DESIGN OF A DEMOUNTABLE STEEL TIMBER FLOOR SYSTEM

Design Rules and Recommendations for the Application of a Demountable Steel Timber Floor System

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Master's Thesis

Faculty of Civil Engineering ang Geosciences

Delft University of Technology

TUDelft

Design of a Demountable Steel Timber Floor System

Design Rules and Recommendations for the application of a demountable steel timber floor system.

By

E. B. Paul

in partial fulfilment of the requirements for the degree of

Master of Science

in Civil Engineering

at the Delft University of Technology,

to be defended publicly on Tuesday, 31st May 2022 at 8:00 AM

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Preface

This thesis is the final deliverable of my graduation work at TU Delft for obtaining my master's degree in Structural Engineering - Steel and Composite Structures.

Before attending TU Delft, I was in many ways ignorant to delve into the aspects of sustainability, especially related to the construction industry. The emphasis that the Civil Engineering and Geosciences faculty (and the university) gave about issues such as combating climate change, and attaining the target of zero emissions, is what directed me to a graduation project in the direction of sustainable construction. In this thesis, steel-timber floor systems are investigated into, as an alternative to the conventional choices of floor systems using concrete and steel. By using timber obtained from sustainably managed forests, building with it can act as natural carbon sinks, storing the CO_2 until the end of its service life.

The project itself involved delving into various steel and timber construction products. Instead of conducting research into a specific detail or method, this thesis started with a broad literature study about various practical aspects of steel timber construction, and gradually focuses into the analysis of a specific steel timber floor system. Finally, the environment benefits of steel-timber construction is evaluated with the help of a Life Cycle Analysis.

This topic was conceived in collaboration with Bouwen met Staal, and they were very much involved in the decision-making process that led to finalizing this thesis. Bouwen met Staal is a consortium of like-minded civil engineers and architects, and as an organization aims for the promotion of steel construction, by actively participating in research in this field. Restrictions due to the Covid-19 pandemic meant that the work had to be done remotely. As a result, I conducted the work for this thesis from the comfort of my home in Kochi, India.

The thesis committee that supervised me during the tenure of this research consists of Prof. Dr. M. Veljkovic, Dr. Ir. G. P. J. Ravenshorst, Dr. Florentia Kavoura and Dr. H.M. Jonkers from Delft University of Technology, and Mic A. Barendsz and Ir. Jan-Pieter den Hollander from Bouwen met Staal. I would like to thank all members of the committee for their guidance and invaluable inputs that allowed to get through all the hurdles that I encountered while conducting this research. A special mention goes to Dr. M. P. Nijgh, who had helped me initially, until the end of his tenure at the university in April 2021.

Finally, I thank my family and friends for all their support, and for the grace of God Almighty.

Eldhose Boben Paul

Delft, May 2022

Abstract

With the effects of climate change being more and more frequent, the European union, and other governments world-wide are looking for sustainable alternatives for processes and products that are part of the daily requirements of people. Among these requirements, the construction industry plays a vital role, as it accounts for the consumption of about 50% of the total raw materials. In line with its targets of 100% recycling by 2050, the industry in Europe, and particularly in the Netherlands is shifting towards the use of circular and sustainable construction products. This thesis investigates into both these aspects: Circularity by addressing the requirements for demountable structural elements, thus providing scope for reusing, and Sustainability by addressing the impact of substituting concrete and steel with timber products.

Structural timber products obtained from sustainable forestry are considered as eco-friendly construction products. The benefits are two-fold: First of all, it is produced or grown naturally, and thus avoids the emission of harmful gases during production. Second, growing timber products helps in CO_2 storage for the duration of the technical service life of the product. Thus, increasing the market share of timber products in the construction industry can play a huge and decisive role to achieve the targets of sustainability.

The focus of this thesis is on steel-timber floor systems i.e., a floor system with timber slabs supported by steel beams. There are many timber products available that can be used as slabs, as a substitute to hollow core slabs and steel-concrete composite slabs, which are the conventional solutions for floor systems in the Netherlands (the former more than the latter). Owing to the disadvantage of timber in stiffness, coupled with the effects of creep, steel beams are considered for traversing larger spans.

From the plethora of steel and timber products that can be coupled together to form a floor system, the best solution is obtained with the help of a Multi Criteria Analysis. This is done by scoring the different floor systems on the aspects of utility, circularity and sustainability. The obtained solution is a conventional non-integrated floor system with Lignatur surface elements as the slab, supported by steel I beams. Lignatur elements are box-shaped slabs that can span over large distances typically required for office use. Being made of sawn timber, it boasts the advantage of being more sustainable than other timber products made of cross laminated timber and laminated veneer lumber. I beams were found to perform better than other steel beams, posing as more accessible for demounting, over the other beams.

With the help of a case study, the benefits of the chosen steel-timber floor system were evaluated and compared against the hollow core slabs and steel-concrete composite slabs. Owing to the lightweight nature of timber, the chosen floor system was approximately 45% lighter than hollow core slabs, and 25% lighter than composite slabs. They were comparable in terms of floor height, to the composite slabs.

The main benefit was the fact that the use of concrete and/or steel could be substituted with the use of timber slabs, and the supporting steel frame could be lighter owing to the lightweight nature of timber. A life cycle analysis was done to compare the different floor systems. Due to the use of sustainably produced timber, the steel-timber floor system had the least environment impact. Considering the effects of carbon storage meant that the total environmental footprint of the floor system could be negative. On further analysis, by excluding the benefit of carbon storage, the

steel-timber floor system was still found to have the least impact (39% lesser than hollow core slabs, 31% lesser than steel-concrete composite slabs).

Another aspect that was investigated in this thesis was whether the consideration of composite action (similar to that in steel-concrete) could lead to any benefits i.e., composite action between the timber slab and the supporting steel beam. Based on the calculations in the case study, it was concluded that there were no such benefits i.e., the reduction in size of the steel beams was not enough to justify the use of added shear connectors.

Another limitation of timber was found to be its lack of reusability. Being a biomaterial, timber is associated with a large reduction in strength for each reuse (although this reduction cannot be quantified using the present state of the art). Other materials such as steel and concrete do not experience this strength loss, and thus assume to have better performance than timber regarding this aspect.

The main barrier for implementing a steel-timber floor system would be in terms of costs. Though not explicitly investigated into in this thesis, it is expected to be more than that for its conventional counterparts. Currently, hollow core slabs are the most widely used, in the Netherlands, which is mainly owed to its low costs. Thus, the circulation of a product is closely related to its costs in the market. By showing light on the benefits of the steel-timber floor system, it is expected that the results of this thesis will help the industry shift to the more responsible choice, in the near future. Wider circulation of steel-timber floors can help in reducing its costs, thus making it more desirable.

Contents

Appendices

Nomenclature

Abbreviations

CLT	Cross Laminated Timber
CLT_OR	CLT Open Rib Elements
CLT_SS	CLT Solid Slab Elements
EC	Eurocode
ECI	Environment Cost Indicator
EoL	End of Life
EPD	Environment Product Declaration
ETA	European Technical Assessment
FRP	Fibre Reinforced Polymers
FU	Functional Unit
GFA	Gross Floors Area
GLT	Glued Laminated Timber
GWP	Global Warming Potential
IFB	Internal Floor Beams
LCA	Life Cycle Assessment
LFE	Lignatur Surface Elements
LKE	Lignatur Box Elements
LVL	Laminated Veneer Lumber
MOE	Modulus of Elasticity
MPG	Milieuprestatie Gebouw
NMD	Dutch National Milieu Database
OSB	Oriented Strand Boards
PCR	Product Category Rules
PEF	Product Environment Footprint
SLS	Serviceability Limit State
ULS	Ultimate Limit State
RHS	Rectangular Hollow Sections
RSL	Reference Service Life of the Building
SBK	Stichting Bouwkwalitiet
SFB	Shallow Floor Beams
STC	Steel Timber Composite
STB	Steel Timber Bolted Connections
THQ	Top Hat Beams

Symbols

δ_{sc}	Shear Connector Slip
Δ	Deflection of Beam
$\epsilon_{T/S}$	Normal Strains in Timber/Steel
κ	Curvature of Beams
η	Degree of Partial Shear Interaction
ψ_i	Combination Factor for Imposed Loads
ρ_k	Characteristic Density of Timber
ρ_{mean}	Mean Density of Timber
$\sigma_{T/S}$	Bending Stresses in Timber/Steel
γ	Steel – timber Interface Cooperation Factor used in Gamma Method
$\gamma_{M,S}$	Steel Material Safety factor
$\gamma_{ULS/SLS}$	Load factors for ULS/SLS
$A_{T/S}$	Cross Sectional Area of Timber/Steel
$b_{T/S}$	Breadth of Timber/Steel members
d	Diameter of dowel
d_H	Bolt hole Clearance
e_{sc}	Edge distances of Shear Connectors
E	Young's Modulus
E_1	MOE in the grain direction of Timber
E_2	MOE in the direction perpendicular to grain of Timber
EI	Bending Stiffness
f_{md}	Design Bending Strength of Timber
f_y	Steel yield strength
F_t	Imposed Load for Reference Period t
F_{t0}	Imposed Load for Default Reference Period t_0
G	Dead Loads
$H_{T/S}$	Height of Timber/Steel members
k_{mod}	Timber strength reduction for duration of load
k_{ser}	Slip Modulus/Stiffness of Shear Shear Connection
K_{sc}	Smeared Stiffness of Shear Connectors
$L_{Beam/Slab}$	Span of Beams/Slab
N_{Ed}	Design Normal Forces
N_{sc}	Number of Shear Connectors
M_{Ed}	Design Bending Moments
M_{Rd}	Bending moment resistance
p_{sc}	Spacing of Shear Connectors
P_{Ed}	Design Loads on Shear Connectors
P_{Rd}	Design resistance of Shear Connectors
$Q_{q,k}$	Live Loads
$q_{SLS/ULS}$	Design loads in ULS/SLS
r	Number of times of reuse
r_H	Geometrical Deviations of Bolt Hole from its actual position
s_0	Initial Slip of STC due to Dead Loads
s_{sc}	Spacing of Shear Connectors
t	Reference Period of Load

t_0	$Default\ Reference\ Period\ of\ Load\ (50\ years)$
$t_{T/S}$	$Thickness\ of\ Timber/\ Steel\ members$
t_{tf}/t_{bf}	$Thickness\ of\ top\ and\ bottom\ flanges$
t_w	$Thickness\ of\ web$
V_{Ed}	$Design\ Shear\ Forces$
w	$Displacements/Deflections\ of\ Beam$
z	$Distance\ of\ NA\ of\ Sections\ from\ top/bottom$

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1. Introduction

Sustainable development is defined as development that meets the needs and aspirations of the present without compromising the ability to meet those of the future. The construction industry holds a large contribution to the deterioration of the environment, through consumption of large amounts of raw materials. Within the construction industry, floor systems are the largest consumer of raw materials. According to a survey in 2010 [143], the requirement for newly constructed buildings in Europe for the year 2023 is expected to be an area of 190 million m^2. Thus, floor systems cause the most amount of damage to the environment.

Thus, it is imperative to come up with alternate solutions to conventional construction materials such as concrete and steel, both of which are non-renewable. The solution presented here in this thesis is to use timber slabs as an alternative. Timber obtained from controlled forestry is a renewable resource i.e., as long as the amount of trees felled is controlled and they are regrown in a sustainable manner. Producing timber elements require less raw materials and energy, and it stores CO_2 during this process.

Apart from being a sustainable building material, timber has many other advantages over concrete and steel. It has a significantly lower density compared to the latter two and offers the benefit of lightweight construction. This, and the fact that timber is easily workable gives us the possibility of opting for prefabricated timber elements, thus boosting the speed of the construction process. Where it fails to match steel and concrete is with respect to its stiffness, and this woe can only be deepened considering the effects of creep. With the advent of each new construction product, be it slabs or beams, the requirements and demands on structures keep on increasing. The basic requirement of a floor system would be to provide maximum free spaces (column-free), and no alternative to what the construction presently offers can be justified without fulfilling this primary need. Thus, to compensate for this, we consider the aid of steel beams, in keeping with the prefabricated nature of construction with timber. Keeping this in mind, the focus of thesis will be on steel timber floor systems i.e., floor systems using timber slabs supported by steel beams.

The other important aspect to reduce the construction industry's environmental footprint is to design structures for reuse i.e., to incorporate the concept of circularity. Thus, this thesis will also research into the prospects of reusing timber i.e., whether there are constraints to reusing timber from the perspective of its load carrying capacity. Apart from this, the key to the transition from a linear to a circular economy lies in the use of demountable connections i.e., structures are designed by avoiding wet joints so that at the end of their intended use, they can be dismantled without causing any damage, and reused for other purposes.

1.1 Problem Definition

Currently in the market, there are many steel and timber products available in Europe and in the Netherlands. Different elements have different advantages. The combination of these various elements presents a vast array of possibilities, and the optimum choice would be depending on the requirements of client. In this thesis, the requirements are set at obtaining the best solution for a steel timber floor system considering sustainability and circularity. Now, it would be good to obtain the best possible combination, as presently this is not known. It is also not known how such a steel timber floor system would compare to conventional floor systems.

In the case of steel-concrete floors, it has been observed that the use of composite action between the concrete slabs and steel beams can result in significant gains [68]. These gains are in the form of reduction in the size of the supporting steel beam, owing to the contribution of the slab in the load bearing mechanism. Presently, it is not known whether such type of composite action between the timber slab and steel beam can result in practically justifiable gains.

1.2 Main Objectives

The main objective of this research is to obtain best choice of a demountable steel timber floor system, and to show its relevance within the current construction industry. This is done by making comparisons with conventional floor systems, and by formulating design recommendations for the chosen steel timber floor system. These recommendations can be in the form of span tables for typical grids, and for conducting design with composite action in steel-timber, if applicable. The focus of this thesis is on demountable steel timber floor systems for office use. To reach the final objective, the following aspects have to be considered:

- Assessment of the requirements for demountability and reusability of floor systems, and to keep these aligned with the current legislative policies for the same.
- Assessment of the commonly used steel and timber products available at present, and to choose the best demountable steel timber floor system from the combination of such products in an objective manner.
- Structural analysis and implementation of the chosen system with the help of a case study to assess the advantages of such a floor system with respect to conventional floor systems. The possibility of the application of composite design is also investigated.
- Assessment of the environment impact of the chosen floor system with respect to conventional floor systems.
- Developing design recommendations for the chosen floor system: Designing steel-timber with composite action, and obtaining span tables for the application of chosen floor system for office use.

It should be noted here that the research in this thesis is done using semi-empirical prediction methods i.e., design codes available for steel timber construction. This is because this research includes a broad analysis of various steel timber floor systems, from which the analysis is narrowed down to a specific floor system. Even for considering composite action in steel timber (for which there are no codes available at present), the approach is to review the analysis of steel-concrete and timber-concrete and adopt what is relevant for steel-timber.

1.3 Research Questions

Based on the above objectives, the main research questions, and the associated sub-questions can be formulated as follows:

- What is the best choice of a demountable steel timber floor system among all the possibilities of the combinations of steel and timber products commonly used in the Netherlands?
  - What are the requirements for a floor system to be reusable?
  - What are the commonly used steel beams and timber slabs in Europe, and in the Netherlands, and what is the scope of their application?
- How does the chosen demountable steel timber floor system compare with conventional solutions for demountable floor systems?
  - Is it advanantageous to use the chosen floor system considering aspects of functionality such as floor height, workability and savings in materials?
  - Is it advantageous to use design with composite action for steel-timber? Can the net gains of composite action in steel-timber be justified? How does it affect the load deflection curve of the structure?
  - Is it advanantageous to use the chosen floor system considering the aspect of environmental impact?

1.4 Thesis Structure

The first part of the thesis involves a literature study to address the issue of sustainability with respect to structural design of the floor system, to determine the requirements of demountable structures, and also to present the state of the art on demountable steel timber floor systems. This is given in Chapter 2 and Chapter 3.

The next part of this thesis is to objectively choose the best solution of a demountable steel timber floor system, and this is done with the help of a Multi Criteria Analysis. This is given in Chapter 4, and the boundary conditions for this is formulated based on the previously mentioned literature study.

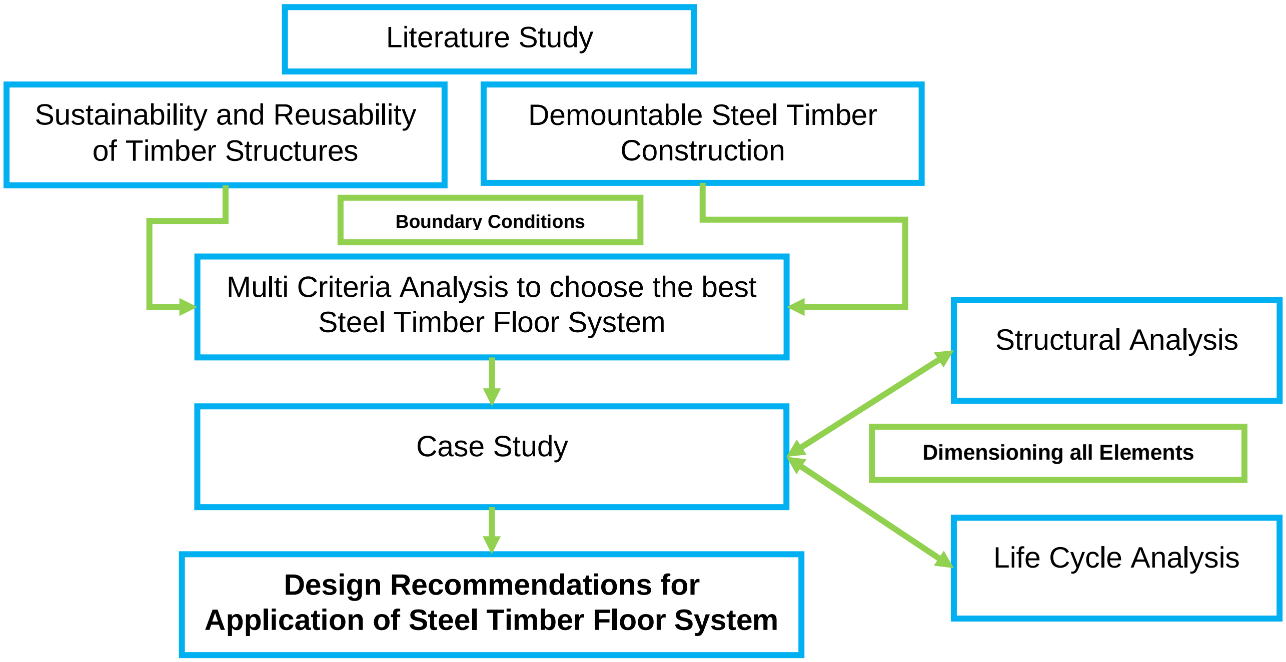
Once a suitable demountable steel timber floor system has been chosen, it is implemented with the help of Case Study (Bouwdeel D). Conventional floor systems such as hollow core slabs and steel-concrete composite slabs are also implemented for the same scenario, and comparison are made with respect to the different floor systems. This is given in Chapter 5.

As mentioned earlier, the research in this thesis is done with design codes. The structural analysis related to all the elements in the case study, for the chosen steel timber floor system is given in Chapter 6. This includes the applicability of composite action for steel timber. Design recommendations based on the structural analysis are also given.

In Chapter 7, a Life Cycle Analysis is conducted on the different variants of the case study, to understand the environmental impacts of the different floor systems. The final comparison between the different floor systems is based on parameters such as height and weight of the floor system, the amount of materials required, and also the environmental impact.

Chapter 8 concludes this thesis with the main results of the chosen floor system, and by ensuring that all the research questions mentioned earlier have been addressed.

Figure 1.1: Overview of Thesis



2. Sustainability and Reusability

In this chapter, terms associated with sustainable development such as sustainability, circularity and reusability are addressed. In Section 2.1, the definition of sustainability is extrapolated to define the term ‘sustainable structural design’, which is one of the main objectives of this thesis. The main differences between a conventional ‘linear’ construction process and a ‘circular’ construction process are addressed in Section 2.2. In Section 2.3, the implications of reuse/recycling of structural elements are addressed. Section 2.4 forms a summary on the current legislation and sustainability targets for sustainability in Europe and in the Netherlands.

2.1 Sustainability

Tasked by the United Nations, the World Commission on Environment and Development (also known as the Brundtland Commission), published a report “Our Common Future” in 1987, which defines sustainability as follows [80]:

“*Sustainable Development is development that meets the needs of the present generation without compromising the ability of the future generations to meet their needs*”.

The Brundtland Commission addresses the 3 main requirements of sustainable development [81]:

- Environmental Aspects, which emphasize limiting the use of finite resources, and preventing the release of harmful substances into the environment.
- Social Aspects, which emphasize that there be social fairness in the distribution of available resources between the present and future generations.
- Economic Aspects, which emphasize that development can/should be achieved even with the constraints of sustainability i.e., it should not occur at the cost of the environmental damage.

In 1988, novelist and environmental entrepreneur John Elkington, in his international bestseller “The Green Consumer Guide”, explains how economic profit can be made which is not at the expense of the environment, and promotes corporate socio-environmental responsibility [81]. He refers to the 3 fundamental pillars of sustainable development as the “3-P Notion”: People (Social Aspects), Planet (Environmental Aspects) and Profit (Economic Aspects).

In the Netherlands, the notion of sustainability was introduced as soon as 1988, with the introduction of the first National Environment Policy (Nationaal Milieu Beleidsplan/ NMP-I) [81].

The notion of sustainability, as defined by the Brundtland Commission and John Elkington are in common use even today. Peters and Wiltjer translate these definitions to the following aspects applicable for structural engineers [82]:

- **Increase service life of buildings:** The structural elements must remain in service for a longer period of time. In terms of structural design, there is an increased probability of occurrence of the peak load. Particularly relevant for timber is that this material experiences a loss of strength over time. Hence it is required to design the structure for larger imposed loads and/or decreased values of the strength parameters of timber.

- **Limit material use:** As far as timber material is concerned, it offers high strength/weight and stiffness/weight ratios, comparable to that of steel. It also offers the freedom to produce elements of differnet shapes and sizes. This property can be utilized to optimize the geometry of elements, and further reduce consumption of materials.
- **Use sustainable materials:** Timber obtained from sustainable forestry is a renewable resource. Alternative construction materials such as steel, concrete and aluminium are produced from non-renewable resources.
- **Consider the environmental impact of construction and transport:** Manufacturing timber is a less intensive process in terms of energy and raw materials, compared to steel and concrete. Also, as timber is a lightweight material, it is easier to handle on-site using cranes. The impact of transportation for timber products is high for timber, particularly in the Netherlands, where a major portion of it is imported. However, easy transportation between sites (production site to construction site) can be achieved when the percentage of use of timber products in the construction industry increases.
- **Design the structure for reuse use in the future:** Timber facilitates the use of lighweight prefabricated structural products and can adapted for demountable (dry) connections easily. This helps in the process of circular construction. However, there is the constraint of the degradation of timber strength over time.

2.2 Linear and Circular Building Process

The traditional building process is linear by nature and consists of 6 distinctive stages [81]. These are given below:

- Stage I : Extraction of raw materials such as limestone, sand, clay, timber, iron ore, etc.
- Stage II : Manufacturing of half – products such as cement, steel, timber planks, etc.
- Stage III: Assembly of products, which is the final building.
- Stage IV: Use phase of product, which is the service life of the building.
- Stage V: Demolition/Disassembly of building, when the building has completed its service life.
- Stage VI: Landfilling, where the waste products are stored together for an indefinite time.

The stages up to the production of half products is referred to as '*cradle to gate'*, and stages I-VI are collectively referred to as '*cradle to grave'*. In the traditional linear building process, the waste generated after the use of buildings are stored as landfill, never to be used again. Hence this process assumes an infinite supply of raw materials, which is far from reality. Governed by this ideology, the global extraction of minerals is projected to increase by 200% in 2020, compared to the level of extraction in 1980 [83]. The construction industry in the Netherlands is responsible for the consumption of 50% of its national resources [81].

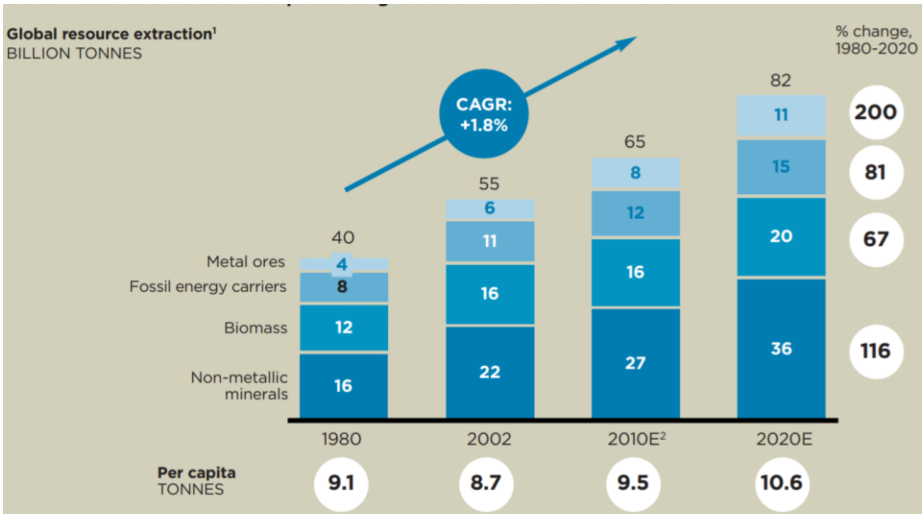


Figure 2.1: Global resource extraction, from [83].

The viable alternative to the traditional linear building process is to give regard to the notions of sustainability during the design phase itself, i.e., reduce the amount of materials, reuse the buildings/components as much as possible, and finally recycle the raw materials from the waste generated to substitute raw materials required for a new cycle. Storing the waste generated as landfills is to be preferred the least. This type of process which replaces the 'end of life' scenario with reducing, reusing, recycling and recovering materials across various stages is known as a circular building process, and such an economic system is referred to as circular economy [84]. According to [83], in an ideal scenario, where the concept of circular economy is adopted without any resistance (in 2010), the amount of new resources which would otherwise have to be extracted, and thus can be saved, by 2040 is 250%. This is shown in Figure 2.2.

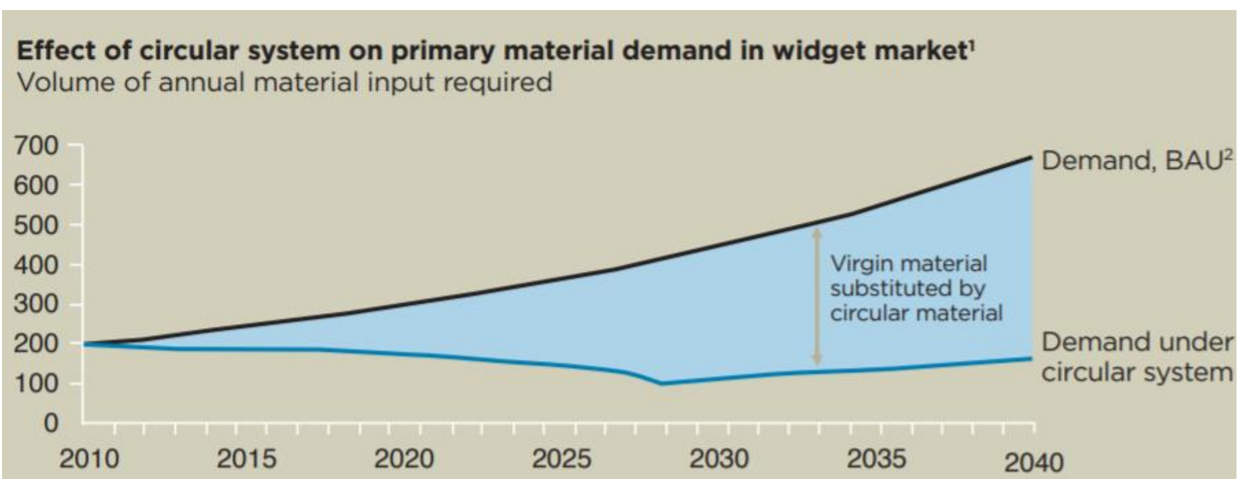


Figure 2.2: Savings in new material to be extracted, assuming a completely circular economy from 2010, from [83].

Founded in 2010, the Ellen McArthur Foundation is one of the leading organisations which try to promote the idea of circular economy, working with businesses, policy makers and institutions to bring about innovative solutions at a global level. According to their report [83], the concept of circular economy urges for solutions for reusing products with their maximum utilization. This is because the process of downcycling of products into raw materials, and further processing to make usable products is much more energy intensive compared to the former. In the construction industry, this means that it is most preferable to reuse the building as whole, followed by reuse at the component level (i.e., slabs, beams, columns and foundations) and finally to recycle the waste generated to produce raw materials.

2.3 Reusability of Structural Elements

Adopting the idea of circular construction implies that the resources (structural elements) remain in use for as much time as possible, and that the minimum amount of waste is generated. Article 4 of the European Waste Framework Directive 2008/98/EC [86] describes the waste hierarchy as comprising of 5 measures: Prevention, Preparing for reuse, Recycling, other Recovery, and Disposal. M. Gharfalkar et al [85] analysed the definitions of these measures, citing the clarifications issued by the European Commission's Director General of Environment in 2012 (DG-ENV 2012) for the same, and other publications by UK's Department of Environment Food and Rural Affairs (DEFRA) and Waste and Resources Actions Plan (WRAP). He proposed an alternative hierarchy of resource use, and this is shown below in Figure 2.3, in the form of a reverse triangle, with preference from top to bottom. In this hierarchy, different levels of reuse are clubbed into a broader term '*Recovery*'. Among these, the highest priority is given for reusing members at the highest level i.e., with the least amount of downgrading of materials. Extraction of raw materials from products at the end of life is given the second least preference, followed by energy recovery.

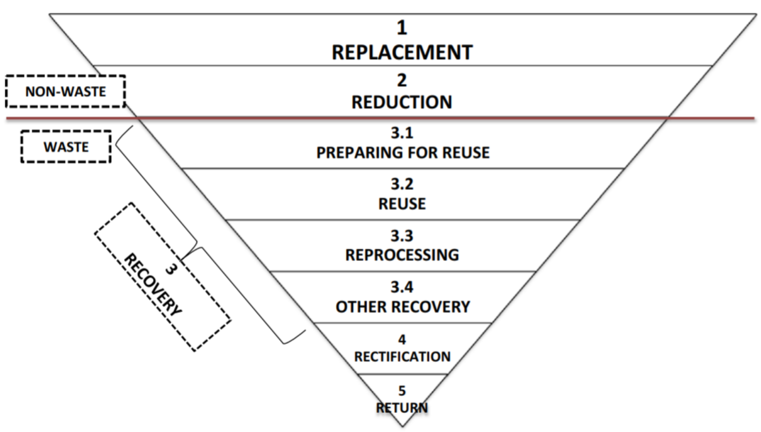


Figure 2.3: Alternative hierarchy of resource use, proposed in [85].

In the construction industry, different levels of reuse can be considered [87]: Building level, System level (floors, foundations, etc), Element level (beams, decks, facades, etc) and the level of raw materials (downgrading materials in construction materials to serve other use). The most important requirement for reusing, is to use prefabricated elements and/or ensure that all the connections are demountable, to mitigate the occurrence of any damages during the process of assembling, disassembling, and reassembling. In this thesis, only reuse at the building level is considered, as it is preferred to reuse at the highest level, and thus, the connections required are assumed to be demountable. Another broad categorisation of reuse can be with respect to where each level can be reused: At the same location i.e., Reorientation, or at a completely different site location i.e., Relocation. These can be applied at each level of reuse.

The need for reorientation of a building, arises when the building must serve multiple functions over its service life. For example, when an office is designed to be used by different clients, it will be good if the building can be redesigned (with new layout of partitions/cubicles etc.) according to their differing needs. Such a design requires a flexible layout of the structure i.e., large column-free spaces. Slabs and beams spanning large distances will help in this regard. Such buildings have to be designed for an increased service life. The aspect related to structural design that must be incorporated in this scenario is the increased possibility of occurrence of maximum loads, as this increases with increase in service life. According to EN 1990 [55], the prescribed value of imposed loads (from EN 1991 [54]) is based on a reference service life of 50 years for buildings and other common structures. For a longer service life, the loads are increased according to Eq. 1 given in the National Annex to EN 1990 [56], to incorporate the increased probability of occurrence of maximum loads.

$$F_t = F_{t0} * \left(1 + \frac{1 - \psi_0}{9} * ln\left(\frac{t}{t_0}\right)\right) \qquad \textbf{\textit{(Eq 1)}}$$

Where,

$$t = Reference\ Period$$

$$t_0 = Default\ Reference\ Period\ (50\ years)$$

$$\psi_0 = Combination\ Factor\ for\ Imposed\ Loads$$

$$F_{t0} = Imposed\ Load\ for\ Default\ Reference\ Period\ t_0$$

$$F_t = Imposed\ Load\ for\ Reference\ Period\ t$$

Reuse by relocation refers to the case where the buildings/structural elements are designed to be taken apart from building and used again in a different location. This is the conventional practise for temporary structures. The implication on structural design is that the design/dimensioning of the elements will have to be done with the strength/stiffness parameters of the construction material in the reused state i.e., depending on the material (concrete, steel, timber), there will reductions in its strength/stiffness. The loads can be maintained as the same, without any changes.

Steel structures are usually designed for large service lives, even under the exposure of severe weather. The Steel Construction Institute (*SCI*) [94] have made recommendations for the reuse of steel structures. Steel elements are visually inspected to check for any signs of plastic deformations. If there are none, then the steel structures can be reused as such without any reduction of strength or stiffness. Apart from having demountable connections, this can be achieved by adopting elastic design i.e., by ensuring that the *ULS* loads are within the elastic limit of steel, the probability of being able to reuse steel is very high. However, even when plastic method is used for design, there is still a possibility not having any plastic deformations, even though it is much smaller. This can be observed in Figure 2.4. The maximum service life that steel structures can be designed with, using plastic resistance of the section, while maintaining that there is less than 5% chance of plastic deformations during this period is about 25 years. At the same time, a service life of well over 100 years can be safely achieved by adopting elastic design. For steel structures subjected to compressive loads (buckling failure), it is recommended to use a conservative value of steel material safety factor $\gamma_{M,S} = 1.15$

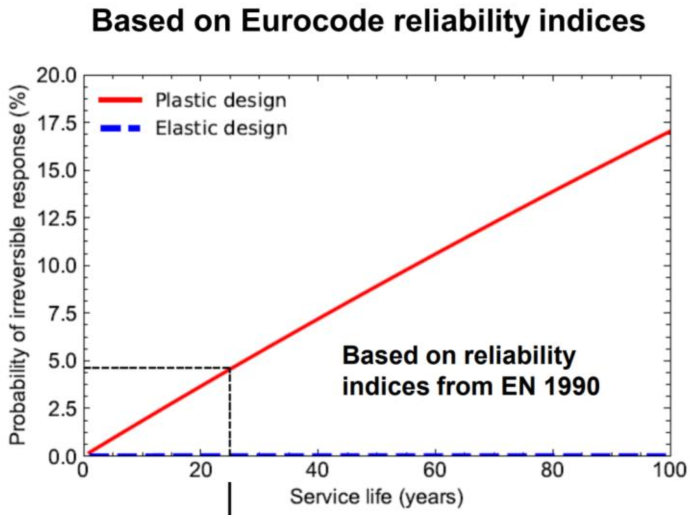


Figure 2.4: Probability of Damage for Plastic and Elastic design of steel structures, adapted from [72].

For concrete structures, no reductions in strength or stiffness have been recommended. A service life of 100 – 200 years is common with respect to concrete structures (bridges, monuments, etc..). Hence for demountable construction, the use of precast structural elements (for example, hollow core slabs), coupled with the application of demountable connections are the only aspects that can be incorporated in design. Ultimately, like steel (and timber), the possibility of reuse of concrete elements will depend on its condition at the end of each service life.

Experimental studies on salvaged timber [129,130] have shown that there are many barriers for reusing timber, from a structural point of view. Typically, houses made of timber in the Netherlands have been in use for 50 to 100 years. Glued timber products such as stressed skin panels, cross laminated timber and glued laminated timber, however, have an even lower declared technical service life, according to the manufacturer's guidelines for these products [5-8], due to the use of adhesives, which can wear out over time. Most of these products have been declared for a service life of 50 years, although it is possible to use them for up to 100 years, in mild environmental conditions. Even under the most optimum conditions, we can observe that the technical service life of timber structures is much lesser than that of steel and concrete.

The main structural barrier for reusing timber elements is the lack of knowledge of its mechanical properties beyond its initial service life [129]. The main aspects that influence the mechanical properties of reused timber are as follows:

- **Mechanical Damage:** This will mainly depend on the type of connections used. Glued connections cannot be demounted without damaging the elements. The same screws cannot be effectively reused in the same holes, as the holes tend to be enlarged. To a large extent, mechanical damages can be mitigated by opting for bolted connections in the design phase itself. Reduced cross sections should be used when dealing with large bolt holes [90]. Presence of cracks or splits can lead to a reduction in stiffness of the elements.
- **Biological Attack:** Being an organic material, timber structures are prone to biological degradation by the attack of fungi and insects. This is dependant on the species of wood used, and the type of environment where the structure is used. *EC5* for design of timber structures takes this phenomenon into account by specifying service classes. For indoor

dry applications, which includes most scenarios of use for floor systems, it is suggested to use service class 1 (or 2) [53]. Thus, the effect of biological attack is negligible for timber floors designed for office use.

- **Effect of Duration of Load:** The effect of duration of load is a characteristic of timber, as strength is dependent on the intensity and duration of applied load [129]. This means that a relatively high load level can cause strength degadration, even when applied for a small duration. Many models have been formulated to quantify this effect. *EC5* uses the k_{mod} value (strength reduction factor for duration of load effect), which is based on the Madsen cruve [93]. According to Cavalli et al [130], this effect is more pronounced on the strength properties of timber rather than its stiffness, as most specimens with a reduction in bending strength did not show the same trend for bending stiffness. At the same time, a certain relaxation can be applied to the value of k_{mod} (to be applied along with reduction in characteristic strength) for each cycle of reuse. This is because, most of the strength reduction would have occurred in the first few years of the lifespan of the element. This can be observed in Figure 2.5.
- **Natural Ageing Phenomena:** Again, due to the fact that wood is a biomaterial, the mechanical properties show variation with respect to the age of the wood used. This represents the time period starting from when the trees are felled, until the end of its life, when they are processed to make wood chips, or incinerated.

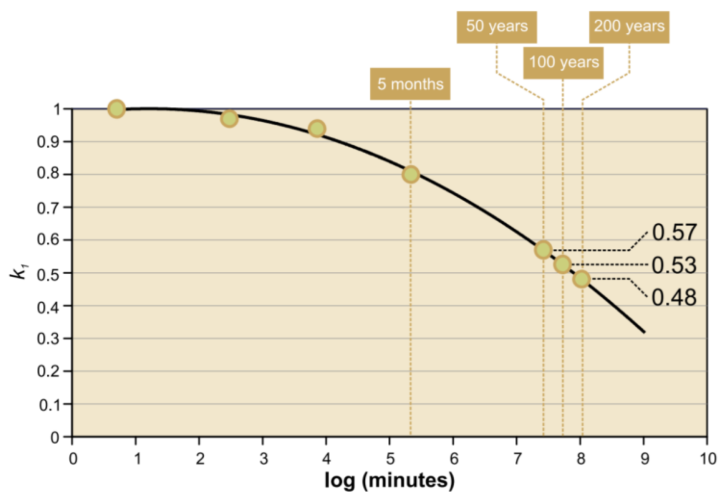


Figure 2.5: Values of k_{mod} based on Madsen Curve from [93]

Among the different aspects discussed above, the possibility of mechanical damages can be mitigated to a large extent by using demountable connections, and the effect of biological attack is negligible for floor systems, especially for indoor office use. The combined effect of the duration of load and ageing phenomena is what determines the reduction in mechanical properties of timber, to a large extent. Cavalli et al [130] has summarized the literature available on experiments on salvaged timber (from 1950s to 2000s), with the primary intent of investigating the effect of ageing of timber. It was observed that there were many inconsistencies in the method of testing used by different researchers over the ages, so as to identify a specific pattern. These include uncertainties in the original mechanical properties of wood, moisture content, state of conservation, presence of cracks and other damages, and most importantly the loading history.

For bending stiffness, the conclusion was that many research works agreed that there was no significant change (20 works reported an increase or no change, compared to 5 works which reported a slight decrease). For bending strength, the results showed a much larger variation. Most of researchers (8 out of 17 works) observed a reduction in bending strength (from 7% up to 60%), and attributed it to the duration of load effect and due to the damages that occurred during the process of dismantling. At the same time, some works also reported no change (7 works) or a slight increase (2 works). Even though the above-mentioned variations were observed in different experiments, it was erroneous to attribute them to the ageing phenomenon, as the net effects were due to the combination of different aspects, with the highest contribution from duration of load effect. Also, there was little information available on other mechanical properties of wood such as compression, tension, and shear, to draw specific conclusions.

Thus, determining the residual mechanical properties of reused timber in the initial design phase poses as a complex problem, as it is the net effect of different interacting factors. The current practise is to grade the salvaged timber elements obtained from dismantled buildings based on visual inspection and non-destructive testing methods, and to reuse them with a design suitable for its new mechanical properties. However, this process of 're-grading' does not have specific set of rules and guidelines. EN 14081 – 1 [131], which contains guidelines for grading new timber only states that these salvaged timber elements should not be graded into the same class that it was previously assigned. Petr Hradil et al [90] suggests a few recommendations for the reuse of timber elements in Finland. If the original certification of grading is available, it is recommended to reduce the grade of the reused timber (and consequently all its mechanical properties) by 2 steps, when the load history is unavailable or if the element is heavily loaded, and to reduce it by 1 step in all other cases. However, it does not specify the time period for which the elements have remained in service, and thus could lead to ambiguous implications. For example, it could be implied that an element which has been in service for 10 years can be assigned the same mechanical properties as an element which has been in service for 50 or even 100 years. Also, upon discussion with experts from TU Delft, it was decided that reduction suggested i.e., by 2 strength grades was too conservative to be used as a recommendation for design.

Thus, since the change in the mechanical properties of timber elements cannot be predicted before-hand, in this thesis, it is opted to compare demountable floor systems only for the case of reuse by reorientation. In this section, the focus was on the aspects to be considered for reusing structural components i.e., to develop demountable (floor) systems. All the parameters considered above were at the level of the structural element, and this is summarised below:

- **Prefabricated Elements :** For concrete, this means that cast in-situ elements will have to be replaced with elements such as hollow core slabs. Otherwise, it will be difficult to dismantle the elements, making it unsuitable for reuse. Steel and timber are already suited for this, as they are currently manfucatured elsewhere and assembled on-site.
- **Demountable (Dry) Connections :** This aspect goes in conjunction with prefabrciated elements. For concrete, the connections to these elements will have to use bolts, avoiding wet joints completely. For steel and aluminium this means that bolts will have to preferred over welded joints. Timber elements are usually executed with dry connections. It is preferreed to use bolts or carpentry joints, instead of screws, dowels, etc. This choice of bolts for timber (steel-timber) is discussed in Section 3.3. Moreover, the geometry of the elements being connected will decide how easily they can be dismantled. This aspect is quantified with the help of a demountability index (Appendix B.2.4).

- **Increased Live Loads:** An increased service could imply an increase in the probability of of occurrence of live loads, as in the case of reuse by reorientation.
- **Degradation of Mechanical Properties over time:** In the case of reuse by relocation, the initial design would have to incorporate the reduced strength/stiffness of the reused structural elements. It was conluded from current recommendations that this aspect was negligible for steel and concrete, but had a drastic effect on timber. The main limitation was that this effect on timber cannot not be quantified with the present state of the art for timber.
- **Elastic Analysis:** Using elastic analysis, instead of utilizing the load carrying capacity of the plastified section can decrease the probability of occurrence of damages (plastic deformations) on the structural elements, and thus increase the chances of being able to reuse them. Analysis of steel and steel-timber composite section are limited to the elastic limit, considering this aspect.

All of these aspects are incorporated in design. As mentioned above, all these aspects are associated at the level of the structural element. On a larger scheme, reusability of structural elements requires standardised dimensions i.e., standard dimensions of column grids. Since the comparison of the steel timber floor system with other conventional floor systems are made with respect to a particular case study, this aspect cannot be considered further in this thesis. In the Netherlands, it is a common practise to adopt standardised layouts for building office structures [132]. Referring to the typical layout for Dutch offices, it can be observed that the size of the column grid is in multiples of 1.8 m, which is associated with the optimum span of composite slabs (3.6 m). This can be considered as a step in the direction of standardised layouts, which will indeed help in the reusability of structural elements. These standardised office layouts [132] are referred again in Section 6.5, to produce span tables for the chosen steel timber floor system.

2.4 Policies on Reuse and Sustainability

Targets by the European Union for waste management are the key drivers for increasing the amount of waste that is recycled [86]. 30 binding targets have been set for the period 2015 – 2030, for different types of wastes (construction waste, e-waste, municipal waste, packaging, etc). The European Commission has shown commitment to tackling key environmental and socio-economic issues by signing the Paris Climate Agreement in 2015 and committing to the United Nations sustainable development goals [120]. The primary goal of the European Commission is to achieve net zero greenhouse gas emissions by 2050 [121]. In keeping with this objective, the European Commission has made legislative proposals to encourage the use of greener products in the construction market. For example, using timber structures as an alternative to steel/concrete. This includes providing incentives to use timber owing to its sustainability aspects, as conventionally these are more expensive than steel/concrete.

The targets set by the Dutch government for the building sector is based on the performance criteria developed by the Stichting Bouwkwalitiet *(SBK).* Known as the ***Milieuprestatie Gebouw (MPG)*** method, it is based on the shadow price method with the data obtained from the Dutch National Milieu Database *(NMD)*. Details on how to calculate the Environment Cost Indicator *(ECI)* value using the shadow price method is given in Section 7.3. The value of the *MPG* performance criteria is calculated as follows:

$$MPG = \frac{ECI}{RSL * GFA} \tag{Eq 2}$$

Where,

$$ECI \text{ is the total environment costs using the shadow price method in Euros}$$

$$RSL \text{ is the reference service life of the building in years}$$

$$GFA \text{ is the gross floor area of the building in } m^2$$

The calculation of MPG is inclusive of all components of the building such as slabs, beams, columns, facades, etc. According to the Dutch Building decree [1], effective from 2018, all newly constructed residential buildings and offices of area larger than 100 m^2 requires an MPG calculation. The requirement for MPG is currently set at $0.8\ \epsilon/year/m^2$, and $0.5\ \epsilon/year/m^2$ for the year 2030. The report 'Nederland Circulair 2050' [118] published by the Dutch government has set ambitious targets in terms of achieving its sustainability goals. The main objective is to shift to a 100% circular economy by 2050, with a waypoint of 50% by 2030. Even though construction waste is currently recycled, the products experience severe downgrading during the process of recycling. For the same reason, the focus of this thesis is to achieve circularity in steel timber floor systems by reusing at the highest possible levels i.e., with the least amount of downgrading.

2.5 Summary

The term 'sustainable structural design' has been defined in this thesis as structural design that gives long service live, using minimum materials of low environment impact. The traditional construction process can be converted from linear to circular by reusing the construction products as much as possible before they are downgraded or sent for landfill. The most important aspect is to ensure demountability of the structure by using dry connections, preferably used in combination with prefabricated elements. Reusing structural components is possible in many levels. However, it is preferable to reuse it at the highest possible level, for optimum efficiency. Thus, in this thesis, focus will be on reusing at the building level. Among the three construction materials: steel, concrete and timber – steel and concrete can be reused without any reductions in their strength and stiffness (except for buckling of steel). However, timber experiences a huge reduction in strength as it is a biomaterial. Hence, this will be one of the governing aspects of design of steel timber floor systems. As explained in this chapter, the strength reduction of timber elements cannot be determined before-hand, using the literature that is currently available. Hence only reuse by reorientation will be considered in this thesis i.e., by considering an increased service life for design. Thus, all the relevant aspects that can be incorporated with respect to the design of a floor system is addressed here and will be adopted further on in this thesis. Lastly, the current legislation on circularity is also addressed in this chapter.

3. Steel Timber Floor Systems

This chapter contains the state of the art on steel timber construction and demountable construction. Various products used in the Netherlands and in Europe are considered here, and it forms the basis for the Multi Criteria Analysis that follows in Chapter 4.

3.1 Timber Decks

There are many types of engineered timber products available in Europe, such as solid timber, cross laminated timber (*CLT*), glued laminated timber (*GLT*), laminated veneer lumber (*LVL*), etc. Oriented strand boards (*OSB*) are also used, but not as commonly as the former ones. Pertaining to the mechanical properties, these products differ in their orthotropic nature. Complex manufacturing processes such as that for *LVL* and *OSB* lead to lower variations in their strength and stiffness characteristics between the longitudinal and transverse directions. They also differ in the amount of glue used, which implies a higher impact on the environment. For example, by comparing the *ECI* values (explained in Chapter 7), we can observe that the value of solid timber is lower than that of steel, whereas that of *LVL* and *OSB* are much higher than steel.

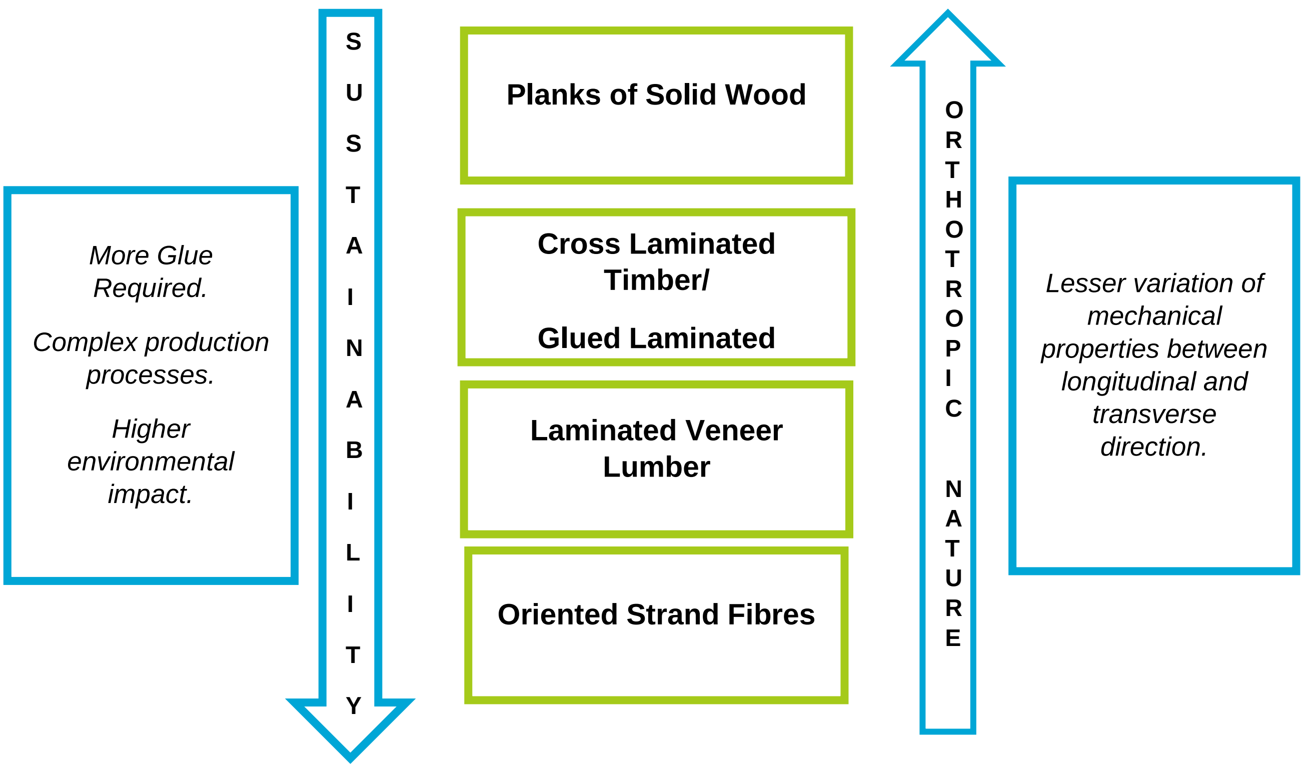


Figure 3.1: Comparison of Different Engineered Timber Products

The different types of timber products used as floor slabs, with respect to the type of engineered timber and geometry of the slab, are given below:

Joist Slabs: This is a commonly used timber floor with timber floorboards resting on timber joists. These are designed according to the requirements of the user and offers cheap and practical solutions for floors with low demands on load carrying capacity. However, these require assembling the individual elements (joists and floorboards) on site, and thus impedes the process of dismantling. On the other hand, prefabricated timber products (discussed below) offer the possibility of rapid execution, and also serves as a better alternative for its load carrying capacity. Hence joist floor systems will not be considered further in this thesis.

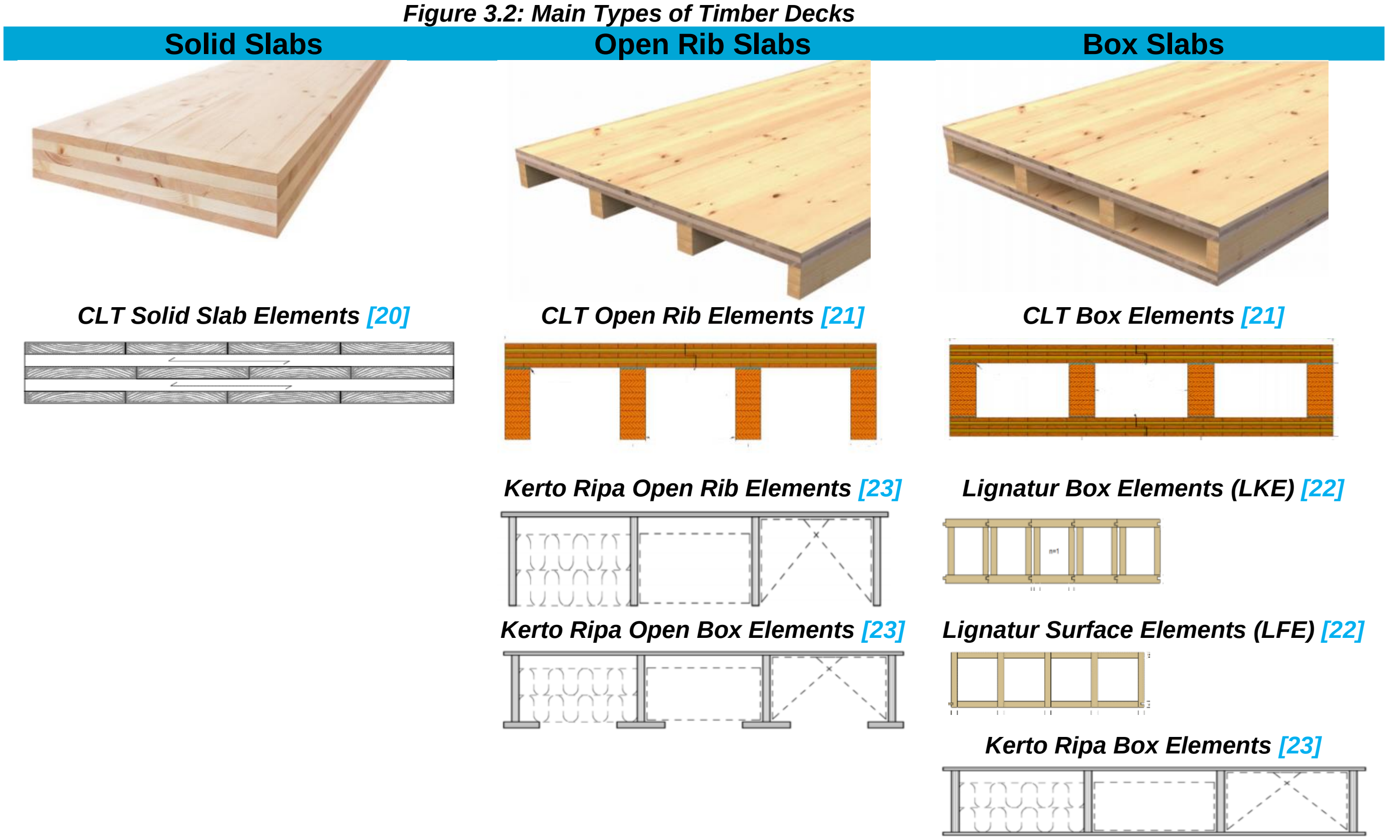
CLT Slabs: *CLT* floors are timber floor panels made of layers of timber board or 'lamellae', with alternate layers oriented in the crosswise direction. Thus, *CLT* floors provide bidirectional load bearing capacity. The leading producers of *CLT* in Europe are KLH Massivholz, Stora Enso, BinderHolz, etc. For this research, it is opted to use CLT products manufactured by Stora Enso. The maximum size of *CLT* panels that can be produced is 16 x 3.45 m, having 3 to 8 layers. The lamellae are made of spruce or equivalent European softwoods. The technical details of the *CLT* floors used is given in Appendix A.2.

Open Rib Slabs: In these types of floors, the top flange is adhesive bonded onto the webs to form a composite unit. These are like joist floors in cross section but are available as completely built-up units. Open rib panels can be produced with *CLT*, solid timber and *LVL*. For *CLT*, the open rib floors manufactured by Stora Enso are considered. The flanges are made of *CLT*, and the webs are made of *GLT*. *LVL* open rib floors manufactured by MetsäWood are also considered in this thesis (referred to as Kerto Ripa floor elements). The flanges are made of Kerto-Q panels and the ribs are made of Kerto-S beams. Additionally, open box elements (with open bottom flange) manufactured by MetsäWood is also considered in this thesis. Details of Kerto Ripa elements are given in Appendix A.4. Generally, open rib elements can span over larger distances, and is very deep. Services/Installations can be integrated into the slab.

Box Slabs: Also known as timber hollow core slabs, these also can be manufactured like the open rib floors, with the addition of a bottom flange. In this thesis, *CLT* box elements manufactured by Stora Enso, Kerto Ripa closed rib elements by MetsäWood, and solid timber box elements by Lignatur are considered. The details of Lignatur elements used is given in Appendix A.3. Generally, box elements are more slender than the corresponding open rib elements, for the same span. Services/installations must be placed outside the timber section, as it is a closed section. Both the open rib elements and the closed rib elements use very thin flanges (and also webs), and hence are referred to as stress skin panels [3]. This results in shear deformations (shear lag), and thus the effective width of the flanges will have to be used for calculations.

The above-mentioned timber decks offer high spans without the use of additional stiffeners in the transverse direction (without steel or timber joists). All timber decks considered in this thesis have been divided into 3 main categories: Solid slabs, open rib slabs and box slabs. The cross sections of different types of timber decks, sorted by these categories is given below in Figure 3.2.

Figure 3.2: Main Types of Timber Decks



3.2 Steel Beams

The objective of this thesis is to obtain floor system that sits well with the concept of circularity. For this reason, the slabs considered above offer high spans for office use (above 7 m). Consequently, only primary steel beams are considered i.e., beams with sufficient load capacity to be used in conjunction with the above mentioned slabs. Smaller steel beams used as stiffeners or joists are not considered in this thesis (i.e., cold formed C sections, Z purlins, etc). In the Netherlands, there are 2 main categories of primary steel beams: Integrated and Non-Integrated. As the name suggests, integrated beams allow for placing the slabs on the bottom flange of steel. This significantly reduces the floor height. The second one refers to conventional beams, where the slabs are placed on top of the steel beams. Keeping this in mind, the different types of steel beams considered for this thesis is given below:

I Beams: The typical European I beams are *IPE*, *HEA*, *HEB* and *HEM* and these long span members are produced by the process of hot rolling. They vary based on their slenderness and thickness of webs/flanges. *IPE* beams are thin and deep. *HEM* beams have thick webs/flanges and are very slender. U section or half – I beams are also used (*UPE*, *UPA*, *UPN*, etc). However, since they are less slender than the corresponding I section, they are not considered further in this thesis.

Castellated I Beams: Castellated I beams are I beams with large web opening for the seamless passage of mechanical, electrical and plumbing (*MEP*) ducts. They offer large spans, thus enabling larger column free spaces. In this thesis, the castellated I beams manufactured by Arcellor Mittal [10] are considered. These are manufacture from standard I beams, by creating the required openings using flame cutting. In comparison to standard I beams, castellated I beams offer larger spans and smaller sections. However, they very deep compared to the former.

Common Integrated Beams: Commonly used integrated beams in the Netherlands are Top Hat Beams (*THQ*), Shallow Floor Beams (*SFB*) and Internal Floor Beams (*IFB*) [4]. THQ is made with welded plates, in the shape of a hat. It is designed in such a way that the slabs can be placed on the bottom flange. SFBs are produces by welding an additional wider bottom plate to standard I beams. IFBs consist of a half I beam (Top flange and half the web) with a wide bottom plate welded to it.

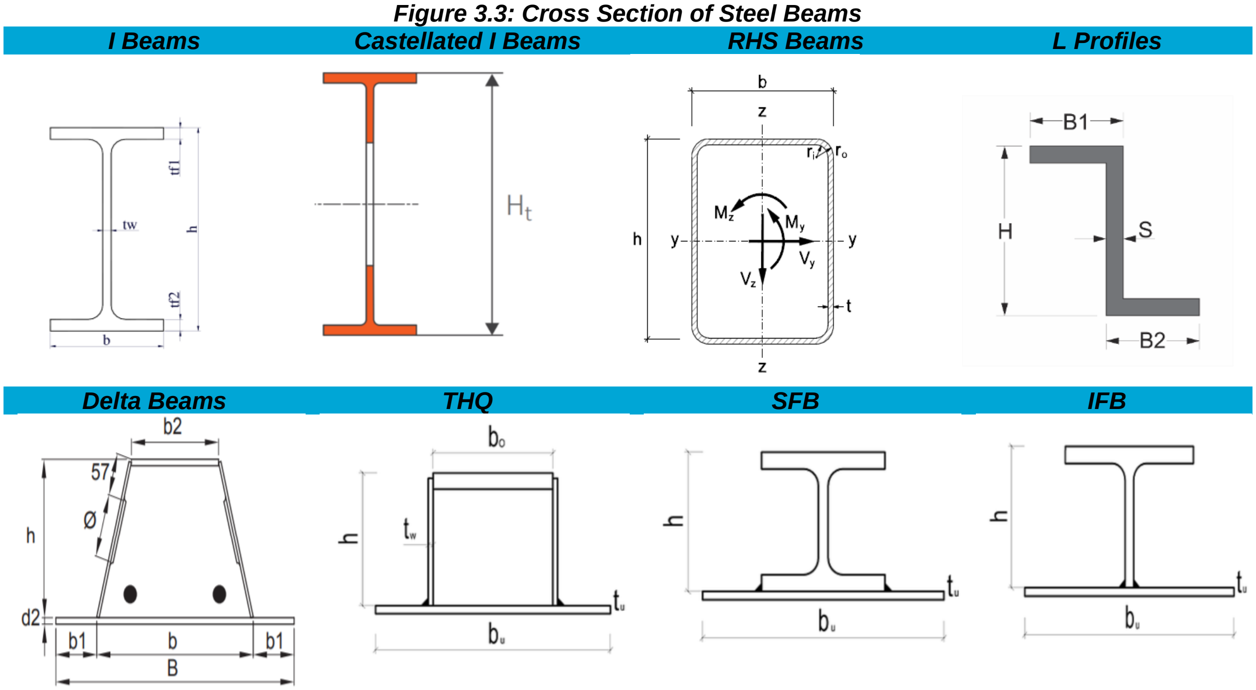
Delta Beams: These can be described as castellated integrated beams with a trapezoidal cross section. Integrated slabs are placed onto the bottom flange. Finally, concrete is cast into the section of the beam to create a seamless monolithic section. In this thesis, Delta beams produced by Peikiko have been considered [12,13]. These can be combined with hollow core slabs and timber slabs. However, both require wet concrete to be cast into the beams. Hence, these are not suitable for demountable beams, and will not be considered further.

RHS Beams: Rectangular hollow sections (*RHS)* can be hot finished or cold formed. In this thesis, hot finished *RHS* manufactured by Tata Steel [14] are considered. They can be used as integrated or non-integrated beams. *RHS* members are usually welded onto other structural members. Hence, a plate is to be welded onto the *RHS* to establish a demountable connection with the timber deck.

Customised Integrated Beams: In the Netherlands, customised solutions for floor systems are available, mostly for integrated systems. These maybe hot rolled or using welded plates. In this thesis, a customised L profile, like the one used in the demountable building Bouwdeel D [19] is also considered for comparison.

The cross sections of the above-mentioned steel beams are given below:

Figure 3.3: Cross Section of Steel Beams



3.3 Demountable Connections

In designing a steel timber composite floor system (*STC)*, the main connections that are needed to be considered for design are: steel to steel (beam-column and beam-beam), steel timber (beam-slab) and timber to timber (slab-slab). However, the focus of this research will be on the shear connectors i.e., the connections between steel beams and timber slabs. The basic types of joints used in timber construction are carpentry joints, glued joints, and joints using various metal fasteners [3]. Carpentry joints are used for timber-timber connections. In a bid to maintain demountability, glued connections cannot be used. Hence it is opted to use metal fasteners as the choice of demountable shear connectors between steel beams and timber decks.

There are 2 main functions that need to be served by these connections: The first one is to enable the diaphragm action of the floor i.e., to transfer the horizontal forces (wind forces) onto the slabs so that they can aid the structure in stability. The second function is only applicable in case we require composite action between the steel beam and the timber slab. In this case, the shear connectors need to transfer the longitudinal shear forces between the steel beam and the timber slabs. Glued connections enable a rigid joint. With the use of metal fasteners, the connections are semi-rigid. This means that there will be a significant slip between the members, and this aspect needs to be accounted for in design. At the same time, semi-rigid joints give some advantages: It ensures ductility, thus giving clear sign of failure, and it gives rise to plastic redistribution of stresses, thus relieving the highly stresses areas [3]. In this thesis, 3 approaches have been considered to predict the behaviour of the steel-timber composite sections, all of which take into account the slip at the interface: Leskela Approach, the Gamma Method used for built-up timber sections, and the Newmark Model. This will be explained further in Chapter 6. For all the methods, the main factor that influences the interaction between the steel and timber is the joint stiffness per unit length. Between diaphragm action of the floor and composite action, the latter is more demanding. Thus, the smeared stiffness of the shear connectors is given more weightage compared to the total load carrying capacity, when we choose a suitable solution for the demountable shear connector.

The main types of metal fasteners used are dowel type, surface connectors and punched metal plate fasteners. Among the various metal fasteners used in practise, the ones that can be used for timber-steel connections are nails, screws, bolts, shear plate connectors and single sided tooth-plate connectors. Staples, split rings and punched metal plate fasteners are used for timber-timber joints. The use of dowels is not recommended for steel-timber connections [3]. It should be noted that the surface connectors are mainly used to transfer large forces with single connectors, as an alternative to using multiple dowel type connectors. Their joint stiffness (and consequently the smeared stiffness) is lower than that of the dowel type connectors. Also, these can be quite expensive compared to conventional bolts and nails. Hence, for the steel-timber connections which require composite action, surface connectors are also avoided.

Now the choice is between nails, screws and bolts. Nails and screws (up to a certain extent) are not considered to be demountable. This is because both of these are tightly fitted inside the joints, when used for the first service life. In the consequent service lives, the process of removing and refitting these will lead to the enlargement of the connection holes. Thus, in this thesis, it is opted to choose bolts as the solution for the demountable shear connectors.

Parameters such as the different failure modes for bolted connections in steel timber, and the joint stiffness, have been addressed in *EC5* [52] for the design of timber structures. The different failure modes for these connections are predicted by the Johannsen yield models [3]. Experiments on steel timber composite sections connected using bolts and nails [24,31] also showed results consistent with the failure modes predicted by this model.

Minimum distances between connectors have been specified by *EC5* [52]. These required edge distances guarantee that almost all brittle failure modes like splitting can be prevented, which paves the way for ductile failure [3]. The load carrying capacity of the dowel type connectors is determined by the Johannsen Yield Models. These consider the interaction of the brittle failure modes (timber crushing/shear resistance of dowels) and the ductile failure modes (plastic hinges in dowels). For the surface type connectors, the shear failure of the connectors and timber crushing when the embedment strength is reached, are considered to calculate the load bearing capacity of the joints. As the penetration into timber is comparatively less for surface connectors, these are associated with brittle failure. Table 3.1: below shows the summary of different metal fasteners that can be used, with their joint stiffnesses. *EC5* [52]. Also specifies the minimum spacings for each connection, from which the maximum permissible smeared stiffnesses can be obtained. However, these composite structures are usually designed for a minimum degree of composite action. All aspects related to the analysis of these bolted steel timber connections are addressed in Section 6.3, and those related to composite action, in Section 6.2.

Table 3.1: Summary of Shear Connectors [3,52]

Connectors	Diameter (d) $[mm]$	Joint Stiffness (K_{ser}) $[N/mm]$	Minimum Spacing (S_i) $[mm]$	Smeared Stiffness (k_{sc}) $[N/mm^2]$	Remarks
Dowel Type Connectors					*Ductile failure*
Nails	*2 - 8*	$2x\frac{d * \rho_m{}^{1.5}}{23}$	$0.7\ x\ 5d$	$0.0248 * \rho_m{}^{1.5}$	*Predrilled for d > 6 mm and ρ_k > 500 kg/m^3.*
Bolts	*10 - 30*	$2x\frac{d * \rho_m{}^{1.5}}{23}$	$5d$	$0.0174 * \rho_m{}^{1.5}$	*Always predrilled.*
Screws	*6 > d*	$2x\frac{d * \rho_m{}^{1.5}}{23}$	$0.7\ x\ 5d$	$0.0248 * \rho_m{}^{1.5}$	*No predrilling required.*
	6 < d <14	$2x\frac{d * \rho_m{}^{1.5}}{23}$	$5d$	$0.0174 * \rho_m{}^{1.5}$	*No predrilling required.*
Surface Connectors					*Brittle failure. Large forces. Lesser joint stiffness*
Single-sided shear plate connectors	*60 - 260*	$\frac{d_c * \rho_m}{2}$	$2d_c$	$0.25 * \rho_m$	*Milling depression, predrilling bolt holes.*
Singled Sided Tooth Plate Connectors (C1-C9)	*38 - 165*	$\frac{1.5 * d_c * \rho_m}{4}$	$1.5d_c$	$0.25 * \rho_m$	*For ρ_k < 500 kg/m^3, connectors can be pressed into timber.*
Singled Sided Tooth Plate Connectors (C10,C11)	*38 - 165*	$\frac{d_c * \rho_m}{2}$	$2d_c$	$0.25 * \rho_m$	

3.4 Steel Timber Construction

In this section, various buildings in Europe with steel – timber floor systems are considered. These include offices, residential buildings and other types of structures. It was observed that in most cases, the reason for choosing such a floor system instead of conventional steel-concrete floors was due to its lightweight nature, and the possibility for rapid execution.

2200 Times Square [122]**:** This tower in London was proposed as a redevelopment project, to add additional storeys to an existing historic structure. As the objective was to increase the number of storeys with minimal changes to the foundations/ supporting structure, this called for innovations in lightweight construction. The proposed solution was to use lightweight steel – timber hybrid floors. The slabs were made of *CLT* solid panels supported on castellated I beams. Opting for this solution meant that there could be weight reduction of 15%, added by the benefits of the reduction of embodied carbon by 30%.

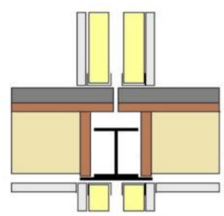
6 Orsman Road [123]**:** Designed by Waugh Thistleton architects, this building was created as a solution for demountable construction for office spaces in London. Similar to 2200 Times Square, this building also used *CLT* solid panels supported on castellated I beams for the floor system.

Student's Residence Hall [126]**:** Created on demand for the Rhode Island School of Design, this residence hall was built to serve as a model for sustainable construction. The main requirements for choosing steel – timber were rapid execution and lightweight structure. The floors were made of *CLT* solid panels resting on a steel frame with I beams.

Karel Doorman [125]**:** This residential building in Rotterdam was one of the first to use to steel - timber hybrid structure, when it was renovated for the purpose of adding additional storeys. The main structural frame was with steel. The floor systems were made of Kerto S joists manufactured by MetsäWood [8]. The slabs were resting on *HEA* beams with a welded bottom plate (*SFB).*

Hurlingham Racquet Centre [124]**:** This is sports hall, where the roof is made of a hybrid steel – timber structure. Steel arches with *RHS* spans across the hall with Kerto Ripa Box Elements used as the roof slabs. The slabs are supported on plates welded onto the *RHS*.

Figure 3.4: Steel Timber Floor systems.



a) Karel Doorman, beam slab junction.

b) Hurlingham Racquet Centre, Roof.

c) 6 Orsman Road

3.5 Demountable Construction

In this section, a few examples of buildings designed for disassembly are discussed, with most of them comprising of steel timber floor systems. This serves as basis for choosing the case study for implementing the steel timber floor obtained from the *MCA* in Chapter 4. It should be noted that the examples considered here are specifically for office spaces. Car parks and residential buildings have been avoided here, as this is not the focus of this research.

Bouwdeel D: Developed by Cepezed architects, Bouwdeel D was designed as a completely demountable, sustainable, and lightweight construction [18]. It is designed for disassembly, in the centre of Delft. Construction was started in May 2019 and completed by September 2019. Being the owner of the building as well, Cepezed architects chose a demountable design to serve as a symbol of the company's role to help achieve the target of complete circularity in the Netherlands by 2050. It measures 11x 21.5 m and has 4 floors with 200 m^2 of lettable space [28]. The largest grid is 1.8 x 6.35 m. The floors consist of Kerto Ripa open rib elements, with the ducts of services integrated into the slabs. The columns are spaced 1.8 m apart and uses very slender hollow sections. The edge beams used are customised L profiles, and support the slabs on the bottom flange, thus forming an integrated system. All structural connections are made with bolts. The facades are supported directly on the edge beams, without the use of any additional frames. An *RHS* beam is used as the as a support for the slabs in between the edge beams. All of this results in a very lightweight construction, enabling easy transportation and erection. With the large open spaces, Bouwdeel D is designed for flexibility in use, depending upon the client. Hence, this design can be reused in the building level for reorientation and can be dismantled for relocation.

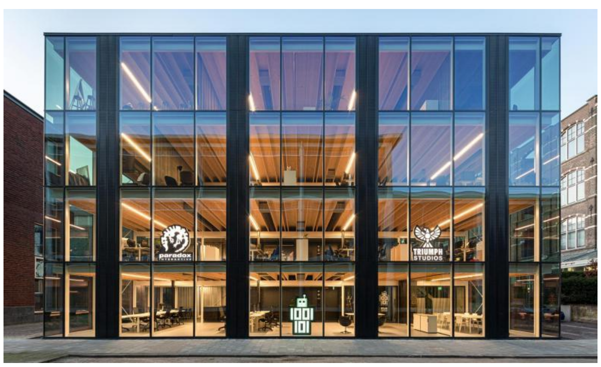


Figure 3.5: Bouwdeel D [18].

Figure 3.6: The Greenhouse Utrecht [27].

The Greenhouse: This building has been developed as a pavilion for catering and meetings, along with the redevelopment of the Knoop Barracks in Utrecht. The Green House was constructed by Strukton, Ballast Nedam and Facilicom together with the catering operator Albron. Cepezed is the architect and Pieters Bouwtechniek the constructor of this future-proof project. The building is designed for 15 years of use, to make the locality lively, after which it will be dismantled from the original plot and reused. This building is designed for reuse at the building level by relocation. Even the foundations can be dismantled from the original plot. They consist of steel stelcon plates founded on steel. Standard concrete blocks have been placed on these to transfer the loads from the columns to the foundation. The building measures 12.15 x 30.25 m, with 2 storeys (3.375, 3.57 m). The column grids are 6.075 x 6.07 m. Standard steel elements such as *HEA* and *IPE* sections are used as the column and beams. For the slabs, prefabricated units with glulam webs and multiplex flanges are used. The roof is made with steel roofing sheets.

Due to the use of lightweight materials, the total weight of the building is 252 tonnes, compared to an expected weight of 539 tonnes from traditional construction [27].

StayOkay Hostel and NatuurPodium: This building was intended for StayOkay and the municipality of Bergen Op Zoom, and designed for disassembly [46]. The building measures 20 x 20 m. The floors are made of Kerto Ripa open rib elements with the services integrated into them. The beams used are cold-formed C sections, as these require produce lesser environmental impact than the hot rolled products. The site imposed a height restriction of 7.8 m for 2 storeys, which led to the use of integrated floor systems. The columns are of glulam, with a grid of 5 x 5m.

