the above equations are to be taken as 0. This results in a minimum concrete cover calculation by:

$$c_{min} = \max[\emptyset; c_{min,dur}; 10]$$

And a nominal concrete cover, determined by:

$$c_{nom} = \max[\emptyset; c_{min,dur}; 10] + 5$$

The assumption is made that the used rebars have a diameter of $\emptyset \leq 10\ mm$. Now the required cover results in:

$$c_{nom} = \max[\emptyset; c_{min,dur}] + 5 = 10 + 5 = 15\ mm$$

The used reinforcing steel is made of type B500. The safety factor is $\gamma_s = 1.15$. The properties are listed below:

$$\rho_s = 7850\ kg/m^3$$
$$E_s = 200000\ N/mm^2$$
$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} = 435\ N/mm^2$$

Timber

The glue laminated beams used are made of two different types of lamellas. For the top part GL24h is used and the bottom part GL32h is used. The partial safety factor of glulam is $\gamma_m = 1.25$. The modification-, deformation- and height factors are equal, or calculated in the same manner, for the two types of wood used:

$$k_{mod,p} = 0.6$$
$$k_{mod,m} = 0.8$$
$$k_{def,p} = 0.6$$
$$k_{def,m} = 0.25$$

If the height under bending or the width under tension of glue laminated timber is less than 600mm, the characteristic bending and tensile strengths may be multiplied with the following factor:

$$k_h = min\begin{cases}\left(\frac{600}{h}\right)^{0.1} \\ 1.1\end{cases}$$

The properties of timber 1 and timber 2 are given in table Table 9-40.

	GL24h	GL32h	Unit
$\boldsymbol{\rho_{m,ti}}$	420	490	$[kg/m^3]$
$\boldsymbol{\rho_{0k,ti}}$	385	440	$[kg/m^3]$
$\boldsymbol{E_{ti}}$	11500	14200	$[N/mm^2]$
$\boldsymbol{k_{h,i}}$	1.07	1.10	
$\boldsymbol{f_{mk,ti}}$	24	32	$[N/mm^2]$
$\boldsymbol{f_{mk,ti}k_{h,i}}$	24	35.2	$[N/mm^2]$
$\boldsymbol{f_{md,ti}}$	15.4	22.5	$[N/mm^2]$
$\boldsymbol{f_{t0k,ti}}$	19.2	25.6	$[N/mm^2]$
$\boldsymbol{f_{t0k,ti}k_{h,i}}$	19.2	28.2	$[N/mm^2]$
$\boldsymbol{f_{t0d,ti}}$	12.3	18.0	$[N/mm^2]$
$\boldsymbol{f_{vk,ti}}$	3.5	3.5	$[N/mm^2]$
$\boldsymbol{f_{vd,ti}}$	2.2	2.2	$[N/mm^2]$
$\boldsymbol{f_{h0k}}$	27.8		$[N/mm^2]$
$\boldsymbol{f_{h0d}}$	17.8		$[N/mm^2]$
$\boldsymbol{f_{ax,k}}$	27.2		$[N/mm^2]$

Table 9-40: Timber properties

The effective modulus of elasticity of timber is calculated with:

$$E_{tim,fin} = E_{tim}\left[\frac{\%q_G}{1+\psi_{tim} * k_{def,p}} + \frac{\%q_Q}{1+\psi_{tim} * k_{def,m}}\right]$$

In which

E_{tim} is the mean modulus of elasticity of timber;

$\%q_i$ is the fraction of the total load consisting of either permanent or variable loading;

ψ_{tim} is the modification factor for the effective creep coefficient of timber which accounts for the influence of the composite action.

$$\psi_{tim} = \begin{cases} 1.0\ for\ t = \infty \\ 0.5\ for\ t = 3\ to\ 7\ years \end{cases}$$

$$E_{tim1,fin} = E_{tim1}\left[\frac{\%q_G}{1+\psi_{tim} * k_{def,p}} + \frac{\%q_Q}{1+\psi_{tim} * k_{def,m}}\right] = 11500\left[\frac{0.44}{1+1*0.6} + \frac{0.56}{1+1*0.25}\right]$$
$$= 8295\ N/mm^2$$

$$E_{tim2,fin} = E_{tim2}\left[\frac{\%q_G}{1+\psi_{tim} * k_{def,p}} + \frac{\%q_Q}{1+\psi_{tim} * k_{def,m}}\right] = 10242\ N/mm^2$$

Connection

The connection used in this example is a notched connection with a screw. The partial safety factor for connections is $\gamma_v = 1.25$. The modification- and deformation factors are:

$$k'_{mod,p} = 0.77$$
$$k'_{mod,m} = 0.89$$
$$k'_{def,p} = 1.2$$
$$k'_{def,m} = 0.5$$

The slip modulus of the notched connection

$$K_{ser} = K_u = 15 * 10^5 * s_{ef} = 15 * 10^5 * 0.59 = 888750\ N/mm$$

The effective slip moduli of the connections are calculated using the following equations:

$$K_{i,fin} = K_i \left[\frac{\%q_G}{1 + \psi_{conn} * k'_{def,p}} + \frac{\%q_Q}{1 + \psi_{conn} * k'_{def,m}} \right]$$

In which

K_i is the slip modulus for SLS or ULS;

ψ_{conn} is the modification factor for the effective creep coefficient of the connection which accounts for the influence of the composite action, found in [90] table 7.1.

$$\psi_{conn} = \begin{cases} 1.0 \quad for\ t = \infty \\ 0.65\ for\ t = 3\ to\ 7\ years \end{cases}$$

$$K_{ser,fin} = K_{u,fin} = K_i \left[\frac{\%q_G}{1 + \psi_{conn} * k'_{def,p}} + \frac{\%q_Q}{1 + \psi_{conn} * k'_{def,m}} \right]$$

$$= 888750 \left[\frac{0.44}{1 + 1 * 1.2} + \frac{0.56}{1 + 1 * 0.5} \right] = 507708\ N/mm$$

The characteristic tensile strength of the screw

$$f_{u,k} = 800\ N/mm^2$$

The connections are first spaced with an equal distance over the beams. Hereafter the spacing is optimized. In Figure 9.9 the indicated distances can be found.

$$l_1 = 350\ mm$$
$$l_2 = s_{min} = 500\ mm$$
$$l_3 = 525\ mm$$
$$l_4 = s_{max} = 870\ mm$$
$$s_{max} = 870 \leq 4 * s_{min} = 2000 \quad Satisfied$$
$$s_{ef} = 0.75 s_{min} + 0.25 s_{max} = 592.5\ mm$$

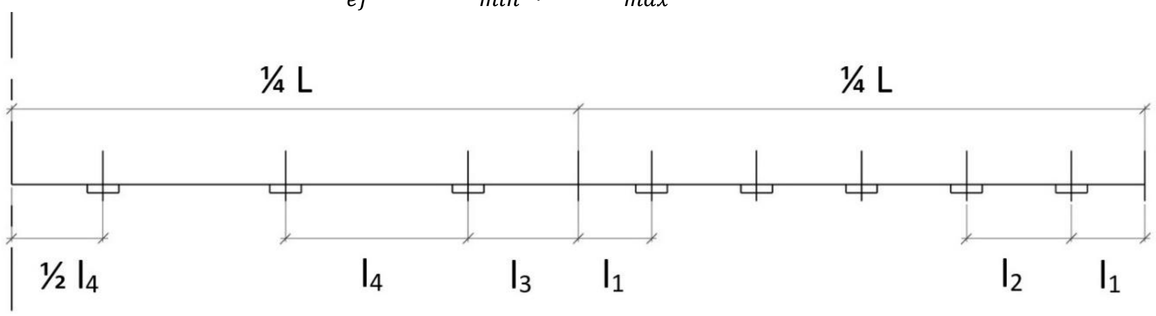

Figure 9.9: Notch distances

9.2.2 Ultimate limit state verification – short term

Several requirements should be satisfied by the concrete, the timber and the connection in the ultimate limit state.

Bending stiffness built-up beam

Cooperation factors

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1 s_1}{K_i l^2}} = 0.92$$

$$\gamma_2 = 1$$

$$\gamma_3 = 1$$

Position of the normal force centre with respect to the top of the element

$$NC = \frac{\sum_{i=1}^{3} \gamma_i A_i E_i a_{i-top}}{\sum_{i=1}^{3} \gamma_i A_i E_i} = 129\ mm$$

The distances between the mid-point of the timber or concrete, and the NC

$$a_1 = NC - \frac{1}{2} h_c = 89\ mm$$

$$a_2 = h_c + \frac{1}{2} h_{t1} - NC = 106\ mm$$

$$a_3 = h_{tot} - \frac{1}{2} h_{t2} - NC = 321\ mm$$

The effective bending stiffness

$$(EI)_{ef} = \sum_{i=1}^{2} (E_i I_i + \gamma_i E_i A_i a_i^2) = 4.7 * 10^{13}\ Nmm^2$$

Concrete

The concrete slab has to be verified in two directions, the longitudinal and transverse direction.

<u>Concrete stresses</u>

Regarding the system in the longitudinal direction, Figure 9.10, the stresses in the outer fibres due to the permanent and variable load can be calculated.

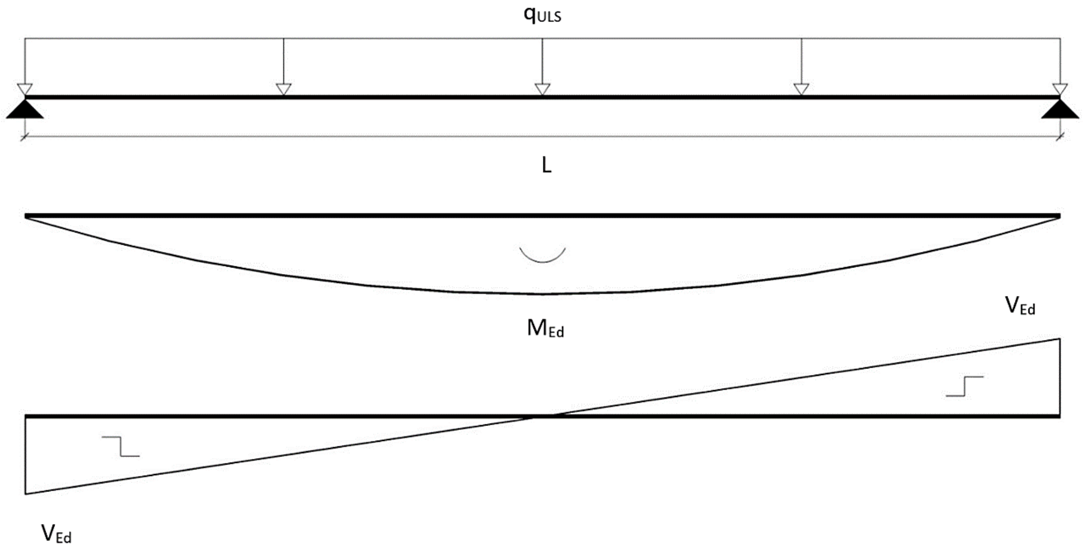

Figure 9.10: Rib floor, forces acting longitudinal

In the top fibre, the unity check (UC) for the concrete compressive strength should be satisfied:

$$\frac{\sigma_{c,top}}{f_{c,d}} \leq 1$$

In which

$\sigma_{c,top}$ is the acting stress in the top fibre of the concrete;
$f_{c,d}$ is the design compressive strength of the concrete.

In the bottom fibre, the tensile stress should not exceed the tensile strength, resulting in UC:

$$\frac{\sigma_{c,bot}}{f_{c,t,d}} \leq 1$$

In which

$\sigma_{c,bot}$ is the acting stress in the bottom fibre of the concrete;
$f_{c,t,d}$ is the design tensile strength of the concrete.

Concrete compressive stress due to axial force

$$\sigma_1 = \frac{\gamma_1 E_1 a_1 M}{(EI)_{ef}} = \frac{0.92 * 33000 * 89 * 77.4 * 10^6}{4.7 * 10^{13}}$$
$$= -4.46\ MPa$$

Concrete bending stress due to bending moment

$$\sigma_{m,1} = \frac{0.5 E_1 h_1 M}{(EI)_{ef}} = \frac{0.5 * 33000 * 80 * 77.4 * 10^6}{4.7 * 10^{13}}$$
$$= \pm 2.18\ MPa$$

Concrete stress in the top fibre

$$\sigma_{c,top} = -4.46 - 2.18 = -6.64\ MPa$$
$$\frac{\sigma_{c,top}}{f_{cd}} \leq 1 \rightarrow \frac{-6.64}{-20} = 0.33 \quad Satisfied$$

Concrete stress in the bottom fibre

$$\sigma_{c,bot} = -4.46 + 2.18 = -2.28\ MPa$$
$$\frac{\sigma_{c,bot}}{f_{ctd}} \leq 1 \rightarrow \frac{-2.28}{1.93} = -1.18 \quad Satisfied$$

The concrete is not cracked so no reduced cross section has to be taken into account.

Concrete reinforcement

The required longitudinal reinforcement is determined by regarding the longitudinal bending moment and shear force. The bending reinforcement is determined by regarding the moment distribution. The part of the acting bending moment which is taken by the concrete part is calculated with:

$$M_{Ed,conc} = \sigma_{m,conc} * W_{conc} = 2.28 * \frac{1}{6} * b_{ef} * h_c^2 = 1.4\ kNm$$

Figure 9.11 shows the regarded cross section and the acting internal forces and distances.

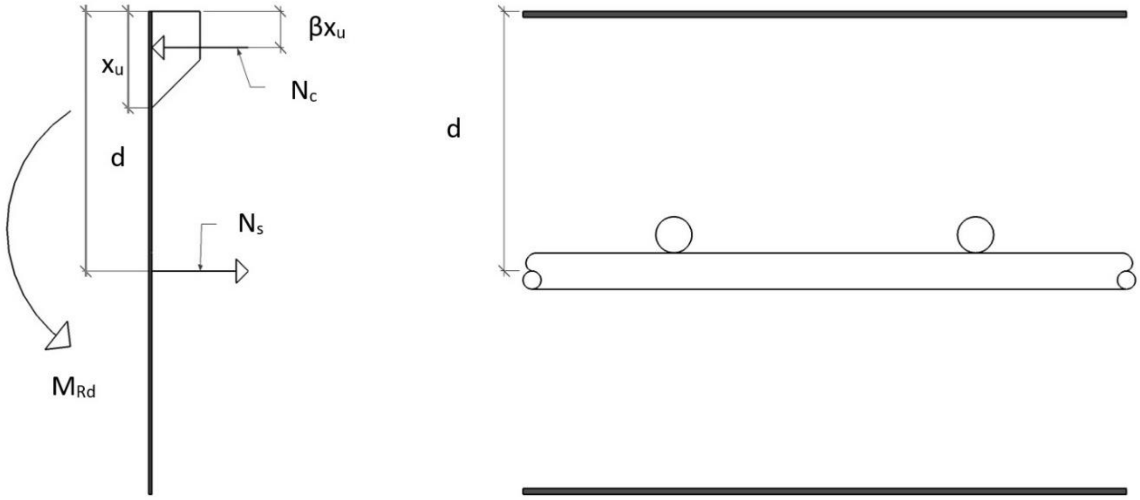

Figure 9.11: Concrete cross-section longitudinal

A rebar diameter is assumed in the longitudinal and transverse direction, $\emptyset_l$ and $\emptyset_t$ respectively, including a number of bars in the regarded cross section.

$$\emptyset_l = \emptyset_t = 6\,mm$$

$$A_{bar} = \frac{1}{4}\pi\emptyset_l^2 = \frac{1}{6} * \pi * 6^2 = 28.3\,mm^2$$

$$n = 8$$

The total area of reinforcement and the spacing of the bars is now

$$A_{sl} = A_{bar} * n = 28.3 * 8 = 226.2\,mm^2$$

$$s_l = \frac{b_{ef,c}}{n} = \frac{600}{8} = 75\,mm$$

The distance from the rebar to the top of the slab is depicted as

$$d = \frac{1}{2}h_c + \frac{1}{2} * \emptyset_l = 40 + 3 = 43\,mm$$

The force that can be resisted by the rebars can be calculated

$$N_s = A_{sl} * f_{yd} = 226.2 * 435 = 98.4\,kN$$

This force must be equal to the concrete compressive force: $N_c = \alpha * b_{ef,c} * f_{cd} * x_u$ for horizontal force equilibrium. Equating these results in a formula to determine the concrete compressive zone height

$$x_u = \frac{N_s}{\alpha * b * f_{cd}} = \frac{98.4 * 10^3}{0.75 * 600 * 20} = 10.9\,mm$$

The distance from the concrete compressive force, N_c, to the top of the slab is βx_u. β is a factor which is used to express this distance in a simplified manner, without many computations. The moment resistance can now be determined:

$$M_{Rd} = N_s(d - \beta x_u) = 98.4 * 10^3(43 - 0.39 * 10.9) = 3.8\,kNm$$

The bending moment resistance is verified with unity check;

$$\frac{M_{Ed,conc}}{M_{Rd}} = \frac{1.4}{3.8} = 0.37 \leq 1 \quad Satisfied$$

Shear reinforcement is not custom in plates. If the shear resistance without shear reinforcement is bigger than the acting shear stress, no shear reinforcement is required. The acting shear stress is calculated by

$$v_{Ed} = \frac{V_{Ed}}{bd} = \frac{28.4 * 10^3}{600 * 43} = 1.10\ MPa$$

The shear resistance without shear reinforcement:

$$v_{Rd,c} = \max\left[C_{Rd,c}k(100\rho_l * f_{ck})^{\frac{1}{3}}, v_{min}\right] = \max[1.48; 1.08] = 1.48\ MPa$$

In which

$C_{Rd,c} = \frac{0.18}{\gamma_c} = 0.12;$

$k = \max\left[1 + \sqrt{\frac{200}{d}}, 2.0\right] = \max[3.2; 2.0] = 3.2$

$\rho_l = \max\left[\frac{A_{sl}}{b*d}, 0.02\right] = \max[0.009; 0.02] = 0.02$

$v_{min} = 0.035 * k^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}} = 0.035 * 3.2^{\frac{3}{2}} * 30^{\frac{1}{2}} = 1.08\ MPa$

Unity check

$$\frac{v_{Ed}}{v_{Rd,c}} = \frac{1.10}{1.48} = 0.74 \leq 1$$

No shear reinforcement is required in the slab.