Figure 3.7: StayOkay Natuupodium [47].

Figure 3.8: Circl Amsterdam [28].

Circl: This building was designed as the headquarters of ABN Amro bank in Amsterdam. In order to promote circularity, this building is designed for disassembly**.** The design was made by de Architects Cie. Most of the elements used in the building have been recycled, as far as possible. The wooden floors are made of rejected wooden frames. Tiles were made of reused concrete. The main support structure is made with locally sourced Larch, and all the connections are demountable. The insulation was provided by using discarded jeans [28].

Temporary Courthouse: The temporary courthouse, Amsterdam is designed for disassembly, with an initial service life of 5 years. As the name suggests, this building is supposed serve as the courthouse, until the construction of the permanent building is completed in 2022 [43]. The dimensions of free spaces and storey heights have been selected such that they can be used as schools, offices etc after the initial use as the temporary courthouse. Demountable connections are used throughout. The slabs used are hollow core slabs, and the beams used are *SFB*. A moment resisting dry connection is established between the slab and beam using bolts [44]. Thus, it can be reused in the building level for both reorientation and relocation. The construction of the temporary courthouse was commissioned by the Dutch government (Rijksvastgoedbedrijf), and the main reason for adopting circular design is to minimise waste and maximise the residual value of the building after its initial service life.

Figure 3.9: Temporary Courthouse Amsterdam [45].

The summary of the floor systems considered above is given below:

Table 3.2: Summary of Demountable Construction in the Netherlands.

Building	Floor System		Original Use	Reason for Design for Disassembly	Level of Reuse
	Slabs	Beams			
Bouwdeel D	*Kerto Ripa open rib slabs*	*Customised L profile*	*Office*	*Promotion of Circular design*	*Building level, reorientation and relocation*
The Green House	*Glulam ribs, multiplex floors.*	*IPE, HEA*	*Catering pavilion*	*Increase activity in locality, then use plot for apartment/offices*	*Building level, relocation*
StayOkay Natuurpodium	*Kerto Ripa open rib slabs*	*Cold-formed C sections.*	*Public space.*	*Promotion of circular design*	*Building level, reorientation and relocation.*
Circl Amsterdam	*Reused wooden frames.*	*Larch beams.*	*Offices, public spaces*	*Promotion of circular design.*	*Building level, relocation.*
Temporary Courthouse Amsterdam	*Hollow core slabs*	*SFB*	*Public office*	*Temporary use until construction of permanent building is finished*	*Building level, reorientation and relocation*

For this research, it is decided to choose Bouwdeel D for the Case study with the new *STC* floor system. As mentioned earlier, the designers of the building, also being the owner, intended this to be model for circular design. The original design is very light, sustainable, and demountable. Hence, it would be good to have a comparison between the STC floor in this building and the *STC* floor chosen from the *MCA*. Comparisons will be made in terms of the advantages in weight of the structural system, and sustainability, through a life cycle analysis.

3.6 Composite Action between Steel and Timber

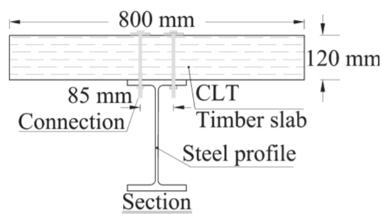
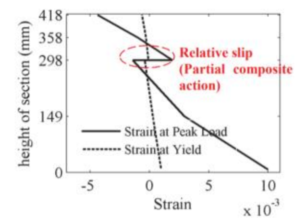
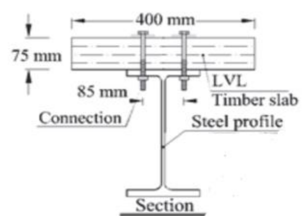
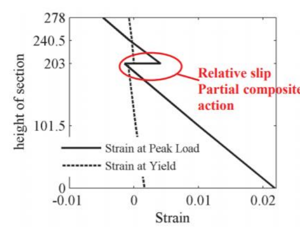
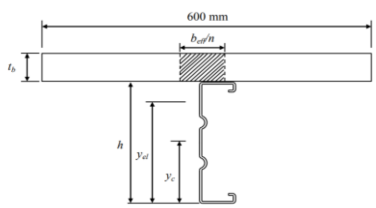
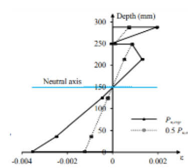
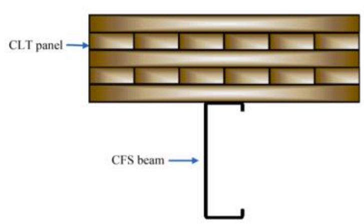
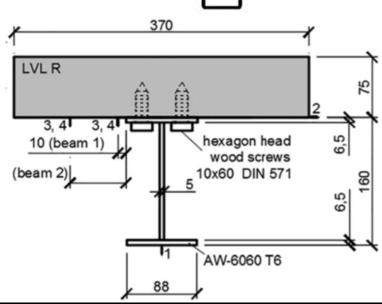
In engineering terms, composite action refers to two individual elements acting as one system. In the case of beam and slabs, it means that at their interface, the load carrying capacity of both materials is utilised. Composite design combines the advantages of the individual elements used. In the case of steel and timber, timber is brittle in tension and ductile in compression. Overall, steel performs better than timber in strength and stiffness, with sufficient ductility. Thus, with the timber slab at the top, composite design can utilise the ductility of timber in compression without worrying about brittle tensile failure (for a simply supported case). Even with the large difference in mechanical properties between steel and timber, there can be gain in stiffness and resistance owing to the increase in the structural height of the section. Composite design will allow us to utilize the individual sections to the maximum. In this thesis, the load carrying capacity of the timber slabs is utilized to the aid the steel beam on which it is supported. Such type of composite action for steel-concrete is already used in design, and can lead to sufficient benefits.

Short-term structural behaviour of steel – *CLT* beams (henceforth referred to as '*STC* sections') under 4-point bending have been investigated by Hassanieh et al [15]. The results showed that there was an increase of at least 50% in stiffness and ultimate load carrying capacity, whilst maintaining sufficient ductility. Similar experiments were done on steel – *LVL* composite beams. The strain distribution for demountable shear connectors (coach screws, bolts) were similar in the sense that both showed significant slip at the interface between steel and timber. Pinelope et al [17] conducted 4-point bending tests on floor system with wooden floorboards and cold – formed C sections. It was concluded from experimental values that there was an increase of 40% in the bending moment capacity and 15% increase in stiffness. The failure modes observed were due to local buckling of the web of steel beams and distortional buckling in the top flanges. This is owed to the use of Class 4 sections as the steel joists. Assuming a plastic stress distribution, the neutral axis was calculated to be either in the timber slab, or in the top part of the steel web. Thus, it can be stated that the individual resistances of the elements are comparable to each other, in this case. It should also be noted that the strain distribution of the specimens without adhesives show significant slip in the interface of steel and timber, similar to Hassanieh's results [15]. Navaratnam et al [25] conducted experiments and finite element analysis on a composite system with cold-formed steel C sections and *CLT*. In this case, it was found that the load carrying capacity of the composite beam increased as much as 800% compared to the steel beam, and 37% compared to the *CLT* slab. It should be noted that with the use of smaller steel sections as joists, there is significant increase due to composite action. Chybinski et al [16] conducted 4 point bending tests on aluminium - timber composite beams. The experimentally obtained ultimate load showed good accuracy with the ultimate load calculated from a plastic stress distribution (upto 92.5%). The summary of the results of the experiments mentioned above is given below in Table 3.3.

Figure 3.10 Failure Modes in STC beams with CLT and cold formed C-sections, from [17]. (Left) Distortional Buckling. (Right) Web local buckling.

The orthotropic nature of timber must be considered, while striving for composite efficiency. For this reason, the experiments above use *CLT* and *LVL*, which offer higher strength and stiffness in the perpendicular direction. In a conventional steel timber floor system, the longitudinal sections of steel and timber are aligned perpendicular to each other. Hence, while considering composite design, we are left with the properties of timber in the direction perpendicular to the grains, which will influence the final benefits to be obtained due to composite action.

Table 3.3: Summary of Experiments on Steel-Timber Composites.

Source	Description	STC Section	Strain Distribution	Increase % Strength	Increase % Stiffness
[24]	*4 Point Bending.* *CLT, Steel I Beam, Bolts.*			*50%*	*50%*
[24]	*4 Point Bending.* *LVL, Steel I Beam, Bolts.*			-	-
[17]	*4 Point Bending.* *Timber Floorboard, Cold-formed C Section, Self-drilling screws.*			*40%*	*15%*
[25]	*4 Point Bending.* *CLT, Cold-formed C Section, Self-drilling screws.*		-	*800% /37%*	
[16]	*4 Point Bending.* *LVL, Aluminium I Beam, Hexagonal wood screws.*		-	-	-

To enable demountability, we require demountable shear connectors at definite intervals i.e., we cannot opt for adhesive bonding. This results in a semi rigid connection between steel and timber, and hence there will be slip at the interface. Hence for the analysis of such *STC* sections, we need to account for this slip at the interface.

It should also be noted here that all the specimens considered above combined large sized timber sections with comparable or smaller steel sections. The large difference in the mechanical properties of steel and timber is factored, while designing these specimens. In contrast to this, the sections which are considered in thesis have large steel sections. Thus, the overall benefits due to composite action will be different from what is observed in these experiments.

3.7 Summary

In the comparison of steel timber floor systems that follow in the next chapter, it is opted to choose the following timber decks and steel beams:

Table 3.4: Summary Steel and Timber Products used in Europe.

Timber Decks		Steel Beams		
1. CLT Solid Slabs	5. Lignature Surface Elements	1. IPE	5. Castellated IPE	9. THQ
2. CLT Open Rib Elements	6. Kerto Ripa Open Rib Elements	2. HEA	6. Castellated HEA	10. SFB
3. CLT Box Elements	7. Kerto Ripa Box Elements	3. HEB	7. Castellated HEB	11. IFB
4. Lignatue Box Elements	8. Kerto Ripa Open Box Elements	4. HEM	8. Castellated HEM	12. RHS Beams
				13. L Profiles

Due to the same reason for avoiding timber joist floors, the examples of *STC* sections considered in the experiments for composite action are not considered in this thesis. The steel beams used are very slender, cold formed, Class 4 sections i.e., they cannot be used to traverse large spans. This would result in the requirement of an increased number of elements (slabs and beams), and also an increased number of shear connectors to establish a demountable connection between them. This would greatly impede the process of disassembly.

For establishing full shear interaction between steel and timber (i.e., for a non-slip shear connection), it is required to use adhesive bonding. It is decided to choose bolts as the demountable shear connector. Using discrete shear connectors such as bolts means that only partial composite action can be achieved. This requires design methods which factors in the slip at the interface. Thus, the Gamma Method (used for built-up timber sections) is proposed for the elastic analysis of *STC* floors, to calculate the effective strength and stiffness.

For the case study, it is opted to choose Bouwdeel D, as this building serves as a model for circular design in the Netherlands. The timber slab used here are the Kerto RIPA floor elements. The reason for choosing this slab was its low price (compared to other timber products). Both this slab, and the customised L shaped beam used in Bouwdeel D are considered for the *MCA* in the following chapter. The result of the *MCA* will help to identify the best possible *STC* floor considering all aspects, especially sustainability and circularity. Calculations in the *MCA* and in the Case Study that follows will shed light on the advantages of this ‘best’ *STC* floor over conventional floor systems.

4. Multi Criteria Analysis

In this chapter, the optimum demountable steel timber floor system is chosen from the all the possible combinations of steel floor beams and timber decks. This is done with the help of a multi criteria analysis (*MCA*). In Section 4.1, the method of conducting the *MCA* is explained. Then the parameters considered for comparison are described in Section 4.2. In Section 4.3, the results of the *MCA* are discussed, from which the best *STC* floor system is chosen. Only this system is considered further on in the thesis. The detailed calculations required for rating the different parameters and applying different weight factors to get a cumulative score for the *MCA* floor system is given in Appendix B.

4.1 Procedure for MCA

'Multi criteria decision analysis' or 'multi criteria analysis' (*MCA*) is an objective method of measuring the preference to a certain choice, among the given alternatives, for fulfilling a specific task/function [70]. Hence a well conducted *MCA* requires the following: 1. Functional Unit (*FU)*, which defines the requirements to be fulfilled, 2. Alternatives of Solutions which need to be compared, 3. Criteria to compare different alternatives, 4. Rating of these criteria based on specific limits, and 5. Method of aggregating different ratings to obtain a final cumulative score. The ratings are aggregated based on the new theory of proper measurement by Barzillai et al [2], which is based on a weighted summation.

In this *MCA*, the requirement is to obtain the best demountable *STC* floor system. The requirement for demountability arises from the need to reuse the structure, to decrease the toll on the environment. Hence the requirements for demountability, reusability and sustainability, along with the nominal requirements of a structural system to comply with the Dutch building decree [1] are converted into the criteria for comparison. The floors are rated from 1-5 (5 being the best and 1 being the worst) based on specified limits. The limits can be relative to the obtained values, or from literature. Once the floor systems have been rated for different parameters, these ratings are aggregated into a cumulative score using different weight factors. The final or cumulative scores are compared to objectively choose the best *STC* floor system. As the final step, the same procedure is repeated with a different functional unit, to get consistent results.

As this stage of the research involves a broad study into the different products available in the Netherlands, manufacturer's span tables were used for the different structural elements. In this thesis, as far as it was possible, an effort has been made to maintain uniformity in selecting the required cross sections. However, there were a few limitations. Assumptions were made whenever required, based on engineering judgement.

Limitations and Assumptions:

1. For the timber decks span tables used were for an equivalent loading of dead load and office category live loads. The specifics for each timber deck is given in Appendix A.
2. For SLS delfection criteria of the timber decks, the limits were set for instantaneous and final deflection. The specifics are given in Appendix A.

3. The structural wood used is with strength class as given in Appendix A.1 for the different products.
4. For all timber decks except *CLT* solid slab elements, the encapsulation strategy for fire protections is used (by providing an additional layer of gypsum) for fire safey. For the latter, the reduced cross section method was used as the manufacturer's span tables were available only for REI90.
5. For sound insulation of the timber decks, specific layups were adopted which have been tested and recorded in literature [40-42]. The same sound insulation layup was used for both the functional units. However, in reality ***FU1*** (defined below) with a larger cross section would require a smaller layup for sound insulation compared to ***FU2***.
6. In practise, for adopting a suitable timber floor, the vibration requirements (in terms of the natural frequency and stiffness) have to be satisfied. In the span tables used, the *CLT* products taken satisfied the vibration requirements with an assumed wet screed, whereas the Lignatur and the Kerto Ripa products did not meet any such criteria. It is assumed that the requirements of *EC5* [53] can be met by using an additional layers.
7. The steel used is assumed to be S355 with f_y = 355 MPa and *MOE* = 200 GPa.
8. The steel beams are designed for *ULS* stresses and *SLS* deflection limit of L/300. They are not precambered.
9. The integated steel beams are additionally checked for the load carrying capacity of the bottom flange.

The functional units used in the *STC* have been defined as follows:

Functional Unit 1 (FU1): *"A continuous one-way spanning floor slab, with* ***span = 7 m****, fire safe time of 90 minutes, and sound insulation complying to the Dutch building decree (R_w > 54 dB and $L_{n,w}$ < 52 dB)* [1]*, designed for Office Category Loads. The slabs are supported using a simply supported steel beam of span 9 m."*

Functional Unit 2 (FU2): *"A continuous one-way spanning floor slab, with* ***span = 5 m****, fire safe time of 90 minutes, and sound insulation complying to the Dutch building decree (R_w > 54 dB and $L_{n,w}$ < 52 dB)* [1]*, designed for Office Category Loads. The slabs are supported using simply a supported steel beam of span 9 m."*

From literature it was found that the lowest of the maximum spans permitted by the timber decks is 7 m (*CLT* solid slab elements), and the highest of the minimum spans that can utilize all the timber decks considered is 5 m (*CLT* open rib elements). It should be noted that even with span 5 m, *CLT* box elements cannot be completely utilized. However, by using a larger span for ***FU2***, there is very less difference with ***FU1***. Hence 5 m was chosen, and consequently *CLT* box elements were left out the *MCA* for ***FU2***. When combined with the respective slabs, it was found that not all the steel beams could span more than 9 m (by limiting SLS deflection to L/300). Hence the dimensions 7 x 9 m and 5 x 9 m have been adopted for ***FU1*** and ***FU2*** respectively.

As explained in Chapter 2, in this thesis 8 timber decks and 13 steel floor beams have been considered. Considering all the possible combinations, we would be left with 104 *STC* floor systems. To minimise efforts, and to avoid unnecessary calculations, the whole *MCA* has been split into 3 parts:

MCA of Timber Decks (MCA1): Considering Timber Decks alone. When combined with steel floor beams to form ST floor system, the only parameter that is different (than when standalone timber decks are compared) is the demountable connection between the slab and beam. This is mainly determined by the cross section of the timber deck. Hence the goal of this *MCA* is to obtain the best timber decks, each having a different type of cross section. This leaves us 3 types of timber decks: 1. Solid slab elements, 2. Box elements and 3. Open rib elements, reducing the total number from 8.

MCA of Steel Floor Beams (MCA2): Of the 13 steel floor beams considered, 8 have 'I' cross section with variation in the thickness of the members. An *MCA* is done among these to choose one. Similarly, the integrated beams considered (*THQ*, *SFB* and *IFB*) serve the same purposes, and are similar in many respects. Another *MCA* is done with these 3 to select the best integrated beam to be combined into a *STC* floor. Using these simplifications, we are left with 4 steel floor beams, based on their cross section and function: 1. (Castellated) I Beams, 2. Integrated Beams, 3. *RHS* Beams, and 4. Double L Profile.

MCA of STC floor systems (MCA3): Combining the 3 timber decks and the 4 steel floor beams we are left with 12 *STC* floor systems, reduced from 104. The *STC* floors obtained thus are distinct in the way they are connected. An *MCA* comparing these systems will provide us the best *STC* floor system that can be adopted. The whole procedure for choosing 1 *STC* floor system out of 104 systems is explained below in Figure 4.1.

4.2 Parameters for Comparison of MCA

All the parameters considered for the *MCA* are listed here. As mentioned above, 3 separates *MCAs* are conducted to get the final result with minimal calculations. Hence for each parameter, the ratings are given separately for timber decks, steel beams and *STC* floor systems. The detailed calculations for different ratings are given in Appendix B.2.

<u>1. Slenderness:</u> The total depth determines the aesthetics of the floor system. Also, deeper floors would require larger area for the façade coverings. Hence it is desirable to have slender floors. For timber decks the depths of the whole layup is considered (i.e. sound insulation, fire safety and structural timber). For steel beams, only the height of the cross section of steel is considered. For comparing *STCs*, the total height of the combined system including service installations is considered.

<u>2. Weight:</u> The weight of the floor system determines the total load on the columns and finally on the foundations. It also determines the ease with which cranes can handle the various elements on-site or for loading/unloading.

<u>3. Building Decree:</u> The Dutch building decree [1] imposes several criteria to design a floor system. Among these, the structural requirements for *ULS* and *SLS* deflections are already satisfied when we choose an adequate cross section for the elements of the functional unit. However, to meet the requirements for *SLS* vibrations and building physics aspects, we need specific additions. Hence the rating for building decree is based on these 3 sub-criteria. For sound insulation and fire protection, the rating is based on the weight of the additions. For vibrations, the rating is based on the natural frequency and stiffness of the floor system.

<u>4. Demountability:</u> This parameter measures the ease of disassembly of the *STC* floor system and depends on the connection between the timber deck and the steel beam. It is measured using the demountability index (Appendix B.2.4). Having lesser number of individual components also aids in the process of disassembly. Hence the ratings for this parameter incorporate both the number of elements and the ease of dismantling.

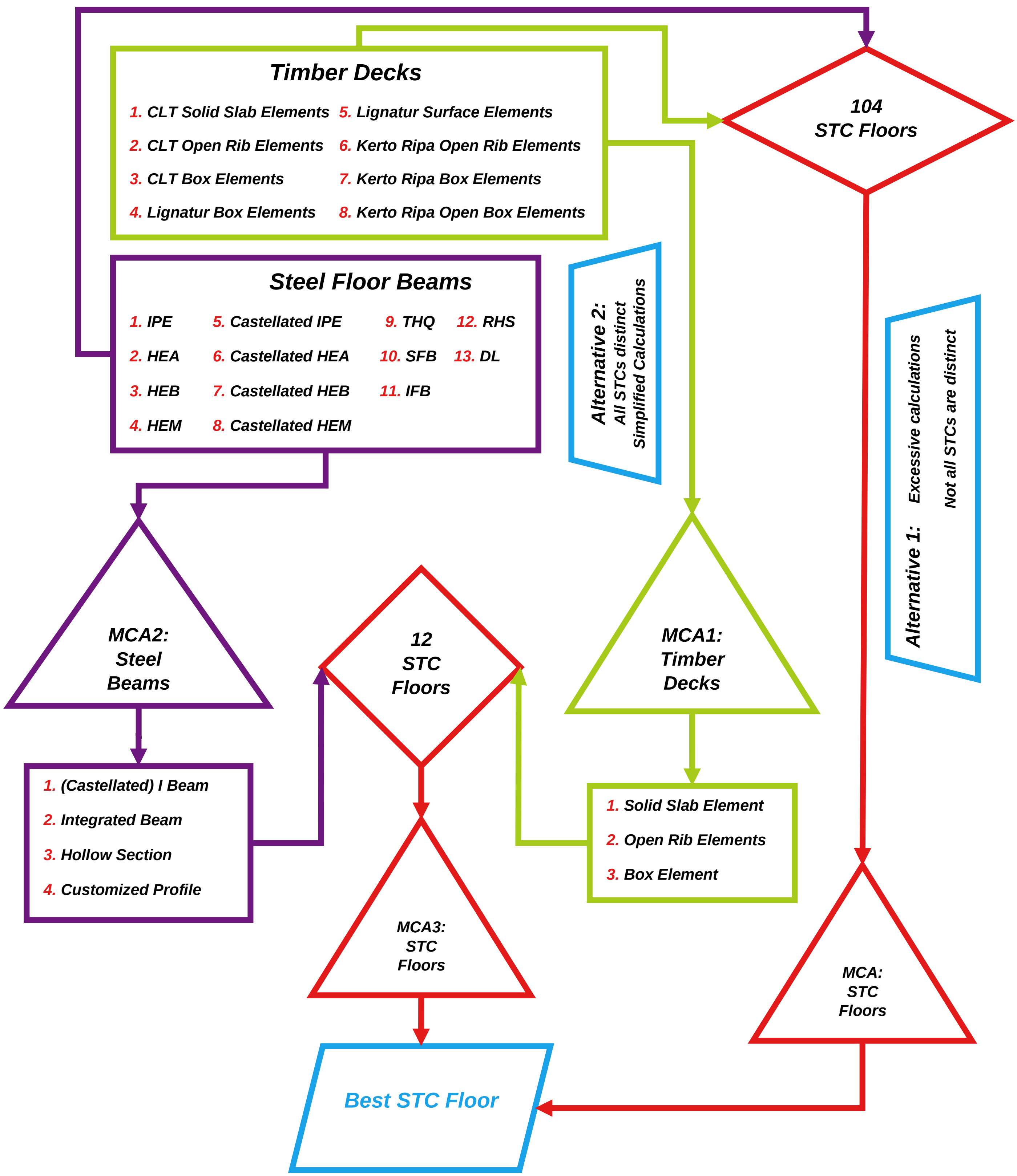


Figure 4.1: Overview of procedure for MCA.

5. Sustainability: The toll on the environment is measured using the *ECI* per unit area of the floor system. For parameter, the *ECI* values of the materials used are obtained from the *NMD*.

6. Logistics: This parameter is used to compare the effort required for transporting the structural elements of the floor system, in terms of the number of trucks required for transportation. As the floor systems are designed for disassembly, this will determine the ease of assembly and reassembly.

7. Flexibility: When the *STC* floor system offers larger spans, there is freedom for re-orientation of the building to cater to the needs of different clients during its life cycle. As this parameter lies outside the boundaries of the functional unit, it is awarded as a bonus point.

4.3 Results of the MCA

The detailed calculations for dimensioning the structural elements, and how the ratings for different parameters were obtained, is given in Appendix B.1.

Once the ratings were obtained, they are aggregated into a single score, for direct comparison. The total score of an element is obtained as the sum of the ratings of an element for each parameter multiplied by the weight factor of the respective parameter. To get consistent results from the *MCA*, the weight factors are changed to see the difference in the results. For this we divide the 7 parameters considered in the *MCA* into 2 main categories.

Functionality (F) → Refers collectively to Slenderness, Weight and Building Decree. These are the main requirements for a conventional floor system. Between these 3 parameters, the ratio of their contribution is 1 : 1 : 1.5 . Building decree is given a higher weightage as it consists of 3 sub-criteria.

Circularity (C) → Refers collectively to Demountability, Sustainability, Logistics and Flexibility. These are the aspects required in addition to the former, for a demountable floor system. Between these 4 parameters, the ratio of their contribution to the total score is 1.67: 1.67 : 1.67 : 1. The contribution of flexibility is lesser as the max rating is only 1 (compared to 5 for the other 3 parameters).

The total score is fixed at 1000. By fixing the distribution of the contribution within the collective criteria *Functionality* and *Circularity*, the percentage contribution between these 2 main criteria is varied. Thus, we obtain 4 different scores based on different weight factors obtained by changing the percentage of contribution of Circularity from 45% to 60% in multiples of 5%.

4.3.1 MCA of Timber Decks

Based on the procedure mentioned above, the results of the *MCA* for timber decks are given below:

Table 4.1: Final Scores of MCA for Timber Decks.

Functional Unit 1				
Timber Decks	**Score 1**	**Score 2**	**Score 3**	**Score 4**
		Solid Slab Elements		
CLT_SS260	*613.5*	*613.8*	*614.1*	*614.4*
		Open Rib Elements		
CLT_OR360	*670*	*656.6*	*643.1*	*629.7*
KR_OR475	*464*	*480.3*	*496.6*	*513*
KR_OB428	*464*	*480.3*	*496.6*	*513*
		Box Elements		
CLT_BE320	*770.8*	*763*	*755.2*	*747.4*
LK240	*746.8*	*736*	*725.2*	*714.4*
LF240	*803*	*792.3*	*781.5*	*770.8*
KR_BE410	*624.7*	*633.4*	*642*	*650.7*
Functional Unit 2				
Timber Decks	**Score 1**	**Score 2**	**Score 3**	**Score 4**
		Solid Slab Elements		
CLT_SS180	*591.6*	*596.1*	*600.6*	*605.1*
		Open Rib Elements		
CLT_OR260	*721.5*	*714.4*	*707.4*	*700.4*
KR_OR325	*581.1*	*591.2*	*601.4*	*611.5*
KR_OB293	*524.9*	*535*	*545*	*555.1*
		Box Elements		
[-]	*[-]*	*[-]*	*[-]*	*[-]*
LK160	*764*	*755.3*	*746.6*	*738*
LF160	*797.3*	*785.8*	*774.4*	*763*
KR_BE250	*708.5*	*713.8*	*719*	*724.2*

It can be observed that for open rib elements, the *CLT_ORE* elements consistently score the highest. This can be attributed to the high spans offered by it. The Kerto Ripa elements score very low on sustainability. The *ECI* value of *LVL* is very high compared to normal and laminated timber, due to the use of large of amounts of adhesives for its production. *CLT* open rib elements are also slightly more slender than the Kerto Ripa elements.

Comparing the box type timber decks, *LFE* scores the highest, consistently. This is due to its low weight, and high sustainability and slenderness. Compared to *LKE*, *LFE* scores higher on the aspect of demountability (as less number of elements are used). It also weighs slightly lesser due to thinner webs.

Thus, it opted to choose *LFE* and *CLT_ORE* to combine with the steel floors from ***MCA2***. As *CLT_SS* elements do not score well in ***MCA1***, it is decided that it will be combined only with an I beam to form an *STC* floor, for comparison in ***MCA3***. The reason for combination with I beams is because they score well in ***MCA2***. At the same time, they have a constraint that it can only form a non-integrated system with the timber decks. Hence it is opted to combine the I beam with 1) *LFE* and 2) *CLT_SSE*, as these are the 2 most slender timber decks.

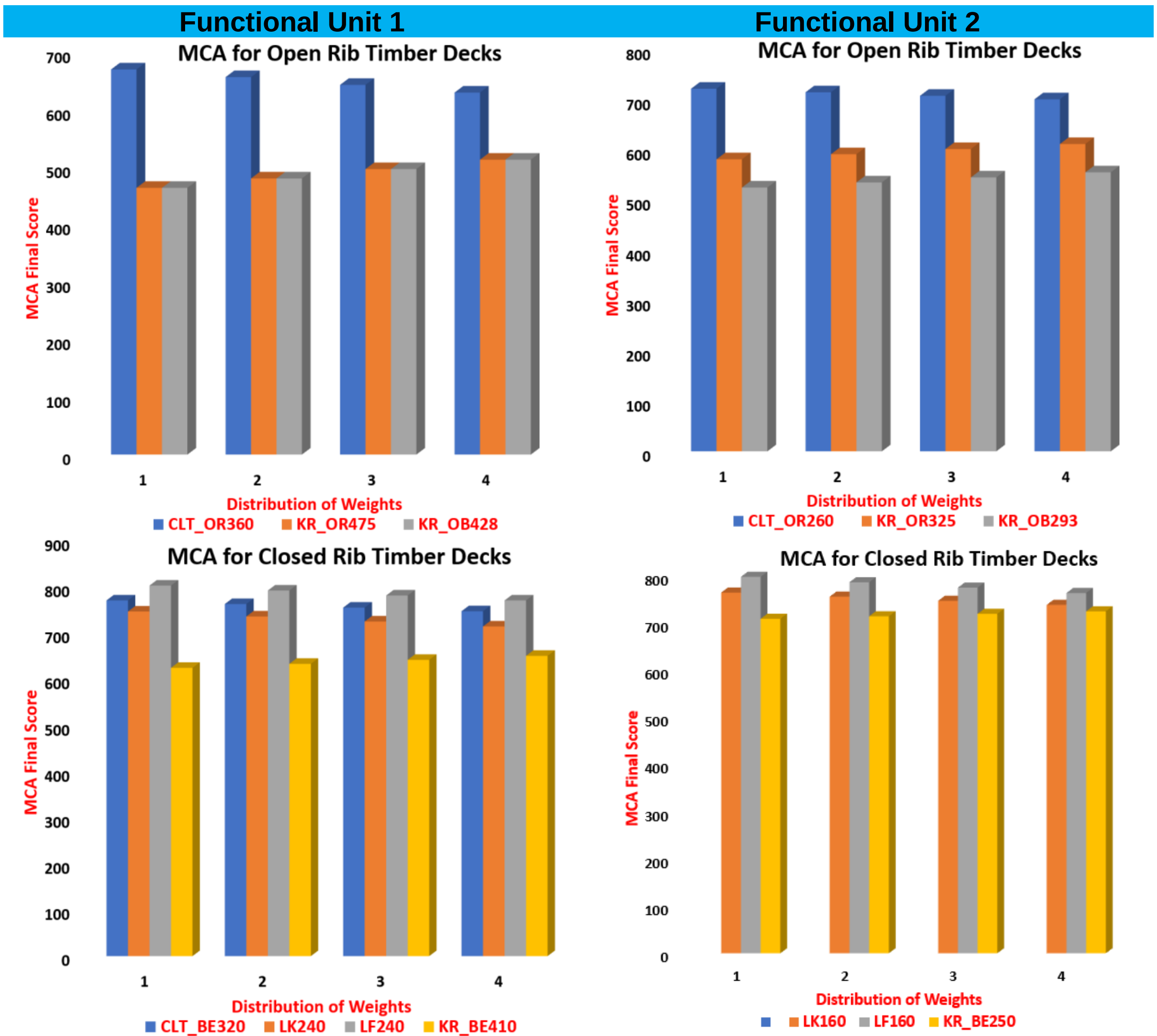


Figure 4.2: Results of MCA for Timber Decks.

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4.3.2 MCA of Steel Beams

Using the similar procedure for timber decks, the results of the *MCA* for steel beams are given below:

Table 4.2: Total Scores for MCA of Steel Beams.

Functional Unit 1				
Timber Decks	**Score 1**	**Score 2**	**Score 3**	**Score 4**
		I Beams		
IPE550	*650*	*643.7*	*637.5*	*631.2*
HEA500	*540*	*545*	*550*	*555*
HEB450	*680*	*677.5*	*675*	*672.5*
HEM320	*580*	*590*	*600*	*610*
CIPE672	*640*	*632.5*	*625*	*617.5*
CHEA581	*600*	*600*	*600*	*600*
CHEB535	*600*	*600*	*600*	*600*
CHEM422	*700*	*700*	*700*	*700*
		Integrated Beams		
THQ420	*610*	*611.2*	*612.5*	*613.7*
SFB410	*710*	*698.7*	*687.5*	*676.2*
IFB410	*710*	*700*	*700*	*700*
		Other Beams		
RHS500	*510*	*523.7*	*537.5*	*551.2*
DL425	*700*	*700*	*700*	*700*
Functional Unit 2				
Timber Decks	**Score 1**	**Score 2**	**Score 3**	**Score 4**
		I Beams		
IPE500	*590*	*588.7*	*587.5*	*586.2*
HEA450	*480*	*490*	*500*	*510*
HEB400	*600*	*600*	*600*	*600*
HEM300	*580*	*590*	*600*	*610*
CIPE594	*640*	*632.5*	*625*	*617.5*
CHEA489	*580*	*577.5*	*575*	*572.5*
CHEB445	*510*	*523.7*	*537.5*	*551.2*
[-]	*[-]*	*[-]*	*[-]*	*[-]*
		Integrated Beams		
THQ340	*500*	*512.5*	*525*	*537.5*
SFB340	*610*	*611.2*	*612.5*	*613.7*
IFB340	*640*	*632.5*	*625*	*617.5*
		Other Beams		
RHS450	*470*	*491.2*	*512.5*	*533.7*
DL345	*640*	*632.5*	*625*	*617.5*

The main aspect that we learn from ***MCA2*** is that there is very little variation between steel beams of the similar type. As we can see from the results of the integrated beams, all 3 integrated beams score similarly. The *IFB* scores slightly higher, and hence this is used to represent integrated beams in ***MCA3*** for *STC* floors.

Among I beams, the castellated *HEM* beam scores the highest consistently through ***MCA2***. However, while comparing *STC* floors, the contribution of the beam to parameters such as weight and sustainability are negligible, i.e. they are mostly influenced by the type of slab used. The *STC* floors formed from combination with I beams are non-integrated systems. Therefore, the main aspect of I beams that affects the results of ***MCA3*** of *STC* floors is the slenderness. Also, it should be noted that all I beams score well compared to the other steel beams considered. Hence, it is decided to choose *HEM* beams (as this is the most slender I beam) for representing I beams in the ***MCA3*** of *STC* floors.

The remaining two beams, namely *RHS* and Double L profile, are taken irrespective of the *MCA* results. This is because the objective of the *MCA* is to compare distinct *STC* floor systems. It should be noted that double L beams score very high as they are customised beams designed for maximum utilization in the functional unit.

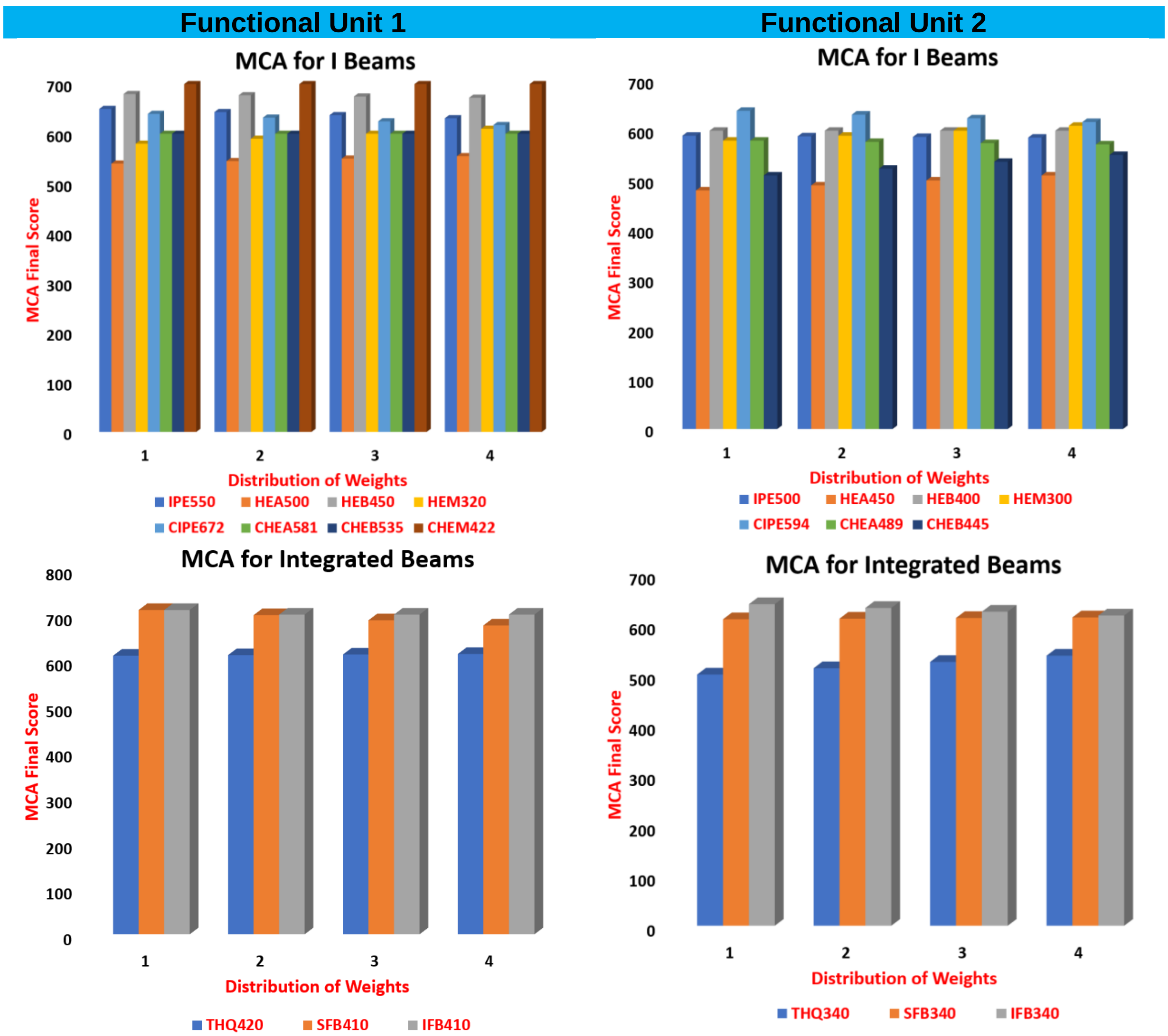


Figure 4.3: Results of MCA for Steel Beams.

4.3.3 MCA of STC Floors

Using the results of ***MCA1*** and ***MCA2***, we choose 8 *STC* Floors formed by the different combinations of timber decks and steel beams. These are given below in Table 4.3:

Table 4.3: STC Floors selected for MCA3.

FU1			FU1		
STC Floors	**Timber Decks**	**Steel Beams**	**STC Floors**	**Timber Decks**	**Steel Beams**
Non-Integrated Systems			**Non-Integrated Systems**		
STC1a	*CLT_SS260*	*HEM320*	***STC1b***	*CLT_SS180*	*HEM300*
STC2a	*LF240*	*HEM320*	***STC2b***	*LF160*	*HEM300*
Integrated Systems			**Integrated Systems**		
STC3a	*CLT_OR360*	*IFB410*	***STC3b***	*CLT_OR3260*	*IFB340*
STC4a	*LF240*	*IFB410*	***STC4b***	*LF240*	*IFB340*
STC5a	*CLT_OR360*	*RHS500*	***STC5b***	*CLT_OR3260*	*RHS450*
STC6a	*LF240*	*RHS500*	***STC6b***	*LF240*	*RHS450*
STC7a	*CLT_OR360*	*DL425*	***STC7b***	*CLT_OR3260*	*DL345*
STC8a	*LF240*	*DL425*	***STC8b***	*LF240*	*DL345*

From ***MCA1***, we are left with 3 timber decks: *LFE*, *CLT_ORE* and *CLT_SSE*. *LFE* is the best among box slabs and *CLT_ORE* is the best among the open rib slabs. *CLT_SSE* is the only solid slab considered for comparison. As it has a mediocre score in ***MCA1***, it is decided to combine this with I beams only. The reason is that *CLT_SSE* are very slender, and hence performs the best as a non-integrated *STC* floor system. From ***MCA2***, 4 steel beams are chosen for combining with timber decks: *HEM*, *IFB*, *RHS*, Double L profile. Except for *HEM*, all the beams form integrated systems with the timber decks. Hence it is decided to combine *HEM* only with the 2 most slender timber decks: *LFE* and *CLT_SSE*. Each of the remaining steel beams are combined with the 2 timber decks chosen from the ***MCA1*** of timber decks: *LFE* and *CLT_ORE*. This is how the above mentioned 8 *STC* floor systems are chosen. In other words, among the 104 possible combinations of *STC* floors, these 8 are the most distinct combinations. ***MCA3*** conducted on these 8 *STC* floors will yield the best solution from all possible combinations. This saves time and effort compared to directly comparing all combinations of *STC* floors.

The ratings are done as mentioned previously. The only parameter that is different from the previous two *MCA* is demountability. Demountability depends on the connection between the timber decks and the steel beams. All *STC* floors are designed to be demountable, however they still differ in the ease of dismantling. The scores are aggregated in the same way, and the ratio of contribution between sub parameters of Functionality and Circularity are the same as for the timber decks. The results of the ***MCA3*** for *STC* floors are given below in Table 4.4 and Figure 4.4:

Table 4.4: Total Score of MCA for STC Floor Systems.

Functional Unit 1				
STC Floors	**Score 1**	**Score 2**	**Score 3**	**Score 4**
STC1a	*569*	*553.3*	*537.6*	*521.9*
STC2a	*736.8*	*719.8*	*702.9*	*686*
STC3a	*519.1*	*527.7*	*536.4*	*545*
STC4a	*666.3*	*663.5*	*660.7*	*657.9*
STC5a	*505.8*	*515.5*	*525.3*	*535*
STC6a	*627*	*622.1*	*617.1*	*612.2*
STC7a	*519.1*	*527.7*	*536.4*	*545*
STC8a	*689.2*	*689.2*	*689.3*	*689.3*
Functional Unit 2				
STC Floors	**Score 1**	**Score 2**	**Score 3**	**Score 4**
STC1b	*467.9*	*467.3*	*466.8*	*466.2*
STC2b	*706*	*692.1*	*678.3*	*664.5*
STC3b	*710*	*718.3*	*726.5*	*734.7*
STC4b	*732.4*	*727.5*	*722.5*	*717.5*
STC5b	*652*	*659.3*	*666.5*	*673.7*
STC6b	*641.1*	*637.9*	*634.7*	*631.5*
STC7b	*660*	*672.4*	*684.8*	*697.2*
STC8b	*710.4*	*713.1*	*715.8*	*718.5*

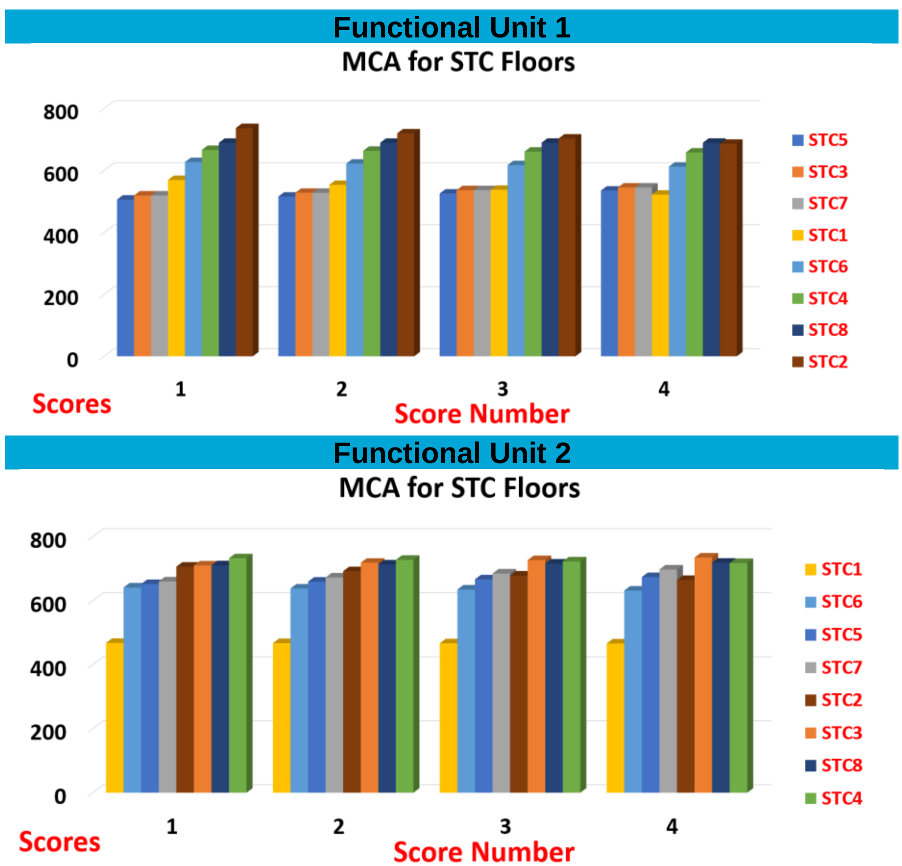


Figure 4.4: Results of MCA for STC Floor Systems

The top 3 *STC* floor systems in each score is given below in Table 4.5, for each functional unit.

Table 4.5: Top 3 STC Floor Systems.

Functional Unit 1			
Score	**First Choice**	**Second Choice**	**Third Choice**
Score1	*STC2*	*STC8*	*STC4*
Score2	*STC2*	*STC8*	*STC4*
Score3	*STC2*	*STC8*	*STC4*
Score4	*STC8*	*STC2*	*STC4*
Functional Unit 2			
Score	**First Choice**	**Second Choice**	**Third Choice**
Score1	*STC4*	*STC3*	*STC8*
Score2	*STC4*	*STC3*	*STC8*
Score3	*STC3*	*STC4*	*STC8*
Score4	*STC3*	*STC8*	*STC4*

From the results of *MCA*, it can be observed that as long as Circularity is given at least 50% weightage to the total score, ***STC2*** consistently scores the highest in ***FU1***. The reason for this lies in the fact that the *LFE* slabs used were best option among all slabs. The I beams considered were also good. The disadvantage in floor height is more than compensated when it comes to the aspect of demountability. Non-integrated floor systems offer ease of demountability, as they are not constrained to move in the sideways direction. It can be observed that the values of the *STC* floor is mostly dominated by the corresponding timber slabs. Thus, the *STC* floors using *LFE* sections come out as the best options.

In ***FU2***, the results are not as consistent, as in the case of ***FU1***, neither is there any agreement between the results of the 2 functional units. ***FU2*** with span of slab = 5 m, represents a value less than the minimum span that most of the slabs can obtain. Hence, ***FU1*** gives a more a correct value. Even in the case of ***FU2***, ***STC2*** is not far behind, although it is not among the top three. Finally, the remaining 3 *STC* floors which give high scores – all of them are integrated floor systems. The possibility of composite action will be more pronounced in the case of ***STC2*** which has the advantage of extra floor height. If this were considered, this could lead to added advantages. Thus, it is opted to choose ***STC2*** the best demountable *STC* floor system, and this will used for further analysis.

4.4 Summary of the MCA

Since the focus of this research is on sustainability and circularity, it is opted to choose ***STC2*** to represent the best possible demountable steel timber floor system, to be investigated further. ***STC8*** (which scores second best in the *MCA* overall) is an integrated system and serves the advantage of small floor heights. However, integrated systems are constrained in the possibilities for movement (for demounting), and it is owing to this aspect that the scores ***STC2*** higher. Apart from all the parameters considered in the *MCA*, there is also the possibility of applying composite action in ***STC2***, which can lead to more savings in materials, and a reduction in floor height. This aspect will be looked into in Section 6.4.

5. Case Study

5.1 Introduction

The building chosen for the case study is Bouwdeel D, which was designed to be demountable. Initially designed as a temporary structure, all components can be disassembled and reused. The original design was chosen such that it would serve as a symbol for circularity, for future projects. Figure 5.1 below shows the 3D impression of the kit of parts of Bouwdeel D.

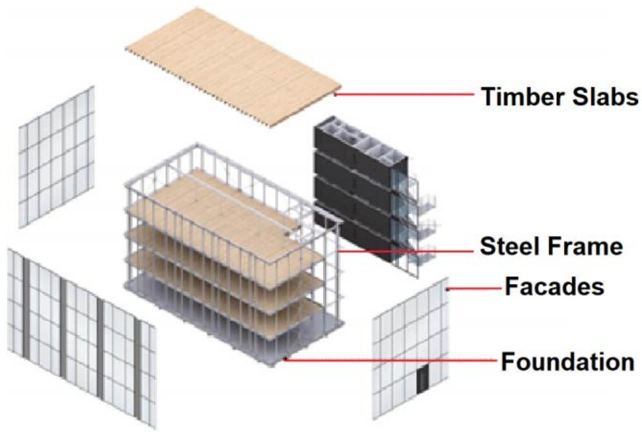


Figure 5.1: Bouwdeel D - Kit of Parts, from [18].

5.1.1 Goal and Scope of Case Study

The goal of this case study is to compare the benefits of using the *STC* floor system obtained from the *MCA* of different steel-timber floor systems in Chapter 4, which is to be designed as completely demountable and reusable i.e., the *STC* floor should serve its use beyond the lifetime of the building. Normally, office buildings are designed for a service life of 50 years [55]. Hence the reference service life for this case study is chosen to be 100 years i.e., at least twice the nominal service life of permanent structures. As mentioned in Section 2.3, 2 scenarios of reuse can be considered: 1) Reorientation, where the floor system is used at the same location for a longer service life of 100 years and 2) Relocation, where the floor is designed to be used in 2 cycles (of service life 50 years each for the same function). However, since the deterioration of the mechanical properties of timber elements during its first service life cannot be predicted during the initial design phase, it is decided to consider only Reuse by Reorientation. Designs for the *STC* floors are done with and without considering composite action between the timber slab and the steel beam. To make an accurate comparison with other industry standard floor systems, design is done using Hollow Core Slabs (*HCS*) and steel-concrete Composite Slabs (*CS*). A comparison is made between the floor systems, in terms on the total weight per unit area, which influences the size of foundations and transportation of the components, and the total floor height. Finally, the structural design in each alternative will serve as an input to determine the amount of material for the *LCA* in Chapter 7. The results of the *LCA* will show light on the advantages of an *STC* floor system in terms of its environmental consequences, as addition to its benefits related to structural aspects. We can also understand whether the gains due to composite action between steel and timber justify the use of extra shear connectors.

To summarize, the following design alternatives will be considered:

- **Design Alternative 1 (DA1_STC):** The floor is designed with steel timber *(STC)* to be used at the same location for a service life of 100 years (Reuse by reorientation). This will cater to the needs of different clients, by providing freedom of remodelling (due to maximum column-free spans). The design is made for the *STC* beam with and without composite action. As mentioned is Section 2.3, when initially designed for 100 years, only the live loads need to be adjusted (according to Eq. 1).
- **Design Alternative 2 (DA2_HCS):** The floor is designed with Hollow Core Slabs (*HCS*). Similar to *STC*, the floor is designed for reuse by roerientation, with the same assumptions.
- **Design Alternative 3 (DA3_CS):** The floor is designed steel-concrete Composite Slabs. For designing the cross beams (supporting the slabs), composite action is considered. Similar to ***DA1_STC*** and ***DA2_HCS***, the floor is designed for reuse by roerientation, with the same assumptions.

Further, it should be noted that all design is done assuming the elastic properties of steel/timber, so that there is maximum probability for reusing the structural elements. The results of this case study can be used to determine its environment impact over a reference period of 100 years. All beams and columns are designed with steel, for uniformity. The floor will be designed by excluding the openings for the stairs/elevators. Foundations and facades are left out of the scope of this case study. Since the goal is to compare different floor systems, the design aspects will be limited to the following elements:

- **Floor Slabs**
- **Primary Beams** (supporting the slabs)
- **Secondary Beams** (edge beams)
- **Columns**

For the ***DA1_STC**,* a few connections will be designed, including those for the diaphragm action and composite action. This is to determine the extra amount shear connectors required for composite action, over the minimum number required for diaphragm action of the floor system. However, the connections will not be designed for the other design alternatives. In Section 6.4, the procedure to design the *STC* with composite action is provided. For ***DA3_CS***, to design with composite action, full shear interaction is assumed, without looking into the details of the shear connectors. This is so that a comparison can be made between the benefits of composite action in *STC* and Composite Slabs.

Finally, for a fair comparison of timber with the other slabs, it is ensured that all the all the slabs satisfy a fire safe time of 90 minutes, and also complies with the requirement of sound insulation according to the Dutch Building decree [1]. The additions required for *LFE* to fulfil this criterion is adopted from Chapter 4. *HCS* and Composite slabs are designed with span tables for 90 minute fire safe time. Hence, without additional checks, it is assumed that they comply to this criterion. For sound insulation, no additional layups are required for the latter 2 slabs. This information is mainly used in the *LCA* calculations in Chapter 7.

5.1.2 Description of the Original Design

In the original design, as shown in Figure 5.2 below, the building has outer dimensions 21.2 x 10.9 m, with a floor area of 231 m^2. The timber slabs span from façade to façade along the short side, with the support of an intermediate beam at 6.35 m. The slabs used are Kerto RIPA floor elements manufactured by MetsäWood, made of *LVL*. The edge beams are RHS sections. Onto these are attached, customised cold formed Z profiles, spanning 1.8 m between the columns to support the slabs. The section of the beam supporting the timber slab shown below in Figure 5.3, is an integrated system with total floor height 400 mm (structural members only). The intermediate beam uses *SHS* and spans 1.8 m between the columns. The wind loads from the façade are transferred onto the edge beams. The total height of the building is 12.52 m (4 storeys with approximately 3 m in height). The structure is designed for a service life of 50 years.

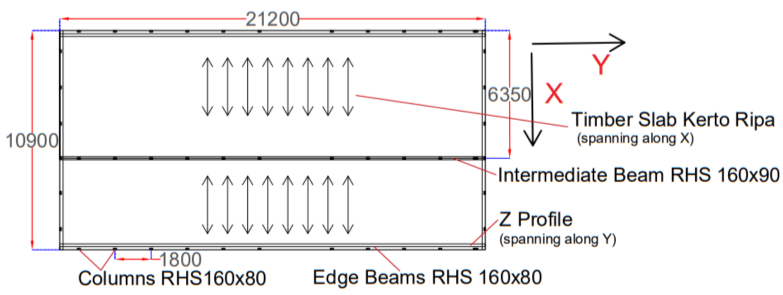


Figure 5.2: Original plan of Bouwdeel D.

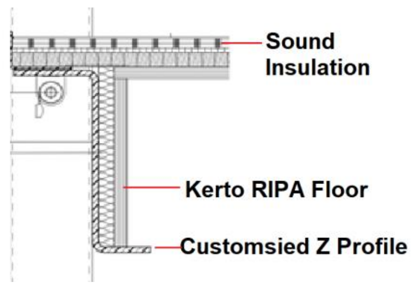


Figure 5.3: Customised Z Profile supporting Kerto RIPA floor elements.

All 4 design alternatives in this case study are made keeping in mind the goal to provide flexibility to the clients. Thus, as a constraint, the intermediate beam was removed. The slabs/beams span from façade to façade without intermediate columns. All columns are located on the edges. Finally, as mentioned earlier, the building is designed for an increased service life of 100 years.

5.1.3 Loads and Load Combinations

Dead Loads:

The total dead loads are calculated as the sum of slab dead loads (G_S), weight of installations (services and ceilings, $G_{C/S}$) and floor finish (G_{FF}). For timber slabs, the additional weight of fire protections and sound insulation ($G_{F/S} = 0.5\ kNm^{-2}$) must be considered. For *HCS* and *CS*, the cross sections used are with sufficient fire safe time without any additions.

$$\rightarrow G = G_S + G_{FF} + G_{C/S} + G_{F/S}$$

Wind Loads:

Wind Loads have been determined according to *EC1* for Wind Loads [59]. This is mainly to check the in-plane strength of the floor system, and to design the connections between the slabs and the cross beams. This gives rise to axial forces in the beams and slabs, which are checked in the design. Only the values for the topmost storey have been considered. The governing values have been taken as the most onerous one, among the 2 scenarios of wind action: 1) Wind perpendicular to the long side, and 2) Wind perpendicular to the short side of the building. Detailed calculations are given in Appendix C.1.1. The characteristic values of wind loads obtained, for a 100-year reference period is shown below in Figure 5.5. Wind loads are considered mainly to determine the number of shear connectors for required for the diaphragm action of the *STC* in design alternative ***DA1***.

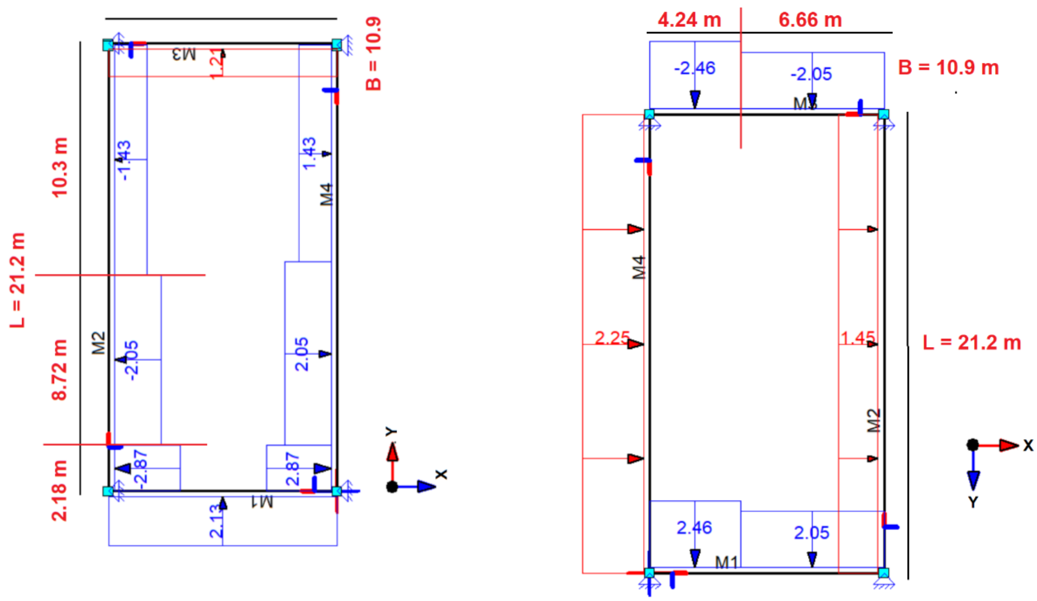


Figure 5.4: Wind Loads on Top Storey. (Right) Wind acting perpendicular to Long side. (Left) Wind acting perpendicular to Short side.

Imposed Loads:

The loads considered are Category B Office loads as per *EC1* for Live Loads [57].

Distributed load, $Q_{q,k,50} \; = \; 3.5 \; kNm^{-2} \; (Including \; 0.5 \; kNm^{-2} \; for \; Partitions)$

For a 100-year reference period,

$Q_{q,k,100} \; = \; 3.5 * \left(1 + \frac{(1-0.7)}{9} * ln \frac{100}{50}\right) \; = 3.6 \; kNm^{-2}$ according to Eq. 1

Roof Loads:

Roofs are considered to of Category I i.e., they are accessible for people, with the same live loads as on the floors, according to *EC1* for Live Loads [57].

Distributed load, $Q_{q,k,50} \; = \; 3.0 \; kNm^{-2}$

$Q_{q,k,100} \; = \; 3 * \left(1 + \frac{(1-0.7)}{9} * ln \frac{100}{50}\right) \; = 3.06 \; kNm^{-2}$ according to Eq. 1

Load Combinations:

The summary of the partial factors and combination factors for the loads considered above, is given below in Table 5.1.

Table 5.1: Partial and Combination Factors for Load Combinations (from [56]).

	ψ_0	ψ_1	ψ_2	ξ	γ_{ULS}	γ_{SLS}
Imposed Loads ($Q_{q/Q}$)	*0.7*	*0.5*	*0.3*	-	*1.5*	*1*
Wind Loads (Q_w)	*0.6*	*0.2*	*0*	-	*1.5*	*1*
Dead Loads (G)	-	-	-	*0.89*	*1.35*	*1*

According to *EC0* [56], the *SLS/ULS* design loads can be obtained as the minimum of the following combinations (for STR/GEO),

$$\begin{aligned} q_{Ed,SLS/ULS} &= min \quad of \quad \{\gamma_G * G + \gamma_Q * \psi_{q,0} Q_q + \gamma_Q * \psi_{w,0} * Q_w \} \\ &\quad and \quad \{ \xi * \gamma_G * G + \gamma_Q * Q_q + \gamma_Q * \psi_{w,0} * Q_w \} \end{aligned} \qquad \textbf{(Eq 3)}$$

For the roof loads, only imposed loads are considered. The wind loads are not considered because it is likely that during the event of a storm, people would be moved inside. In line with the goal of this case study, only the cross beams are checked against deflections in *SLS*. Hence wind loads are not considered for *SLS* i.e., the stability of the framed structure with respect to deflections are not checked.

5.2 Description of DA1_STC

5.2.1 Geometry

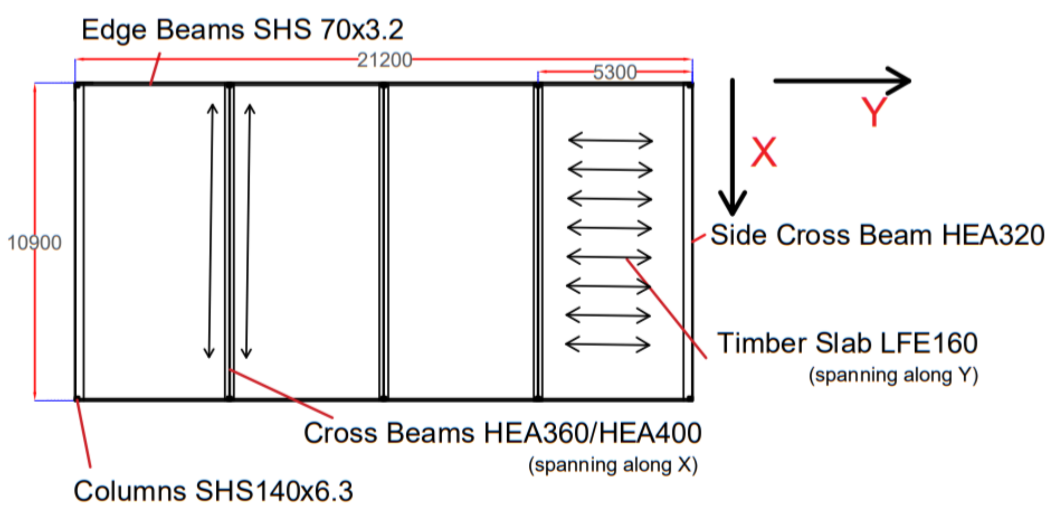


Figure 5.5: Floor Plan of DA1_STC.

DA1_STC is with the *STC* floor system chosen from the *MCA* in Chapter 4. It is designed for a service life of 100 years at the same location. The slabs are with *LFE* sections, spanning 5.3 m. The cross beams supporting the slabs are from façade to façade, spanning 10.9 m. These are supported by the columns, with a distance of 5.3 m *C/C*. The edge beams spanning along the side of the building are of length 5.3 m between the supports, with a total length of 21.2 m.

5.2.2 Structural Elements

Slabs:

The timber slabs are designed with ***LFE160*** sections, as simply supported over the cross beams. This is because using a continuous system would require additional moment resistance connections. Also, as the design is with composite action, it would pose a disadvantage to timber, as at the supports the section would be subject to double bending. At the façade ends also, they are simply supported. The span is 5.3 m, and the timber sections carry the load through unidirectional bending (about y-z plane). Other than carrying the imposed floor loads, they also contribute to the horizontal stiffness through diaphragm action i.e., they carry the axial loads induced by wind. The cross section is given below in Figure 5.6. Transverse stiffeners of thickness ***25 mm*** are placed at ***1050 mm C/C***. These helps transfer the axial loads in the x – direction. Detailed calculations are given in Appendix C.2.2.

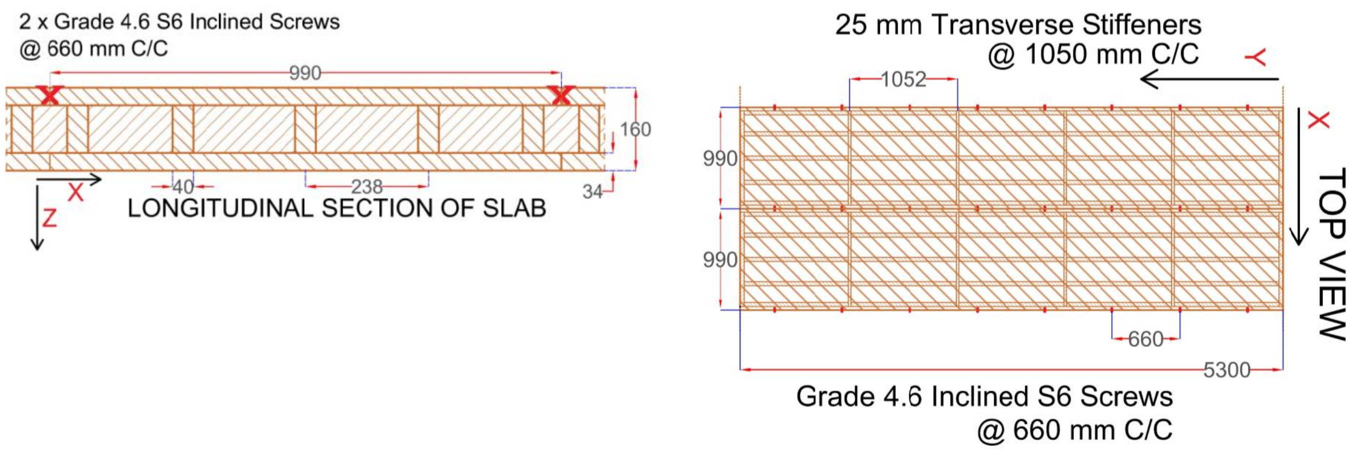


Figure 5.6: Dimensions of Timber Slab, LFE160.

Beams:

All the beams are designed to be simply supported. The edge beams are design with hot rolled square hollow sections ***SHS70x3.2.*** Their only role is to resist the wind loads transferred from the facades (axial loads), and to provide restraint against buckling for the columns. The side cross beams are of ***HEA320***. Apart from supporting the timber slabs and transferring the loads to the columns, they are also designed to withstand the axial forces due to wind load. The other cross beams are also designed for the same. However, since they are loaded more onerously, they use higher sections. We consider 2 design scenarios here: 1) Design with composite action, in which case ***HEA320*** is used. 2) Design without composite action, in which case the section used is ***HEA400***. The cross beams are checked for combinations of bending, and axial forces. It is assumed that these are restrained laterally by the timber slabs, such that they do not undergo lateral torsional deformations. The detailed checks for the edge beams, side cross beams and middle cross beams are given in Appendix C.2.

Columns:

The beams are simply supported onto the columns. All the out of plane loads on the floors, and the self-weights of the structural elements are transferred onto the columns. Again, based on whether the middle cross beams are designed for composite action or not, the load on the column varies slightly. However, ultimately this did not result in different sections being used. For both designs, with and without composite action, hot rolled square hollow sections, ***SHS140x6.3*** were used as the column. The detailed calculations for buckling of the columns are given in Appendix C.2.9.

5.2.3 Connections

Slab – Slab Connections:

Slab – slab connection is to transfer the lateral forces induced by the wind loads, due to the diaphragm action of the floor. This connection is with ***Grade 6.6 self – tapping inclined screws of diameter 6mm at 660 mm C/C***. These connections transfer the shear forces in the plane of the slab by their axial withdrawal capacity. The detail is given above in Figure 5.7 Since the timber slab is one – way spanning, these connections need not withstand the actions of the imposed floor loads. Detailed calculations are given in Appendix C.2.3.

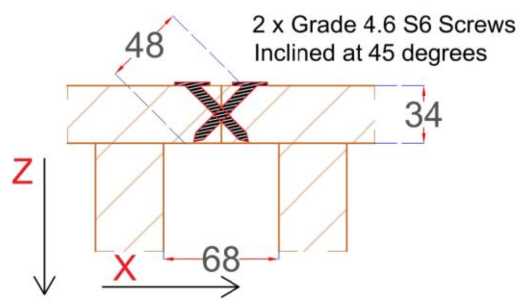


Figure 5.7: Slab - Slab connections for Timber.

Slab – Cross Beam Connections:

The connection between the slabs and the side cross beam uses ***1 x 19 Grade 4.6 M14 bolts at 566 mm C/C***. The main purpose of this is to transfer the in – plane loads generated by wind i.e., transferring axial forces onto the timber slab, and resisting shear forces due to wind on the long side. The resultant force acts at an angle to the timber grain direction. Hence the resistance of the bolt is calculated with respect to that angle. Detailed calculations are given in Appendix C.2.6. The connection detail is given below in Figure 5.8a.

The connection between the slabs and the other cross beams are different based on 2 scenarios:

1) With Composite Action: The middle cross beam is designed for composite action. In this case, the connection uses ***2 x 68 Grade 4.6 M14 bolts at 156 mm C/C***. The shear connectors are designed as flexible connectors to withstand the longitudinal shear force between the steel and timber section. Aspects of composite action are given in detail in Section 6.4. The loads due to wind are also acting on these connectors. The connection detail is given in Figure 5.8b.

2) Without Composite Action: In this case, the shear connectors only have to withstand the wind loads. Compared to the loads acting on the side cross beams, the magnitude is lesser. Hence, the connection used is ***2 x 7 Grade 4.6 M14 bolts at 1540 mm C/C***. The beam is connected to the slab on either side, which requires the same amount shear connectors. Thus, 2 bolts are required in each bolt row. The connection detail is given below in Figure 5.8c, and the detailed calculations are given in Appendix C.2.8.

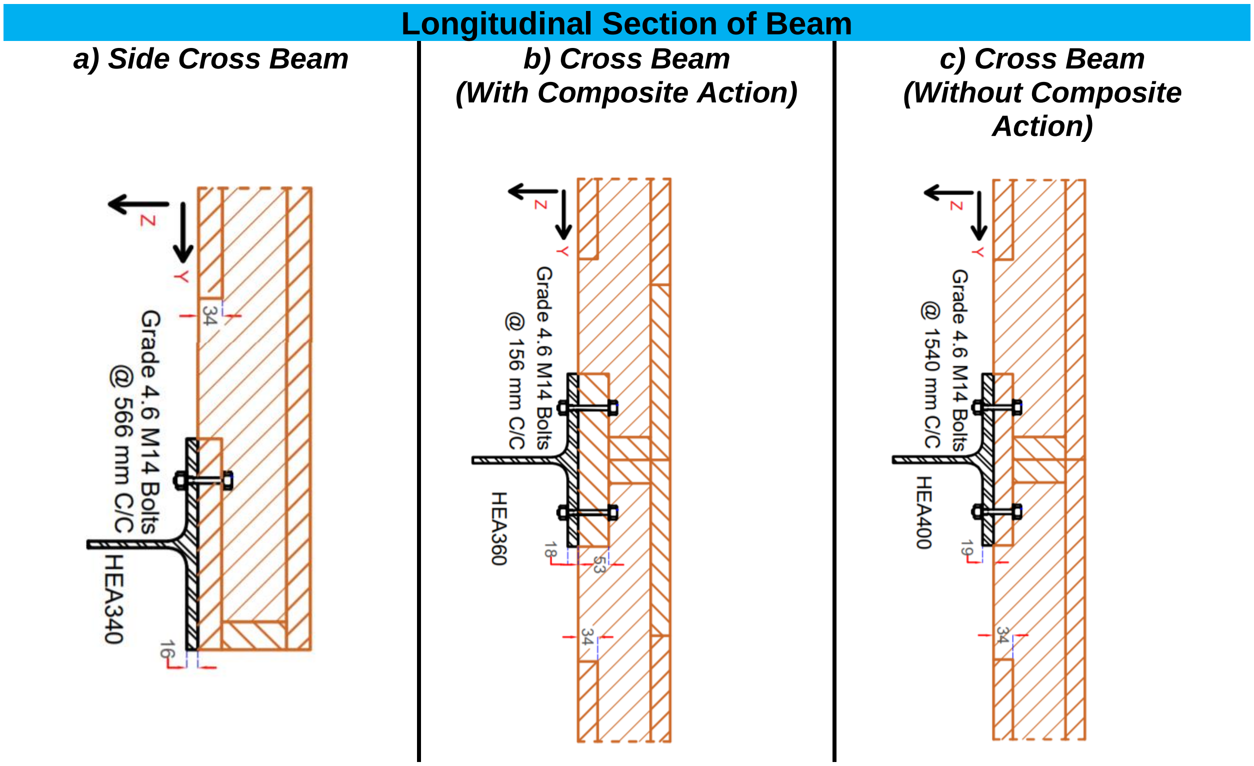


Figure 5.8: Slab - Cross Beam Connections.

To summarize, by designing the *STC* for composite action, the size of the cross beam can be reduced only by 1 size i.e., from *HEA400* to *HEA360*. This results in a decrease of floor height by 40 mm. The amount of savings in steel material due to the reduction of the size of the steel section is $1.8\ kg/m^2$, which is a very small amount. For achieving this, we need 2x68 shear connectors spread across the total span of the beam. Without composite action, 2x7 shear connectors (of the same type) are required, to transfer the wind loads (diaphragm action of the timber slab). Thus, it can be concluded that using an extra 122 Grade 4.6 M14 Bolts cannot be justified for obtaining the gains from composite action. This is also because shear connectors are an expensive component, with the added costs of labour. As mentioned earlier, the detailed calculations for composite action is given in Chapter 6. For fire safety and sound insulation, the layup shown in Table A.7 is used. This will be considered in the *LCA* in Chapter 7.

5.3 Description of DA2_HCS

5.3.1 Geometry

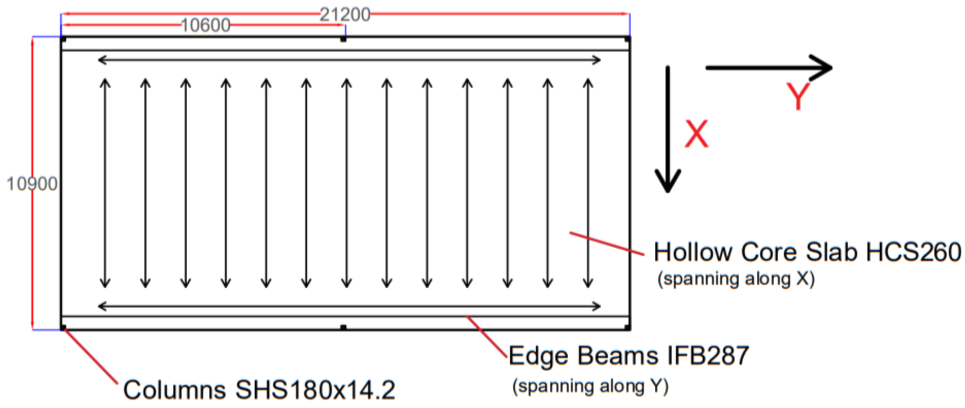


Figure 5.9: Floor Plan of HCS_2R50.

The structural scheme of ***DA2_HCS*** is shown in Figure 5.9. Like ***DA1_STC***, this design is also for a total service life of 100 years each, where all the structural elements can be reused as it. The slabs are spanning from façade to façade, with length of 10.9m. They are supported on integrated steel beams, with the slabs resting on the bottom flange. The span of the beam is 10.6 m. The wind loads are taken by the steel beams at the long side, and by the slabs along the shorter side. In this section, the design is limited to the dimensioning of the structural members. The design of connections is not dealt with here.

5.4.2 Structural Elements

Slab:

The slabs used are ***HCS260*** with a depth of 260 mm, produced by Consolis VBI, one of the leading producers of *HCS* in the Netherlands. They are prestressed precast members, thus allowing large spans [98]. They are simply supported at the façade ends and transfer the loads by unidirectional bending along the x – direction. The use of holes in the concrete section saves a large amount of concrete (weight), but with a slight increase in section height. The section of ***HCS260*** is given below in Figure 5.10.

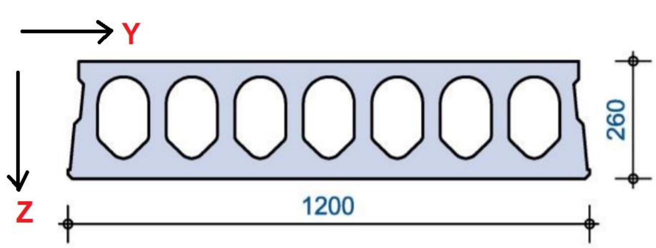


Figure 5.10: HCS260 Cross Section, adapted from [98].

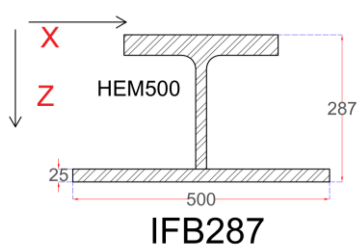


Figure 5.11: IFB287 Cross Section.

Beams:

All the beams are designed to be simply supported. The edge beams are design with integrated beams ***IFB287***. This is basically half of a *HEM500* section with a plate 500x25 mm welded at the bottom. This is shown above in Figure 5.11. The slabs are resting on the bottom flange, which means that the bottom flange will have sufficient thickness to prevent local damages. The beams are spanning along the y – direction with span 10.6 m, and also aid in transferring the axial forces due to the wind loads. Detailed calculations are given in Appendix C.4.3.

Columns:

The beams are simply supported onto the columns, placed at 10.6 m *C/C*. All the out of plane loads on the floors (office live loads and dead loads of the floors), and the self-weights of the structural elements are transferred onto the columns. The column section used is hot rolled ***SHS180x14.2***. The detailed calculations for buckling of the columns are given in Appendix C.4.4.

5.4 Description of DA3_CS

5.5.1 Geometry

The structural scheme of ***DA3_CS*** with steel – concrete composite slabs is shown below in Figure 5.12. This design is also for a service life of 100 years, where all the structural elements can be reused. The slabs are supported by cross beams spanning from façade to façade, with length of 10.9 m. These in turn are supported by the edge beams, the span of which is 10.6 m. The calculations for this case study are kept to just the structural elements and is given in Appendix C.5.

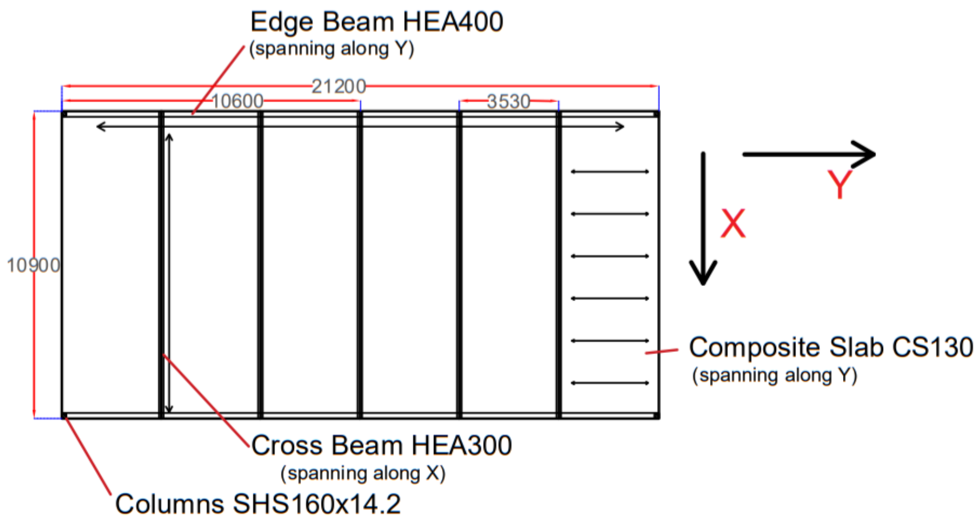


Figure 5.12: Floor Plan of DA3_CS.

5.5.2 Structural Elements

Slab:

The slabs used are *ComFlor60* [99], with a total depth of 130 mm. These are steel – concrete composite slabs with cold formed steel sheets of depth 60 mm at the bottom, over which 120 mm thick layer of concrete is cast into (hereafter referred to as ***CS130***). They are available as units of double spans, each of length 3.53 m. For the first use, the concrete is cast on – site, whereas these can be used as completely prefabricated units. The complete floor is made of 6 such units of composite slabs. To maintain the demountability of the slab elements, bolted connections are used (instead of welded shear studs) for achieving composite action with the supporting steel beams. The loads on the slabs are transferred by bending along the x – axis. Figure 5.13 below shows the cross section of ***CS130***.

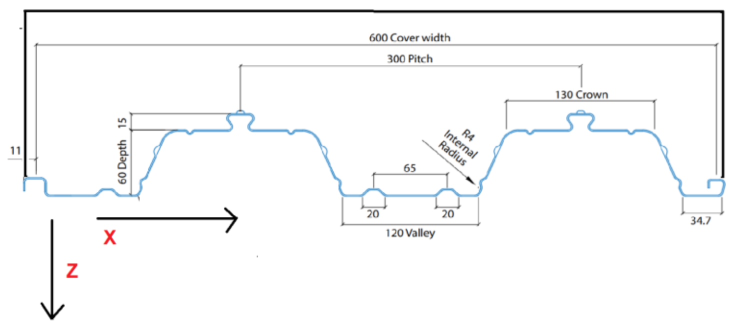


Figure 5.13: CS130 Cross Section, adapted from [99].

Beams:

All the beams are designed to be simply supported. The cross beams are with *HEA300*, spanning 10.9 m along the x – direction, from façade to facade. Composite action is utilized, according to *EC4* [64], while designing this beam. These are supported at either façade ends by the edge beams. These are with *HEA400.* The cross – beams are supported on the edge beams by connections at the web. The axial forces due to wind are taken by the edge beams and the side cross beams. Since the edge beams are laterally constrained by the cross beams, buckling failure is not checked. These are supported by columns at the middle i.e., spanning 10.6 m. Detailed calculations are given in Appendix C.5.3 and Appendix C.5.4 respectively for the cross beams and edge beams.

Columns:

The edge beams are simply supported onto the columns. All of the floor loads and the weight of all the structural elements are transferred onto the columns. The column section used is hot rolled ***SHS160x14.2***. The detailed calculations for buckling of the columns are given in Appendix C.5.5.

5.5 Summary

Design of all structural members were done for a service life of 100 years, with the conservative approach of elastic analysis. Combined with the use of demountable connections, it is expected that all structural elements can be reused, beyond its initial service life. Although the specific guidelines for reusing timber could not be used, it is expected that the timber elements also can be reused at the end of its initial service life, as was the case observed in literature on reused structural timber elements [129].

As a consequence of the lightweight nature of timber, ***DA1_STC*** was much lighter than its conventional counterparts, with ***DA3_CS*** and ***DA2_HCS*** being up to 33.5% and 79.8% heavier respectively. This translates into large savings on the cost of foundations. Also, *STCs* are easier to handle on-site overall, which is also a consequence of its lightweight nature. ***DA3_CS*** is advantageous when it comes to ease of handling. *ComFlor60* acts as propping, on to which the in-situ concrete is poured. However, there is lesser degree of prefabrication (and thus more execution time). Moreover, after the first use, these are detachable as a complete unit with concrete, which could prove quite difficult to handle. ***DA2_HCS*** with the highest weight for the slabs, is the least advantageous, for transporting, assembling, reassembling, and also considering the cost of foundations.

Calculations on composite action in *STC* sections showed that the savings in the size of the supporting cross beams was only 1 size i.e. reduction of amount of steel required by $1.8\ kg/m^2$. That too, after considering reoriented timber flanges at the top and bottom, owing to its orthotropic nature. Even though only a minor change, this would lead to extra costs for production and labour. On top of all this, it comes at the added costs for shear connectors. Thus, the use of composite action in steel-timber cannot be justified in this case. The optimum combinations of steel beams and timber slabs in *STC* floors is looked into further in Section 6.5, to see the applicability of composite action in a wider sense. Figure 5.14 below shows the comparison of *STCs* with/without composite action. In comparison, ***DA3_CS***, which uses a similar design with the same span of cross beams, also gives savings of only 2 steel section sizes (*HEA300* instead of *HEA340*, with reduction in amount of steel used by $5.44\ kg/m^2$). However, as we will see in Section 6.4, composite action in steel – concrete can lead to other advantages, both directly in terms of increase in mechanical properties, and indirectly in terms of the number of shear connectors used. Conversely, the benefits shown here for composite action in steel – timber represent the best-case scenario for this system.

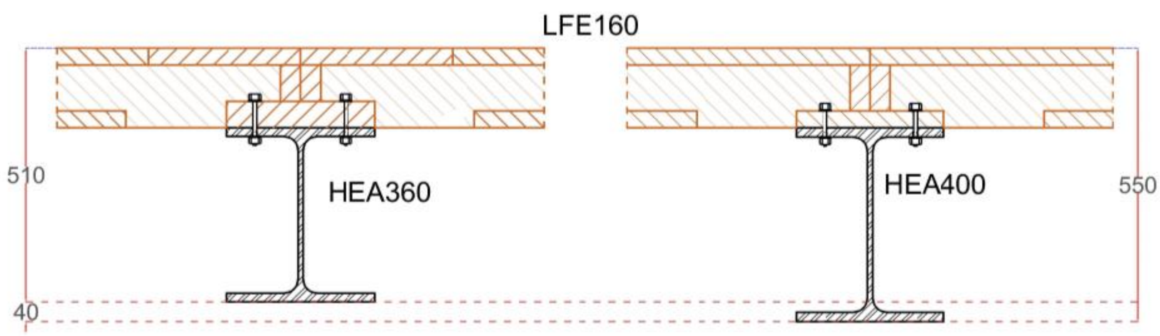


Figure 5.14: Comparison of STC with/without Composite Action.

It should also be noted that *HCS* and Composite slabs satisfy the Dutch building decree requirements [1] of sound insulation. For *STCs,* additional layer of insulating materials are required to meet these requirements. Fire protection for the latter is achieved by the encapsulation strategy. For *HCS* and composite slabs, these are achieved using larger sections. Finally, owing to the large spans possible in *HCS,* these were the most slender floor system. This was also due to the fact that *HCS* could be combined with an integrated steel beam. ***DA1_STC*** and ***DA3_CS*** formed non – integrated floor systems, with the slabs resting on top of the top flange of the steel beam. The result was that the floor heights were 77.7% and 81.1% for ***DA1_STC*** and ***DA3_CS*** respectively, with respect to ***DA2_HCS***. Larger spans meant that larger column grids could be used for *HCS* (10.9 m x 10.6 m). For *STCs,* the column grids were smaller owing to the lesser spans possible using *LFE* slabs (5.3 m x 10.9 m). For ***DA3_CS*** the same column grid as for ***DA2_HCS*** was used, but this required the use of additional steel beams (cross beams supported on the edge beams, instead of directly supporting on the columns).

The comparison of all the floor systems considered is summarized below in Table 5.2. The amount of materials calculated in this section are used as input for the Life Cycle Analysis (*LCA*) in Chapter 7. The increase % of different parameters indicated in Table 5.2 is with respect to the minimum values (marked as the benchmark) for each parameter. Here, the results of the *LCA* is also presented, based on the analysis of Global Warming Potential (*GWP*), including the effects of carbon storage. This is explained in detail in Chapter 7.

Table 5.2: Summary of Results of Case Study.

Design Alternative 1: Steel Timber Floor System (With Composite Action)				
Component	***Remark***	***Weight*** $[kg/m]$	$[kg/m^2]$	***Percentage of Total Weight***
Slab	*Span 5.3 m, Simply Supported. Additional layup for Fire protection and Sound insulation. Depth 160 mm.*	**-**	***278.9***	***90.3***
Lignature Box Elements, LFE160		-	*37*	*12*
Fire Protection		-	*27.2*	*8.8*
Sound Insulation		-	*112.7*	*36.5*
Floor Finish		-	*51*	*16.5*
Services and Installations		-	*51*	*16.5*
Beams	*All beams Simply Supported. Cross Beams spanning 10.9, façade to façade. Edge beams spanning 5.3 m.*	**-**	***26.3***	***8.5***
Side Cross Beams: HEA320		*97.6*	*9.2*	*3*
Main Cross Beams: HEA360 ***(With composite action)***		*124.8*	*15.9*	*5.2*
Edge Beams: SHS 70x3.2		*6.6*	*1.2*	*0.4*
Column: *SHS 140x6.3*	*Simply Supported, Height = 3.1 m.*	*26.1*	***3.5***	***1.2***
Total Weight ***(Benchmark)***			$\mathbf{308.7\ kg/m^2}\ (-)$	
Total Floor Height *(Compared to HCS Design)*			$\mathbf{510\ mm}\ (+78\%)$	
Total Environmental Impact ***(Benchmark)***			$\mathbf{-26.1\ CO_2\ eq/m^2}\ (-)$	
Design Alternative 1: Steel Timber Floor System (Without Composite Action)				
Component	***Remark***	***Weight*** $[kg/m]$	$[kg/m^2]$	***Percentage of Total Weight***
Slab	*Span 5.3 m, Simply Supported. Additional layup for Fire protection and Sound insulation. Depth 160 mm.*	**-**	***278.9***	***89.8***
Lignature Box Elements, LFE160		-	*37*	*11.9*
Fire Protection		-	*27.2*	*8.8*
Sound Insulation		-	*112.7*	*36.3*
Floor Finish		-	*51*	*16.4*
Services and Installations		-	*51*	*16.4*
Beams	*All beams Simply Supported. Cross Beams spanning 10.9,*	**-**	***28.1***	***9***
Side Cross Beams: HEA320		*97.6*	*9.2*	*2.9*
Main Cross Beams: HEA400 ***(no composite action)***		*124.8*	*17.7*	*5.7*

Edge Beams: SHS 70x3.2	*façade to façade. Edge beams spanning 5.3 m.*	*6.6*	*1.2*	*0.4*
Column: *SHS 140x6.3*	*Simply Supported, Height = 3.1 m*	*26.1*	***3.5***	***1.2***
Total Weight *(Compared to STC Design with Composite Action)* **Total Floor Height** *(Compared to HCS Design)* **Total Environmental Impact** *(Compared to STC Design with Composite Action)*		$\mathbf{310.5\ kg/m^2}$ $(+0.6\%)$ $\mathbf{550\ mm}$ $(+92\%)$ $\mathbf{-24.1\ CO_2\ eq/m^2}$ $(+8\%)$		
Design Alternative 2: Hollow Core Slab Floor System				
Component	***Remark***	***Weight*** $[kg/m]$	$[kg/m^2]$	***Percentage of Total Weight***
Slab	*Span 10.9 m, façade to façade. Simply Supported. No additions for Fire protection and Sound Insulation. Depth 260 mm.*	***-***	***505.7***	***91.1***
Hollow Core Slabs, HCS260 *Floor Finish* *Services and Installations*		*-* *-* *-*	*383.3* *71.4* *51*	*69.1* *12.9* *9.1*
Beams	*Integrated edge beams supporting HCS260, spanning 10.6 m, along facades. Simply Supported.*	***-***	***43.6***	***7.9***
Edge Beams: IFB287 (1/2 x HEM500 + Bottom Plate 50x25)		*237.5*	*43.6*	*7.9*
Column: *SHS 180x14.2*	*Simply Supported, Height = 3.1 m*	*72.2*	***5.8***	***1***
Total Weight *(Compared to STC Design with Composite Action)* **Total Floor Height** *(Benchmark)* **Total Environmental Impact** *(Compared to STC Design with Composite Action)*		$\mathbf{555.1\ kg/m^2}$ $(+80\%)$ $\mathbf{287\ mm}$ $(-)$ $\mathbf{102.2\ CO_2\ eq/m^2}$ $(+491\%)$		
Design Alternative 3: Composite Slab Floor System				
Component	***Remark***	***Weight*** $[kg/m]$	$[kg/m^2]$	***Percentage of Total Weight***
Slab	*Continuously Spanning 3.53 m. Total depth 130 mm. No additions for Fire protection and Sound Insulation.*	***-***	***354.9***	***86.2***
Composite Slab, ComFlor60, thickness 1 mm *Concrete, depth 70 mm* *Rebars (0.2%)* *Floor Finish* *Services and Installations*		*-* *-* *-*	*11.2* *240.6* *1.1* *51* *51*	*2.7* *58.4* *0.3* *12.4* *12.4*
Beams	*All beams Simply Supported. Cross beams spanning 10.9 m, façade to façade. Supported on edge beams. Edge beams spanning 10.6 m along facades.*	***-***	***52.1***	***12.6***
Cross Beams: HEA300 (With Composite Action) *Edge Beams: HEA400*		*88.3* *124.8*	*29.2* *22.9*	*7.1* *5.6*
Column: *SHS 180x14.2*	*Simply Supported, Height = 3.1 m*	*63.3*	***5.1***	***1.2***
Total Weight *(Compared to STC Design with Composite Action)* **Total Floor Height** *(Compared to HCS Design)* **Total Environmental Impact** *(Compared to STC Design with Composite Action)*		$\mathbf{412.1\ kg/m^2}$ $(+33\%)$ $\mathbf{520\ mm}$ $(+81\%)$ $\mathbf{92.3\ CO_2\ eq/m^2}$ $(+453\%)$		

6. Structural Analysis of STC Beams

The starting point for analysis is the design checks on the timber slabs and this is given in Section 6.1. Based on these sections, we move on to design the *STC* Beam. In Section 6.2, the validation of partial shear interaction for *STC* sections are done, based on experimental literature for the same. The validation is done by using by comparing the methods used for shear interaction in timber – concrete structures and steel – concrete structures. The main input for this is the stiffness of the shear connectors used. In Section 3.3, it was concluded to use bolts as the longitudinal shear connectors for providing composite action between steel and timber. Structural analysis of these shear connectors is one of the most important aspects of the design of a composite beam, and this is done in Section 6.3. The design of the *STC* Beam for the case study is done in Section 6.4. The first step for this is to consider the case of full shear interaction, to assess the total savings that can be made. Finally, in Section 6.5, design recommendations for the application of the *STC* floor system are made, by producing the span tables for typical layouts of Dutch offices. The results are summarised in Section 6.6. For detailed calculations, the reader is referred to Appendix D.

6.1 Verification of Timber Sections

The timber slabs are checked for various criteria for obtaining their span tables. Since the degraded properties of reused timber cannot be predicted during the initial design phase, the verifications are done only for reuse by reorientation. *LFE* sections use very slender webs and flanges. Figure 6.1 below shows the cross section. Table 6.1 shows the range of values that can be used for the individual components of the glued composite section (obtained from Table A.6). Many aspects must be considered to design the members. The checks that have been carried out are mentioned below. The detailed calculations and unity checks are given in Appendix D.1.

Table 6.1: Dimensions of LFE.

LFE Dimensions	
Top Flange Thickness ($t_{T,tf}$)	*25-82 mm*
Bottom Flange Thickness ($t_{T,bf}$)	*25-82 mm*
Web Thickness ($t_{T,w}$)	*27-80 mm*
Total Height (H_T)	*120-360 mm*
Spacing of Webs ($b_{T,f}$)	*Upto 250 mm c.t.c.*
Width (b_T)	*< 1000 mm*
Length of Slab (L_S)	*< 18000 mm*
Transverse Stiffener Spacing ($S_{T,S}$)	*< 1200 mm C/C*

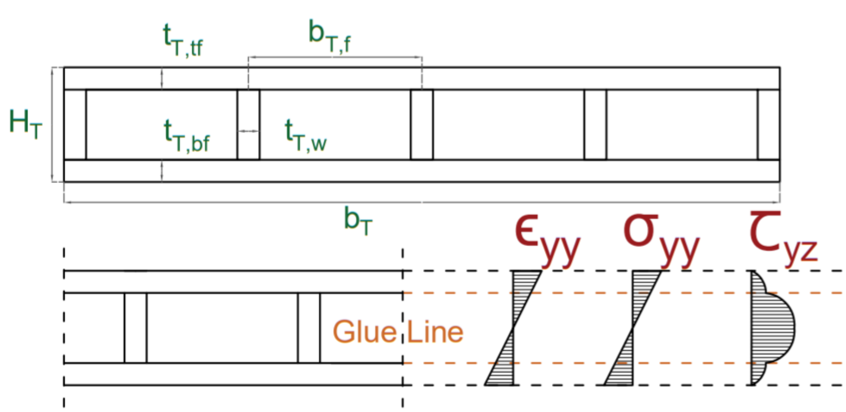


Figure 6.1: Cross Section of LFE.

6.1.1 Effective Flange Width

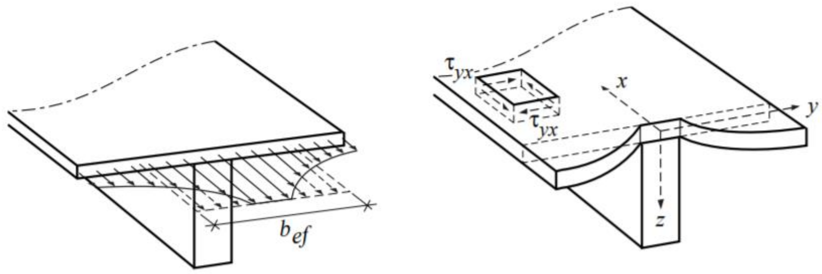


Figure 6.2: Stress Distribution in the Flange [3].

Shear deformations in the plane of the flanges imply that the normal stresses in the flanges are non-uniformly distributed [3]. Möhler et al. [33] derived the effective width of a fictitious flange considering its shear deformations, to obtain the equivalent uniformly distributed normal stress. The effective flange width ($b_{T,f,eff}$) thus obtained would depend on the ratio of spacing between webs and the span of the slab $\left(\frac{b_{T,f}}{L_{Slab}}\right)$, and the ratio of *MOE* in the direction perpendicular to grain direction of timber and the Shear modulus $\left(\frac{E_x}{G}\right)$ of the flanges. With increasing value of both these ratios, $b_{T,f,eff}$ decreases. From calculations, it was observed that this formula was beyond the scope of *LFE* sections. The value of E_x was very small, which would lead to complex solutions for $b_{T,f,eff}$ based on this formula. Physically, this implies that the shear deformations are negligible for this cross section. Thus, it was opted to take the value of $b_{T,f,eff}$ as the maximum permissible value (as per *EC5* [52]), considering shear deformation and the buckling failure of the compression flange. The values adopted were that of plywood with outer fibre direction parallel to that of the webs from Table D6-1 of *EC5*. The formula of Möhler and the detailed calculations are given in Appendix D.1.1.

6.1.2 Stresses in the Flange

The maximum bending stresses occur at the extreme fibres in the top and bottom flanges and are checked against the design resistance to bending. Apart from this, the flange in compression is also susceptible to buckling around the minor axis. Hence *ULS* checks are done for the same. The formulas and calculations are given in Appendix D.1.2.

6.1.3 Shear Stresses in the Web

Slender webs are susceptible to buckling due to shear force acting on it. Hence, they are checked against the design shear strength in the plane parallel to the direction of timber grains, in *ULS*. Checks for the maximum shear stresses (that occur at the neutral axis of the cross section) are also done (also in *ULS*). The formulas and calculations are given in Appendix D.1.3.

6.1.4 Shear Stresses in Glue Line between Flange and Web

The strength of correctly glued joints between the flange and web normally exceeds that of the flange/web materials. Hence, the weakest element in such a joint may be the rolling shear strength of the flange/web in the plane perpendicular to the direction of timber grains ($f_{v,90,d}$) [3]. This is checked against the total longitudinal shear flow in the section, in *ULS*. The formulas and calculations are given in Appendix D.1.4.

6.1.5 Deflection and Vibrations

In *SLS*, checks are done for the initial and final deflections of the floor element. These are checked against the limits provided in *EC0* [[55]]. *EC5* [52] also mandates checks for the vibration performance of the floor elements, in terms of the first fundamental frequency and stiffness. Hence, this has been carried out. The formulas and calculations are given in Appendix D.1.5.

6.1.6 Results

By checking all the parameters given in Section 6.1.1 to 6.1.5, the span tables for *LFE* have been obtained, designed for office use. As mentioned in Section 2.3, since the mechanical properties of timber cannot be predicted during the initial design phase, only reuse by reorientation is considered, by increasing the value of live loads. The properties of the *LFE* section for different spans are given below in Table 6.2.

Table 6.2: Span Tables for LFE for Reuse by reorientation.

Section	H [m]	Reorientation		
		$t_{T,w}$ [mm]	$t_{T,f}$ [mm]	Weight [kg/m^2]
LFE120	*4	42	31	31.16
LFE140	4.6	39	31	32.43
LFE160	5.1	37	31	33.66
LFE180	5.7	35	31	34.72
LFE200	6.2	34	31	35.9
LFE220	6.7	32	31	36.66
LFE240	7.2	31	31	37.63
LFE260	7.7	31	31	38.93
LFE280	8.1	31	31	40.24
LFE300	8.6	31	31	41.54
LFE320	9.1	31	31	42.84
LFE340	9.5	31	31	44.14
LFE360	9.9	31	31	45.44

It can be observed that the *LFE* sections may not be the most slender (compared to *HCS* and Composite slabs), but they offer a very light-weight solution, while providing sufficient spans. Though it may not be as good as *HCS* which offers spans up to 16 m, it seems to be a much better solution compared to Composite Slabs, for which the optimum span without propping is only around 3.6 m.

6.2 Composite Action in Steel – Timber

Steel is a ductile material, which offers high performance of strength and stiffness. Timber is brittle in tension and ductile in compression. Consequently, analysis of timber is limited to the elastic limit when subjected to tension, but plasticity can be attained by timber in compression. According to the recommendations of the Steel Construction Institute (SCI) [94] for reusing structural steel, as long as there are no plastic deformations on the structure, steel can be reused without any reductions for strength and stiffness. Thus, it is preferable to have the *ULS* loads within its elastic limit, although this is not mandatory. Based on reliability indices for limit state loads, the probability of having plastic deformations can be limited to less than 5% in this manner. As in the case for steel, it is preferable to limit the analysis of timber, and thus the *STC* section, also to its elastic limit, for the purpose of reusing.

The governing criterion for beams is mostly its deflections, and by default it is calculated within elastic limit. Thus, the bending stiffness of beams is of utmost importance, and the primary role of enabling composite action between timber slabs and steel beams is to increase this. Using elastic analysis overall would also require that other checks such as for shear and bending moments be done within the elastic limit, in contrast to the conventional practise of assuming plastic strains/stresses to calculate the ultimate load carrying capacity for the sections for steel and also for steel – concrete composite structures.

Composite action arises from the shear interaction between the different components used. When there is no shear interaction between the steel and timber, both the components undergo bending action separately, while keeping the same curvature (to maintain compatibility). As shown in Figure 6.3 (Left), it results in slip at the interface of the 2 elements, as there is no mechanism to resist shear force. This is what happens in a conventional floor system, especially those with prefabricated elements. The shear connections provided are minimal, usually to transfer forces in the plane of the floor (diaphragm action). This produces a very low degree of shear interaction, and the contribution of the slabs to aid the beams in load bearing is not considered in design. When there is full shear interaction, the 2 constituent elements behave as one, and undergoes bending. The bending stiffness is increased due to composite action, and there is no slip at the interface, as shown in Figure 6.3 (Right).

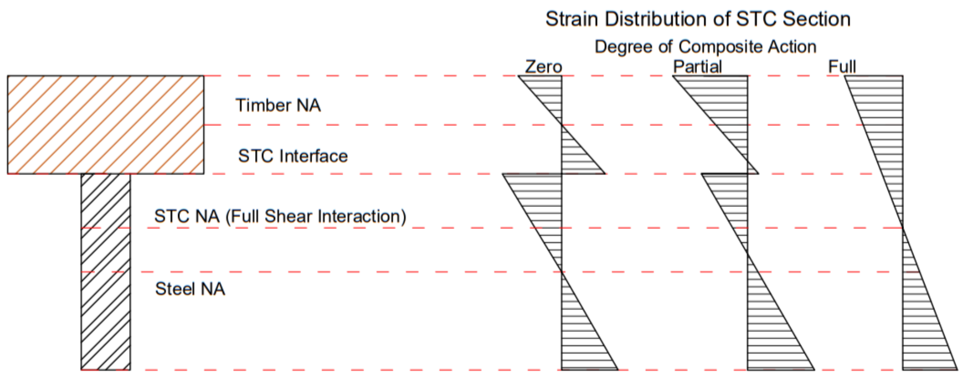


Figure 6.3: Distribution of Normal Strains in STC Section for varying degrees of Composite Action. (Left) No Composite Action. (Middle) Partial Composite Action. (Right) Full Composite Action.

However, in all realistic scenarios, the actual behaviour of the composite beam lies in between the above-mentioned ideal scenarios (Figure 6.3, center). This is because, using discrete shear connections such as bolts would provide semi-rigid joints i.e., they give resistance to the shear forces generated while undergoing some degree of deformations. This has been observed in the experimental work of Hassanieh [24]. In Hassanieh's research, 4-point bending tests were conducted on steel timber composite beams with different types of shear connectors. It was found that using a continuous shear connection such as glued connection could result in complete composite action, with no slip at the interface – but this is not suitable for demountable construction. On the specimens connected with screws, partial shear interaction was observed, which resulted in small amounts of slip at the interface. The strains have been recorded at the yield loads. Figure 6.4 below shows the strain distribution of the *STC* section along its depth at the yield load and ultimate loads.

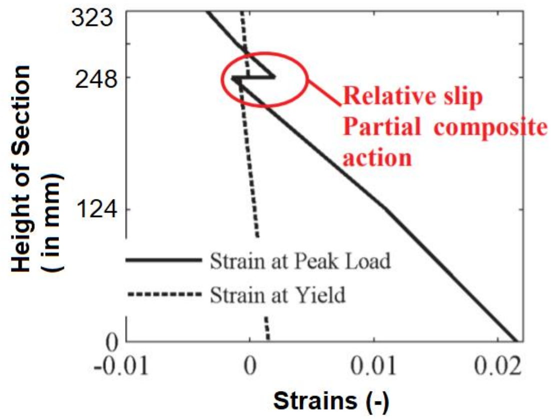


Figure 6.4: Strain Distribution of STC Section from Hassanieh's Experiments, adapted from [24]. The results shown here are for Steel - LVL specimens with Coach Screws as shear connectors (Specimen #4).

Based on the above results for strain distribution (for Hassanieh's Steel-LVL Specimen #4), and other such specimens with discrete shear connectors, it is suggested to use the Gamma Method for analysing partial shear interaction in *STC*. The Gamma Method is an elastic method of analysis commonly used for built-up timber sections, which takes into account the combined effect of Euler-Bernoulli bending and the slip at the interface due to the flexible nature of the shear connectors [3]. The normal forces developed in the individual components are transferred between each other with the help of shear connectors. During this process, as the shear connectors transfer the loads, they deform, thus generating slip at the interface. Using this method, the effective bending stiffness EI_{eff} and the distribution of strains/stresses over the *STC* section can be determined. The formulas for the same are given in Appendix D.2.1. L. van Glabbeek [78] used this method to analyse timber-concrete composite structures, which showed good accuracy. The assumptions used in this method are given below:

- Both constituent elements being combined are linear elastic in tension and compression.
- The load slip behaviour of the shear connection is linear elastic.
- Discrete shear connectors are modelled as continuous with equivalent smeared stiffness K_{sc}.
- Curvature κ of constituent elements is the same, and equal to that of the composite beam. It is obtained as $\kappa = \frac{M}{EI_{eff}}$, where EI_{eff} is the effective bending stiffness.
- Frictional forces, uplift effects and shear deformation are neglected.
- The steel beam is not deformed when establishing the shear connection i.e., either the *STC* beam is propped, or the steel beam is precambered to take all the dead loads.

Thus, steel and timber can be analysed for partial shear interaction, as both materials show linear-elastic behaviour. Calculations using the Gamma Method have been done on the *STC* section mentioned above. The summary of properties of the specimen considered is given below in Table 6.3.

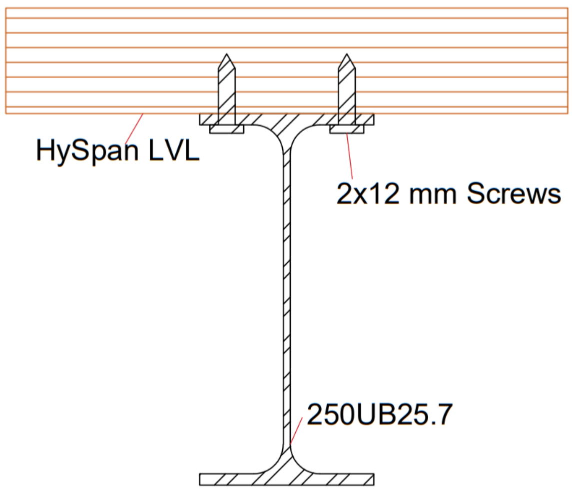


Figure 6.5: Steel - LVL composite from Hassanieh's experiments. Specimen #4, from [24].

Table 6.3: Properties of Steel - LVL STC beam used in Hassanieh's experiments. Specimen #4, from [24].

Steel Beam: **250UB25.7** (Australian standard Hot rolled Beams)
$h_S = 248\,mm, b_S = 124\,mm$ $t_{S,w} = 5\,mm, t_{S,f} = 8\,mm$ Distance of NA to top and bottom fibre of Steel, $z_{S,t} = 124\,mm = z_{S,b}$ $E_S = 200000\,MPa, f_y = 320\,MPa$
Timber Slab: **HySpan LVL**
$h_T = 75\,mm, b_T = 400\,mm$ Distance of NA to top and bottom fibre of Timber: $z_{T,t} = 37.5\,mm = z_{T,b}$ $E_T = 13200\,MPa, f_b = 50\,MPa$
Shear Connectors: **Screws**
Diameter, $d_{sc} = 12\,mm$ *Spacing*, $s_{sc} = 250\,mm$

Based on the above properties of components and shear connectors, the Gamma Method has been used to predict EI_{eff} of the *STC* beam. Using this, the maximum moment carrying capacity (based on yielding od steel/timber), and thus the yield load of the specimen under 4-point bending can be determined. The strain distribution is computed for this theoretical yield load and compared with the experimentally obtained values (Figure 6.4), recorded at the experimental yield load of the specimen. This is shown below in Figure 6.6. The calculations for the same are given in Appendix D.2.2.

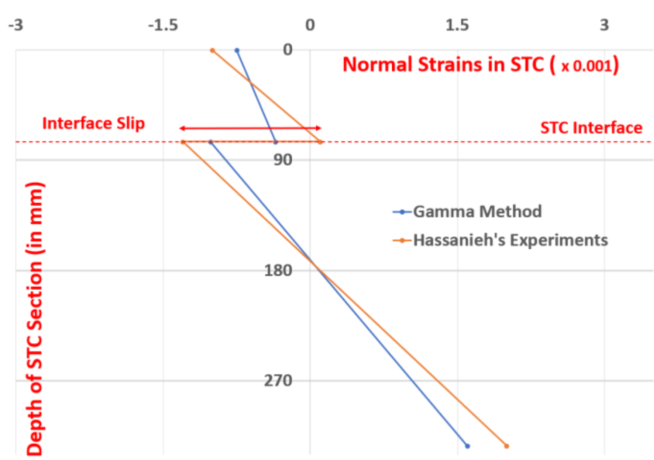


Figure 6.6: Distribution of Normal Strains over depth of STC. Comparison between Gamma Method and Experimental values.

From the Figure 6.6, it can be observed that the Gamma Method can predict the value of normal strains with good accuracy. The yield load predicted by the Gamma Method of 165 kN, using the mechanical properties of the section and the shear connectors, is also in agreement with the experimentally obtained value of 130 kN (+26.9%). The slip is estimated to be 1.3 mm using the Gamma Method, compared to the experimentally obtained value of 2.2 mm (-40.9%). Thus, it can be concluded that the Gamma Method can predict the behaviour of *STC* beams with sufficient accuracy. The above-mentioned values are for partial shear interaction. When there is full shear interaction, the *STC* behaves as a solid section, and the properties in this case can be calculated using Steiner's rule. Table 6.4 below compares the properties of *STC* with partial and full shear interaction with respect to the properties of the steel beam only.

Table 6.4: Summary of benefits of Composite Action. Hassanieh's Steel - LVL Specimen #4.

Parameter	Steel Section	STC Section	
		Partial Shear Interaction (Specimen #4)	Full Shear Interaction
Bending Stiffness $(\boldsymbol{EI})$ $[\boldsymbol{x10^{12}\ Nmm^2}]$	*10.9 [-]*	*15.64* ***[+43%]***	*18.54* ***[+70%]***
Elastic Bending Moment Resistance $(\boldsymbol{M_{el,Rd}})$ $[\boldsymbol{kNm}]$	*140.7 [-]*	*165.2* ***[+17%]***	*175.6* ***[+25%]***

Thus, it can be observed that considering composite action in *STC* sections can lead to significant gains in its mechanical properties (up to 70.1% increase in EI_{eff}, and up to 24.8% increase in $M_{el,Rd}$). However, a few aspects must be noted here. First of all, the spans under consideration in Hassanieh's experiments (and consequently the size of the supporting steel sections) are quite small. Typical spans in Dutch offices are of the order 6 – 12 meters compared to the 6m *STC* beams considered here. The contribution of timber sections to gains in properties due to composite action when combined with these larger steel sections is bound to much lower. Another aspect is the orthotropic nature of timber. The timber products considered in Hassanieh's experiments – *CLT* and *LVL* both offer greater timber strength and stiffness properties in the transverse direction. We are considering the transverse direction because that is what will be active in a typical slab – beam system for composite action. However, the *STC* floor system chosen in this thesis for further analysis is *LFE,* which offers much less advantageous transverse direction properties. The reason that *CLT* and *LVL* were not considered further was due to their huge toll in terms on environment impact. Ironically, this increased environmental costs came due to the use of large amounts of adhesives, which in turn resulted in less degree of orthotropic nature (i.e. stronger in the transverse direction). Calculations done in Section 6.4 will shed more light on both of these aspects, and a comparison of the benefits of composite action between *STC* and steel – concrete will help conclude whether the added costs of shear connectors can be justified.

Another aspect to be considered while looking into composite action in *STC* is the creep effects of timber. All the experiments conducted by Hassanieh are short-term, which means that the obtained results do not reflect on the creep effects in timber. The deflections in timber structures are calculated based on different creep factors for different types of loads, according to *EC5* [53], according to Eq. 26 and Eq. 28. Dead loads have a far greater effect due to creep compared to live loads which only act for a short duration at a time. Thus, precambering steel beams to take all the dead loads of the *STC* (timber slabs, installations, floor finish, fire protection, sound insulation, etc) will help in this respect. In other words, the composite beam only takes the imposed floor loads for deflection. This also helps to keep in line with the objective of rapid execution for reusability. By using precambering, we can utilize higher stiffness properties of timber (with reference to the *STC* in the Case Study: 8461 MPa instead of 6875 MPa).

Last but not least, we look into the applicability of plastic stress distribution in the *STC* section, to calculate the plastic bending moment resistance $M_{pl,Rd}$ of the section. Even though the structural analysis in this thesis is limited to the elastic limit, for increasing the reusability of the structural elements, $M_{pl,Rd}$ of the composite section is important to see the benefits of the *STC* section over that of the steel section alone, as steel is a ductile material. From Figure 6.4, it can be seen that at the ultimate load (well beyond the elastic limit), the strains in the section have gone well beyond their corresponding yield strains ($1.78x10^{-3}$ for S355 steel, $1.6 - 1.7x10^{-3}$ for timber). Thus, the assumptions of plastic stresses in the *STC* section can be justified up to a certain extent. From the experiments of Chybinksi et al [16] the theoretical value of $M_{pl,Rd}$ calculated using the latter assumption predicted the value of ultimate load with 92.5% accuracy. Thus, calculations for $M_{pl,Rd}$ of the *STC* section are done, to make a comparison with that of the steel beams.

It should be noted here that just like the Gamma Method, the Newmark Model [72] and Leskela Approach [77] also can be used to investigate partial shear interaction between 2 linear elastic materials. All the 3 methods are based on the same assumptions and conditions of compatibility. They vary in the method used for obtaining the final solutions. The Newmark Model is the most accurate method, as it can be used to model different boundary conditions. The other 2 methods are restricted to the case of simply supported beams with uniform loads. The main parameter that influences the degree of shear interaction is the connection stiffness, which will be addressed in Section 6.3.

6.3 Bolted Connections for STC

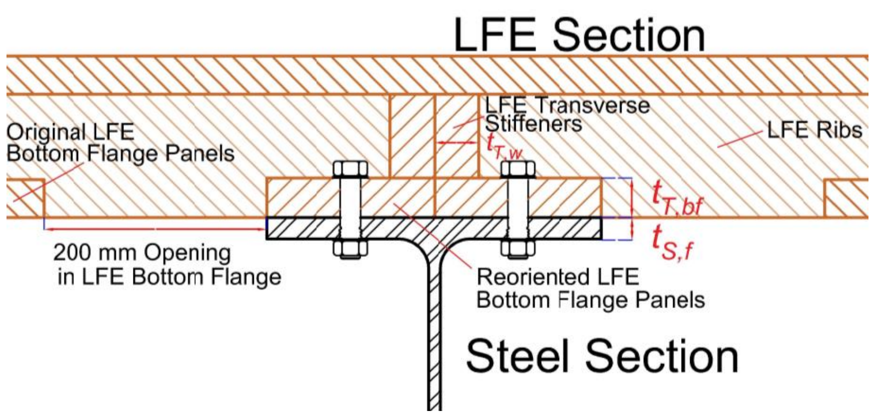


Figure 6.7: Steel Timber connection with Bolts.

As mentioned in Section 3.3, bolts are the preferred option for establishing demountable connections. Figure 6.7 above shows the longitudinal section of the beam, which is the transverse section of timber slab. From this, it can be observed that the bolted shear connection in the *STC* beam is established between the bottom flange of timber to the top flange of the steel section. *LFE* is a closed section. Hence an opening of at least 200 mm is required in the timber bottom flange near the supports to access the connections (for assembly/disassembly/reassembly).

Since the slabs are simply supported, it is opted to have the entire timber bottom flange removed from the edge of the steel flange, up to a distance of 200 mm. This will lead to reduction in materials and ease of prefabrication due to uniformity in section throughout the span of the slab. The bolts used are conventional hexagonal bolts with washer-nut assembly, according to the specifications of EN 15048 (and EN14399 for preloaded bolts). They are placed from the top, and tightened with the help of the nut, from the bottom.

With respect to the orthotropic nature of wood, it is the transverse section of timber that is utilized for composite action, with material properties of timber perpendicular to the grain direction. For increased efficiency due to composite action, the bottom flange of *LFE* near the supports must be reoriented, such that the timber parallel to grain direction is active in composite action. This can be seen in Figure 6.7. For manufacturing these sections, the timber bottom flange is removed from the support ends up to the end of the hole openings (up to 250 – 350 mm). The reoriented timber bottom flanges with width equal to half the width of the steel section is placed at the support ends. The webs that are active in composite action are the transverse stiffeners and are already oriented optimally. The thickness is the same as that of the ribs of *LFE* section used. It is preferred to leave the top flange of timber is left unchanged.

6.3.1 Resistance of Bolted Connections

Although it is better to restrict the analysis of *STC* to the elastic limit from the perspective of reusability, it is decided to design the connections for ductile failure modes. As we will see further ahead in this section, the ductile failure modes of connections are associated with the steel components and the brittle failure modes are associated with timber. As steel is much stronger than timber, we will obtain the optimum design with minimum material by ensuring this criterion. Also, by using ductile shear connectors, the assumption of plastic stress distribution can be used to design of steel – timber composite structures for their effective plastic bending moment resistance $M_{pl,Rd}$. According to *EC4* for steel-concrete composite structures [64], bolted shear connectors can be classified as ductile only if they have a deformation capacity of 6 mm, as obtained from push-out tests. From push-out tests on steel-timber bolted connections [15-17,24-25], it was observed that shear connectors used in *STC* beams fulfil this criterion. Also, the ultimate load carrying capacity obtained was in line with the predictions of Johannsen Yield models [3] for steel timber connections. The major distinction for different failure modes is whether the steel plate connected is thin or thick. For thick steel plates, steel is assumed to be a rigid vertical support, and prevents any rotation of the bolt. For this criterion, it is required that the steel plate is at least as thick as the diameter of the bolt. The different failure modes that can occur in the shear connectors are given below:

- **Bearing Stress of Timber:** As the name suggests, this failure mode occurs when the bearing stress in timber exceeds the design value. For thin plates, due to the rotation of the bolt in the steel flange, the effective area of bearing on timber is reduced. For thick steel flanges, due to constraints in rotation, the whole area of timber can be utilized in bearing, thus resulting in higher strength. Compared to steel, timber is brittle. Hence this failure mode can be considered as a brittle failure mode, associated with timber crushing.
- **Formation of 1 Plastic Hinge**: This is ductile failure mode, associated with the formation of a plastic hinge in the bolt region supported by timber. This failure mode can be observed when connect to thin and thick steel plates.
- **Formation of 2 Plastic Hinges:** This also is a ductile failure mode, and is associated with the formation of 2 plastic hinges in the bolt. The first one is in the region supported by

timber. The second one is in the steel flange. This failure mode occurs only when the steel plates are considered to be thick.

- **Bearing Stress of Resin:** In Section 6.3.2, it is opted to proceed with resin injected bolts, as a solution to obtain slip resistant connections. Hence, it is imperative to check that the bearing stress does not exceed the limits of the resin used.
- **Shear Failure of Bolt:** This failure mode is a brittle one because the dimension under consideration (*diameter of Bolt*) is small compared to the other dimensions of the bolted connection. From calculations, it was observed that, though not governing, this failure mode was close to the governing failure mode for lesser grade bolts (4.6,4.8,5.6).
- **Failure Modes of Steel Plate:** Bearing and punching failure of the steel plate, are not relevant to bolted connectors for steel-timber. This is because the strength of steel is significanlty higher than that of timber. Compared to the failure modes of Johannsen yield theory, the load carrying capacities of failure in the steel flange is significantly higher. Hence, these are not checked further.

The formulas for the different failure modes are given in Appendix D.3.1. It should be noted that the failure modes with the formation of plastic hinges is ductile in nature. Hence it is required to ensure that this is the governing failure mode, to justify plastic stress distribution in the *STC* beam in *ULS*. In other words, the bearing strength of timber should be greater than the strength of the mechanisms formed with either 1 or 2 plastic hinges. From this criterion, the minimum thickness of the bottom flange, can be determined for combination with different bolt sizes. This can be observed in Figure 6.8. From these calculations, it was also observed that the failure of the connection by bolt rupture or increased bearing stress in the resin is not governing in almost all scenarios.

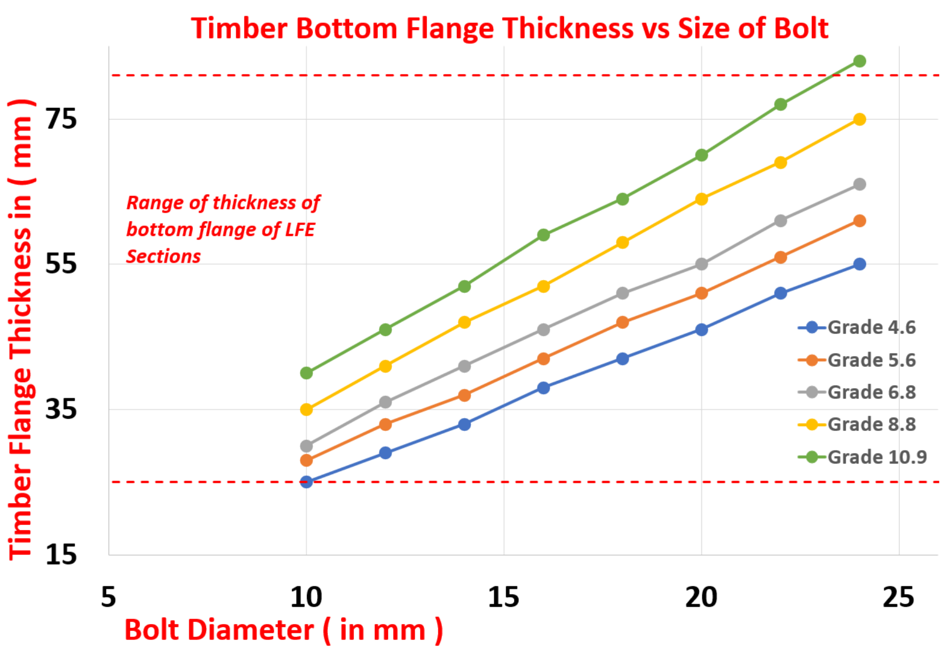


Figure 6.8: Thickness of timber bottom flange vs Bolt Diameter for different grades of Bolts.

As mentioned in Section 6.2, the stiffness of the shear connector is what governs the design of the *STC* beam. And this parameter is mostly influenced by the diameter of the bolt. The increase in bending stiffness of the *STC* beam is attributed mostly to the height of timber section (rather than the strength of timber). Thus, having thicker bottom flanges of timber is only adds the amount of material, and is not desirable. This gives rise to the need for using minimum thickness, while ensuring ductile connections. Figure 6.8 also shows a sensitivity study with the Grade of Bolts used. The data for this graph is given in Table D.4. It can be observed that it is most desirable to use lesser grade bolts (Grades 4.6,5.6 and 6.8) of higher diameter (M20 – M24).

From Eq. 45 and Eq. 46, we can see that the load carrying capacity of *STC* connections with thick steel plates is much greater than those with thin steel plates (1.5x). As explained earlier, the rotational constraint on the bolt imposed by thick steel plates ensures that all the timber thickness is available for bearing support. For thin plates, the effective area of timber support is reduced due to the rotation of the bolt at the steel plate end. Hence it is desirable to ensure the *STC* connections are with thick steel plate i.e., the diameter of the bolt should be less than that of the steel plate. Bolts require predrilling of holes, which is increasingly difficult with larger thickness of steel flange. It is decided to restrict the thickness of the steel flange to 25 mm, considering this aspect, and considering the aspect of the scope of the use of *STC* beams for offices.

As mentioned in Section 5.2.3, the connections used in the case study are Grade 4.6 M14 bolts, connected between the timber bottom flange (with thickness 34 mm, from Section 6.4) and the steel top flange (with thickness 17.5 mm). The reason for this choice is based on the fact that the same bolts are used to design the system for diaphragm action of the floor (low shear forces), and for composite action between the steel beam and timber slab.

6.3.2 Bolt Hole Clearances for Demountability

The degree of shear interaction is mostly determined by the stiffness and spacing of the shear connectors, according to Eq 37. Even when composite action is not required for design, longitudinal shear connectors are required to enable the diaphragm action of the floor. For a demountable floor system with *STC*, drilling of bolt holes both in the timber and steel section is a time-consuming process onsite. In line with the aspect of rapid execution, it is preferred to have the steel/timber elements with the holes predrilled in a factory before its execution on-site. This will lead to an efficient process of assembly, disassembly, and reassembly.

The nominal hole clearances for bolts specified by EN 1090 -2 [62] assures us that steel structures can be assembled with sufficient certainty [34]. The nominal hole clearances for normal round holes for bolts ranges from 1 – 3 mm, depending upon the diameter of the bolt. For connections in timber, it is specified by *EC5* [53] to have predrilled holes of clearance 1mm (irrespective of the size of the bolt). However, these clearances are under the assumption that the holes are predrilled onsite. When it is required to have the holes also prefabricated, there must be extra clearance in either one of the members (steel beam or timber slab) to account for the geometrical and dimensional deviations of both the members, and the deviations within the structural grid [34].

Nijgh et al [34] investigated these deviations to determine the hole clearance required for the rapid execution of steel-concrete composite structures. In his experiments [35], the shear connectors were embedded in prefabricated concrete slabs, and bolt holes with large clearances were required in the steel flange. In this thesis, the generic formula used by Martin et al [34] is used to determine the hole clearances for the rapid execution of the *STC* floors. These depend on the following factors:

- **Geometric Deviation of the location of Bolt Hole:** This is the deviation in the position on the centreline of the actual bolt with respect to the centreline of the nominal bolt hole. Tolerance limits have been specified by EN 1090 – 2 [62].
- **Out of Straightness of the Beam**: This refers to the initial out of straightness of the beam, arising from the manufacturing processes of the steel beams.
- **Slip due to the execution of the STC floor:** This refers to the slip that occurs as the steel carries the load of the dead weight of the timber slab by bending.
- **Position of Shear Connector within the Timber Slab:** Similar to that of steel beams, this refers to the deviation in the position on the centreline of the actual bolt with respect to the centreline of the nominal bolt hole. In this thesis, this is assumed to be 1mm.
- **Column Offset:** This refers to the deviations in the structural grid. Tolerance limits have been specified by EN 1090 – 2 [62].

The total deviations r_H is given by Eq.4,

$$r_H = \sqrt{(\Delta x_{Hole} + \Delta x_c - \Delta x_{slip} - \Delta x_{sc})^2 + (\Delta y_{Hole} + \Delta y_c + \Delta y_{str,u} - \Delta y_{sc})^2} \quad \textbf{\textit{(Eq 4)}}$$

And the minimum required hole clearance d_H is given by Eq.5.

$$d_H > 2 * r_H \quad \textbf{\textit{(Eq 5)}}$$

The derivation for Eq.4 is given in Appendix D.3.2. As most of the variables considered above are random variables, a Monte – Carlo simulation was done to obtain the minimum bolt hole clearance, based on the condition of 95% probability of success, for Eq.5. Using the value of d_H obtained in this manner, a circular bolt-hole clearance is provided.

Monte Carlo simulation is a Level-III reliability method, in which the probability of success is calculated numerically [74]. This was done using MS Excel, using the function NORMINV, which generates values for normally distributed variables with the mean and standard deviation of the data as input. The uniformly distributed variables were generated with the RAND function which produces random numbers within the specified limits. The number of simulations was so chosen that the probability remains unchanged with a difference of less than 0.5%, each time the program is run.

The bolt hole clearances of *STC* beam chosen in Section 5.2 (*HEA360*) is determined using the procedure mentioned above. The loads under consideration here are the dead loads of the timber slab with all its additions (for sound insulation and fire safety), and the dead load of the steel beam, as these are the loads that are responsible for the slip during execution.

Span of Beam: $L_{Beam} = 10900\,mm$, Timber dead load: $q_g = 5.3 * 0.504 + 1.1 = 3.77\,N/mm$

Bending Stiffness of Beam: $EI_S = 63.3 * 10^{12}\,Nmm^2$,Distance between NA: $r = 267\,mm$

Slip: $\delta_0 = \frac{3.77*10900^3*267}{24*63.3*10^{12}} = 0.41\,mm$, *according to* Eq. 55

According to the derivation in Appendix D.3.2, the values of the different variables are given below in Table 6.5.

Table 6.5: Values of different variables to compute the bolt hole clearance.

Basic Variables	Tolerance Class 1 [mm]			Tolerance Class 2 [mm]		
Normally Distributed Random Variables						
	Tolerance Limit (TC)	*Average* (μ)	*Standard Deviation* (σ)	*Tolerance Limit* (TC)	*Average* (μ)	*Standard Deviation* (σ)
R	-2	0	1.02	-1	0	0.51
A_0	-	3.89	1.91	-	3.89	1.91
c_0	-1	0	0.51	-1	0	0.51
$\Delta Y_{c,1/2}$	-10	0	5.1	-5	0	2.55
$\Delta X_{c,L}$	-10	0	5.1	-5	0	2.55
Uniformly Distributed Random Variables						
	Lower Limit	*Upper Limit*				
x	0	10900				
Θ	0	2π				
Ψ	0	2π				
Deterministic Variables						
	Value					
s_0	0.41					

The probability of success was obtained as the number of outcomes where the criteria is met divided by the total number of simulations. Due to the presence of the edge beams, the column offset value would be the same for all cross beams, due to the axial rigidity of the edge beam. Hence the probability of successful installation of only one beam should be considered here. It was observed that at least 10^4 simulations were required to obtain concurrent results with an accuracy of 0.5%. It was opted to go for elements with Tolerance Class 2 for minimizing the bolt hole clearance required. Also, no braces were used. The bolt hole clearance was determined to be 16 mm (irrespective of the size of the bolt). The probability curve for successful installation of bolts with respect to the size of the bolt hole clearance is given below in Figure 6.9. For comparison, the value obtained by Martin et al [34] for steel-concrete composite beams was 19 mm for Tolerance Class 2, for unbraced systems. The latter value is larger due to the larger span and higher dead weight imposed by the concrete slab, leading to more slip. For the *STC* beam without composite action (*HEA400*) of the same Tolerance Class, the bolt hole clearances were determined to be the same, at 16 mm. Despite the higher stiffness of the steel beam leading to lesser slip during execution, the total deviations do not decrease significantly.

From these calculations it was observed that the main factors affecting the size of the bolt hole clearance was the tolerance class of the structure and the size of the structural grid (i.e., span of steel beam: 10.9 m, and spacing between them: 5.3 m). The deviations due to slip during execution were minimal (less than 1 mm). The largest deviations were on account of the column offset and by the out of straightness of the beam (up to 10 mm). The deviations due to the out of straightness of the beam can be reduced by using braces – however this was not done, as it would increase the overall footprint of the structure. The tolerance class of the elements is what determines the amount of deviations for each parameter. Thus, the requirement of large bolt clearances was mitigated to a certain extent by using elements with tolerances class 2 (thus the final value of 16 mm bolt hole clearance)

Considering the aspect of predrilling these oversized bolt holes, it is much easier to have them in timber rather than in steel (even despite the difference in thickness: 34 mm for timber vs 17.5 mm for steel). Thinking on similar lines, it would also prove to cost much lesser. Thus, to conclude, it is opted to have circular 16 mm oversized holes for the M14 bolts, in the timber flange.

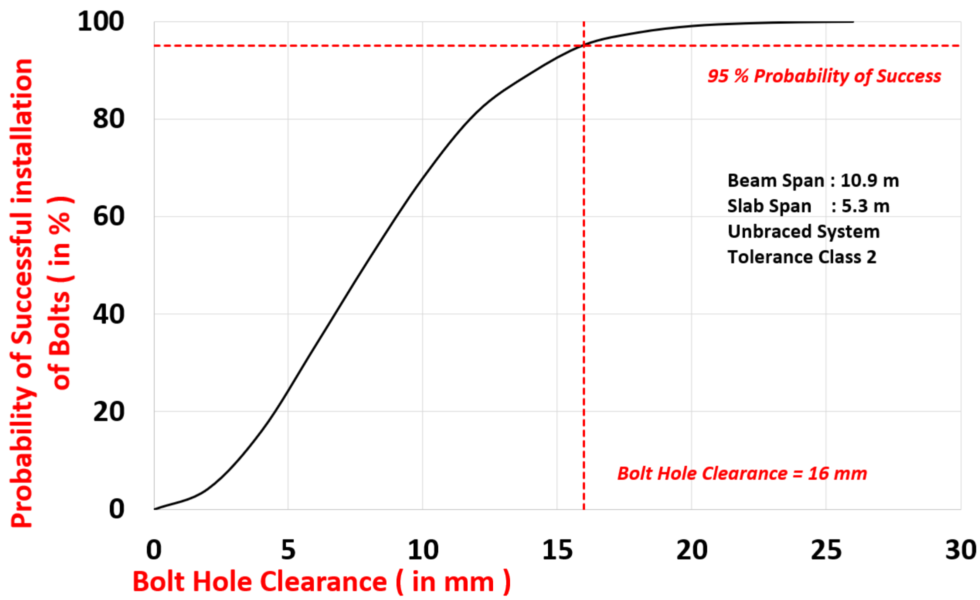


Figure 6.9: Probability of successful installation of bolts designed to be demountable.

Large hole clearances required for the successful execution of the demountable shear connectors implies that there will be slip i.e., the bolt will undergo displacement until it establishes contact with the encasing surface. This leads to a decrease in the stiffness of the bolts, which ultimately influences the degree of shear interaction between steel and timber. Hence it is required to use slip – resistant shear connectors. There are 2 conventional solutions for this:

- **Preloaded Bolts:** By loading the bolts in tension during execution, it generates frictional forces between the surfaces in contact. Hence there will be no slip until this frictional force is overcome, resulting in high initial stiffness.
- **Resin Injected Bolts:** By injecting epoxy resins into the bolt hole clearances, these provide resistance against slip.

However, due to the limited strength of timber loaded in compression perpendicular to the grain, preloading bolts is not a viable solution. Thus, the viable solution for steel-timber is to use resin injected bolted connections. Bearing failure of resin (considering a conservative value of bearing stress for resin) was not found to be governing, in calculating the design resistances of the connection in Section 6.3.1. The main aspect to be considered is the decrease in the stiffness of the connection, as we are replacing timber material in the oversized hole with resin. At present, there are no experimental studies available to investigate further into the stiffness of the resin injected bolted connections for steel-timber. Thus, this aspect is not considered further in this thesis. *FRP* material bears resemblance to timber in the aspect that both are made of a resin-fibre matrix. Fruzsina et al [36] successfully conducted experiments on steel – *FRP* bolted connections with steel reinforced resin injected into the oversized holes in *FRP*. One of the main observations from their experiments was that, in these connections, the failure was observed with damage in the resin and the bolt i.e., there was no observable damage in the *FRP* material. Thus, such a connection with timber could also give similar benefits.

6.4 STC Beam Design

The *STC* beam under consideration here are the middle cross beams used case study ***DA1_STC*** from Section 5.2. The design has been made with and without considering composite action between the timber slab *LFE160* and the steel beam *HEA360/HEA400* i.e., without considering composite action *HEA400* is required to carry all the loads, and by considering composite action, the steel section can be reduced to *HEA360.* Figure 6.10 below shows the section of the *STC* beam with composite action.

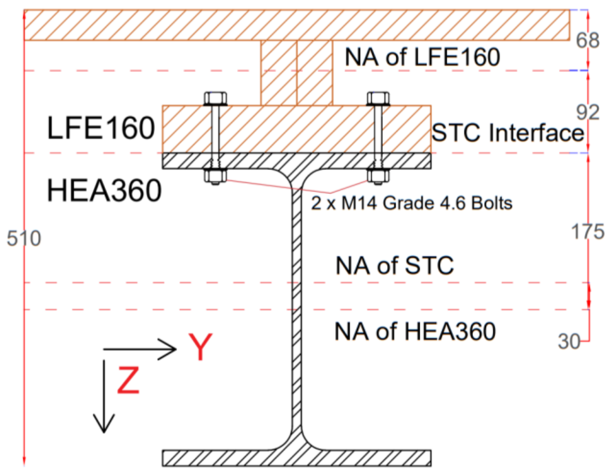


Figure 6.10: STC Section.

Table 6.6:Properties of STC Beam.

Steel Beam ***HEA360***
$h_S = 350\,mm, b_S = 300\,mm$
$t_{S,w} = 10\,mm, t_{S,f} = 17.5\,mm$
Distance of NA to top and bottom fibre of Steel:
$z_{S,t} = 175\,mm = z_{S,b}$
Timber Slab ***LFE160***
$h_T = 160\,mm, b_{T,tf} = 610\,mm$
$t_{T,tf} = 34\,mm, t_{T,bf} = 53\,mm,$
$t_{T,w} = 80\,mm$
Distance of NA to top and bottom fibre of Timber:
$z_{T,t} = 68\,mm, z_{T,b} = 92\,mm$

Composite Section ***STC510***
Distance of NA of Timber/Steel from the top of the STC section: $e_T = 68\,mm, e_S = 335\,mm$
Distance of NA of Timber/Steel to the STC Interface: $r_T = 92\,mm, r_S = 175\,mm,\ r = 267\,mm,$
Distance of Timber/Steel NA to NA of STC: $a_T = 237\,mm, a_S = 30\,mm$
$*\,Subscript\ T \rightarrow Timber, S \rightarrow Steel$

Table 6.7: Load Considerations for Design of STC Beam.

Characteristic Values	**Description**
$q_G = 10.9\,kN/m$	Dead load of steel beam, timber slab, floor finish, installations, fire protection and sound insulation.
$q_Q = 19.08\,kN/m$	Office Category B Imposed Loads (including partitions) [57]
Design Values	
$q_{SLS} = 29.8\,kN/m$ $q_{ULS} = 41.5\,kN/m$	
Precamber of Steel Beam	
$w_p = 31.5\,mm$	

The longitudinal section of the *STC* under consideration is given above in Figure 6.10 (which is the transverse section of the timber slab). The effective width of the timber top flange active in composite action depends on shear lag, as mentioned in Section 6.1.1. It was required to use reoriented panels for this as well, to have net savings in the size of the steel section. This is determined to be $610\,mm$ (see Appendix D.4). It is assumed that the maximum thickness of the reoriented timber bottom flange that can be used is one-third of the total height of the section. This gives us $t_{T,bf} = 53\,mm$. As obtained from Section 6.3, the optimum bolts that can be used are Grade 4.6 M22 for the connections, for highest stiffness. It was observed that the magnitude of shear forces to be resisted by the connections are low in this example. Hence it is decided to adopt Grade 4.6 M14 bolts. The basis for this design choice is so that there can be uniform comparison between the number of bolts required for composite action, and the minimum number required for diaphragm action of the floor system. The optimum size of the bolt hole clearance for rapid execution d_H was determined to be 16 mm (for Tolerance Class 2), which is independent of the size of bolts used. The load carrying capacity of each bolt, $P_{Rd,Bolt} = 13.1\,kN.$ From Figure 6.10, we can observe that there are 2 such bolts in each row.

6.4.1 STC Beam with Full Composite Action

Before looking into partial shear interaction for the *STC* beam, we look into the case with full shear interaction. This will help us get an overview of the maximum gains that can be obtained using composite action. As mentioned earlier, an *STC* beam with full composite action can be analysed with the bending stiffness for full shear interaction obtained using Steiner's rule. The detailed calculations are given in Appendix D.4.2.

The main objective in designing the *STC* beam with composite action is to increase EI and $M_{el,Rd}$. As mentioned earlier, *STC* composite action is achieved only after the whole beam – slab setup has been erected, and the steel beam should be undeformed at the onset of composite action. The dead load is completely taken by the steel beam, and the steel beam is precambered to negate the deflections due to the dead loads. Thus, this is the starting point for design of a *STC* beam. The amount of precamber required for the steel is obtained as follows:

$$w_p = \frac{5 * q_G * {L_B}^4}{384 * EI_S} = 31.51\,mm$$

$$where\ q_G = 10.85\,kN/m\ is\ the\ total\ Dead\ Load$$

Here, it should be checked that the dead loads acting on the steel beam is within the elastic limit load of the steel section:

$$\rightarrow q_{el,S} = \frac{8 * f_y * I_S}{z_{S,t/b} * {L_B}^2} = 43.23 > q_G = 10.85\,kN/m$$

The deformations due to the dead loads are negated by precambering. However, these produce stresses in the steel section:

$$\sigma_{S,G,t/b} = \frac{M_G * z_{S,t/b}}{I_S} = +-89.1\,Nmm^{-2}$$

Where,

$$M_G = \frac{q_G * {L_B}^2}{8} \; is\ the\ maximum\ bending\ moment\ due\ to\ the\ dead\ loads$$

The remaining loads are taken by the *STC* section:

$$q_{STC,SLS} = q_{SLS} - q_G = 18.98\ kN/m, \qquad q_{STC,ULS} = q_{ULS} - q_G = 30.64\ kN/m$$

The total bending stiffness due to composite action is obtained as $EI_{STC} = 8.712 * 10^{13}\ Nmm^2$. The elastic yield load for the composite section is obtained from the yielding of extreme fibres in either steel or timber, similar to the approach in Section 6.2. This will depend on the geometry of the sections being combined. In the situation where the timber is yielding first, there is reserve strength in steel that can be utilized, since the steel beam is precambered i.e., the steel beam (and hence the system) can take more dead loads, which is an additional load from the loads that can be taken by the *STC* section. This was the case in the example used in this thesis. For the *STC* section only, the top of the timber component was yielding first, which gave an elastic load limit obtained as follows:

$$q_{el,STC} = \frac{8 * f_{md} * EI_{STC}}{E_T * z_{STC,T,t} * {L_B}^2} = 43.65\ kN/m$$

It should be checked that the loads in the *STC* section (i.e., the live loads) are within its elastic limit,

$$\rightarrow q_{el,STC} = 43.65 > q_{STC,ULS} = 30.64\ kN/m$$

Considering the whole precambered system, the bottom of the steel component was yielding, which gave an elastic load limit for the whole system as follows:

$$q_{el,STC,Total} = \frac{8 * (f_y - \sigma_{G,S}) * EI_{STC}}{E_S * z_{STC,S,b} * {L_B}^2} + q_G = 54.5\ kN/m$$

It should be checked that the total *ULS* loads in the system are within its elastic limit,

$$\rightarrow q_{el,STC,Total} = 54.5 > q_{ULS} = 41.5\ kN/m$$

The mid-span deflection of the beam under *SLS* live loads is given as:

$$\Delta_{mid-span} = \frac{5 * q_{STC,SLS} * {L_B}^4}{384 * EI_{STC}} = 42.12\ mm$$

The maximum permissible deflection for a steel beam is:

$$\Delta_{max} = L_B/250 = 43.6\ mm$$

Figure 6.11 below shows the load deflection curve of the *STC* beam. The total increase in bending stiffness due to composite action is 37.7%, from $EI_S = 6.33 * 10^{13}\ Nmm^2$ (for the steel beam only), to $EI_{STC} = 8.712 * 10^{13}\ Nmm^2$. The increase in the total elastic load limit is 13.2%, from $q_{el,S} = 43.23\ kN/m$ to $q_{el,STC} = 48.9\ kN/m$. This corresponds to an increase in elastic bending moment capacity from $M_{el,Rd,S} = 642.1\ kNm$ to $M_{el,Rd,STC} = 726.9\ kNm$.

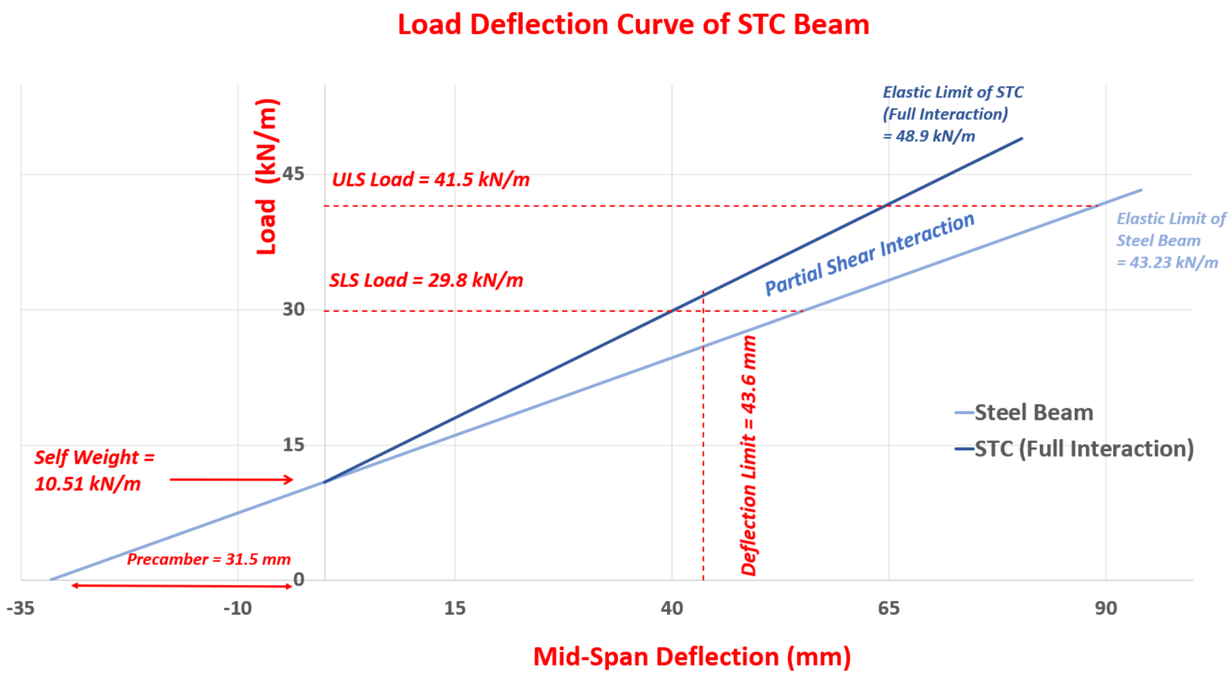


Figure 6.11: Load - Deflection Curve of the STC Beam.

Finally, the normal stresses in the *STC* section are calculated with EI_{STC}, using Euler Bernoulli's theorem. As mentioned earlier, precambering results in stresses in the steel beam. All the excess loads beyond the dead loads are taken by the composite beam. Thus, the total stresses in the *STC* section is the sum of these two stresses, as shown in Figure 6.12. In this manner, the total stresses at *ULS* are obtained for the *STC* section, as shown in Figure 6.13. This also shows the values of normal stresses divided by their respective bending strengths. The calculations are given in Appendix D.4.2, Table D.8. As shown in Figure 6.13, the decrease in the maximum bending stresses (at the bottom of steel section, in tension) is 11.1%, from 341 MPa to 303 MPa.

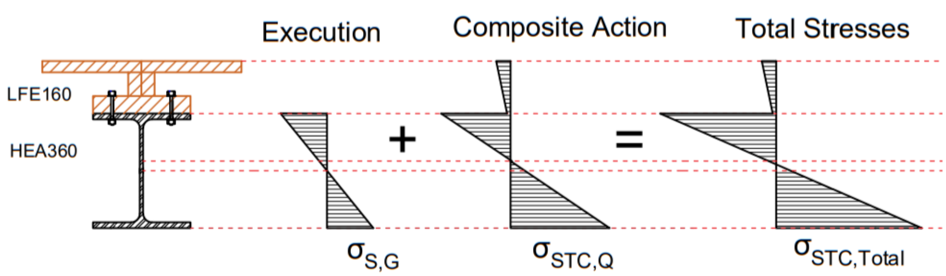


Figure 6.12: Load mechanism of STC Section, showing the distribution of normal stresses.

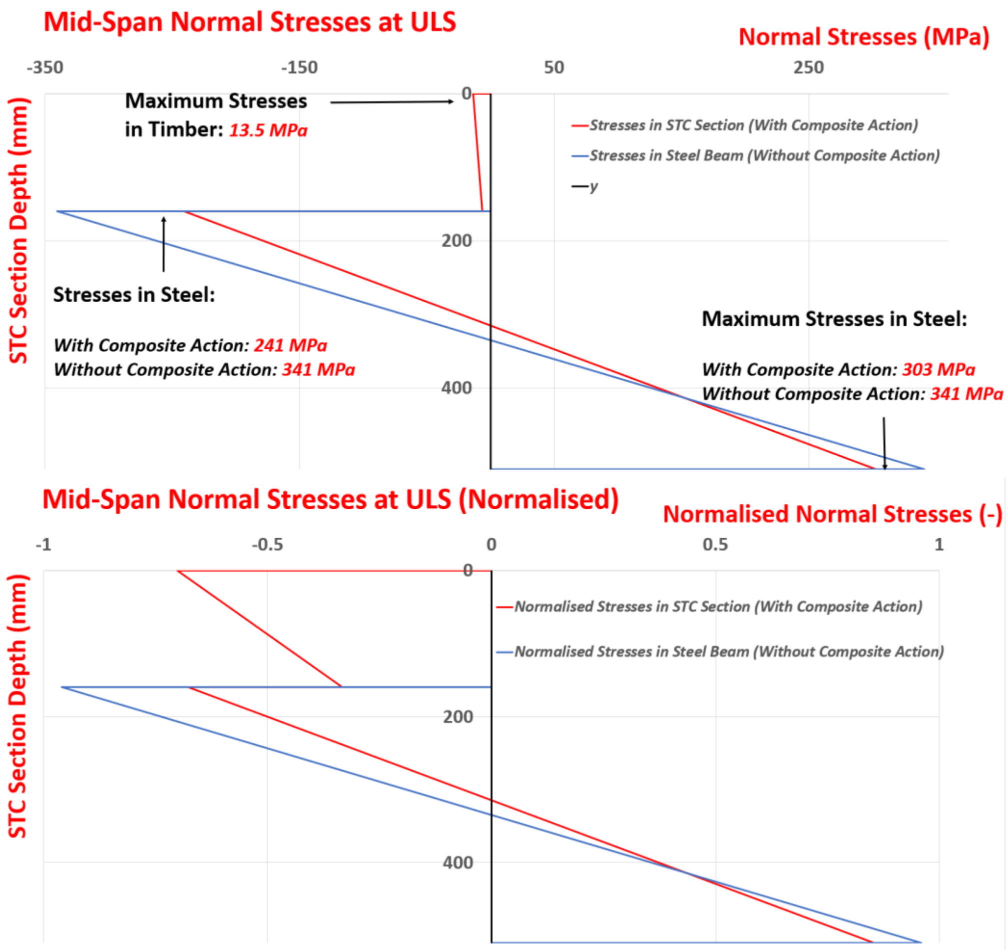


Figure 6.13: Stresses in the STC Section. (Top) Values in MPa. (Bottom) Values normalised to bending strengths of steel/timber.