In the transverse direction, the rib floor can be schematised as a continuous beam on four supports with two cantilevering parts. Transverse reinforcement should be designed to resist in-plane shear and transverse bending. The acting shear stress should be smaller than the shear strength of the shear surface:

$$\frac{\tau_{Ed}}{\tau_{Rd}} \leq 1$$

To determine the in-plane shear reinforcement, two shear planes have to be checked. These are shown in Figure 9.12, denoted by; a-a and b-b. The required reinforcement is calculated for both planes and the biggest requirement is applied.

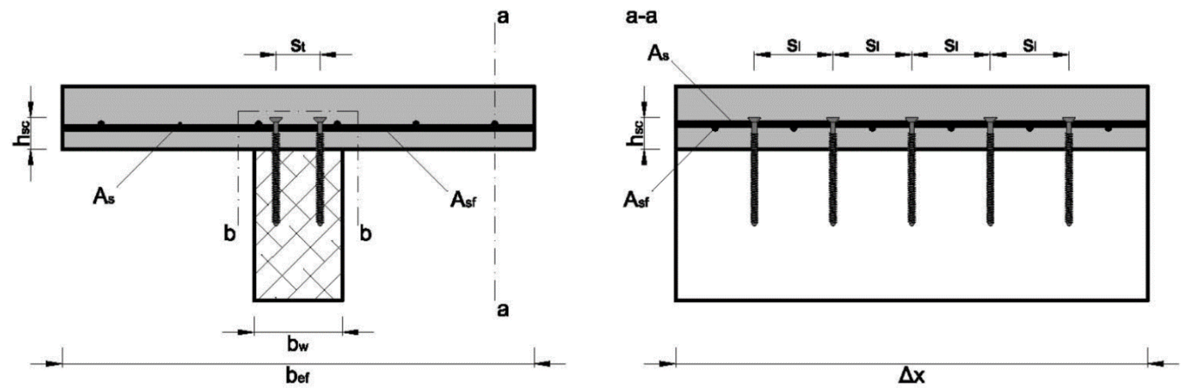

Figure 9.12: The connection between flange and web [90]

The acting longitudinal shear stress is calculated with:

$$\tau_{Ed} = \frac{\Delta F_d}{h_\tau \Delta x}$$

In which

ΔF_d is the change in longitudinal shear over a length Δx of the beam, taking into account the number of shear planes and the length of the shear surface.

Δx is usually taken as 1000 mm and is maximally half the distance between the point of zero moment and the point of maximum moment;

$$h_\tau = \begin{cases} h_f \;\; for\ plane\ a-a \\ 2h_{sc} + \emptyset \;\; for\ plane\ b-b, single\ row\ of\ fasternes \\ 2h_{sc} + \emptyset + s_t \;\; for\ plane\ b-b, fasteners\ arranged\ in\ paisr \end{cases}$$

is the length of the shear surface.

ΔF_d is calculated as the change in shear force due to stresses arising from the acting moment from x=0 to x=1000 mm.

$$M(\Delta x) = M(1000) = \frac{1}{2} * q_{ULS} * L * 1000 - \frac{1}{2} * q_{ULS} * 1000^2$$

$$= \frac{1}{2} * 5.2 * 10900 * 1000 - \frac{1}{2} * 5.2 * 1000^2 = 25.8\ kNm$$

$$\sigma_{tot}(1000) = \frac{\gamma_1 E_1 a_1 M(1000)}{(EI)_{ef}} + \frac{\frac{1}{2} * E_1 h_1 M(1000)}{(EI)_{ef}}$$

$$= \frac{0.92 * 33000 * 89 * 25.8 * 10^6}{4.7 * 10^{13}} + \frac{\frac{1}{2} * 33000 * 80 * 25.8 * 10^6}{4.7 * 10^{13}} = |-1.49 - 0.73|$$

$$= 2.21\ MPa$$

$$\Delta\sigma = \sigma_{tot}(0) + \sigma_{tot}(1000) = 0 + 2.21 = 2.21\ MPa$$

$$\Delta \mathrm{F}_d = \Delta\sigma A_1 = 2.21 * 48000 = 106.2\ kN$$

The shear strength of the flange is calculated by assuming the flange is a system of compressive struts and tensile ties, schematised in Figure 9.13. The transverse reinforcement per unit length can be calculated by:

$$\frac{A_{sf}}{s_f} \geq \frac{\tau_{Ed} * h_\tau}{f_{yd} \cot\theta}$$

In which

A_{sf} is the cross-sectional area of the transverse reinforcement;

s_f is the spacing of the transverse reinforcement;

f_{yd} is the design tensile strength of the steel reinforcement;

θ is the angle of the concrete strut.

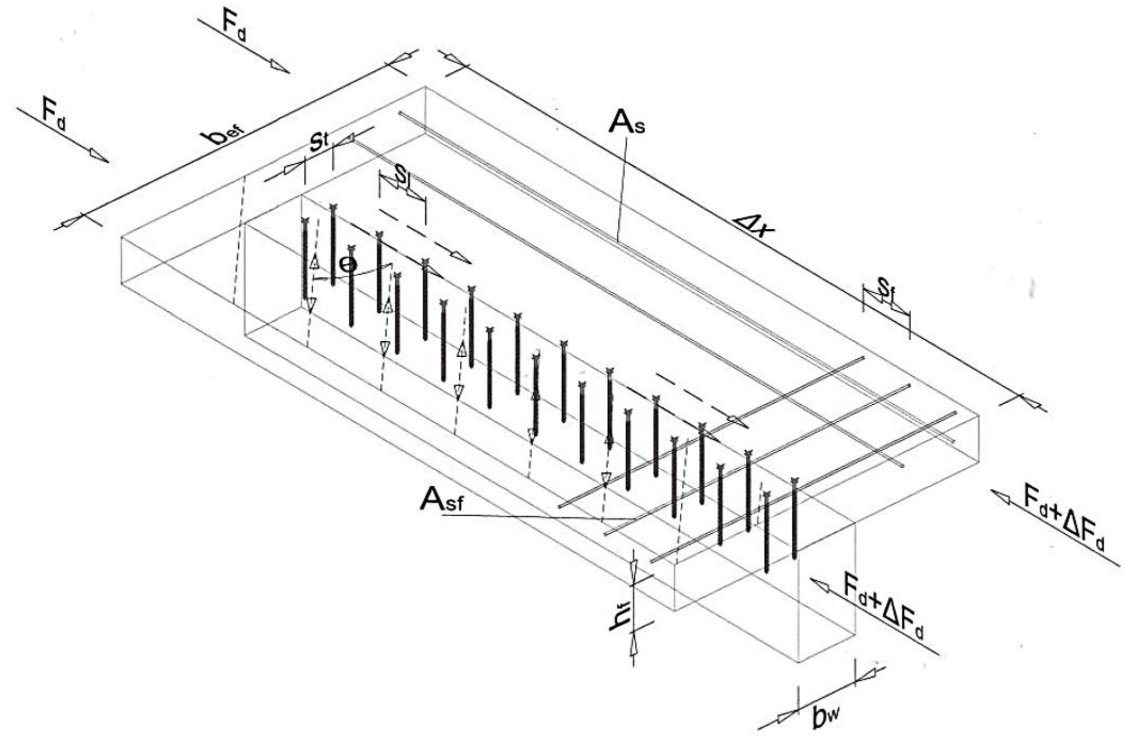

Figure 9.13: The connection between flange and web, 3D view [90]

Plane a-a

The length of the shear surface

$$h_\tau = h_c = 80\ mm$$

The acting longitudinal shear stress

$$\tau_{Ed} = \frac{\Delta F_d}{h_\tau \Delta x} = \frac{106.2 * 10^3}{80 * 1000} = 1.33\ MPa$$

Assumed angle of the concrete strut

$$\theta = 30°$$

To ensure that compressive failure of the struts won't occur

$$\tau_{Ed} = 1.33 < \frac{\nu f_{cd}}{(\cot\theta + \tan\theta)} = \frac{0.53 * 20}{(\cot 30 + \tan 30)} = 4.57 \quad Assumed\ angle\ OK$$

The transverse reinforcement per unit length

$$\frac{A_{sf}}{s_f} \geq \frac{\tau_{Ed} * h_\tau}{f_{yd}\cot\theta} = \frac{1.33 * 80}{435 * \cot 30} = 0.141\ mm^2/mm = 141\ mm^2/m$$

The assumed bar diameter and bar area

$$\emptyset_l = \emptyset_t = 6\ mm$$

$$A_{bar} = \frac{1}{4}\pi\emptyset_l^2 = \frac{1}{6} * \pi * 6^2 = 28.3\ mm^2$$

The required number of bars

$$n = \frac{A_{sf}}{A_{bar}} = \frac{141}{28.3} = 5$$

The total area of reinforcement and the spacing of the bars

$$A_{sf} = A_{bar} * n = 28.3 * 5 = 141\ mm^2$$

$$s_f = \frac{1000}{n} = \frac{1000}{5} = 200\ mm$$

Plane b-b

The length of the shear surface

$$h_\tau = 2h_{in\,c} + d_s = 2 * 60 + 12 = 132\ mm$$

The acting longitudinal shear stress

$$\tau_{Ed} = \frac{\Delta F_d}{h_\tau \Delta x} = \frac{106.2 * 10^3}{132 * 1000} = 1.61\ MPa$$

Assumed angle of the concrete strut

$$\theta = 30°$$

To ensure that compressive failure of the struts won't occur

$$\tau_{Ed} = 1.61 < \nu f_{cd} = 0.53 * 20 = 10.6 \quad Assumed\ angle\ OK$$

The transverse reinforcement per unit length

$$\frac{A_{sf}}{s_f} \geq \frac{\tau_{Ed} * h_\tau}{f_{yd} \cot\theta} = \frac{1.61 * 132}{435 * \cot 30} = 0.282\ mm^2/mm = 282\ mm^2/m$$

The required number of bars

$$n = \frac{A_{sf}}{A_{bar}} = \frac{324}{28.3} = 10$$

The total area of reinforcement and the spacing of the bars

$$A_{sf} = A_{bar} * n = 28.3 * 10 = 283\ mm^2$$

$$s_f = \frac{1000}{n} = \frac{1000}{10} = 100\ mm$$

The normative amount of reinforcement is found for shear plane b-b.

For determination of the transverse bending reinforcement, the rib floor is schematised as a continuous beam on four supports with two cantilevering parts. The acting bending moment is calculated with simplified rules. The moment at the first support, the second support, first mid-span and second mid-span are calculated with:

$$M_{sup,1} = \frac{1}{2} * q_{ULS,t} * l_{cant}^2 = \frac{1}{2} * 8.7 * 300^2 = 0.40\ kNm$$

$$M_{sup,2} = \frac{1}{12} * q_{ULS,t} * l_{cant}^2 = \frac{1}{12} * 8.7 * 300^2 = 0.26\ kNm$$

$$M_{mid,1} = \frac{M_{sup,1} + M_{sup,2}}{2} - \frac{1}{8} * q_{ULS,t} * l_{mid}^2 = \frac{0.40 + 0.26}{2} - \frac{1}{8} * 8.7 * 600^2 = 0.07\ kNm$$

$$M_{mid,2} = \frac{M_{sup,2} + M_{sup,2}}{2} - \frac{1}{8} * q_{ULS,t} * l_{mid}^2 = \frac{0.26 + 0.26}{2} - \frac{1}{8} * 8.7 * 600^2 = 0.13\ kNm$$

A check is done to ensure that these values are close to the real values by calculating the bending moments with the programme MatrixFrame. The bending moment distribution is shown in Figure 9.14. This computation results the following bending moments

$$M_{sup,1}^{mf} = 0.39\ kNm$$

$$M_{sup,2}^{mf} = 0.23\ kNm$$

$$M_{mid,1}^{mf} = 0.08\ kNm$$

$$M_{mid,2}^{mf} = 0.15\ kNm$$

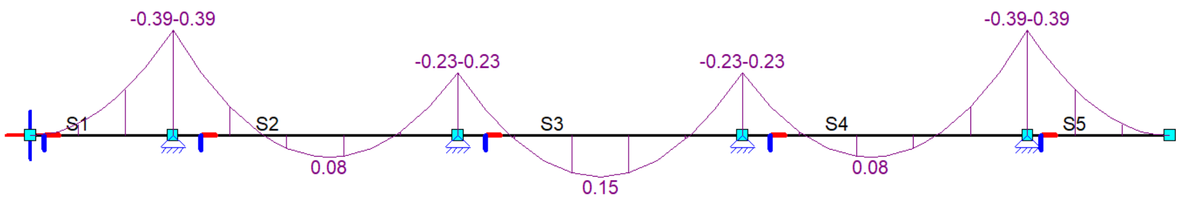

Figure 9.14: Transverse moment calculation Matrixframe

The acting moments at the supports are a bit overestimated and the moments in the span are a little bit underestimated with the hand calculations. The difference is very small so for simplification, the hand calculated values are used.

The required amount of reinforcement is calculated in the same manner as for the longitudinal reinforcement. The only difference is that negative and positive moments occur and thus a top and bottom rebar has to be implemented.

The used rebar diameter in the calculation procedure for the longitudinal reinforcement is used, an assumption is made for the number of bars.
The bending moment resistance is verified with unity check;

$$\frac{M_{Ed,conc}}{M_{Rd}} \leq 1$$

In which

$$M_{Ed,conc} = \begin{cases} \max(M_{sup,1}, M_{sup,2}) \\ \max(M_{mid,1}, M_{mid,2}) \end{cases}$$

The calculation is shown for the support position, where a negative bending moment occurs, Figure 9.15 shows the situation.

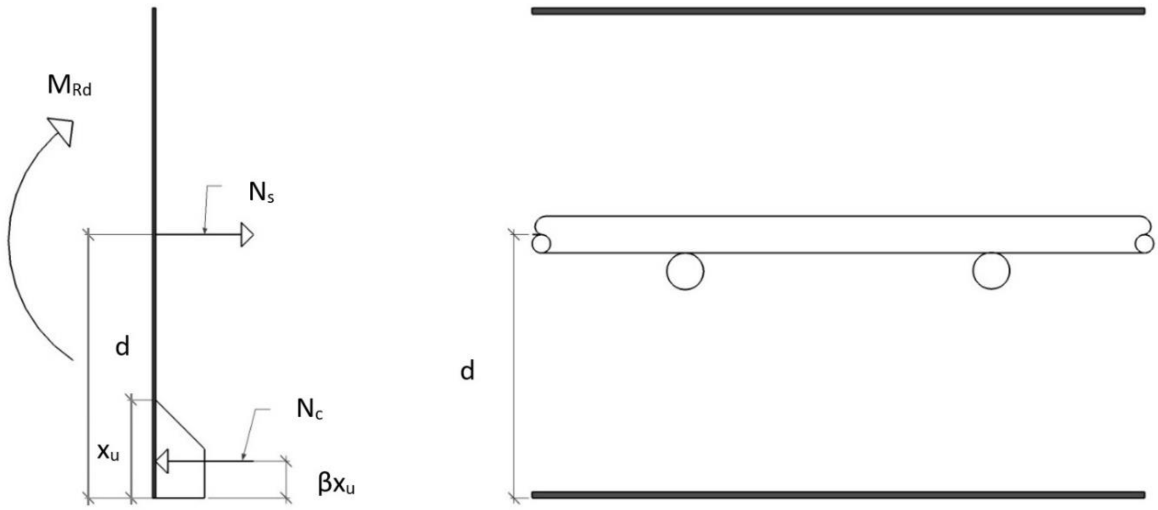

Figure 9.15: Concrete cross-section, negative transverse bending

The assumed bar diameter, bar area and number of bars

$$\emptyset_l = \emptyset_t = 6\ mm$$

$$A_{bar} = \frac{1}{4}\pi\emptyset_l^2 = \frac{1}{6} * \pi * 6^2 = 28.3\ mm^2$$

$$n = 7$$

The total area of reinforcement and the spacing of the bars

$$A_{st,M} = A_{bar} * n = 28.3 * 7 = 197.9\ mm^2$$

$$s_t = \frac{b_{ef}}{n} = \frac{1000}{7} = 142.9\ mm$$

The effective height of the cross section

$$d = \frac{1}{2}h_c + \frac{1}{2} * \emptyset_t = 40 + 3 = 43\ mm$$

The force in the rebars

$$N_s = A_{st,M} * f_{yd} = 197.9 * 435 = 86.1\ kN$$

The height of the concrete compressive zone

$$x_u = \frac{N_s}{\alpha * b * f_{cd}} = \frac{86.1 * 10^3}{0.75 * 1000 * 20} = 5.7\ mm$$

The moment resistance due to the assumed reinforcement

$$M_{Rd} = N_s(d - \ \beta x_u) == 86 * 10^3(43 - 0.39 * 5.7) = 3.5\ kNm$$

The acting negative bending moment

$$M_{Ed,neg} = \max(M_{sup,1}, M_{sup,2}) = \max(0.40; 0.26) = 0.40\ kNm$$

Unity check

$$\frac{M_{Ed,neg}}{M_{Rd}} = \frac{0.40}{3.5} = 0.11 \leq 1\ \ Satisfied$$

This procedure is repeated for the positive bending moment at the mid-span. The only difference is the effective height.

The acting positive bending moment

$$M_{Ed,pos} = \max(M_{mid,1}, M_{id,2}) = \max(0.065; 0.13) = 0.13\ kNm$$

Unity check

$$\frac{M_{Ed,pos}}{M_{Rd}} = \frac{0.13}{2.99} = 0.04 \leq 1\ \ Satisfied$$

In-plane shear and transverse bending occur at the same time. Therefore, the required amount of reinforcement in the transverse direction is calculated as:

$$A_{st} = \max\left(A_{sf}; \frac{1}{2}A_{sf} + A_{st,M}\right) = \max\left(282.7; \frac{1}{2} * 282.7 + 197.9\right) = 339.3\ mm^2$$

Using the reinforcement bars stated above, the amount of rebars and the spacing thereof becomes

$$n = \frac{339.3}{28.3} = 12$$

$$s_t = \frac{1000}{12} = 83.3\ mm$$

The maximum and minimum reinforcement and the spacings between bars must be checked. The transverse reinforcement is the principal reinforcement since it is much more than the longitudinal reinforcement.

The minimum principal (transverse) reinforcement is calculated as:

$$A_{s,min} = min\begin{cases} A_{s,min1} \\ A_{s,min2} \end{cases}$$

In which

$$A_{s,min1} = \frac{f_{ctm}W}{f_{yd}z}$$

$$A_{s,min2} = 1.25A_{st}$$

The secondary (longitudinal) reinforcement should be at least 20% of the principal reinforcement. The maximum reinforcement is calculated as:

$$A_{s,max} = 0.04 * A_c$$

The minimum spacing is determined as:

$$s_{min} = max\begin{cases} \emptyset_{bar} \\ d_g + 5 \\ 20 \end{cases}$$

In which

d_g is the maximum grain diameter.