As mentioned in Section 6.2, calculations are done on the $M_{pl,Rd}$ of the *STC* section, to make a comparison with that of the steel beam. This is done using the approach for steel – concrete composite structures, according to *EC4* [64]. The values are not used further ahead in thesis and is only for comparison. The calculations for the same are given in Appendix D.4.4. The value thus obtained is $M_{pl,Rd,STC} = 1074.7\ kNm$, which showed an increase of 18.2% over that of the steel beam alone ($M_{pl,Rd,S} = 909.4\ kNm$).

Table 6.8 below shows the summary of the change in the mechanical properties due to composite action. These include the bending stiffness, the decrease in the peak stresses in the steel section (at the bottom, in tension), and the elastic and plastic Bending Moment Capacity. In the last column, the same values for the benefits of composite action in steel – concrete from ***DA3_CS*** is also given. The calculations for the latter are given in Appendix D.4.4. The structural scheme for ***DA3_CS*** is the same as ***DA1_STC***, to make a fair comparison. The cross beams are spanning 10.9 m façade to façade, and the span of the composite slabs is 3.53 m (compared to 5.3 m for the *LFE* slab.

Table 6.8: Summary of Change in Properties due to Composite Action in STC.

Property	Steel Beam	STC Beam	Percentage Change	Percentage Change for Steel Concrete Composite Action
Bending Stiffness $(\boldsymbol{EI})\ [\boldsymbol{x10^{13}\ Nmm^2}]$	*6.331*	*8.718*	**+38**	***+210.24%***
Peak Stresses in Steel $(\boldsymbol{\sigma_{S,b}})\ [\boldsymbol{Nmm^{-2}}]$	*341*	*303*	**-11**	***NA***
Elastic Bending Moment Capacity $(\boldsymbol{M_{el,Rd}})\ [\boldsymbol{kNm}]$	*642.1*	*727*	**+13**	***+33***
Plastic Bending Moment Capacity $(\boldsymbol{M_{pl,Rd}})\ [\boldsymbol{kNm}]$	*741.4*	*876.4*	**+18**	***+80%***

Composite action in steel – concrete has been chosen as a standard for comparison, because it is accepted in industry, to a certain extent and the gains are sufficient, to justify the use of extra shear connectors (as concluded in C. Braendstrup's master's thesis, [68]). Since timber is a much weaker material than concrete (with the difference being much more pronounced compared to steel), it is understandable that benefits of composite action in *STC* will not as large as that in steel – concrete. However, as observed in Table 6.8, the difference is very large. For steel – concrete, the corresponding value is up to 5x larger for bending stiffness, and 3 – 4x larger for bending moment capacity, compared to *STC.*

Comparing the total savings in steel per unit area, it is $5.44\ kg/m^2$ for steel – concrete and $1.8\ kg/m^2$ for *STC.* It might seem contradictory that the effective gain in steel is comparable between the steel – concrete and *STC*. The benefits of reduction of floor height is also the same, at $40\ mm$. The reason for this is as follows: The calculations on *STC* represent the best possible outcome i.e., it involves reorientation of the timber flanges, which takes extra effort. Also, the deflection of the system is governing, which is also the area where *STC* gives the largest gains. Thus, it can be said that no more benefits of composite action can be achieved for this system. For steel – concrete, no extra efforts are required. It is the bending moments which are governing. This is because elastic analysis and design is done, owing to reusability. Had there been the possibility for plastic design, the steel sections could be reduced further to *HEA240*, thus obtaining a net savings of $14.7\ kg/m^2$ of steel, and a reduction of floor height by $100\ mm$. This is much larger than what is obtained by composite action in *STC.*

Last but not least, a comparison is made on the number of shear connectors required for composite action in *STC* and in steel – concrete. Shear connectors are quite expensive and labour intensive to execute. It is the sole additional cost that needs to be borne for achieving composite action (apart from reorientation of timber flanges in *STC*). Thus, the governing criteria for justifying the use of composite action is always a comparison between the added costs of shear connectors versus the savings in steel material obtained. The standard value (base value) for this comparison is the total number of shear connectors required for diaphragm action of the floor system. For *STC,* the minimum number of shear connectors required is 2x7 Grade 4.6 M14 bolts. For composite action in *STC,* the total number of shear connectors required is 2x68 bolts of the same type bolts per cross beam, compared to 2x46 bolts for steel – concrete. It is assumed that designing the connections to withstand the longitudinal shear flow at the interface of steel – timber or steel – concrete will provide full composite action with elastic distribution of stress. Moreover, there is the possibility of lesser number of bolts for steel – concrete, by increasing the diameter and grade of the bolts. For *STC,* this option is limited, to maintain ductile failure modes in the connection. The resistance of the bolts are much smaller for *STC.* For the above mentioned bolts used, the resistance for *STC* is $13.1\ kN$, compared to $39.4\ kN$ for steel – concrete (about 2.6x). Thus, as you can see, more number of shear connectors are required for *STC,* while giving minimal benefits.

Combining all these aspects, it is concluded that the benefits of composite action for the particular *STC* floor chosen, is not sufficient to be of any practical significance. This conclusion is based on a comparison with composite action in steel – concrete, which is assumed to the benchmark to compare the advantages of composite action in any system. However, the drawback to this comparison is as follows: In steel – concrete composite slabs, the concrete component is two-way spanning (as concrete is isotropic). Apart from the fact that timber is an orthotropic material, the particular slab chosen is a one-way spanning box slab i.e., there is a large difference between the mechanical properties of the section of slab in the x and y directions. Thus, this comparison between *STC* floors using *LFE* slabs, and steel-concrete composite slabs, cannot be considered as a fair comparison, for assessing the benefits of composite action in steel-timber.

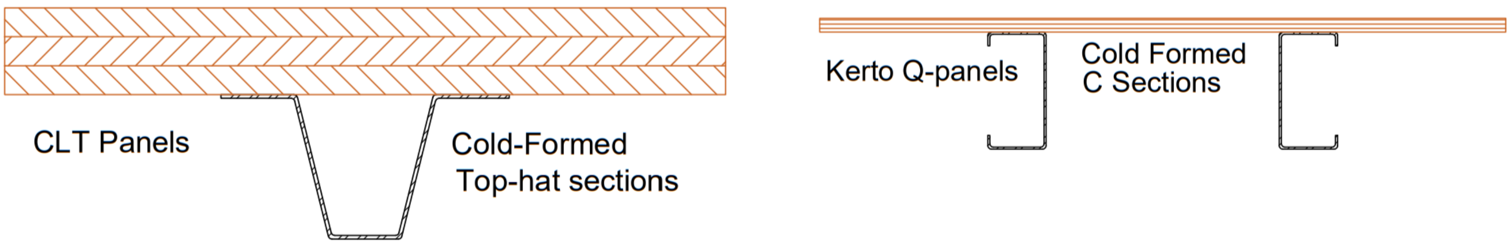
The most relevant type of timber slab, that can be compared to composite slabs are the *CLT* solid slabs. As discussed in Section 3.1, the use of alternate lamellas oriented parallel and perpendicular to each other ensures that these elements can produce sufficient capacity for double bending. Thus, the same calculations are done for such an *STC* floor system with *CLT* solid slabs (**STC1** from Table 4.3). These are given in Appendix D.4.5. From the calculations, it was observed that the effective bending stiffness in the x-direction $EI_{T,eff,x}$ (active in composite action) was lesser than what was obtained for the *LFE* slabs. This is because for the lattter, reoriented timber flanges have been used to increase its mechanical properties. While *CLT* solid slabs offer the advantage of more section height (200 mm vs 160 mm for *LFE*) and more material, the outer fibres active in load bearing are transversely oriented (low bending stiffness). Since the bending stiffness of the slab is lesser, the net benefits due to composite action will also be lesser. Thus the chosen *STC* system with *LFE* slabs (which use reoriented timber flanges near the supports for composite action) gives a representative value for composite action in steel-timber.

Based on these calculations, from the calculations of composite action in the chosen *STC* floor system i.e., Lignature box elements combined with I beams, the conclusions can expanded to be applicable to all such steel-timber floor systems. The reason for the limited benefits is that the mechanical properties of timber are too weak to have added benefits of the section of the 'composite beam', in combination with steel. The above-mentioned calculations were for the case of full shear interaction. In the case of partial shear interaction (based on the procedure in Section 6.2), the net benefits will be even lesser. The only advantage is that lesser shear connectors can be used, in the case of partial shear interaction.

The case of composite action considered in this section was for large span elements, to see whether there would be any benefits for the steel beam supporting the slab, and it was concluded that there could be no such practical gains. However, the converse can be true i.e., using small steel sections as stiffeners can aid the performance of timber slabs. This conclusion points in the direction of new hybrid steel-timber slabs, and thus was not considered in the *MCA* in Chapter 4 (as it addressed only steel timber floor systems which were currently available in the market). Experiments have been conducted on such systems and are discussed in Section 3.6. For example, a hybrid element with cold formed C section bonded onto the bottom of *CLT* panels is considered for 4-point bending in [25]. The result was that there was an increase in bending stiffness by about 8x compared to the *CLT* section alone.

A plethora of such hybrid steel-timber slabs can be obtained. The best solution for timber is to use products such as *CLT* or Kerto Q – panels manufactured by Metsawood, which offer higher transverse stiffness. For steel, it is recommended to use cold – formed elements such as C – sections or Z profiles. The role of steel is to act as stiffeners or webs, and give higher mechanical properties by composite action, compared to the elements with timber only. A few such examples are given below in Figure 6.14. The gains in mechanical properties provided by composite action can lead to higher spans, which ultimately leads to lesser weight per unit area and environmental costs.

Figure 6.14: Optimum STC Solutions for Composite Action.



6.5 Design Recommendations for STC Floors

In Section 6.4, it was concluded that the use of composite action in *STC* sections cannot be justified based the analysis of perceived gains. This is mainly due to the use of expensive shear connectors, which cannot be met from the savings in material (steel) per floor. The only advantage that composite action can provide, is a reduction in floor height, although not to a large extent. The main advantage for reduction in floor height is for high-rise buildings, where the reduction of the area of façade claddings can come into play. However, this is rarely the case, especially in the Netherlands.

On the other hand, *STC* floors provide a huge advantage with respect to the other floor systems (*HCS* and Composite Slabs), even without the use of composite action. As we have seen in Chapter 5, DA1_STC was the preferred option for a floor system, considering aspects of total weight of the floor system, and the total environmental impact (as we will see in Chapter 8). Apart from this, *STC* floors also allowed for easier handling onsite and transportation (owing to its lightweight nature). Thus, even without the use of composite action, *STC* floors provide us a competitive option, as alternative to the conventional floor systems mentioned earlier. The main disadvantage for *STC* floors was due to its large floor height, as it was a non – integrated floor system. Even regarding this aspect, the *STC* floors were comparable to that of Composite Floors.

In order to make design recommendations for this *STC* floor, span tables are made these floors for office use, with respect to the typical layouts used for Dutch offices, according to [132]. Typically, Dutch offices are of 4 generic types:

- ➢ **Cocoon Office and Combination Office:** Shown in Figure 6.15a. Usually, these include 2 rows of middle columns between the facades. The size of the column grid varies from 3.6 – 7.2 m along the façade, and 5.4 – 9 m for the internal supports.
- ➢ **Cell Office:** Shown in Figure 6.15b. This layout includes 1 row of centre columns, between the façades. The size of the column grid for this layout also varies from 3.6 – 7.2 m along the façade, and 5.4 – 9 m for the internal supports.
- ➢ **Group Office:** Shown in Figure 6.15c. This layout is designed without the use of internal columns i.e. with the floor system spanning from façade to façade. This is the most demanding in terms of structural design, with the column spacing varying from 1.8 – 7.2 m along the façade. The depth of the building varies from 10.8 – 16.2 m.

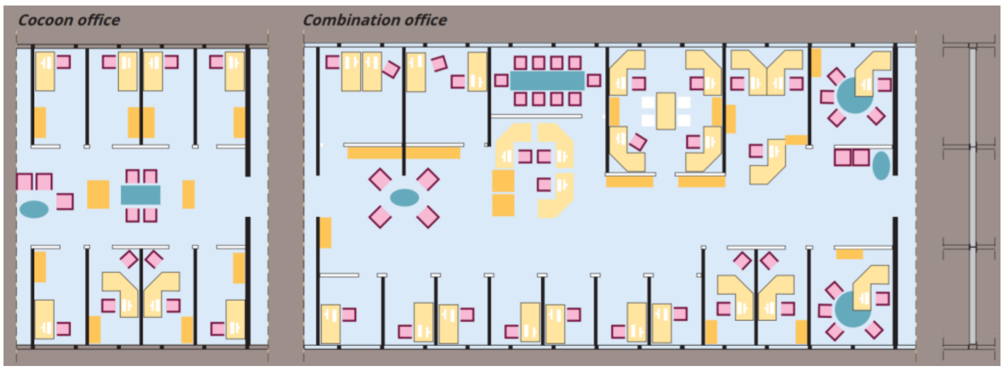
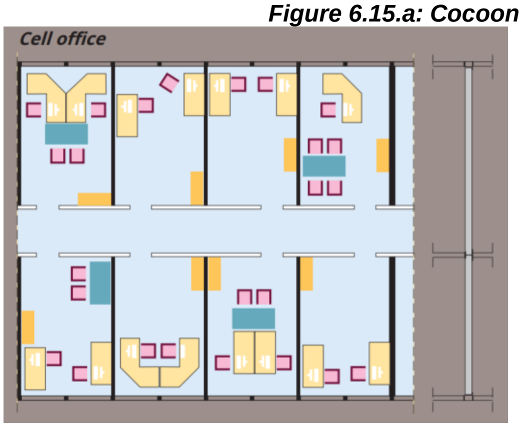


Figure 6.15.a: Cocoon Office (Left), and Combination Office (Right).



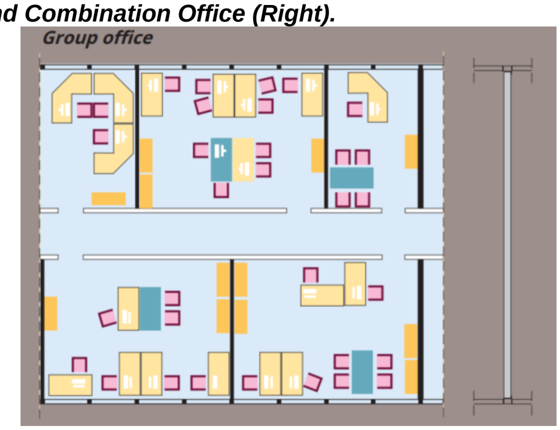


Figure 6.15.b: Cell Office.

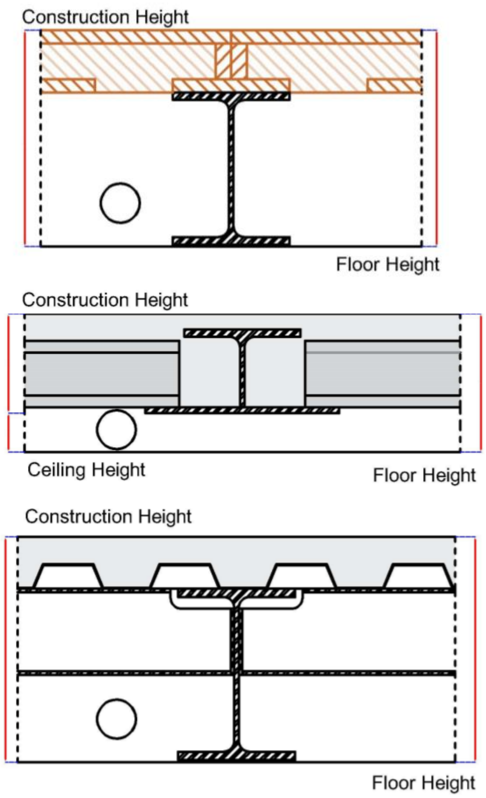
Figure 6.15.c: Group Office.

Figure 6.15: Typical Layouts of Dutch Offices, from [132].

From the perspective of flexibility of office use, it is preferred to have maximum column – free spans. This is the reason that the design of Bouwdeel D was changed from the original Cell Office layout to Group Office layout, in Chapter 5. The depth of the sections required to implement the *STC* floor has been obtained for the different layouts and spans of Dutch Offices. Even though, it was concluded that the use of composite action could not lead to any benefits, the sections are determined for the case of composite action as well as for without composite action. This is to see the sensitivity of benefits of composite action, with respect to the spans of the timber slabs and steel beams. All of this data is summarized below in Table 6.9. The main goal of obtaining these span tables is so that they can serve as reference for the design of *STC* floor, for architects and structural designers. The assumptions and calculations for the same are given in Appendix D.5.

Finally, the obtained depths of the *STC* floors are compared with the 2 industry standard floors: *HCS* and Composite Slabs, as given in [132]. All the floor systems compared are with steel framework. The sections of the three floor systems compared is given below in Figure 6.16. Among the three floor systems compared, *STC* floors and Composite Slab floors are supported by I-beams (as a non-integrated system). Thus, the pipes for services can be placed within the height of the construction. For floors with *HCS,* the slabs are supported using integrated beams. Thus, extra height must be provided below the height of construction, to provide space for the services. For composite slabs, the optimum span without the use of propping is 3.6 m [132,99]. Hence, wherever required, secondary beams were used to span across larger dimensions.

Figure 6.16: Sections for different Floors Systems.

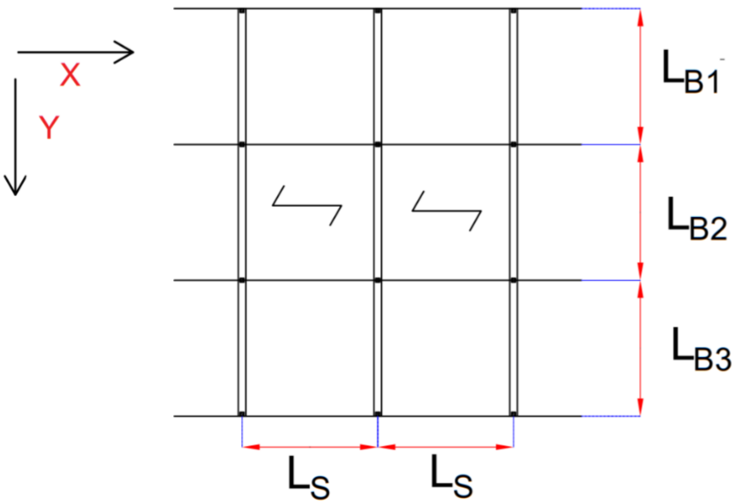


(Top) STC floor system.

(Middle) HCS floor system.

(Bottom) Composite Slab floor system.

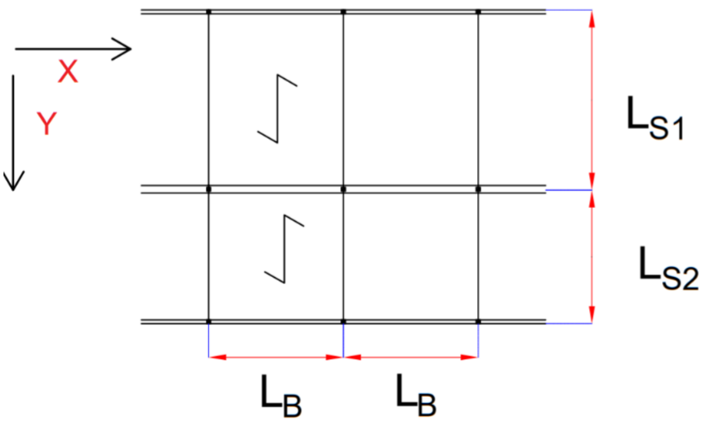
Table 6.9: Span Tables for STC for Dutch Office layouts.
a) Cocoon/Combination Office



Timber Slabs (*LFE*) spanning in x-direction, supported by I-Beams (*HEA*)
I-Beams with 2 intermediate column supports, spanning in y-direction.

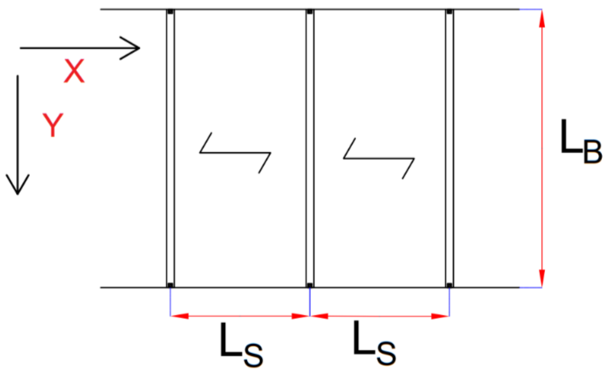
Building Depth (Façade to Façade) $(L_B = L_{B1} + L_{B2} + L_{B3})$ $[m]$	**STC Floor System**					**Floor Height for HCS** (d_{HCS}) $[mm]$	**Floor Height for Composite Slab** (d_{CS}) $[mm]$
	Slab (LFE)		**Beam (HEA)**				
	Span of Slab (L_S) $[m]$	**Depth of Slab** (d_S) $[mm]$	**Maximum Span of Beam** $(L_{B,max})$ $[m]$	**Floor Height for STC** (d_{STC}) $[mm]$			
				With Composite Action	**Without Composite Action**		
16.2 (5.4+5.4+5.4)	*3.6*	*120*	*5.4*	***313***	***370***	***NA***	***370***
	5.4	*160*	*5.4*	***353***	***430***	***295***	***390***
	7.2	*240*	*5.4*	***414***	***530***	***315***	***NA***
	9	*320*	*5.4*	***494***	***630***	***375***	***NA***
18 (5.4+7.2+5.4)	*3.6*	*120*	*7.2*	***390***	***410***	***NA***	***440***
	5.4	*160*	*7.2*	***430***	***490***	***NA***	***480***
	7.2	*240*	*7.2*	***471***	***590***	***375***	***540***
	9	*320*	*7.2*	***551***	***690***	***385***	***NA***
19.8 (5.4+9+5.4)	*3.6*	*120*	*9*	***430***	***450***	***NA***	***490***
	5.4	*160*	*9*	***490***	***530***	***NA***	***580***
	7.2	*240*	*9*	***550***	***630***	***NA***	***630***
	9	*320*	*9*	***610***	***770***	***425***	***630***
19.8 (7.2+5.4+7.2)	*3.6*	*120*	*7.2*	***390***	***410***	***NA***	***440***
	5.4	*160*	*7.2*	***430***	***490***	***NA***	***480***
	7.2	*240*	*7.2*	***471***	***590***	***375***	***540***
	9	*320*	*7.2*	***551***	***690***	***385***	***NA***

b) Cell Office



Timber Slabs (*LFE*) spanning in y-direction, from façade to façade, supported by I-Beams (*HEA*), with an intermediate support.
I-Beams spanning in x-direction, along the façade. 1 intermediate I-beam to support slabs.

Building Depth (Façade to Façade) $(L_S = L_{S1} + L_{S2})$ $[m]$	**STC Floor System**					**Floor Height for HCS** (d_{HCS}) $[mm]$	**Floor Height for Composite Slab** (d_{CS}) $[mm]$
	Slab (LFE)		**Beam (HEA)**				
	Maximum Span of Slab (L_S, max) $[m]$	**Depth of Slab** (d_S) $[mm]$	**Span of Beam** (L_B) $[m]$	**Floor Height for STC** (d_{STC}) $[mm]$			
				With Composite Action	**Without Composite Action**		
12.6 (5.4+7.2)	*7.2*	*240*	*3.6*	***336***	***396***	***315***	***440***
	7.2	*240*	*5.4*	***354***	***414***	***315***	***480***
	7.2	*240*	*7.2*	***411***	***471***	***375***	***540***
	9	*320*	*3.6*	***416***	***476***	***375***	***490***
14.4 (5.4+9)	*9*	*320*	*5.4*	***453***	***513***	***375***	***580***
	9	*320*	*7.2*	***491***	***551***	***375***	***630***
	7.2	*240*	*3.6*	***336***	***396***	***315***	***440***
	7.2	*240*	*5.4*	***354***	***414***	***315***	***480***
14.4 (7.2+7.2)	*7.2*	*240*	*7.2*	***411***	***471***	***375***	***540***
	9	*320*	*3.6*	***416***	***476***	***375***	***490***
	9	*320*	*5.4*	***453***	***513***	***375***	***580***
	9	*320*	*7.2*	***491***	***551***	***375***	***630***
16.2 (7.2+9)	*7.2*	*240*	*3.6*	***336***	***396***	***315***	***440***
	7.2	*240*	*5.4*	***354***	***414***	***315***	***480***
	7.2	*240*	*7.2*	***411***	***471***	***375***	***540***
	9	*320*	*3.6*	***416***	***476***	***375***	***490***



Timber Slabs (*LFE*) spanning in x-direction, supported by I-Beams (*HEA*).
I-Beams spanning in y-direction, from façade to façade, without the use of intermediate column supports.

Building Depth (Façade to Façade) (L_B) $[m]$	STC Floor System: Slab (LFE): Span of Slab (L_S) $[m]$	STC Floor System: Slab (LFE): Depth of Slab (d_S) $[mm]$	STC Floor System: Beam (HEA): Span of Beam (L_B) $[m]$	STC Floor System: Beam (HEA): Floor Height for STC (d_{STC}) $[mm]$: With Composite Action	STC Floor System: Beam (HEA): Floor Height for STC (d_{STC}) $[mm]$: Without Composite Action	Floor Height for HCS (d_{HCS}) $[mm]$	Floor Height for Composite Slab (d_{CS}) $[mm]$
10.8	*3.6*	*120*	*10.8*	***410***	***470***	***250***	***650***
	5.4	*160*	*10.8*	***470***	***530***	***250***	***610***
	7.2	*240*	*10.8*	***550***	***610***	***315***	***650***
	9	*320*	*10.8*	***610***	***670***	***NA***	***NA***
12.6	*3.6*	*120*	*12.6*	***470***	***530***	***250***	***740***
	5.4	*160*	*12.6*	***550***	***610***	***250***	***670***
	7.2	*240*	*12.6*	***630***	***690***	***315***	***740***
	9	*320*	*12.6*	***710***	***770***	***NA***	***NA***
14.4	*3.6*	*120*	*14.4*	***560***	***620***	***435***	***810***
	5.4	*160*	*14.4*	***600***	***660***	***435***	***730***
	7.2	*240*	*14.4*	***680***	***740***	***435***	***810***
	9	*320*	*14.4*	***810***	***870***	***NA***	***NA***
16.2	*3.6*	*120*	*16.2*	***610***	***670***	***515***	***890***
	5.4	*160*	*16.2*	***700***	***760***	***515***	***800***
	7.2	*240*	*16.2*	***780***	***840***	***515***	***890***
	9	*320*	*16.2*	***860***	***920***	***NA***	***NA***

From the span tables given above for *STC* floors, by comparing the sections for the case with and without composite action, it can be observed that the benefits from composite action are still very small. Obviously, the largest amount of gains are present in the case where large timber sections are combined with small steel sections. This can be used to substantiate the claim made earlier about hybrid steel-timber slabs, and how composite action can be used to increase their properties.

6.6 Summary

The span tables for *LFE* sections have been determined for office use. Compared to *HCS* and Composite slabs, *LFE* sections are very light weight. Added to this benefit, they also offer much larger spans than Composite slabs (without the use of propping), although not as much as *HCS.*

The shear connectors opted for use in *STC* floors are bolted connections. The resistance of bolts is governed by Johannsen yield models [3]. Failure of bolt in shear or due to bearing failure of resin is not governing. For plastic distribution of stresses, it is required to have ductile connections. This makes it necessary to not have the governing failure mode of the connection as timber in bearing, as this is brittle. From this we get the minimum thickness of timber bottom flange. It is advised to have larger bolts for stiffness, but with lesser grades, to use minimum amount of timber material. *STC* bolted connections with thick plates give much higher load carrying capacity than those with thin plates (approximately 1.5x). The diameter of the bolts should be chosen accordingly. Considering the aspect of drilling bolt holes, it is decided to limit the thickness of steel flange to 25 mm in this thesis.

For rapid execution of these prefabricated elements, the bolt holes for the *STC* beam must be predrilled in the timber and steel flanges. For a successful execution onsite, we require additional bolt hole clearances to account for the geometrical deviations in elements and the structural grid. This results in bolts with oversized holes and calls for the need of slip resistant connections to provide sufficient stiffness for achieving composite action in the *STC* beam. It is decided to use resin injected bolted connections rather than pretensioned bolts as the latter is limited by the compressive strength of timber perpendicular to the grain.

Analysis of *STC* beams can be done with elastic and plastic methods. As the Gamma method showed good corelation with the results of experiments on the short term behaviour of *STC* beams, it was concluded that such elastic methods which incorporate slip at the steel-timber interface (partial shear interaction) could predict the behaviour of *STC* beams with sufficient accuracy. The main aspect to be considered while designing *STC* beams for partial shear interaction is the shear connector stiffness. The elastic methods required that the steel beam be undeformed at the onset of composite action. It was decided to precamber the steel beams to negate the deflections due to the dead load of the system. This also provided an efficient way to incorporate the creep effects of timber.

For the purpose of reusability, all design was made within the elastic limit, as mentioned in Chapter 5. However, the plastic bending resistance of the *STC* was calculated, so that a comparison could be made with steel – concrete. Based on the assumption of plastic stress distribution, to calculate the plastic bending moment of resistance in *ULS*, the Eurocode method for steel – concrete composite structures was applied. This was based on the fact that the experimentally obtained values of ultimate strains (from [24]) in *STC* beams showed that timber had yielded in compression, and steel in tension, thus providing sufficient ductility.

For specific case of ***DA1_STC***, before looking into the case of partial shear interaction, the case of full shear interaction was considered, to have an overview of the maximum gains that can be achieved. From calculation, it was observed the maximum gains was 37.8% for bending stiffness, and 13.3% and 18.2% for the elastic and plastic bending moment resistance respectively, compared to that of steel alone. These figures were much lesser than what can gained by composite action in steel – concrete, as given in Table 6.8. The total savings in steel per unit area by using composite action for *STC* was only $1.8\ kg/m^2$. Thus, it was concluded that the use of composite action in *STC* cannot be justified because of the small amounts of gains compared to the large costs of added shear connectors required. Comparison of the transverse mechanical properties of *LFE* slabs (chosen *STC*) and *CLT* slabs (timber slab with better bidirectional bending properties) showed that the former had better performance (due to the assumption of reoriented timber flanges). Thus, the calculations on the chosen *STC* floor system can be used to draw the same conclusion about composite action in steel-timber for all such combinations of large span steel beams and timber slabs, which are currently available in the European market. The experiments conducted for composite action in steel-timber, which have been cited in this thesis, and have yielded positive results, was mainly for the combination of large timber sections with smaller cold formed C/Z sections. This points towards the possibilities for developing new hybrid steel-timber slabs. In this thesis, it is concluded that such possibilities do not exist for combinations of large span steel and timber elements.

Finally, using the aspects that were addressed, for the case of full shear interaction, design recommendations were made for *STC* by considering generic Dutch office layouts. The sensitivity of composite action with respect to the spans of steel/timber elements (which correspond to the sizes of the respective sections) was looked into, and it was found that the results did not alter much. The benefits of composite were still too less, to be of any practical significance, for most of the combinations considered. Thus, it is concluded in this research that the use of composite action in steel – timber, between typical slab – beam configurations could not lead to any justifiable benefits.

7. Life Cycle Analysis

The Life Cycle Analysis (*LCA)* is a method that has been developed over several decades. Developed by the Institute of Environmental Sciences (CML) of Leiden University, Netherlands [81], it can be used to quantify different aspects of environmental performance i.e., the impact of a specific product or process on the environment. In this chapter, an *LCA* is conducted on the design alternatives considered in the case study in Chapter 5. This is done to gain insight into the environmental performance of *STC* floor systems, and make comparisons to that of conventional floor systems used in the Netherlands. The rules for *LCA* were formally drafted as European Standards ISO 14040 and ISO 14044, which were adapted specifically for construction works as EN 15978 [66] (assessment of buildings) and EN15804 [96,97] (environment product declarations) [81]. In this thesis, the fast – track method of *LCA* is used, where the information on the environment impacts of different materials/products, which have already been carried out by manufacturers, are aggregated to obtain the final values for the different floor systems. As opposed to this, the classical *LCA* method would require determining the environmental impact of all constituent materials (steel, timber, concrete, e.t.c) from scratch. Thus, the fast – track approach is built on the results *LCAs* conducted on each of the constituent materials, and is more suited with respect to the goals of this thesis. A *LCA* can be divided into specific stages of the life cycle. For constuction processes, the typical life cycle stages are given below in Table 7.1.

Table 7.1: Life Cycle Stages of Constructions works [81].

Modules	Life Cycle Stages		Desciption
Module A	***Prodcution Stage***	***A1***	*Supply of raw materials*
		A2	*Transport to production site*
		A3	*Production processes*
	Construction Stage	***A4***	*Transport to construction site*
		A5	*Installation/constuction process*
Module B	***Use Stage***	***B1***	*Use*
		B2	*Maintenance*
		B3	*Repair*
		B4	*Refurnishment*
		B5	*Replacement*
Module C	***End of Life Stage***	***C1***	*Demolition/dismantling*
		C2	*Transport to reuse/ recycle/ landfill/ incineration facility*
		C3	*Waste processing*
		C4	*Disposal*
Module D	***Benefits beyond End of Life***	***D***	*Reuse, Recycling, and Recovery (energy)*

The procedure for performing an *LCA* requires 4 steps, and the sections in this chapter have been designed to deal with each of these aspects:

- **Defining the Goal and Scope of *LCA***: This refers to the extent to which the *LCA* is conducted, specifying the life cycle stages and the intended application. This is given in Section 7.1.
- **Life Cycle Inventory Analysis:** In this step, the inputs for the *LCA* are identified i.e., the quantity of materials. The main calculations for these have already been done in Chapter 5. In Section 7.2, this is summarized. The data on the environment impacts of different materials is also collected. This is done with the help of environment product declarations *(EPDs)*, and is given in Section 7.3.
- **Environment Impact Assessment:** The collected data on environment impact can be aggregated using different methods. The different methods used are described in Section 7.3, out of a specific method is adopted.
- **Interpretation of the Results:** Finally, the results of the *LCA* are obtained. These are classified by life cycle stages and by elements, to identify the environment hotspots in the complete process. The total environment impact of the design alterantives are obtained and compared here. All of this is given in Section 7.4.

7.1 Goal and Scope of LCA

7.1.1 Goal of LCA

The most important goal of the *LCA* is to compare the environmental performance of the *STC* floor systems. Thus, for this *LCA,* based on the case study of Bouwdeel D in Chapter 5, 3 floors systems have been considered, of which the *STC* floors have been designed with and without composite action. This is summarized in Table 7.2. All the floor systems considered are assumed to be demountable, as they are executed with bolted connections.

Table 7.2: Summary of Design Alternatives from Chapter 5.

Design Alternatives		
1	***DA1_STC***	*Steel Timber Floor Systems. Cross beams with Composite Action.*
2	***DA1_STC***	*Steel Timber Floor Systems. Cross beams without Composite Action.*
3	***DA2_HCS***	*Floor system with Hollow Core Slabs.*
4	***DA3_CS***	*Floor system with Composite Slabs. Cross beams with Composite Action.*

7.1.2 Scope of LCA: System Boundaries

Keeping in line with the goal of the *LCA* (and this thesis), the system boundaries are formulated with the notion that only the life cycle stages that show significant differences between the floor systems being compared have to be considered. Thus, the use stage (Module B) is completely left out for the *LCA.* Apart from the main constraint that data on Module B is not readily available, the environment impact of the use stage of structural elements is negligible. Most of the components are designed for a technical service life of 50-100 years, which means that no repairs or maintenance would be required during this period. The main environmental impact for the use stage of buildings arises from the operational expenses of the client such as energy demands for heating and electricity, use of water, etc. Apart from the difference in insulation for the different building materials considered (timber, steel and concrete), the quantification of whose impacts is beyond the scope of this thesis, all the other aspects related to these operational expenses would be the same for all design alternatives. All the other stages of the *LCA* are different for different materials (different floor systems), and must be considered, for effective comparison.

According to the *MPG* method (explained in Section 2.4), it is required that the environmental data of a product has at least the production stage of the material declared (life cycle stages A1-A3). This only makes it possible to conduct a 'cradle to gate', where the End of Life (*EoL)* scenario and the benefits/loads of reusing cannot be included. However, in this thesis, as we will see in Section 7.3, this method is not used due to the 'blackbox' nature of the data in the Dutch *NMD.* Rather, a more investigative approach is used with the help of the data in standard environmental product declarations *(EPDs)*, which have clearly explained the underlying assumptions for the data declared.

With respect to the *EPDs* currently available for different materials, there is a clear demarcation with respect to the standards (Eurocode or ISO) on which it is based on. With respect to sustainability in civil engineering, in Europe, there are 2 main standards: EN 15978 [66] which provides rules for the assessment of new and existing buildings, and EN 15804 [96,97] which focuses on the product category rules for the development of *EPDs* of construction products. EN 15804 has had 2 major revisions (amendments) since its conception in 2012: Amendment A1 in 2013 (EN 15804+A1 [96]) and Amendment A2 in 2019 (EN15804+A2 [97]). Based on this, the *EPDs* currently available can be classified as two (hereafter referred to as "***A1 EPDs***" and "***A2 EPDs***" respectively). The main differences between the two are discussed further in Section 7.3.1. ***A1 EPDs*** need only declare the production stage data, whereas ***A2 EPDs*** need to declare the production stage, end of life stage and a detailed description of the benefits/loads of the material beyond its end of life. This would make a proper 'cradle to grave' *LCA* possible, as is fitting for demountable construction. However, the requirements of EN 15804+A2 becomes mandatory only by July 2022, and thus, we are now in a transition period, where manufacturers have the freedom to declare ***A1 EPDs*** or ***A2 EPDs****,* based on their own requirement. To summarise, not all materials required for analysis would have *EPDs* with data on the *EoL* scenarios. Thus, the *LCA* in this thesis is inclusive of the construction stage and end of life stage, including the benefits and loads beyond the technical service life of the products (Modules A, C and D), while being constrained to the availability of data for different materials, for different stages of the life cycle.

7.1.3 Reference Service Life

The reference service life of the for the *LCA* is 100 years and is the same as that for which the different floor systems in the case study have been designed for. Among the different structural elements used, steel and concrete products are conventionally guaranteed a much larger technical service life (up to 150 years for bridges, etc.). Even though timber products are specified with a technical service life of only 50 years in most European Technical Assessment documents [5-7], they can be used for longer periods (up to 100 years) when not subjected to harsh environment conditions (service class 1 and 2), such as in the case of indoor floors. For further reuse, the structural members will have to be subject inspection of damage and graded again. This limitation is only in the case of timber, and does not apply for steel and concrete, as discussed in Section 2.3. In the initial design stage (such as for this thesis), the only change that can be done to minimise damage, is to provide demountable connections (which is provided for all design variants). Thus, the structural elements can be assumed to be reusable at the end of the 100 years reference period of the *LCA.* Wherever available, the data for benefits/loads beyond the system boundaries is taken as that for the End of Life (*EoL)* scenario for reuse.

7.1.4 Modelling of Reuse

As mentioned in Section 2.3, in this thesis, the type of reuse considered is reuse by reorientation, wherein the structural elements are used for an extended service life of 100 years. The elements are provided with demountable connections so that they can be dismantled and reused beyond the initial 100 – year service life. Modelling this in the *LCA* requires that the *EPD* related to the specific product has declared its reuse *EoL* scenario. However, this is rarely the case. As we will see in Section 7.4, reuse *EoL* scenario is mostly declared for steel products. Other products have declared other *EoL* scenarios such as incineration (for timber products), recycling and landfill. Thus, due to the limitation of availability of data for the *EoL* reuse scenario, the calculations in this thesis are made with what is available. This approach is sufficient as the primary reuse of the building (in reuse by reorientation) is done within its initial service life (i.e., within the 100 – year reference period), as the design is meant to cater to the needs of differing clients. The benefits of the structural elements due to further reuse is what could have been credited in this *LCA,* had reuse *EoL* scenario been declared. The *EoL* scenarios adopted for different materials will be discussed in Section 7.4.

Thus, the approach for including reuse by reorientation in the *LCA* is simple and direct, and only includes adding the benefits/loads of the *EoL* scenario (Module D), along with Modules A and C, to obtain the total value. For considering reuse by relocation, the only difference is that aspects related to reuse such as stages A4, A5, (transport to new construction site and assembly on site), and stages C1, C2 (disassembly, transport to *EoL* location) would have to considered twice. The summary of how both types of reuse can be modelled in the *LCA* is given below in Figure 7.1.

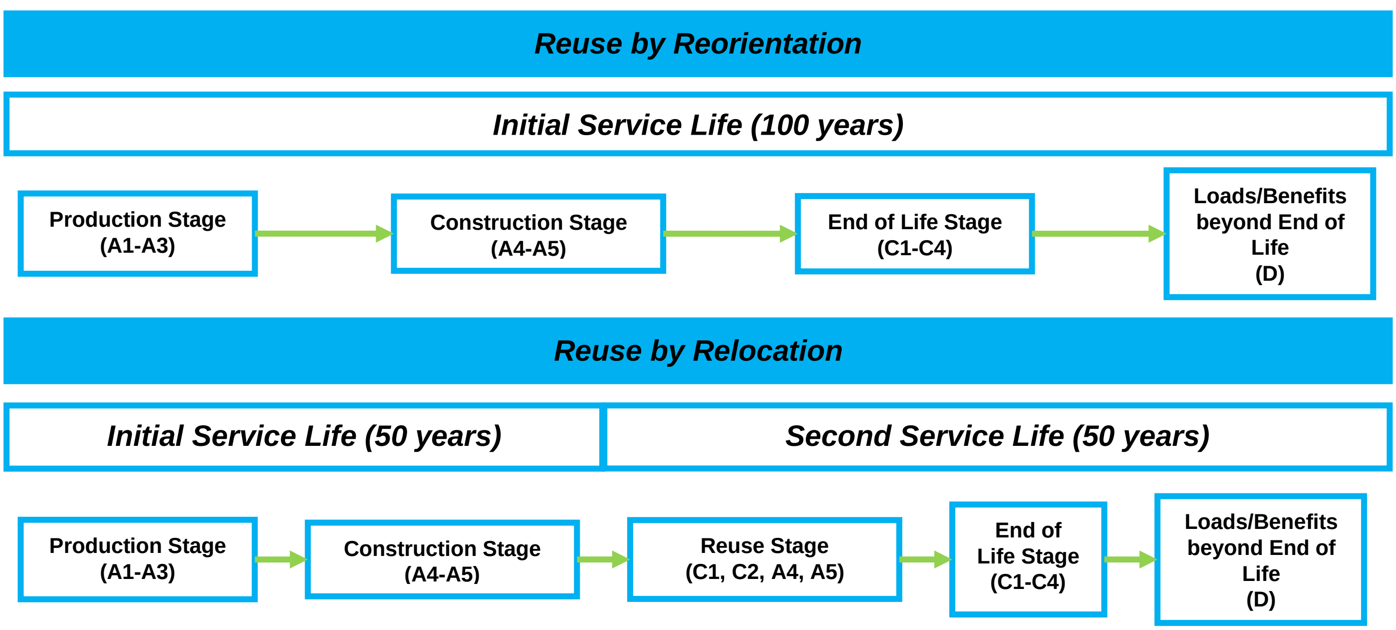


Figure 7.1: Modelling Reuse in LCA.

7.1.5 Functional Equivalent of LCA

The functional equivalent is a representation of the required technical characteristics and functionalities of the building [66] i.e., it refers to aspects (in this case the structural elements) of the building that are chosen to represent the whole building for the *LCA.* In all respects, 'Functional Equivalent' is synonymous with the term 'Functional Unit' used in Chapter 5. The functional equivalent of the *LCA* includes all the structural elements associated with the floor systems: Slabs, Secondary and Primary Beams, and Columns. It was due to this that the dimensioning of structural elements also was limited to these specific elements. A major point of difference of *STCs* with respect to *HCS* is that the latter has been dimensioned for a fire safe time of 90 minutes. Also, concrete slabs do not require any additional layers to meet the requirement of the Dutch Building decree [1]. The same holds for composite slabs. Thus, for uniformity is comparison, the *STCs* must also meet the same requirements, which calls for additional layers. In Chapter 4, it has been calculated that an additional 40 mm gypsum plasterboard layer is sufficient for a fire safe time of 90 minutes. The layers required for sound insulation is also given in Appendix A.3. Thus, the functional equivalent can be defined as follows:

1 floor unit of Bouwdeel D with outer dimensions 21.2 x 10.9 m, including slabs and all beams and columns. The floor must provide a fire safe time of 90 minutes, and should comply with requirements of the Dutch building decree (R_w > 54 dB and $L_{n,w}$ < 52 dB) [1] *for sound insulation. The total service life of the structure is 100 years.*

Foundations are required for calculations based on the *MPG* method. However, as explained in Section 7.3, this method is not considered, and hence foundations are not included in this *LCA*. It should also be noted that since the reference period of the *LCA* is the same (100 years) for all design alternatives compared, the final results will be reported as the total value for 100 years, and is not comparable to the results of the *MPG* method, which requires the final result in $\epsilon/year/m^2$ (i.e., environmental impact per year).

7.2 Quantification of Materials

The dimensions of the structural elements for the 4 design alternatives have been determined in Chapter 5. This is used as input to calculate the quantity of materials required in the *LCA.* The data in *EPDs* is given in different units, based on the type of material. For example, the data for timber is declared for 1m^3 of material, whereas for steel, it is for 1 tonne. For uniformity, all data is converted to the equivalent value for 1 kg of the material, using appropriate conversion factors. The total environmental impact is calculated per Gross Floor Area *(GFA).* For Bouwdeel D, *GFA* = 231.08 m^2. The total quantity of materials required is determined for 1 floor (including slabs, beams and columns), and divided by the *GFA*. Table 7.3 below shows the quantity of materials used in $\boldsymbol{kg/m^2}$. The calculations for the same is given in Table E.1 in Appendix E.1.

Table 7.3: Quantity of Materials for LCA. a) For STC Design Alternatives (DA1_STC).

DA1_STC				
Materials			**Quantity $[kg/m^2]$**	
			With Composite Action	**No Composite Action**
Floor	***Slab***	***Timber***	*36.73*	*36.73*
	Sound	***Chipboard***	*19.02*	*19.02*
	Insulation	***Mineral Wool***	*4.8*	*4.8*
	Fire Protection	***Gypsum***	*27.2*	*27.2*
Beams		***Steel***	*26.3*	*28.1*
Columns		***Steel***	*3.5*	*3.5*

b) For HCS Design Alternative (DA2_HCS) and

DA2_HCS			
Materials			***Quantity $[kg/m^2]$***
Floor	***Slab***	***HCS260***	***383.3***
	Floor Finish	***Concrete***	***71.4***
Beams		***Steel***	***43.6***
Columns		***Steel***	***5.8***

c) Composite Slab Design Alternative (DA3_CS)

DA3_CS			
Materials			***Quantity $[kg/m^2]$***
Floor	***Composite Deck***	***ComFlor60***	***11.2***
		Concrete	***240.6***
		Rebars	***1.1***
Beams		***Steel***	***52.1***
Columns		***Steel***	***5.1***

7.3 Calculation of Environment Impact

The next step in the *LCA* calculation is to translate the amount of resources (construction materials) used into a metric of environment impact. The environment impact of different construction materials is divided into several impact categories i.e., the environment impact categories.

As mentioned earlier, in this thesis, the 'fast track' method of *LCA* is used, where the final results of the case study design alternatives are built on the *LCA* results on the individual components. Thus, it is required to obtain the environment impact data collected by different organizations and consultant firms. In the Netherlands, this data is obtained from the from the Dutch National Milieu Database *(NMD)*. Different methods for the assessment of environment impact use different environment impact categories. In the Netherlands, the most commonly used method of environment impact assessment is the ***CML-2 baseline method***, developed by the Institute of Environment Sciences (CML) at Leiden University, in 2001. According to this method, at least 11 impact categories must be included in the calculation of environment impact of a building. These are given in Table 7.4, and this is also part of the requirements of *MPG* assessment method (explained in Section 2.4). In this method, the total environment impact is aggregated into a final 'single score', known as the 'shadow price' for easier comparison and interpretation. As the name suggests, the term 'shadow price' refers to the internalisation of the product's environment impact [81], calculated as the aggregate of environment impact, using specific conversion factors (Table 7.4, in Euros per Unit equivalents), and reflects the costs due to society owing to pollution i.e., the costs incurred for removing the amount of pollutants released due to construction activity. The shadow costs calculated using the ***CML-2 baseline method*** requires at least 11 environment impact categories, which are available in the Dutch *NMD,* and the final output is in the form of Euros per unit quantity of the material.

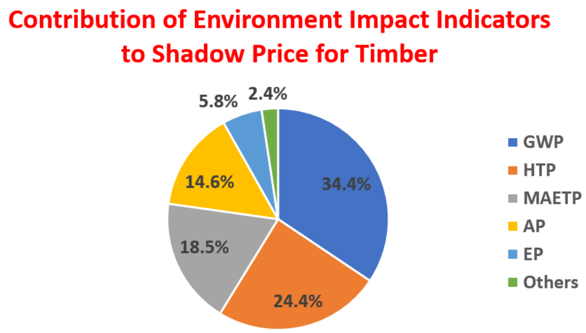
Table 7.4: Shadow Costs of Environment Impact Indicators, adapted from [81].

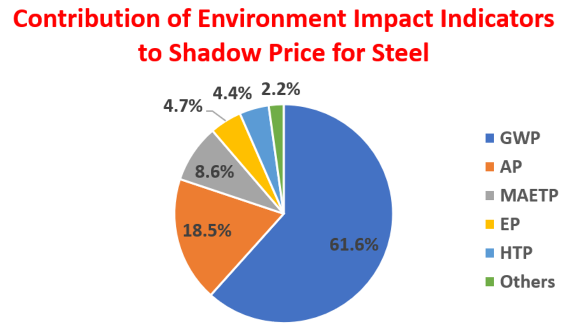
Environment Impact Category	Abbreviation	Unit Equivalent $[UE]$	Shadow Costs $[€/UE]$	Remark
Global Warming Potential	***GWP***	$kg\ CO_2$	*0.05*	*Net effect of emissions due to human activities on heat absorption capacity of atmosphere*
Ozone Layer Depletion	***ODP***	$kg\ CFC-11$	*30*	*Destruction of ozone layer in stratosphere, which protects us from UV radiations*
Acidification Potential	***AP***	$kg\ SO_2$	*4*	*Associated with the harmful effects of production of acids when mixed with rainwater*
Eutrophication Potential	***EP***	$kg\ PO_4{}^{3-}$	*9*	*Seepage of added agricultural nutrients (nitrogen, phosphorus) into groundwater, and excess growth of algae.*
Photochemical Oxidation Potential	***POCP***	$kg\ C_2H_4$	*2*	*Chemically reactive air – borne pollutants leading to oxidation reactions (such as production of ozone in lower atmosphere)*
Abiotic Depletion Potential (Non-fuel)	***ADPE***	$kg\ Sb$	*0.16*	*Depletion of minerals, which are non – renewable.*
Abiotic Depletion Potential (Fuel)	***ADPF***	$*\ kg\ Sb$	*0.16*	*Depletion of fossil fuels, which are non – renewable.*
Human Toxicity Potential	***HTP***	$kg\ 1{,}4\ -\ C_6H_6Cl_2$	*0.09*	*Refers to harmful chemicals which produce adverse effects on humans and freshwater, marine and terrestrial organism respectively.*
Freshwater Aquatic Eco – Toxicity Potential	***FAETP***	$kg\ 1{,}4\ -\ C_6H_6Cl_2$	*0.03*	
Marine Aquatic Eco – Toxicity Potential	***MAETP***	$kg\ 1{,}4\ -\ C_6H_6Cl_2$	*0.0001*	
Terrestrial Eco – Toxicity Potential	***TETP***	$kg\ 1{,}4\ -\ C_6H_6Cl_2$	*0.06*	

*Conversion factor: $4.81E-4\ kg\ Sb/MJ$ when ADPF is declared in MJ

However, in other countries (European and other), such *LCA* results are documented by individual program operators, as Environment Product Declarations (*EPDs)*, created according to the guidelines in EN15978 [66] and EN 15804 [96,97]. ***A1 EPDs*** need to declare a minimum of 7 impact categories. This excludes 4 extra impact categories declared in the *NMD* (HTP, FAETP, MAETP and TTP, from Table 7.4). Thus, one could say that the information in *EPDs* is less extensive than that of the *NMD*, in terms of the number of environment impact categories. However, in all other aspects, the converse holds true. The *NMD* follows a 'blackbox' approach where the underlying assumptions to obtain the given values are not mentioned [67]. There is also another problem that the data for specific timber products are not avialable, as most of the structural timber is imported in the Netherlands. This means that the data required for the *LCA* calculations can sometimes only be obtained using *EPDs.* Thus, in this thesis, it decided to choose the data from the *EPDs* over the *NMD* data.

As the *EPDs* are used, and these do not have the data for the additional 4 environment impact categories (related to the 4 toxicity potentials) required for calculating the shadow price, the Dutch methodology cannot be used i.e., the shadow price calculated without considering all 11 environment impact categories does not bear any significance. Thus, in this thesis, it is decided to consider only one of the main environment impact categories declared in the *EPDs* – Global Warming Potential (*GWP*). This is because *GWP* is the indicator which has the most significance. Figure 7.2 below shows the distribution of the contribution of different environmental impact indicators towards the shadow price, for the main construction materials – timber, steel and concrete (with a contribution to the total shadow price ranging from 35% up to 62%). The data for the same obtained from the Dutch *NMD* for Module A only. These values obtained from the Dutch *NMD* are for representation only and will not be used further in this thesis. For the specific scenario chosen, it can be observed that for timber, the 2nd highest contributor is *GWP* for timber and *MAETP* for concrete, and this supports the assumption in this thesis that the shadow price calculated without the use of the toxicity potentials does hold any significance.





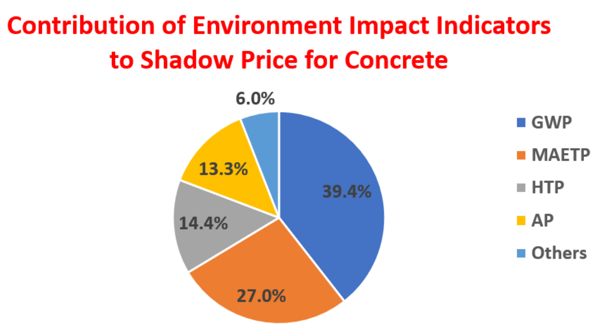


Figure 7.2: Contribution of different environment impact indicators towards the shadow price for Module A, for Timber, Steel and Concrete. Data obtained from Dutch NMD.

7.3.1 EN 15804: Amendment A1 vs A2

As mentioned earlier, *EN 15804 – Sustainability of Construction works – Environment product declarations – Core rules for the product category of construction products* is the European standard for creating *EPDs.* The first amendment EN 15804+A1 [96] in 2013 made it compulsory to apply to common factors for aggregating the different environment impact categories, similar to the CML-2 baseline method. The international standard ISO 21930 is a mirror image of EN 15804+A1 for creating *EPDs.* The latest amendment EN 15804+A2 [97] was added in 2019 and does not comply with ISO21930. This amendment, steers toward a new method, known as the Product Environment Footprint method *(PEF)*, and aims to bring uniformity for processing environmental data within Europe. As a limitation, this brings about potential trade barriers between Europe and the rest of the world. It is expected that all organisations issue the new *EPDs* conforming to EN 15804+A2 by July 2022 at the latest, although currently it is allowed to report the data in the form of *A1 EPDs* also. Thus, in this period of transition from *A1 EPDs* to *A2 EPDs*, users of environmental data are left with the choice to use either form of *EPDs*, and so it is essential to differentiate between the two.

The first point of difference between the two forms of *EPDs* is in the declaration of different stages (modules) of the *LCA*, specifically which modules are to be declared as a mandate. The new amendment makes it compulsory to declare Module A, C and D, whereas only Module A (A1-3) was compulsory according to EN 15804+A1. This means that more complex calculations can be made on the *EoL* scenario, and the loads/benefits beyond the system boundaries.

Using any method of analysis for the environment impact data requires additional information, for the assumptions of *EoL* and material flow, all of which is required to be standardised. This is done with the help of Product Category Rules (*PCR*). Providing the *PCR* for different construction products in one of the main objectives of the standard EN 15804. However, with the new amendment A2 to the EN 15804, it is inevitably required that the *EPD* program operators revise their *PCR* i.e., to use the new standard, it is required to formulate new *PCR,* which is based on EN 15804+A2. Ultimately, this should be in the form of revised/separate *PCRs* for different construction materials (steel, concrete, timber). EN 17662 [95] is the *PCR* for steel, iron and aluminium structural products. In this thesis, this standard has been referred to obtain additional information on steel products.

With respect to the core environment impact indicators, EN 15804+A1 required the reporting of at least the 7 core indicators (Table 7.4), whereas amendment A2 now requires reporting of 13 core indicators. These are given below in Table 7.5. The additional indicators are as follows: *GWP*, which initially reported as the total values (total *GWP*) has been split into 3 sub-categories related to biogenic carbon, and the impact of usage of fossil fuels and land use. *EP* also has been declared as 3 sub-categories pertaining to freshwater, marine and terrestrial separately. Finally depletion of water as a resource needs to be declared. Thus, the indicators of *A1 EPDs* cannot be directly compared with the indicators of *A2 EPDs*.

Table 7.5: Environment Impact Indicators according to EN 15804+A2.

Environment Impact Category		Unit
Global Warming Potential	**Fossil Fuels**	$kg\ CO_2$
	Biogenic Carbon	$kg\ CO_2$
	Land use and land transformation	$kg\ CO_2$
	Total	$kg\ CO_2$
Ozone Layer Depletion		$kg\ CFC-11$
Acidification Potential		$mol\ H^+$
Eutrophication Potential	**Aquatic Freshwater**	$kg\ {PO_4}^{3-}$
	Aquatic Marine	$kg\ N$
	Terrestrial	$mol\ N$
Photochemical Oxidation Potential		$kg\ NMVOC$
Abiotic Depletion Potential (Non-fuel)		$kg\ Sb$
Abiotic Depletion Potential (Fuel)		MJ
Water Use		m^3

The *GWP* declared in *A1 EPDs* is actually the total value, which comprises the effect of biogenic carbon captured during the production of timber i.e., for *A1 EPDs*, the positive effects of carbon storage during production was included in Module A, and the same value was subtracted in Module C to account for the CO_2 that was released, once these timber products are incinerated (or left to disintegrate in landfills). This is explained further in Section 7.3.3. Thus, the total value for cradle to gate analysis would accurately contain information on the storage and release of biogenic CO_2. Thus, to make a fair comparison of *GWP* between *A1 EPDs* and *A2 EPDs* ,the value of total *GWP* would have to be considered from the latter.

Apart from these major differences, EN 15804+A2 also makes it compulsory for program operators to declare their products in the International Reference Life Cycle Data System (ILCD) format, which is technically an XML file. This makes the transfer of data easier and more accessible for the users of environmental data.

7.3.2 Product Environment Footprint (PEF) Method

As mentioned earlier, EN 15804+A2 steers towards the *PEF* method. The main reason for this is to have a universal method for comparison of environmental impact in Europe, with the help of common method of assessment. The *PEF* method is based on the *LCA* standards (with different environment impact categories and life cycle stages), but with more specific requirements. This is to increase the reproducibility, comparability, consistency, and practicality of *EPDs.*

The *PEF* method was developed by the Joint Research Centre of the European Commission in 2013 and is a multi – criteria method of environment impact assessment that can be applied to a wide range of products. Apart of including the specific environment impacts or emissions of a product, this method also includes the flow of materials (outputs/inputs), makes a distinction between primary materials obtained from virgin materials and those obtained from recovery processes (recycling/reusing), and most importantly includes all the different *EoL* scenarios declared for the product [97]. Thus, it avoids the 'open nature' of including one (out of many) *EoL* scenario for a conventional *LCA*, by requiring that environment impacts of all *EoL* scenarios (recycling, reusing, energy recovery and landfill) be considered [127]. This is the reason that the *PEF* method could lead to harmonised method of measuring environment impact.

Although the *PEF* method is more accurate and is heading the trajectory of harmonised environment impact assessment for the future, it is left out of scope for this thesis. This is done mainly to keep the *LCA* aligned with the main goals of this thesis – the main goal is to compare different floor systems and provide design recommendations for the application of demountable floor system using *STC.* The expectation from an environment impact assessment on different floor systems, is to compare the benefits of *STC* floor systems over *HCS* and composite slabs. Thus, the methodology is limited to using an *LCA* based on total *GWP*, using data from *EPDs* of the considered construction products.

7.3.3 Carbon Sequestration in Timber Structures

Sustainable forestry is the practise of forest management in which timber harvesting is carried out in a controlled manner i.e., in a way that does not disrupt the biodiversity of the forest. Organisations such as the Forest Stewardship Council (FSC) and the Program for the Endorsement of Forest Certification Schemes (PEFC) promotes sustainable forestry by providing certificates/approvals for timber products obtained from such forests [3]. Being a carbon – based product, the production of timber structures store CO_2 (in the form of raw materials, although the production processes might cause release CO_2 and other harmful products), until it fulfils its technical service life. Thus, timber structures obtained from sustainable forestry can be considered 100% sustainable when the technical service life of such products exceeds the time for the same amount of trees to be regrown.

As the amount of CO_2 stored is released at the end of life of the product, the flow of the carbon is zero. For Life Cycle Calculations within the reference period of structures, the biogenic carbon content of timber structures (amount of CO_2 stored) is considered as follows: During the production stage (Module A), the positive effects of carbon storage within the timber products are added. During the stage of its end of life (Module C), these same benefits are reduced as the CO_2 stored in timber is sent back to the environment either in the form of harmful gases released during incineration or by biodegradation of the structures. This is the case in all *A1 EPDs*. For *A2 EPDs*, the data for biogenic carbon is given separately, along with the data for *GWP* related to combustion of fossil fuels, and land use and land transformation. Therefore, the total *GWP* in ***A2 EPDs*** is the sum of *GWP* related to biogenic carbon and the latter two aspects. Only this value is comparable to *GWP* declared in ***A1 EPDs****.* The amount of sequestered carbon i.e., the CO_2 stored in wood and wood – based products can be calculated using the formula given in EN 16449 [105] as follows:

$$p_{CO_2} = \frac{44 * c_f * \rho_w * V_w}{12 * (1 + \frac{w}{100})} \qquad \textbf{\textit{(Eq 6)}}$$

Where,

$$c_f \text{ is the carbon fraction of oven dry wood mass (default } = 0.5)$$

$$w \text{ is the moisture content of wood (default } = 12\,\%)$$

$$\rho_w \text{ is the density of wood at moisture content } w$$

$$V_w \text{ is the volume of wood considered for calculation}$$

To create a net positive effect on the environment by using timber structures, the technical service life needs to be increased as mentioned earlier. However, the constraint of using a bio – material such as timber is its strength loss over time. Typically, structural timber products are declared for a service life of 50 years [5-8], although they can be used for longer periods in service Classes 1 & 2, which are less susceptible to biological degradation. As the advantage of carbon sequestration cannot be included in the *LCAs* of individual buildings/projects, it offers less incentive to use timber structures as a substitute to conventional building materials. However, the effect of sequestered carbon can be considered in the *LCAs* involving the timber industry as whole, although this is left out of the scope of this thesis. Increasing the percentage market share of timber in the construction industry as whole can lead to long term benefits in terms of sustainability.

The above-mentioned case is applicable to *EPDs* which declare incineration or landfill as the *EoL* scenario. In the case where reuse of recycling is declared, it is assumed that the sequestered carbon is not released back into the atmosphere. However, it is very rare to find *EPDs* which declare this information, as the most commonly declared *EoL* scenario is incineration.

7.3.4 Effect of Actual Transport Distances

The data declared in *EPDs* is region specific. This is especially true about the data on transportation of raw materials/products. For obtaining accurate results, it is required to modify the values of environment impact related to transportation (Modules A4, C2). This issue is especially applicable to timber structures, as most of the timber products in the Netherlands are imported. Thus, it is decided to see the effects of transportation, by comparing the case of assumed distances in the *EPDs* with the actual distances.

To obtain the data modified to account for the actual transport distances, we can use the method of linear interpolation. This requires knowledge of the assumed distances (provided in the *EPDs*) and the actual distances. In the example considered in this thesis, the timber slabs (*LFE*) are assumed to be manufactured in a production centre (nearest centre) in Waldstaat, Switzerland [7]. The distance from the production centre to the construction site in Delft is 850 km. In the *LFE* considered in this thesis (explained in Section 7.3.5), the typical distances for Module A4 related to transport vary from 50 – 100 kms [100-105]. Thus, the modified values can be obtained based on this information. Similarly, for the distances related to the various *EoL* scenarios (Module C2), the Dutch Institute for Building Biology and Ecology (NIBE) [67] provides recommendations: 50 km for reuse and recycle, 100 km for landfill and 150 km for incineration. Thus, the modified data for this Module also can be obtained, when the information is available in the *EPDs*.

For steel products (rebars, steel sheets and profiles), when the assumed distances for Module A4 are not provided, EN 17662 [95] gives us the average distances that can be used within the Europe. As mentioned in Section 7.3.1, EN 17662 provides *PCR* specifically related to steel and aluminium structures. Since steel is readily available (in contrast to timber) in the Netherlands, the value for steel from local markets can be adopted, equal to 50 km. For Module C2, when the distances to the different end of life scenarios are not given, the value that can be used is 200 km (average distance from site of deconstruction to storage). Using these distances, the values of environmental impact for these Modules (A4, C2) can be corrected to represent realistic values. However, unlike timber, steel is readily available in the market in the Netherlands, most of which is produced locally. Thus, the difference between the assumed distances and the distances recommended in EN 17662 will not be significantly different, as in the case of timber (for Module

A4, the actual distances are up to 8 – 10 times more). Owing to this, it is decided not to modify the data related to transport of steel, as the change in the total values of environmental impact is expected to be negligible.

7.3.5 EPD Data Used in LCA

As concluded earlier in this section, it is opted to proceed ahead with the *LCA* using the total *GWP* data available from the *EPDs* of different construction products. Since there are many *EPDs* for the same product, a critical review of the data used should be done, for conducting an accurate *LCA*. As long as we are investigating timber structures in the Netherlands, this aspect is particularly relevant, as most of the timber products are imported from other European countries such as Sweden, Austria, Germany, etc.

The timber product used for design, Lignatur Surface Elements (*LFE*), consists of stressed – skin panels glued together i.e., each member (web/flange) consists of pieces of saw timber which are glued together to form a composite box-shaped structure. The specific *EPD* for *LFE* sections could not be obtained. Hence, the approach used in this thesis is to aggregate data for sawn timber from different producers, to obtain a representative value for the *LCA.* Compared to the other types of structural timber (such as *LVL,* Glulam), sawn timber was found to be the most representative one for *LFE,* although it underestimates the negative impacts by not considering the small amounts of adhesives used in the manufacturing process. Table 7.6 below shows the summary of the *EPDs* used for timber and steel.

Table 7.6: Summary of EPDs used for Construction Products (Timber and Steel).

Timber Products					
Producer	**EPD Operator**	**Modules Not Declared**	**EoL Scenario Declared**	**Region**	**Validity**
Binderholz [104]	*IBU*	*A4, C1, C4*	*Incineration*	*Germany*	*2019 – 2024*
Bergen Holme [100]	*Norwegian EPD Foundation*	*NA*	*Incineration*	*Norway*	*2021 – 2026*
Møbelindustrien [102]	*EPD Danmark*	*A5, C1, C4*	*Incineration and Recycling*	*Norway, Sweden and Finland*	*2020 – 2025*
Generic Finnish Sawn Timber [133]	*The Building Information Foundation RTS*	*A5*	*Incineration*	*Finland*	*2018 – 2023*
Swedish Wood [103]	*EPD International*	*A4, A5*	*Incineration*	*Sweden*	*2021 – 2026*
Egger [101]	*IBU*	*A4, A5*	*Incineration*	*Germany*	*2021 – 2026*
Stora Enso [134]	*EPD International*	*A4, A5*	*Reuse, Recycle, Incineration and Landfill*	*Austria, Sweden and Finland*	*2021 – 2026*

Steel Products					
Producer	**EPD Operator**	**Modules Not Declared**	**Applicable Products**	**Region**	**Validity**
ArcelorMittal [108]	*IBU*	*A4, A5, C1, C2, C4*	*Hot – rolled steel sections*	*Europe*	*2019 – 2024*
Bauforuhmstahl [109]	*IBU*	*A4, A5, C1, C2, C4*	*Hot – rolled steel sections*	*Europe*	*2018 – 2023*
DS Staalconstruktion A/S [135]	*EPD Danmark*	*NA*	*Hollow sections, beams and plates*	*Denmark*	*2021 – 2026*
BE Group Sverige AB [136]	*EPD International*	*A5*	*Hot – rolled steel beams.*	*Sweden*	*2021 – 2026*
Give Steel [106]	*EPD Dannmark*	*NA*	*Galvanised steel beams and plates.*	*Denmark*	*2020 – 2025*
	***EPDs according to EN 15804+A1 for Timber and Steel. All other EPDs are according to EN 15804+A1.**				

The timber *EPDs* chosen for comparison and analysis are such that they incorporate the large variation in the data with respect to the geographical scope of the *EPDs.* With respect to this aspect, *EPDs* from Sweden, Germany, Finland, Norway and Austria have been considered, which are the major producers of structural timber in Europe. The next aspect is regarding the *EoL* scenario for timber. As can be observed in Table 7.6, the most commonly declared *EoL* scenario for timber is incineration, wherein the timber products undergo combustion to produce thermal energy, which in turn is converted into electricity. A few *EPDs* have declared other *EoL* scenarios such as recycling, reusing and landfill. Thus, an accurate analysis requires us to assign the appropriate *EoL* scenario, and this is elaborated in Section 7.4. Finally, another aspect of concern is the type of *EPDs:* the total dataset is a combination of 4 ***A1 EPDs*** and 3 ***A2 EPDs***, although it is ensured that all *EPDs* considered are well within their period of validity. As we discussed earlier, the two types of *EPDs* cannot be directly compared with each other. However, this issue is resolved by the fact we are only considering *GWP* for comparison i.e., we are comparing *GWP* data of ***A1 EPDs*** with the total *GWP* data of ***A2 EPDs****,* (the issue of biogenic carbon, discussed in Section 7.3.3, is also resolved this way). With the help of these 7 timber *EPDs*, the idea is to adopt a representative value for accurate comparison.

For *STC* floors, apart from timber, the other main building material is steel. Thus, a similar approach is used here also, and the 5 steel *EPDs* considered are given in Table 7.6. For steel *EPDs*, the *EoL* scenario considered is a mixture of reuse and recycling (with varying percentages for each according to the region). A small percentage of steel (approximately 1%) is sent for landfill. The main difference would be in the production process for manufacturing these steel elements. Since we are dealing with large structural steel sections, the use of *EPDs* narrowed down to only hot – rolled steel products. Finally, as in the case of timber, both types of *EPDs* are considered in this analysis. This has a negligible effect since we are comparing only total *GWP.* There is no biogenic carbon involved for steel products, and the analysis done is similar to that for timber.

For the other design variants, the main construction materials are ready-mix concrete and precast hollow core slabs. A similar approach as in the case of steel and timber is done, where multiple *EPDs* are compared to get a representative value. The different floor systems also require other materials such as gypsum, rebar, etc. These are required in small quantities, and have very littile influence on the total values. Thus, a single *EPD* for each material, the details of which is given in Appendix E.2.1 in Table E.4, E.5 and E.6. The *GWP* data for all the *EPDs* considered is given in Appendix E.2.1.

7.4 Results of LCA

Figure 7.3 below shows the comparison of the total *GWP* value, obtained using the relevant *EPDs* for timber. All values are converted into $CO_2\ equivalents/kg$. The values are displayed separately for the production stage (Module A), the different *EoL* scenarios (Modules C+D), and the total values for different *EoL* scenarios (Modules A+C+D). For module A, the average value for the 7 *EPDs* considered is $-1.44\ CO_2\ eq/kg$ (with a variation of +-5.5%). This is mainly due to the difference is energy requirements, transport distances, etc during the production stage. All *EPDs,* assume storage of carbon during production, and hence the large negative values. At this point, it is essential to distinguish between the various *EoL* scenarios declared in the timber *EPDs.* The Stora Enso *EPD* is the only one which has declared 4 different *EoL* scenarios. For landfill *EoL* scenario, the implication on total *GWP* is that the CO_2 stored during production is released back into the atmosphere: thus, landfill gives the most onerous value of environment impact among all the *EoL* scenarios considered ($+2.32\ CO_2\ eq/kg$). As mentioned earlier, the most commonly declared *EoL* is incineration, where the timber is burnt to produce energy. Like in the case of landfill, here also, the stored carbon is released back into the atmosphere. The only difference is that the energy produced by combustion of timber is added as credit to the total value. Thus, incineration gives a slightly less impact than landfill (average value: $+1.02\ CO_2\ eq/kg$).

The other 2 *EoL* scenarios declared are reuse and recycle. For reuse, the structural elements are subjected to inspection and some degree of refurbishment, before they are sent to be used again. For recycling, the timber structures are downgraded to woodchips, and sent for use. In both these scenarios, the stored CO_2 remains as such, and is not released back into the atmosphere. Thus, these have the least environment impact (recycling: $-0.11\ CO_2\ eq/kg$). However, it should be noted that these *EoL* scenarios are rarely declared in the *EPDs.* Among the different timber *EPDs* considered, only 2 have declared recycle (Stora Enso and Mobilindustrien), and only 1 for reuse (Stora Enso).

Apart from this, another *EoL* scenario is considered, wherein the timber structures are assumed to be stored in an adequate place such that they are not allowed to disintegrate, as in the case of landfill i.e., the stored CO_2 is not released back into the atmosphere. This *EoL* scenario (hereafter referred to as “Storage”) can be compared to artificial carbon sinks, but without the infrastructure to absorb CO_2 from the atmosphere (as this is already done during the growth of trees). The benefit of this *EoL* scenario over reuse and recycle is that it is a conservative approach, as it does not require that the structures can be reused, nor does it require extra processing as in the case of recycling. In terms of analysis, the data can be obtained from existing *EPDs.* Stages C1 (dismantling the structure) and C2 (transport away from construction site) can be taken as such, from whichever *EoL* is declared. The value of stage C3 is zero (i.e., no CO_2 released). The other stages can be assumed to be zero (C4, D). The net result is a slightly more conservative value than that for reuse/recycling.

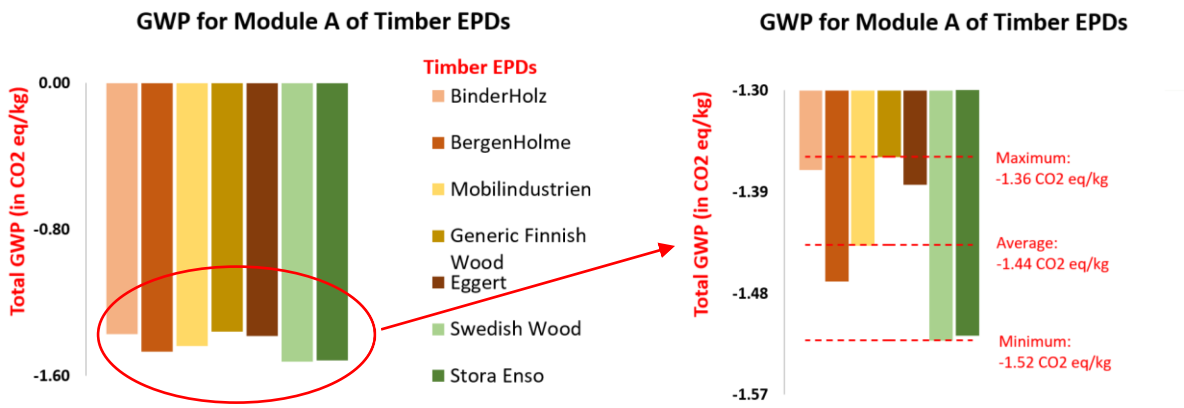


Figure 7.3a: Module A.

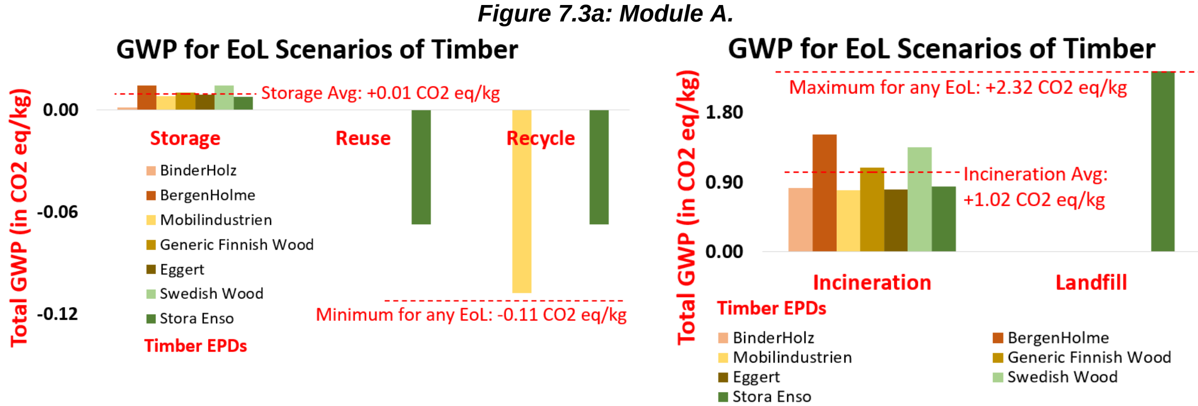


Figure 7.3b: End of Life Scenarios (Module C+D).

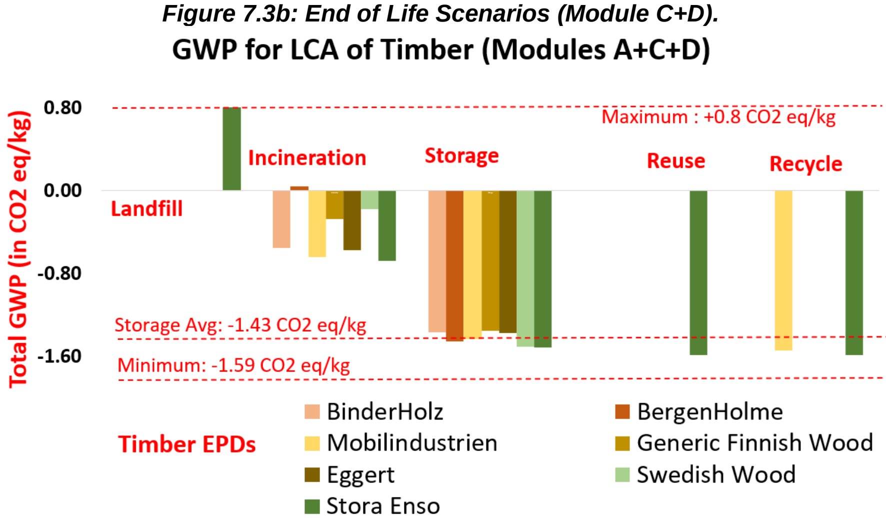


Figure 7.3c: Total Values. (Modules A+C+D).

Figure 7.3: Total GWP values from Timber EPDs.

Thus, in this thesis, for further analysis, storage *EoL* scenario is adopted, and the average value for all 7 *EPDs* is taken. The total value of *GWP* for the complete life cycle (Module A+C+D) is obtained as $-1.43\, CO_2\, eq/kg$. The maximum value (highest environmental impact) is $+0.8\, CO_2\, eq/kg$ (+155.9%) for landfill, and the lowest value is $-1.59\, CO_2\, eq/kg$ (-11.2%) for recycling. Thus, as we can see, there can be a large variation in the total value of *GWP* for timber based on how the *EoL* scenario is considered. The average value is taken in order to get a representative value, which is not dependant on the geographical scope of the *GWP,* as none of the timber products considered here are from the Netherlands.

In the case of steel, a similar approach is used, with the help of average values. Considering the *EoL* scenario is less complicated in the case of steel, as explained earlier in Section 7.3.5. This is mainly because only one *EoL* scenario is declared per *EPD.* This is given below in Figure 7.4. The average value of *GWP* is $+0.75\, CO_2\, eq/kg$, with a variation of -7.5% to +9.6%. The variation is much smaller than in the case of timber. These small variations arise due to differences in energy for production, transport distances, difference in shape of final products, the presence of any coating (galvanised steel), etc. This variation is for the total life cycle of steel. By splitting the *GWP* by different modules, it can be observed that while the total values show less variation, this is not the case with each module (except module C). The reason for this can be explained based on the amount of scrap (recycled) steel that is used as primary material in the production stage. The steel products which use a larger share of recycled steel during production have a lower environmental impact in the production stage (module). For the same product, when in comes to module D, the net credit applied for reusing/recycling steel is comparatively lower. This is because the net credit value assigned is based on the difference in the amount of steel recycled and the amount of recycled steel used in production. When the environmental impact for all the life cycle stages are combined, the net effect is that all steel products are comparable, with a very small value.

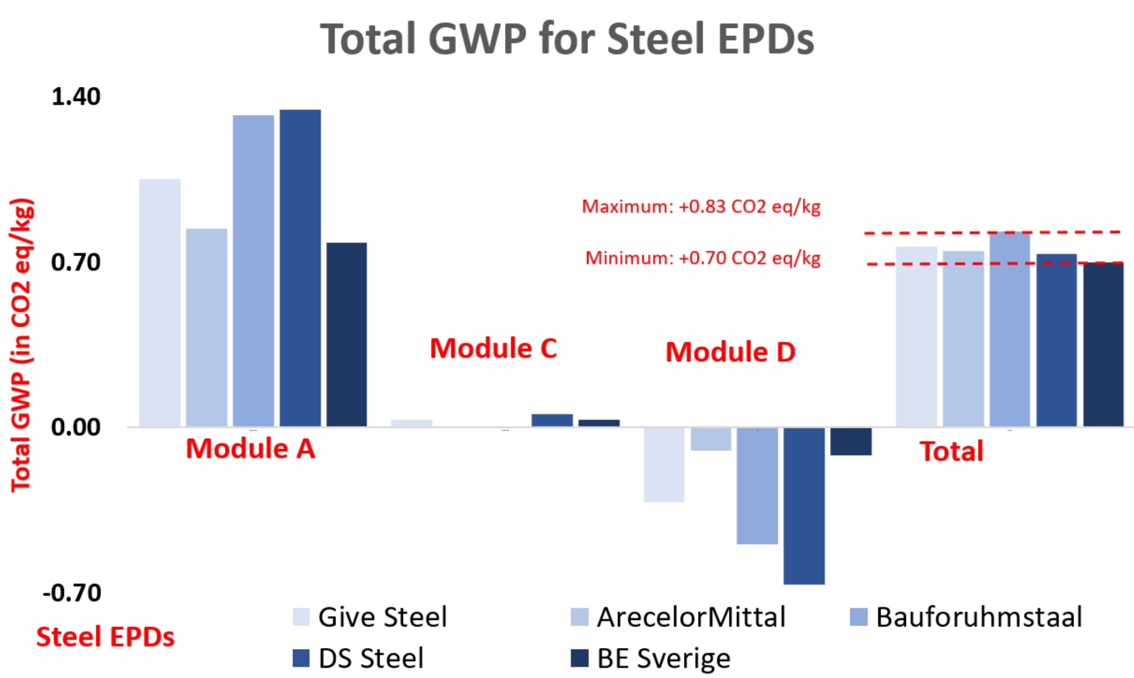


Figure 7.4: GWP Values from Steel EPDs.

It should be noted that all the values presented above are based on the actual transport distances (stages A4 and C2), for timber. These have been calculated based on the procedure given in Section 7.3.4. Further in this section, a comparison is made between the total values of *GWP* based on actual and assumed distances. This will give more light on the consequences of such type of analysis.

Steel and timber are the primary construction materials for ***DA1_STC***. For ***DA2_HCS*** and ***DA3_CS***, concrete and steel have the most influence. Since the analysis on steel and timber *EPDs* showed that there could be some variation based on which *EPD* is chosen, it is important to look at the variation in environment impact for concrete *EPDs* too. Thus, a representative value is chosen (average) from multiple *EPDs*, both for ready-mix concrete (used for composite slabs and floor finish) and for precast hollow core slabs. This is given in Appendix E.2.1. From the analysis, it was observed that the sensitivity to the *EPDs* was minimal for ready-mix concrete. For precast concrete hollow core slabs, some of amount of variation was there, though not as pronounced as in the case of timber.

As mentioned earlier in Section 7.3.5, all the other materials considered required specific *EPDs*, and this was what was done i.e., 1 *EPD* was used, specific to each material. The summary of total *GWP* values for all the materials used for analysis is given in Table E.7. For the main construction materials i.e., steel, timber and concrete (ready-mix and precast hollow core slabs), the maximum and minimum values were calculated for a sensitivity analysis.

7.4.1 STC Design Alternatives

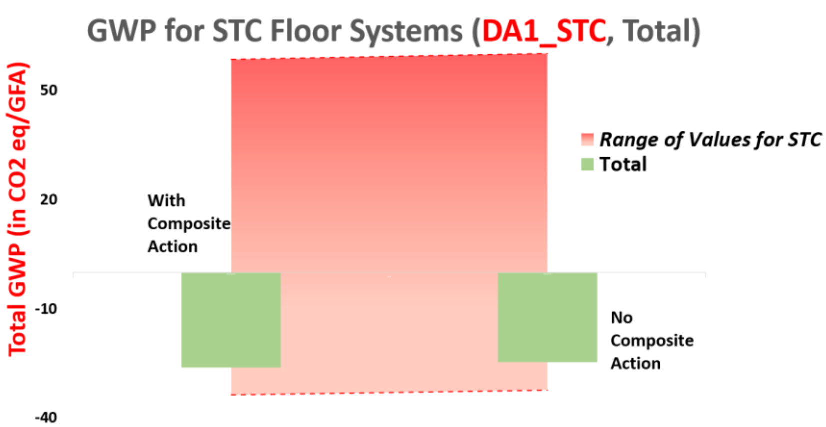


Figure 7.5: Total GWP for STC Design Variants.

The total *GWP* values have been obtained for the *STC* design variants (***DA1_STC***) and is shown above in Figure 7.5. This is done for the case of design with and without composite action in steel beam. All the values given below are in CO_2 $equivalents\ per\ GFA$. The first aspect to be noted here is that for the total value for the functional equivalent considered, the *STC* design variants give a negative value i.e., positive environment impact ($-26.1\ CO_2\ eq/m^2$ and $-24.6\ CO_2\ eq/m^2$ for the case with and without composite action respectively). The contribution of timber (with negative value) negates the effect of environment impact of other materials such as steel for the beams and columns, gypsum for fire protection, etc. This can be understood with the help of Figure 7.6 below, which shows the contribution to *GWP* by each element. Then timber slabs have the largest contribution, with a net negative value. This is primarily due the effect of CO_2 storage, based on the scenario of *EoL* considered. The data for the graphs is given in Appendix E.2.3.

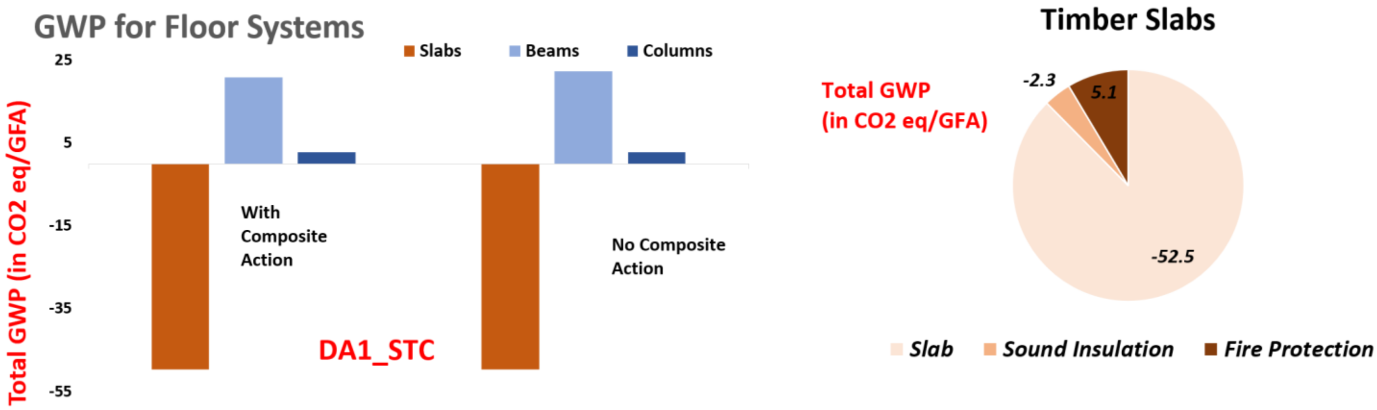


Figure 7.6: GWP for DA1_STC. Contribution by each element.

As we have seen in Chapter 5, the net savings in steel due to composite action in the cross beams is $1.8\ kg/m^2$ – a very small amount. This translates to a difference in *GWP* of $+1.42\ CO_2\ eq/m^2$. Comparing the values for whole floor system, this means an increase of +5.5%. Thus, based on the quantification of environment impact also, the net gains due to composite action is very small.

Also given in Figure 7.5 is a sensitivity analysis, based on different values of *GWP* as per the different steel and timber *EPDs.* Only the maximum and minimum values are considered: the upper limit is formed by using the maximum values for steel and timber, and the lower limit is formed by using the minimum values of steel and timber. As discussed earlier in this section, the variation with respect to different steel *EPDs* is negligible. The variation arises from different data on timber, and the reason for this is based on which *EoL* scenario we consider. The minimum value obtained ($-33.6\ CO_2\ eq/m^2$ and $-32.3 CO_2\ eq/m^2$ respectively) does vary significantly with respect to the base value used for analysis (average value of all *EPDs*). The reason being that storage *EoL* scenario does not give much difference from the minimum value used (Stora Enso *EPD*, recycling *EoL* scenario). The maximum value obtained ($+58.6\ CO_2\ eq/m^2$ and $+60.2\ CO_2\ eq/m^2$ respectively) shows a large difference (+324% and +344% respectively), and this is because landfill is considered as the *EoL* scenario, which excludes the benefits of carbon storage from the *LCA*. Thus, to conclude, the *EoL* scenario considered in analysis has a significant impact on *LCA* results for timber structures. We will see in Section 7.4.3 how this maximum value for *STC* floors compare with the other floor systems.

7.4.2 Effect of Actual Transport Distances

As mentioned in Section 7.3.4, the effects of considering the actual transportation distances for timber are discussed in this section. This analysis is done with storage as the *EoL* scenario. Figure 7.7 below shows the difference in *GWP* for timber material alone, based on the assumed and actual transport distances. For the total *GWP,* it can be observed that the net change is an increase by $+0.01\ CO_2\ eq/kg$ (+0.7%). For the total *GWP* of the functional equivalent, this translates to an increase of $+0.1\ CO_2\ eq/m^2$ (shown in Figure 7.8), which is a mere 0.4% increase. Thus, for analysis with respect to *GWP,* it can be observed that the transport distances do not have much effect on the total results. The reason for this lies in the effect of biogenic carbon. Figure 7.7 (Left) shows the total value including biogenic carbon. The image on the right shows the value of *GWP* excluding the biogenic carbon, also showing distribution of *GWP* with respect to the difference stages. In this case, the effect is more pronounced than before, but the increase is still only 6.7%. Thus, from this it can be concluded that the minimal effects of transport distances on the *LCA* is mostly due to the chosen environment impact indicator. Had any other impact indicator, or some other method of analysis been chosen, the effects would have been more prominent. All the data for this analysis is given in Appendix E.2.2. Since the details of transport (related to Module A4) is not declared in the *EPD* with information on landfill *EoL* scenario (Stora Enso), a similar analysis could not be conducted for this *EoL* scenario.

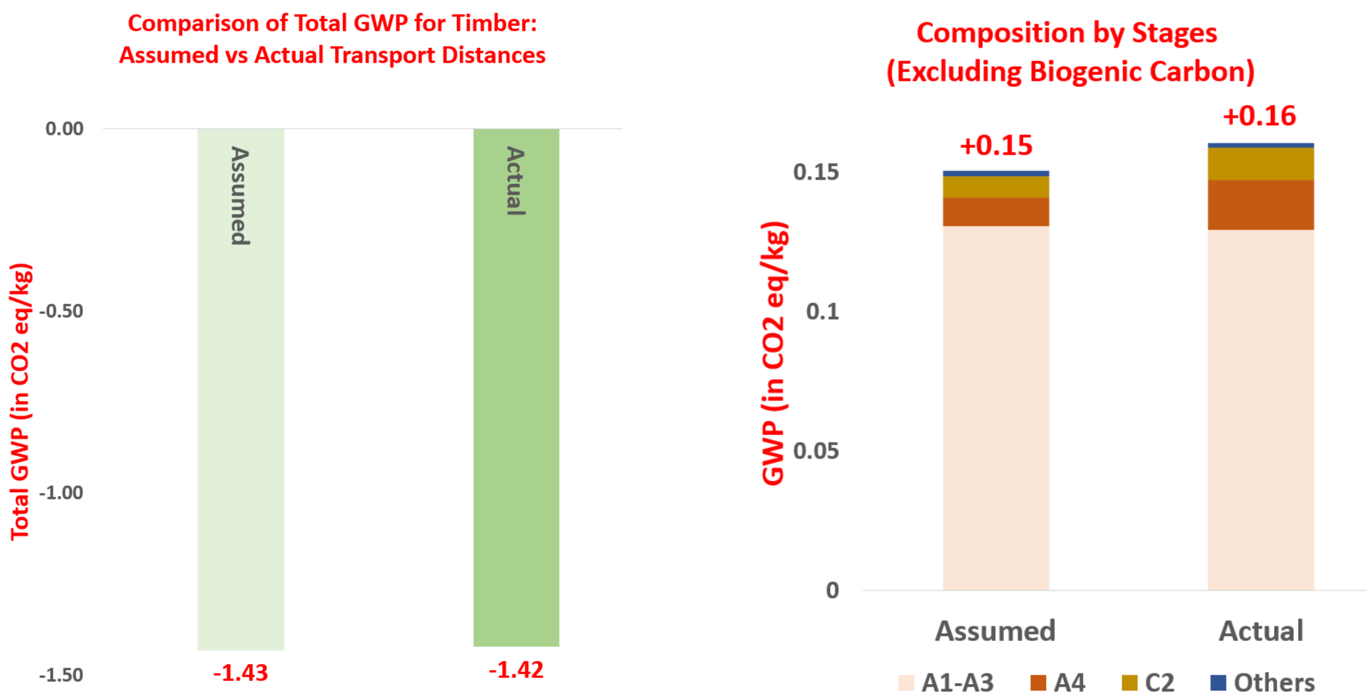


Figure 7.7: Effect on Transport Distances on GWP of Timber. (Left) Total GWP. (Right) Composition by stages, excluding Biogenic Carbon.

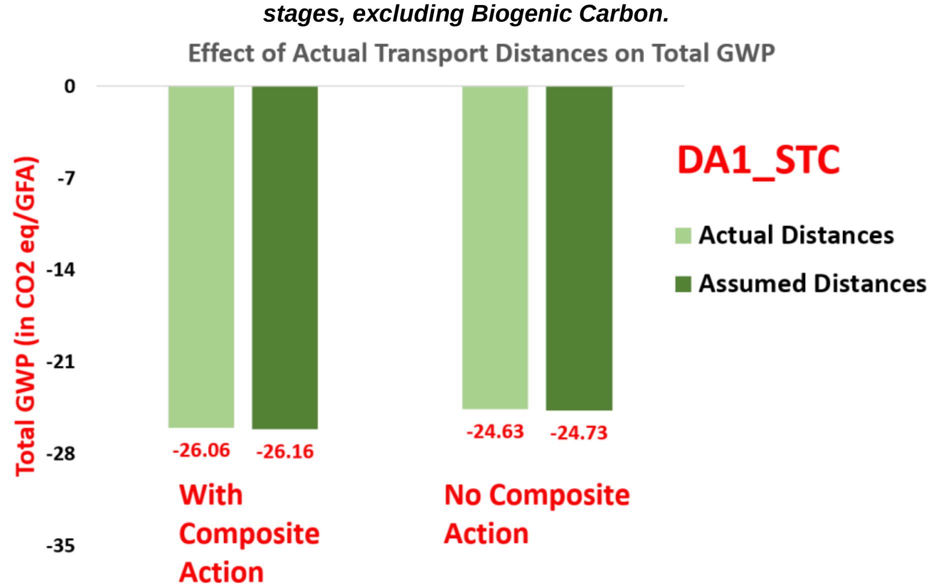


Figure 7.8: Effect on Transport Distances on Total GWP of Functional Equivalents.

7.4.3 Comparison of STC Floors with other Design Alternatives

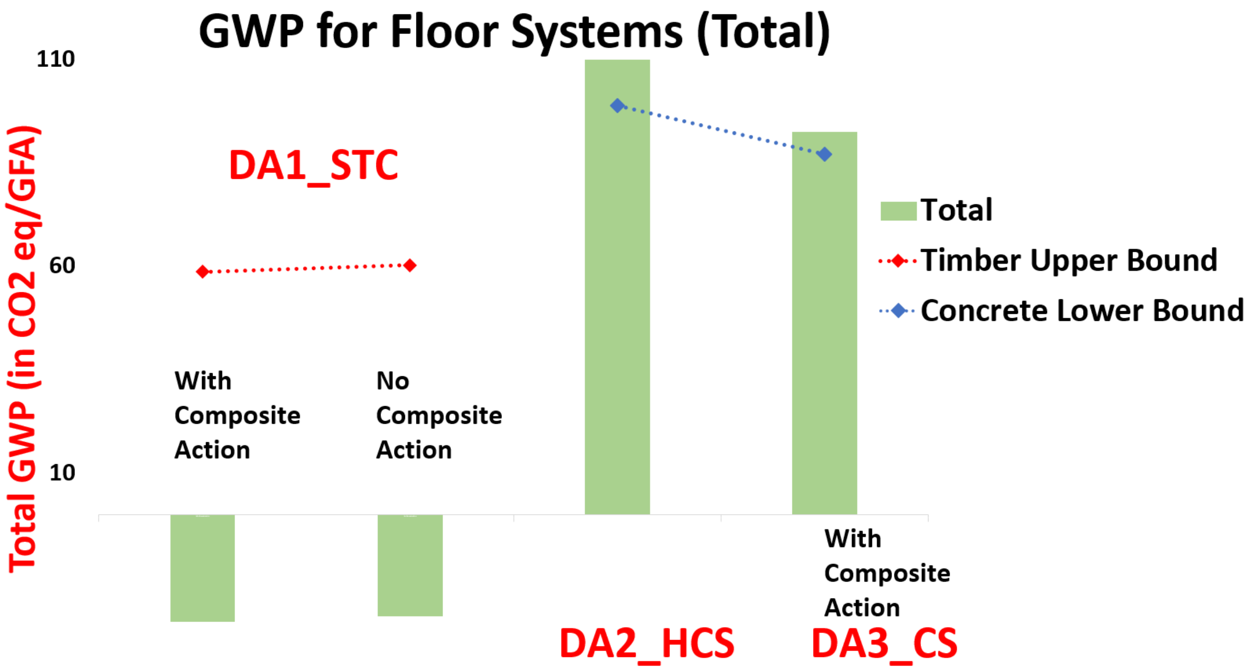


Figure 7.9: Comparison of GWP for all Design Variants.

Figure 7.9 above shows the comparison of total *GWP* of all design alternatives done for the case study. The data for the same is given in Appendix E.2.3. The bar graphs represent the nominal values, obtained as the average of data from multiple *EPDs* of the same material. For the *STC* design variants, the most onerous values obtained from the sensitivity analysis is marked. As we discussed earlier in Section 7.4.1, using the average values for timber and steel (with Storage as the timber *EoL* scenario) gives us net positive environment impact. Comparing these to the nominal values for ***DA2_HCS*** and ***DA3_CS***, we see that not only do they have positive values, but also that they have very high values ($+109.8\ CO_2\ eq/m^2$ and $+92.4\ CO_2\ eq/m^2$ respectively). This is because, using *STC* floors essentially means that we are replacing concrete/steel (which have a high environment impact) with timber material (which has a positive effect on the environment, mainly due to the storage of CO_2). Figure 7.10 below also shows the distribution of total *GWP* with respect to the constituent elements in both the floor systems.

Also in Figure 7.9 is marked the lower bound values for ***DA2_HCS*** and ***DA3_CS*** ($+98.8\ CO_2\ eq/m^2$ and $+86.9\ CO_2\ eq/m^2$ respectively). Now if we compare these with the upper bound values for ***DA1_STC*** – the case without composite action, using the maximum value of *GWP* from different *EPDs*, with landfill as the *EoL* scenario ($+60.2\ CO_2\ eq/m^2$), it can be observed that the former are still much greater (+64% and +44% respectively). Such an analysis with landfill as the *EoL* scenario represents the results for *STC* that excludes the benefits of carbon storage. It should also be noted that *STC* floors require additional material to provide the same level of fire protection and sound insulation as the other floors. Thus, to conclude, *STC* floors offer much lesser environmental impact that the conventional floor systems, even without considering the advantages of carbon storage.

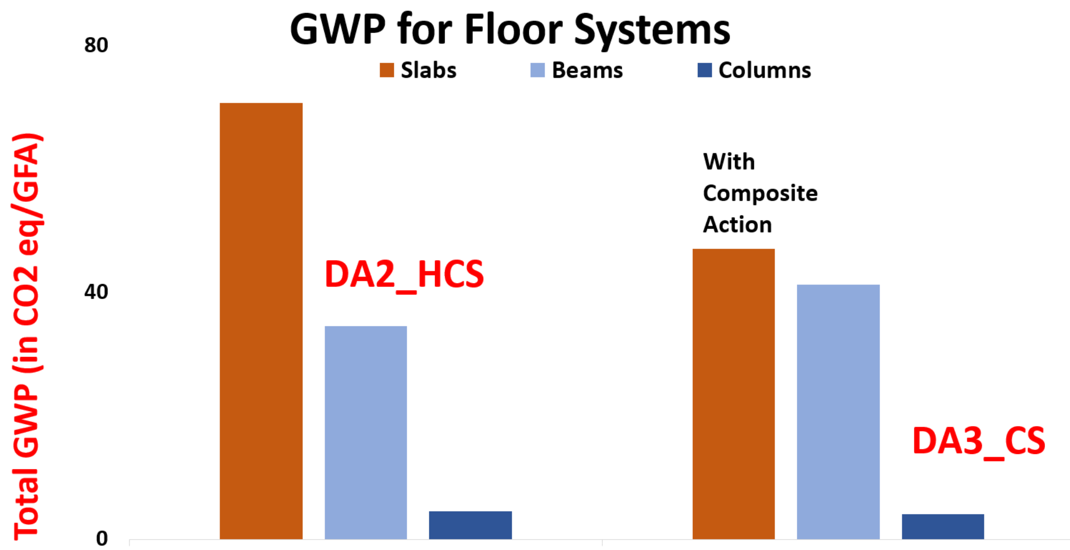


Figure 7.10: Total GWP for other design variants. Contribution by elements.

From Figure 7.10, it can be observed that the highest contribution for *GWP* for ***DA2_HCS*** and ***DA3_CS*** comes from the slabs. In fact, this is true for any floor system. Slabs are the material intensive component of buildings. Conventional construction materials such as steel and concrete produce negative impacts on the environment. However, for ***DA1_STC***, the use of concrete (and large amounts of steel) is replaced by timber. By virtue of carbon storing potential, *STC* floors can produce a positive impact on the environment. A comparison with Figure 7.6 will shed light on this aspect.

7.5 Summary

To compare the environment impacts of different floor systems, an *LCA* is conducted with a 100-year reference period, based on the dimensioned structural elements from Chapter 5. The main goal of this analysis is to compare the performance of *STC* floor systems with conventional floor systems. To have a better understanding of the underlying assumptions of the environmental impact data, it is opted to use *EPDs* instead of the data from the Dutch NMD. As a consequence, the shadow price method cannot be used for analysis, as the *EPDs* do not provide data on the 4 additional environment impact indicators (toxicity potentials). Out of the 7 environment impact indicators declared in all *EPDs*, it is decided to proceed ahead with the total *GWP.* Total *GWP* is chosen because this parameter constitutes the largest contribution towards environmental impact. A small comparison is made using the information from the Dutch NMD. Figure 7.2. shows us that for steel, concrete and timber, the largest share towards the shadow price comes from *GWP*, thus indicating that the latter has the highest contribution.

A key aspect in the analysis of environmental data using *EPDs* is the sensitivity to the *EPD* used. Thus, for all the main construction materials considered (steel, timber and concrete), multiple *EPDs* were referred, from which an appropriate value was chosen. The observation was that, as expected, there were some variations in the total values based on the *EPD* considered. This was most pronounced in the case of timber. Apart from variation based on the scope of the *EPDs* used, timber showed a large variation with respect to the *EoL* scenario considered. The stored carbon in timber is assumed to remain intact or released, based on the *EoL* scenario assumed.

Consequently, *EoL* scenarios such as incineration and landfill gave the highest environment impact, and those such as reuse and recycle gave minimum environment impact. Since the latter two *EoL* scenarios are rarely declared in *EPDs*, it opted to conduct the analysis with another type of *EoL* scenario – Storage. This *EoL* scenario only assumes that the stored carbon is not released back into the atmosphere (with suitable infrastructure), and does not assign any credits for Module D. It gives a slightly more conservative value (compared to reuse and recycle), while maintaining the benefits of stored carbon.

The net effect of using the respective *EoL* scenarios is that either it gives a large negative value of *GWP* for those which assume that the carbon stored remains intact i.e., a positive environment impact, and a positive value of *GWP* for those *EoL* scenarios in which the stored carbon is released back into the atmosphere. Since incineration allocates some credit for the energy produced, the total value is also negative in this case. However, this scenario can be counterproductive, as it promotes the production of energy using a less sustainable method, compared to other methods such as solar, wind, etc.

The other main construction materials (steel and concrete) show a much lesser degree of variation and assigning the *EoL* scenario is less complex – usually only one scenario is declared: recycling. A comparison of the total *GWP* values for the functional unit using the nominal (average) values of steel, timber and concrete showed that the performance of the different floors compared were on extreme ends of the spectrum – the *STC* floors gave a negative value (positive environment impact) whereas the concrete alternatives a large positive value. This was based on the assumption of storage as the *EoL* scenario for timber.

Now, by using an analysis which neglects the positive effects of carbon storage of timber – by considering the most onerous value for timber with landfill as the *EoL* scenario, it was observed that the *STC* floors still performed better that its concrete counterparts. This is because production of timber is a natural process (growth of trees) and manufacturing these timber elements is much less energy intensive compared to steel and concrete.

The effect of using the actual transport distances for timber on the analysis was also studied, and the results showed that this was negligible. This due because the effect of carbon storage (declared in stages A1-A3) overshadows the effect of the other life cycle stages. The effect on the total *GWP* of the floor system was negligible. By excluding the effect of carbon storage, the influence of actual transportation distances was found to be higher than earlier, but still limited.

To conclude, an analysis based on the total *GWP* environment impact indicator showed us that the performance of *STC* floors is much better than its conventional concrete counterparts. This claim is substantiated even further with the help of an *EPD* sensitivity analysis. Now, the question that arises is how the result will vary while considering other environment impact indicators. This question is particularly relevant since the parameter *GWP* can be advantageous to timber based on the *EoL* scenario that is assumed. The analysis of timber with landfill as the *EoL* scenario can help answer this question. In this analysis, timber gives a positive value for total *GWP,* similar to steel and concrete: and still, *STC* floors perform better. This is largely related to the fact that the quantity of timber used (in kg) for construction is much less than what is required for concrete and steel. This is a consequence of its low density ($7850\ kg/m^3$ for steel vs $2400\ kg/m^3$ for concrete vs $400-500\ kg/m^3$ for timber). Although only a full-scale analysis with other environment impact indicators can give accurate results, it is believed that *STC* floors will perform better while considering these parameters as well.

8. Conclusions and Recommendations

In this chapter, the main results of the thesis are compiled and summarized. The beginning of this chapter provides a summary of the results for the case study and calculations on composite action. In Section 8.1, the research questions given in the beginning of this thesis are addressed and answered. The aspects relevant to *STC* floor systems that could not covered in this thesis and provides a knowledge gap for further research is discussed in Section 8.2.

Keeping in mind the need to switch from the use of steel/concrete as the primary construction material, to a renewable sustainable material such as timber, the main objective of this thesis is to determine the best solution of a demountable steel-timber floor system from all the possible combinations of timber slabs and steel beams that are currently available in the market, in Europe, and to see how this floor system compares with conventional floor systems that are currently used.

First, the best combination of such a demountable steel-timber (*STC*) floor system is chosen objectively, based on a multi-criteria analysis (*MCA*). The chosen floor system is using Lignatur surface elements (*LFE*) as the timber slabs, and steel I beams. *LFE* offers box-shaped slender timber slabs, made of planks of solid timber glued together. The combination of these slabs with steel I beams offer ease of demountability, compared to the other integrated floor solutions.

The performance of the chosen *STC* floor system is compared to that of conventional floor systems with the help of a case study. The 'conventional floor systems' considered in this thesis are with hollow core slabs and composite slabs, both in combination with a steel frame (as in the case of the *STC* floors). For this comparison, the building chosen is Bouwdeel D, which is a building that was designed to be reused, and also uses a steel-timber floor system. The most important results of the case study are summarized in Table 8.1. The following conclusions can be drawn:

- ❖ The spacing of columns depends on the spans that be obtained using the respective slabs and beams.The floor system with hollow core slabs offer the largest column grid (***10.9 m x 10.6 m***). The floor system with composite slabs also use the same column grid, but require additional supporting steel beams. In this respect, the *STC* floor system does not perform as well as the others (column grid ***5.3 m x 10.9 m***).
- ❖ Weight of the *STC* floor system is significantly lesser than its conventional counterparts (***reduction by 24 – 45%***). This is due to the lightweight nature of timber material i.e., its low density. Lighter slabs mean lighter steel frames, and thus lighter foundations. Timber slabs are also easier to handle on-site.
- ❖ Floor height is another area where the *STC* floors do not perform well, being ***77 – 92% more*** compared to hollow core slabs, which are the most slender solutions. The reason for this being that the *STC* floor system is a non-integrated floor system. They are comparable in height to the floor system with composite slabs. This is because, beyond a span of 3.6 m (which is the optimum span of composite slabs without propping), the latter requires the use of extra steel beams (primary and secondary steel beams).

- With regard to environmental impact, *STC* floor systems produced the best results. With respect to an anlysis using the *GWP* indicator, *STC* floors gave a positive environmental impact, and came out as an eco-friendly choice of a floor system. As expected, the other floor systems gave a negative environment impact, with the largest contribution coming from slabs. Such an analysis was based on the carbon storing potential of timber elements, which is why the *STC* floors got net negative values. With the help of a sensitivity analysis, it was established that even by excluding the benefits of carbon storage, the *STC* floors still produced the least environment impact (***reduction by 40 – 53%***).

Table 8.1: Overview of Case Study Results.

	STC Floor System (With Composite Action)	STC Floor System (Without Composite Action)
Total Weight	$\mathbf{308.7}\ \boldsymbol{kg/m^2}$	$\mathbf{310.5}\ \boldsymbol{kg/m^2}$
Compared to Hollow Core Slabs	-44%	-44%
Compared to Composite Slabs	-25%	-24%
Floor Height	$\mathbf{510}\ \boldsymbol{mm}$	$\mathbf{550}\ \boldsymbol{mm}$
Compared to Hollow Core Slabs	$+78\%$	$+92\%$
Compared to Composite Slabs	-2%	$+6\%$
Environmental Impact ***(Including Benefits of Carbon Storage)***	$\mathbf{-26.1}\ \boldsymbol{CO_2\ eq/m^2}$	$\mathbf{-24.6}\ \boldsymbol{CO_2\ eq/m^2}$
Compared to Hollow Core Slabs	-125%	-124%
Compared to Composite Slabs	-128%	-126%
Environmental Impact ***(Excluding Benefits of Carbon Storage)***	$\mathbf{+58.6}\ \boldsymbol{CO_2\ eq/m^2}$	$\mathbf{+60.1}\ \boldsymbol{CO_2\ eq/m^2}$
Compared to Hollow Core Slabs	-52%	-51%
Compared to Composite Slabs	-42%	-41%

Apart from this, the practical benefits of composite action between the timber slabs and steel beams, in such a floor system is investigated. By using similar structural grids for the floor system, a comparison is made between composite action in steel-concrete and that in steel-timber. The benefits of composite action in steel-concrete have been well established, and has been identified as worth considering, to have significant reduction in the size of the supporting steel beams. The use of reoriented flanges for the timber slabs at the supports provided a higher transverse bending stiffness compared to two-way spanning timber slabs such as *CLT*. This assumption was sufficient to make up for the fact that the considered timber slab is one-way spanning (compared to concrete in composite slabs, which is two-way spanning). An overview of the comparison of composite action between the two, in terms of gains in mechanical properties, and savings in materials is given below in Table 8.2. The following results have been obtained regarding composite action in steel-timber:

- It can be observed that there is an increase in the mechanical properties of steel-timber composite beam, but in comparison to extent of gains in steel-concrete, this is very small. The extent of gains in steel-concrete is approximately ***5x*** for bending stiffness, ***2x*** for elastic bending moment capacity, and ***4x*** for plastic bending moment capacity, compared to steel-timber.
- The practical net benefits due to composite action is measured as the total savings in the amount of steel required for the cross beams. For steel-timber, this is $\mathbf{1.8}\ \boldsymbol{kg/m^2}$,

compared to the $\mathbf{5.44}\,\boldsymbol{kg/m^2}$ steel saved for steel-concrete. The overall effect of composite action in steel-timber for the case study design is a reduction of weight of the floor system by ***1%***, and a reduction of floor height by $\mathbf{50}\,\boldsymbol{mm}$.

- ❖ The use of elastic analysis on the design of composite beam had a huge effect on composite action in steel-concrete, as the governing criterion was the bending moment resistance. By using plastic design, net savings in steel up to $\mathbf{14.7}\,\boldsymbol{kg/m^2}$ can be achieved, combined with a reduction in floor height by $\mathbf{100}\,\boldsymbol{mm}$. Since the governing criterion for design of the steel-timber composite beam was deflection, no further savings could be obtained for plastic design.
- ❖ Considering the aspect of environmental impact also, the net gains due to composite action in steel-timber were minimal. The decrease in *GWP* was just $\mathbf{1.5}\,\boldsymbol{CO_2\,eq/m^2}$.
- ❖ The design resitances of the same shear connectors were signifiacntly lesser in the case of steel-timber connections, than for steel-concrete connections. Thus, for transferring the same shear forces, more number of shear connectors would be required, and this would significantly increase the costs.

Table 8.2: Overview of results for Composite Action in steel-timber.

Parameters	**Composite Action in Steel-Timber Floor Size: $L_S = 5.3 * L_B = 10.9\,m$**		**Composite Action in Steel-Concrete Floor Size: $L_S = 3.53 * L_B = 10.9\,m$**	
	Composite Beam	*Compared to Steel Beam alone*	**Composite Beam**	*Compared to Steel Beam alone*
Bending Stiffness	$8.72 * 10^{13} Nmm^2$	$\mathbf{+37}\%$	$11.3 * 10^{13} Nmm^2$	$\mathbf{+210}\%$
Peak Stresses in Steel Section	$303\,MPa$	$\mathbf{-11}\%$	$316.6\,MPa$	$\boldsymbol{NA}$
Elastic Bending Moment Capacity	$727\,kNm$	$\mathbf{+13}\%$	$593.4\,kNm$	$\mathbf{+32}\%$
Plastic Bending Moment Capacity	$876.4\,kNm$	$\mathbf{+18}\%$	$886.4\,kNm$	$\mathbf{+80}\%$
Amount of Steel for Cross Beams	$25.1\,kg/m^2$	$\mathbf{-6}\%$	$28.9\,kg/m^2$	$\mathbf{-14}\%$

Considering all these aspects, it is concluded in this thesis that applying composite action between timber slabs and steel beams cannot lead to practically justifiable gains, in terms of reduction of size of the steel beam. This is mainly because timber material is much weaker compared to steel, with respect to its mechanical properties.

Finally, design recommendations are provided for the chosen *STC* floor system and is given in Section 6.5. This is in the form of span tables that are made for the layouts of typical Dutch offices. This is meant to serve as a reference for designers for using this particular floor system.

To summarise, this thesis investigates the applicability and relevance of demountable steel-timber floor systems for the construction industry in the Netherlands, and in Europe, through comparisons with conventional floor systems such as hollow core slabs and composite slabs. The main findings of this research work, that can be of benefit to society are three-fold:

- Identifying and choosing the best demountable steel timber floor system, from all possible combinations. Most timber elements enjoy the benefit of being lightweight in nature, but those which also hold the advantage of low environmental impact are the ones made with sawn timber (instead of *CLT*, *GLT* and *LVL*). Thus *LFE* slabs were obtained to be the best choice, in combination with steel I beams.
- Showing light on the chosen steel-timber floor system as a lightweight alternative to conventional floor solutions, thus avoiding the depletion of extra raw materials required for the remaining parts of the building (building frame and foundations).
- Showing light on the chosen steel-timber floor system as the most sustainable choice compared to its conventional counterparts. By shifting to the former, one can change the floor system of a building from being the most contributing element towards environmental impact, to the least contributing element. Also, by choosing the former, we are replacing the consumption of non-renewable resources (steel/concrete) with timber, which is a renewable resource.

By emphasizing the key benefits of the chosen steel-timber floor system, it is expected that this research will help practitioners in the construction industry to opt for the more responsible choice, with steel-timber floor systems. An approximation for the amount of buildings to be newly constructed in Europe in the year 2023, in terms of floor area is $190\ million\ m^2$, according to a survey in 2010 [143]. In a developing country such as India or China, with a smaller percentage of built environment, this figure would be much higher. With the help of this data, and using the results of the *LCA*, the total environment impact produced by the construction industry in Europe is estimated (assuming that it is entirely constituted of one of the three types of floors systems considered in this thesis). This is given below in Table 8.3.

Table 8.3: Calculation of Total Environment Impact for the different floor systems.

$Total\ Floor\ Area\ to\ be\ Constructed\ in\ 2023{:}\ 190\ million\ m^2$

$Total\ GWP\ for\ 2023\ =\ 190\ million\ m^2\ *\ GWP\ per\ Gross\ Floor\ Area$

Floor System	GWP per Gross Floor Area $[CO_2\ equivalents/m^2]$	Total GWP for 2023 $[billion\ CO_2\ equivalents]$
Steel-Timber Floors	60.2	+11.42
Floors with Hollow Core Slabs	102.1	+19.4
Floors with Composite Slabs	92.3	+17.54

From Table 8.3, it can be observed that the total environmental impact is $+19.4\ billion\ CO_2\ equivalents/year$ and $+17.54\ billion\ CO_2\ equivalents/year$, for the scenario where the whole industry is comprised of hollow core slabs or steel-concrete composite slabs respectively (assuming a 100 year service life). The same value for *STC* floors is estimated to be $+11.42\ billion\ CO_2\ equivalents/year$ (by considering the most onerous scenario: landfill at the end of life) i.e., a reduction by at least $6.12\ billion\ CO_2\ equivalents/year$ compared to the next best option.

Now, if we consider the same scenario, but with the additional requirement that 10% of the floors be reused after 20 years, we can apply some credit to these floor systems, as all of these are demountable, and can be reused. According to the conclusions of this thesis, all the steel and concrete elements can be reused, without considering any reductions in its mechanical properties. For timber elements, as they are associated with a loss in strength, they cannot be reused for the same function. A conservative approach is used here, and thus no credits are applied for reusing the timber elements. Now, total environmental impact is calculated, by applying the credits for reusing these floor systems. This is given below in Table 8.4.

Table 8.4: Calculation of Total Environment Impact for the different floor systems, considering reusability.

$Total\ Credits\ for\ Reusing: 10\%\ of\ 190\ million\ m^2$
$Contribution\ of\ Timber\ Elements\ in\ Steel - Timber\ Floors: 32.6\ CO_2\ equivalents/m^2$
$Reduction\ in\ Total\ GWP\ for\ 2023 = 0.1 * Total\ GWP\ for\ 2023\ for\ Steel/Concrete\ elements$
$and\ 0\ for\ Timber\ Elements.$
→ $Steel - Timber\ Floors: 0.1 * 0.19*(60.2\text{-}32.6) = 0.524\ billion\ CO_2\ equivalents$
→ $Floors\ with\ Hollow\ Core\ Slabs: 0.1 * 19.4 = 1.94\ billion\ CO_2\ equivalents$
→ $Slabs\ and\ Composite\ Slabs: 0.1 * 17.54 = 1.754\ billion\ CO_2\ equivalents$

Floor System	Reduction in GWP for 2023 $[\boldsymbol{billion\ CO_2\ equivalents}]$	Total GWP for 2023 $[\boldsymbol{billion\ CO_2\ equivalents}]$
Steel-Timber Floors	0.524	+10.9
Floors with Hollow Core Slabs	1.94	+17.46
Floors with Composite Slabs	1.754	+15.79

It can be observed that large credits are assigned for reusing steel/concrete elements, compared to timber elements. However, even still, the steel-timber floors have the least environmental impact, thus providing incentive to switch from the conventional floor systems.

8.1 Research Questions

In this section, all the research questions that have been identified initially, are looked into, and answered.

8.1.1 What is the best choice of a demountable steel timber floor system among all the possibilities of the combinations of steel timber products commonly used in the Netherlands?

"What are the requirements for a Floor System to be Reusable?"

This sub-research question has been answered with the help of a literature review given in Section 2.2 and Section 2.3.

At the level of the structural element, the requirements for the floor system to be reusable have been identified. The most important aspect is to use prefabricated elements in conjunction with demountable connections. We saw that in the case of composite slabs, there is the option of having the slab cast in-situ in the first service life. However, the final connections of the slab to the remaining structure will have to be executed with dry connections, to ensure demountability. In the case of precast concrete units (such as hollow core slabs), the top layer of finishing concrete will have to be replaced with dry topping, added to the fact that the final connections to the main structure will also have to be using dry connections. In this respect, timber elements are the most suitable, since the conventional practise is to execute them with dry connections. The prefabricated timber elements identified in this thesis were found to give sufficient spans (from 7m up to 13 m). Added to this, timber also offers the advantage of low weight, which helps in the process of transportation between sites.

Bolts have been identified as most suitable option for demountable connections. The process of assembly/disassembly and reassembly can be sped up by incorporating the possible geometric deviations in the structural elements and column grids. This is done with the help of oversized holes for bolts. The connections that were in focus in this thesis were the shear connectors between the timber slabs and the steel beams. For the case study considered in this thesis, it was observed that an oversize hole of 16 mm was sufficient for this, using elements of tolerance class 2. Using elastic analysis for the design of structural elements can reduce the probability of occurrence of damage, which is also a requirement for reusing them. Thus, all the design checks in the ultimate limit state are done such that they are within the elastic limit. This had a pronounced effect on the design of cross beams for the design alternative with composite slabs. The fact that the plastic bending moment capacity could not be utilized led to a reduction in savings of the steel material by about $9.26\ kg/m^2$.

A direct implication of reusing structural elements is that they will have to remain in service for a larger period i.e., increased serviced life. Based on the type of reuse that is considered, the initial design will have to incorporate either an increase in the live loads, or a decrease in the strength properties, specifically relevant for timber. Since the reduction of strength observed in reused timber elements cannot be quantified in the initial design phase, the former is used for considering reuse of the floor systems (Reuse by Reorientation). From calculations, it was observed that considering this type of reuse produced negligible change on the live loads (from $3.5\ kN/m^2$ to $3.6\ kN/m^2$), and consequently the dimension of the structural elements.

Thus, the requirements for a floor system have been defined i.e., prefabricated elements, demountable connections, elastic design, and an increased service life. This serves as the boundary conditions for identifying the best solution of a demountable steel-timber floor system. All of these 4 aspects have been incorporated into design.

"What are the commonly used steel beams and timber slabs in Europe, and in the Netherlands, and what is the scope of their application?"

This sub-research question has been answered with the help of a broad literature study into the various steel and timber products currently available in the market, using their product catalogues and *ETA* documents. Examples of steel timber construction and demountable construction also have been looked into, to identify the commonly used products. This is given in Chapter 3.

Various products have been identified both for steel and timber. The primary constraint was these products should offer high spans, thus reducing the number of elements required per unit area of a floor. This aids in making the process of assembly/disassembly and reassembly faster. Consequently, for steel, smaller cold formed sections were avoided, and it was decided to use larger hot-rolled sections or welded sections, which offered larger spans, suitable for office spaces. The ones that were considered were typical I beams and castellated I beams, and integrated beams such as *IFB*, *SFB*, *THQ* and *RHS* beams.

For timber, this meant that typical joist slabs were avoided, as these had less degree of prefabrication, and offered lower spans. Instead, the decision was to use high span, prefabricated units: *CLT* solid slabs and ribbed elements, Lignatur elements, and Kerto RIPA floor elements were considered. The timber slabs so obtained were classified based on the geometry of the slabs and based on the type of engineered timber products used to make these elements.

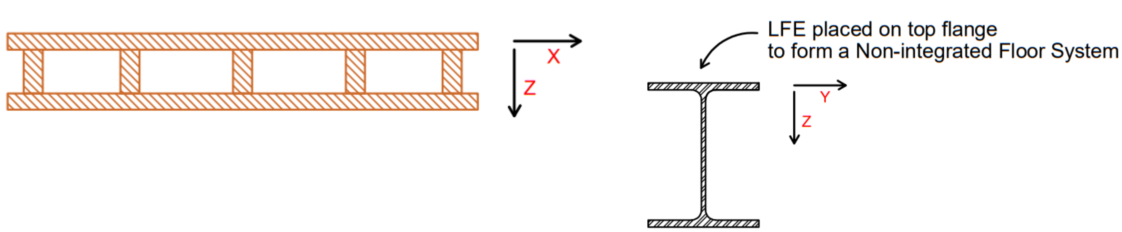
Multi Criteria Analysis to identify the best demountable floor system with steel beams and timber slabs:

Once the various steel timber products have been identified, and the boundary conditions for reusing have been formulated, a Multi Criteria Analysis (*MCA*) was conducted to determine the best possible demountable steel timber floor system. The different floor systems were rated based on parameters such as weight, floor height, sustainability, circularity and building decree. This is given in Chapter 4.

The *STC* floor system obtained as the result of the *MCA* was with *LFE* slabs and steel I Beams (***STC2***). The main advantages of this system were as follows:

- Lightweight nature of *LFE.*
- Low environment impact of *LFE* over other timber products.
- Low weight of I beams compared to other steel beams.
- Being a non-integrated system, it was found to be more demountable that integrated systems with the same *LFE* slabs.

It was observed that the contribution of slabs was dominating for parameters such as weight and sustainability. The combination with different steel beams mainly affected the demountability of the floor system and the floor height. Thus, among all the *STC* floors that scored well in the *MCA,* the common element was the *LFE* slabs. These were the best slabs compared to the other products, owing to its slender, lightweight geometry. Also, between the different timber products considered, sawn timber had a significantly lower environmental impact that the other engineered timber products (such as *CLT* and *LVL*). Thus, the main decision was between ***STC2***, which was a non-integrated system and ***STC8***, which was an integrated system. Due to the ease of demountability, ***STC2*** was chosen. The fact that ***STC2*** could give more benefits due to composite action (owing to its larger sectional height), also helped in making this decision.



Cross Section of LFE. Cross Section of I Beam.

Figure 8.1: Chosen STC Floor System (STC2).

8.1.2 How does the chosen demountable steel timber floor system compare with conventional solutions for demountable floor systems?

The conclusions of the case study in Chapter 5, and the Life Cycle Analysis corresponding to this, given in Chapter 7, provide answers to this research question. The case study building 'Bouwdeel D' is compared by implementing different floor systems. The 3 design alternatives considered are: *STC* floor system, the floor system with hollow core slabs (*HCS*), and the floor system with composite slabs. *HCS* is chosen as this is the most commonly used slab in the Netherlands, and Composite slab is chosen to make comparisons for composite action for steel-timber with steel-concrete.

"Is it advantageous to use the chosen floor system considering aspects of functionality such as floor height, workability and savings in materials?"

The main results of the case study have already been summarised in the beginning of this chapter. To reiterate, the main advantages of using the chosen *STC* floor system is related to its low environmental impact (explained further in this section) and its lightweight nature. As a consequence of its low weight, *STC* floors offer ease of handling on-site and for transportation, and also results in low cost of foundations. The use of steel/concrete for the slabs is replaced by sustainable lightweight timber, and the requirement of steel for the frame of the building is reduced by using a lighter slab.

The disadvantage of *STC* floors is related to its floor height. It does not offer a slender solution like that obtained for floors with *HCS*, although they are comparable in height to floors with composite slabs. They do not offer the possibility of high spans like that for *HCS,* and this is evident in the column grids obtained for the design variants (5.3 m x 10.9 m for *STC* vs 10.9 m x 10.6 m for *HCS*). To a large extent, the issue of floor height can be solved by using an integrated steel beam (like that for *HCS*). The reason for choosing a non-integrated solution from the *MCA* was considering the aspect of demountability. In all other aspects, the combination of *LFE* slabs with different steel beams fared similarly.

Another aspect to be considered is the circularity of the different floors systems considered. Since all the floor systems use a steel frame for the building, it mainly comes down to the slabs used i.e., timber, concrete or steel. The fact that *LFE* slabs offer better workability on-site, and is much easier to transport has already been discussed. From the perspective of structural elements, the question is whether they can remain in use for long periods, and the implications of process of disassembling and reusing over and over on the material. In this respect, timber performs poorly compared to steel and concrete, as the latter two can be reused easily without any conservative reductions in strength and stiffness. Though it is not possible to determine how the mechanical properties of timber change over each period of use, two conclusions can be drawn from available literature: First of all, from experiments on reused structural timber, it was determined that these structural elements can used again for structural purposes. The second aspect is that timber as a structural material most certainly does experience deterioration in its mechanical properties. This is due to various factors such as damages in the elements, biological attack, the effect of duration of load, and the natural ageing phenomena of timber.

The current practise to reuse structural timber is to grade the timber elements again, after its initial service life. Many researchers have proposed guidelines for reusing structural timber, by suggesting a decrease in its strength. However, the main drawback of all these is that they do not specify any particular time period for this strength reduction to occur i.e., there arises the ambiguity that the same strength reduction is proposed for timber elements that remain in service for a short period (say 6 months), and for those that remain in service for larger periods (say 50 years). Since the properties of reused timber could not calculated before-hand, in this thesis, the performance of demountable floor systems could not be compared for more than one service life. This is the reason that reuse by reorientation had to be considered. Thus, this drawback of timber heavily reduces the applications of timber as a reusable structural material.

"Is it advantageous to use design with composite action for steel timber? Can the net gains of composite action in steel timber be justified? How does it affect the load deflection curve of the structure?"

Calculations on composite action in steel-timber have been done in this thesis to identify whether it could lead to any practical benefits in design. The method used here was to draw conclusions based on a comparison between composite action in steel-concrete (which is assumed to set the standards for net gains due to composite action) and steel-timber. Ultimately, it comes down to whether the added costs for shear connectors can be compensated by the net savings in steel (reduction of steel section size).

The effective mechanical properties of the composite section with full shear interaction can be predicted using steiner's rule for elastic analysis. For partial shear interaction, the Gamma method used for built-up timber sections showed good correlation with the experimental results of 4-point bending tests on *STC* sections joined with metal fasteners. The requirement is that the slip at the steel-timber interface be incorporated in modelling the behaviour of the *STC* section. The Newmark model presents a more accurate solution, with the ability to model different arrangements of shear connectors, and different boundary conditions for the structural systems. The main requirement for any method is that there be no deformations in the system at the onset of composite action. This problem can be solved by precambering the steel beams to take all the dead loads. This also serves another purpose, that the timber component of the *STC* can analysed with smaller creep factors (lesser creep effects), as the *STC* section only takes the live loads. In this way, the use of propping can be avoided, making full utilization of the steel section. The load deflection curve of the composite beam can be predicted using the new effective bending stiffness, obtained using the methods for partial or full shear interaction. The properties of the plastified *STC* section is determined using the Eurocode method for steel-concrete composite structures, as experiments on the same showed sufficient ductility before failure.

The calculations done in this thesis are specifically for the case study, for the *STC* floor system chosen from the *MCA.* Since the timber slab considered is a one-way spanning box element, reoriented timber flanges are assumed near the supports when calculating the effective properties due to composite action. For a fair comparison with steel-concrete, it would have been accurate to consider a bi-directional timber slab, such as *CLT.* The assumption of reoriented timber flanges meant that optimal properties of timber could be used (direction perpendicular to the grain angle), and this resulted in higher bending stiffness for the section in the direction of the bending action of the steel beam. Thus, the conclusions drawn for this specific case of *STC* floor system can be expanded to include a wide array of combination of timber slabs and steel beams.

To understand the maximum benefits of composite action in steel-timber, the case of full shear interaction is considered. The main results of the calculations have already been given in the beginning of the chapter. It was observed that the net savings in steel was only $1.8\ kg/m^2$, about 18% of what could be achieved for steel-concrete. Not only that, but this came at the cost of a large number of of shear connectors, since the design resistances of these were low in a steel-timber connection, compared to the same for a connection in steel-concrete. Moreover, a comparison of the increase in mechanical properties such as bending stiffness, elastic and plastic bending moment resistance, and decrease in the peak stresses in the section (for elastic analysis) showed consistently low values. As explained earlier, this is because timber is a weaker material compared to steel and concrete. Since there are no practical gains due to composite action in steel-timber, it is concluded in this research that it is not worth incorporating in design.

The scenario considered here is the combination of large span timber slabs and steel beams, and conclusion is that there are no significant advantages: timber being a weak material cannot aid steel, which is a significantly stronger material. Conversely, the use of small steel sections with timber slabs can help increase the mechanical properties of the latter. Utilizing this effect can lead to the development of new hybrid steel-timber slabs. This aspect was not considered in this research as the focus was mainly on *STC* floor systems currently available in the market.

"Is it advantageous to use the chosen floor system considering the aspect of environmental impact?"

Timber is a sustainable renewable material, as opposed to steel and concrete, which are non-renewable resources. The case study of different floor systems revealed that the use of a *STC* floor system helped replace the use of steel and concrete by using timber. The lightweight nature of timber slabs meant that there was less load on the steel frame of the building, which further reduced the amount of steel required.

The comparison of environment impact of the different floor systems was done with the help of a *LCA*. Using data from the *EPDs* of different materials meant that the analysis could not be done with the shadow price method, due to the unavailability of data on the additional 4 toxicity potentials (which are conventionally not declared in the *EPDs*). Thus, the analysis was limited to the use of just one parameter – *GWP,* which holds the largest contribution towards environmental impact. From the *LCA* on the different floor systems, the following conclusions can be drawn:

- The values for *GWP* for different materials were sensitive to the *EPD* used: The main materials considered for this sensitivity study are steel, timber and concrete i.e., the largest contributors in the floor systems considered. This effect was most pronounced on timber, mostly owing to the allocation of the *EoL* scenario. The value varied from $-\mathbf{1.59}\ \boldsymbol{CO_2\ eq/kg}$ for recycling up to $+\mathbf{0.8}\ \boldsymbol{CO_2\ eq/kg}$ for landfill, based on whether the stored CO_2 was released or not in the *EoL* scenario considered.
- By comparing the *GWP* results for the different floor systems, the *STC* floor system came out as the best ($-\mathbf{24.6}\ \boldsymbol{CO_2\ eq/m^2}$, without considering composite action), with the other floor systems obtaining very large values of environmental impact. For this analysis, the *EoL* scenario considered for timber was Storage, which assumes that the CO_2 remains intact i.e., it includes the benefits of carbon storage.
- The upper bound value for the *STC* floor system was obtained to be $+\mathbf{60.2}\ \boldsymbol{CO_2\ eq/m^2}$, assuming Landfill as the *EoL* scenario for timber i.e., analysis that exlcudes the benefits of carbon storage of timber. Comparing this with the lower bound values obtained for the floor systems with concrete, *STC* was still the better option.
- The impact of importing most of the timber elements from other European countries to the Netherlands was also studied, by considering the actual transport distances. The result was an increase of just $\mathbf{0.1}\ \boldsymbol{CO_2\ eq/kg}$ with respect to the base values of $-\mathbf{1.43}$ and $+\mathbf{0.15}\ \boldsymbol{CO_2\ eq/kg}$ (corresponding to the analyses including and excluding the benefits of carbon storage respectively). This had a very small effect on the total *GWP* of the *STC* floors.
- The largest contribution to *GWP* came from the slabs, for those floor systems with *HCS* and Composite slabs, and these slabs mainly comprised of concrete and/or steel. By replacing these with timber, as in the case of the *STC* floors, the total impact could be

reduced significantly, even up to the extent of obtaining a positive environment impact, as we observed above.

According to the author, it is believed that these results will not be significantly different (i.e., the *STC* floors will still have the least environment impact) if we consider a different environment impact indicator, other than *GWP.* The reason for this is based on the low value of density of timber compared to steel and concrete. Any analysis with other environment impact indicators will show similar results to that of *GWP* analysis without the effect of carbon storage.

Thus, *STC* floors provide a good alternative to the conventional floor systems in terms of structural and environmental aspects. Where this falls short is in the costs of construction. *HCS* floor systems are the cheapest option, only because they are widely used in the construction industry. Thus, making a responsible choice with *STC* floors can lead to wider circulation of this floor type, and consequently lower the costs for construction.

8.2 Possibilities for Future Research

The assessment of *STC* sections done in this thesis were founded on the correlation of the experimental results of Hassanieh's experiments on *STC* beams [15] with the Gamma Method. Even though it was concluded that considering composite action in typical steel-timber floor systems could not lead to any practical benefits, it has been concluded in this thesis that it is a very good choice for sustainable construction. In this Section, some aspects related to steel-timber construction, that could not be covered in this thesis, and provides scope for future research is addressed.

8.2.1 Reusing Timber Elements

The reason that reusability of timber could not be included in calculations (i.e., Reuse by Relocation) is because the mechanical properties of the timber elements cannot be determined during the initial design phase. At the same time, all experiments on reused timber suggested that it is possible to reuse timber, although with reduced mechanical properties. Current recommendations available for reused timber elements [90-93] suggest that reductions be applied on the mechanical properties of timber but show ambiguity in the fact that none recommend the service life of these elements on which the reductions are to be applied. In this thesis, the research work of Cavalli et al [130] was cited, where all the experiments on reused timber elements were summarized. Though there were large variations in the results, most of them could be partly attributed to the inconsistencies in the methods of testing, as these experiments were conducted by different researchers spread across different places. Thus, for addressing this issue, a large-scale testing programme is suggested. The aim is to investigate the residual mechanical properties of reused timber structural elements. The experiments should include aspects such as the species of timber, duration of load effects (by including load history), moisture content, etc.

8.2.2 Resin Injected Bolted Connections for Steel – Timber

At present, there is no research available on resin injected bolted connections for steel-timber with oversized holes. In this thesis, it was opted to have to oversized holes in timber. Due to this gap in literature, this aspect could not be considered further, as it was not possible to predict the reduced stiffness of such connections. It was concluded in this thesis that such a connection with resin inside the oversized hole in timber would provide a good prospect in terms of a demountable shear connection. Thus, it is suggested here that this might be worth investigating. This type of assessment would require accurately designed experiments, to study the reductions in stiffness compared to the stiffness that is predicted by *EC5*. The reductions would depend on the type of resin (stiffness, strength), the geometry of the oversized holes (diameter and thickness), and the geometry of the bolts used (diameter, length). The proposed type of connection is shown below in Figure 8.2. Push-out tests of such connections are required to obtain the load-slip behaviour of such connections, and to investigate its various failure modes.

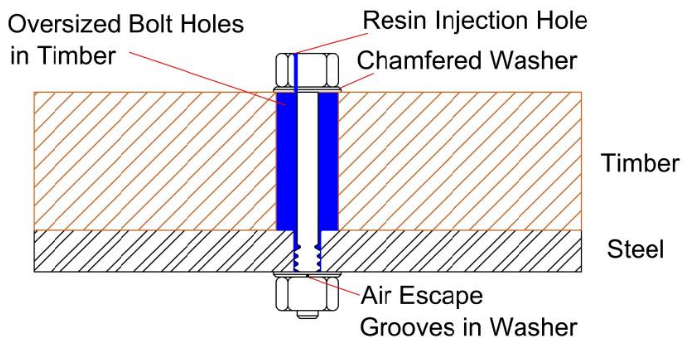


Figure 8.2: Proposed Resin Injected Bolted Connection with Oversize Hole in Timber.

In this figure, it is suggested to have the bolt inserted from the timber end and tightened at the steel end. Holes for injecting the resin should be provided inside the bolt head, with the an air escape groove at the other end, to avoid air bubbles in the resin. Chamfered washers are provided to give small amount of pretensioning, to ensure that there are no gaps at the interface through which the resin might escape. The level of force applied should be in line with the maximum permissible forces on timber (considering compression perpendicular to the grain.

8.2.3 Combination of Stresses in STC Sections

For the case study design alternatives with *STC*, it is opted have the slabs as simply supported, instead of spanning continuously. Even though this design choice would lead to savings in connections at the slab ends, it would also increase the sagging moments and deflections to a certain extent. This decision was made to avoid stresses in timber due to double bending i.e., hogging moments (σ_{yy}) in the direction of the span of the slab, and sagging moments (σ_{xx}) due to STC composite action. The same applies for shear stresses $(\tau_{yz}\ and\ \tau_{xz})$. Factoring in the orthotropic nature of timber, these combined stresses would lead to failure in the timber slab for *LFE* sections.

It was concluded in this thesis that it is not beneficial to apply composite action to typical steel – timber floor systems currently available in the market. At the same time, it was suggested that manufacturing new types of hybrid steel – timber slabs by using composite action can very beneficial. In such cases, it is essential to consider the above-mentioned effect of the combination of stresses. For steel, the effect of combination of stresses is incorporated using the von – mises criterion. Thus, for timber an equivalent orthotropic von – mises criterion can be used. The assessment of *STC* using such a 3D damage criterion also holds good promise for future research.

9. Bibliography

[1] Dutch Building Decree (Bouwbesluit), (2012).

[2] Jonathan Barzillai, *Preference Function Modelling: The mathematical foundations of decision theory*, Trends in Multiple Criteria Decision Analysis, Chapter 3, 57-86 (2009).

[3] Hans Joachin Blass and Carmen Sandhass, *Timber Engineering: Principles for Design*, KIT Scientific Publising, (2017).

[4] Bouwen met Staal, Technical File: *Vloeren van Kanaalplaten met Geintegreerde Stalen Liggers,* (2007).

[5] Stora Enso, Technical Brochure: *CLT by Stora Enso*, (2017). Available from: https://www.storaenso.com/en/products/wood-products/massive-wood-construction/clt/brochures-and-downloads, Accessed: April 2021.

[6] Stora Enso, Technical Brochure: *CLT rib panels by Stora Enso*. Available from: https://www.storaenso.com/en/products/wood-products/massive-wood-construction/rib-panels. Accessed: April 2021.

[7] Lignature, Technical Brochure: *Lignatur Workbook*, (2014). Available from: https://www.lignatur.ch/en/downloads. Accessed: April 2021.

[8] MetsäWood, Technical Brochure: *Kerto for Load bearing Structures*. Available from: https://www.metsawood.com/global/Tools/MaterialArchive/MaterialArchive/Kerto-for-load-bearing-structures-English.pdf. Accessed: April 2021.

[9] Eurocode Applied.com, Online Tool: *Table of design properties for Flanged Steel Profiles (IPE, HEA, HEB and HEM),* Available from: https://eurocodeapplied.com/design/en1993/ipe-hea-heb-hem-design-properties. Accessed: April 2021.

[10] Arcelor Mittal, Technical Brochure: *ACB and Angelica Beams: A new generation of beams with large web openings,* Available from: https://constructalia.arcelormittal.com/en/products/acb. Accessed: April 2021.

[11] Staalmeesters, Technical Files: *Hat Beams: THQ, IFB and SFB beams,* Available from : https://www.staalmeesters.com/hoedliggers. Accessed: April 2021.

[12] Peikiko Group, Technical Brochure: *Delta Beams Composite Delta Beams: Slim Floor Structure with integrated Fireproofing,* (2021). Available from: https://www.peikko.com/products/product/deltabeam-product-information/. Accessed: April 2021.

[13] Peikiko Group, Technical Brochure: *Delta Beam Slim Floor Structure: Slim Floor Systems in Wood Construction,* (2021). Available from: https://www.peikko.com/products/product/deltabeam-product-information/. Accessed: April 2021.

[14] Tata Steel, Technical File: *Celsius: Hot Finished RHS,* Available from: https://www.tatasteeleurope.com/ts/construction/products/tube/structural-hollow-sections. Accessed: April 2021.

[15] A. Hassanieh, H. R. Valipour, M.A. Bradford, *Experimental and Numerical investigation of short-term behaviour of CLT-steel composite beams,* Engineering Structures144 (2017) 43 – 57.

[16] Marcin Chybinski, Lukasz Polus, *Theoretical, numerical and experimental study of aluminium – timber composite beams with screwed connections,* Construction and Building Materials 226 (2019) 317 – 330.

[17] Pinelopi Kyvelou, Leroy Gardner, David A. Nethercot, *Composite Action between Cold-formed steel beams and wood-based Floorboards*, International Journal of Structural Stability and Dynamics, (2015).

[18] Cepezed Architects, Technical File: *Bouwdeel D,* (2019).

[19] Bouwen met Staal, Technical File: *Duurzame stalen vloersystemen,* (2013).

[20] Austrian Institute of Construction Engineering: Vienna, Austria, European Technical Assessment document: *CLT-Cross Laminated Timber,* ETA-14/0349 of 06.04.2020.

[21] Austrian Institute of Construction Engineering: Vienna, Austria, European Technical Assessment document: *Stora Enso CLT-Rib Panels,* ETA-20/0893 of 2020/12/01.

[22] Austrian Institute of Construction Engineering: Vienna, Austria, European Technical Assessment document: *LIGNATUR-box element (LKE), -surface element (LFE) and -shell element (LSE),* ETA-11/0137 of 04.11.2019.

[23] VTT Expert Services LTD: Espo, Finnland, European Technical Assessment document: *VVR Wood Kerto Ripa Elements,* ETA-17/0941 of 2018/01/15.

[24] A. Hassanieh, Phd Dissertation: *Development of Steel Timber Composite system for large scale construction*, University of New South Wales, Australia, (2017).

[25] S. Navaratnam, D. W. Small, P. Gatheeshgar, K. Poologanathan, J. Thamboo, C. Higgins and P. Mendins, *Development of cross laminated timber-cold formed steel composite beam for floor system to sustainable modular building construction*, Structure 32 (2021) 681-690.

[26] Cepezed Architects, Technical File: *Bouwdeel D drawings,* (2019).

[27] Nationale Staal Prijs 2020, *The Green House, Utrecht*, Available from: https://www.nationalestaalprijs.nl/project/green-house.Accessed: June 2021.

[28] Nationale Staal Prijs 2020, *Bouwdeel D, Delft*, Available from: https://www.nationalestaalprijs.nl/project/bouwdeel-demontabel. Accessed: June 2021.

[29] C. Loss, M. Piazza and R. Zandonini, *Connections for steel-timber hybrid prefabricated buildings, Part-I,* Construction and Building Materials 122 (2016), 781 – 795.

[30] C. Loss, M. Piazza and R. Zandonini, *Connections for steel-timber hybrid prefabricated buildings, Part-II,* Construction and Building Materials 122 (2016), 796 – 808.

[31] A. Hassanieh, H. R. Valipour, M. A. Bradford, *Load-slip behaviour of steel-cross laminated timber (CLT) composite connections,* Journal of Constructional Steel Research 122 (2016), 110 – 121.

[32] A. M. G. Coelho, M. Lawson, D. Lam and J. Yang, Technical Document: *Guidance on demountable composite construction systems for UK practise,* Publication number: SCI P428, Steel Construction Institute, Ascot, UK (2020).

[33] Möhler K., Abdel-Sayed G. and Ehlbeck J, Zur Berechnung doppelschaliger, geleimter Tafelelemente, Holz als Roh- und Werkstoff 21:328-333, (1963).

[34] M. P. Nijgh and M. Veljkovic, *Requirements for oversized holes for reusable steel – concrete composite floor systems,* Structures 24, 489 – 498, (2020).

[35] M. P. Nijgh, Girbacea I. A. and M. Veljkovic, *Elastic behaviour of tapered Composite Beam optimized for reuse,* Engineering Structures, 183:366 – 74, (2019).

[36] F. Csillag and M. Pavlovic, *Push – out behaviour of demountable inject vs blind – bolted connectors in FRP decks,* Composite Structures 270 114043, (2021).

[37] B. Zafari, J. Qureshi, J. T. Mottram and R. Rusev, *Static and fatigue performance of resin injected bolts for a slip and fatigue resistant connection in FRP bridge engineering,* Structures 7 71 – 84, (2016).

[38] M. P. Nijgh, Master's Thesis, *New Materials for Injected Bolted Connections: A Feasibility study for demountable connections,* Delft University of Technology, (2017).

[39] A. M. Koper, Master's Thesis, *Assessment of Epoxy Resins for Injected Bolted Shear Connectors,* Delft University of Technology, (2017).

[40] Holzforschung Austria, *Floor slab - gdmtxn01-00 Storey ceiling, solid wood construction, without, dry, with fill, wood visible.* Available from: https://www.dataholz.eu/bauteile/geschossdecke/detail/kz/gdmtxn01.htm. Accessed: April 2021.

[41] Lignum.ch, *Lignatur_k08__22 box girder.* Available from: https://lignumdata.ch/detail.cfm?page=detail&type=single&uuid=7D8108D2-9E68-3CC9-92D9-6259DB8FB48D&bauteilgruppe=decke. Accessed: April 2021.

[42] MetsaWood, *Kerto Ripa Sound Insulation Online design tool*, [cited: 04/28/2021]; Available from: *http://ripaschuif.nl/indexR.html.* Accessed: April 2021.

[43] Cepezed Architects, *Temporary Courthouse Amsterdam,* Available from: https://www.cepezed.nl/nl/project/tijdelijke-rechtbank-amsterdam/30529/. Accessed: June 2021

[44] IMd Raadgevende Ingeniuers, *IMd develops solution for demountable construction,* March 2016, Available from: https://imdbv.nl/Nieuws/IMd-ontwikkelt-oplossing-demontabel-bouwen-(met-VIDEO). Accessed: June 2021.

[45] Bouwen met Staal, *Temporary Courthouse Amsterdam*, March 2016, Available from: https://www.bouwenmetstaal.nl/publicaties/nieuwsbrief-architect-staal/architect-staal-maart-2016/tijdelijke-rechtbank-amsterdam/. Accessed: June 2021.

[46] Bouw Totaal, Technical File, *Steel and wood provide height and environmental gain, September*, 2016, Available from: https://www.bouwtotaal.nl/2016/09/staal-en-hout-leveren-hoogte-en-milieuwinst-op/. Accessed: June 2021.

[47] IMd Raadgevende Ingeniuers, *StayOkay and Natuurpodium,* Available from: https://imdbv.nl/Projecten/Hergebruik/StayOkay-en-Natuurpodium. Accessed: June 2021.

[48] D.W. Green, J. E. Winandy and D. E. Kretschmann, *Mechanical Properties of Wood,* Wood Handbook, Chapter 4.

[49] M. P. Nijgh, I. A. Girbacea and M. Veljkovic, *Elastic behaviour of a tapered steel – concrete composite beam optimized for reuse,* Engineering Structures 183 366 – 374, (2019).

[50] European Committee for Standardization, Eurocode: *NEN-EN 338 - Structural timber - Strength classes.* 2016, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[51] European Committee for Standardization, Eurocode: *NEN-EN 1194 - Structural timber – Glued Laminated Timber - Strength classes and Determination of Characteristic values,* 1999, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[52] European Committee for Standardization, Eurocode*: NEN-EN 1995-1-1 C1+A1, Design of timber structures – Part 1-1: General - Common rules and rules for buildings*, 2011*,* Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[53] European Committee for Standardization, National Annex *to NEN-EN 1995-1-1+C1+A1 - Design of timber structures - Part 1-1: General - Common rules and rules for buildings (includes NEN-EN 1995-1-1+C1+A1/C1:2012),* 2013, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[54] European Committee for Standardization, Eurocode*: NEN-EN 1995-1-2+C2 Design of timber structures - Part 1-2: General - Structural fire design* 2011, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[55] European Committee for Standardization, Eurocode*: NEN-EN 1990+A1+A1/C2, Basis of structural design,* 2019, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[56] European Committee for Standardization, National annex *to NEN-EN 1990+A1+A1/C2, Eurocode: Basis of structural design,* 2019*,* Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[57] European Committee for Standardization, Eurocode: *NEN-EN 1991-1- 1+C1+C11. Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings,* 2019, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[58] European Committee for Standardization, National annex *to NEN-EN 1991-1-1+C1+C11: Eurocode 1: Actions on the structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings,* 2019, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[59] European Committee for Standardization, *NEN-EN 1991-1-4+C1+C11: Eurocode 1: Actions on the structures - Part 1-4: General actions – Wind Actions,* 2005, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[60] European Committee for Standardization, Eurocode: *NEN-EN 1993 – Design of Steel Structures – Part 1-8: Design of Joints,* 2005, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[61] International Organisation for Standardization, *ISO 898-1:2009 - Mechanical properties of fasteners made of carbon steel and alloy steel - Part 1: Bolts, screws and studs with specified property classes - Coarse thread and fine pitch thread,* (2009).