The maximum spacing is calculated as:

$$s_{max,p} = min\begin{cases} 2h_c \\ 250\ mm \end{cases}$$

$$s_{max,s} = min\begin{cases} 3h_c \\ 400\ mm \end{cases}$$

The maximum spacing between the transverse rebars: $s_{max,t} = 150\ mm$

A summary of the limit values for the used area of reinforcement and the spacing of the rebars are given in Table 9-41.

	$\boldsymbol{A_{s,min}}$	$\boldsymbol{A_{s,max}}$	$\boldsymbol{s_{min}}$	$\boldsymbol{s_{max}}$
Longitudinal $\boldsymbol{A_{sl}}$	$0.2 * A_{st} = 67.9\ mm^2$	$0.04 * A_c = 1920\ mm^2$	$\max(\emptyset_{bar}; 20) = \max(6; 20) = 20\ mm$	$\min(3h_c; 400) = \min(240; 400) = 240\ mm$
Transverse $\boldsymbol{A_{st}}$	$\min\left(\frac{f_{ctm}W}{f_{yd}z}; 1.25A_{st}\right) = \min(110.1; 424.1) = 110.1\ mm^2$	$0.04 * A_c = 1920\ mm^2$	$\max(\emptyset_{bar}; 20) = \max(6; 20) = 20\ mm$	$\min(2h_c; 250; 150) = \min(160; 250; 150) = 150\ mm$

Table 9-41: Limit values reinforcement area and spacing

Longitudinal

$$67.9 < 226.2 < 1920\ \ Satisfied$$

$$20 < 75 < 240\ \ Satisfied$$

Transverse

$$110.1 < 339.3 < 1920\ \ Satisfied$$

$$20 < 83.3 < 150\ \ Satisfied$$

Timber

Timber stresses

In the timber beams, the combination of bending- and tensile stresses should be verified in both timber parts.

Timber 1 tensile stress due to axial force

$$\sigma_2 = \frac{\gamma_2 E_2 a_2 M}{(EI)_{ef}} = \frac{1 * 11500 * 106 * 77.4 * 10^6}{4.7 * 10^{13}}$$
$$= 2.02\ MPa$$

Timber 1 bending stress due to bending moment

$$\sigma_{m,2} = \frac{0.5 E_2 h_2 M}{(EI)_{ef}} = \frac{0.5 * 11500 * 310 * 77.4 * 10^6}{4.7 * 10^{13}}$$
$$= \pm 2.95\ MPa$$

Unity check timber 1

$$\frac{\sigma_2}{f_{t,0,d,1}} + \frac{\sigma_{m,2}}{f_{m,d,1}} = \frac{2.02}{12.3} + \frac{2.95}{15.4} = 0.36 \leq 1 \quad Satisfied$$

Timber 2 tensile stress due to axial force

$$\sigma_3 = \frac{\gamma_3 E_3 a_3 M}{(EI)_{ef}} = \frac{1 * 14200 * 321 * 77.4 * 10^6}{4.7 * 10^{13}}$$
$$= 7.53\ MPa$$

Timber 2 bending stress due to bending moment

$$\sigma_{m,3} = \frac{0.5 E_3 h_3 M}{(EI)_{ef}} = \frac{0.5 * 14200 * 120 * 77.4 * 10^6}{4.7 * 10^{13}}$$
$$= \pm 1.41\ MPa$$

Unity check timber 1

$$\frac{\sigma_3}{f_{t,0,d,2}} + \frac{\sigma_{m,3}}{f_{m,d,2}} = \frac{7.53}{18.0} + \frac{1.41}{22.5} = 0.48 \leq 1 \quad Satisfied$$

Since the timber beams contribute the most to the height of the cross section, it is assumed that the total shear force is taken by the timber.

$$\tau_{max} = 1.5 * \frac{V_{Ed}}{A_t} = 1.5 * \frac{28.4}{60200} = 0.71\ MPa$$
$$\frac{\tau_{max}}{f_{v,d}} = \frac{0.71}{2.2} = 0.32 \leq 1 \ Satisfied$$

Connection

The connections must be verified for two positions because the spacing between the notches in not uniform. The spacing optimization results in a spacing for the two outer sides, each having a length L/4, and a spacing for the middle part, having a length of L/2. The difference spacing results in different maximum forces in the notches at the outer sides and in the middle part. The unity checks for the notches are done for a maximum acting shear force at the end of the slab, and a reduced shear force of halve the maximum shear at L/4 from the edge of the slab.

The design load carrying capacity of the notch is the smallest capacity found with the following formulae:

$$F_{v,Rd} = min \begin{cases} f_{v,c,d} b_n l_n \\ f_{c,d} b_n h_n \\ f_{v,t,d} b_n \min(l_v, l_s, 8h_n) \\ f_{h,d} b_n h \end{cases}$$

In which

- $f_{c,d}$ is the design compressive strength of the concrete;
- $f_{v,t,d}$ is the design shear strength of the timber;
- $f_{h,d}$ is the design embedment strength of the timber parallel to the grain;
- $f_{v,c,d}$ is the design shear strength of the concrete

$$F_{v,Rd} = min \begin{cases} 96.0 \\ 112.0 \\ 100.4 \\ 99.6 \end{cases} kN = 96.0\ kN$$

The maximum shear stress in the connection, from $L = 0 \rightarrow \frac{1}{4}L$ and $L = \frac{3}{4}L \rightarrow L$

$$\tau_{conn,max} = \frac{\gamma_1 E_1 A_1 a_1}{b_2 (EI)_{ef}} V_{Ed} = \frac{0.92 * 33000 * 48000 * 89}{140 * 4.7 * 10^{13}} 28.4 * 10^3$$
$$= 0.56\ MPa$$

Maximum shear force in the connection

$$F_{v,conn,max} = \tau_{conn,max} * b_2 * s_{min} = 0.56 * 140 * 500 = 39.3\ kN$$

The shear stress in the connection, from $L = \frac{1}{4}L \rightarrow \frac{3}{4}L$

$$\tau_{conn,1/4L} = \frac{\gamma_1 E_1 A_1 a_1}{b_2 (EI)_{ef}} V_{1/4L} = \frac{0.92 * 33000 * 48000 * 89}{140 * 4.7 * 10^{13}} 14.2 * 10^3$$
$$= 0.28\ MPa$$

Shear force in the connection at $\frac{1}{4}L$

$$F_{v,conn,1/4L} = \tau_{conn,1/4L} * b_2 * s_{max} = 0.28 * 140 * 870 = 34.2\ kN$$

Besides the shear of timber and concrete and the crushing of timber and concrete, the fastener should be able to withstand the uplift force. This is the tensile force between the timber and the concrete. The design tensile resistance of the screw

$$F_{t,Rd} = f_{tens,k} = f_{u,k} * \frac{1}{4} * \pi * d_1^2 = 800 * \frac{1}{4} * \pi * 10.5^2 = 69.3\ kN$$

Maximum tensile force in the screw at $L = 0$

$$F_{t,conn,max} = \max(tan\,\theta * F_{v,Rd}, 0.1 * F_{v,conn,max}) = \max(55.4; 3.93) = 55.4\ kN$$

Tensile force in the screw at $\frac{1}{4}L$

$$F_{t,conn,1/4L} = \max(tan\,\theta * F_{v,Rd}, 0.1 * F_{v,conn,1/4L}) = \max(55.4; 3.42) = 55.4\ kN$$

Unity checks

$$\frac{F_{v,conn,max}}{F_{v,Rd}} = \frac{39.3}{96.0} = 0.41 \leq 1 \quad Satisfied$$

$$\frac{F_{t,conn,max}}{F_{t,Rd}} = \frac{55.4}{69.3} = 0.69 \leq 1 \quad Satisfied$$

Unity checks

$$\frac{F_{v,conn,1/4L}}{F_{v,Rd}} = \frac{34.2}{96.0} = 0.36 \leq 1 \quad Satisfied$$

$$\frac{F_{t,conn,1/4L}}{F_{t,Rd}} = \frac{55.4}{69.3} = 0.69 \leq 1 \quad Satisfied$$

9.2.3 Serviceability limit state verification – short term

In the SLS design, limits are given for deformations, vibrations and crack width control.

Bending stiffness built-up beam

Usually the resulting bending stiffness is different from the one found in ULS short term because the slip modulus is different for these situations. For notched connections the slip modulus used for ULS is the same one used for SLS, therefor the bending stiffness is also equal.

Deformations

For a timber simply supported beam, the maximum initial deflection is

$$w_{inst,max} \leq \frac{l}{300} = \frac{10900}{300} = 36.3\ mm$$

The instantaneous deflection due to self-weight and the variable loading respectively

$$w_{inst,G} = \frac{5}{384} * \frac{q_G * L^4}{(EI)_{ef}} = \frac{5}{384} * \frac{1.7 * 10900^4}{4.7 * 10^{13}} = 6.73\ mm$$

$$w_{inst,Q} = \frac{5}{384} * \frac{q_Q * L^4}{(EI)_{ef}} = \frac{5}{384} * \frac{2.1 * 10900^4}{4.7 * 10^{13}} = 8.24 mm$$

It is possible that a precamber should be added to ensure compliance with the deformation limit, w_c. The used precamber is taken equal to the instantaneous deflection caused by the self-weight of the concrete and the timber, calculated by

$$w_{inst,sw,conc} = \frac{5}{384} * \frac{q_{sw,conc} * L^4}{(EI)_{ef}} = \frac{5}{384} * \frac{1.18 * 10900^4}{4.7 * 10^{13}} = 4.64\ mm$$

$$w_{inst,sw,tim} = \frac{5}{384} * \frac{q_{sw,tim} * L^4}{(EI)_{ef}} = \frac{5}{384} * \frac{0.26 * 10900^4}{4.7 * 10^{13}} = 1.02\ mm$$

$$w_c = 4.64 + 1.02 = 5.65\ mm$$

The total instantaneous deflection results in

$$w_{inst,tot} = w_{inst,G} + w_{inst,Q} - w_c = 6.73 + 8.24 - 5.65 = 9.3\ mm$$

Unity check

$$\frac{w_{inst,tot}}{w_{inst,max}} = \frac{9.3}{36.3} = 0.26 \leq 1 \quad Satisfied$$

Vibrations

it is verified whether a vibration calculation is required. If the quasi-static loading is at least 5 kN/m², no calculation is required.

$$q_{Q-S} = \sum_{j\geq 1} G_{k,j} + \sum_{i\geq 1} \Psi_{2,i} * Q_{k,i} = \frac{1.7 + 0.3 * 2.1}{b_{ef}} = 3.91 < 5 \quad Not\ satisfied$$

If the quasi-static load is smaller than the required value, either the fundamental frequency must be at least 3 Hz or the deflection in the short term under quasi-static loading may not be bigger than 34mm.

The fundamental frequency for timber beams can be determined by

$$f_1 = \frac{\pi}{2l^2}\sqrt{\frac{(EI)_l}{m}} = \frac{\pi}{2 * 10900^2}\sqrt{\frac{4.7 * 10^{13}/10.9}{243.8}} = 1.76 < 3\,Hz \quad Not\ satisfied$$

In which

m is the mass per unit area in $\left[\frac{kg}{m^2}\right]$;

l is the span of the floor in $[m]$;

$(EI)_l$ is the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction in $[Nm^2/m]$.

The deflection under quasi-static loading

$$w_{Q-S} = \frac{5}{384} * \frac{q_{Q-S} * L^4}{(EI)_{ef}} = \frac{5}{384} * \frac{3.91 * 10900^4}{4.7 * 10^{13}} = 9.21 \leq 34\,mm \quad Satisfied$$

Cracking of concrete

Cracks in concrete in indoor environments should be limited to ensure a good appearance of the structure. The maximum crack width is $w_{max} = 0.4\,mm$. This value may be higher if specific limits are stated for an acceptable appearance.

The cracking is assumed controlled if a minimum reinforcement is applied in mm2/m. The minimum required reinforcement to control concrete cracking can be determined by using table 9.1 in [90].

Longitudinal reinforcement

$$\rho_L = \frac{226.2}{b_{ef,c}} = 377.0 > 80\,mm^2/m \quad Satisfied$$

Transverse reinforcement

$$\rho_T = 339.3 > 80\,mm^2/m \quad Satisfied$$

9.2.4 Ultimate limit state verification – long term

Again, the start of the calculation is to determine the cooperation factor, the position of the NC, the distances between the mid-point of the timber and concrete the NC and the effective bending stiffness. The difference with the short-term calculation is the use of the effective moduli of elasticity.

Bending stiffness built-up beam

Cooperation factors

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1 s_1}{K_i l^2}} = 0.97$$

$$\gamma_2 = 1$$

$$\gamma_3 = 1$$

Position of the normal force centre with respect to the top of the element

$$NC = \frac{\sum_{i=1}^{3} \gamma_i A_i E_i a_{i-top}}{\sum_{i=1}^{3} \gamma_i A_i E_i} = 212\ mm$$

The distances between the mid-point of the timbers and concrete , and the NC

$$a_1 = NC - \frac{1}{2} h_c = 172\ mm$$

$$a_2 = h_c + \frac{1}{2} h_{t1} - NC = 23\ mm$$

$$a_3 = h_{tot} - \frac{1}{2} h_{t2} - NC = 238\ mm$$

The effective bending stiffness

$$(EI)_{ef,fin} = \sum_{i=1}^{2} (E_i I_i + \gamma_i E_i A_i a_i^2) = 2.2 * 10^{13}\ Nmm^2$$

Forces arising due to concrete shrinkage

The shrinkage of concrete results stresses acting in the concrete and timber. First the shrinkage strain acting in the concrete is calculated. Because the strain is subjected to relaxation, it may be lowered by multiplying the total final shrinkage with a factor 0.9.

$$\varepsilon_{cs,\infty} = 0.9(\varepsilon_{cd,\infty} + \varepsilon_{ca,\infty})$$

The drying shrinkage strain is calculated with

$$\varepsilon_{cd,\infty} = \varepsilon_{cd,0} * k_h$$

In which

k_h is a coefficient dependent on the notional size;
$\varepsilon_{cd,0}$ is the drying shrinkage strain at $t = 0$.

$$\varepsilon_{cd,0} = 0.85 \left[(220 + 110\alpha_{ds1}) * e^{-\alpha_{ds2} \frac{f_{cm}}{f_{cm0}}} \right] 10^{-6} * \beta_{RH}$$

In which

$\beta_{RH} = 1.55 \left[1 - \left(\frac{RH}{RH_0} \right)^3 \right]$;
α_{ds1}, α_{ds2} are coefficients depending on the cement type;
RH is the relative humidity of the surrounding;
$f_{cm0} = 10\ MPa$;
$RH_0 = 100\ \%$.

The drying shrinkage strain follows

$$\varepsilon_{cd,0} = 0.85\left[(220 + 110 * 4) * e^{-0.12\frac{38}{10}}\right]10^{-6} * 1.36 = 0.48\ ‰$$

$$\varepsilon_{cd,\infty} = \varepsilon_{cd,0} * k_h = 0.48 * 1 = 0.48\ ‰$$

The autogenous shrinkage strain is calculated with

$$\varepsilon_{ca,\infty} = 2.5(f_{ck} - 10)10^{-6} = 2.5(30 - 10)10^{-6} = 0.00005\ ‰$$

The total shrinkage strain can now be determined

$$\varepsilon_{cs,\infty} = 0.9(\varepsilon_{cd,\infty} + \varepsilon_{ca,\infty}) = 0.9(0.48 + 0.00005) = 0.43\ ‰$$

Secondly the section properties of all the layers and the properties of the total element are determined. The axial stiffness per layer, the axial stiffness of the element and the bending stiffness per layer:

$$(EA)_c = 2.95 * 10^8\ N$$
$$(EA)_{T1} = 3.60 * 10^8\ N$$
$$(EA)_{T2} = 1.72 * 10^8\ N$$
$$(EA)_{tot} = 8.27 * 10^8\ N$$
$$(EI)_c = 1.57 * 10^{11}\ Nmm^2$$
$$(EI)_{T1} = 28.8 * 10^{11}\ Nmm^2$$
$$(EI)_{T2} = 2.06 * 10^{11}\ Nmm^2$$

The bending stiffness of the total element and the location of the neutral axis have already been calculated.

Now the forces occurring in each layer are determined. The concrete slab wants to shrink but is kept in place due to the (assumed rigid) connection to the timber beams. This normal shrinkage force is calculated as

$$N^* = \varepsilon_{cs,\infty}(EA)_c = \frac{0.43}{1000} * 2.95 * 10^8 = 127.9\ kN$$

Because the concrete slab is kept in place, the normal shrinkage force results in a tensile force in the slab. This force can be moved to the neutral axis which will result in a normal compressive force and a bending moment on the total cross section. This bending moment is determined by

$$M^* = N^* * e = 127.9\left(NC - \frac{1}{2} * h_c\right) = 127.9(212 - 40) = 22.0\ kNm$$

This bending moment can be rewritten into a fictitious load on the element by

$$q_{shr} = \frac{8M^*}{l^2} = 1.48\ kN/m$$

Every layer is subjected to a normal force due to the normal shrinkage force, N_{i,N^*}, a normal force due to the total bending moment, N_{i,M^*}, and a bending moment due to the total bending moment, M_i. They are calculated with the following formulae

$$N_{c,N^*} = \frac{(EA)_c}{(EA)_{tot}}N^* = \frac{2.95 * 10^8}{8.27 * 10^8} * 127.9 * 10^3 = 45.6\ kN$$

$$N_{c,M^*} = \frac{(EA)_c}{(EI)_{eff,fin}}N^* = \frac{2.95 * 10^8}{2.2 * 10^{13}} * 127.9 * 10^3 = 51.5\ kN$$

$$M_c = \frac{(EI)_c}{(EI)_{eff,fin}} M^* = \frac{1.57 * 10^{11}}{2.2 * 10^{13}} * 22.0 * 10^6 = 0.16\ kNm$$

$$N_{T1,N^*} = 55.7\ kN$$

$$N_{T1,M^*} = 8.4\ kN$$

$$M_{T1} = 2.9\ kNm$$

$$N_{T2,N^*} = 26.6\ kN$$

$$N_{T2,M^*} = 41.6\ kN$$

$$M_{T2} = 0.21\ kNm$$

Concrete

Concrete stresses

In the long-term ultimate limit state, the concrete stresses are verified. The differences from the verification in the short-term is the use of an effective modulus of elasticity and the addition of stresses arising due to concrete shrinkage.