[62] European Committee for Standardization, Eurocode: *NEN-EN 1090 – Execution of Steel Structures and Aluminium Structures – Part 2: Technical requirements for Steel Structures,* 2018, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[63] European Committee for Standardization, Eurocode: *EN 14399 – High Strength structural bolting assemblies for Preloading – Part 4: System HV – Hexagon bolt and nut assemblies,* 2018, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[64] European Committee for Standardization, Eurocode: *NEN – EN 1994 – Design of Steel and Concrete composite structures – Part 1 – 1: General rules and rules for Buildings,* 2011, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[65] European Committee for Standardization, Eurocode: *NEN – EN 1993 – Design of Steel Structures – Part 1 – 1: General rules and rules for Buildings,* 2016, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands

[66] European Committee for Standardization, Eurocode: *NEN – EN 15978 – Sustainability of Construction works – Assessment of environment performance of buildings – Calculation method,* 2011, Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands

[67] W. W. van Wijnen, Master's Thesis, *Sustainable timber structures: Quantitative research evaluating the potential effects of carbon sequestration and cascading strategies in the Netherlands based on a comparison of the Dutch and European life cycle assessment methodolgies,* Delft University of Technology, (2020).

[68] C. Braendstrup, Master's Thesis, *Conceptual design of a demountable, reusable composite flooring system: Structural behaviour and environmental advantages,* Delft University of Technology, (2017).

[69] M. S. Pavlovic, Phd Thesis, *Resistance of Bolted Shear Connectors in prefabricated steel-concrete composite decks,* University of Belgrade, (2013).

[70] Ruud Binnekaamp, Lecture Slides: *Trade Off Matrices: Preference Measurement*, Construction Technology of Civil Engineering Structures (CIE4170), Delft University of Technology, (2020).

[71] M. P. Nijgh, Lecture Slides: *Introduction to Composite Structures*, Steel Structures 3 (CIE4121), Delft University of Technology, (2021).

[72] M. P. Nijgh, Lecture Slides: *Elastic Design of Composite Floor systems*, Steel Structures 3 (CIE4121), Delft University of Technology, (2021).

[73] M. P. Nijgh, Lecture Slides: *Plastic Design of Composite Floor systems*, Steel Structures 3 (CIE4121), Delft University of Technology, (2021).

[74] M. P. Nijgh, Lecture Slides: *Introduction to Reliability of Steel Structures*, Capita Selecta Steel and Aluminium Structures (CIE5122), Delft University of Technology, (2019).

[75] Yuri De Santis and Massimo Fragiacomo, *Timber to Timber and Steel to Timber screw connections: Derrivation of slip-modulus via beam on elastic foundation model,* Engineering Structures 244 112798 (2021).

[76] H. H. Snijder and H. M. G. M. Steenbergen, Bouwen met Staal, Technical File, *Annex for the Netherlands to Structural Basics (Steel Design 1),* Zoetermeer, Netherlands (2012).

[77] M. V. Leskelä, *Shear connections in composite flexural members of Steel and Concrete*, ECCS - TC11 WG1 Shear Connections (2017).

[78] L. van Glabbeek, Master's Thesis, *Development of an Innovative Floor System: Structural Design and Verification,* Delft University of Technology (2019).

[79] J. Song, S. Kim and S. Oh, *The Compressive stress-strain relationship of Timber*, International Conference on Sustainable Buildings Asia, Rotterdam (2007).

[80] The World Commission on Environment and Development, *Our Common Future,* New York: Oxford University Press, ISBN: 978-0192820808 (1987).

[81] Jonkers, H.M., *Materials and Ecological Engineering*, Course Reader, Materials and Ecological Engineering (CIE4100), Delft University of Technology (2018).

[82] Sustainable Engineering: The Future of Structural Design J.A. Ochsendorf 1… P. Peters, et al., Duurzaam Construeren, 10 jaar later. in Cement. 2019, Aeneas Media: 's-Hertogenbosch, the Netherlands. p. 42-47.

[83] Ellen McArthur Foundation, *Towards the Circular Economy: Economic and business rationale for an accelerated transition,* (2013).

[84] J. Kirchherr, D. Reike and M. Hekkert, *Conceptualising the Circular Economy: An analysis of 114 definitions,* Resources, Conservations and Recycling 127 221-232 (2017).

[85] M. Gharfalkar, R. Court, C. Campbell, Z. Ali and G. Hillier, *Analysis of waste hierarchy in European waste directive 2008/98/EC,* Waste Management 38 305-313 (2015).

[86] European Parliament and European Union, *Waste Formwork Directive 2008/98/EC,* November 2008.

[87] E. Durmisevic, PhD Thesis: *Transformable building structures,* Delft University of Technology, (February 2006).

[88] Alba Concepts, Dutch Green Building Council, Rijksdienst voor Onderneemend Nederland and W/E Adviseurs, Technical Document: *Meetmethodiek Losmaakbaarheid*, Eindhoven (2019).

[89] Madaster Services BV, Technical File: *Explanation of Madaster building circularity indicator,* Utrecht (2018).

[90] P. Hadril, A. Talja, M.Walhström, S. Huuhka, J. Lahdensivu and J. Pikkuvirta, *Re-use of structural elements: Environmentally efficient recovery of building components,* VTT Technical Research Centre of Finland, Espo (2014).

[91] R. H. Falk, D.G. Maul, S. M. Cramer, J. Evans and V. Herian, *Engineering properties of douglas-fir lumber reclaimed from deconstructed buildings*, United States Department of Agriculture, Forest Service, Forest Products Laboratory: Washington D.C. (2008).

[92] K. Crews and C. MacKenzie, *Development of grading rules for re-cycled timber used in structural applications*, 10th World Conference on Timber Engineering, Miyazaki, Japan (2008).

[93] K. Crews, D. Hayward, and C. MacKenzie, *Interim Industry Standard Recycled Timber – Visually Stress Graded Recycled Timber for Structural Purposes*, Forest & Wood Products Australia, ISBN: 978-1-920883-35-5, Melbourne, Australia (2008).

[94] D. G. Brown, R. J. Pimentel, M. R. Sansom, Technical Document: *Structural Steel Reuse: Assessment, testing and design principles,* Steel Construction Institute, Ascot (2019).

[95] European Committee for Standardization, Draft of Eurocode: *EN – 17662 – Execution of steel structures and aluminium structures – Environmental Product Declarations – Product Category rules complementary to EN – 15804 for steel, iron and aluminium structural products used in construction works ,* (2021), Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[96] European Committee for Standardization, Eurocode: *NEN – EN – 15804 + A1 – Sustainability of construction works – Environmental Product Declarations – Core rules for the product category of construction products,* (2019), Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[97] European Committee for Standardization, Eurocode: *NEN – EN – 15804 + A2 – Sustainability of construction works – Environmental Product Declarations – Core rules for the product category of construction products,* (2019), Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[98] Consolis VBI, Technical File: *Hollow Core Slab Floor 260: Product Data Sheet,* (2019).

[99] Tata Steel, Technical File: *ComFlor manual: Composite Floor decking design and technical information,* (2017).

[100] The Norwegian EPD Foundation, Environment Product Declaration: *Pine and Spruce Lumber from Bergen Holme AS,* Majorstruen, Norway (2021).

[101] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Egger Sawn Timber*, EPD-EGG-20200248-IBC1-DE, Germany, (2021).

[102] EPD Danmark, Environmental Product Declaration: *Møbelindustrien – Sawn and Dried Construction wood products of Pine and Spruce*, EPD No: MD-20002-EN, Denmark, (2020).

[103] The EPD International, Environmental Product Declaration: *Swedish Wood – Swedish sawn dried timber of spruce or pine*, SP-02527, Sweden, (2021).

[104] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Binderholz Dolid Wood Panels*, EPD-BBS-20190170-IBA1-EN, Germany, (2019).

[105] European Committee for Standardization, Eurocode: *NEN – EN – 16449 – Wood and wood-based products – Calculation of the Biogenic Carbon content of wood and conversion to carbon dioxide,* (2014), Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[106] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Give Steel A/S*, MD-20042-EN, Germany, (2020).

[107] Tata Steel UK, Environmental Product Declaration: *Structural Hollow Sections*, UK, (2019).

[108] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *ArcelorMittal Structural Steel Sections*, EPD-ARM-20190015-CBD1-EN, Germany, (2019).

[109] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Bauforuhmstaal Hot rolled steel sections and heavy plates*, EPD-BFS-20180167-IBG1-EN, Germany, (2018).

[110] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *UK manufactured generic ready-mix concrete – produced by BMRCA*, EPD-RMC-20180095-CBG2-EN, UK, (2018).

[111] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Concrete C3/37 – Informations Zentrum Beton GmbH*, EPD-IZB-20180102-IBG1-DE, Germany, (2018).

[112] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Eurospan raw chipboard – Frtiz Egger GmbH*, EPD-EGG-20200249-IBC1-DE, Germany, (2021).

[113] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Mineral wool insulation material in high bulk density range – FMI Association of Mineral Wool Industry*, EPD-EGG-20210021-IBG1-DE, Germany, (2021).

[114] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Gypsum Plasterboard – Knauf Bulgaria EOOD*, EPD-KNB-20190071-IAC1-EN, Bulgaria, (2019).

[115] Tata Steel UK, Environmental Product Declaration: *ComFlor60 1 steel structural floor deck*, UK, (2021).

[116] Institut Bauen und Umwelt ev (IBU), Environmental Product Declaration: *Reinforcing Steel bars – ArcelorMittal*, EPD-ARM-20160051-IBD3-EN, Luxembourg, (2016).

[117] VBI Consolis, Environmental Product Declaration: *(A260) Channel Plate Floor 260*, EPD-NIBE-20200708-9564, Netherlands, (2020).

[118] Ministerie van Infrastructuur en Milieu en het ministerie van Economische Zaken, *Nederland circulair in 2050*, the Netherlands (2016).

[119] Statistical Office of the European Union (Eurostat), Municipal wastes by waste management operations, Available from: http://appsso.eurostat.ec.europa.eu/nui/show.do?dataset=env_wasmun, Accessed: November 2021.

[120] World Green Building Council, European Advocacy Manifesto – *A Sustainable built environment at the heart of Europe's future,* (2019).

[121] European Commission, Vision for a long-term EU strategy for reducing greenhouse gas emissions – *A Clean planet for all – A European strategic long-term vision for a prosperous, modern, competitive and climate neutral economy,* Document 52018DC0773, (2018).

[122] Heyne Tillet Steel, Technical File: *2200 Timber Square,* Available from: https://hts.uk.com/projects/timber-square, Accessed: July 2021.

[123] Waugh Thistleton Architects, Technical File: *6 Orsman Road,* Available from: https://waughthistleton.com/6-orsman-road/, Accessed: July 2021.

[124] MetsäWood, Technical File: *Hurlingham Racquet Centre,* Available from: https://www.metsawood.com/global/news-media/references/Pages/Hurlingham-Kerto-LVL-for-roof-structure.aspx, Accessed: July 2021.

[125] R. V. M. Coebelens, Master's Thesis: *Building Light and Comfortabel – Concept development of a lightweight steel timber building system regarding human induced vibrations,* Delft University of Technology, (2018).

[126] D. J. Odeh and P. Kuehnel, *The hybrid CLT-steel residence hall,* Rhode Island School of Design, (2019).

[127] S. Manfredi, K. Allacker, K. Chomkhamrsi, N. Pelletier and D. M. de Souza, *Product Environment Footprint Guide,* European Commission, Joint Research Centre, (2012).

[128] IMd Raadgevende Ingenieurs, Website Image – *Bouwdeel D*, Available from: https://imdbv.nl/Projecten/Kantoren/Bouwdeel-D(emontabel), Accessed: November (2012).

[129] Y. Niu, K. Raasi, M. Hughes, M. Halme and G. Fink, *Prolonging life cycles of construction materials and combating climate change by cascading – The case of reusing timber in Finland*, Resources, Conservation and Recycling, 170 105555, (2021).

[130] A. Cavalli, D. Cibecchini, M. Togni and H. S. Sousa, *A review on the Mechanical properties of aged wood and salvaged timber*, Construction and Building Materials, 114 681-687, (2016).

[131] European Committee for Standardization, Eurocode: *NEN – EN – 14081 – Timber structures – Strength graded structural timber with rectangular cross section – Part 1: General requirements,* (2019), Koninklijk Nederlands Normalisatie Instituut: Delft, the Netherlands.

[132] Bouwen met Staal, Technical File, *Verdiepingbouw met Staal – Ontwerpboek voor Archittecten*, (2005).

[133] The Building Informtation Foundation RTS, Environmental Product Declaration: *Finnish sawn dried timber of spruce and pine*, RTS-27-19, Finland, (2018).

[134] The EPD International, Environmental Product Declaration: *Classic sawn timber by Stora Enso*, SP-02150, Sweden, (2021).

[135] EPD Danmark, Environmental Product Declaration: *DS Staalconstruktion A/S – Structural Steel*, MD-21007-EN, Denmark, (2021).

[136] EPD International, Environmental Product Declaration: *BE Group Sverige AB – Steel Beams*, SP-02936-EN, Denmark, (2021).

[137] EPD International, Environmental Product Declaration: *Holcim Romania – Ready Mix Concrete*, SP-00526-EN, Sweden, (2020).

[138] EPD Danmark, Environmental Product Declaration: *CRH Concrete A/S – Hollow Core Slab Elements*, MD-21065-DA, Denmark, (2020).

[139] EPD International, Environmental Product Declaration: *Perdanga UAB – Precast concrete products: Hollow Core Slabs*, SP-04749, Lithuania, (2021).

[140] EPD Norway, Environmental Product Declaration: *Åkra Sementstøperi AS – Concrete M45/40*, NEPD-2849-1541-NO, Norway, (2021).

[141] EPD International, Environmental Product Declaration: *INHUS Prefab UAB – Precast concrete hollow core slabs*, SP-03858, Lithuania, (2021).

[142] EPD International, Environmental Product Declaration: *Skandinaviska Byggelement – Hollow Core Slabs*, SP-01519, Sweden, (2019).

[143] New Energy Efficient Demonstration for Buildings, *Economic and Market Analysis of the EU Building Sector*, NEED4B_WP8_T8.1_D8.1, (2015).

Appendices

A. Technical Data on Timber Decks

A.1 Timber Strength Classes

For *CLT* elements, strength class C24 has been adopted as per the recommendations of [5]. Consequently, all parameters have been taken from NEN-EN 338 [50], except the mean and characteristic densities. The values 510 and 470 kg/m^3 have been taken respectively, as given in [5]. These values are different from typical C24 timber as it accounts for the amount of adhesives required for producing *CLT* panels. The webs of *CLT* rib panels are adopted as GL24h, as recommended [6]. All parameters for GL24h have been taken from NEN-EN 1194 [51].

For Lignatur elements, the softwood planks also belong to strength class C24 [22]. Kerto *LVL* is made of 3 mm veneers of European softwood. For the kerto-S beams and kerto-Q panels, the properties of the elements depend on the orientation of the layup and its thickness, and their properties have been adopted from [8]. The strength and stiffness parameters, and the densities of the materials adopted in the timber floor elements have been tabulated below in Table A.1.

Table A.1: Properties of Timber Decks

		CLT		Lignatur	Kerto LVL		
Strength Class		*C24*	*GL24h*	*C24*	*Kerto-Q Panels*		*Kerto-S*
Thickness [mm]					*21-24*	*27-75*	*21-75*
Strength Parameters [N/mm^2]							
Bending	$f_{m,k}$	24	24	24	28	32	44
Tension //	$f_{t,0,k}$	14	16.5	14	19	26	35
Tension ⊥	$f_{t,90,k}$	0.4	0.4	0.4	6	6	0.8
Compression //	$f_{c,0,k}$	21	24	21	19	26	35
Compression ⊥	$f_{c,90,k}$	2.5	2.7	2.5	9	9	6
Shear	$f_{v,k}$	4	2.7	4	4.5	4.5	4.2
Stiffness Parameters [N/mm^2]							
Mean E-Modulus //	$E_{0,mean}$	11000	11600	11000	10000	10500	13800
Mean E-Modulus ⊥	$E_{90,mean}$	370	390	370	2400	2400	430
5% E-Modulus //	$E_{0,0.05}$	7400	9400	7400	-	-	-
Mean Shear Modulus	G_{mean}	690	720	690	600	600	600
Densities [kg/m^3]							
Mean Density	ρ_{mean}	510	460	420	510	510	510
5% Density	ρ_k	470	380	350	480	480	480

A.2 CLT Elements by Stora Enso

The *CLT* panels are of strength class C24. Solid *CLT* panels are classified as C panels (with the cover layer timber grain direction perpendicular to the length of the element) and as L panels (with the cover layer timber grain direction along the length of the element). These are shown below in Figure A.1.

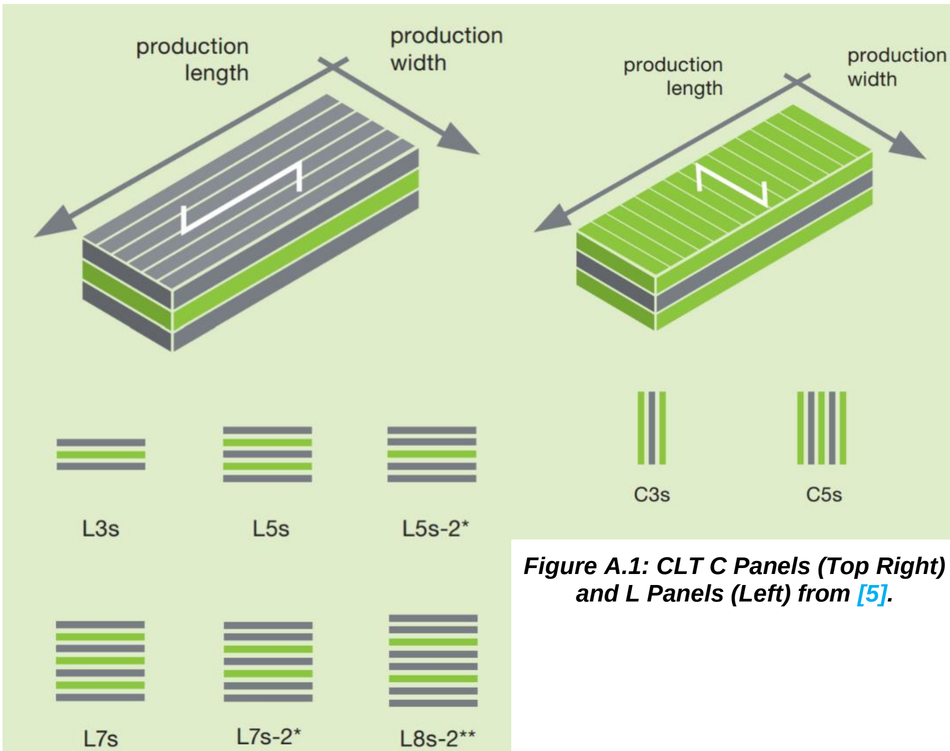


Figure A.1: CLT C Panels (Top Right) and L Panels (Left) from [5].

The *CLT* Rib elements (open and closed) are made by bonding the *CLT* panels (flanges) to *GLT* ribs (webs), to create a cohesive composite unit. The *GLT* webs belong to strength class GL24h. The information on the geometric properties of *CLT* solid and rib elements, as given in the technical brochure by Stora Enso [5,6] and the respective European Technical Assessment (*ETA*) documents [20,21] is given below in Table A.2.

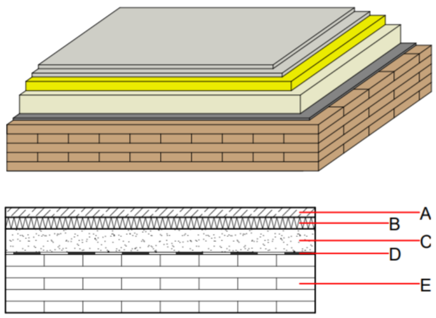
Table A.2: Geometric Properties of CLT Floor Elements

Parameter	**Solid Panels**		
Length	*Up to 16 m*	***Thickness of Lamella***	*20 – 80 mm*
Width	*2.25 - 3.45 m*	***Thickness of Slab***	*60 – 320 mm*
	Open Rib	**Closed Rib**	
Top Flange Thickness	*60-140 mm*	*60-120 mm*	
Bottom Flange Thickness	*NA*	*60-80 mm*	
Web Thickness	*80-200 mm*	*80-120 mm*	
Web Height	*160-480 mm*	*160-400 mm*	
Rib Spacing	*400-800 mm*	*400-800 mm*	
Width	*680-2480 mm*	*680-2480 mm*	

Table A.3: Sound Insulation for CLT Elements

Solid CLT Panels

Figure A.2: Dry Screed adopted for CLT solid panels [40].



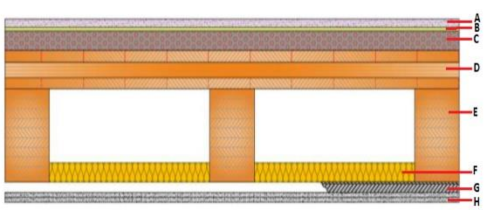
R_w (C;C_{tr}): 62(-5;13) > 54 dB
$L_{n,w}$(C_I): 50(-1) < 52 dB

Layers	Material	Thickness [mm]	Densities [kg/m³]	Areal Weight [kg/m²]
A	*Dry Screed*	25	900	22.5
B	*Impact sound insulating Mineral Wool (s = 40 MN/m³)*	30	160	4.8
C	*Elastically Bound Fill*	60	1500	90
D	*Trickle Protection*	0.2	-	-
E	**CLT Solid Panel**	**140**	500	

***Additional Dead Load due to Sound Insulation:* 117.3 kg/m²**

Open Rib CLT Panels

Figure A.3: Dry Screed adopted for CLT Open Rib panels [21].



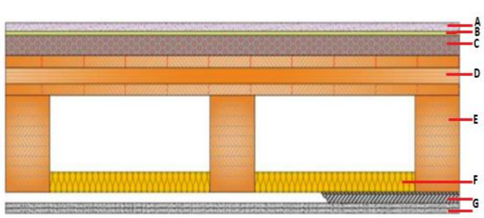
R_w (C;C_{tr}): 69(-6;-14) > 54 dB
$L_{n,w}$(C,C_I,50): 45(3,9) < 52 dB

Layers	Material	Thickness [mm]	Densities [kg/m³]	Areal Weight [kg/m²]
A	*Dry Screed*	20	900	18
B	*Impact sound insulating Mineral Wool*	10	160	1.6
C	*Loose Fill*	50	1500	75
D	**CLT Solid Panel**	**100**	500	-
E	**GLT Webs (x240)**	**160**	460	
F	*Mineral Wool*	50	16	0.8
G	*Resilient Metal Channel*	27	-	-
H	*Gypsum Plasterboard*	25	680	17

***Additional Dead Load due to Sound Insulation:* 112.4 kg/m²**

Closed Rib CLT Panels

Figure A.4: Dry Screed adopted for CLT Closed Rib panels [21].



***R_w (C;C_{tr}): 69(-6;-14) > 54 dB**
$L_{n,w}$(C,C_I,50): 45(3,9) < 52 dB

Layers	Material	Thickness [mm]	Densities [kg/m³]	Areal Weight [kg/m²]
A	*Dry Screed*	20	900	18
B	*Impact sound insulating Mineral Wool*	10	160	1.6
C	*Loose Fill*	50	1500	75
D	**CLT Solid Panel**	**100**	500	-
E	**GLT Webs (x240)**	**160**	460	-
F	*Mineral Wool*	50	16	0.8
G	**CLT Solid Panel**	**50**	500	-

***Additional Dead Load due to Sound Insulation:* 95.4 kg/m²**

Sound Insulation: Dry screed system is adopted here. The values of sound insulation mentioned above are from testing [40,21]. A specific detail was not found for closed rib panels. Hence it is assumed that the same dry screed system as for open rib panels is used. The bottom layer of suspended gypsum is substituted with the bottom flange made of *CLT*. For further reference, it assumed that the dry screed system coupled with *CLT* elements of larger dimensions (than given above) will successfully fulfil the building decree requirements for sound insulation.

Fire Protection: For *CLT* solid panels, the span tables are provided for a fire safety of R90, and hence no extra fire protection is required. For the *CLT* rib panels (open and closed), span tables for R0 fire safety are taken. Additional fire protection is provided in the form of gypsum plasterboards (type A, F or H) to make the fire safety of all the slabs uniform. The starting time of charring of the protected timber member (in minutes) is given as follows [54]:

$$t_{ch} = 2.8 * h_p - 14 \qquad \textbf{(Eq 7)}$$

Where,

$$h_p \textit{ is the thickness of the gyspum boards in mm}$$

The thickness of the gypsum board required is calculated by substituting $t_{ch} = 90\ minutes$

$h_p = (90 + 14)/2.8 = 37.14 \sim 40\ mm$

When gypsum plasterboard is used as sound insulation, only the extra amount to have 40 mm gypsum layer is used. Table A.4 below summarizes the requirements of the *CLT* elements to meet the fire safety value of 90 minutes.

Table A.4: Fire Protection for CLT Elements

	CLT Solid Panels	**CLT Open Rib Panels**	**CLT Closed Rib Panels**
Thickness of Gypsum Plasterboard [mm]	Not Required	**15*	40
Density [kg/m³]		680	
Total Additional Load due to Fire Protection [kg/m²] :	**0 kg/m²**	**10.2 kg/m²**	**27.2 kg/m²**

Span tables have been extracted from [5,6,20,21], according to the functional unit of the *MCA* in Chapter 4 and is given below in Table A.5.

Table A.5: Span Tables for CLT solid and rib elements by Stora Enso [5,6,20,21]

		Solid Panels	Open Rib Panels	Closed Rib Panels
ULS Checks:				
1.	*Flexural Stress*			
2.	*Shear Stress*			
SLS Checks:				
3.	*Instantaneous Deflection*	*< L/300*	*< L/300*	
4.	*Final Deflection*	*< L/250*	*< L/150*	
5.	*Vibration*	*Floor class I ζ = 4%, 5 cm cement screed*	*Floor class I Screed = 6 cm E = 26,000 N/mm²* *Damping coefficient ζ = 4%*	
Building Physics:				
6.	*Sound Insulation: R_w > 54 dB, $L_{n,w}$ < 52 dB (According to Dutch Building Decree [1])*			
7.	*Fire Safety: R90*			
Live Loads				
1.	*Imposed Floor Load : 3kN/m2 (Category B , according to Eurocode 1 [57])*			
Permanent Loads				
2.	*Floor Finish: 0.5 kN/m2*			
3.	*Installations: 0.5 kN/m2*			
4.	*Slab Dead Load: 5 kN/m3 (CLT), 4.5 kN/m3 (GLT)*			
6.	*Sound Insulation*	*1.15 kN/m2*	*1.1 kN/m2*	*0.94 kN/m2*
5.	*Fire Protection*	*0 kN/m2*	*0.1 kN/m2*	*0.27 kN/m2*
Total Dead Load:				
Slab Weight + :		***2.15 kN/m²***	***2.2 kN/m²***	***2.21 kN/m²***

Spans [m]	Solid Panels [mm]	Open Rib Panels [mm] *Spacing of Ribs: 800 mm* *Internal and External Web Thickness: 120, 80 mm* Top Flange	 Total Height	Closed Rib Panels [mm] Top Flange	 Bottom Flange	 Total Height
3	*120L3s*	-	-	-	-	-
3.5	*120L3s*	-	-	-	-	-
4	*140L5s*	-	-	-	-	-
4.5	*160L5s*	-	-	-	-	-
5	*180L5s*	*100L3s*	*260*	-	-	-
5.5	*200L5s*	*120L3s*	*280*	-	-	-
6	*220L7s-2*	*120L3s*	*320*	*60L3s*	*60L3s*	*280*
6.5	*240L7s-2*	*120L3s*	*320*	*60L3s*	*60L3s*	*320*
7	*260Ls-2*	*120L3s*	*360*	*60L3s*	*60L3s*	*320*
7.5	-	*100L3s*	*380*	*80L3s*	*60L3s*	*340*
8.5	-	*120L3s*	*400*	*60L3s*	*60L3s*	*360*
9	-	*100L3s*	*420*	*60L3s*	*60L3s*	*400*
9.5	-	*120L3s*	*440*	*60L3s*	*60L3s*	*400*
10	-	*120L3s*	*480*	*80L3s*	*60L3s*	*420*
10.5	-	*120L3s*	*520*	*80L3s*	*60L3s*	*460*
11	-	*120L3s*	*520*	*90L3s*	*60L3s*	*470*
11.5	-	*120L3s*	*560*	*80L3s*	*60L3s*	*500*
12	-	*100L3s*	*580*	*80L3s*	*60L3s*	*540*

A.3 Lignatur Elements

Lignatur elements use softwood planks of strength class C24, glued into box-shaped elements. The geometrical properties of the Lignatur elements *LKE* (Lignatur Box Elements) and *LFE* (Lignatur Surface Elements) are given below in Table A.6, and are according to the information provided in the Lignatur *ETA* document [22]. The main difference between the two is that *LKE* comes in units of 1 box (with width up to 250 mm), whereas *LFE* offers the possibility of making such elements with up to 4 boxes (i.e., up to 1000 mm width per element)

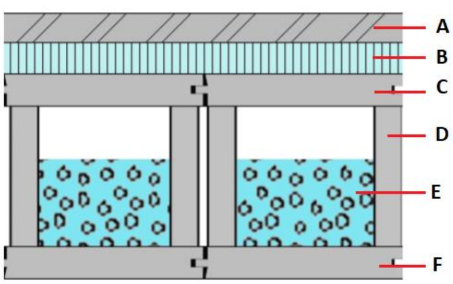
Table A.6: Geometric Properties of Lignatur Floor Elements

Parameter	LKE	LFE
Top Flange Thickness	*25-82 mm*	*25-82 mm*
Bottom Flange Thickness	*25-82 mm*	*25-82 mm*
Web Thickness	*27-33 mm*	*27-80 mm*
Total Height	*120-400 mm*	*120-360 mm*
Number of Boxes	*1*	*Up to 4*
Width	*< 250 mm*	*< 1000 mm*
Length	*< 18 m*	*< 18 m*
Transverse Stiffener Spacing	*< 1.2 m*	*< 1.2 m*

Table A.7: Sound Insulation for Lignatur Elements

LKE / LFE

Figure A.5: Dry Screed adopted for Lignatur [41].



R_w (C;$C_{50-3150}$): 61(-6;-9) > 54 dB
$L_{n,w}$(C_I,$C_{150-2500}$): 52(2,5) < 52 dB

Layers	Material	Thickness [mm]	Densities [kg/m^3]	Areal Weight [kg/m^2]
A	*Chipboard*	28	680	19.02
B	*Impact sound insulating Mineral Wool*	30	160	4.8
C	**Lignatur**	**31**	**420**	**-**
D	**Lignatur**	**138**	**420**	**-**
E	*Fill (Gravel)*	87	1400	88.9
F	**Lignatur**	**31**	**420**	**-**

***Additional Dead Load due to Sound Insulation:* 112.7 kg/m^2**

<u>Sound Insulation:</u> Dry screed system is adopted here. The values of sound insulation mentioned above are from testing [41]. The same set up is used for both *LFE* and *LKE*. Same approach as in Section A.2.

<u>Fire Protection:</u> Same approach as in Section A.2. Thickness of gypsum board: 40 mm. The additional dead load due to 40 mm gypsum 27.2 kg/m^2.

Span tables for Lignatur elements, as given in the *ETA* document [22], based on the requirement for the functional units of the *MCA* in Chapter 4 is given below in Table A.8:

Table A.8: Span Tables for Lignatur Elements [22].

LKE / LFE

ULS Checks:

1. *Flexural Stress*
2. *Shear Stress*

SLS Checks:

3. *Instantaneous Deflection: < L/300*

Building Physics:

4. *Sound Insulation: Rw > 54 dB, Ln,w < 52 dB (According to Dutch Building Decree [1])*
5. *Fire Safety: R90*

Live Loads:

1. *Imposed Floor Load : 3kN/m2 (Category B , according to Eurocode 1 [57])*

Permanent Loads:

2. *Floor Finish: 0.5 kN/m2*
3. *Slab Dead Load: 4.2 kN/m3 (C24)*
4. *Sound Insulation: 1.1 kN/m2*
5. *Fire Protection: 0.5 kN/m2*

➔ ***Total Dead Load: Slab Weight + 5.1 kN/m2***

Spans [m]	**LKE [mm]** *Top and Bottom Flange Thickness: 31 mm* *Width per Box: 250 mm* *Rib Thickness: 27 mm* *Max no of Boxes: 1* **Total Height**	**LFE [mm]** *Rib Thickness: 31mm* *Max no of Boxes: 4* **Total Height**
4.1	*120*	*120*
4.7	*140*	*140*
5.3	*160*	*160*
5.8	*180*	*180*
6.3	*200*	*200*
6.8	*220*	*220*
7.5	*240*	*240*
8.5	*280*	*280*
9.5	*320*	*320*

A.4 Kerto Ripa Floor Elements by MetsäWood

Kerto *LVL* is produced by bonding 3mm thick rotary peeled softwood veneers. The strength class of *LVL* depends on the layup of the product used. For Kerto Ripa elements, the flanges are made of Kerto-Q panels, and the ribs are made of Kerto-S beams. The main difference is that for Kerto-Q, 20% of the veneers are bonded crosswise, thus giving it higher transverse load carrying capacity. For Kerto-S beams, all the veneers are oriented longitudinally. The properties of Kerto *LVL* related to structural design are mentioned in Section A.1.

Kerto Ripa elements are produced by adhesive bonding of Kerto-Q panels (flanges) and Kerto-S beams (ribs) to form composite slab units. The ribs may have one sided taper up to 10 degrees, which can help in optimizing the shape of the roof/floor elements. The information on the geometric properties of Kerto Ripa elements, and general information on Kerto *LVL*, as given in the technical brochures from MetsäWood [7,8] and the respective *ETA* document [23] is given below in Table A.9.

Table A.9: Geometric Properties of Kerto-Ripa Elements.

Kerto-S Beams			
Length	*2 - 25 m*	Thickness	*27 – 75 mm*
Width	*40 – 2500 mm*		
Kerto-Q Panels			
Length	*2 - 25 m*	Thickness of Slab	*27 – 75 mm*
Width	*200 – 2500 mm*		
Kerto Ripa Floor Elements			
Parameter	**Open Rib**	**Closed Rib Panels**	**Open Box Panels**
Length	*4 - 24 m*		
Width	*2 – 4 m*		
Total Height	*120 – 1500 mm*		
Web Thickness	*39 – 75 mm*		
Web Height	*100 – 900 mm*		
Top Flange Thickness	*19 – 68 mm*		
Bottom Flange Thickness	*19 – 68 mm*		
Rib Spacing	*400-800 mm*		

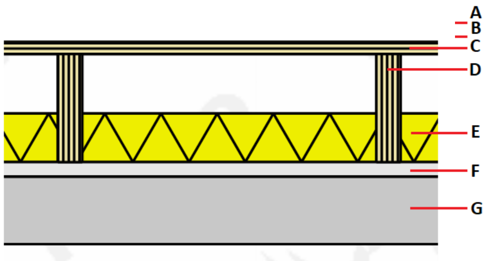
Sound Insulation: Dry screed system is adopted here. The values of sound insulation mentioned below in table are from the online tool Ripaschuif by MetsäWood [42]. A specific detail was found only for open rib elements. For box elements, the thickness of the bottom gypsum plasterboard is reduced by the thickness of the bottom flange. For open box elements, the same layup as for open rib elements is used. For further reference, it assumed that the dry screed system coupled with Kerto RIPA elements of larger dimensions (than given above) will successfully fulfil the building requirements for sound insulation.

Fire Protection: Same approach as in Section A.2. As the gypsum plasterboard thickness is greater than 40 mm for all 3 elements, no extra fire protection is required.

Table A.10: Sound Insulation for Kerto-Ripa Elements.

Kerto Ripa Open Rib Elements

Figure A.6: Dry Screed adopted for Kerto-Ripa Open Rib Elements [42].



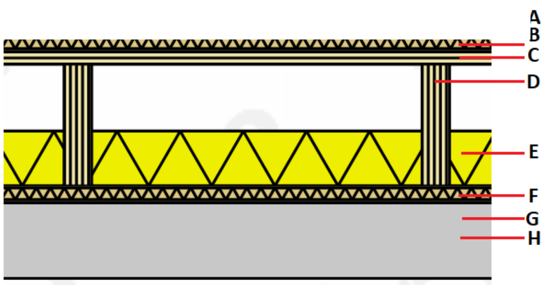
R_w: 65 > 54 dB
$L_{n,w}$: 51 < 52 dB

Layers	Material	Thickness [mm]	Densities [kg/m³]	Areal Weight [kg/m²]
A	*Gypsum Fibre Board*	25	1150	28.75
B	*Wood Fibre Insulation*	20	45	0.9
C	**Kerto-Q Panel**	**25**	**510**	**-**
D	**Kerto-S Beam (x240)**	**45**	**510**	**-**
E	Mineral Wool	90	16	1.44
F	Resilient Metal Chanel	27	7850	-
G	Gypsum Plasterboard	125	680	85

***Additional Dead Load due to Sound Insulation:* 116.1 kg/m²**

Kerto-Ripa Box Elements

Figure A7: Kerto-Ripa Box Elements.



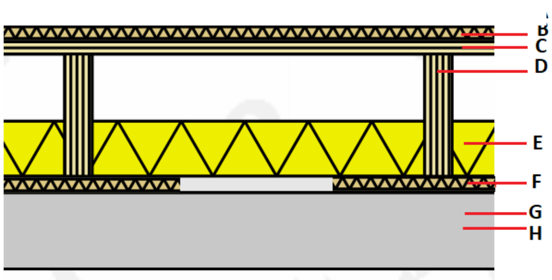
***R_w: 65 > 54 dB**
***$L_{n,w}$: 51 < 52 dB**

Layers	Material	Thickness [mm]	Densities [kg/m³]	Areal Weight [kg/m²]
A	*Gypsum Fibre Board*	25	1150	28.75
B	*Wood Fibre Insulation*	20	45	0.9
C	**Kerto-Q Panel**	**25**	**510**	**-**
D	**Kerto-S Beam (x240)**	**45**	**510**	**-**
E	Mineral Wool	90	16	1.44
F	**Kerto-Q Panel**	**25**	**510**	**-**
G	Resilient Metal Chanel	27	7850	-
H	Gypsum Plasterboard	100	680	68

***Additional Dead Load due to Sound Insulation:* 99.1 kg/m²**

Kerto-Ripa Open Box Elements

Figure A.8: Dry Screed adopted for Kerto-Ripa Open Box Elements.



***R_w: 65 > 54 dB**
***$L_{n,w}$: 51 < 52 dB**

Layers	Material	Thickness [mm]	Densities [kg/m³]	Areal Weight [kg/m²]
A	*Gypsum Fibre Board*	25	1150	28.75
B	*Wood Fibre Insulation*	20	45	0.9
C	**Kerto-Q Panel**	**25**	**510**	**-**
D	**Kerto-S Beam (x240)**	**45**	**510**	**-**
E	Mineral Wool	90	16	1.44
F	**Kerto-Q Panel**	**25**	**510**	**-**
G	Resilient Metal Chanel	27	7850	-
H	Gypsum Plasterboard	125	680	85

***Additional Dead Load due to Sound Insulation:* 116.1 kg/m²**

Based on the above assumptions, the appropriate span tables have been created using FinnWood software by MetsäWood and is given below in Table A.11.

Table A.11: Span Tables for Kerto-Ripa Elements

Kerto Ripa Floor Elements	**Solid Panels**	**Open Rib Panels**	**Closed Rib Panels**
<u>ULS Checks:</u>			
1) Flexural Stress			
2) Shear Stress			
<u>SLS Checks:</u>			
3) Instantaneous Deflection:		*< L/300*	
4) Final Deflection:		*< L/250*	
5) Vibrations:		*f_1 > 9 Hz and U < 0.5 mm*	
<u>Building Physics:</u>			
6) Sound Insulation: R_w > 54 dB, $L_{n,w}$ < 52 dB (According to Dutch Building Decree [1])			
7) Fire Safety: R90			
<u>Live Loads:</u>			
1) Imposed Floor Load : 3kN/m^2 (Category B , according to Eurocode 1 [102])			
<u>Permanent Loads:</u>			
1) Floor Finish: 0.5 kN/m^2			
2) Installations: 0.5 kN/m^2			
3) Slab Dead Load: 5.1 kN/m^3 (Kerto LVL)			
4) Sound Insulation:	*1.14 kN/m^2*	*0.97*	*1.14*
5) Fire Protection:	*0*	*0*	*0*
→Total Dead Load: Slab Weight +:	***2.14***	***1.97***	***2.14***

Spans [m]	**Open Rib Elements [mm]** *Total Width: 2500 mm* *Top Flange Thickness: 25 mm* *Number of Ribs: 5 mm*		**Box Elements [mm]** *Bottom Flange: 2500x25 mm*		**Open Box Elements [mm]** *Bottom Flange: 1500x43 mm*	
	Web Thickness	**Total Height**	**Web Thickness**	**Total Height**	**Web Thickness**	**Total Height**
4	*51*	*285*	*51*	*200*	*45*	*268*
5	*51*	*325*	*51*	*250*	*45*	*293*
6	*51*	*385*	*51*	*350*	*57*	*328*
7	*51*	*475*	*51*	*410*	*45*	*428*
8	*51*	*625*	*51*	*650*	*57*	*518*
9	*-*	*-*	*51*	*650*	*-*	*-*

B. Multi Criteria Analysis

B.1 Dimensioning the Structural Elements

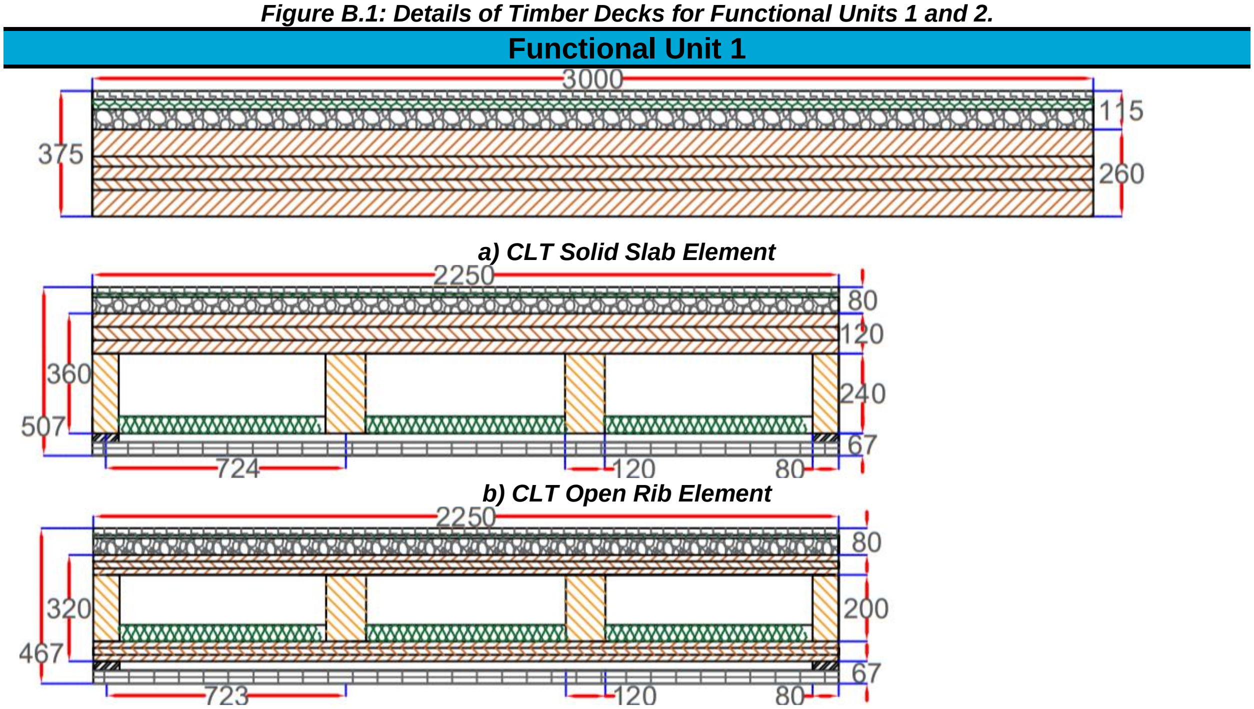
B.1.1 Dimensioning the Timber Decks

The timber decks are dimensioned using manufacturer's span tables. As mentioned in Appendix A.2, *CLT* solid slabs have fire safe time of 90 minutes. All the remaining timber decks inherently have no fire safe time. All the criteria used for design in the span tables are given in Appendix A and are according to the loads adopted in the Functional Unit. The drawings for the timber decks including the total layup for sound insulation and fire safety is given below in Table B.1.

Table B.1: Key for Drawings of Timber Sections

Structural Timber (CLT, Glulam, Spruce, LVL)	Fill (Elastically bound, Loose)
Dry Screed (Gypsum, Chipboard)	Suspended Ceiling (Gypsum Plasterboard Type A/F)
Sound Insulation (Mineral Wool, Wood Fibre)	Steel

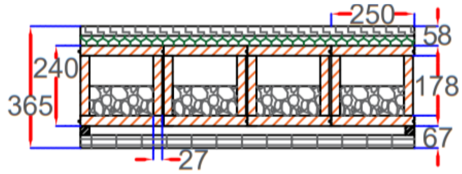
Figure B.1: Details of Timber Decks for Functional Units 1 and 2.



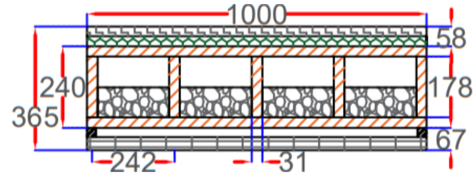
a) CLT Solid Slab Element

b) CLT Open Rib Element

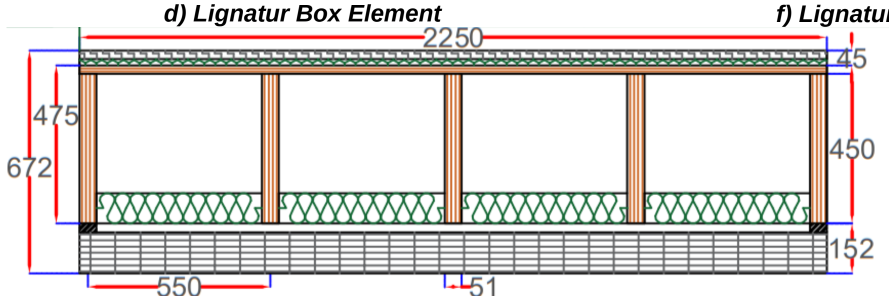
c) CLT Box Element



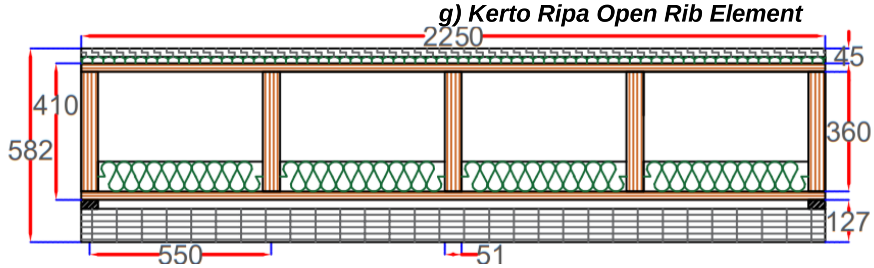
d) ***Lignatur Box Element***



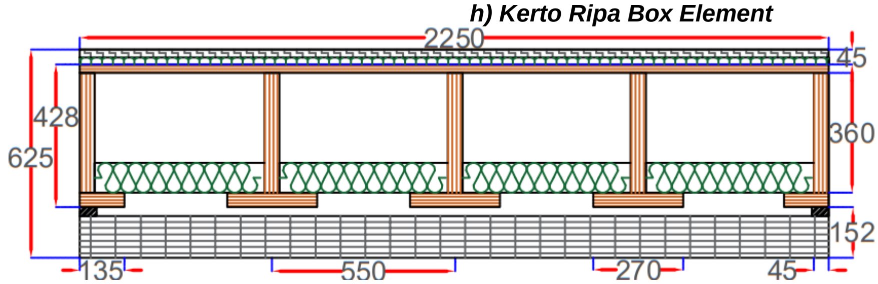
f) ***Lignatur Surface Element***



g) ***Kerto Ripa Open Rib Element***

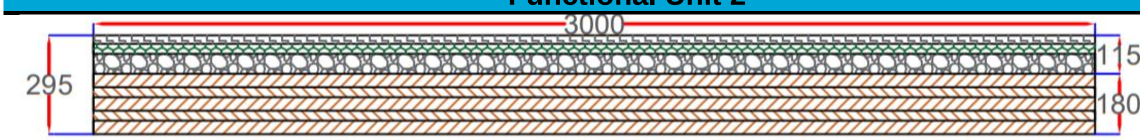


h) ***Kerto Ripa Box Element***

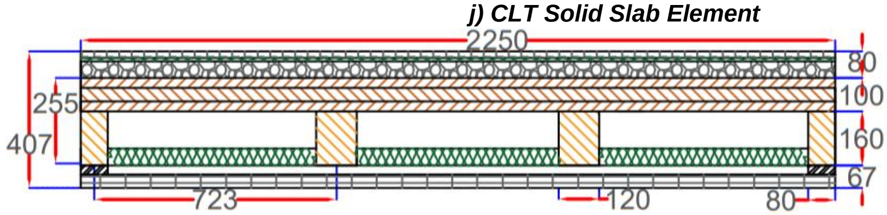


i) ***Kerto Ripa Open Box Element***

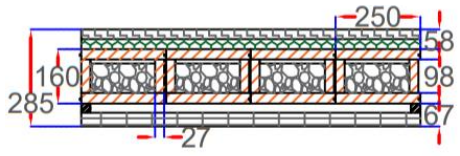
Functional Unit 2

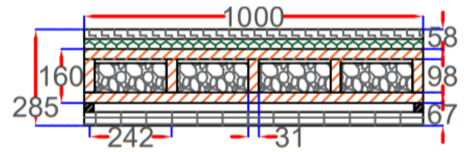


j) ***CLT Solid Slab Element***



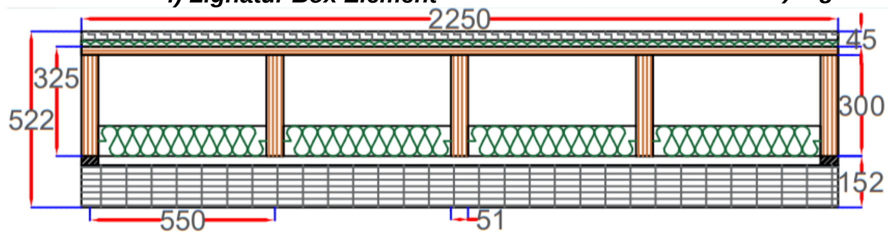
k) ***CLT Open Rib Element***



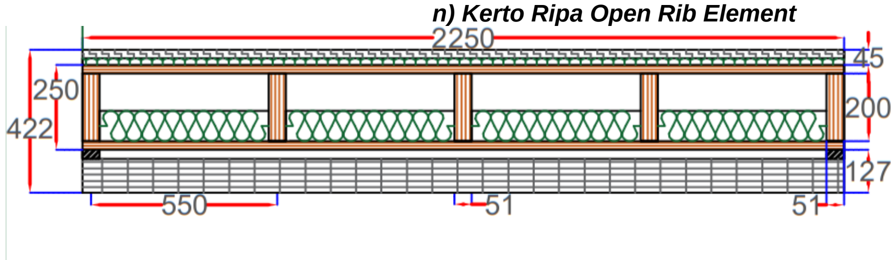


l) Lignatur Box Element

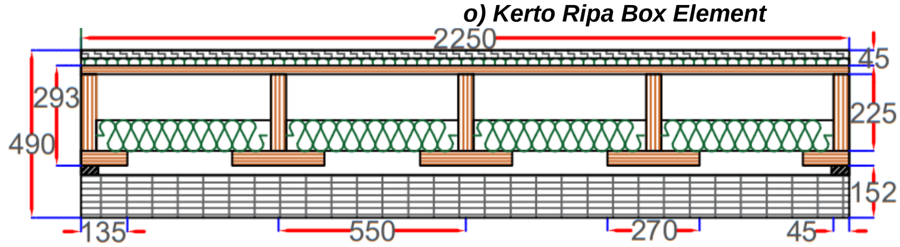
m) Lignatur Surface Element



n) Kerto Ripa Open Rib Element



o) Kerto Ripa Box Element



p) Kerto Ripa Open Box Element

The summary of the cross sections used, and their dead loads is given below in Table B.2, which will be used further ahead in the *MCA*. Total height is the height of the total layup. The height of the structural timber is tabulated separately.

Table B.2: Details of Timber Decks for MCA

Assumptions: *Live Load: 3 kN/m², Floor Finish: 0.5 kN/m², Installed Services: 0.5 kN/m².*

Functional Unit 1				
Timber Deck	**Total Dead Load [kN/m²]**	**Total Height [mm]**	**Height of Timber Section [mm]**	**Code**
CLT Solid Slabs	*3.45*	*375*	*260*	**CLT_SS260**
CLT Open Rib Elements	*3*	*507*	*360*	**CLT_OR360**
CLT Box Elements	*2.96*	*467*	*320*	**CLT_BE320**
Lignatur Box Elements	*3.05*	*365*	*240*	**LK240**
Lignatur Surface Elements	*2.99*	*365*	*240*	**LF240**
Kerto Ripa Open Rib Elements	*2.48*	*672*	*475*	**KR_OR475**
Kerto Ripa Box Elements	*2.38*	*582*	*410*	**KR_BE410**
Kerto Ripa Open Box Elements	*2.54*	*625*	*428*	**KR_OB428**

Functional Unit 2				
Timber Deck	**Total Dead Load [kN/m^2]**	**Total Height [mm]**	**Height of Timber Section [mm]**	**Code**
CLT Solid Slabs	3.05	*295*	180	**CLT_SS180**
CLT Open Rib Elements	2.83	*407*	260	**CLT_OR260**
CLT Box Elements	*[-]*	*[-]*	*[-]*	*[-]*
Lignatur Box Elements	2.97	*285*	160	**LK160**
Lignatur Surface Elements	2.94	*285*	160	**LF160**
Kerto Ripa Open Rib Elements	2.4	*522*	325	**KR_OR325**
Kerto Ripa Box Elements	2.3	*422*	250	**KR_BE250**
Kerto Ripa Open Box Elements	2.48	*490*	293	**KR_OB293**

B.1.2 Dimensioning the Steel Beams

The steel beams are dimensioned based on *ULS* and *SLS* requirements, as given in the Eurocodes. In both the functional units, the steel beam considered is simply supported. Hence compressive stresses occur at the top part of the beam i.e., the bottom part is completely under tension. As the top part of the steel beams are restrained against lateral torsional buckling by the timber decks, no further checks are carried out in this regard. It is assumed that during the execution phase (when there could be unsymmetric loading), the beams are restrained against torsion (by propping). In the final stage, as the loading on the steel beams is symmetric, no checks are done for torsion.

For the integrated beams (and the double Z profile), additional checks must be done for the bottom plate supporting the timber decks, as they are subject to transverse bending and shear. Since the beam is only subjected to sagging moments, only the members under a positive moment are considered for cross section classification. All the checks done on the steel beams are given below in Table B.3:

Table B.3: Checks on Steel Beams.

Formulae:

For a simply supported beam,

$M_{Ed,x} = \frac{q*L^2}{8}$, $V_{Ed,yz} = \frac{q*L}{2}$, $\sigma_{xx} = \frac{M_x*z}{I}$, $\tau_{el,yz} = \frac{V_{yz}*S}{I*t_i}$

$\delta = \frac{5*q*L^4}{384*EI}$ For SLS, w_{max} = L/250

For an Integrated Beam

$m_{y,bp} = \frac{q*e}{4}$, $v_{xz,bp} = \frac{q}{2}$

$\sigma_{yy,bp} = \frac{6*m_{y,bp}}{t_{bp}^2}$ $\tau_{el,xz,bp} = \frac{3*q}{4*t_{bp}}$,

Ultimate Limit State	
Bending: $M_{Rd} = \frac{W_{pl}*f_y}{\gamma_{M.0}}$ for Class 1,2 Sections $M_{Rd} = \frac{W_{el,min}*f_y}{\gamma_{M.0}}$ for Class 3 Sections $M_{Rd} = \frac{W_{eff}*f_y}{\gamma_{M.0}}$ for Class 4 Sections Unity Check → $\frac{M_{Ed}}{M_{Rd}} \leq 1$ For integrated beams, M_{Rd} is reduced by 5% to incorporate the effects of transverse bending. Bending + Shear: For Class 1,2	Shear: $V_{pl,\ Rd} = \frac{A_v*f_y}{\sqrt{3}*\gamma_{M.0}}$ for Class 1,2 Sections $v_{el,\ Rd} = \frac{f_y}{\sqrt{3}*\gamma_{M.0}}$ for Class 3,4 Sections Unity Check → $\frac{V_{Ed}}{V_{pl,Rd}} \leq 1$ OR $\frac{\tau_{Ed}}{v_{el,Rd}} \leq 1$ For integrated beams, M_{Rd} is reduced by 5% to incorporate the effects of transverse bending. Transverse Loading Thickness of Bottom Plate

Only for $V_{Ed} > 0.5 * V_{pl,Rd}$

$\rho = \{\frac{2*V_{Ed}}{V_{pl,Rd}} - 1\}^2$

$M_{V,Rd} = \frac{\{W_{pl} - (\rho * A_w{}^2)/(4*t_w)\}*f_y}{\gamma_{M.0}}$ For I Sections,

$M_{V,Rd} = \rho * M_{Rd}$ For other sections

Unity Check: $\frac{M_{Ed}}{M_{V,Rd}} \leq 1$

For Class 3,4

$\sigma_1 = \sqrt{\{(\sigma_{xx})^2 + (3 * \tau_{yz})^2\}}$

Unity Check → $\frac{\sigma_1}{f_y} \leq 1$

Unity Check → $3 * (\frac{2*v_{xz,bp}}{f_y*t_{bp}})^2 + (\frac{8*m_{y,bp}}{f_y*t_{bp}{}^2}) \leq 1$

Serviceability Limit State

Deflection:

Unity Check → $\frac{\delta}{w_{max}} \leq 1$

Stresses in Bottom Plate (Integrated Beams):

Bottom of the bottom plate,

$\sigma_1 = \sqrt{\{(\sigma_{xx})^2 + \sigma_{xx} * \sigma_{xx} + (\sigma_{yy})^2\}}$

Middle of bottom plate,

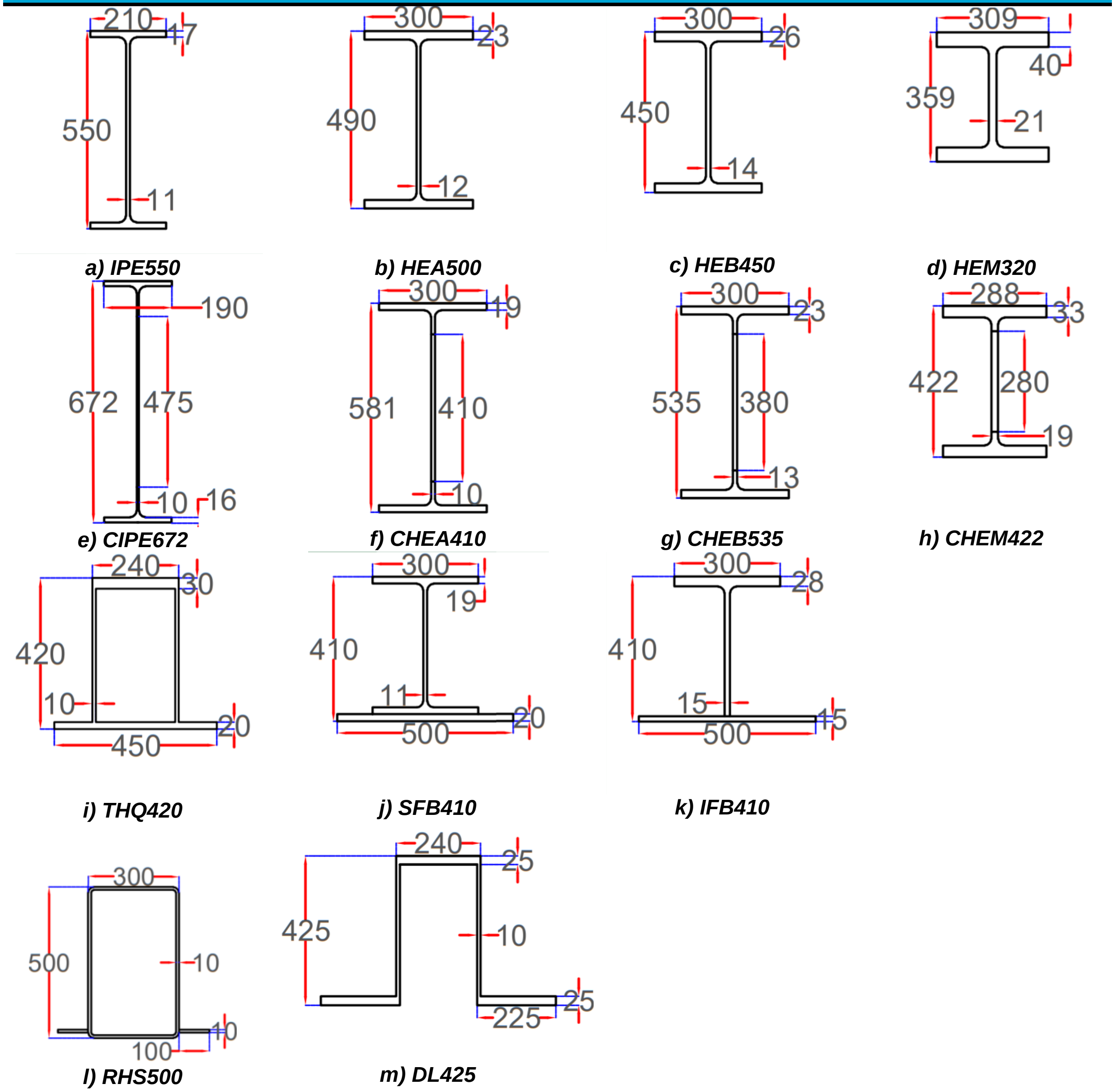
$\sigma_1 = \sqrt{\{(\sigma_{xx})^2 + 3 * (\tau_{xz,bp})^2\}}$

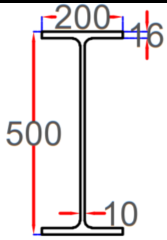
Unity Check → $\frac{\sigma_1}{f_y} \leq 1$

The cross sections used for the *MCA* are given below:

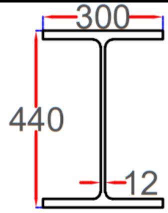
Figure B.2: Detail of Steel Beams for both Functional Units.

Functional Unit 1

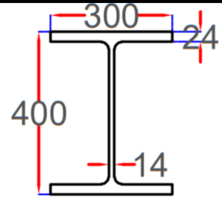




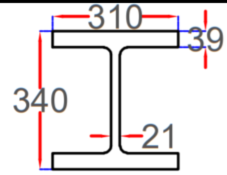
o) IPE500



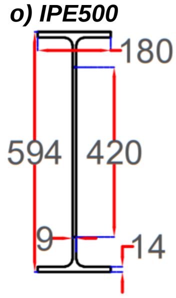
p) HEA450



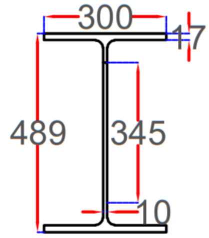
q) HEB400



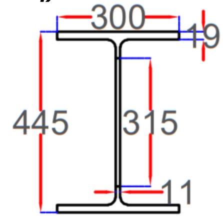
r) HEM300



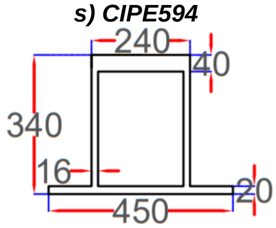
s) CIPE594



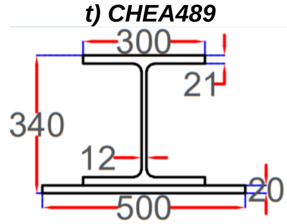
t) CHEA489



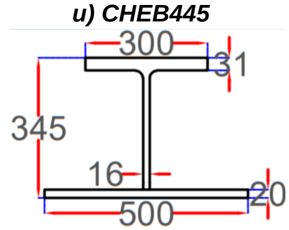
u) CHEB445



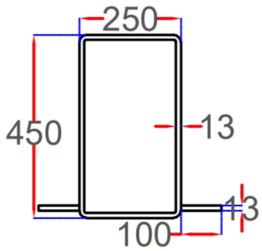
v) THQ340



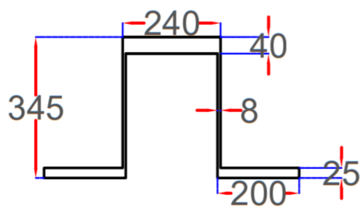
w) SFB340



x) IFB345



y) RHS450



z) DL345

The checks mentioned have been done on the steel beams. This has been tabulated below in Table B.4.

Table B.4 Design Checks for Steel Beams.

Assumptions: *f_y = 355 MPa, E = 200 GPa, $\gamma_{M,0}$ = 1*
q_{DL} = 3.5 kN/m², q_{LL} = 3 kN/m²
Formulas:

$$q_{ULS} = L_S * (1.5 * q_{LL} + 1.35 * q_{DL}) + 1.35 * q_{G,s} \qquad q_{SLS} = L_S * (1 * q_{LL} + 1 * q_{DL}) + 1 * q_{G,s}$$

$$q_{ULS,bp} = L_S * (1.5 * q_{LL} + 1.35 * q_{DL}) \qquad q_{SLS,bp} = L_S * (1 * q_{LL} + 1 * q_{DL})$$

Functional Unit 1									
Steel Beams	**Cross Section Class**	**$q_{G,s}$ [kg/m]**	**ULS Unity Checks**			**SLS Unity Checks**	**Integrated Beam Bottom Plate Unity Checks**		
			Bending	**Shear**	**Bending + Shear**	**Deflection**	**Thickness [-]**	**Elastic [-] Bottom**	**Stresses Middle**
I Beams									
IPE550	*1*	*105.5*	*0.67*	*0.2*	*NA*	*0.98*	*1*	*105.5*	*0.67*
HEA500	*1*	*155.1*	*0.48*	*0.1*	*NA*	*0.76*	*1*	*155.1*	*0.48*
HEB450	*1*	*171.1*	*0.47*	*0.09*	*NA*	*0.84*	*1*	*171.1*	*0.47*
HEM320	*1*	*245*	*0.43*	*0.06*	*NA*	*1*	*1*	*245*	*0.43*
CIPE672	*3*	*81.67*	*0.69*	*0.91*	*0.98*	*0.91*	*3*	*81.67*	*0.69*
CHEA581	*1*	*126.98*	*0.48*	*0.99*	*0.98*	*0.92*	*1*	*126.98*	*0.48*
CHEB535	*1*	*144.31*	*0.46*	*1.06*	*0.98*	*0.7*	*1*	*144.31*	*0.46*
CHEM422	*1*	*144.31*	*0.46*	*1.06*	*0.98*	*0.7*	*1*	*144.31*	*0.46*
Integrated Beams									
THQ420	*1*	*193.6*	*0.48*	*0.18*	*NA*	*0.91*	*1*	*193.6*	*0.48*
SFB410	*1*	*207.2*	*0.6*	*0.14*	*NA*	*0.95*	*1*	*207.2*	*0.6*
IFB410	*1*	*174.4*	*0.52*	*0.18*	*NA*	*0.95*	*1*	*174.4*	*0.52*
Others									
RHS500	*1*	*134.2*	*0.6*	*0.08*	*NA*	*1.03*	*1*	*134.2*	*0.6*
DL425	*1*	*162.88*	*0.54*	*0.12*	*NA*	*1*	*1*	*162.88*	*0.54*
Functional Unit 2									
Steel Beams	**Cross Section Class**	**$q_{G,s}$ [kg/m]**	**ULS Unity Checks**			**SLS Unity Checks**	**Integrated Beam Bottom Plate Unity Checks**		
			Bending	**Shear**	**Bending + Shear**	**Deflection**	**Thickness [-]**	**Elastic [-] Bottom**	**Stresses Middle**
I Beams									
IPE500	*1*	*90.7*	*0.61*	*0.16*	*NA*	*0.98*	*1*	*90.7*	*0.61*
HEA450	*1*	*139.8*	*0.42*	*0.08*	*NA*	*0.75*	*1*	*139.8*	*0.42*
HEB400	*1*	*155.3*	*0.42*	*0.07*	*NA*	*0.83*	*1*	*155.3*	*0.42*
HEM300	*1*	*237.9*	*0.34*	*0.04*	*NA*	*0.83*	*1*	*237.9*	*0.34*
CIPE594	*2*	*69.01*	*0.65*	*0.81*	*0.99*	*0.96*	*2*	*69.01*	*0.65*
CHEA489	*1*	*104.9*	*0.49*	*0.98*	*0.86*	*0.8*	*1*	*104.9*	*0.49*
CHEB445	*1*	*117.38*	*0.48*	*1.03*	*0.99*	*0.87*	*1*	*117.38*	*0.48*
Integrated Beams									
THQ340	*1*	*189.8*	*0.42*	*0.2*	*NA*	*0.99*	*1*	*189.8*	*0.42*
SFB340	*1*	*209*	*0.52*	*0.16*	*NA*	*1.01*	*1*	*209*	*0.52*
IFB340	*1*	*194.4*	*0.4*	*0.15*	*NA*	*0.95*	*1*	*194.4*	*0.4*
Others									
RHS450	*1*	*144.1*	*0.46*	*0.05*	*NA*	*0.89*	*1*	*144.1*	*0.46*
DL345	*1*	*167.36*	*0.43*	*0.13*	*NA*	*1*	*1*	*167.36*	*0.43*

B.1.3 Dimensioning the STC Floor Systems

Based on the results of *MCA1* and *MCA2*, 8 *STC* floors have been chosen. A summary of the *STC* floors taken for *MCA3* are given below:

Table B.5: STC Floors for MCA3.

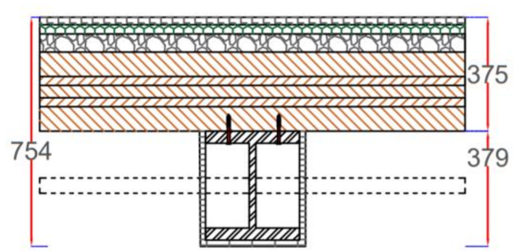
FU1			FU1		
STC Floors	**Timber Decks**	**Steel Beams**	**STC Floors**	**Timber Decks**	**Steel Beams**
Non-Integrated Systems			**Non-Integrated Systems**		
STC1	*CLT_SS260*	*HEM320*	**STC1**	*CLT_SS180*	*HEM300*
STC2	*LF240*	*HEM320*	**STC2**	*LF160*	*HEM300*
Integrated Systems			**Integrated Systems**		
STC3	*CLT_OR360*	*IFB410*	**STC3**	*CLT_OR3260*	*IFB340*
STC4	*LF240*	*IFB410*	**STC4**	*LF240*	*IFB340*
STC5	*CLT_OR360*	*RHS500*	**STC5**	*CLT_OR3260*	*RHS450*
STC6	*LF240*	*RHS500*	**STC6**	*LF240*	*RHS450*
STC7	*CLT_OR360*	*DL425*	**STC7**	*CLT_OR3260*	*DL345*
STC8	*LF240*	*DL425*	**STC8**	*LF240*	*DL345*

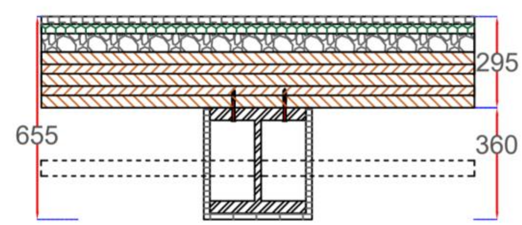
The dimensioning of the timber decks remains the same, as they are imposed with the same requirements. In this *MCA*, for the steel beams, the slab dead load (previously adopted as 3.5 kN/m^2 in *MCA2*) is according to the weight of the different slabs used. However, in most cases there is no change in the cross sections of the steel beams. As the dead loads are equal to or less than the values used in *MCA2*, the design checks on steel beams are not repeated. Finally, the drawings of the *STC* floors, showing the section along the steel beams are given below. All the *STC* floors use bolts/screws as the demountable connectors. For some slabs, additions/openings are required for a demountable connection. *LFE* is provided with openings to access the connections. The constraint of minimum floor height was applied in the placement of the installations.

Figure B.3: Details of STC Floors for MCA3.

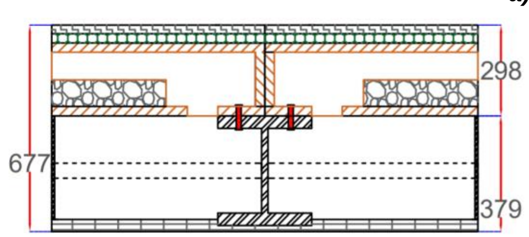
Functional Unit 1	Functional Unit 2

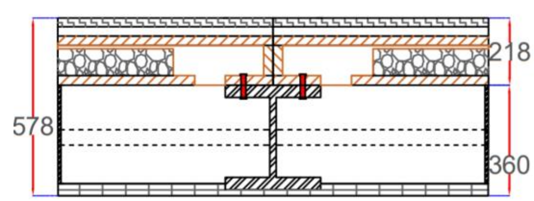
Non-Integrated STC Floor Systems





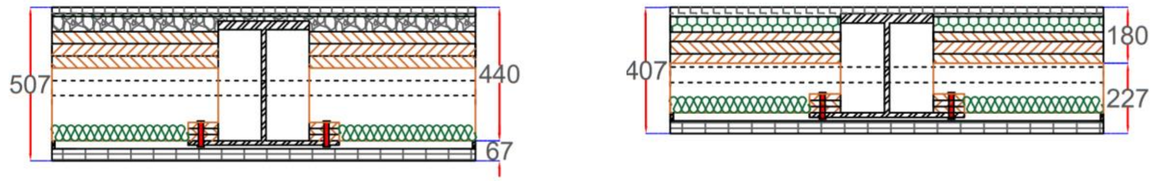
a) STC1





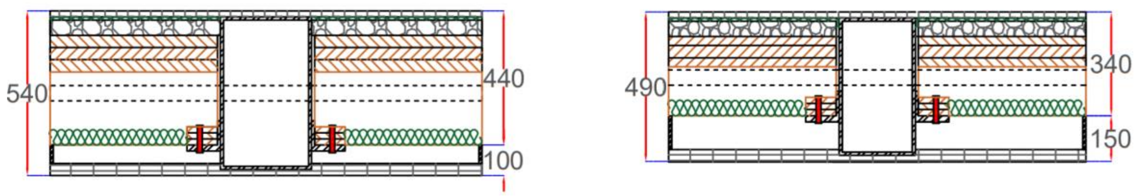
b) STC2

Integrated STC Floor Systems

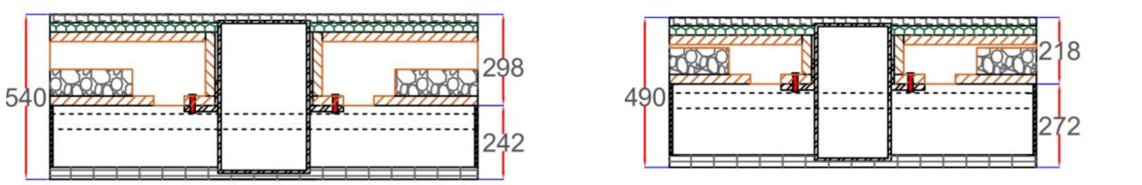


c) STC3

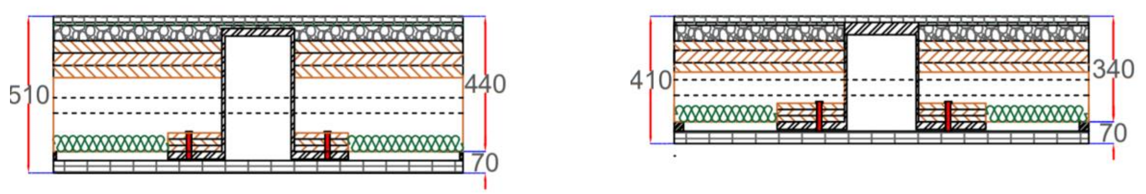
d) STC4



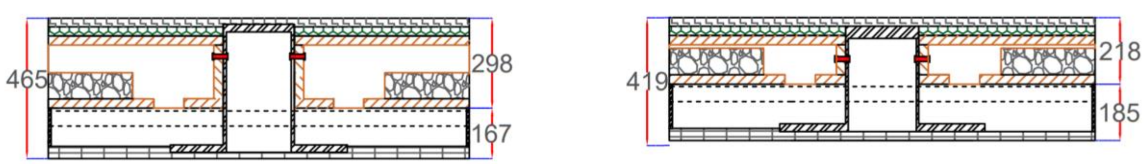
e) STC5



f) STC6



g) STC7



h) STC8

B.2 Rating for Different Parameters

B.2.1 Slenderness

Table B.6: Slenderness Rating for Timber Decks.