First the stresses arising due to the concrete shrinkage are determined.

$$\sigma_{c,N^*} = -\frac{N^*}{A_c} = -\frac{127.9 * 10^3}{48000} = 2.67\ N/mm^2$$

$$\sigma_{c,N_{c,N^*}} = +\frac{N_{c,N^*}}{A_c} = \frac{45.6 * 10^3}{48000} = -0.95\ N/mm^2$$

$$\sigma_{c,N_{c,M^*}} = \pm\frac{N_{c,M^*}}{A_c} = \frac{51.5 * 10^3}{48000} = -1.07\ N/mm^2$$

$$\sigma_{c,M_c} = \pm\frac{M_c}{W_c} = \frac{0.16 * 10^6}{\frac{1}{6} * b_{ef} h_c^2} = \pm\ 0.25\ N/mm^2$$

$$\sigma_{c,shr,top} = 2.67 - 0.95 - 1.07 - 0.25 = 0.39 N/mm^2$$

$$\sigma_{c,shr,bot} = 2.67 - 0.95 - 1.07 + 0.25 = 0.89\ N/mm^2$$

Concrete compressive stress due to axial force

$$\sigma_1 = \frac{\gamma_1 E_{1,fin} a_1 M}{(EI)_{ef,fin}} = \frac{0.97 * 6140 * 172 * 77.4 * 10^6}{2.2 * 10^{13}} = -3.67\ MPa$$

Concrete bending stress due to bending moment

$$\sigma_{m,1} = \frac{0.5 E_{1,fin} h_1 M}{(EI)_{ef,fin}} = \frac{0.5 * 6140 * 80 * 77.4 * 10^6}{2.2 * 10^{13}} = \pm 0.88\ MPa$$

Concrete stress in the top fibre

$$\sigma_{c,top} = -3.67 - 0.88 + 0.39 = \ -4.15\ MPa$$

$$\frac{\sigma_{c,top}}{f_{cd}} \leq 1 \rightarrow \frac{-4.15}{-20} = 0.21 \leq 1 \quad Satisfied$$

Concrete stress in the bottom fibre

$$\sigma_{c,bot} = -3.67 + 0.88 + 0.89 = -1.90\ MPa$$

$$\frac{\sigma_{c,bot}}{f_{ctd}} \leq 1 \rightarrow \frac{-1.90}{1.93} = \ -0.98 \leq 1 \quad Satisfied$$

Timber

<u>Timber stresses</u>

As for concrete, the only difference from the verification in the short-term is the use of an effective modulus of elasticity and the addition of stresses arising due to concrete shrinkage.
First the stresses arising due to the concrete shrinkage are determined.

$$\sigma_{T1,N_{T1,N^*}} = -1.28\ N/mm^2$$
$$\sigma_{T1,N_{T1,M^*}} = 0.19\ N/mm^2$$
$$\sigma_{T1,M_{T1}} = 1.31\ N/mm^2$$
$$\sigma_{T2,N_{T2,N^*}} - 1.58\ N/mm^2$$
$$\sigma_{T2,N_{T2,M^*}} = 2.48\ N/mm^2$$
$$\sigma_{T2,M_{T2}} = 0.62\ N/mm^2$$

Timber 1 tensile stress due to axial force

$$\sigma_2 = \frac{\gamma_2 E_{2,fin} a_2 M}{(EI)_{ef,fin}} = \frac{1 * 8295 * 23 * 77.4 * 10^6}{2.2 * 10^{13}}$$
$$= 0.68\ MPa$$

Total axial stress timber 1

$$\sigma_{2,tot} = \sigma_2 + \sigma_{T1,N_{T1,N^*}} + \sigma_{T1,N_{T1,M^*}} = -0.41\ MPa$$

Timber 1 bending stress due to bending moment

$$\sigma_{m,2} = \frac{0.5 E_{2,fin} h_2 M}{(EI)_{ef,fin}} = \frac{0.5 * 8295 * 310 * 77.4 * 10^6}{2.2 * 10^{13}}$$
$$= \pm 4.59\ MPa$$

Total bending stress timber 1

$$\sigma_{m,2,tot} = \sigma_{m,2} + \sigma_{T1,M_{T1}} = 5.90\ MPa$$

Unity check timber 1

$$\frac{\sigma_{2,tot}}{f_{t,0,d,1}} + \frac{\sigma_{m,2,tot}}{f_{m,d,1}} = \frac{-0.41}{12.3} + \frac{5.90}{15.4} = 0.35 \leq 1 \quad Satisfied$$

Timber 2 tensile stress due to axial force

$$\sigma_3 = \frac{\gamma_3 E_{3,fin} a_3 M}{(EI)_{ef,fin}} = \frac{1 * 10242 * 321 * 77.4 * 10^6}{2.2 * 10^{13}}$$
$$= 8.71\ MPa$$

Total axial stress timber 2

$$\sigma_{3,tot} = \sigma_3 + \sigma_{T2,N_{T2,N^*}} + \sigma_{T1,N_{T2,M^*}} = 9.61\ MPa$$

Timber 2 bending stress due to bending moment

$$\sigma_{m,3} = \frac{0.5 E_{3,fin} h_3 M}{(EI)_{ef,fin}} = \frac{0.5 * 10242 * 120 * 77.4 * 10^6}{2.2 * 10^{13}}$$
$$= \pm 2.20\ MPa$$

Total bending stress timber 2

$$\sigma_{m,3,tot} = \sigma_{m,3} + \sigma_{T2,M_{T2}} = 2.82\ MPa$$

Unity check timber 2

$$\frac{\sigma_{3,tot}}{f_{t,0,d,2}} + \frac{\sigma_{m,3,tot}}{f_{m,d,2}} = \frac{9.61}{18.0} + \frac{2.82}{22.5} = 0.66 \leq 1 \quad Satisfied$$

9.2.5 Serviceability limit state verification – long term

Bending stiffness built-up beam

Because the slip modulus used for ULS is the same one used for SLS, the bending stiffness is also equal.

Deformations

The maximum final deflection is

$$w_{fin} \leq \frac{l}{250} = \frac{10900}{250} = 43.6\ mm$$

The limit for the additional deflection which results from the variable loading and the long term effects of creep

$$w_{add,max} \leq \frac{3}{1000} l = 32.7\ mm$$

The final deflection due to self-weight and the part of the deflection caused by creep

$$w_{fin,G} = \frac{5}{384} * \frac{q_G * L^4}{(EI)_{ef,fin}} = \frac{5}{384} * \frac{1.7 * 10900^4}{2.2 * 10^{13}} = 14.57\ mm$$

$$w_{G,cr} = w_{fin,G} - w_{inst,G} = 14.57 - 6.73 = 7.83\ mm$$

The final deflection due to the variable loading, including too much creep, the creep deflection caused by the variable loading and the final deflection due to the variable loading

$$w_{fin,Q+cr} = \frac{5}{384} * \frac{q_Q * L^4}{(EI)_{ef,fin}} = \frac{5}{384} * \frac{2.1 * 10900^4}{2.2 * 10^{13}} = 17.82\ mm$$

$$w_{Q,cr} = \left(w_{fin,Q+cr} - w_{inst,Q}\right)\Psi_2 = (17.82 - 8.24) * 0.3 = 2.87\ mm$$

$$w_{fin,Q} = w_{inst,Q} + w_{Q,cr} = 8.24 + 2.87 = 11.11\ mm$$

The deflection due to shrinkage

$$w_{shr} = \frac{5}{384} * \frac{q_{shr} * L^4}{(EI)_{ef,fin}} = \frac{5}{384} * \frac{1.48 * 10900^4}{2.2 * 10^{13}} = 12.57\ mm$$

The total additional deflection

$$w_{add} = w_{G,cr} + w_{fin,Q} + w_{shr} = 7.83 + 11.11 + 12.57 = 31.51\ mm$$

The total final deflection

$$w_{fin,tot} = w_{fin,G} + w_{fin,Q} + w_{shr} - w_c = 14.57 + 11.11 + 12.57 - 5.65 = 32.59\ mm$$

Unity checks

$$\frac{w_{add}}{w_{add,max}} = \frac{31.51}{32.7} = 0.96 \leq 1 \quad Satisfied$$

$$\frac{w_{fin,tot}}{w_{fin,max}} = \frac{32.59}{43.6} = 0.75 \leq 1 \quad Satisfied$$

9.3 Appendix C – Verification demountable connections

In this appendix the determination of the tolerances to be accommodated in the connections and the verification of the demountable connectors designed in chapter 6 are given.

9.3.1 Tolerances

In all the figures given in this paragraph, the used colours represent the following:

- Blue = the intended position of the bolt;
- Black = the intended position of the hole and the building, and the final size of the hole;
- Red and Green = deviated position due to tolerance.

Connection 1

To ensure structural soundness and a vertical fixation of the floor slab, an angle section is screwed onto the outer sides of the outer timber beams during fabrication of the floor slab. The angle section is fastened with a bolt to the edge beam during erection. If this connection would be loaded, the force acting would only be a vertical tensile force. Therefor in both horizontal directions the distance between the bolt shank and the connecting plates is not of importance for force transfer. The choice is made to use an oversized hole and not a slotted hole because these are less costly.

The used bolt is an M20 bolt which will be welded to the edge beam at the manufacturing plant. The position of the bolt can have a fabrication deviation of 2mm in both horizontal directions. A standard hole deviation of 4mm is required for oversized holes used for steel bolted connections. Therefor the starting diameter of the bolt hole is $d_H = d + 4 = 24\ mm$. The punching of this hole in the angle section has a fabrication tolerance of 2mm in both horizontal directions. Another fabrication tolerance which should be regarded is the tolerance on the length of the floor slab. To ensure that the connection will fit, this tolerance is added to the tolerance of the punching of the bolt hole.

In Figure 9.16 the fabrication tolerance of the bolt hole in both horizontal directions are shown with the green and red colours. Left the direction transverse to the edge beam is shown and right the direction longitudinal to the edge beam. Only the deviation of the bolt hole is shown. The possible deviation of the position of the bolt hole is equal to that of the possible deviation of the bolt. Since the bolt hole has a diameter which is 4mm bigger than the hole, the bolt will always fall in area where the hole will be applied. The required bolt diameter regarding the fabrication tolerance of the bolt hole is $d_H = 28\ mm$.

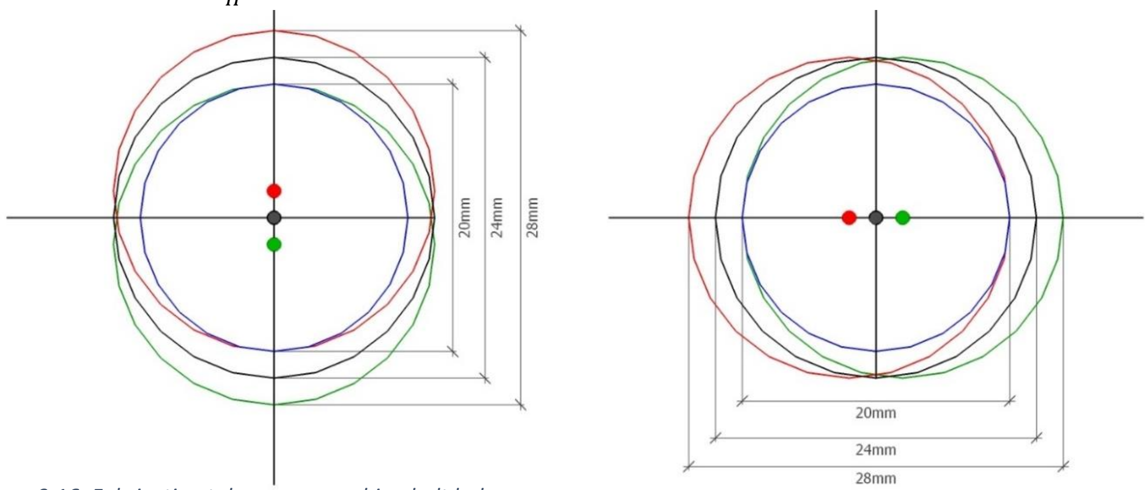

Figure 9.16: Fabrication tolereance punching bolt hole

In Figure 9.17 the fabrication tolerance of the length of the floor slab is shown left and the final bolt hole diameter taking into account all the fabrication tolerances is shown to the right.

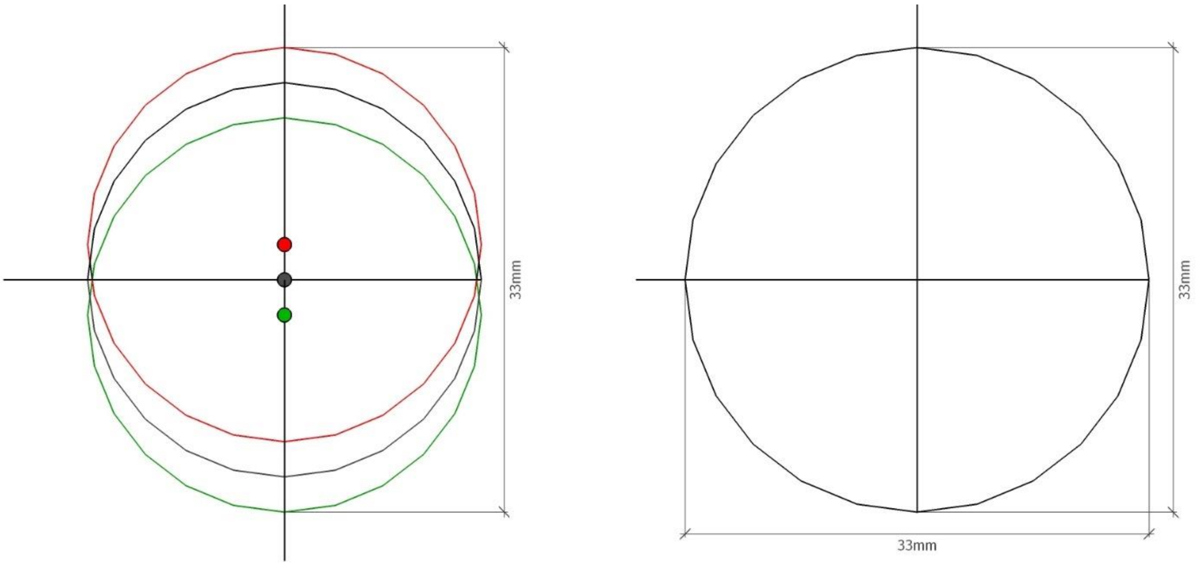

Figure 9.17: Fabrication tolerance bolt hole, length floor slab and final

Besides the fabrication tolerances, the assembly tolerances should be regarded as well. In the direction longitudinal to the edge beam, a tilt of the columns can result in a shift of the edge beams and thus a shift of the position of the bolt. Figure 9.18 shows this tilt, the value of the horizontal displacement of the edge beam is calculated as $\Delta = \frac{h_{column}}{500} = \frac{3205}{500} = 6.4\ mm$. The biggest column length is used to calculate this displacement to ensure that the elements of the connections at all positions in the building fit. At both sides of the building the columns could be tilted so the total tolerance is $2\Delta = 12.8\ mm$.

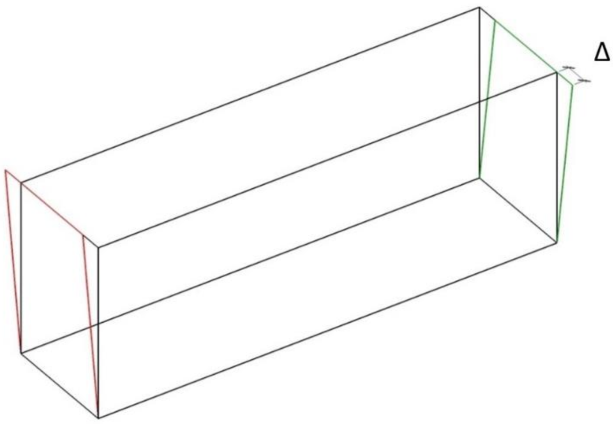

Figure 9.18: Assembly tolerance, tilt of columns in longitudinal direction

In the direction transverse to the edge beam, there are quite a few mistakes that can be made during erection due to which the edge beam could shift: a deviation of the straightness of the edge beam, a tilt of the columns, a deviation of the intended c-t-c distance of the edge beams and a deviation in the placement of the edge beams at the beam-column connection. Illustrations of these assembly tolerances are given in Figure 9.19, Figure 9.20, Figure 9.21 and Figure 9.22 respectively.

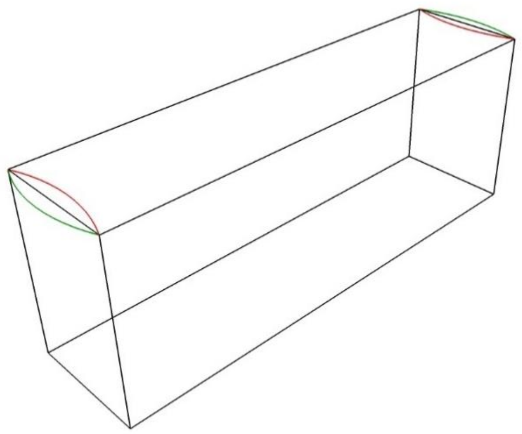

Figure 9.19: Assembly tolerance, deviation of straightness edge beam

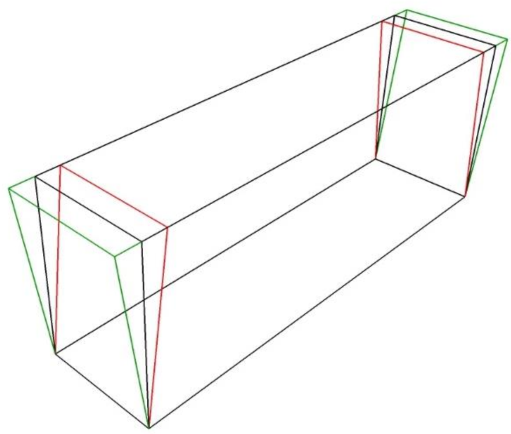

Figure 9.20: Assembly tolerance, tilt columns

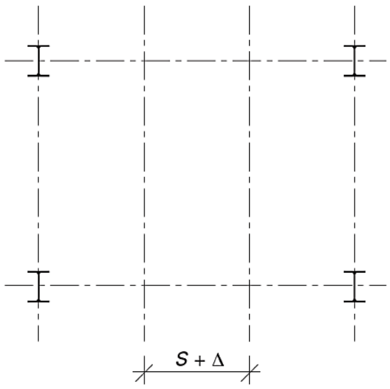

Figure 9.21: Assembly tolerance, centre-to-centre distance edge beams [112]

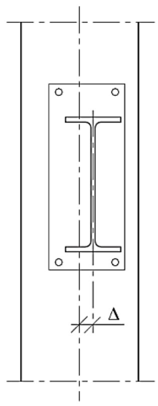

Figure 9.22: Assembly tolerance, deviation of placement edge beam at beam-column connection [112]

The maximum deviation of straightness is calculated as $\Delta = \frac{L_{beam}}{1000} = 1.8\ mm$. This tolerance can occur at both sides so a maximum deviation of $\Delta_{straight} = 3.6\ mm$ is found. The tilt of the columns results in a shift of the beam of $\Delta = \frac{h_{column}}{500} = \frac{3205}{500} = 6.4\ mm$. Again, both sides can deviate so the total tolerance is $\Delta_{tilt} = 12.8\ mm$. The maximum difference in c-t-c distance of the edge beams is $\Delta_{ctc} = 5mm$. Lastly the deviation of the placement of the edge beam is $\Delta = 3mm$. Both edge beams can be out of place, so the total tolerance becomes $\Delta_{place} = 6mm$.