Timber Decks	Total Height [mm]	Slenderness (S) [-]	Rating		Limits	Rating
Functional Unit 1						
CLT_SS260	*375*	*26.9*	*5*		17 ≤ S	5
CLT_OR360	*507*	*19.4*	*3*		15 ≤ S < 17	4
CLT_BE320	*467*	*21.8*	*3*		13 ≤ S < 15	3
LK240	*365*	*29.1*	*5*		11 ≤ S < 13	2
LF240	*365*	*29.1*	*5*		11 > S	1
KR_OR475	*672*	*14.7*	*1*			
KR_BE410	*582*	*17*	*2*			
KR_OB428	*625*	*16.3*	*2*			
Functional Unit 2						
Timber Decks	**Total Height [mm]**	**Slenderness (S) [-]**	**Rating**		**Limits**	**Rating**
CLT_SS180	*295*	*27.7*	*4.5*		17 ≤ S	5
CLT_OR260	*387*	*20.8*	*2.5*		15 ≤ S < 17	4
[-]	*NA*	*NA*	*NA*		13 ≤ S < 15	3
LK160	*285*	*31.2*	*5*		11 ≤ S < 13	2
LF160	*285*	*31.2*	*5*		11 > S	1
KR_OR325	*522*	*15.3*	*1.5*			
KR_BE250	*422*	*20*	*2.5*			
KR_OB293	*490*	*17*	*1.5*			

Table B.7: Slenderness Rating for Steel Beams.

Steel Beams	Total Height [mm]	Slenderness (S) [-]	Rating		Limits	Rating
Functional Unit 1						
IPE550	*550*	*16.3*	*2*		24 ≤ S	5
HEA500	*490*	*18.3*	*3*		21 ≤ S < 24	4
HEB450	*450*	*20*	*3*		17 ≤ S < 21	3
HEM320	*359*	*25*	*5*		14 ≤ S < 17	2
CIPE672	*672*	*13.3*	*1*		14 > S	1
CHEA581	*581*	*15.4*	*2*			
CHEB535	*535*	*16.8*	*2*			
CHEM422	*422*	*21.3*	*4*			
THQ420	*420*	*21.4*	*4*			
SFB410	*410*	*21.9*	*4*			
IFB410	*410*	*21.9*	*4*			
RHS500	*500*	*18*	*3*			
DL425	*425*	*21.1*	*4*			
Functional Unit 2						
Steel Beams	**Total Height [mm]**	**Slenderness (S) [-]**	**Rating**		**Limits**	**Rating**
IPE500	*500*	*18*	*2*		25 ≤ S	5
HEA450	*440*	*20.4*	*3*		22 ≤ S < 25	4
HEB400	*400*	*22.5*	*3*		19 ≤ S < 22	3
HEM300	*340*	*26.4*	*5*		15 ≤ S < 19	2
CIPE594	*594*	*15.1*	*1*		15 > S	1
CHEA489	*489*	*18.4*	*2*			
CHEB445	*445*	*20.2*	*3*			
[-]	*[-]*	*[-]*	*[-]*			
THQ340	*340*	*26.4*	*5*			
SFB340	*340*	*26.4*	*5*			
IFB340	*340*	*26.4*	*5*			
RHS450	*450*	*20*	*3*			
DL345	*345*	*26*	*5*			

For the timber decks, the slenderness is obtained as the span of the slab divided by the height of the total layup, including sound insulation and fire safety. The placement of installations is not considered in this rating. The rating limits and the ratings for the timber decks for *MCA1* are given above in Table B.6. For the steel beams, the slenderness is obtained as the span of the beam divided by the height of the cross section of steel. The rating limits and the ratings for the steel beams for *MCA2* are given above in Table B.7. For the *STC* floors, slenderness is obtained as the span of the beam divided by the total height of the floor. This includes height of steel, timber deck, sound insulation, fire protection and the placement of installations. The rating limits and the ratings for the *STC* floors for *MCA3* are given below in Table B.8.

Table B.8: Slenderness Ratings for STC Floors.

Functional Unit 1					
STC Floors	**Total Height [mm]**	**Slenderness (S) [-]**	**Rating**	**Limits**	**Rating**
STC1a	*754*	*11.9*	*1*	19 ≤ S	5
STC2a	*677*	*13.2*	*2*	17 ≤ S < 19	4
STC3a	*507*	*17.7*	*4*	15 ≤ S < 17	3
STC4a	*524*	*17.1*	*4*	13 ≤ S < 15	2
STC5a	*540*	*16.6*	*3*	13 > S	1
STC6a	*540*	*16.6*	*3*		
STC7a	*510*	*17.6*	*4*		
STC8a	*465*	*19.3*	*5*		
Functional Unit 2					
Timber Decks	**Total Height [mm]**	**Slenderness (S) [-]**	**Rating**	**Limits**	**Rating**
STC1b	*655*	*13.7*	*1*	21 ≤ S	5
STC2b	*578*	*15.5*	*2*	19 ≤ S < 21	4
STC3b	*407*	*22.1*	*5*	17 ≤ S < 19	3
STC4b	*460*	*19.5*	*4*	15 ≤ S < 17	2
STC5b	*490*	*18.3*	*3*	15 > S	1
STC6b	*490*	*18.3*	*3*		
STC7b	*410*	*21.9*	*5*		
STC8b	*410*	*21.9*	*5*		

B.2.2 Weight

Table B.9: Weight Rating for Timber Decks.

Functional Unit 1				
Timber Decks	**Weight (W) [kg/m²]**	**Rating**	**Limits**	**Rating**
CLT_SS260	*132.6*	*1*	40 ≥ W	5
CLT_OR360	*80.82*	*2*	40 < W ≤ 60	4
CLT_BE320	*77.55*	*3*	60 < W ≤ 80	3
LK240	*42.18*	*4*	80 < W ≤ 100	2
LF240	*36.13*	*5*	100 < W	1
KR_OR475	*34.88*	*5*		
KR_BE410	*41.67*	*4*		
KR_OB428	*41.15*	*4*		
Functional Unit 2				
Timber Decks	**Weight (W) [kg/m²]**	**Rating**	**Limits**	**Rating**
CLT_SS180	*91.8*	*1*	30 ≥ W	5
CLT_OR260	*64.08*	*3*	30 < W ≤ 50	4
[-]	*0*	*0*	50 < W ≤ 70	3
LK160	*34.93*	*4*	70 < W ≤ 90	2
LF160	*31.59*	*4*	90 < W	1
KR_OR325	*27.08*	*5*		
KR_BE250	*33.35*	*4*		
KR_OB293	*34.96*	*4*		

For timber decks, the rating for weight is based on the total weight per unit area. The weight of sound insulation and fire protection have been considered in the parameter Building Decree. Hence, to avoid double counting of the same parameter, only the weight of the timber is considered for this rating. The rating limits and the ratings for the timber decks for *MCA1* are given above in Table B.9. For the steel beams, the weight rating is based on the weight per unit length. The rating limits and the ratings for the steel beams for *MCA2* are given below in Table B.10.

Table B.10: Weight Rating for Steel Beams.

Functional Unit 1					
Steel Beams	**Weight (W) [kg/m]**	**Rating**		**Limits**	**Rating**
IPE550	*105.5*	*5*		110 ≥ W	5
HEA500	*155.1*	*3*		110 < W ≤ 150	4
HEB450	*171.1*	*3*		150 < W ≤ 180	3
HEM320	*245*	*1*		180 < W ≤ 210	2
CIPE672	*81.6*	*5*		210 < W	1
CHEA581	*126.9*	*4*			
CHEB535	*144.3*	*4*			
CHEM422	*191.9*	*2*			
THQ420	*193.6*	*2*			
SFB410	*207.2*	*2*			
IFB410	*174.4*	*3*			
RHS500	*134.2*	*4*			
DL425	*162.8*	*3*			
Functional Unit 2					
Steel Beams	**Weight (W) [kg/m]**	**Rating**		**Limits**	**Rating**
IPE500	*90.7*	*5*		100 ≥ W	5
HEA450	*139.8*	*3*		100 < W ≤ 130	4
HEB400	*155.3*	*3*		130 < W ≤ 160	3
HEM300	*237.9*	*1*		160 < W ≤ 200	2
CIPE594	*69*	*5*		200 < W	1
CHEA489	*104.9*	*4*			
CHEB445	*117.3*	*4*			
[-]	*[-]*	*[-]*			
THQ340	*189.8*	*2*			
SFB340	*209*	*1*			
IFB340	*194.4*	*2*			
RHS450	*144.1*	*3*			
DL345	*167.3*	*2*			

Table B.11: Weight Rating for STC Floors.

Functional Unit 1					
Timber Decks	**Weight (W) [kg/m²]**	**Rating**		**Limits**	**Rating**
STC1a	*167.6*	*1*		75 ≥ W	5
STC2a	*71.1*	*5*		75 < W ≤ 105	4
STC3a	*105.7*	*3*		105 < W ≤ 135	3
STC4a	*61*	*5*		135 < W ≤ 165	2
STC5a	*99.9*	*4*		165 < W	1
STC6a	*55.3*	*5*			
STC7a	*105.9*	*3*			
STC8a	*61.2*	*5*			
Functional Unit 2					
Timber Decks	**Weight (W) [kg/m²]**	**Rating**		**Limits**	**Rating**
STC1b	*139.3*	*1*		75 ≥ W	5
STC2b	*79.1*	*4*		95 < W ≤ 75	4
STC3b	*98.9*	*3*		115 < W ≤ 95	3

STC4b	*66.4*	*5*	135 < W ≤ 115	2
STC5b	*87.4*	*4*	135 < W	1
STC6b	*54.9*	*5*		
STC7b	*97.5*	*3*		
STC8b	*65*	*5*		

For the *STC* floors, weight is obtained as the total weight of beam and slab per unit area. Due to the same reasoning as for weight rating of timber decks, only the weight of structural timber and steel is considered. The rating limits and the ratings for the *STC* floors are given above in Table B.11.

B.2.3 Building Decree

For timber decks, the final rating for building decree is obtained from 3 sub criteria: sound insulation, fire protection and vibration comfort. The rating for sound insulation is based on the weight of the additions required to comply with the regulation of the Dutch building decree [1] for sound. Wet screeds are avoided, for demountability. As mentioned in Appendix A, the encapsulated strategy is used for fire protection (apart from *CLT_SS*). As part of sound insulation, some timber decks require a suspended ceiling with gypsum plasterboard. Therefore, the rating for fire protection is based on the weight of the additional layer of gypsum required. It should be noted here that the same layup is used for both the functional units. The rating limits and the ratings for the timber decks for sound insulation and fire protection for *MCA1* are given below:

Table B.12: Sound Insulation Rating and Fire Protection Rating for MCA1.

Sound Insulation		Fire Protection	
Limits	**Rating**	**Limits**	**Rating**
$W_S \le 100$	5	$W_F \le 10$	5
$100 < W_S \le 105$	4	$10 < W_F \le 20$	4
$105 < W_S \le 110$	3	$20 < W_F \le 30$	3
$110 < W_S \le 115$	2	$30 < W_F \le 40$	2
$115 < W_S$	1	$40 < W_F$	1

Functional Unit 1&2				
Timber Decks	**Sound Insulation**		**Fire Protection**	
	Weight (W_S) [kg/m²]	**Rating**	**Weight (W_F) [kg/m²]**	**Rating**
CLT_SS	*117.3*	*1*	*0*	*5*
CLT_OR	*112.4*	*2*	*10.2*	*4*
CLT_BE	*95.4*	*5*	*27.2*	*3*
LK	*112.7*	*2*	*53.6*	*1*
LF	*112.7*	*2*	*53.6*	*1*
KR_OR	*116.1*	*1*	*0*	*5*
KR_BE	*99.1*	*5*	*0*	*5*
KR_OB	*116.1*	*1*	*0*	*5*

For vibration comfort of the timber decks, the rating is based on the first natural frequency and the stiffness of the deck. Rather than checking whether the timber decks conform to the criteria for vibrations according to *EC5* [53], rating limits provided by Hamm and Richter [3] are used to classify them. It is assumed that the timber decks that do not clear the criteria *EC5* are provided with structural additions to improve their performance. The rating limits and the ratings for the timber decks for vibration comfort are given below:

Table B.13: Vibration Ratings for Timber Decks.

Formulae:

First Natural Frequency of the Timber Deck,

$\rightarrow f_1 = \frac{\pi *}{2 * {L_S}^2} * \sqrt{\{\frac{(EI_x)_l}{M}\}}$

For Biaxial slabs (such as CLT),

$\rightarrow f_{1,plate} = f_1 * \sqrt{\{1 + \frac{1}{\alpha^4}\}}$, where $\alpha = \frac{L_B *}{L_S} * \sqrt[4]{\{\frac{(EI_x)_l}{(EI_y)_l}\}}$

Stiffness of the Timber Deck, measured as the deflection under load of 2kN

$\rightarrow u = \frac{5 * P * {L_S}^4}{384 * EI_x}$

Timber Decks	First Natural Frequency (f_1) [Hz]	Stiffness (u) [mm]	Rating
Functional Unit 1			
CLT_SS260	*6.78*	*0.93*	*3*
CLT_OR360	*6.88*	*1.01*	*2*
CLT_BE320	*8.1*	*0.87*	*4*
LK240	*5.33*	*1.65*	*1*
LF240	*5.25*	*1.74*	*1*
KR_OR475	*9.3*	*0.67*	*4*
KR_BE410	*9.72*	*0.63*	*4*
KR_OB428	*9.63*	*0.6*	*4*
Functional Unit 2			
Timber Decks	**First Natural Frequency (f_1) [Hz]**	**Stiffness (u) [mm]**	**Rating**
CLT_SS180	*14.01*	*0.34*	*5*
CLT_OR260	*13.86*	*0.37*	*5*
[-]	*[-]*	*[-]*	*[-]*
LK160	*7.68*	*1.14*	*2*
LF160	*7.54*	*1.2*	*2*
KR_OR325	*13.18*	*0.48*	*5*
KR_BE250	*13.8*	*0.45*	*5*
KR_OB293	*14.02*	*0.41*	*5*

Limits		Rating
$f_1 \geq 8$	& $u \leq 0.5$	5
$f_1 \geq 8$	& $0.5 < u \leq 1.0$	4
$6 \leq f_1 < 8$	& $0.5 < u \leq 1.0$	3
$4.5 \leq f_1 < 6$	& $u > 1.0$	2
$f_1 < 4.5$	& $u > 1.0$	1

The total rating for building decree for timber decks is obtained as follows:

Building Decree Total Rating = 0.2*Sound Insulation + 0.3*Fire Protection + 0.5*Vibration

The summary of results for building decree and the total rating is given below.

Table B.14: Summary of Ratings for Building Decree for Timber Decks.

Timber Decks	Sound Insulation	Fire Protection	Vibration	Total Rating
Functional Unit 1				
CLT_SS260	*1*	*5*	*3*	*3.2*
CLT_OR360	*2*	*4*	*2*	*2.6*
CLT_BE320	*5*	*3*	*4*	*3.9*
LK240	*2*	*1*	*1*	*1.2*
LF240	*2*	*1*	*1*	*1.2*
KR_OR475	*1*	*5*	*4*	*3.7*
KR_BE410	*5*	*5*	*4*	*4.5*
KR_OB428	*1*	*5*	*4*	*3.7*
Functional Unit 2				
Timber Decks	**Sound Insulation**	**Fire Protection**	**Vibration**	**Total Rating**
CLT_SS180	*1*	*5*	*5*	*4.2*
CLT_OR260	*2*	*4*	*5*	*4.1*
[-]	*[-]*	*[-]*	*[-]*	*[-]*
LK160	*2*	*1*	*2*	*1.7*
LF160	*2*	*1*	*2*	*1.7*
KR_OR325	*1*	*5*	*5*	*4.2*
KR_BE250	*5*	*5*	*5*	*5*
KR_OB293	*1*	*5*	*5*	*4.2*

For steel beams, only the fire protection aspect of building decree is considered. The amount of fire protection required depends on the section factor of the beam. Hence the rating is based on the section factor, obtained as the exposed perimeter per cross section area of the steel beam. The steel beam with the least section factor gets the highest rating, and vice versa. The rating limits and the ratings for the steel beams for fire protection are given below:

Table B.15: Fire Protection Rating for Steel Beams.

Functional Unit 1

Steel Beams	Section Factor (SF) [1/m]	Rating
IPE550	*139.6*	*1*
HEA500	*106.8*	*3*
HEB450	*92.9*	*4*
HEM320	*59.7*	*5*
CIPE672	*127.6*	*2*
CHEA581	*114.6*	*3*
CHEB535	*99.2*	*3*
CHEM422	*63.7*	*5*
THQ420	*81*	*4*
SFB410	*89*	*4*
IFB410	*106.4*	*3*
RHS500	*127.6*	*2*
DL425	*146*	*1*

Limits	Rating
75 ≥ SF	5
75 < SF ≤ 95	4
95 < SF ≤ 115	3
115 < SF ≤ 135	2
135 < SF	1

Functional Unit 2

Steel Beams	Section Factor (SF) [1/m]	Rating
IPE500	*150.9*	*1*
HEA450	*112.9*	*3*
HEB400	*97.4*	*3*
HEM300	*60.4*	*5*
CIPE594	*139.3*	*2*
CHEA489	*121.3*	*2*
CHEB445	*106.1*	*3*
[-]	*[-]*	*[-]*
THQ340	*77.2*	*4*
SFB340	*83*	*4*
IFB340	*90.2*	*4*
RHS450	*87*	*4*
DL345	*121*	*2*

Limits	Rating
74 ≥ SF	5
74 < SF ≤ 96	4
96 < SF ≤ 118	3
118 < SF ≤ 140	2
140 < SF	1

For the *STC* floors, the above ratings for timber decks and steel beams are combined. The ratings for sound insulation and vibration are taken as it from timber decks. For fire protection, the rating for the *STC* floor is obtained as follows:

Total Fire Protection Rating = 0.3* Beam Rating + 0.7* Slab Rating

Higher weightage is given to slabs as they require more surface of fire protection. The steel ratings are based on the new section factor, which considers parts of the steel beam covered by the timber slabs. The total rating for building decree for the *STC* floor is obtained using the same method as for timber decks:

Building Decree Total Rating = 0.2*Sound Insulation + 0.3*Fire Protection + 0.5*Vibration

The summary of ratings, and the total rating for building decree of the *STC* floor is given below:

Table B.16: Summary of Ratings for Building Decree of STC Floors.

Timber Decks	Sound Insulation	Fire Protection			Vibration	Total Rating
Functional Unit 1						
		Timber Decks	Steel Beams	STC Floors		
STC1a	*5*	*4*	*4.7*	*3*	*3.11*	*5*
STC2a	*1*	*5*	*2.2*	*1*	*1.56*	*1*
STC3a	*4*	*4*	*4*	*2*	*2.6*	*4*
STC4a	*1*	*3*	*1.6*	*1*	*1.38*	*1*
STC5a	*4*	*4*	*4*	*2*	*2.6*	*4*
STC6a	*1*	*2*	*1.3*	*1*	*1.29*	*1*
STC7a	*4*	*4*	*4*	*2*	*2.6*	*4*
STC8a	*1*	*3*	*1.6*	*1*	*1.38*	*1*
Functional Unit 2						
Timber Decks	**Sound Insulation**	**Fire Protection**			**Vibration**	**Total Rating**
		Timber Decks	**Steel Beams**	**STC Floors**		
STC1b	*1*	*5*	*4*	*4.5*	*5*	*4.05*
STC2b	*2*	*1*	*5*	*3*	*2*	*2.3*
STC3b	*2*	*4*	*4*	*4*	*5*	*4.1*
STC4b	*2*	*1*	*2*	*1.5*	*2*	*1.85*
STC5b	*2*	*4*	*3*	*3.5*	*5*	*3.95*
STC6b	*2*	*1*	*1*	*1*	*2*	*1.7*
STC7b	*2*	*4*	*4*	*4*	*5*	*4.1*
STC8b	*2*	*1*	*3*	*2*	*2*	*2*

B.2.4 Demountability

For the timber decks, demountability is measured as the number of elements required for the functional unit, which is equal to the span of the beam divided by the width of the slab. When many timber decks are required, the rating is the least, and vice versa. At the same time, timber decks with small widths give more modularity. They can be replaced/removed easily, thus giving flexibility in creating open spaces. Hence a bonus point is awarded when the width of the timber element is less than 250 mm, as in the case of Lignatur box elements. The rating limits and the ratings of the timber decks for demountability is given below:

Table B.17: Ratings for Demountability of STC Floors.

Timber Decks	Width (B) [mm]	Number of Elements (NE) [-]	Rating
Functional Unit 1&2			
CLT_SS	*3000*	*3*	*4*
CLT_OR	*2250*	*4*	*4*
CLT_BE	*2250*	*4*	*4*
LK	*250*	*36*	*2*
LF	*1000*	*9*	*3*
KR_OR	*2250*	*4*	*4*
KR_BE	*2250*	*4*	*4*
KR_OB	*2250*	*4*	*4*

Limits	Rating
NE ≤ 5	4
5 < NE ≤ 10	3
10 < NE ≤ 40	2
NE > 40	1
Bonus Point: B ≤ 250	1

For the steel beams alone, this parameter is not considered in *MCA2*. For the *STC* floors, demountability is obtained from 2 sub criteria. The first sub criteria is the number of elements required, which includes the timber decks, steel beams and any extra elements required for the connection. For example, the *RHS* sections require 2 additional channel plates to be welded onto to it, for a demountable connection between the beam and slab. The rating limits and the ratings for the number elements in the *STC* floors is given below:

Table B.18: Ratings for Demountability of Timber Decks.

Functional Unit 1&2				
STC Floors	**Number of Elements (NE) [-]**	**Rating**	**Limits**	**Rating**
STC1	*4*	*5*	6 ≥ NE	5
STC2	*10*	*3*	6 < NE ≤ 8	4
STC3	*5*	*5*	8 < NE ≤ 13	3
STC4	*10*	*3*	13 < NE	2
STC5	*7*	*4*		
STC6	*12*	*2*		
STC7	*5*	*5*		
STC8	*10*	*3*		

The second sub criteria is the demountability index, developed and tested by a consortium of Alba Concepts, Dutch Green Building Council (*DGBC*) and other agencies [88]. This index is calculated based on 4 aspects: 1). Type of connection used, 2). Accessibility of the connection, 3). Degree of integration of installations, and 4). Form of encasement. Scores for these aspects have been provided in [88], from the values relevant to this research is extracted. This is tabulated below. The final Demountability Index is calculated as the average these 4 scores.

Table B.19: Scores for Aspects of Demountability of STC Floors.

Aspects of Demountability	Score	Remark
Type of Connection		
Bolt and Nut *Screws* *Weld*	*0.8* *0.8* *0.1*	*The demountability between steel beam and timber decks is considered here. Hence welds used in the connection for RHS beams is not considered. For all STC Floors, bolts/screws are used, and the score for the type of connection is 0.8*
Accessibility of Connection		
Freely Accessible *Accessible with additional actions that do not cause any damage*	*1* *0.8*	*For the solid slabs and the open rib slabs, the connections are freely accessible, and hence score 1. For the box slabs, additional openings must be created to access the connections. Once installed, no additional damage is caused in disassembly. Hence, they score 0.8.*
Degree of Integration of Installations		
Modular: Does not pass through any element. *Intersection with Beams* *Integrated into Slabs*	*1* *0.4* *0.1*	*STC floors are dimensioned for minimum height. When possible, installations pass through the beams, close to its middle. In these situations, they score 0.4 (STCs 1,2,4&8). For open rib elements, the installations are integrated into the slabs. In such cases, they score 0.1 (STCs 3,5&7). When there is not enough clearance through the beams, installations are provided below it, thus scoring 1 (STC4). However, this increases the total floor height.*
Form of Encasement		
No Inclusions *Overlap on 1 Side*	*1* *0.8*	*For I beams, the slabs can be easily removed. Hence, they score 1. For the integrated beams, the slabs are overlapped on side. Hence, they score 0.8.*

The rating limits and ratings for demountability index is given below.

Table B.20: Demountability Index Ratings for STC Floors.

Functional Unit 1&2				
STC Floors	**Demountability Index (DI) [-]**	**Rating**	**Limits**	**Rating**
STC1	*0.8*	*5*	0.8 ≤ DI	5
STC2	*0.75*	*5*	0.75 ≤ DI < 0.8	4
STC3	*0.675*	*1*	0.73 ≤ DI < 0.75	3
STC4	*0.7*	*2*	0.7 ≤ DI ≤ 0.73	2
STC5	*0.675*	*1*	0.7 > DI	1
STC6	*0.7*	*2*		
STC7	*0.675*	*1*		
STC8	*0.7*	*2*		

The total rating for demountability is obtained as follows:

Demountability Total Rating = 0.4*Number of Element + 0.6*Demountability Index

The summary of ratings and total ratings for demountability is given below:

Table B.21: Summary of Ratings for Demountability of STC Floors.

Functional Unit 1&2			
Timber Decks	**Number of Elements**	**Demountability Index**	**Total Rating**
STC1	*5*	*5*	*5*
STC2	*3*	*5*	*2.6*
STC3	*5*	*1*	*2.4*
STC4	*3*	*2*	*2.2*
STC5	*4*	*1*	*2*
STC6	*2*	*2*	*2.6*
STC7	*5*	*1*	*2.4*
STC8	*3*	*2*	*4.2*

B.2.5 Sustainability

The parameter sustainability is measured with the help of the *ECI* value of the materials used. *ECI* values, obtained from the *NMD* is given below:

Table B.22: ECI values of the Materials used for the STC floors.

Material Used	Material from NMD	Material Code		ECI [€/kg]
		Use Stage	End of Life Stage	
Steel	*Heavy Steel Construction Products*	*329*	*329r*	*0.0326*
Lignatur	*Spruce, Sustainable Forestry*	*271*	*17v*	*0.0005*
CLT	*Pine Planks Laminated, Sustainable Forestry*	*77*	*17v*	*0.0285*
GLT	*Pine Planks Laminated, Sustainable Forestry*	*77*	*17v*	*0.0285*
LVL	*Spruce Multiplex, Sustainable Forestry*	*167*	*25v*	*0.0967*
Dry Screed	*Gypsum Plasterboard*	*69*	*10r*	*0.0182*
Gypsum Fibreboard	*Gypsum Plasterboard*	*69*	*10r*	*0.0182*
Chipboard	*Chipboard Sustainable*	*237*	*025v*	*0.0266*
Wood Fibre Insulation	*Woodchips*	*89*	*16v*	*-0.0209*
Impact Sound_MW	*Rockwool*	*252*	*35r*	*0.0953*
Mineral Wool	*Rockwool*	*252*	*35r*	*0.0953*
Loose Fill	*Mixed Granulate*	*142*	*14r*	*-0.0003*
Gravel	*Mixed Granulate*	*142*	*14r*	*-0.0003*
Elastically Bound Fill	*Mixed Granulate*	*142*	*14r*	*-0.0003*
Trickle Protection Layer	*Plastic Layer*	*118*	*1*	*3.6799*

The sustainability of the timber decks is measured with the *ECI* value per unit area of the slab. The timber decks with the least *ECI* per area gets the highest rating, and vice versa. The rating limits and ratings for sustainability for the timber decks are given below. The contribution of the metal channels for the suspended ceilings are not considered.

Table B.23: Ratings for Sustainability of the Timber Decks.

Functional Unit 1

Timber Decks	ECI per Area (ECI_A) [€/m^2]	Rating
CLT_SS260	*5.28*	*2*
CLT_OR360	*3.28*	*4*
CLT_BE320	*3.19*	*4*
LK240	*1.43*	*5*
LF240	*1.43*	*5*
KR_OR475	*6.06*	*1*
KR_BE410	*6.53*	*1*
KR_OB428	*6.67*	*1*

Limits	Rating
$ECI_A \leq 2.2$	5
$2.2 < ECI_A \leq 3.4$	4
$3.4 < ECI_A \leq 4.6$	3
$4.6 < ECI_A \leq 5.8$	2
$ECI_A \geq 5.8$	1

Functional Unit 2

Timber Decks	ECI per Area (ECI_A) [€/m^2]	Rating
CLT_SS180	*4.11*	*2*
CLT_OR260	*2.82*	*3*
[-]	*[-]*	*[-]*
LK160	*1.43*	*5*
LF160	*1.43*	*5*
KR_OR325	*5.31*	*2*
KR_BE250	*5.73*	*1*
KR_OB293	*6.07*	*1*

Limits	Rating
$ECI_A \leq 1.5$	5
$1.5 < ECI_A \leq 2.8$	4
$2.8 < ECI_A \leq 4.1$	3
$4.1 < ECI_A \leq 5.4$	2
$ECI_A \geq 5.4$	1

For the steel beams, sustainability is measured as the *ECI* per unit length. The rating limits and ratings for sustainability for the steel beams are given below:

Table B.24: Sustainability Rating of Steel Beams.

Functional Unit 1

Steel Beams	ECI (ECI_L) [€/m]	Rating
IPE550	*0.492*	*5*
HEA500	*0.723*	*3*
HEB450	*0.798*	*3*
HEM320	*1.143*	*1*
CIPE672	*0.381*	*5*
CHEA581	*0.592*	*4*
CHEB535	*0.673*	*4*
CHEM422	*0.895*	*3*
THQ420	*0.903*	*2*
SFB410	*0.967*	*2*
IFB410	*0.813*	*3*
RHS500	*0.626*	*4*
DL425	*0.76*	*3*

Limits	Rating
$0.5 \geq ECI_L$	5
$0.5 < ECI_L \leq 0.7$	4
$0.7 < ECI_L \leq 0.9$	3
$0.9 < ECI_L \leq 1.1$	2
$1.1 < ECI_L$	1

Functional Unit 2

Steel Beams	ECI (ECI_L) [€/m]	Rating
IPE500	*0.592*	*4*
HEA450	*0.913*	*3*
HEB400	*1.014*	*2*
HEM300	*1.554*	*1*
CIPE594	*0.45*	*5*
CHEA489	*0.685*	*4*
CHEB445	*0.767*	*3*
[-]	*[-]*	*[-]*
THQ340	*1.24*	*2*
SFB340	*1.365*	*1*
IFB340	*1.27*	*2*
RHS450	*0.941*	*3*
DL345	*1.093*	*2*

Limits	Rating
$0.5 \geq ECI_L$	5
$0.5 < ECI_L \leq 0.7$	4
$0.7 < ECI_L \leq 1.0$	3
$1.0 < ECI_L \leq 1.3$	2
$1.3 < ECI_L$	1

For the *STC* floors, sustainability is measured as the *ECI* per unit area by combining the contribution of the timber deck and steel beam. The rating limits and ratings for sustainability for the steel beams are given below:

Table B.25: Sustainability Rating for STC Floors.

Functional Unit 1				
Timber Decks	**ECI per Area (ECI_A) [€/m^2]**	**Rating**	**Limits**	**Rating**
STC1a	*6.429*	*1*	$ECI_A \leq 2.8$	5
STC2a	*2.574*	*5*	$2.8 < ECI_A \leq 3.7$	4
STC3a	*4.096*	*5*	$3.7 < ECI_A \leq 4.6$	3
STC4a	*2.245*	*3*	$4.6 < ECI_A \leq 5.7$	2
STC5a	*3.909*	*5*	$ECI_A \geq 5.7$	1
STC6a	*2.057*	*3*		
STC7a	*4.103*	*5*		
STC8a	*2.251*	*3*		
Functional Unit 2				
Timber Decks	**ECI per Area (ECI_A) [€/m^2]**	**Rating**	**Limits**	**Rating**
STC1b	*5.674*	*1*	$ECI_A \leq 2.6$	5
STC2b	*2.986*	*5*	$2.6 < ECI_A \leq 3.5$	4
STC3b	*3.961*	*4*	$3.5 < ECI_A \leq 4.4$	3
STC4b	*2.571*	*3*	$4.4 < ECI_A \leq 5.3$	2
STC5b	*3.583*	*5*	$ECI_A \geq 5.3$	1
STC6b	*2.193*	*4*		
STC7b	*3.915*	*5*		
STC8b	*2.525*	*3*		

B.2.6 Logistics

The parameter logistics is measured as the number of trucks used to transport the structural elements. For this, a standard articulated trailer, that is commonly used in the Netherlands is considered. The constraints for transportation are given below:

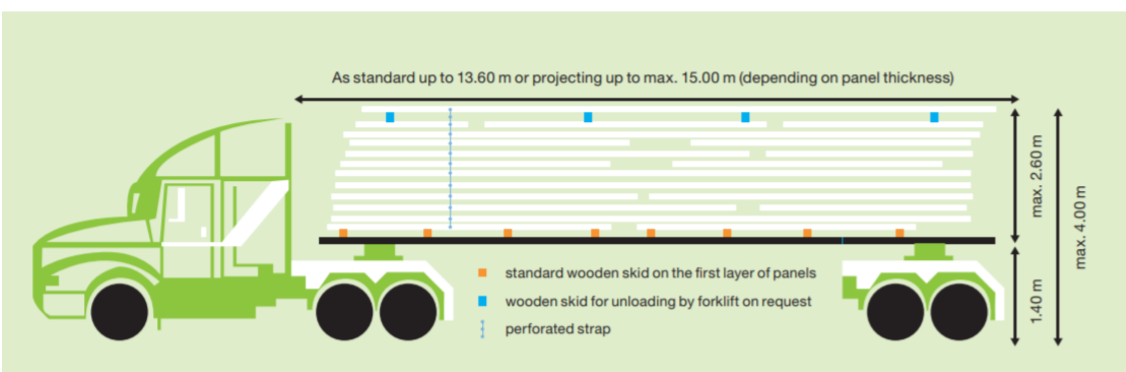


Figure B.4: Standard Articulated Trailer, from [5].

Max Dimensions for Goods:
Height: 2600 mm ***Width: 2950 mm*** ***Length: 13600 mm***
Max Weight: 25 tonnes

Based on these constraints, the total number of structural elements required for 1000 m^2 area of *STC* floors is calculated. The rating limits and the ratings for the timber decks, steel beams and the *STC* floors are given below:

Table B.26: Transportation Rating for a) Timber Decks.

Functional Unit 1

Timber Decks	Number of Trucks (NT) [-]	Rating
CLT_SS260	*6*	*5*
CLT_OR360	*8*	*3*
CLT_BE320	*7*	*4*
LK240	*5*	*5*
LF240	*6*	*5*
KR_OR475	*11*	*1*
KR_BE410	*9*	*2*
KR_OB428	*11*	*1*

Limits	Rating
NT = 6	5
NT = 7	4
NT = 8	3
NT = 9	2
NT ≥ 10	1

Functional Unit 2

Timber Decks	Number of Trucks (NT) [-]	Rating
CLT_SS180	*4*	*4*
CLT_OR260	*4*	*4*
[-]		
LK160	*3*	*5*
LF160	*3*	*5*
KR_OR325	*5*	*3*
KR_BE250	*4*	*4*
KR_OB293	*5*	*3*

Limits	Rating
NT = 3	5
NT = 4	4
NT = 5	3

b) Steel Beams

Functional Unit 1

Steel Beams	Number of Trucks (NT) [-]	Rating
IPE550	*1*	*2*
HEA500	*1*	*2*
HEB450	*2*	*1*
HEM320	*2*	*1*
CIPE672	*1*	*2*
CHEA581	*1*	*2*
CHEB535	*1*	*2*
CHEM422	*2*	*1*
THQ420	*2*	*1*
SFB410	*2*	*1*
IFB410	*2*	*1*
RHS500	*1*	*2*
DL425	*1*	*2*

Limits	Rating
NT = 1	1
NT = 2	2

Functional Unit 2

Steel Beams	Number of Trucks (NT) [-]	Rating
IPE500	*1*	*2*
HEA450	*2*	*1*
HEB400	*2*	*1*
HEM300	*2*	*1*
CIPE594	*1*	*2*

Limits	Rating
NT = 1	1
NT = 2	2

CHEA489	*1*	*2*
CHEB445	*2*	*1*
[-]	*[-]*	*[-]*
THQ340	*2*	*1*
SFB340	*2*	*1*
IFB340	*2*	*1*
RHS450	*2*	*1*
DL345	*2*	*1*

c) STC Floors

Functional Unit 1				
Timber Decks	**Number of Trucks (NT) [-]**	**Rating**	**Limits**	**Rating**
STC1a	*7*	*5*	NT = 7	5
STC2a	*7*	*5*	NT = 8	3
STC3a	*9*	*1*	NT = 9	1
STC4a	*7*	*5*		
STC5a	*9*	*1*		
STC6a	*7*	*5*		
STC7a	*9*	*1*		
STC8a	*7*	*5*		

Functional Unit 2				
Timber Decks	**Number of Trucks (NT) [-]**	**Rating**	**Limits**	**Rating**
STC1b	*6*	*1*	NT = 4	5
STC2b	*4*	*5*	NT = 5	3
STC3b	*5*	*3*	NT = 6	1
STC4b	*4*	*5*		
STC5b	*5*	*3*		
STC6b	*4*	*5*		
STC7b	*5*	*3*		
STC8b	*4*	*5*		

B.2.7 Flexibility

The parameter flexibility is measured as the extra span that can be achieved using the structural element, beyond the functional unit. As this lies outside the functional unit, it is awarded as a bonus point. The limits for the bonus points and ratings for timber decks and steel beams are given below.

Table B.27: Flexibility Bonus Points for Timber Decks.

Functional Unit 1&2				
Timber Decks	**Maximum Span ($L_{S, max}$) [m]**	**Rating**	**Limit**	**Bonus Point**
CLT_SS	*7*	*0*	$L_{S, max} > 9$	1
CLT_OR	*12*	*1*		
CLT_BE	*12*	*1*		
LK	*9.5*	*1*		
LF	*9.5*	*1*		
KR_OR	*8*	*0*		
KR_BE	*9*	*1*		
KR_OB	*8*	*0*		

Table B.28: Flexibility Bonus Points for Steel Beams.

Functional Unit 1				
Steel Beams	**Maximum Span ($L_{B, max}$) [m]**	**Rating**	**Limit**	**Bonus Point**
IPE550	*9.9*	*0*	$L_{B, max}$ > 12	1
HEA500	*11.5*	*0*		
HEB450	*12.2*	*1*		
HEM320	*13.6*	*1*		
CIPE672	*9.2*	*0*		
CHEA581	*10.7*	*0*		
CHEB535	*10.4*	*0*		
CHEM422	*13.2*	*1*		
THQ420	*10.3*	*0*		
SFB410	*10.2*	*0*		
IFB410	*10.5*	*0*		
RHS500	*10.4*	*0*		
DL425	*12.4*	*1*		
Functional Unit 2				
Steel Beams	**Maximum Span ($L_{B, max}$) [m]**	**Rating**	**Limits**	**Rating**
IPE500	*11*	*0*	$L_{B, max}$ > 13	1
HEA450	*12.8*	*0*		
HEB400	*13.6*	*1*		
HEM300	*15.1*	*1*		
CIPE594	*10.3*	*0*		
CHEA489	*12.8*	*0*		
CHEB445	*12.4*	*0*		
[-]	*14.6*	*1*		
THQ340	*11.5*	*0*		
SFB340	*11.3*	*0*		
IFB340	*11.7*	*0*		
RHS450	*11.6*	*0*		
DL345	*13.8*	*1*		

For the STC floors, the bonus points from both the relevant timber deck and the respective steel beam is added to get the total rating. The criteria for bonus points are changed to L_S > 10 m for slabs, and $L_{B, FU1}$ > 11 m and $L_{B, FU2}$ > 12 for beams.

Table B.29: Flexibility Bonus Points for STC Floors.

	Functional Unit 1			**Functional Unit 2**		
Timber Decks	**Slab**	**Beam**	**Total**	**Slab**	**Beam**	**Total**
STC1	*1*	*0*	*1*	*1*	*0*	*1*
STC2	*1*	*0*	*1*	*1*	*0*	*1*
STC3	*0*	*1*	*1*	*1*	*1*	*2*
STC4	*0*	*0*	*0*	*1*	*0*	*1*
STC5	*0*	*1*	*1*	*0*	*1*	*1*
STC6	*0*	*0*	*0*	*0*	*0*	*0*
STC7	*0*	*1*	*1*	*0*	*1*	*1*
STC8	*0*	*0*	*0*	*0*	*0*	*0*

B.3 Effect of Weight Factors

Based on the procedure explained in Section 4.3, the percentage of contribution for each parameter is tabulated below. These are obtained from the required distribution of weights between ***F*** and ***C***, and also within ***F*** and ***C*** depending on the prescribed ratio of contribution of each parameter.

Table B.30: Percentage Distribution of Weights for different Parameters.

Timber Decks									
Scores (Total:100)	**Functionality (F)**				**Circularity (C)**				
	Slenderness	**Weight**	**Building Decree**	**Total**	**Demountability**	**Sustainability**	**Logistics**	**Flexibility**	**Total**
Score1	*11.42*	*11.42*	*17.16*	***40***	*16.67*	*16.66*	*16.67*	*10*	***60***
Score2	*12.86*	*12.86*	*19.28*	***45***	*15.27*	*15.27*	*15.27*	*9.19*	***55***
Score3	*14.28*	*14.28*	*21.44*	***50***	*13.88*	*13.88*	*13.88*	*8.36*	***50***
Score4	*15.71*	*15.71*	*23.58*	***55***	*12.5*	*12.5*	*12.5*	*7.5*	***45***
Steel Beams									
Scores (Total:100)	**Functionality (F)**				**Circularity (C)**				
	Slenderness	**Weight**	**Building Decree**	**Total**	**Demountability**	**Sustainability**	**Logistics**	**Flexibility**	**Total**
Score1	*15*	*15*	*10*	***40***	*[-]*	*30*	*12*	*36*	***60***
Score2	*16.875*	*16.875*	*11.25*	***45***	*[-]*	*27.5*	*11*	*16.5*	***55***
Score3	*18.75*	*18.75*	*12.5*	***50***	*[-]*	*25*	*10*	*15*	***50***
Score4	*20.625*	*20.625*	*13.75*	***55***	*[-]*	*22.5*	*9*	*13.5*	***45***
STC Floors									
Scores (Total:100)	**Functionality (F)**				**Circularity (C)**				
	Slenderness	**Weight**	**Building Decree**	**Total**	**Demountability**	**Sustainability**	**Logistics**	**Flexibility**	**Total**
Score1	*11.42*	*11.42*	*17.16*	***40***	*16.67*	*16.66*	*16.67*	*10*	***60***
Score2	*12.86*	*12.86*	*19.28*	***45***	*15.27*	*15.27*	*15.27*	*9.19*	***55***
Score3	*14.28*	*14.28*	*21.44*	***50***	*13.88*	*13.88*	*13.88*	*8.36*	***50***
Score4	*15.71*	*15.71*	*23.58*	***55***	*12.5*	*12.5*	*12.5*	*7.5*	***45***

The total contribution of a parameter is the product of the maximum rating of a parameter with the respective weight factor. Thus, from the required percentage, the weight factors are obtained.

B.3.1 MCA of Timber Decks

The summary of the ratings for timber decks is given below:

Table B.31: Summary of Ratings for Timber Decks.

Functional Unit 1							
Timber Deck	**Slenderness**	**Weight**	**Building Decree**	**Number of Elements**	**Sustainability**	**Logistics**	**Flexibility**
CLT_SS260	*5*	*1*	*3.2*	*4*	*2*	*5*	*0*
CLT_OR360	*3*	*2*	*2.6*	*4*	*4*	*3*	*1*
CLT_BE320	*3*	*3*	*3.9*	*4*	*4*	*4*	*1*
LK240	*5*	*4*	*1.2*	*2*	*5*	*5*	*1*
LF240	*5*	*5*	*1.2*	*3*	*5*	*5*	*1*
KR_OR475	*1*	*5*	*3.7*	*4*	*1*	*1*	*0*
KR_BE410	*2*	*4*	*4.5*	*4*	*1*	*2*	*1*
KR_OB428	*2*	*4*	*3.7*	*4*	*1*	*1*	*0*
Functional Unit 2							
Timber Deck	**Slenderness**	**Weight**	**Building Decree**	**Number of Elements**	**Sustainability**	**Logistics**	**Flexibility**
CLT_SS180	*4*	*1*	*4.2*	*4*	*2*	*4*	*0*
CLT_OR260	*2*	*3*	*4.1*	*4*	*3*	*4*	*1*
[-]	*[-]*	*[-]*	*[-]*	*[-]*	*[-]*	*[-]*	*[-]*
LK160	*5*	*4*	*1.7*	*2*	*5*	*5*	*1*
LF160	*5*	*4*	*1.7*	*3*	*5*	*5*	*1*
KR_OR325	*1*	*5*	*4.2*	*4*	*2*	*3*	*0*
KR_BE250	*2*	*4*	*5*	*4*	*1*	*4*	*1*
KR_OB293	*1*	*4*	*4.2*	*4*	*1*	*3*	*0*

The weight factors used to obtain the final scores are given below.

Table B.32: Weight Factors used for MCA of Timber Decks.

Score (Total:1000)	Slenderness	Weight	Building Decree	Number of Elements	Sustainability	Logistics	Flexibility
Score 1	22.8	22.8	34.2	33.3	33.3	33.3	100
Score 2	25.7	25.7	38.5	30.5	30.5	30.5	91.6
Score 3	28.5	28.5	42.8	27.7	27.7	27.7	83.3
Score 4	31.4	31.4	47.1	25	25	25	75

B.3.2 MCA of Steel Beams

The summary of the ratings for steel beams is given below:

Table B.33: Summary of Ratings for Steel Beams.

Functional Unit 1						
Steel Beams	**Slenderness**	**Weight**	**Fire Protection**	**Sustainability**	**Logistics**	**Flexibility**
IPE550	2	5	1	5	2	0
HEA500	3	3	3	3	2	0
HEB450	3	3	4	3	1	1
HEM320	5	1	5	1	1	1
CIPE672	1	5	2	5	2	0
CHEA581	2	4	3	4	2	0
CHEB535	2	4	3	4	2	0
CHEM422	4	2	5	3	1	1
THQ420	4	2	4	2	1	0
SFB410	4	2	4	2	1	0
IFB410	4	3	3	3	1	0
RHS500	3	4	2	4	2	0
DL425	4	3	1	3	2	1
Functional Unit 2						
Steel Beams	**Slenderness**	**Weight**	**Fire Protection**	**Sustainability**	**Logistics**	**Flexibility**
IPE500	2	5	1	4	2	0
HEA450	3	3	3	3	1	0
HEB400	3	3	3	2	1	1
HEM300	5	1	5	1	1	1
CIPE594	1	5	2	5	2	0
CHEA489	2	4	2	4	2	0
CHEB445	3	4	3	3	1	0
[-]	[-]	[-]	[-]	[-]	[-]	[-]
THQ340	5	2	4	2	1	0
SFB340	5	1	4	1	1	0
IFB340	5	2	4	2	1	0
RHS450	3	3	4	3	1	0
DL345	5	2	2	2	1	1

The weight factors used to obtain the final scores are given below.

Table B.34: Weight Factors used for MCA of Steel Beams.

Score (Total:1000)	Slenderness	Weight	Fire Protection	Sustainability	Logistics	Flexibility
Score 1	30	30	20	60	60	180
Score 2	33.75	33.75	22.5	55	55	165
Score 3	37.5	37.5	25	50	50	150
Score 4	41.25	41.25	27.5	45	45	135

B.3.3 MCA of STC Floors

The summary of the ratings for *STC* Floors is given below:

Table B.35: Summary of Ratings for STC Floors.

Timber Deck	Slenderness	Weight	Building Decree	Number of Elements	Sustainability	Logistics	Flexibility
Functional Unit 1							
STC1a	*1*	*1*	*3.11*	*5*	*1*	*5*	*1*
STC2a	*2*	*5*	*1.56*	*4.2*	*5*	*5*	*1*
STC3a	*4*	*3*	*2.6*	*2.6*	*3*	*1*	*1*
STC4a	*4*	*5*	*1.38*	*2.4*	*5*	*5*	*0*
STC5a	*3*	*4*	*2.6*	*2.2*	*3*	*1*	*1*
STC6a	*3*	*5*	*1.29*	*2*	*5*	*5*	*0*
STC7a	*4*	*3*	*2.6*	*2.6*	*3*	*1*	*1*
STC8a	*5*	*5*	*1.38*	*2.4*	*5*	*5*	*0*
Functional Unit 2							
Timber Deck	**Slenderness**	**Weight**	**Building Decree**	**Number of Elements**	**Sustainability**	**Logistics**	**Flexibility**
STC1b	*1*	*1*	*4.05*	*5*	*1*	*1*	*1*
STC2b	*2*	*4*	*2.3*	*4.2*	*4*	*5*	*1*
STC3b	*5*	*3*	*4.1*	*2.6*	*3*	*3*	*2*
STC4b	*4*	*5*	*1.85*	*2.4*	*5*	*5*	*1*
STC5b	*3*	*4*	*3.95*	*2.2*	*4*	*3*	*1*
STC6b	*3*	*5*	*1.7*	*2*	*5*	*5*	*0*
STC7b	*5*	*3*	*4.1*	*2.6*	*3*	*3*	*1*
STC8b	*5*	*5*	*2*	*2.4*	*5*	*5*	*0*

The weight factors used to obtain the final scores are given below.

Table B.36: Weight Factors used for MCA of STC Floors.

Score (Total:1000)	Slenderness	Weight	Building Decree	Number of Elements	Sustainability	Logistics	Flexibility
Score 1	*22.8*	*22.8*	*34.2*	*33.3*	*33.3*	*33.3*	*100*
Score 2	*25.7*	*25.7*	*38.5*	*30.5*	*30.5*	*30.5*	*91.6*
Score 3	*28.5*	*28.5*	*42.8*	*27.7*	*27.7*	*27.7*	*83.3*
Score 4	*31.4*	*31.4*	*47.1*	*25*	*25*	*25*	*75*

C. Calculations on Case Study

C.1 Wind Loads

Wind Loads have been determined according *EC1* for Wind Loads [59].

$$v_{b,0} = 27.5\, m/s\,, basic\ wind\ velocity\ for\ Terrain\ Category\ II, c_{dir} = 1, c_{season} = 1$$

Basic Wind Velocity, $v_b = c_{dir} * c_{season} * v_{b,0} = 27.5\, m/s$

Assumed to be in Terrain Category IV, $z_{min} = 10\, m$, $z_0 = 1$, $z_{0,II} = 0.05\, m$

$$k_r = 0.19 * \left(\frac{z_0}{z_{0,II}}\right)^{0.07} = 0.234, \quad k_l = 1, \quad c_0(z) = 1\ (flat\ terrain\ assumed)$$

For $z_{min} < z < z_{max}$, $c_r(z) = k_r * ln(\frac{z}{z_0}) = 0.234 * ln(z)$

Mean Wind Velocity at height z from the ground, $v_m(z) = c_r(z) * c_0(z) * v_b = 6.435 * ln(z)$

For $z_{min} < z < z_{max}$, Turbulence Intensity, $I_v(z) = \frac{k_l}{c_o(z)*ln(\frac{z}{z_0})} = \frac{1}{ln(z)}$

Air Density, $\rho_{air} = 1.25\, kgm^{-3}$

For calculations, $z = 12.52\, m$, Storey Height, $H_s = 3.205\, m$

Peak Velocity Pressure, $q_p(z) = (1 + 7 * I_v(z)) * \frac{\rho_{air}*v_m(z)^2}{2} = 0.623\, kNm^{-2}$

External pressure coefficients (c_{pe}) have been determined based on wind loading in the parallel and perpendicular direction. The respective internal pressure coefficients (c_{pi}) have been conservatively taken as the more onerous value of +0.2 and -0.3, even though it is less likely that the openings would remain open in the event of heavy winds.

Total wind loads on the edge beam have been calculated as follows:

$$q_w = H_s * c_{p,net} * q_p = 1.99 * c_{p,net}\ in\ kNm^{-1}$$

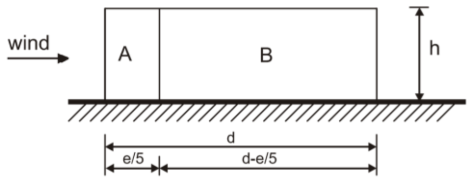
Finally, these loads are adjusted for a 100-year reference period, according to Eq. 1. (with $\psi_{w,0} = 0.6$).The wind loads have been tabulated below in Table C.1.

Table C.1: Wind Load Calculations.

$q_p = 0.623\ kNm^{-2}$, $h = 12.5\ m$

$Area\ of\ panels > 10\ m^2\ for\ all\ Sections$

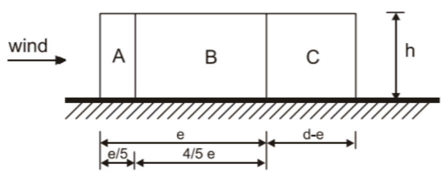
Wind Perpendicular to Long Side



$b = 21.2\ m, d = 10.9\ m, e = 21.2\ m$

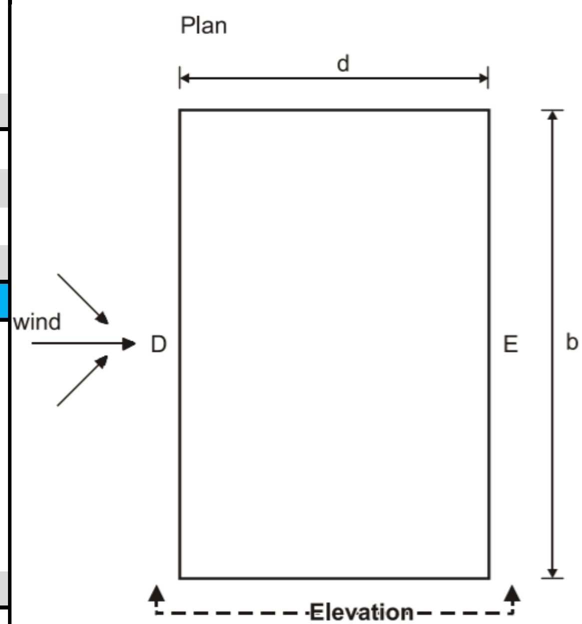
	A	B	C	D	E
c_{pe} [-]	-1.2	-0.8	NA	0.8	-0.51
c_{pi} [-]	0.2	0.2	NA	-0.3	0.2
$c_{pe,net}$ [-]	-1.2	-1	NA	1.1	-0.71
q_w[kNm^{-1}]	-2.46	-2.05	NA	2.25	-1.45

Wind Perpendicular to Short Side



$b = 10.9\ m, d = 21.2\ m, e = 10.9\ m$

	A	B	C	D	E
c_{pe} [-]	-1.2	-0.8	-0.5	0.74	-0.39
c_{pi} [-]	0.2	0.2	0.2	-0.3	0.2
$c_{pe,net}$ [-]	-1.4	-1	-0.7	1.04	-0.59
q_w[kNm^{-1}]	-2.87	-2.05	-1.43	2.13	-1.21

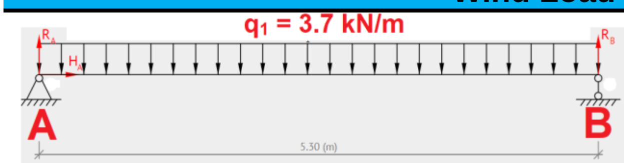


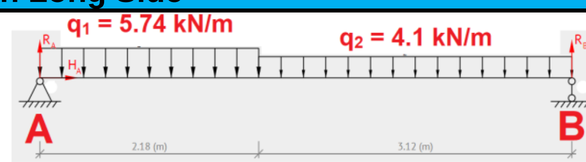
C.2 Calculations on DA1_STC

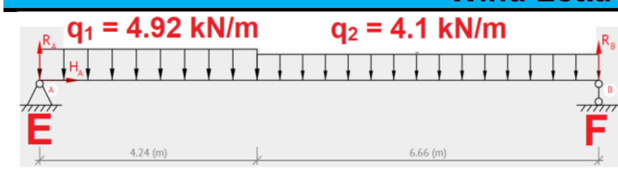
C.2.1 Action of Wind Loads

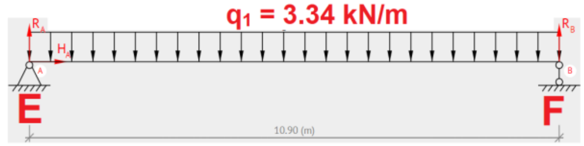
Characteristic Wind loads for the top-most storey have been determined in Appendix C.1. Based on this, and the geometry of the design (according to Section 5.2.1), the action of wind loads is determined. It is conservatively assumed that all connections are pinned, to obtain the most conservative actions of wind loads. 2 Scenarios of wind loads are considered: 1) Wind direction perpendicular to long side and 2) Wind direction perpendicular to short side. The maximum value is taken. Due to the presence of cross beams, it is assumed that the wind load from the long side is not transferred to the timber slabs. Only wind loads transferred to the slabs are those acting on the short side. The values of wind actions is given below in Table C.2.

Table C.2: Action of Wind Loads.

<table>
<tr><th>Wind Perpendicular to Long Side</th><th>Wind Perpendicular to Short Side</th></tr>
<tr><td colspan="2">Wind Load on Long Side</td></tr>
<tr><td>

$V_{Ek,A/B} = 9.8\ kN,\ M_{Ek,max} = 13\ kNm$</td>
<td>

$V_{Ek,A} = 13.7\ kN, V_{Ek,B} = 11.6\ kN$
$M_{Ek,max} = 16.41\ kNm$</td></tr>
<tr><td colspan="2">Wind Load on Short Side</td></tr>
<tr><td>

$V_{Ek,E} = 25.2\ kN, V_{Ed,F} = 23\ kN$
$M_{Ek,max} = 64.63\ kNm$</td>
<td>

$V_{Ek,E/F} = 18.2\ kN,\ M_{Ek,max} = 49.6\ kNm$</td></tr>
<tr><td colspan="2">Axial Force in Cross Beams</td></tr>
<tr><td colspan="2">$b = 10.9\ m, d = 5.3\ m$
Deep Beam, $b < 3 * d = 15.9\ m$, → YES
Lever Arm, $z = 0.2 * b + 0.4 * d = 4.3\ m$</td></tr>
<tr><td>Side Cross Beams (AJ/EF)
$M_{Ek,S} = M_{Ed,max} = 64.63\ kNm$
Force due to Moment, $= \frac{M_{Ek,S}}{z} = 15\ kN$
Total, $N_{Ek,cb,S} = 15 + 9.8 = 24.8\ kN$
Middle Cross Beams (BI/CH/DG)
Total, $N_{Ek,cb,M} = 9.8\ kN$</td>
<td>Side Cross Beams (AJ/EF)
$M_{Ek,S} = M_{Ed,max} = 49.6\ kNm$
Force due to Moment, $= \frac{M_{Ek,S}}{z} = 11.53\ kN$
Total, $N_{Ek,cb,S} = 13.7 + 11.53 = 25.23\ kN$
Middle Cross Beams (BI/CH/DG)
Total, $N_{Ek,cb,M} = 11.6\ kN$</td></tr>
<tr><td colspan="2">Axial Force in Edge Beams</td></tr>
<tr><td colspan="2">$b = 5.3\ m, d = 10.9\ m$
Lever Arm, $z = d = 10.9\ m$, due to presence of cross beams</td></tr>
<tr><td>$M_{Ek,L} = M_{Ed,max} = 13\ kNm$
Force due to Moment, $= \frac{M_{Ek,L}}{z} = 1.2\ kN$</td>
<td>$M_{Ek,L} = M_{Ed,max} = 16.41\ kNm$
Force due to Moment, $= \frac{M_{Ek,S}}{z} = 1.5\ kN$</td></tr>
</table>

Total, $N_{Ek,eb} = 25.2 + 1.2 = 25.4\ kN$	*Total,* $N_{Ek,eb} = 18.2 + 1.5 = 19.7\ kN$
Axial Force on Timber Slabs	
Along y - direction (Longitudinal) $n_{y,Ek,T} = 4.42\ kN/m\ (Avg)$ ***Along x - direction (Transverse)*** $n_{x,Ek,T} = \frac{M_{Ek,S}}{z} = 15\ kN\ over\ 4.3\ m$	***Along y - direction (Longitudinal)*** $n_{y,Ek,T} = 3.34\ kN/m$ ***Along x - direction (Transverse)*** $n_{x,Ek,T} = \frac{M_{Ek,S}}{z} = 11.53\ kN\ over\ 4.3\ m$
Force on Connections	
Slab – Side Cross Beam Connection	
$V_{x,Ek,s-scb,Total} = N_{Ek,cb,S} = 24.8\ kN$ $V_{y,Ek,s-scb,Total} = n_{y,Ek,T} * B = 48.2\ kN$	$V_{x,Ek,s-scb,Total} = N_{Ek,cb,S} = 25.23\ kN$ $V_{y,Ek,s-scb,Total} = n_{y,Ek,T} * B = 36.41\ kN$
Slab – Middle Cross Beam Connection	
$V_{x,Ek,s-mcb,Total} = N_{Ek,cb,M} = 9.8\ kN$ $V_{y,Ek,s-mcb,Total} = n_{y,Ek,T} * B = 48.2\ kN$	$V_{x,Ek,s-mcb,Total} = N_{Ek,cb,M} = 11.6\ kN$ $V_{y,Ek,s-mcb,Total} = n_{y,Ek,T} * B = 36.41\ kN$
Slab – Slab Connection	
$Width\ of\ Timber\ Slab, b_T = \frac{10.9}{11} = 990\ mm$ $V_{x,Ek,s-s,Total} = n_{x,Ek,T} = 15\ kN$ $V_{y,Ek,s-s,Total} = V_{Ek,E} - 0.99 * 4.92 = 20.32\ kN$	$Width\ of\ Timber\ Slab, b_T = \frac{10.9}{11} = 990\ mm$ $V_{x,Ek,s-s,Total} = n_{x,Ek,T} = 11.53\ kN$ $V_{y,Ek,s-s,Total} = V_{Ek,E} - 0.99 * 3.34 = 14.9\ kN$

C.2.2 Design of Timber Slabs

Span of Timber Slab, $L_S = 5.3\ m$

Timber Section Used, $\boldsymbol{LFE160}$

Cross Section Dimensions,

$$H_T = 160\ mm,\ b_T = 990\ mm,\ t_{T,w} = 40\ mm,\ t_{T,f} = 34\ mm,\ b_{T,f} = 237.5\ mm$$

Cross Section Over Middle Supports,

$$H_T = 160\ mm,\ b_{T,eff} = 610\ mm,\ S_{T,t} = 40\ mm,\ t_{T,tf} = 34\ mm,\ t_{T,bf} = 53\ mm$$

Transverse Stiffener Dimensions,

$$S_{T,s} = \frac{(5300 - 610)}{4} = 1172.5\ mm,\ S_{T,t} = 25\ mm$$

Weight of Timber Slabs, $G_S = 0.36\ kNm^{-2}$

Total Dead Load, $G = 0.36 + 1.5 = 1.86\ kNm^{-2}$

Live Load, $Q = 3.6\ kNm^{-2}, (for\ 100\ years\ with\ Reorientation)$

Since imposed loads are much higher than wind loads, the governing load combination will be with the imposed loads as the leading variable (from Table 5.1)

$$q_{ULS} = 0.89 * 1.35 * G + 1.5 * Q_q = 7.85\ kNm^{-2}$$

$$q_{SLS} = 1 * G + 1 * Q_q = 5.64\ kNm^{-2}$$

As calculated in Table C.2, wind loads produce axial forces in the timber slabs in the longitudinal and transverse direction. However, these are the secondary variable load (lower combination factor).

ULS Axial Loads, $n_{y,Ed,T} = 1.5 * 0.6 * 4.42 = 3.98 kN/m,\ N_{x,Ed,T} = 1.5 * 0.6 * 15 = 13.5\ kN$

Strength Parameters:

Strength reduction for Size Effects, $t_{max} = 51\ mm \rightarrow k_h = min(1.3, \frac{150}{t_{max}}^{0.2}) = 1.24$

Material Factor, $\gamma_M = 1.3$

Strength reduction for duration of load (ULS), $k_{mod} = 0.8$

MOE of Timber, $E_{1,T,ULS} = 11000/(1 + 0.5 * 0.6) = 8461\ MPa$

Design strengths,

$$f_{m,d} = \frac{0.8 * 1.24 * 24}{1.3} = 18.25\ MPa, \qquad f_{v,0,d} = \frac{0.8 * 1.24 * 4}{1.3} = 3.05\ MPa,$$

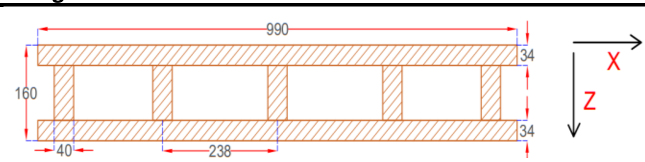
$$f_{v,90,d} = \frac{0.8 * 1.24 * 1}{1.3} = 0.763\ MPa, \qquad f_{c,0,d} = \frac{0.8 * 1.24 * 21}{1.3} = 16.02\ MPa,$$

$$f_{t,0,d} = \frac{0.8*1.24*14}{1.3} = 10.7\ MPa, \quad f_{t,90,d} = \frac{0.8*1.24*0.4}{1.3} = 0.31\ MPa,$$

$$f_{c,90,d} = \frac{0.8 * 1.24 * 2.5}{1.3} = 1.91\ MPa,$$

The slabs are simply supported. Different checks are done on the timber slabs according to the formulas given in Section D.1. These are given below in Table C.3.

C.3: Design Checks on LFE160.

General Properties of LFE160	
$I_T = 320.4 * 10^6\ mm^4/m$ $A_T = 87000\ mm^2/m$ $z_{T,max} = 80\ mm$ $S_{T,yz,web} = 2.491 * 10^6 mm^3/m$	 ***Figure C.1: LFE160 Cross Section.***
Maximum Bending Stresses	**Buckling of Compression Flange**
$m_{y,Ed,ULS} = 27.65\ kNm/m$ $\sigma_{T,max} = \frac{27.65 * 80}{320.4} = 6.9\ MPa$ $\boldsymbol{UC1{:}= \frac{6.9}{18.25} = 0.378 < 1}$	$n_{y,Ed,ULS} = 3.978\ kN/m$ $z_{T,tf} = 80 - 0.5 * 34 = 63\ mm$ $\sigma_{T,c,bf} = 6.8 + \frac{3.98 * 1000}{87000} = 6.94\ MPa$ $\lambda_z = 18.54, k_z = 0.55, k_{cz} = 0.99$ $\boldsymbol{UC2{:} \frac{6.94}{0.99 * 16.8} = 0.343 < 1}$

Shear Stress in Web

$$v_{y,Ed,ULS} = 20.8\ kN/m \rightarrow \tau_{T,w,max} = \frac{20.1 * 2.491 * 1000}{320.4 * 200} = 0.87\ MPa$$

$$\boldsymbol{UC3}:\ \frac{\mathbf{0.87}}{\mathbf{3.2}} = \mathbf{0.2474 < 1}$$

Check for Shear Buckling of Webs

$\lambda_{T,w} = \frac{92}{40} = 2.3 < 70$,

$F_{T,v,Ed} = \frac{20.8}{5} = 4.2\ kN\ per\ web$

$F_{T,v,Rd} = 92 * 40 * 3.05 = 11.2\ kN$

$for\ \lambda_{T,w} < 35$

$$\boldsymbol{UC4}:\ \frac{\mathbf{4.72}}{\mathbf{11.2}} = \mathbf{0.42 < 1}$$

Shear Stress in Glue Line

$S_{T,yz,Glue\ Line} = 3.06 * 10^{6} mm^{3}/m$

$V_{T,Ls,Ed} = \frac{7.85*3.06*5300^{2}}{320.4*4*1000} = 391\ kN/m$

$V_{T,Ls,Rd} = \frac{5300*200*0.8}{2} = 392.2\ kN/m$

$$\boldsymbol{UC5}:\ \frac{\mathbf{355.7}}{\mathbf{392.2}} = \mathbf{0.997 < 1}$$

Check for Axial Force in Transverse Stiffeners:

$N_{x,Ed,T} = 1.5 * 0.6 * 15 =$
$13.5\ kN\ over\ 4300\ mm$

$S_{T,s} = 1172.5\ mm$, $S_{T,t} = 25\ mm$

Number of stiffeners effective, = $\frac{4300}{1172.5} \sim 3$

Resistance for Tension, = $3 * 25 * 92 * 10.7 = 73.83\ kN$

$\lambda_z = 32.9, k_z = 0.67, k_{cz} = 0.94$

Resistance under compression, = $3 * 25 * 92 * 0.94 * 16.02 = 103.9\ kN$

$$N_{x,Rd,T} = 73.83\ kN$$

$$\boldsymbol{UC10}:\ \frac{\mathbf{15}}{\mathbf{73.83}} = \mathbf{0.2 < 1}$$

Check for Deflections

$w_{T,initial,G} = 4.9\ mm$, $w_{T,initial,Q} = 10.4\ mm$

$w_{T,initial} = 15.8\ mm$

$$w_{T,initial,max} = \frac{5300}{300} = 17.66\ mm,$$

$$\boldsymbol{UC6}:\ \frac{\mathbf{15.8}}{\mathbf{17.66}} = \mathbf{0.894 < 1}$$

$w_{T,final} = 4.9 * 1.6 + 10.1 * 1.3 = 20.97\ mm$

$w_{T,final,max} = \frac{5300}{250} = 21.2\ mm,$

$$\boldsymbol{UC7}:\ \frac{\mathbf{20.97}}{\mathbf{21.2}} = \mathbf{0.99}$$

Check for Vibrations:

$M = 36.3\ kg/m^{2}$,

$(EI_{T,final})_{l} = 3.5 * 10^{12}\ Nmm^{2}/m$

$$f_1 = \frac{3.14}{2 * 5300^{2}} * \sqrt{\frac{3.5 * 10^{12}}{36.3}} = 17.2\ Hz$$

$$\boldsymbol{UC8}:\ \boldsymbol{f_1} = \mathbf{17.2 > 8}\ \boldsymbol{Hz}$$

$$a = \frac{5300^{3}}{48 * 3.5 * 10^{12}} = 0.88\ mm/kN$$

$$\boldsymbol{UC8}:\ \boldsymbol{a} = \mathbf{0.88 < 1}\ \boldsymbol{mm/kN}$$

$(EI_{T,final})_{b} = 0.163 * 10^{12}\ Nmm^{2}/m$,

$\zeta = 0.01$

$$n_{40} = \left[\left\{\left(\frac{40}{17.2}\right)^{2} - 1\right\} * \left\{\frac{10900^{4} * 3.6}{5300^{4} * 0.163}\right\}\right]^{0.25} = 6.2$$

$$v = \frac{4 * (0.4 + 0.6 * 6)}{36.3 * 5.3 * 10.9 + 200} = 0.007$$

$$v_{limit} = 120^{17.2*0.01 - 1} = 0.019$$

$$\boldsymbol{UC9}:\ \boldsymbol{v} = \mathbf{0.007 < 0.019}$$

C.2.3 Design of Slab – Slab Connections

Actions:

$V_{x,Ek,s-s,Total} = 15\ kN$ $V_{y,Ek,s-s,Total} = 20.32\ kN$ *(Wind Load Perpendicular to Long Side)*

$V_{screw,X,Ed,ULS} = \frac{1.5*15}{16} = 1.4\ kN, V_{screw,Y,Ed,ULS} = \frac{1.5*20.32}{16} = 1.9\ kN\ (16\ screws\ per\ slab)$

Resultant Force, $V_{R,Ed,ULS} = 2.36\ kN$

Grade 4.6 2xS6 Inclined Screws crosswise at 660 mm C/C

Lateral load carrying capacity of Inclined Screws,

$\alpha = 45°,\ d = 6\ mm, t_{T,tf} = 34\ mm\ ,\ l_{screw} = \frac{34}{sin45} = 48\ mm,$

$f_{ax,0,k} = 0.0036 * 350^{1.5} = 23.6\ MPa\ ,\ f_{ax,\alpha,k} = \frac{23.6}{sin45^2+1.5*cos45^2} = 18.85\ MPa$

$F_{ax,\alpha,Rd} = (3.14 * 6 * 48)^{0.8} * 18.85 = 2.72\ kN$

$$\boldsymbol{UC}: \ \frac{\mathbf{2.36}}{\mathbf{2.72}} = \mathbf{0.87 < 1}$$

C.2.4 Design of Edge Beams

Actions:

$N_{Y,Ed,ULS} = 1.5 * 25.4 = 38.1kN$

SHS 70x3.2

$L_{cr} = 5300\ mm, l = b = 70\ mm, t = 3.2\ mm, r = 4\ mm$

$\epsilon = 0.81,\ \lambda_{w/f} = \frac{70-2*(3.2+4)}{3.2*0.81} = 17.4 < 33$ → *Class 1 Section*

$I = 0.623 * 10^6\ mm^4, A = 844\ mm^2, i_{z/y} = \sqrt{\frac{0.623*10^6}{844}} = 27.2\ mm, \lambda_{z/y} = \frac{5300}{27.2} = 194.8$

$\lambda_1 = 93.9 * 0.81 = 74.5, \boldsymbol{\lambda} = \frac{194.8}{74.5} = 2.58, \Phi = 4.18, \chi = 0.134$

$N_{b,Rd} = \frac{0.134*844*355}{1} = 40.1\ kN$

$$\boldsymbol{UC}: \ \frac{\mathbf{38.1}}{\mathbf{40.1}} = \mathbf{0.95 < 1}$$

C.2.5 Design of Side Cross Beams

Actions:

Imposed loads, $q_{ULS} = 0.5 * 5.3 * (1.2 * 1.86 + 1.5 * 3.6) + 1.2 * 0.96 = 21.3\ kN/m$

$V_{x,Ed,ULS} = 0.5 * 10.9 * 21.3 = 116.5\ kN,\ M_{x,Ed,ULS} = 0.125 * 10.9^2 * 21.3 = 317.4\ kNm,$

$q_{SLS} = 0.5 * 5.3 * (1.86 + 3.6) + 0.96 = 15.4\ kN/m$

Axial force due to wind load, $N_{X,Ed,ULS} = 0.9 * 25.23 = 22.7\ kN$

Table C.4: Design Checks on Side Cross Beams.