The chance is very small that all these tolerances occur at the same time so the normative combination of possible deviations is sought. The following can occur:

- Column tilt and beam straightness deviations;
- Column tilt and placement deviation;
- C-t-c deviation

$$\Delta_{transverse} = \max \begin{cases} 12.8 + 6.4 \\ 12.8 + 6 \\ 5 \end{cases} = 18.8\ mm$$

In Figure 9.23 the influence of the assembly tolerances on the required bolt hole dimensions are shown and in Figure 9.24 the final diameter of the bolt hole is given, taking into account fabrication and erection tolerances.

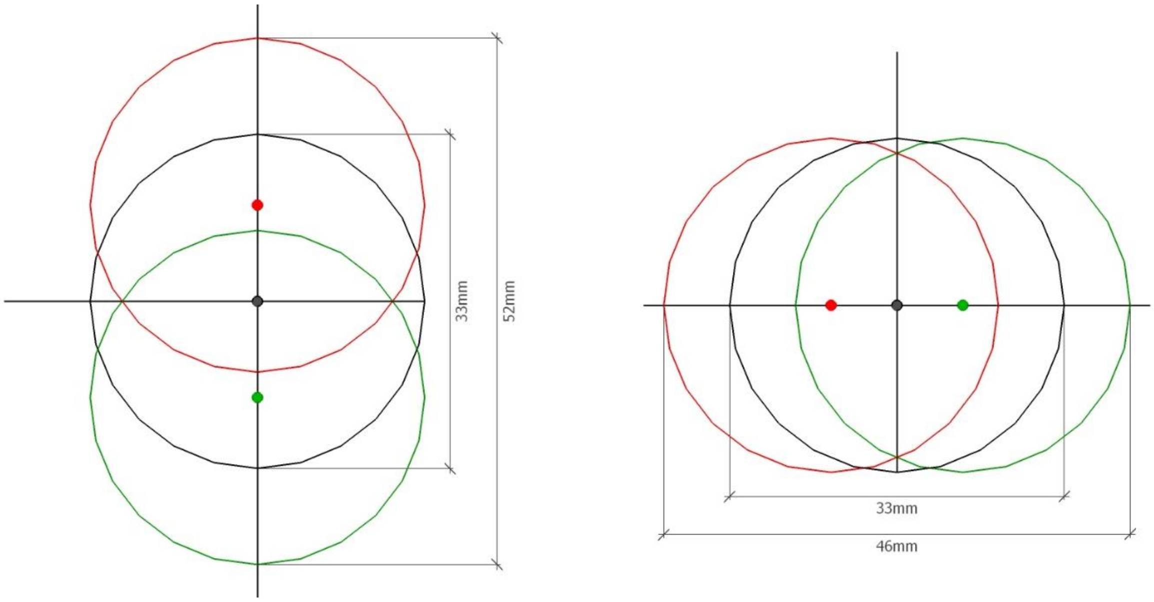

Figure 9.23: Assembly tolerances. Left: transverse, right: longitudinal

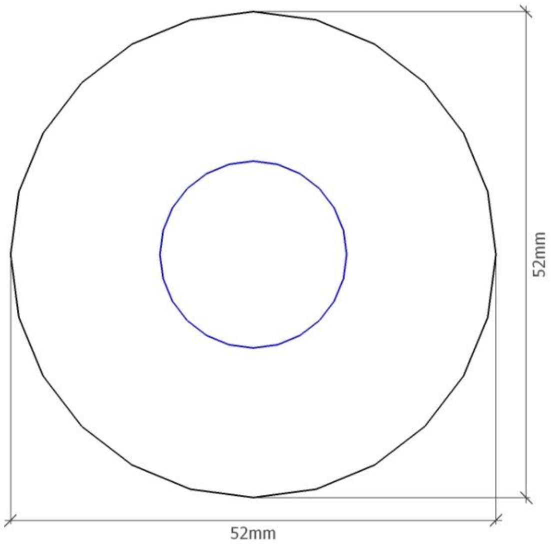

Figure 9.24: Final diameter bolt hole angle section

Connection 2

<u>Dimensions V-shape</u>

The used bolt is in this part of the connection is a M20 bolt. A standard oversize is required for all bolt holes. For slotted holes, this oversize is bigger. The assumption is made that this slotted hole will be short and thus in the slotted direction the hole must have a starting length of $l_H = d + 6 = 26\ mm$. In the direction perpendicular to the slot, the tolerance is equal to that of a regular hole, which results in $d_H = d + 2 = 22\ mm$.

Figure 9.25 shows the fabrication tolerances of the bolt hole which are required in vertical and horizontal direction. Only the deviation of the bolt hole is shown for the same reason mentioned for the bolt hole in connection 1.

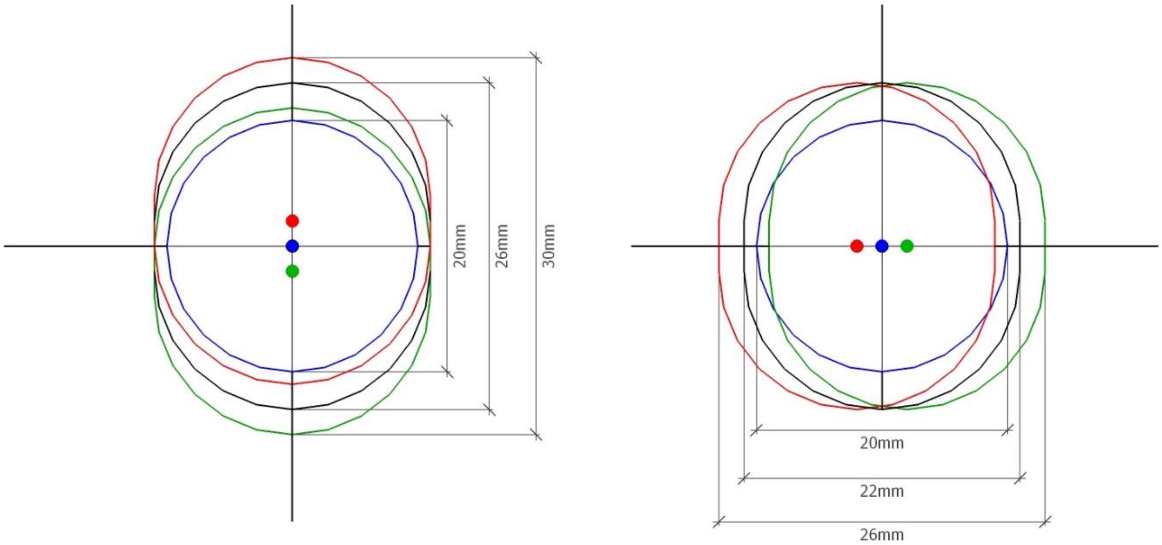

Figure 9.25: Fabrication tolerances V-shape

The fabrication tolerances result in required bolt hole dimensions and a V-shape shown in Figure 9.26. For the arched bottom of the V-shape is ¼ of a circle with a diameter of 60mm.

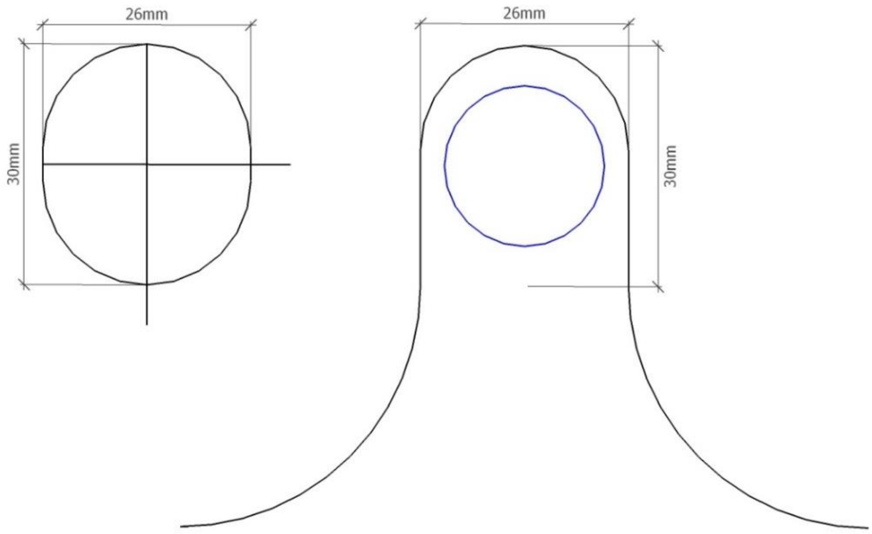

Figure 9.26: Slotted hole dimensions and V-shape due to fabrication tolerances

With the hole adapted for the fabrication tolerances, the assembly tolerances are determined. Only the possible tilt of the columns supporting the edge beam influence the possible fit of connection elements. This tilt is equal to the one calculated for the assembly tolerance for the bolt hole in the angle section in connection 1. Also, the value is equal. Applying this tolerance to the bolt hole gives the final required dimensions of the V-shape, shown in Figure 9.27.

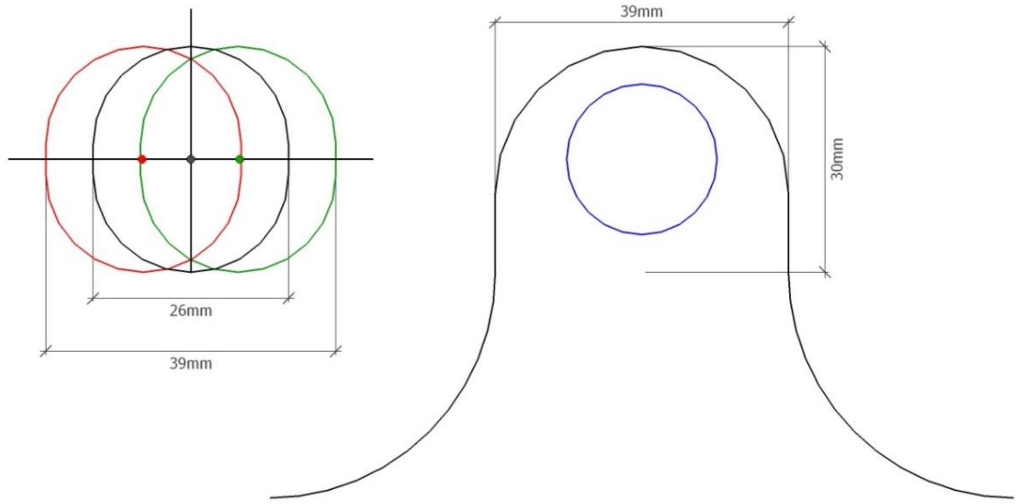

Figure 9.27: Assembly tolerances and final V-shape

<u>Dowel hole</u>

The determination of the required dimensions of the dowel hole is equal to that of the determination of the dimensions of the bolt hole in the angle section in connection 1. Since the used dowel also has a diameter of 20mm, the size of the oversized bolt hole is completely the same.

Slab-slab and slab-beam side connection

The required diameter of the bolt hole in the steel plate is determined. The tolerances that have to be accommodated in this bolted connection are:

- The deviation of the placement of the concrete bolt anchors;
- The punching of the bolt holes in the steel plate;
- The length of the floor slab;
- The width of the floor slab.

The placement of the anchors is a concrete fabrication tolerance just like the tolerances for the dimensions of the slab. Therefor the normative tolerance in both directions, and thus the possible positions of the bolt anchors, is found from these three possible deviations.

The placement of the bolt anchors can deviate 2mm in both directions, as can the placement of the punched hole. For the same mentioned reason mentioned above, only the deviation of the bolt hole is shown in Figure 9.28. The required diameter of the hole is in this case $22\ mm$.

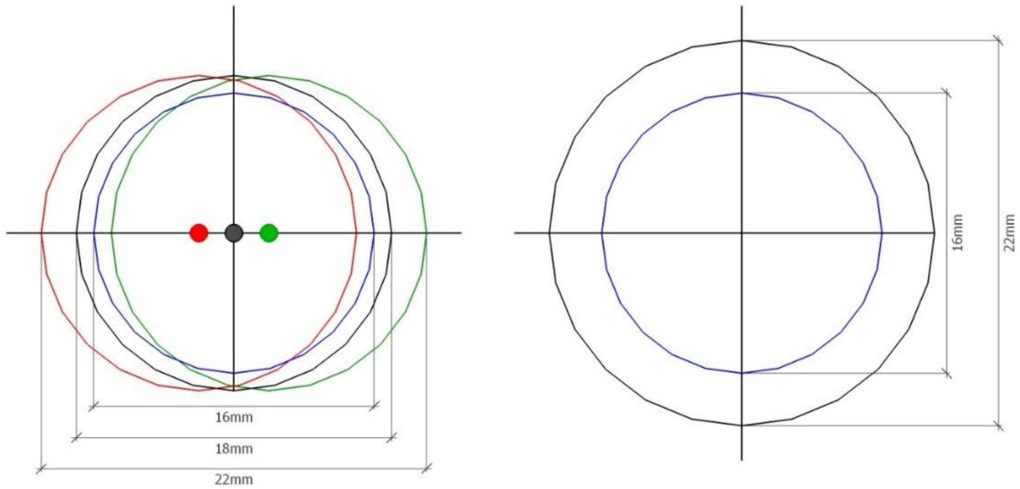

Figure 9.28: Tolerance bolt hole and bolt anchor deviation

The length of the left floor slab can be 5mm shorter than intended and the length of the right floor slab can be 5mm larger than intended. This deviation is shown in Figure 9.29 resulting in a bolt hole diameter of $28\ mm$. The smaller black diameter shown in the figure represents the required diameter for the deviation in the punched bolt hole.

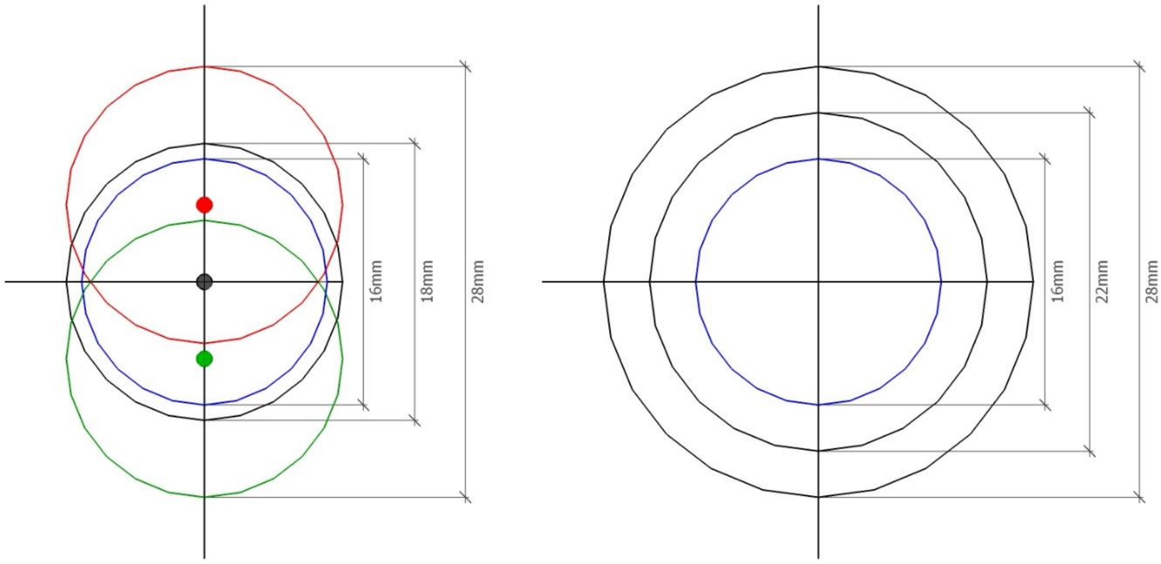

Figure 9.29: Tolerance slab length deviation

The width the floor slab can be either 3mm smaller or 3mm bigger than the intended width. This results in a +- deviation of $1.5\ mm$ on the bolt anchors. This deviation is given in Figure 9.30 and results in a required bolt hole diameter of $21\ mm$.

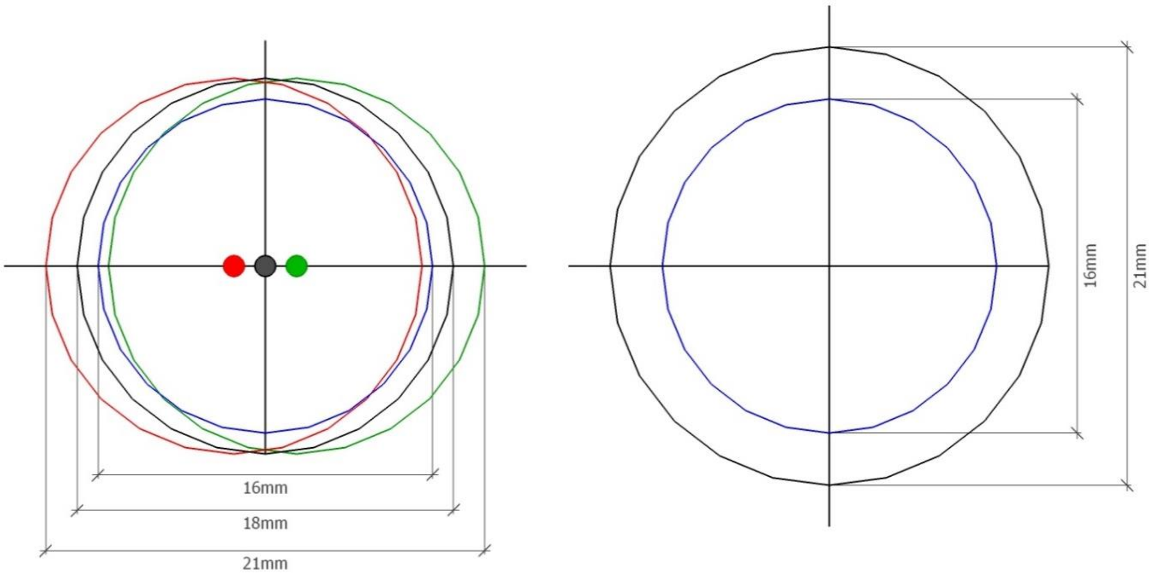

Figure 9.30: Tolerance slab width deviation

The maximum required bolt hole diameter is $28\ mm$, which is found for the tolerance for the length of the slab. Assembly tolerances are not determined separately because they are taken up in the slab-beam connection at the head ends.

9.3.2 Acting forces

Before the connections can be verified, the acting forces must be known. The forces resulting from the floor slab loading are determined separately for every element in the paragraphs below. When all the characteristic loads are known, the design load is determined by finding normative load combination from the following equation. The bottom part results in two values, one for taking the wind loading as the leading variable loading and one for taking the variable floor loading as the leading variable loading.

$$F_{Ed} \; or \; q_{Ed} = \max \begin{cases} \gamma_G G_k + \gamma_Q \Psi_{0,1} Q_{k,1} + \sum \gamma_Q \Psi_{0,i} Q_i \\ \gamma_G \xi G_k + \gamma_Q Q_{k,1} + \sum \gamma_Q \Psi_{0,i} Q_i \end{cases}$$

Wind loading

In this part, the global wind loads are determined with use of [113] and [114]. The detail loads acting in the connections are determined separately for every element in the paragraphs below.

The distributed wind load when wind is acting on the long side of the building is determined, see Figure 9.31. After calculating the peak velocity pressure on the outside of the building, the internal and external wind pressures can be determined. With these wind pressures, the forces acting on the sides of the building and on the floor slabs are determined for both wind on the longs side and wind on the short side of the building.

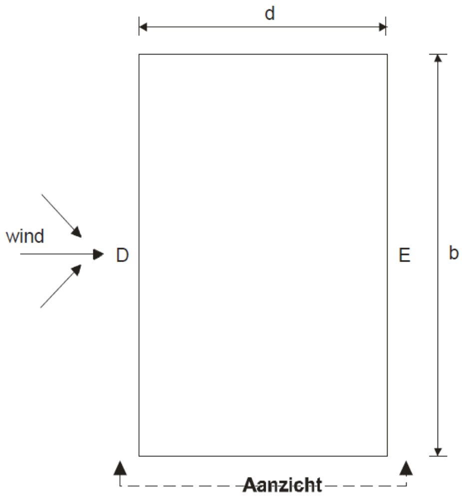

Figure 9.31: Wind direction and dimensions

$d = 10.9\ m$ and $b = 21.2\ m$.

It is assumed that the hall will be built in wind area 2 in a built environment, so in terrain category 3. The basic wind velocity represents the wind velocity at 10m height and is calculated with

$$v_b = c_{dir} c_{season} v_{b,0}$$

Since in the Netherlands, $c_{dir} = 0$ and $c_{season} = 0$, this results in

$$v_b = v_{b,0} = 27\ m/s$$

Now the number of wind pressure planes on the windwards side is determined. The wind pressure can be either constant, divided in two pressures planes or in two constant pressure planes and a transition zone, see Figure 9.32.

$$h = 12.5 \leq b = 21.2$$

This means that only one pressure plane occurs, the top option in the figure.

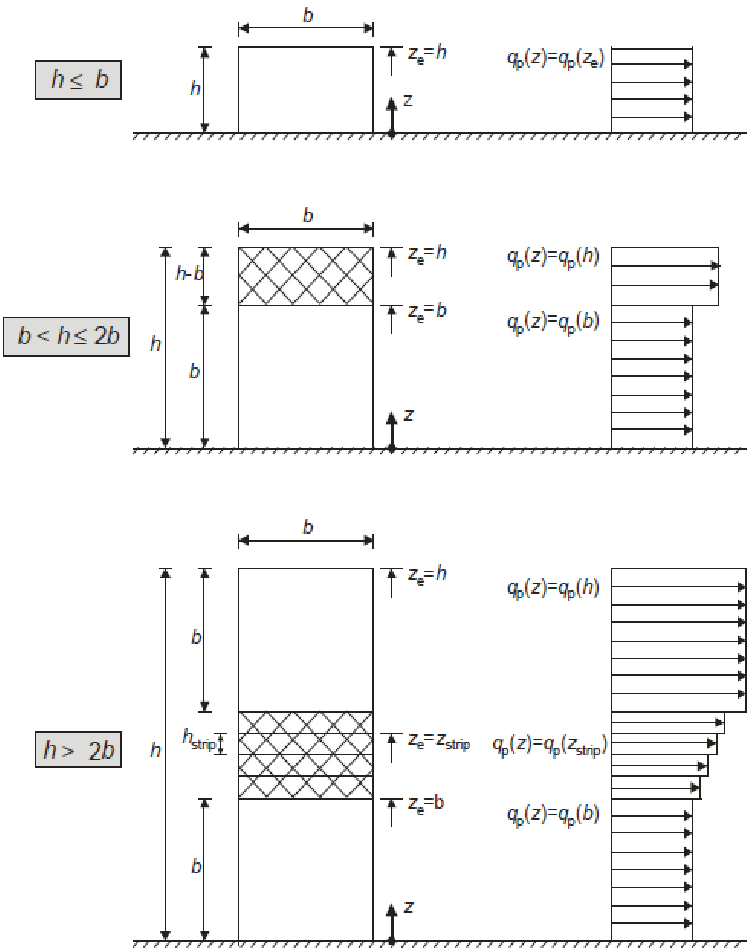

Figure 9.32: Wind pressure planes

The peak velocity pressure is determined for $z = H$. If the peak velocity pressure has a changing value over the building height it is assumed that the maximum acting peak velocity pressure is acting over the entire height of the building to ensure that the floor slab suffices when it is placed on the first floor but also when it is placed on the top floor.

$$q_p(z) = [1 + 7l_v(z)] * \frac{1}{2}\rho v_m^2(z) = [1 + 7 * 0.31] * \frac{1}{2} * 1.25 * 19.4^2 = 0.75\ kN/m^2$$

In which

$l_v(z)$ is the turbulence intensity,
$\rho = 1.25\ kg/m^3$ is the air density,
$v_m(z)$ is the mean wind velocity at height z.

The mean wind velocity is calculated with

$$v_m(z) = c_r(z)c_o(z)v_b = 0.72 * 1 * 27 = 19.4\ m/s$$

In which

$c_r(z)$ is the roughness factor;
$c_o(z)$ is the orography factor.

The assumption is made that the wind velocity is not increased due to the orography of the landscape, so $c_o(z) = 1$.

The roughness factor is determined by

$$c_r(z) = \begin{cases} k_r \ln\left(\frac{z}{z_0}\right) & for\ z_{min} \le z \le z_{max} \\ c_r(z_{\min}) & for\ z \le z_{min} \end{cases}$$

- $z = H = 12.5\ m$
- $z_{max} = 200\ m$
- $z_0 = 0.5\ m$
- $z_{min} = 7\ m$
- $z_{0,||} = 0.05\ m$

$$k_r = 0.19\left(\frac{z_0}{z_{0,||}}\right)^{0.07} = 0.19\left(\frac{0.5}{0.05}\right)^{0.07} = 0.22$$

$$c_r(z) = 0.22 * \ln\left(\frac{12.5}{0.05}\right) = 0.72$$

The turbulence intensity is determined with

$$l_v(z) = \begin{cases} \dfrac{k_|}{c_o(z)\ln\left(\frac{z}{z_0}\right)} & for\ z_{min} \le z \le z_{max} \\ l_v(z_{\min}) & for\ z \le z_{min} \end{cases}$$

Using $k_| = 1$ results in

$$l_v(z) = \frac{1}{1 * \ln\left(\frac{12.5}{0.05}\right)} = 0.31$$

The internal and external wind pressures are calculated by multiplying the peak velocity pressure with a pressure coefficient. The external pressure has a constant value on the windward (D) and leeward (E) side but on the facades perpendicular to these faces, the pressure can vary. Depending on the value of $e = \min(b, 2h)$ compared to d, the amount of different pressure zones can be found, see Figure 9.33.

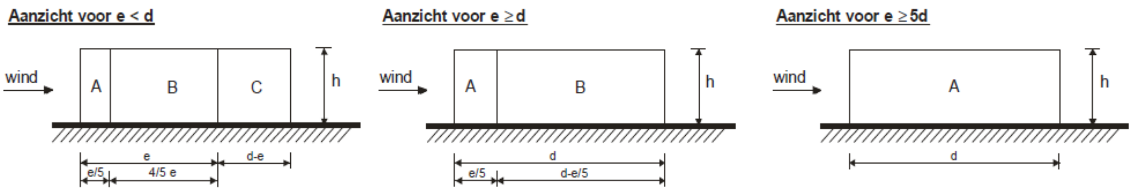

Figure 9.33: Amount of pressure zones

$$e = \min(21.2; 25.0) = 21.2\ m$$
$$e = 21.2 \ge d = 10.9$$

Two pressure planes occur at the head ends of the building. The pressure coefficients can be found in the national annex:

$$c_{pe,10,A} = -1.2$$
$$c_{pe,10,B} = -0.8$$
$$c_{pe,10,D} = \ \ 0.8$$

The pressure coefficient on side E depends on the factor $\frac{h}{d}$

$$c_{pe,10,E} = \begin{cases} -0.5 & \frac{h}{d} \le 1 \\ -0.7 & \frac{h}{d} = 5 \end{cases}$$

If the factor is in between the two, linear interpolation should be used. Here $\frac{h}{d} = \frac{12.5}{10.9} = 1.1$. This results in

$$c_{pe,10,E} = -0.51$$

The external wind pressure on side D is determined by combining the pressure on side D and the suction on side E. The lack of correlation between these pressures can be taken into account when combining them. The resulting force may be multiplied with a factor 0.85.

$$q_{w,D} = \left[c_{pe,10,D}q_p(z) - c_{pe,10,E}q_p(z)\right]0.85 = [0.8 * 0.75 + 0.51 * 0.75]0.85 = 0.83\ kN/m^2$$

The internal pressure coefficients are dependent on the openings in the building. Because the opening ratio can't be determined accurately, the internal pressure coefficients are taken as

$$c_{pi,pos} = \quad 0.2$$
$$c_{pi,neg} = -0.3$$

When the total area of the planes parallel to the wind direction is smaller than four times the total area of the planes perpendicular to the wind direction, the effects of wind friction can be neglected. This holds true in this case:

$$A_{par} = 273 < 4 * A_{perp} = 4 * 531$$

The external wind pressure acting on plane D results in a global bending moment and shear force on the structure. They are illustrated in Figure 9.34.

$$M_{Ed,L} = \frac{1}{8} * q_{w,D} * b^2 = \frac{1}{8} * 0.83 * 21.2^2 = 149.5\ kNm$$
$$V_{Ed,L} = \frac{1}{2} * q_{w,D} * b = 28.2\ kN$$

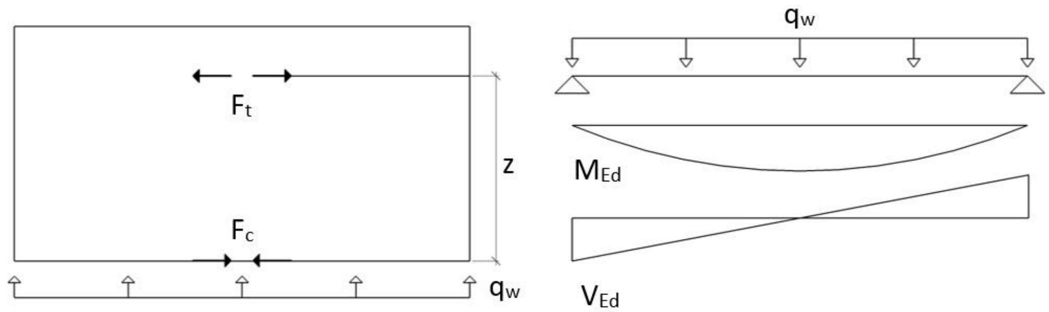

Figure 9.34: Wind on long side, resulting forces

For the determination of the global forces acting in the floor field, the field is regarded as a simply supported beam. A check is done to determine whether the beam is a deep beam: $b = 21.2 < 3d = 3 * 10.9 = 32.7$ so the beam is a deep beam. The acting moment results in a compressive and tensile force acting in the floor field and/or in the steel edge beams as shown in Figure 9.34. The internal lever arm between the tensile and compressive force, and the resulting forces become

$$z = 0.2 * b + 0.4 * d = 8.6 \leq 0.6b = 12.7\ m$$
$$F_{t,L} = F_{c,L} = \frac{M_{Ed}}{z} = \frac{149.5}{8.6} = 17.4\ kN$$

The same calculation is done for wind acting on the short side only now the floor field is simplified as a cantilevering beam, see Figure 9.35. Again, a check is done to determine whether the beam is to be regarded as a deep beam:

$b = 10.9 < 3d = 3 * 21.2 = 63.6$ so the beam is a deep beam.

$$q_{w,D} = [c_{pe,10,D}q_p(z) - c_{pe,10,E}q_p(z)]0.85 = [0.8 * 0.75 + 0.48 * 0.75]0.85 = 0.81\ kN/m^2$$

$$M_{Ed,S} = \frac{1}{8} * q_{w,D} * b^2 = \frac{1}{8} * 0.81 * 10.9^2 = 154.7\ kNm$$

$$V_{Ed,S} = \frac{1}{2} * q_{w,D} * b = 28.4\ kN$$

$$z = 0.2 * b + 0.4 * d = 10.7 \leq 0.6b = 6.5\ m$$

$$F_{t,S} = F_{c,S} = \frac{M_{Ed}}{z} = \frac{28.4}{6.5} = 23.7\ kN$$

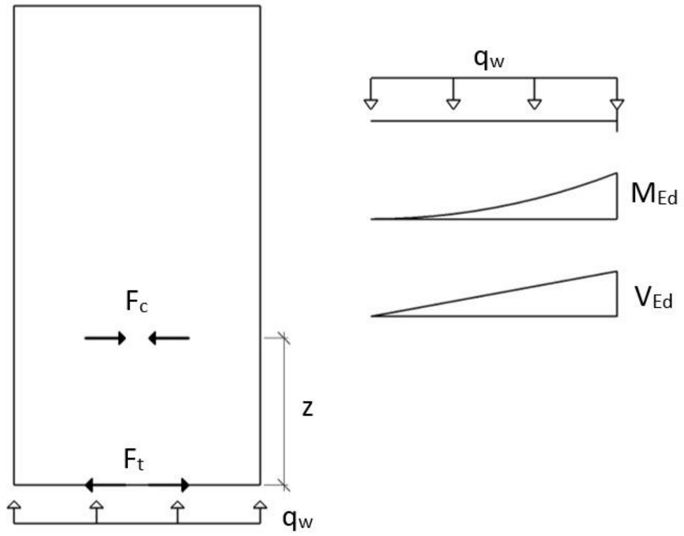

Figure 9.35: Wind on short side, resulting forces

Lastly the forces acting in the plane of the connections are determined. In Figure 9.36 the two combinations of internal and external pressures that have to be regarded are shown. The normative load is determined from these two combination options.

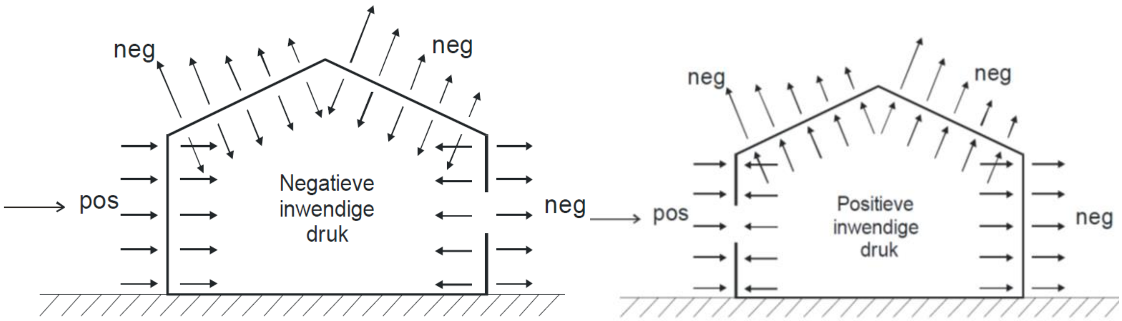

Figure 9.36: Combination internal and external wind pressures

Figure 9.37 shows the internal and external wind pressures acting on the building for wind on the long side of the building. The red load represents negative internal pressure and the green load represents positive internal pressure.

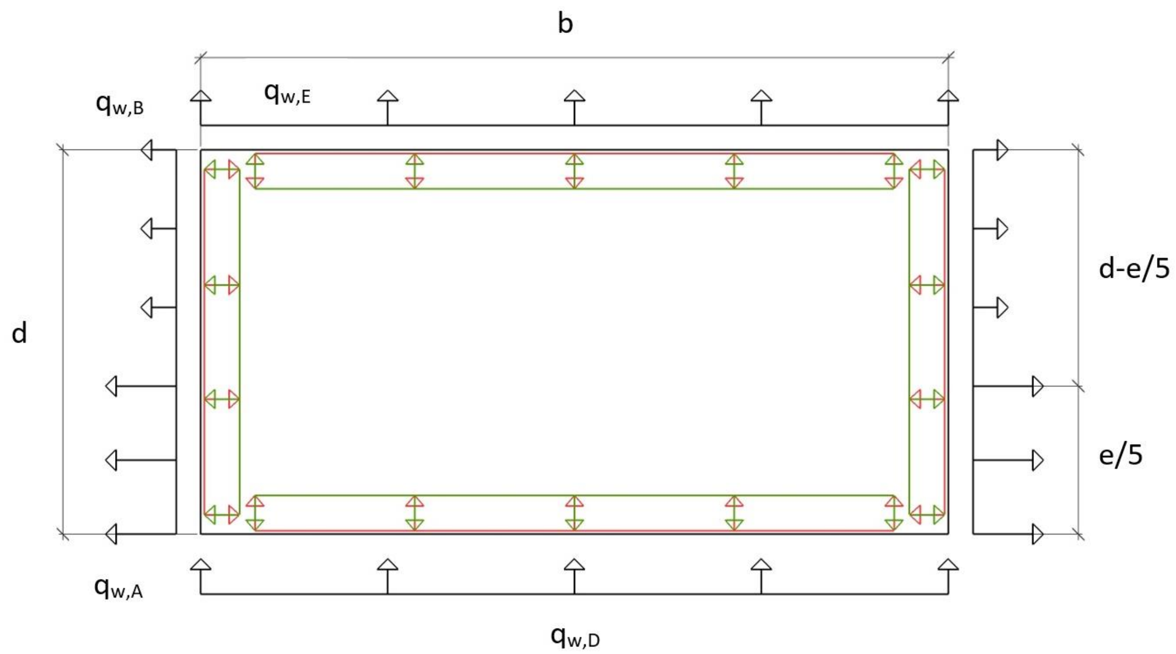

Figure 9.37: Internal and external wind pressures, wind on long side

The maximum total acting force on the sides is found for positive internal pressure

$$F_{side,L} = \left[\frac{e}{5}q_{w,A} + \left(d - \frac{e}{5}\right)q_{w,B} + q_{wi}d\right]h_{storey} = 30.2\ kN$$

The maximum compressive force acing on one slab width occurs when negative internal pressure is acting on side D. The maximum tensile force acing on one slab width occurs when positive internal pressure is acting on side E

$$F_{c,L} = \left[\left(c_{pe,10,D} - c_{pi,neg}\right)q_p(z)\right]h_{storey}W_{slab} = 4.7\ kN$$
$$F_{t,L} = \left[\left(-c_{pe,10,E} + c_{pi,pos}\right)q_p(z)\right]h_{storey}W_{slab} = 3.0\ kN$$

The same can be done for wind on the short side. In this situation three pressure planes occur on the long sides. The length over which the different wind pressures act can be found in Figure 9.33. The maximum total acting force on the sides is found for positive internal pressure

$$F_{side,S} = \left[\frac{e}{5}q_{w,A} + \frac{4}{5}eq_{w,B} + (d - e)q_{w,C} + q_{wi}d\right]h_{storey} = 58.0\ kN$$

The maximum compressive force acing on the head end occurs when negative internal pressure is acting on side D. The maximum tensile force acing on the head end occurs when positive internal pressure is acting on side E

$$F_{c,L} = \left[\left(c_{pe,10,D} - c_{pi,neg}\right)q_p(z)\right]h_{storey}L_{slab} = 28.7\ kN$$
$$F_{t,L} = \left[\left(-c_{pe,10,E} + c_{pi,pos}\right)q_p(z)\right]h_{storey}L_{slab} = 17.7\ kN$$

9.3.3 Verification slab-beam connection, head end

The chosen slab-beam connection at the head end of the slabs is made of several components. All the components have to be verified in ULS and one in SLS. Per component the acting forces are determined after which the resistances are calculated and the unity checks are given. 11 full plates and 2 fitting plates are used which results in a total of 35 toothed-plate connectors used on each long side of the building.

Edge beam

The edge beam will be verified in ULS for the longitudinal and transverse bending moments, shear force and tensile or compressive stresses, and in SLS for the deflection. Figure 9.38 shows the L-section beam. The maximum possible eccentricity of the vertical force is determined by first determining the minimum possible bearing length of the timber beams:

$$l_{min} = b_{fl,out} - t_{turn} - 0.5\Delta L_{slab} = 200 - 30 - 2.5 = 167.5\ mm$$

$$e_1 = \frac{1}{2}l_{min} + \frac{1}{2}t_w + t_{turn} + \Delta L_{slab} = 83.8 + 9 + 30 + 5 = 127.8\ mm$$

The eccentricity of the compression bolt

$$e_2 = h_w - \frac{1}{2}t_{fl} - e_{bolt} = 528 - 9 - 40 = 479.0\ mm$$

The distance of the neutral axis (na) to the bottom of the beam

$$NC_{b1} = \frac{h_w * t_w * \left(\frac{1}{2}h_w + t_{fl}\right) + b_{fl,out} * t_{fl} * \left(\frac{1}{2}t_{fl}\right)}{A} = 193.9\ mm$$

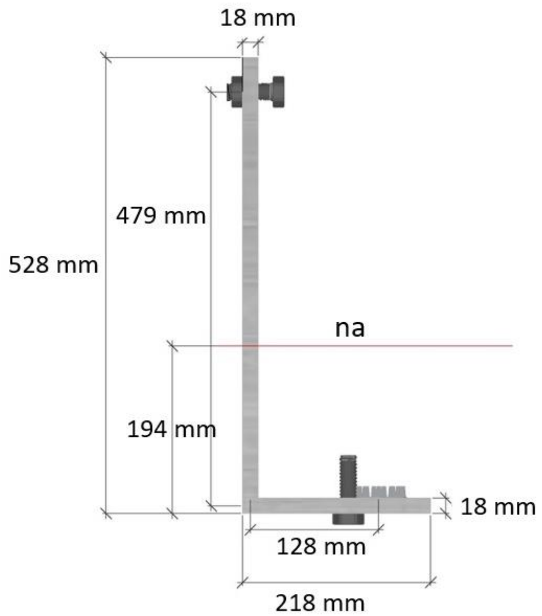

Figure 9.38: Dimensions L-section

To determine whether a plastic or elastic calculation can be used, the cross section class is determined. For an open section like this one the class of the web determines the class of the entire cross section. The found values result in a cross section class 2.

$$c_w = h_w - a_{weld} = 510 - 10 = 500\ mm$$

$$\frac{c_w}{t_w} = \frac{500}{18} = 27.8 \leq \frac{10\varepsilon}{\alpha} = \frac{10}{0.34} = 29.4$$

Now the acting loads will be determined. The floor slab rests on the edge beam with three timber beams. These result in three point loads on the flange. A distributed load is also present, which represents the self-weight of the beam. Figure 9.39 shows the mechanical scheme of the beam with the loads acting on it.

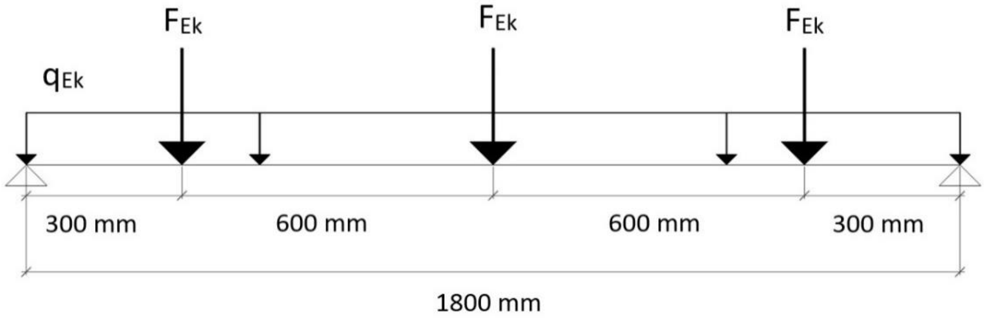

Figure 9.39: Mechanical scheme edge beam

The point loads are all equal and consist of a permanent part and a variable part.

$$G = q_{duct} + q_{top\ floor\ \&\ ceiling} + q_{sw} = 0.25 + 0.25 + 2.34 = 2.94\ kN/m^2$$
$$Q = q_{walls} + q_{imposed} = 1 + 2.5 = 3.5\ kN/m^2$$
$$F_{Ek,1,G} = F_{Ek,2,G} = F_{Ek,3,G} = F_{Ek,G} = G * b_{ef,c} * \frac{1}{2} * L_s = 2.94 * 0.6 * 5.4 = 9.5\ kN$$
$$F_{Ek,Q} = Q * b_{ef,c} * \frac{1}{2} * L_s = 3.5 * 0.6 * 5.4 = 11.3\ kN$$
$$q_{Ek,G} = \frac{A_{gir} * \rho_s}{100} = \frac{13104 * 10^{-6} * 7850}{100} = 1.0\ kN/m$$

The normative moment occurs at mid-span and the normative shear force at the supports. They are determined by calculating the moment and shear force with all the possible load combinations:

$$M_{Ed,long} = 21.8\ kNm$$
$$V_{Ed,long} = 33.3\ kNm$$

<u>Transverse bending</u>

Figure 9.40 shows the cross section of the edge beam, the eccentric loading, and the transverse bending moment distribution over the cross section.

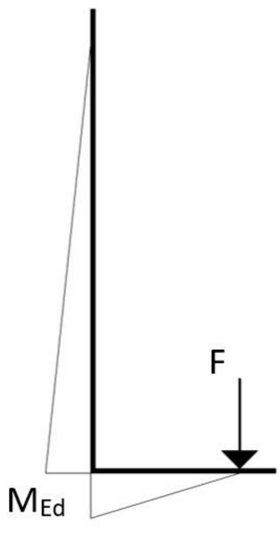

Figure 9.40: Transverse bending L-section

The load acting on the bottom flange due to one timber beam:

$$F = 0.89 * 1.35 * F_{Ek,G} 1.5 * F_{Ek,Q} = 28.4\ kN$$

The acting bending moment

$$M_{Ed} = 3F * e_1 = 85.3 * 0.128 = 10.9\ kNm$$

The bending moment resistance of the flange and the web

$$M_{Rd} = W_{pl} f_{yd} = \frac{1}{4} * L_{gird} * t_{fl}^2 = 235 * \frac{1}{4} * 1800 * 18^2 = 34.3\ kNm$$

The resulting unity check for transverse bending

$$UC = \frac{10.9}{34.3} = 0.32 \quad \rightarrow Satisfied$$

The compression bolt will also result in transverse bending of the L-section. This bending acts in the opposite direction of the bending caused by the eccentric load. The compressive force inducing this other bending moment is smaller than the eccentric load. Therefor the actual acting moment will be lower, but will never exceed the value used here. The calculation is therefore safe and doesn't have to be refined by including the other bending moment.

<u>Longitudinal bending, mid span</u>

Since transverse and longitudinal bending occur at the same time, the area available for the resistance of the longitudinal moment has to be reduced by the area required for the transverse bending moment and the shear force. These areas are determined with help of [115]. Figure 9.41 shows the cross section of the edge beam with the areas required for transverse actions, they are not to scale. The blue areas are for shear forces and the red area is for the bending moment. First the required areas for the shear force are determined. The required area over 1 m' in the length of the beam is determined by rewriting the formula to determine the plastic shear resistance:

$$V_{pl,Rd} = \frac{A_v f_{yd}}{\sqrt{3}\gamma_{M0}} \rightarrow A_v = \frac{3F\sqrt{3}\gamma_{M0}}{f_{yd}} = t_\tau$$

The thickness of the area, t_τ, when regarding the cross section instead of the longitudinal system is the same as the area. The area in the cross section now becomes for the flange and the web respectively:

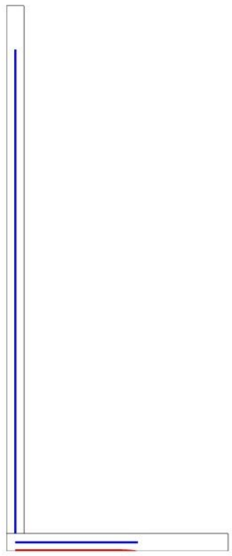

Figure 9.41: L-section with reduced areas due to transverse actions

$$A_{\tau,fl} = e_1 t_\tau = e_1 \frac{3F\sqrt{3}\gamma_{M0}}{f_{yd}} = 127.8 * 0.6 = 80.4\ mm^2$$

$$A_{\tau,w} = e_2 t_\tau = e_2 \frac{3F\sqrt{3}\gamma_{M0}}{f_{yd}} = 479.0 * 0.6 = 301.3\ mm^2$$

The determination of the area required for bending is more elaborate than for the area required for the shear force because the area has a partly curved shape. Figure 9.42 shows a zoomed in part of the bottom flange.

Figure 9.42: Zoomed in bending area

The acting moment results in tensile stresses at the top and compressive stresses at the bottom of the flange. When setting the internal moment equal to the external moment (for variable y), a formula results for the thickness development of the bending area.

$$F(e_1 - y) = t_\sigma f_{yd}\left(t_{fl} - t_\sigma\right) \rightarrow t_\sigma = \frac{1}{2}t_{fl} - \sqrt{\frac{1}{4}t_{fl}^2 - \frac{F(e_1 - y)}{f_{yd}}} = 1.4\,mm$$

Integrating the function for the thickness of the area results in the area.

$$A_\sigma = \frac{1}{2}t_{fl}e_1 - \frac{1}{12(3F)}t_{fl}^2 f_{yd}\left[1 - \sqrt{\left(1 - \frac{4e_1(3F)}{t_{fl}^2 f_{yd}}\right)^3}\right] = 185.2\,mm^2$$

It is assumed that the area is a triangle and that the distance from the centre of the bending area to the bottom of the flange is

$$\alpha = \frac{1}{3}t_\sigma = 0.5\,mm$$

The halve-line is determined after which the distance of the normal force centre to the bottom of the flange can be determined

$$z = \frac{h_w t_w - b_{fl}t_{fl} + A_\tau + A_\sigma}{2t_w} = 153.4\,mm$$

$$NC_{b2} = z + t_{fl} = 171.4\,mm$$

The reduced moment resistance becomes

$$M_{Rd} = \left[\frac{1}{2}(h_{tot} - NC_{b2})^2 t_w + \frac{1}{2}\left(NC_{b2} - t_{fl}\right)^2 t_w + \left(b_{fl}t_{fl} - A_\tau\right)\left(NC_{b2} - \frac{1}{2}t_{fl}\right) - A_\sigma(NC_{b2} - \alpha)\right] f_{yd} = 436.0\,kNm$$

The unity check follows

$$UC = \frac{21.8}{436.0} = 0.05 \quad \rightarrow Satisfied$$

Shear force, supports

A plastic calculation method is adopted and therefor the area resisting shear can be determined by

$$A_v = \eta * h_w t_w = 1.2 * 510 * 18 = 11016\,mm^2$$

The shear resistance is now determined as

$$V_{pl,Rd} = \frac{A_v * f_{yd}/\sqrt{3}}{\gamma_{M0}} = \frac{11016 * 235/\sqrt{3}}{1.0} = 1494.6\,kN$$

The resulting unity check

$$UC = \frac{33.3}{1494.6} = 0.02$$

Deflection

Lastly the initial and additional deflections are determined. To do so the bending stiffness is first ascertained.

$$EI = E_s\left[\frac{1}{12} * t_w * h_w^3 + t_w h_w\left(\frac{1}{2}t_w + t_{fl} - NC_{b1}\right)^2 + \frac{1}{12}b_{fl,o}t_{fl}^3 + b_{fl,o}t_{fl}\left(NC_{b1} - 0.5t_{fl}\right)^2\right]$$
$$= 21000 * 3.9 * 10^8 = 8.2 * 10^{13}\ Nmm^2$$

The maximum initial and additional deflections

$$w_{ini,max} = \frac{1800}{300} = 6\ mm$$
$$w_{add,max} = \frac{3 * 1800}{1000} = 5.4\ mm$$

The deflection of the two outer point loads is equal. The distance from the left support to the first point load and the distance from this point load to the right support and the following deflection

$$a_1 = 300\ mm$$
$$b_1 = 1500\ mm$$
$$w_{F1} = w_{F3} = \frac{F_1 * b_1 * \frac{1}{2}L_{gird}}{6 * EI * L_{gird}}\left[L_{gird}^2 - b_1^2 - \left(\frac{1}{2}L_{gird}\right)^2\right] = 0.006\ mm$$

The deflection due to the middle point load and the self-weight of the beam and the final initial deflection

$$w_q = \frac{5}{384}\frac{q_{Ek,G} * L_{gird}^2}{EI} = 0.002\ mm$$
$$w_{F2} = \frac{F_2 L_{gird}^3}{48\ EI} = 0.031\ mm$$
$$w_{ini} = 0.02 + 0.06 + 0.06 + 0.31 = 0.044\ mm$$

The additional deflection is determined just like the initial deflection but now only the variable part of the loading that is used. This results in

$$w_{F1} = w_{F3} = 0.03\ mm$$
$$w_{F2} = 0.017\ mm$$
$$w_{add} = 0.023\ mm$$

The unity checks for the deflections follow

$$UC = \frac{0.044}{6} = 0.007 \quad \rightarrow Satisfied$$
$$UC = \frac{0.023}{5.4} = 0.004 \quad \rightarrow Satisfied$$

Toothed-plate connector

On the toothed-plate connector shear forces act in different directions. Longitudinal to the floor beams, a shear force is induced due to the tensile component of the bending moment induced by the eccentric load on the edge beam. The shear force due to the permanent part and due to the variable part of the floor loading are determined separately

$$F_{v,Ek,G,//} = \frac{F_{Ek,G} e_1}{e_2} = \frac{9.5 * 10^3 * 127.8}{479.0} = 2.5\ kN$$

$$F_{v,Ek,Q,//} = \frac{F_{Ek,Q} e_1}{e_2} = \frac{11.3 * 10^3 * 127.8}{479.0} = 3.0\ kN$$

The global wind loads are determined in paragraph 9.3.2. Now the loads acting on the toothed-plate have to be determined. The acting loads are determined for wind on the long side and for wind on the short side separately. A force indicated with subscript ⊥ acts perpendicular to the timber beams, a force indicated with subscript // acts longitudinal to the timber beams. Forces with subscript L arise due to wind on the long side of the building, forces with subscript S arise due to wind on the short side of the building:

$$F_{v,Ek,L,\perp} = \frac{F_{t,L}}{\#\ connectors} = \frac{17.4 * 10^3}{3 * 11 + 2} = 0.5\ kN$$

$$F_{v,Ek,L,//} = \frac{F_{t,L}}{\#\ connectors} = \frac{3.0 * 10^3}{3} = 1.0\ kN$$

$$F_{v,Ek,S,\perp} = \frac{V_{Ed,S}}{\#\ connectors} = \frac{28.4 * 10^3}{3 * 11 + 2} = 0.8\ kN$$

$$F_{v,Ek,S,//} = \frac{F_{side,S}}{\#\ connectors} = \frac{58.0 * 10^3}{3 * 11 + 2} = 1.7\ kN$$

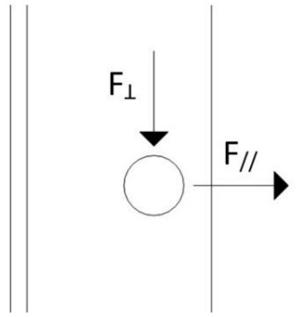

Figure 9.43: Forces on toothed-plate

The characteristic loads are all known so the normative design load can be determined. The design loads are determined for wind on the long side and for wind on the short side of the building. Figure 9.43 shows the separate loads in the two different directions. As a simplification the loads in the different directions are only added and not decomposed into the final direction of the total shear load. This is a safe simplification because the total load decreases if a decomposition is done.

$$F_{v,Ed,L} = \max \begin{cases} 1.35 * 2.5 + 1.5 * 0.5 * 3.0 + 1.5 * 0 * (0.5 + 1.0) = 5.7\ kN \\ 0.89 * 1.35 * 2.5 + 1.5 * 0.5 * 3.0 + 1.5 * (0.5 + 1.0) = 7.6\ kN \\ 0.89 * 1.35 * 2.5 + 1.5 * 3.0 + 1.5 * 0 * (0.5 + 1.0) = 7.6\ kN \end{cases} = 7.6\ kN$$

$$F_{v,Ed,S} = \max \begin{cases} 1.35 * 2 + 1.5 * 0.5 * 2.4 + 1.5 * 0 * (0.8 + 1.7) = 5.7\ kN \\ 0.89 * 1.35 * 2 + 1.5 * 0.5 * 1.4 + 1.5 * (0.8 + 1.7) = 9.0\ kN \\ 0.89 * 1.35 * 2 + 1.5 * 1.4 + 1.5 * 0 * (0.8 + 1.7) = 7.6\ kN \end{cases} = 9.0\ kN$$

The determination of the resistance of the toothed-plate connector depends on the type used. Here type C11 is used. The diameter of the plate is given as d_c, length of the steel teeth embedded in the timber is denoted as h_e, the thickness of the timber member is $t_1 = t_2$.

$$F_{v,Rk} = 25k_1k_2k_3d_c^{1.5} = 25 * 1 * 0.75 * 1.26 * 65^{1.5} = 12.4\ kN$$

$$d_c = 65\ mm$$

$$h_e = h_{e,max} - \Delta h_e = 15 - 5 = 10\ mm$$

$$k_1 = \min\left[1; \frac{t_1}{3h_e}; \frac{t_2}{5h_e}\right] = \min[1; 14.3; 8.6] = 1$$

$$k_2 = \min\left[1; \frac{a_{3,t}}{2d_c}\right] = \min[1; 0.75] = 0.75$$

$$a_{3,t} = \max[1.5d_c; 80] = 97.5\ mm$$

$$k_3 = \min\left[1.5; \frac{\rho_k}{350}\right] = \min[1.5; 1.26] = 1.26$$

The design resistance results

$$F_{v,Rd} = \frac{12.4}{1.25} = 9.9\ kN$$

The unity check can now be found

$$UC = \frac{9.0}{9.9} = 0.91 \quad \rightarrow Satisfied$$

The UC is below 1 so the simplified way for determining the acting load without decomposition is fine for this initial design verification.

Timber beams

The floor slab is supported on the edge girder by placing the timber beams on top of the bottom flange. At these positions bearing stresses will occur, resulting in compression perpendicular to the grain in the timber. This acting stress should not exceed the compressive strength of the timber. The acting stress and the strength

$$\sigma_{c,90,t} = \frac{V_{Ed}}{l_b b_t} = \frac{28400}{169.5 * 140} = 1.2\ MPa$$

$$f_{c,90,d} = \frac{0.8 * 2.5}{1.25} = 1.6\ MPa$$

The resulting unity check for the compressive stress perpendicular to the grain of the timber

$$UC = \frac{1.2}{1.6} = 0.75 \quad \rightarrow Satisfied$$

In the design it is assured that the compression bolt presses against the concrete deck and not the timber beams, therefor the compressive stress parallel to the grain of the timber doesn't have to be verified.

9.3.4 Verification slab-slab connection

The only forces acting on the slab-slab connection are wind loads. The detail forces will be determined from the global wind loads found in paragraph 9.3.2. Only wind on the long side results in forces on the slab-slab connection. Figure 9.44 shows the forces acting on the connection. Six connections will be used over the length of the slabs. The reinforcing bar that is used has a diameter of 10mm.

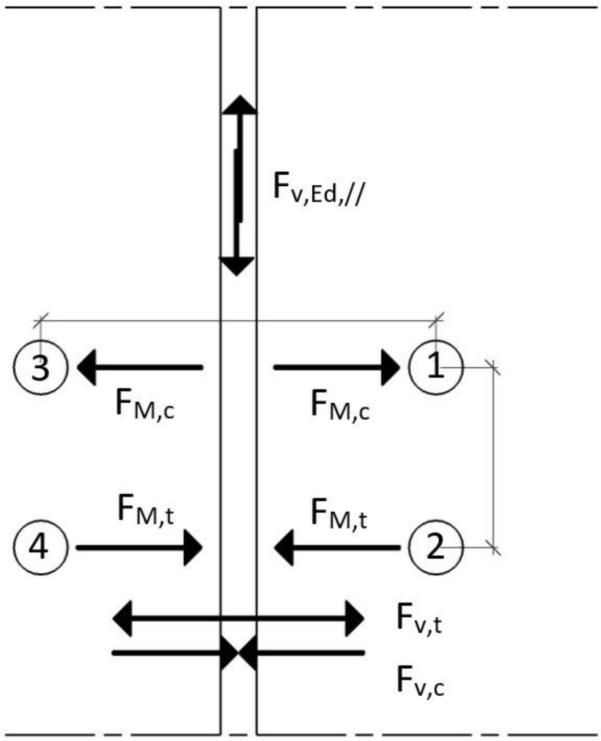

Figure 9.44: Forces on the slab-slab connection

$V_{Ed,L}$ acts as a shear force between the plates due to the diaphragm action of the floor field. The characteristic value has already been determined as $V_{Ed,L} = 28.2\ kN$. This shear force induces a bending moment on the bots due to its eccentric position. The design shear force per connection and the tensile and compressive component of the moment are

$$F_{v,Ed,//} = \frac{1.5V_{Ed,L}}{6} = 7.1\ kN$$

$$F_{M,c} = F_{M,t} = \frac{F_{v,Ed,//}e_A}{2e_B} = \frac{7.1 * 0.106}{2 * 0.1} = 3.7\ kN$$

The moment acting in the floor field can results in a tensile and a compressive force in the floor field. They are transferred in the transverse direction through the floor field and act in the connection as shear forces through the bolts, as bearing on the plate, as compression or tension on the concrete slab and as a tensile force in the reinforcement. It is assumed that this force is distributed over all the connections between the floor slabs in one line. 11 full plates and 2 fitting plates are used so 12 connections are present over the length. The tensile and compressive components of the bending moment have already been determined.

$$F_{v,t,Ed} = F_{v,c,Ed} = F_{t,Ed,rein} = \frac{1.5F_t}{12} = 2.2\ kN$$

In Figure 9.45 and Figure 9.46 the forces acting on the bolts are shown. Because the forces on bolt 1 & 3 and on bolt 2 & 4 are equal and result in the same type of loading on the slab (tension or compression) only the forces on bolt 1 and bolt 2 are shown and determined.

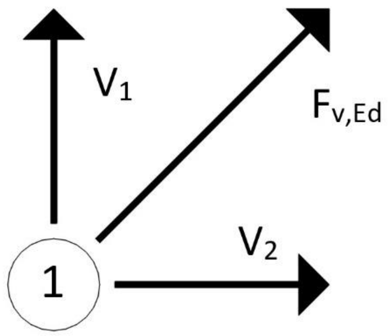

Figure 9.45: Forces on bolt 1&3, slab-slab

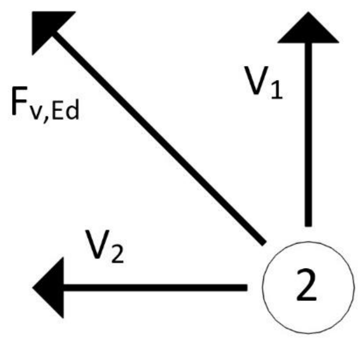

Figure 9.46: Forces on bolt 2&4, slab-slab

Bolt 1:

$$V_1 = F_{v,Ed,//} = 7.1\ kN$$
$$V_2 = F_{M,c} + F_{v,c,Ed} = 5.9\ kN$$
$$F_{v,Ed} = \sqrt{V_1^2 + V_2^2} = 9.2\ kN$$

Bolt 2:

$$V_1 = F_{v,Ed,//} = 7.1\ kN$$
$$V_2 = F_{M,t} + F_{v,t,Ed} = 5.9\ kN$$
$$F_{v,Ed} = \sqrt{V_1^2 + V_2^2} = 9.2\ kN$$
$$F_{t,Ed,conc} = V_2 = 5.9\ kN$$

The shear resistance of the bolt, the bearing resistance of the plate and the tensile resistance of the reinforcement are

$$F_{v,Rd} = \frac{\alpha_v f_{ub} d}{\gamma_{M2}} = \frac{0.6 * 800 * 16}{1.0} = 77.2\ kN$$
$$F_{b,Rd} = \frac{\alpha_b k_1 f_{up} t_p d}{\gamma_{M2}} = \frac{0.56 * 2.5 * 360 * 10 * 16}{1.0} = 64.0\ kN$$
$$F_{t,Rd,rein} = A_{reinf} * f_{yd} = 78.5 * 235 = 18.5\ kN$$

The resistance of the concrete to breaking when a tensile force is acting on the side of the slab is determined from [116]. In this document the found resistance is

$$F_{t,Rd,conc} = 12.4\ kN$$

Finally, the unity checks for shear of the bolt, bearing of the plate, tension in the reinforcement and tension on the concrete are given by respectively

$$UC = \frac{9.2}{77.2} = 0.12$$
$$UC = \frac{7.1}{64.0} = 0.11$$
$$UC = \frac{2.2}{18.5} = 0.12$$
$$UC = \frac{5.9}{12.4} = 0.48$$

9.3.5 Verification slab-beam connection, side

The only forces acting on the slab-slab connection are wind loads. The detail forces will be determined from the global loads found in paragraph 9.3.2. Wind forces act on this detail when wind is acting on the long side and when wind is acting on the short side of the building. Both are determined and the normative values are used in the unity checks. Figure 9.47 shows the forces acting on the connection. Just like for the slab-slab connection, six connections are used over the slab length.

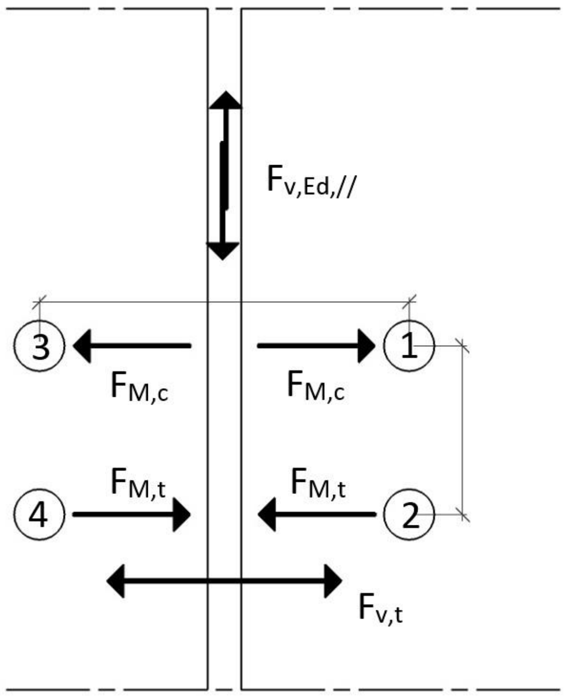

Figure 9.47: Forces on slab-beam connection

First forces arising due to wind on the long side are regarded. The global shear force, $V_{Ed,L}$, acts as a shear force between the beam and the floor plate. The characteristic value has already been determined as $V_{Ed,L} = 28.2\ kN$. This shear force induces a bending moment on the bots due to its eccentric position. The design shear force per connection and the tensile and compressive component of the moment are

$$F_{v,Ed,//} = \frac{1.5V_{Ed,L}}{6} = 7.1\ kN$$

$$F_{M,c} = F_{M,t} = \frac{F_{v,Ed,//}e_A}{2e_B} = \frac{7.1 * 0.106}{2 * 0.1} = 3.7\ kN$$

The tensile force acting on the head ends of the building is found when equating the internal and internal pressures. It results in a shear force in the perpendicular direction of the floor beams:

$$F_{v,Ed,\perp} = \frac{1.5F_{side,L}}{6} = \frac{1.5 * 30.2}{6} = 7.5\ kN$$

In Figure 9.48 and Figure 9.49 the forces acting on the bolts are shown. Because the forces on bolt 1 & 3 and bolt 2 & 4 are equal and result in the same type of loading on the slab (tension or compression) only the forces on bolt 1 and bolt 2 are shown and determined.

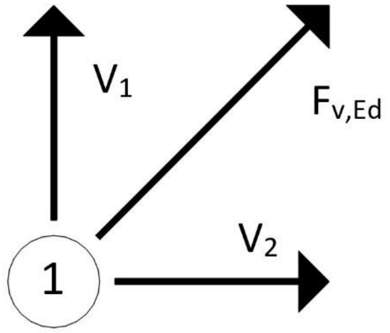

Figure 9.48: Forces on bolt 1&3, slab-beam

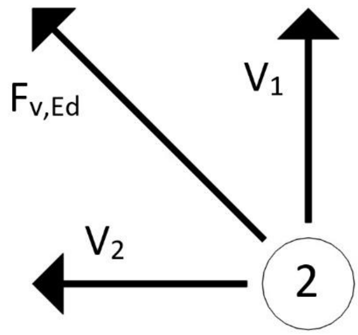

Figure 9.49: Forces on bolt 2&4, slab-beam

Bolt 1:

$$V_1 = F_{v,Ed,//} = 7.1\ kN$$
$$V_2 = F_{M,c} - F_{v,Ed,\perp} = -3.8\ kN$$
$$F_{v,Ed} = \sqrt{V_1^2 + V_2^2} = 8.0\ kN$$
$$F_{t,Ed,conc} = V_2 = 3.8\ kN$$

Bolt 2:

$$V_1 = F_{v,Ed,//} = 7.1\ kN$$
$$V_2 = F_{M,t} + F_{v,Ed,\perp} = 11.3\ kN$$
$$F_{v,Ed} = \sqrt{V_1^2 + V_2^2} = 13.3\ kN$$
$$F_{t,Ed,conc} = V_2 = 11.3\ kN$$

Now the forces acting in the toothed plate due to wind on the short side are determined. They are shown in Figure 9.50.

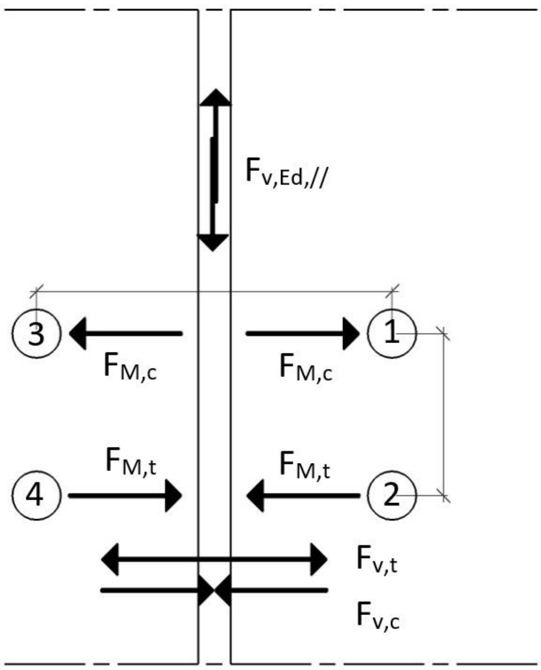

Figure 9.50: Forces on slab-beam connection, wind acting on the short side

The tensile component of the bending moment acting on the floor field acts as a shear force between the beam and the plate. The characteristic value has already been determined as $F_{t,S} = 23.7\ kN$. This shear force induces a bending moment on the bots due to its eccentric position. The design shear force per connection and the tensile and compressive component of the moment are

$$F_{v,Ed,//} = \frac{1.5F_{t,S}}{6} = \frac{1.5 * 23.7}{6} = 5.9\ kN$$

$$F_{M,c} = F_{M,t} = \frac{F_{v,Ed,//}e_A}{2e_B} = \frac{5.9 * 0.106}{2 * 0.1} = 3.1\ kN$$

The tensile and compressive forces resulting from equating the internal and external wind pressures also act on the connection. These result in shear forces on the bolts, bearing on the plates, tension on the concrete edge and tension in the reinforcement.

$$F_{v,t,Ed} = F_{t,Ed,rein} = \frac{1.5F_{t,L}}{6} = \frac{1.5 * 17.7}{6} = 4.4\ kN$$

$$F_{v,c,Ed} = \frac{1.5F_{c,L}}{6} = \frac{1.5 * 28.7}{6} = 7.2\ kN$$

The forces acting on the bolts are similar to the ones for wind on the long side, they can be found in Figure 9.48 and Figure 9.49. Because the forces on bolt 1 & 3 and bolt 2 & 4 are equal and result in the same type of loading on the slab (tension or compression) only the forces on bolt 1 and bolt 2 are shown and determined.

Bolt 1:

$$V_1 = F_{v,Ed,//} = 5.9\ kN$$

$$V_2 = F_{M,c} + F_{v,c,Ed} = 10.3 kN$$

$$F_{v,Ed} = \sqrt{V_1^2 + V_2^2} = 11.9\ kN$$

Bolt 2:

$$V_1 = F_{v,Ed,//} = 5.9\ kN$$

$$V_2 = F_{M,t} + F_{v,t,Ed} = 7.6\ kN$$

$$F_{v,Ed} = \sqrt{V_1^2 + V_2^2} = 9.6\ kN$$

$$F_{t,Ed,conc} = V_2 = 7.6\ kN$$

The shear resistance of the bolt, the bearing resistance of the plate, the tensile resistance of the reinforcement and the resistance of the concrete to breaking are the same as in the slab-slab connection.

Finally, the unity checks for shear of the bolt, bearing of the plate, tension in the reinforcement and tension on the concrete are given by respectively

$$UC = \frac{13.3}{77.2} = 0.17$$

$$UC = \frac{11.3}{64} = 0.18$$

$$UC = \frac{2.2}{18.5} = 0.12$$

$$UC = \frac{11.3}{12.4} = 0.91$$