HEA320

$h = 310\ mm, b = 300\ mm,\ t_f = 15.5\ mm, t_w = 9\ mm, r = 27\ mm, q_g = 0.96\ kN/m$

$\epsilon = 0.81,\ \lambda_f = \frac{0.5*(300-2*27-9)}{15.5*0.81} = 9.43 < 10, \lambda_w = \frac{310-2*(27+15.5)}{9*0.81} = 30.8 < 72 \rightarrow$ *Class 2*

$L = 10900\ mm,\ A = 12437\ mm^2, S_{yz} = 0.685 * 10^6\ mm^3,$

$I_y = 229.3 * 10^6\ mm^4, W_{el,y} = 1.479 * 10^6\ mm^3$

Check for Shear

$\tau_{Ed} = \frac{116.5*1000*0.685}{229.3*9} = 38.7\ Nmm^{-2}$

$\boldsymbol{UC1}: \frac{38.7}{355/\sqrt{3}} = \mathbf{0.19 < 1}$

Check for Bending

$M_{el,Rd} = \frac{355*1.479*10^6}{1} = 525.1\ kNm$

$\boldsymbol{UC2}: \frac{\mathbf{317.4}}{\mathbf{525.1}} = \mathbf{0.61 < 1}$

Check for Flexural Buckling

$N_{pl,Rd} = \frac{12437*355}{1} = 4415\ kN, L_{cr} = 10900\ mm$

Major Axis (yy)

$I_y = 229.3 * 10^6\ mm^4,$

$i_y = 135.7\ mm,\ \overline{\lambda_y} = \mathbf{1}.08,$

$\Phi_y = 1.23, \chi_y = 0.55$

$N_{b,pl,Rd} = 0.55 * 4415 = 2424\ kN$

$\boldsymbol{UC3}: \frac{\mathbf{22.7}}{\mathbf{2424}} = \mathbf{0.009 < 1}$

Minor Axis (zz)

$I_z = 69.85 * 10^6\ mm^4,$

$i_z = 74.9\ mm,\ \overline{\lambda_z} = \mathbf{1}.95,$

$\Phi_z = 2.83, \chi_z = 0.204$

$N_{b,pl,Rd} = 0.204 * 4415 = 903\ kN$

$\boldsymbol{UC3}: \frac{\mathbf{22.7}}{\mathbf{903}} = \mathbf{0.025 < 1}$

Check for Bending, Shear and Axial Force

$\sigma_{xx,Ed} = \frac{22.7*1000}{12437} = 1.82\ Nmm^{-2} \rightarrow$ Axial force

$\sigma_{xx,tf,Ed} = \frac{317.4}{1.479} = 214.6\ Nmm^{-2} \rightarrow$ Bending

$\tau_{yz,w,Ed} = 38.7\ Nmm^{-2} \rightarrow$ Shear

At Supports, $\sigma_{von\ mises,Ed} = 67.03\ Nmm^{-2}$

$\boldsymbol{UC4}: \frac{\mathbf{67.03}}{\mathbf{355}} = \mathbf{0.19 < 1}$

At Mid-span, $\sigma_{von\ mises,Ed} = 216.4\ Nmm^{-2}$

$\boldsymbol{UC4}: \frac{\mathbf{216.4}}{\mathbf{355}} = \mathbf{0.61 < 1}$

Check for Beam – Column Buckling

$C_{my} = 0.95, k_{yy} = 0.957, k_{zy} = 0$

$\chi_y = 0.55\ ,\ \chi_{LTB} = 1$

$\boldsymbol{UC7}: \frac{\mathbf{22.7}}{\mathbf{0.55 * 2424}} + \mathbf{0.957} * \frac{\mathbf{317.4}}{\mathbf{1 * 525.1}} = \mathbf{0.587 < 1}$

Check for LTB

The slabs provide lateral restrains to the beams, therefore , $\chi_{LTB} = 1$

$\boldsymbol{UC6}: \frac{\mathbf{317.4}}{\mathbf{525.1}} = \mathbf{0.61 < 1}$

Check for Deflections

$q_{SLS,G} = 5.88\ kN/m,\ q_{SLS,Q} = 9.54\ kN/m$

Precamber $\rightarrow \Delta_G = 23.6\ mm$

$\Delta_{SLS,Q} = 38.2\ mm$

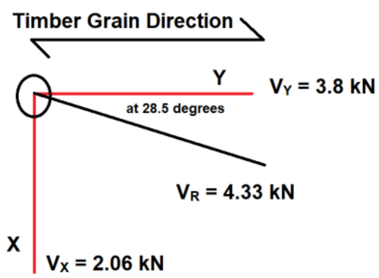
$w_{max} = 10900/250 = 43.6\ mm$

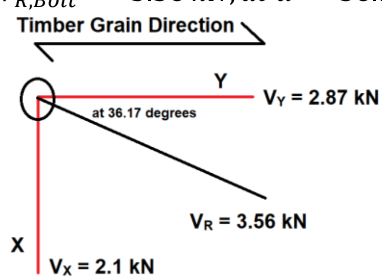
$\boldsymbol{UC8}: \frac{\mathbf{38.2}}{\mathbf{43.6}} = \mathbf{0.87 < 1}$

C.2.6 Design of Slab – Side Cross Beam Connections

Wind is the leading Variable Load.

Table C.5: Design Checks on Slab – Side Cross Beam Connections.

<table>
<tr><td colspan="2">19 x Grade 4.6 M14 Bolts @ 566 mm C/C
$s_{sc} = 566\,mm, d = 14\,mm$
$N_{sc,eff,x} = 19, \quad N_{sc,eff,y} = 18$
$f_{h,k} = 24.68\,MPa, M_{y,Rk} = 0.114\,kNm$</td></tr>
<tr><td>Wind Perpendicular to Long Side</td><td>Wind Perpendicular to Short Side</td></tr>
<tr><td>$V_{X,Bolt} = \frac{1.5*24.8}{19} = 2.06kN$,
$V_{Y,Bolt} = \frac{1.5*48.2}{18} = 3.805\,kN,$
$V_{R,Bolt} = 4.33\,kN, at\ \alpha = 28.5\,°$


$f_{h,k,\alpha} = 22.86\,MPa,$
$F_{Rd,Bolt,\alpha} = 4.35\,kN$
$\boldsymbol{UC}: \ \frac{\mathbf{4.33}}{\mathbf{4.35}} = \mathbf{0.994 < 1}$</td><td>$V_{X,Bolt} = \frac{1.5*25.2}{19} = 2.1\,kN$,
$V_{Y,Bolt} = \frac{1.5*36.4}{18} = 2.87\,kN,$
$V_{R,Bolt} = 3.56\,kN, at\ \alpha = 36.17\,°$


$f_{h,k,\alpha} = 22.\,MPa,$
$F_{Rd,Bolt,\alpha} = 4.19\,kN$
$\boldsymbol{UC}: \ \frac{\mathbf{3.56}}{\mathbf{4.19}} = \mathbf{0.85 < 1}$</td></tr>
</table>

C.2.7 Design of Cross Beams

<u>With Composite Action</u>

Actions:

Imposed loads, $q_{ULS} = 5.3 * (1.2 * 1.86 + 1.5 * 3.6) + 1.2 * 1.1 = 41.7\,kN/m$

$V_{x,Ed,ULS} = 0.5 * 10.9 * 41.7 = 227.6\,kN,\ M_{x,Ed,ULS} = 0.125 * 10.9^2 * 41.7 = 620.3\,kNm,$

$q_{SLS} = 5.3 * (1.86 + 3.6) + 1.1 = 30.03\,kN/m$

Axial force due to wind load (secondary variable load), $N_{X,Ed,ULS} = 0.9 * 11.6 = 10.44\,kN$

Calculations on Composite Action i.e., $M_{el,STC,Rd}$ and EI_{eff} are given in Section 6.4. For the remaining checks, it is assumed that the loads are taken by the beam alone.

Table C.6: Design Checks on Cross Beams (with Composite Action).

HEA360

$h = 350\ mm, b = 300\ mm,\ t_f = 17.5\ mm, t_w = 10\ mm, r = 27\ mm, q_g = 1.1\ kN/m$

$\epsilon = 0.81,\ \lambda_f = \frac{0.5*(300-2*27-10)}{17.5*0.81} = 8.32 < 9, \lambda_w = \frac{350-2*(27+17.5)}{10*0.81} = 32.3 < 72 \rightarrow$ *Class 1*

$L = 10900\ mm,\ A = 14276\ mm^2, S_{yz} = 0.874 * 10^6\ mm^3,$

$I_y = 330.9 * 10^6\ mm^4, W_{el,y} = 1.89 * 10^6\ mm^3$

Check for Shear

$\tau_{Ed} = \frac{227.6*1000*0.874}{330.9*10} = 60.1\ Nmm^{-2}$

$\boldsymbol{UC1}:\ \frac{\mathbf{60.1}}{\mathbf{355/\sqrt{3}}} = \mathbf{0.293 < 1}$

Check for Bending

$M_{el,Rd,STC} = 809.5\ kNm$

$\boldsymbol{UC2}:\ \frac{\mathbf{620.3}}{\mathbf{809.5}} = \mathbf{0.766 < 1}$

Check for Flexural Buckling

$N_{pl,Rd} = \frac{14276*355}{1} = 5068\ kN,\ L_{cr} = 10900\ mm$

Major Axis (yy)

$I_y = 330.9 * 10^6\ mm^4,$

$i_y = 152.2\ mm,\ \overline{\lambda_y} = 0.96,$

$\Phi_y = 1.09, \chi_y = 0.62$

$N_{b,pl,Rd} = 0.62 * 5068 = 3153\ kN$

$\boldsymbol{UC3}:\ \frac{\mathbf{10.44}}{\mathbf{3153}} = \mathbf{0.003 < 1}$

Minor Axis (zz)

$I_z = 78.9 * 10^6\ mm^4,$

$i_z = 74.3\ mm,\ \overline{\lambda_z} = 1.96,$

$\Phi_z = 2.86, \chi_z = 0.201$

$N_{b,pl,Rd} = 0.201 * 5068 = 1022\ kN$

$\boldsymbol{UC3}:\ \frac{\mathbf{10.44}}{\mathbf{1022}} = \mathbf{0.01 < 1}$

Check for Bending, Shear and Axial Force

$\sigma_{xx,Ed} = \frac{22.7*1000}{12437} = 1.82\ Nmm^{-2} \rightarrow$ Axial force

$\sigma_{xx,tf,Ed} = \frac{620.3*10^9*200*145}{8.72*10^{13}}$

$= 240.6\ Nmm^{-2} \rightarrow$ Bending

$\tau_{yz,w,Ed} = 38.7\ Nmm^{-2} \rightarrow$ Shear

At Supports, $\sigma_{von\ mises,Ed} = 67.03\ Nmm^{-2}$

$\boldsymbol{UC4}:\ \frac{\mathbf{67.03}}{\mathbf{355}} = \mathbf{0.19 < 1}$

At Mid-span, $\sigma_{von\ mises,Ed} = 216.4\ Nmm^{-2}$

$\boldsymbol{UC4}:\ \frac{\mathbf{242.42}}{\mathbf{355}} = \mathbf{0.68 < 1}$

Check for LTB

The slabs provide lateral restrains to the beams, therefore , $\chi_{LTB} = 1$

$\boldsymbol{UC6}:\ \frac{\mathbf{631.3}}{\mathbf{1 * 735}} = \mathbf{0.858 < 1}$

Check for Beam – Column Buckling

$C_{my} = 0.95, k_{yy} = 0.952, k_{zy} = 0$

$\chi_y = 0.62\ ,\ \chi_{LTB} = 1$

$\boldsymbol{UC7}:\ \frac{\mathbf{10.44}}{\mathbf{0.62 * 5068}} + \mathbf{0.952} * \frac{\mathbf{631.3}}{\mathbf{1 * 735}}$

$= \mathbf{0.72 < 1}$

Check for Deflections

$q_{SLS,G} = 11.6\ kN/m,\ q_{SLS,Q} = 19.1\ kN/m$

Precamber $\rightarrow \Delta_G = 31.52\ mm$

$EI_{STC} = 8.72 * 10^{13}\ Nmm^2$

$\Delta_{SLS,Q} = 42.12\ mm$

$w_{max} = 10900/250 = 43.6\ mm$

$\boldsymbol{UC8}:\ \frac{\mathbf{42.12}}{\mathbf{43.6}} = \mathbf{0.966 < 1}$

Without Composite Action

Actions:

Imposed loads, $q_{ULS} = 5.3 * (1.2 * 1.86 + 1.5 * 3.6) + 1.2 * 1.22 = 41.9 kN/m$

$V_{x,Ed,ULS} = 0.5 * 10.9 * 41.9 = 228.4\ kN,\ M_{x,Ed,ULS} = 0.125 * 10.9^2 * 41.9 = 622.5\ kNm,$

$q_{SLS} = 5.3 * (1.86 + 3.6) + 1.22 = 30.2\ kN/m$

Axial force due to wind load (secondary variable load), $N_{X,Ed,ULS} = 0.9 * 11.6 = 10.44\ kN$

Table C.7: Design Checks on Cross Beams (without Composite Action).

HEA400

$h = 390\ mm, b = 300\ mm,\ t_f = 19\ mm, t_w = 11\ mm, r = 27\ mm, q_g = 1.22 kN/m$

$\epsilon = 0.81,\ \lambda_f = \frac{0.5*(300-2*27-11)}{19*0.81} = 7.6 < 9, \lambda_w = \frac{390-2*(27+19)}{11*0.81} = 33.4 < 72$ → *Class 1*

$L = 10900\ mm,\ A = 15898\ mm^2, S_{yz} = 1.06 * 10^6\ mm^3,$

$I_y = 450.7 * 10^6\ mm^4, W_{el,y} = 2.31 * 10^6\ mm^3$

Check for Shear

$\tau_{Ed} = \frac{228.4*1000*1.06}{450.7*11} = 48.8\ Nmm^{-2}$

$\boldsymbol{UC1}: \ \frac{\mathbf{48.8}}{\mathbf{355/\sqrt{3}}} = \mathbf{0.24 < 1}$

Check for Bending

$M_{el,Rd} = \frac{355*2.31*10^6}{1} = 820.5\ kNm$

$\boldsymbol{UC2}: \ \frac{\mathbf{622.5}}{\mathbf{820.5}} = \mathbf{0.76 < 1}$

Check for Flexural Buckling

$N_{pl,Rd} = \frac{15898*355}{1} = 5643\ kN,\ L_{cr} = 10900\ mm$

Major Axis (yy)

$I_y = 450.7 * 10^6\ mm^4,$

$i_y = 168.4\ mm,\ \overline{\lambda_y} = 0.87,$

$\Phi_y = 0.95, \chi_y = 0.75$

$N_{b,pl,Rd} = 0.75 * 5643 = 4256\ kN$

$\boldsymbol{UC3}: \ \frac{\mathbf{10.44}}{\mathbf{4256}} = \mathbf{0.002 < 1}$

Minor Axis (zz)

$I_z = 85.64 * 10^6\ mm^4,$

$i_z = 73.4\ mm,\ \overline{\lambda_z} = \mathbf{1}.99,$

$\Phi_z = 2.79, \chi_z = 0.21$

$N_{b,pl,Rd} = 0.21 * 5643 = 1189\ kN$

$\boldsymbol{UC3}: \ \frac{\mathbf{10.44}}{\mathbf{1189}} = \mathbf{0.008 < 1}$

Check for Bending, Shear and Axial Force

$\sigma_{xx,Ed} = \frac{10.44*1000}{15898} = 0.66\ Nmm^{-2}$ → Axial force

$\sigma_{xx,tf,Ed} = \frac{622.5}{2.31} = 269.3\ Nmm^{-2}$ → Bending

$\tau_{yz,w,Ed} = 48.8\ Nmm^{-2}$ → Shear

At Supports, $\sigma_{von\ mises,Ed} = 84.5\ Nmm^{-2}$

$\boldsymbol{UC4}: \ \frac{\mathbf{84.5}}{\mathbf{355}} = \mathbf{0.24 < 1}$

At Mid-span, $\sigma_{von\ mises,Ed} = 270\ Nmm^{-2}$

$\boldsymbol{UC4}: \ \frac{\mathbf{270}}{\mathbf{355}} = \mathbf{0.76 < 1}$

Check for Beam – Column Buckling

$C_{my} = 0.95, k_{yy} = 0.952, k_{zy} = 0$

$\chi_y = 0.75\ ,\ \chi_{LTB} = 1$

$\boldsymbol{UC7}: \ \frac{\mathbf{10.44}}{\mathbf{0.75 * 5643}} + \mathbf{0.952} * \frac{\mathbf{622.5}}{\mathbf{1 * 820.5}} = \mathbf{0.724 < 1}$

Check for Deflections

$q_{SLS,G} = 11.1\ kN/m,\ q_{SLS,Q} = 19.1\ kN/m$

Precamber → $\Delta_G = 22.6\ mm$

$\Delta_{SLS,Q} = 38.9\ mm$

$w_{max} = 10900/250 = 43.6\ mm$

$\boldsymbol{UC8}: \ \frac{\mathbf{38.9}}{\mathbf{43.6}} = \mathbf{0.834 < 1}$

Check for LTB

The slabs provide lateral restrains to the beams, therefore , $\chi_{LTB} = 1$

$\boldsymbol{UC6}: \ \frac{\mathbf{622.5}}{\mathbf{1 * 820.5}} = \mathbf{0.76 < 1}$

C.2.8 Design of Slab – Cross Beam Connections

Cross Beam with Composite Action

Wind is the secondary Variable Load.

Table C.8: Design Checks on Slab – Cross Beam Connections (with Composite Action).

34 x 2 Grade 4.6 M14 Bolts per $L_B/2$

$d = 14\,mm, \quad s_{sc} = 156.2\,mm\ C/C$

$F_{Ed,X,Bolt,Composite\ Action} = \frac{875.2}{34} = 25.7\,kN$

All Bolts are not maximally loaded, hence $N_{sc,eff,x} = 34 * 2 = N_{sc,eff,y}$

$f_{h,k} = 24.68\,MPa, M_{y,Rk} = 0.114\,kNm$

Wind Perpendicular to Long Side	Wind Perpendicular to Short Side
$V_{X,Bolt} = \frac{0.9*9.8}{34*2} + 25.7 = 25.8\,kN,$ $V_{Y,Bolt} = \frac{0.9*48.2}{34*2} = 0.7\,kN, V_{R,Bolt} = 25.82\,kN, at\ \alpha = 0.77 \sim 0\,°, F_{Rd,Bolts} = 26.2\,kN$ $\boldsymbol{UC}: \frac{\mathbf{25.82}}{\mathbf{26.2}} = \mathbf{0.98 < 1}$	$V_{X,Bolt} = \frac{0.9*11.6}{34*2} + 25.7 = 25.85\,kN,$ $V_{Y,Bolt} = \frac{0.9*36.4}{34*2} = 0.52\,kN, V_{R,Bolt} = 25.85\,kN, at\ \alpha = 0.57 \sim 0\,°, F_{Rd,Bolts} = 26.2\,kN$ $\boldsymbol{UC}: \frac{\mathbf{25.85}}{\mathbf{26.2}} = \mathbf{0.98 < 1}$

Cross Beam without Composite Action

Wind is the leading Variable Load.

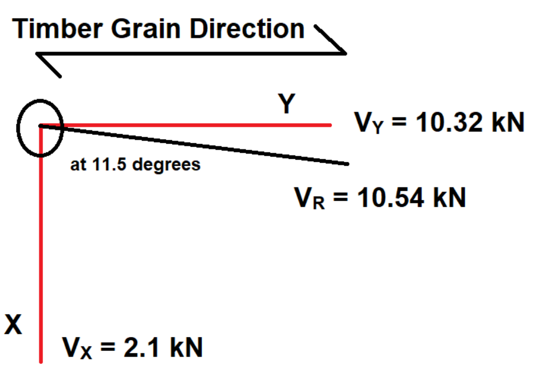
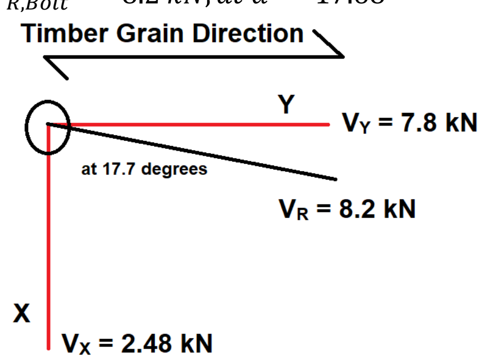
Table C.9: Design Checks on Slab – Cross Beam Connections (without Composite Action).

7 x 2 Grade 4.6 M14 Bolts per L_B

$d = 14\,mm, \quad s_{sc} = 1540\,mm\ C/C$

$N_{sc,eff,x} = 7, \quad N_{sc,eff,y} = 7$

$f_{h,k} = 24.68\,MPa, M_{y,Rk} = 0.114\,kNm$

Wind Perpendicular to Long Side	Wind Perpendicular to Short Side
$V_{X,Bolt} = \frac{1.5*9.8}{7} = 2.1\,kN, V_{Y,Bolt} = \frac{1.5*48.2}{7} = 10.32\,kN,\ V_{R,Bolt} = 10.54\,kN, at\ \alpha = 11.5\,°$  $f_{h,k,\alpha} = 24.34\,MPa, F_{Rd,Bolt,\alpha} = 11.27\,kN$ $\boldsymbol{UC}: \frac{\mathbf{10.54}}{\mathbf{11.27}} = \mathbf{0.934 < 1}$	$V_{X,Bolt} = \frac{1.5*11.6}{7} = 2.48\,kN, V_{Y,Bolt} = \frac{1.5*36.4}{7} = 7.8\,kN, V_{R,Bolt} = 8.2\,kN, at\ \alpha = 17.68\,°$  $f_{h,k,\alpha} = 23.9\,MPa,$ $F_{Rd,Bolt,\alpha} = 11.13\,kN$ $\boldsymbol{UC}: \frac{\mathbf{8.2}}{\mathbf{11.13}} = \mathbf{0.73 < 1}$

C.2.9 Design of Columns

Cross Beam with Composite Action

Actions:

$N_{Z,Ed,ULS}$ *per Storey*

$= 0.5 * q_{Cross\ Beam,ULS} * 10.9 + 1.2 * q_{G,Edge\ Beam} * 5.3 + 1.2 * q_{Column} * 3.2 = 228.5\ kN$

$N_{Z,Ed,ULS} = 4 * 228.5 = 914\ kN$

SHS 140x6.3

$H = 3025\ mm, l = 140\ mm, t = 6.3\ mm, r = 7.85\ mm$

$\lambda_{w/f} = \frac{140-2*(6.3+7.85)}{6.3*0.81} = 21.88 < 33$ → *Class 1 Section*

$I = 9.839 * 10^6\ mm^4, A = 3327\ mm^2, i_{z/y} = \sqrt{\frac{9.839*10^6}{3327}} = 54.4\ mm, \overline{\lambda_{z/y}} = 0.746$

$\Phi_{z/y} = 0.835, \chi_{z/y} = 0.824$

$N_{b,Rd} = \frac{0.835*3327*355}{1} = 974.2\ kN$

$\boldsymbol{UC}: \frac{\mathbf{914}}{\mathbf{974.2}} = \mathbf{0.94 < 1}$

Cross Beam without Composite Action

Actions:

$N_{Z,Ed,ULS}$ *per Storey*

$= 0.5 * q_{Cross\ Beam,ULS} * 10.9 + 1.2 * q_{G,Edge\ Beam} * 5.3 + 1.2 * q_{Column} * 3.2 = 229.6\ kN$

$N_{Z,Ed,ULS} = 4 * 229.6 = 918.3\ kN$

SHS 140x6.3

$\chi_{z/y} = 0.824$

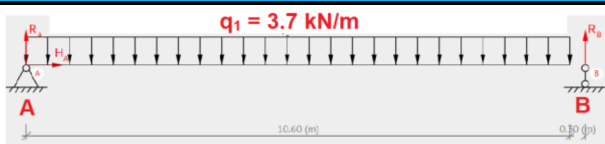
$N_{b,Rd} = \frac{0.835*3327*355}{1} = 974.2\ kN$

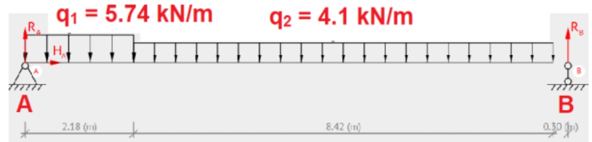
$\boldsymbol{UC}: \frac{\mathbf{918.3}}{\mathbf{974.2}} = \mathbf{0.942 < 1}$

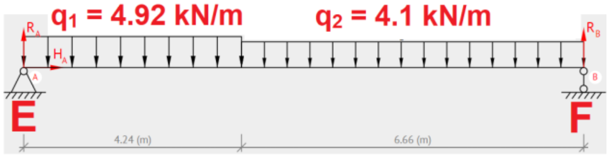
C.3 Calculations on DA2_HCS

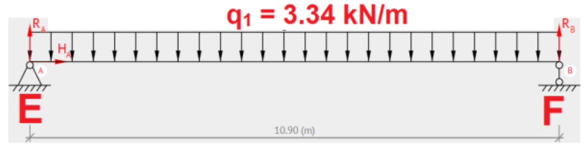
C.3.1 Action of Wind Loads

Table C.10: Action of Wind Loads.

<table>
<tr><th>Wind Perpendicular to Long Side</th><th>Wind Perpendicular to Short Side</th></tr>
<tr><td colspan="2">Wind Load on Long Side</td></tr>
<tr><td>

$V_{Ek,A/B} = 19.61\ kN,\ M_{Ek,max} = 52\ kNm$</td><td>

$V_{Ek,A} = 25.5\ kN, V_{Ek,B} = 21.5\ kN\ \ M_{Ek,max} = 62.8\ kNm$</td></tr>
<tr><td colspan="2">Wind Load on Short Side</td></tr>
<tr><td>

$V_{Ek,E} = 25.2\ kN, V_{Ed,F} = 23\ kN\ \ M_{Ek,max} = 64.63\ kNm$</td><td>

$V_{Ek,E/F} = 18.2\ kN,\ M_{Ek,max} = 49.6\ kNm$</td></tr>
<tr><td colspan="2">Axial Force in Edge Beams</td></tr>
<tr><td colspan="2">$b = 10.9\ m, d = 10.6\ m$
Deep Beam, $b < 3 * d = 15.9\ m$, → *YES*
Lever Arm, $z = 0.2 * b + 0.4 * d = 6.42\ m$</td></tr>
<tr><td>$M_{Ek,S} = M_{Ed,max} = 52\ kNm$
Force due to Moment, $= \frac{M_{Ek,S}}{z} = 8.1\ kN$
Total, $N_{Ek,cb,S} = 25.2 + 8.1 = 33.3\ kN$</td><td>$M_{Ek,S} = M_{Ed,max} = 62.8\ kNm$
Force due to Moment, $= \frac{M_{Ek,S}}{z} = 9.8\ kN$
Total, $N_{Ek,cb,S} = 18.2 + 9.8 = 28\ kN$</td></tr>
</table>

C.3.2 Design of Hollow Core Slab

For obtaining the cross section of *HCS*, span tables given in the product catalogue of Consolis VBI was used [98]. For a live load of $3.6\ kN/\ m^2$ (Office Category B including partitions), *HCS260* can sufficiently withstand spans up to 11 m. The assumed weight of additions to the floor is given to be a dead load $1.2\ kN/\ m^2$. It is assumed that constitutes the weight of the Floor Finish ($G_{FF} = 0.7\ kN/\ m^2$) and that of Ceilings and Services ($G_{C/S} = 0.5\ kN/\ m^2$) .Hence, this is used without doing any specific calculations. The technical data of *HCS260* is given below in Table C.11.

Table C.11: Technical Specification of HCS260.

Weight	$3.76\ kN/\ m^2$	***Fire Resistance***	*90 minutes*
Environment Class	*XC1, XC3*	***Concrete Strength Class***	*C45/55*
Width	$1200\ mm$	***Neutral Axis***	$122.9\ mm$
Cross Section Area	$177829\ mm^2$	***Second Moment of Area***	$1434 * 10^6 mm^4$

C.3.3 Design of Edge Beam

Actions:

Imposed loads, $q_{ULS} = 0.5 * 10.9 * (1.2 * 4.96 + 1.5 * 3.6) + 1.2 * 2.33 = 64.7\ kN/m$

$V_{y,Ed,ULS} = 0.5 * 10.6 * 64.7 = 343\ kN,\ M_{y,Ed,ULS} = 0.125 * 10.6^2 * 64.7 = 908.2\ kNm,$

$q_{SLS} = 0.5 * 10.6 * (4.96 + 3.6) + 2.13 = 49\ kN/m$

Axial force due to wind load (secondary variable load), $N_{Y,Ed,ULS} = 0.9 * 33.3 = 30\ kN$

Table C.12: Design Checks on Edge Beam.

IFB287 (1/2 x HEM500 + Bottom Plate 500x25)

$h = 287\ mm, b_{f,t} = 300\ mm,\ t_{f,t} = 40\ mm, t_w = 21\ mm, r = 27\ mm,$

$b_{f,b} = 500\ mm, t_{f,t} = 25\ mm, q_g = 2.33\ kN/m$

$\epsilon = 0.81,\ \lambda_{f,t} = \frac{0.5*(300-2*27-21)}{40*0.81} = 1.74 < 9, \lambda_w = \frac{(222-27)}{21*0.81} = 11.4 < 72$

→ *Cross Section Class 1 under Sagging Moments.*

$L = 10600\ mm,\ A = 29162\ mm^2, S_{xz} = 1.68 * 10^6\ mm^3,$

$z_t = 150\ mm, z_b = 137\ mm, I_x = 425 * 10^6\ mm^4, W_{el,x} = 2.83 * 10^6\ mm^3$

Check for Shear	**Check for Bending**
$\tau_{Ed} = \frac{343*1000*1.68}{425*21} = 64.8\ Nmm^{-2}$ $\boldsymbol{UC1}: \frac{\mathbf{64.8}}{\mathbf{355/\sqrt{3}}} = \mathbf{0.32 < 1}$	$M_{el,Rd} = 2.83 * 355 = 1005.6\ kNm$ $\boldsymbol{UC2}: \frac{\mathbf{908.2}}{\mathbf{1005.6}} = \mathbf{0.903 < 1}$

Check for Flexural Buckling

$N_{pl,Rd} = 29162 * 355 = 10352\ kN,\ L_{cr} = 10600\ mm$

Major Axis (xx)	Minor Axis (zz)
$I_x = 425 * 10^6\ mm^4,$ $i_x = 120.7\ mm,\ \overline{\lambda_y} = 1.17,$ $\Phi_x = 1.29, \chi_x = 0.54$ $N_{b,pl,Rd} = 0.54 * 10352 = 5631\ kN$	$I_z = 350.6 * 10^6\ mm^4,$ $i_z = 109.6\ mm,\ \overline{\lambda_z} = 1.29,$ $\Phi_z = 1.45, \chi_z = 0.47$ $N_{b,pl,Rd} = 0.47 * 10352 = 4885\ kN$

$\boldsymbol{UC3}: \frac{\mathbf{30}}{\mathbf{5631}} = \mathbf{0.005 < 1}$	$\boldsymbol{UC3}: \frac{\mathbf{30}}{\mathbf{4885}} = \mathbf{0.006 < 1}$
Check for Bending, Shear and Axial Force $\sigma_{yy,Ed} = \frac{30*1000}{29162} = 1.03\ Nmm^{-2}$ → Axial force $\sigma_{yy,tf,Ed} = \frac{908.2}{2.83} = 321\ Nmm^{-2}$ → Bending $\tau_{xz,w,Ed} = 64.8\ Nmm^{-2}$ → Shear At Supports, $\sigma_{von\ mises,Ed} = 112\ Nmm^{-2}$ $\boldsymbol{UC4}: \frac{\mathbf{112}}{\mathbf{355}} = \mathbf{0.32 < 1}$ At Mid-span, $\sigma_{von\ mises,Ed} = 322\ Nmm^{-2}$ $\boldsymbol{UC4}: \frac{\mathbf{322}}{\mathbf{355}} = \mathbf{0.91 < 1}$	**Check for Beam – Column Buckling** $C_{my} = 0.95, k_{yy} = 0.952, k_{zy} = 0$ $\chi_x = 0.54\ ,\ \chi_{LTB} = 1$ $\boldsymbol{UC7}: \frac{\mathbf{30}}{\mathbf{0.47 * 10352}} + \mathbf{0.952} * \frac{\mathbf{908.2}}{\mathbf{1 * 1005.6}} = \mathbf{0.909 < 1}$
Check for LTB The slabs provide lateral restrains to the beams, therefore , $\chi_{LTB} = 1$ $\boldsymbol{UC6}: \frac{\mathbf{908.2}}{\mathbf{1 * 1005.6}} = \mathbf{0.903 < 1}$	**Check for Deflections** $q_{SLS,G} = 29.4\ kN/m,\ q_{SLS,Q} = 19.6\ kN/m$ Precamber → $\Delta_G = 56.8\ mm$ $\Delta_{SLS,Q} = 37.9\ mm$ $w_{max} = 10600/250 = 42.4\ mm$ $\boldsymbol{UC8}: \frac{\mathbf{37.9}}{\mathbf{42.4}} = \mathbf{0.89 < 1}$

Check for Bottom Plate (SLS)

$e = 0.5 * 300 + 0.25 * (500 - 300) = 200mm,\ q_{bp} = 45.34\ kN/m$

$m_{x,bp} = 45.34 * 0.2 = 9.068\ kNm/m\ ,\ \sigma_{xx,bp} = \frac{6*9.068*1000}{25^2} = 92.2\ MPa,$

$v_{yz,bp} = 45.34\ kN/m\ , \tau_{el,yz,bp} = \frac{45.34*12}{25*4} = 5.44\ MPa,$

$q_{SLS} = 47.7\ kN/m, M_{y,bp} = \frac{47.7*10.6^2}{8} = 670\ kNm/m,\ \sigma_{yy,bp} = \frac{670*137}{425} = 216\ MPa$

Check for Von mises strength at bottom of bottom plate, $\sigma_{R1} = \sqrt{3 * 0^2 + (92.2 + 216)^2} = 308.2\ MPa$

Check for Von mises strength at centre of bottom plate, $\sigma_{R2} = \sqrt{3 * 5.44^2 + (0 + 216)^2} = 216.2\ MPa$

$$\boldsymbol{UC9}: \frac{\boldsymbol{max}(\mathbf{308.2, 216.2})}{\mathbf{355}} = \mathbf{0.87 < 1}$$

C.3.4 Design of Columns

Actions:

$N_{Z,Ed,ULS}\ per\ Storey$

$= q_{Edge\ Beam,ULS} * 10.6 + 1.2 * q_{Column} * 3.1 = 688.1\ kN$

$N_{Z,Ed,ULS} = 4 * 688.1 = 2752.3\ kN$

SHS 180x14.2

$H = 3025\ mm, l = 180\ mm, t = 14.2\ mm, r = 17.8\ mm$

$\lambda_{w/f} = \frac{180-2*(14.2+17.8)}{14.2*0.81} = 10.1 < 33$ → *Class 1 Section*

$I = 41.5.28 * 10^6\ mm^4, A = 9201\ mm^2, i_{z/y} = \sqrt{\frac{41.5*10^6}{9201}} = 67.2\ mm, \overline{\lambda_{z/y}} = 0.604$

$\Phi_{z/y} = 0.724, \chi_{z/y} = 0.89$

$N_{b,Rd} = \frac{0.89*9201*355}{1} = 2907\ kN$

$$\textbf{UC: } \frac{\mathbf{2752}}{\mathbf{2907}} = \mathbf{0.95 < 1}$$

C.4 Calculations on DA3_CS

C.4.1 Action of Wind Loads

The action of wind loads on the longer side is negligible due to the presence of cross beams 3.53 m spacing. Hence, only the action of wind loads on the shorter side is considered, which is the same as given in Table C.10.

C.4.2 Design of Composite Slabs

Similar to the approach for *HCS*, the span tables of ComFlor by Tata Steel [99] were referred. For unpropped construction with double span units, ComFlor60 is sufficient to carry office category live loads ($3.6\ kN/\ m^2$) with fire safe design of 90 minutes, for a span of $3.53\ m$.. The technical data of *CS190* is given below in Table C.13.

Table C.13: Technical Specification of CS190 (per meter width).

Thickness of Plate	$1\ mm$	***Cross Section Area***	$1424\ mm^2$
Profile Weight	$0.11\ kN/m^2$	***Second Moment of Area***	$1.06 * 10^6 mm^4$
Steel Strength (f_y)	*350 MPa*	***Concrete Used***	*C30/37*
Concrete MOE (E_{cm})	*31 GPa*	***Design Strength (f_{cd})***	$20\ MPa$
Concrete Depth	$130\ mm$	***Concrete Weight***	$2.36\ kN/m^2$

The dead loads are calculated as follows:

Slab Dead loads, $G_S = 2.47\ kN/m^2, G_{C/S} = 0.5\ kN/m^2, G_{FF} = 0.5\ kN/m^2$

→Total, $G = 3.47\ kN/m^2$

C.4.3 Design of Cross Beam

<u>Without Composite Action</u>

(For Reference Only)

Actions:

Imposed loads, $q_{ULS} = 3.53 * (1.2 * 3.47 + 1.5 * 3.6) + 1.2 * 1 = 35\ kN/m$

$V_{x,Ed,ULS} = 0.5 * 10.9 * 35 = 191\ kN,\ M_{x,Ed,ULS} = 0.125 * 10.9^2 * 35 = 520\ kNm,$

$q_{SLS} = 3.533 * (3.47 + 3.6) + 1 = 26\ kN/m$

Table C.14: Design Checks on Cross Beams (without Composite Action).

HEA340 $h = 330\,mm, b_f = 300\,mm, t_f = 16.5\,mm, t_w = 9.5\,mm, r = 27\,mm, q_g = 1\,kN/m$ $\epsilon = 0.81, \lambda_f = \frac{0.5*(300-2*27-9.5)}{16.5*0.81} = 8.84 < 9, \lambda_w = \frac{330-2*(27+16.5)}{9.5*0.81} = 31.57 < 72$ → *Cross Section Class 1*, $L = 10900\,mm, A_S = 13347\,mm^2, S_{yz} = 1.72*10^6\,mm^3$ $I_x = 276.9*10^6\,mm^4, W_{el,x} = 1.68*10^6\,mm^3$	
Check for Deflections $q_{SLS,G} = 13.3\,kN/m, q_{SLS,Q} = 12.7\,kN/m$ Precamber → $\Delta_G = 44.06\,mm$ $EI_S = 5.54*10^{13}Nmm^2$ $\Delta_{SLS,Q} = 42.2\,mm$ $w_{max} = 10900/250 = 43.6\,mm$ $\boldsymbol{UC1}$: $\boldsymbol{\frac{42.2}{43.6} = 0.97 < 1}$	**Check for Bending** $M_{el,Rd} = \frac{355*1.68*10^6}{1} = 595.8\,kNm$ $\boldsymbol{UC2}$: $\boldsymbol{\frac{520}{595.8} = 0.873 < 1}$
Check for Shear $\tau_{Ed} = \frac{190.8*1000*1.72}{276.9*9.5} = 124.8\,Nmm^{-2}$ $\boldsymbol{UC3}$: $\boldsymbol{\frac{124.8}{355/\sqrt{3}} = 0.61 < 1}$	**Check for LTB** The slabs provide lateral restrains to the beams, therefore , $\chi_{LTB} = 1$ $\boldsymbol{UC4}$: $\boldsymbol{\frac{520}{1*595.8} = 0.873 < 1}$

<u>With Composite Action</u>

Actions:

Imposed loads, $q_{ULS} = 3.53*(1.2*3.47 + 1.5*3.6) + 1.2*0.86 = 34.8\,kN/m$

$V_{x,Ed,ULS} = 0.5*10.9*34.8 = 189.8\,kN, M_{x,Ed,ULS} = 0.125*10.9^2*34.8 = 517\,kNm,$

$q_{SLS} = 3.533*(3.47 + 3.6) + 0.86 = 25.8\,kN/m$

Table C.15: Design Checks on Cross Beams (With Composite Action).

HEA300

$h = 290\,mm, b_f = 300\,mm, t_f = 14\,mm, t_w = 8.5\,mm, r = 27\,mm, q_g = 0.86\,kN/m$

$\epsilon = 0.81, \lambda_f = \frac{0.5*(300-2*27-8.5)}{14*0.81} = 9.89 < 10, \lambda_w = \frac{290-2*(27+14)}{8.5*0.81} = 30.2 < 72$

→ *Cross Section Class 2*, $L = 10900\,mm, A_S = 11253\,mm^2,$

$I_{S,x} = 182.6*10^6\,mm^4, W_{S,el,x} = 1.26*10^6\,mm^3$

Concrete C30/37

$E_{C,eff} = 31/2 = 16.5\,GPa, n = \frac{16.5}{200} = 0.0825, h_p = 60\,mm, h_C = 70\,mm$

$b_{C,eff} = n*min(0.25*10900,3533) = 224.8\,mm, A_{C,eff} = 15736\,mm^2$

$I_{C,eff} = 6.42*10^6\,mm^4$

Composite Beam

$x_{el} = 135\,mm, z_{S,b} = 285\,mm$

$EI_{Comp} = 11.33*10^{13}\,Nmm^2, S_{yz,S,w} = 1.56*10^6\,mm^3$

<table>
<tr>
<td>Check for Deflections
$q_{SLS,G} = 13.1\ kN/m,\ q_{SLS,Q} = 12.7\ kN/m$
Precamber → $\Delta_G = 66\ mm$
$\Delta_{SLS,Q} = 20.62\ mm$
$w_{max} = 10900/250 = 43.6\ mm$
$\boldsymbol{UC1}: \frac{\mathbf{20.62}}{\mathbf{43.6}} = \mathbf{0.473 < 1}$</td>
<td rowspan="3">Check for Bending
$M_{G,Ed} = 195\ kNm$
→ $\sigma_{Ed,S} = 154.6\ Nmm^{-2}$
$q_{Comp} = q_{ULS} - q_{SLS,G} = 21.7\ kN/m$
→ $M_{Comp,Ed} = 322.3\ kN/m$
$q_{Comp,elastic\ limit} = q_{SLS,G} + \frac{(f_y-\sigma_{Ed,S})*EI_{Comp}}{z_{S,b}*E_S*0.125*L_B{}^2} = 39.95\ kN/m > q_{Comp}$
(From yielding of steel bottom fibre)
→ $\sigma_{Ed,Comp} = \sigma_{Ed,S} + \frac{M_{Comp,Ed} * z_{S,b} * E_S}{EI_{Comp}}$
$= 316.6 < 355\ Nmm^{-2}$
$M_{Comp,el,Rd} = 39.95 * 0.125 * 10.9^2$
$= 593.7\ kNm$
$\boldsymbol{UC2}: \frac{\mathbf{517}}{\mathbf{593.7}} = \mathbf{0.871 < 1}$</td>
</tr>
<tr>
<td>Check for Shear
$\tau_{Ed} = \frac{V_{x,Ed}*S_{yz,S,w}}{EI_{Comp}*t_{S,w}} = 61.7\ Nmm^{-2}$
$\boldsymbol{UC3}: \frac{\mathbf{61.7}}{\mathbf{355/\sqrt{3}}} = \mathbf{0.59 < 1}$</td>
</tr>
<tr>
<td>Check for LTB
The slabs provide lateral restrains to the beams, therefore , $\chi_{LTB} = 1$
$\boldsymbol{UC4}: \frac{\mathbf{517}}{\mathbf{1 * 593.7}} = \mathbf{0.871 < 1}$</td>
</tr>
</table>

C.4.4 Design of Edge Beams

Actions:

Imposed loads (equivalent), $Q_{ULS} = 0.5 * 10.9 * 34.8 = 191\ kN$ at 1/3rd span and 2/3rd span points.

$V_{x,Ed,ULS} = 190\ kN,\ M_{x,Ed,ULS} = \frac{190*10.6}{3} = 671\ kNm,\ Q_{SLS} = 0.5 * 10.9 * 25.8 = 141\ kN$

Axial force due to wind load (secondary variable load), $N_{X,Ed,ULS} = 0.9 * 25.2 = 22.7\ kN$ (Same as in Appendix C.3.1)

Table C.16: Design Checks on Cross Beams (with Composite Action).

<table>
<tr>
<td colspan="2">HEA400
$h = 390\ mm, b = 300\ mm,\ t_f = 19\ mm, t_w = 11\ mm, r = 27\ mm, q_g = 1.2\ kN/m$
$\epsilon = 0.81,\ \lambda_f = \frac{0.5*(300-2*27-11)}{19*0.81} = 7.63 < 9, \lambda_w = \frac{390-2*(27+19)}{11*0.81} = 33.4 < 72$ → Class 1
$L = 10600\ mm,\ A = 11552\ mm^2, S_{xz} = 1.39 * 10^6\ mm^3,$
$I_y = 450.7 * 10^6\ mm^4, W_{el,y} = 2.31 * 10^6\ mm^3$</td>
</tr>
<tr>
<td>Check for Shear
$\tau_{Ed} = \frac{V_{x,Ed}*S_{xz,S,w}}{EI_{Comp}*t_{S,w}} = 53.6\ Nmm^{-2}$
$\boldsymbol{UC3}: \frac{\mathbf{53.6}}{\mathbf{355/\sqrt{3}}} = \mathbf{0.26 < 1}$</td>
<td>Check for Bending
$M_{el,Rd} = 820.5\ kNm$
$\boldsymbol{UC2}: \frac{\mathbf{671}}{\mathbf{820.5}} = \mathbf{0.82 < 1}$</td>
</tr>
<tr>
<td>Check for Flexural Buckling
Lateral constraints at 3.53 m by the HEA300.
Hence no check for buckling.
Check for LTB
Lateral constraints at 3.53 m by the HEA300. Hence no check for buckling.</td>
<td>Check for Deflections
$Q_{SLS,G} = 71.4\ kN,\ Q_{SLS,Q} = 69.6\ kN$
Precamber → $\Delta_G = \frac{23*Q_{SLS,G}*L^3}{648*EI}\ 33.5\ mm$
$\Delta_{SLS,Q} = 32.6\ mm$
$w_{max} = 10600/250 = 42.4\ mm$
$\boldsymbol{UC4}: \frac{\mathbf{32.6}}{\mathbf{42.4}} = \mathbf{0.77 < 1}$</td>
</tr>
<tr>
<td colspan="2">Check for Bending and Shear</td>
</tr>
</table>

$\tau_{Ed,top\ of\ web} = 40.5\ Nmm^{-2}, \sigma_{Ed,top\ of\ web} = 262\ Nmm^{-2}, \sigma_{Von\ Mises,top\ of\ web} = 271\ Nmm^{-2}$

$$\boldsymbol{UC5}:\ \frac{\mathbf{272}}{\mathbf{355}} = \mathbf{0.763 < 1}$$

C.4.5 Design of Columns

Actions:

$N_{Z,Ed,ULS}\ per\ Storey$

$= Q_{Edge\ Beam,ULS} * 3 + 1.2 * q_{Column} * 3.1 = 586.3\ kN$

$N_{Z,Ed,ULS} = 4 * 586.3 = 2345.3\ kN$

SHS 160x14.2

$H = 3025\ mm, l = 160\ mm, t = 14.2\ mm, r = 17.8\ mm$

$\lambda_{w/f} = \frac{160-2*(14.2+17.8)}{14.2*0.81} = 8.34 < 33$ → *Class 1 Section*

$I = 28.1 * 10^6\ mm^4, A = 8065\ mm^2, i_{z/y} = \sqrt{\frac{28.1*10^6}{8065}} = 59\ mm, \overline{\lambda_{z/y}} = 0.687$

$\Phi_{z/y} = 0.788, \chi_{z/y} = 0.853$

$N_{b,Rd} = 0.853 * 8065 * 355 = 2443\ kN$

$$\boldsymbol{UC}:\ \frac{\mathbf{2345.3}}{\mathbf{2443}} = \mathbf{0.96 < 1}$$

D. Structural Analysis of STC Beams

D.1 Verification of Timber Sections

Table D.1: Mechanical Properties of C24 Spruce (adopted from Table A.1).

$$k_h = min\left\{\frac{150}{t_{max}}, \quad 1.3\right\}$$

$$t_{max} = max\left\{t_w, \quad t_f\right\}$$

$$k_{def} = 0.6, (Service\ Class\ 1)$$

$$k_{mod} = 0.8$$

$$\gamma_M = 1.3\ for\ Sawn\ Timber$$

$$k_{sr} = {k_{sr,1}}^{r-1}$$

$$f_d = \frac{k_h * k_{mod} * k_{sr} * f_k}{\gamma_M}$$

Parameter	Value [MPa]
$E_{0,mean}$	*11000*
$E_{90,mean}$	*370*
$E_{0.05}$	*7400*
G_{mean}	*690*
$f_{m,k}$	*24*
$f_{v,0,k}$	*4*
$f_{c,0,k}$	*21*
$f_{v,90,k}$	*1*

Dead Loads:

Floor Finish, $G_{FF} = 0.5\ kNm^{-2}$

Fire Protection and Sound Insulation, $G_{FS} = 0.5\ kNm^{-2}$

Installations, $G_I = 0.5\ kNm^{-2}$

Timber Dead Load, $G = 0.5 + 0.5 + 0.5 + G_T$,

where $G_T\ is\ the\ weight\ of\ timber\ slab$

Live Loads:

Office Category B Live Loads, $Q = 3.5\ kNm^{-2}$. ($Q = 3.6\ kNm^{-2}$ for Reorientation)

Load Combinations:

$q_{ULS} = 1.2 * G + 1.5 * Q$

$q_{SLS} = 1 * G + 1 * Q$

D.1.1 Effective Width of Flange

The effective flange width ($b_{T,f,eff}$) according to Möhler et. Al [33]:

$$\frac{b_{T,f,eff}}{b_{T,f}} = \frac{(\lambda_1 * tanh\ \alpha_1 + \lambda_2 * tanh\ \alpha_2) * 2 * L_{Slab}}{\pi * ({\lambda_1}^2 - {\lambda_2}^2) * b_{T,f}} \quad \textbf{(Eq 8)}$$

Where,

$$\alpha_i = \frac{\lambda_i * \pi * b_{T,f}}{2 * L_{Slab}}$$

$$\lambda_i = \sqrt{a +- \sqrt{a^2 - c}}$$

$$a = \frac{E_x}{2{*}G_T} - \mu_{xy} \qquad \text{and} \qquad c = \frac{E_x}{E_y}$$

For C24 timber, according to the conventions in this thesis,

$$E_y = E_{0,mean} = 11000\ MPa$$

$$E_x = E_{90,mean} = 370\ MPa$$

$$G_T = G_{mean} = 690\ MPa$$

The Poisson's ratio (μ_{xy}) for C24 spruce has been taken from D.W. Greene er al [48] as 0.43, obtained as the average value for longitudinal-tangential plane (μ_{LT}) and longitudinal-radial plane (μ_{LR}) for 2 species of spruce.

Thus,

$$a = \frac{370}{2 * 690} - 0.43 = -0.161\,, \qquad c = \frac{370}{1100} = 0.0336$$

This would give complex values of λ_i

Maximum permissible Effective Flange Width,

$$b_{T,f,eff} = minimum\ of\ \{\ 0.1 * L_{Slab}\ (from\ Shear\ Deformation)$$
$$and\ \ 25 * t_{T,f}\ (from\ Buckling\ of\ Compression\ Flange)\ \}$$

For all cross sections of LFE considered in this thesis,

Maximum spacing of webs, $b_{T,f,max} = 250\ mm\ c.t.c.$

Minimum thickness of flange, $t_{T,f} = 31\ mm$

Minimum span considered, $L_{Slab,min} = 4100\ mm$

This gives us $b_{T,f,eff} = min\ (0.1 * 4100, 25 * 31) = 410 > b_{T,f,max}$

Hence for all cases considered, $b_{T,f,eff} = b_{T,f}$

For the longitudinal timber sections near the supports, the timber sections are used for composite action. The width of the top flange that is effective in composite action is determined using the procedure mentioned above. Apart from shear lag between the individual components of the timber section, and buckling of the flange in compression, *EC4* [64] for steel-concrete composite structures suggests a limit, based on the shear lag between the steel beam and the slab (timber as in our case).

$$b_{T,f,eff} = 0.25 * L_{Slab}$$

D.1.2 Stresses in the Flange

Maximum Bending Stresses in the Flanges,

$$\sigma_{T,f,max} = \frac{m_{y,Ed,ULS} * z_{T,max}}{I_T} \qquad \textbf{(Eq 9)}$$

Where,

$$m_{y,Ed,ULS} \text{ is the bending moment per unit length along the span of } LFE$$

$$z_{T,max} \text{ is the distance of the NA of } LFE \text{ to the outermost fibre}$$

$$I_T \text{ is the Second Moment of Area of } LFE \text{ per unit length}$$

Design Check for bending in flanges,

$$DC1: \ \frac{\sigma_{T,f,max}}{f_{m,d}} < 1 \qquad \textbf{(Eq 10)}$$

Axial stress in c.g. of compression flange (due to bending),

$$\sigma_{T,c} = \frac{m_{y,d,ULS} * z_{T,f}}{I_T} \qquad \textbf{(Eq 11)}$$

Where,

$$z_{T,f} \text{ is the distance of the NA of } LFE \text{ to the c.g. of the flange in compression}$$

Design Check for buckling of flange in compression,

$$DC2: \ \frac{k_{c,z} * \sigma_{T,c}}{f_{c,0,d}} < 1 \qquad \textbf{(Eq 12)}$$

Where,

$$k_{c,z} \text{ is the Coefficient of Buckling in the xy plane of compression flange of } LFE$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{{k_z}^2 + {\lambda_{rel,z}}^2}}$$

$$k_z = 0.5 * (1 + 0.2 * (\lambda_{rel,z} - 0.3) + {\lambda_{rel,z}}^2), \qquad \text{for sawn timber}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} * \sqrt{\frac{f_{c,0,k}}{E_{0,0.05}}}, \qquad \lambda_z = \frac{l_{crit}}{i_z}$$

l_{crit} *is the distance between lateral supports of the compression flange,* $= s_{T,ts}$

i_z *is the radius of gyration of the compression flange about the xy plane*

D.1.3 Shear Stresses in the Web

Maximum Shear Stress in Webs,

$$\tau_{T,w,max} = \frac{v_{y,d,ULS} * S_{T,yz,max}}{I_T * b_{T,w}} \qquad \textbf{(Eq 13)}$$

Where,

$v_{y,Ed,ULS}$ *is the maximum shear force per unit length along the span of LFE*

$S_{T,yz,max}$ *is the maximum first moment of area of LFE section*

Design Check for shear stress in web,

$$DC3: \ \frac{\tau_{T,w,max}}{f_{v,0,d}} < 1 \qquad \textbf{(Eq 14)}$$

Slenderness of Web,

$$\lambda_{T,w} = \frac{h_{T,w}}{t_{T,w}} \qquad \textbf{(Eq 15)}$$

Where,

$$h_{T,w} = H_T - 2 * t_{T,f} \text{ , is the height of the web}$$

For $\lambda_{T,w} < 70$

Design shear force on each web,

$$F_{T,v,Ed} = \frac{v_{y,d}}{5} \qquad \textbf{(Eq 16)}$$

Design resistance to shear for each web,

$$F_{T,v,Rd} = h_{T,w} * t_{T,w} * f_{v,0,d} \ for \ \lambda_{T,w} < 35$$

$$F_{T,v,Rd} = 35 * {t_{T,w}}^2 * f_{v,0,d} \ for \ 35 <= \lambda_{T,w} < 70 \qquad \textbf{(Eq 17)}$$

Design Check for shear buckling of web,

$$DC4: \ \frac{F_{T,v,Ed}}{F_{T,v,Rd}} < 1 \qquad \textbf{(Eq 18)}$$

D.1.4 Shear Stress in Glue Line between Flange and Web

Shear Stress in Glue Line,

$$\tau_{T,Glue\ Line} = \frac{v_{y,Ed,ULS} * S_{T,yz,Glue\ Line}}{I_T * b_{T,w}} \quad \textbf{(Eq 19)}$$

Longitudinal shear flow per unit length,

$$\mathrm{v}_{T,Ls} = \tau_{T,Glue\ Line} * b_{T,w} \quad \textbf{(Eq 20)}$$

Total longitudinal shear flow in the glue line,

$$\mathrm{V}_{T,Ls,Ed} = \int_0^{\frac{L_{Slab}}{2}} \tau_{T,Glue\ Line} * b_{T,w} * dx = \frac{q_{T,ULS,Ed} * S_{T,yz,Glue\ Line} * {L_{Slab}}^2}{I_T * 4} \quad \textbf{(Eq 21)}$$

Design strength of Glue Line,

$$\mathrm{V}_{T,Ls,Rd} = \frac{\mathrm{L}_{Slab} * b_{T,w} * f_{v,90,d}}{2} \quad \textbf{(Eq 22)}$$

Design Check for shear stress in Glue Line,

$$DC5: \frac{V_{T,Ls,Ed}}{V_{T,Ls,Rd}} < 1 \quad \textbf{(Eq 23)}$$

D.1.5 Deflection and Vibrations

Initial Deflection limit for timber,

$$w_{T,initial,max} = \frac{L_{Slab}}{300} \quad \textbf{(Eq 24)}$$

Initial Deflection of LFE,

$$w_{T,initial} = w_{T,G} + w_{T,Q} \quad \textbf{(Eq 25)}$$

Where,

$$w_{T,G} = \frac{5 * q_G * {L_{Slab}}^4}{384 * EI_{T,initial}}, deflection\ due\ to\ dead\ load$$

$$w_{T,Q} = \frac{5 * q_Q * {L_{Slab}}^4}{384 * EI_{T,initial}}, deflection\ due\ to\ Imposed\ load$$

$$EI_{T,initial} = E_x * I_T\ , \qquad is\ the\ Initial\ Bending\ Stiffness$$

Design Check for Initial Deflection,

$$DC6: \frac{w_{T,initial}}{w_{T,initial,max}} < 1 \quad \textbf{(Eq 26)}$$

Final Deflection limit for timber,

$$w_{T,final,max} = \frac{L_{Slab}}{250} \quad \textbf{(Eq 27)}$$

Final Deflection of LFE,

$$w_{T,final} = w_{T,G,final} + w_{T,Q,final} \quad \textbf{(Eq 28)}$$

Where,

$$w_{T,G,final} = w_{T,G} * (1 + k_{def})$$

$$w_{T,Q,final} = w_{T,Q} * (1 + \psi_{Q,1} * k_{def}), \qquad \psi_{Q,1} = 0.5$$

Design Check for Initial Deflection,

$$DC7\text{:} \quad \frac{w_{T,final}}{w_{T,final,max}} < 1 \quad \textbf{(Eq 29)}$$

First fundamental frequency of LFE,

$$f_1 = \frac{\pi}{2 * L_{Slab}{}^2} * \sqrt{\frac{(EI_{T,final})_l}{M}} \quad \textbf{(Eq 30)}$$

Where,

$(EI_{T,final})_l$ *is the equivalent bending stiffness per unit length of the longitudinal section of timber floor*

$$EI_{T,final} = \frac{E_x * I_T}{1 + k_{def}} \;, \qquad \textit{is the reduced Bending Stiffness due to Creep}$$

M *is the mass per unit area of the floor*

Frequency Criteria,

$$DC8\text{:} \quad f_1 > 8\,Hz \quad \textbf{(Eq 31)}$$

Stiffness of the floor under the action of a Point Load,

$$a = \frac{L_{Slab}{}^3}{48 * EI_{T,final}} \quad \textbf{(Eq 32)}$$

Stiffness Criteria

$$DC8\text{:} \quad a < \mathrm{a}_{limit} = 1\,mm/kN \quad \textbf{(Eq 33)}$$

Unit Impulse Velocity response,

$$v = \frac{4 * (0.4 + 0.6 * n_{40})}{M * L_{Slab} * L_{Beam} + 200} \quad \textbf{(Eq 34)}$$

Where,

$$n_{40} = \left[\left\{\left(\frac{40}{f_1}\right)^2 - 1\right\} * \left\{\frac{L_{Beam}{}^4 * (EI_{T,final})_l}{L_{Slab}{}^4 * (EI_{T,final})_b}\right\}\right]^{0.25}$$

$(EI_{T,final})_b$ *is the equivalent bending stiffness per unit length of the floor about its transverse axis*

Unit Impulse Velocity response Criterion,

$$DC9: \ v < v_{limit} = 120^{f_1 * \zeta - 1} \quad \textbf{(Eq 35)}$$

The unity checks for each design criteria are given below in Table D.2.

Table D.2: Unity Checks for all design criteria.

Design for Reuse by Reorientation											
Section	L_{Slab} [m]	$t_{T,w}$ [mm]	$t_{T,f}$ [mm]	**DC1**	**DC2**	**DC3**	**DC4**	**DC5**	**DC6**	**DC8**	**DC9**
LFE120	4	42	31	0.37	0.31	0.26	**0.98**	0.93	0.96	0.92	0.55
LFE140	4.6	39	31	0.38	0.34	0.26	**0.99**	0.94	0.97	0.93	0.47
LFE160	5.1	37	31	0.38	0.35	0.26	**0.97**	0.91	0.94	0.9	0.41
LFE180	5.7	35	31	0.4	0.38	0.26	**0.99**	0.95	0.98	0.94	0.37
LFE200	6.2	34	31	0.4	0.39	0.26	**0.96**	0.94	0.97	0.93	0.33
LFE220	6.7	32	31	0.41	0.41	0.27	**0.98**	0.95	0.98	0.93	0.3
LFE240	7.2	31	31	0.42	0.43	0.27	**0.98**	0.95	**0.98**	0.93	0.27
LFE260	7.7	31	31	0.43	0.45	0.26	0.94	0.96	**0.99**	0.94	0.25
LFE280	8.1	31	31	0.43	0.45	0.26	0.9	0.93	**0.97**	0.91	0.23
LFE300	8.6	31	31	0.44	0.47	0.25	0.88	0.95	**0.98**	0.92	0.21
LFE320	9.1	31	31	0.45	0.49	0.25	0.86	0.96	**0.99**	0.93	0.2
LFE340	9.5	31	31	0.45	0.5	0.24	0.83	0.95	**0.98**	0.92	0.19
LFE360	9.9	31	31	0.45	0.51	0.24	0.8	0.94	**0.97**	0.9	0.17

D.2 Composite Action in Steel – Timber

For reference in this section,

$$Subscripts\ 'T'\ and\ 'S'\ refer\ to\ the\ Timber\ and\ Steel\ respectively$$

$$Subscripts\ 't'\ and\ 'b'\ respectively\ refer\ to\ the\ top\ and\ bottom\ of\ the\ sections$$

$$E\ is\ the\ Modulus\ of\ Elasticity$$

$$I\ is\ the\ Second\ Moment\ of\ Inertia$$

$$A\ is\ the\ cross\ sectional\ Area$$

$$z_{S/T}\ is\ the\ \ distance\ of\ NA\ of\ Steel/Timber\ to\ the\ NA\ of\ STC\ with\ full\ Composite\ Action$$

D.2.1 The Gamma Method

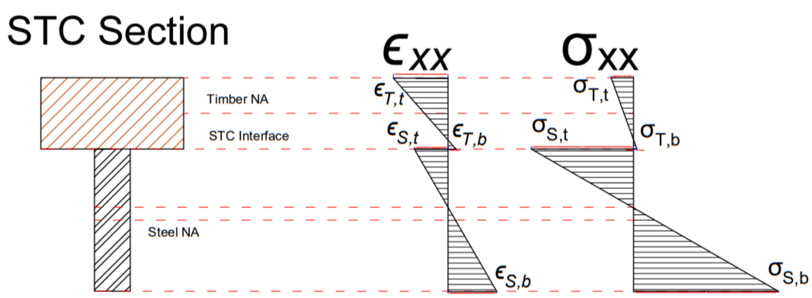


Figure D.1: Normal Strains and Stresses for Elastic Analysis of STC.

The stiffness of steel-timber shear connectors (with dowels) in N/mm,

$$k_{sc} = \frac{2 * \rho_{mean}{}^{1.5} * d}{23} \qquad \textbf{(Eq 36)}$$

Where,

$$d\ is\ the\ diameter\ of\ the\ metal\ fastener$$

$$\rho_{mean}\ is\ the\ mean\ density\ of\ Timber$$

The Co-operation factor for built-up section (*STC*),

$$\gamma = \frac{1}{1 + \frac{\pi^2 * E_T * A_T * s_{sc}}{k_{sc} * L_B{}^2}} \qquad \textbf{(Eq 37)}$$

Where,

$$L_B\ is\ the\ length\ of\ the\ Steel\ Beam$$

$$s_{sc}\ is\ the\ spacing\ of\ \ Shear\ Connector$$

The distance of the effective *NA* of the *STC* from the *NA* of steel and timber sections,

$$a_S = \frac{E_T * A_T * (z_{T,b} + z_{S,t})}{\gamma * E_T * A_T + E_S * A_S}$$
$$a_T = z_{T,b} + z_{S,t} - a_S \qquad \textbf{(Eq 38)}$$

The Effective bending stiffness, EI_{eff} is obtained as follows:

$$EI_{eff} = E_T I_T + E_S I_S + \gamma * (E_T A_T * a_T^{\,2} + E_S A_S * a_S^{\,2}) \qquad \textbf{(Eq 39)}$$

The normal strains and stresses (as shown in Figure D.1) are obtained as follows:

$$\begin{aligned}
\epsilon_{T,t} &= -\kappa * (\gamma * a_T + z_{T,t}), & \sigma_{T,t} &= E_T * \epsilon_{T,t} \\
\epsilon_{T,b} &= \kappa * (-\gamma * a_T + z_{T,b}), & \sigma_{T,b} &= E_T * \epsilon_{T,b} \\
\epsilon_{S,t} &= \kappa * (\gamma * a_S - z_{S,t}), & \sigma_{S,t} &= E_S * \epsilon_{S,t} \\
\epsilon_{S,b} &= \kappa * (\gamma * a_S + z_{S,b}), & \sigma_{S,b} &= E_S * \epsilon_{S,b}
\end{aligned} \qquad \textbf{(Eq 40)}$$

D.2.2 Analysis of Hassanieh's Specimens using the Gamma Method

All calculations done in this section are specific to ***Steel – LVL Specimen #1*** used by Hassanieh in his experiments [24]. The length of *STC* beam $L_B = 6000\,mm$.

Properties of Timber Component:

HySpan LVL, $E_T = 13200\,MPa, f_{m,k} = 50\,MPa, k_h = 1.1, k_{mod} = 1\ (instantaneous), \gamma_M = 1.25$

$$\rightarrow f_{m,d} = \frac{50 * 1.1 * 1}{1.25} = 45.8\,MPa$$

$A_T = 30000\,mm^2, EA_T = 0.396 * 10^9\,N, I_T = 14.06 * 10^6\,mm^4, EI_T = 0.185 * 10^{12}\,Nmm^2$

Properties of Steel Component:

Australian Standard Beam 250UB25.7, $E_S = 200000\,MPa, f_y = 320\,MPa,\ \gamma_{M1} = 1$

$A_S = 5128\,mm^2, EA_S = 1.025 * 10^9\,N, I_S = 54.5 * 10^6\,mm^4, EI_S = 10.9 * 10^{12}\,Nmm^2$

Properties of Shear Connectors:

2 x 12 mm Coach Screws @ 250 mm C/C

$\rho_{mean} = 600\,kg/m^{-3},\ k_{sc} = 2 * \frac{2*12*600^{1.5}}{23} = 30671\,N/mm,\ s_{sc} = 250\,mm$

Partial Shear Interaction using the Gamma Method:

$\gamma = 0.531, a_S = 27.5\,mm, a_T = 134\,mm,$

$$\rightarrow EI_{eff} = 15.64 * 10^{12}\,Nmm^2$$

To find the yield load of the *STC,* we assume that either the top fibre of the timber in compression or the bottom fibre of steel in tension has yielded. This will give us the maximum permissible value of curvature κ of the *STC* section.

κ from yielding of Steel bottom fibre (in tension),

$$\kappa = \frac{f_y}{E_S*(a_S+z_{S,b})} = 10.65 * 10^{-6}\ 1/mm$$

κ from yielding of Timber top fibre (in compression),

$$\kappa = \frac{f_y}{E_T*(\gamma*a_T+z_{T,t})} = 20.02 * 10^{-6}\ 1/mm$$

$$\rightarrow \kappa = min(10.65 * 10^{-6}, 22.08 * 10^{-6}) = 10.65 * 10^{-6}\ 1/mm$$

Distribution of strains/stresses at yield load,

$$\epsilon_{T,t} = -0.75 * 10^{-3}, \qquad \sigma_{T,t} = -9.2\ Nmm^{-2}$$

$$\epsilon_{T,b} = -0.36 * 10^{-3}, \qquad \sigma_{T,b} = -4.7\ Nmm^{-2}$$

$$\epsilon_{S,t} = -1.02 * 10^{-3}, \qquad \sigma_{S,t} = -203.9\ Nmm^{-2}$$

$$\epsilon_{S,b} = +1.6 * 10^{-3}, \qquad \sigma_{S,b} = +320\ Nmm^{-2}$$

Maximum moment of resistance of Steel section alone,

$$\rightarrow M_{el,Rd,S} = \frac{f_y * I_S}{z_{S,t/b}} = 140.7\ kNm$$

Maximum moment of resistance of *STC* section,

$$\rightarrow M_{el,Rd,STC} = EI_{eff} * \kappa = 165.2\ kNm$$

The experimental setup is of 4-point bending, as shown in the structural scheme given below in Figure D.2.

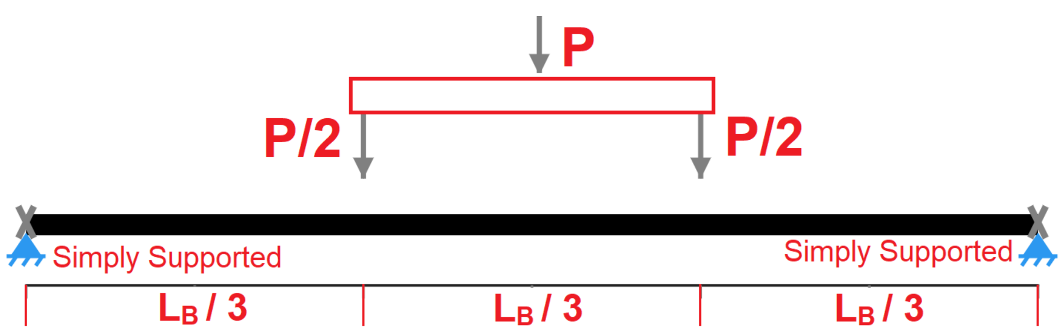


Figure D.2: Structural Scheme of Hassanieh's Experiments. 4-Point Bending.

Maximum Bending moment (at mid-span), $M_{Ed} = P * L_B/6$

Thus, the theoretical yield load of the *STC* section can be calculated as follows,

$$\rightarrow P_y = \frac{M_{Rd,STC} * 6}{L_B} = 165.2\ kN$$

Maximum end slip at yield load,

$$\rightarrow \delta_e = \frac{\Delta\epsilon_{Interface} * L_B}{3} = 1.31\ mm$$

Where,

$$\Delta\epsilon_{Interface} = \epsilon_{T,b} - \epsilon_{S,t}, is\ the\ slip\ strain\ at\ STC\ interface\ at\ the\ mid-span$$

Maximum end slip at yield load from Experiments,

$$\rightarrow \delta_{e,Exp} = 2.3\ mm$$

Full Shear Interaction:

$\gamma = 1, EI_{eff} = EI_{\infty} = 18.54 * 10^{12}\ Nmm^2$

$\kappa_{max}(from\ yielding\ of\ steel) = 9.46 * 10^{-6}\ 1/mm \rightarrow M_{el,STC,Rd,\infty} = 175.56\ kNm$

Strain Distribution of STC Section at mid-span:

The strain distribution observed from experiments have been tabulated below in Table D.3. The experimental values are taken from Figure 6.4, by visual observation. At the yield load, they have been compared to the values obtained from the Gamma method.

Table D.3: Normal Strains at various points over the depth of STC at mid-span.

Location	**Depth** $[\ mm\ ]$	**Strains at Yield Load** $(*\ 10^{-3})\ [-]$	
		Gamma Method	**From Experiment**
Top of Timber	*0*	*-0.75*	*-1*
Bottom of Timber	*75*	*-0.36*	*+0.05*
Top of Steel	*75*	*-1.02*	*-1.1*
Bottom of Steel	*323*	*+1.6*	*2*

D.3 Bolted Connections for STC

D.3.1 Resistance of Bolts in STC

Bearing Strength of Timber in direction parallel to timber grains (*in MPa*),

$$f_{h,0,k} = 0.082 * (1 - 0.01 * d) * \rho_k \quad \textbf{(Eq 41)}$$

Where,

$$\rho_k = 350\ kg/m^3\ is\ the\ characteristic\ density\ of\ timber$$

$$d\ is\ the\ diameter\ of\ the\ Bolt\ (in\ mm)$$

Characteristic Plastic Moment of Resistance of Bolt *(in Nmm)*

$$M_{y,Rk} = 0.3 * f_{ub} * d^{2.6} \quad \textbf{(Eq 42)}$$

Characteristic Pull-out strength of Bolt,

$$F_{ax,k} = (\pi * d * l_{b,eff})^{0.8} * f_{ax,k} \quad \textbf{(Eq 43)}$$

Where,

$$f_{ax,k} = 0.0036 * \rho_k{}^{2.6}$$

$$l_{b,eff} = t_{T,bf} + t_{S,f}\ is\ the\ effective\ length\ of\ the\ Bolt\ in\ STC$$

$$t_{S,f}\ is\ the\ thickness\ of\ Steel\ Flange$$

$$t_{T,bf}\ is\ the\ thickness\ of\ Timber\ Bottom\ Flange$$

Rope Effect Term,

$$F_{Rope} = max(0.25 * F_{ax,k}\,,\ 0.25 * F_{Rd2,3,4}) \quad \textbf{(Eq 44)}$$

Where,

$$F_{Rd2,3,4}\ is\ the\ corresponding\ Johannsen\ Part$$

Bearing Failure of Timber:

Thin plates, $t_{S,f} < 0.5 * d$,

$$F_{Rd1} = 0.4 * f_{h,0,k} * d * t_{T,bf} \quad \textbf{(Eq 45)}$$

Thick plates, $t_{S,f} >= d$,

$$F_{Rd3} = f_{h,0,k} * d * t_{T,bf} \quad \textbf{(Eq 46)}$$

Formation of 1 Plastic Hinge in Bolt:

Thin plates, $t_{S,f} < 0.5 * d$,

$$F_{Rd2} = 1.15 * \sqrt{2 * M_{y,Rk} * f_{h,0k} * d} + F_{Rope} \quad \textbf{\textit{(Eq 47)}}$$

Thick plates, $t_{S,f} >= d$,

$$F_{Rd4} = f_{h,0,k} * d * t_{T,bf} * \left[\sqrt{2 + \frac{4 * M_{y,Rk}}{f_{h,0k} * d * {t_{T,bf}}^2}} - 1\right] + F_{Rope} \quad \textbf{\textit{(Eq 48)}}$$

Formation of 2 Plastic Hinge in Bolt:

Only for Thick plates, $t_{S,f} >= d$,

$$F_{Rd5} = 2.3 * \sqrt{M_{y,Rk} * f_{h,0k} * d} + F_{Rope} \quad \textbf{\textit{(Eq 49)}}$$

Bearing Failure of Resin:

Design Bearing strength of Resin,

$$F_{Rd6} = \frac{k_t * k_s * d * t_{T,bf} * f_{b,resin} * \beta}{\gamma_{M4}} \quad \textbf{\textit{(Eq 50)}}$$

Where,

$$k_t = 1\ for\ SLS, 1.2\ for\ ULS, \qquad k_s = 1$$

$$\beta = 1\ for\ \frac{t_{T,bf}}{t_{S,f}} > 2, \gamma_{M4} = 1$$

$$f_{b,resin} = 200\ MPa\ is\ the\ bearing\ strength\ of\ resin$$

Shear Failure of Bolt:

Shear plane passes through threaded region of bolt

Design Shear Strength of Bolt,

$$F_{Rd7} = \frac{\alpha * A_s * f_{ub}}{\gamma_{M2}} \quad \textbf{\textit{(Eq 51)}}$$

Where,

$$\alpha = 0.6\ for\ Grades\ 4.6, 5.6\ and\ 8.8.\ \alpha = 0.5\ for\ Grades\ 4.8, 5.8, 6.8\ and\ 10.9$$

$$A_s\ is\ the\ Tensile\ stress\ area\ of\ Bolt, \gamma_{M2} = 1.25$$

Parametric Study to determine minimum thickness of Timber Bottom Flange:

Minimum thickness of timber bottom flange, to have ductile connections with bolts of different sizes according to EN 14399 – 4 [63], for connections with thick steel flange, is given below in Table D.4, along with design resistances for each failure modes. This is repeated for bolts of different grades.

Table D.4: Minimum thickness of timber bottom flange for Ductile connections with Bolts.

d [mm]	$t_{T,bf}$ [mm]	Design Resistance [kN]								
		F_{Rd1}	F_{Rd2}	F_{Rd3}	F_{Rd4}	F_{Rd5}	F_{Rd6}	F_{Rd7}	F_{Rd_THIN}	F_{Rd_THICK}
Grade 4.6 Bolts										
10	**25**	2.58	7.14	6.45	6.33	9.67	9.6	50	2.58	6.33
12	**29**	3.51	9.8	8.78	8.68	13.18	13.82	69.6	3.51	8.68
14	**33**	4.56	12.79	11.4	11.31	17.11	18.81	92.4	4.56	11.31
16	**38**	5.86	16.07	14.65	14.27	21.47	24.57	121.6	5.86	14.27
18	**42**	7.11	19.63	17.79	17.41	26.14	31.1	151.2	7.11	17.41
20	**46**	8.44	23.37	21.12	20.77	31.14	38.4	184	8.44	20.77
22	**51**	10.04	27.38	25.11	24.4	36.49	46.46	224.4	10.04	24.4
24	**55**	11.51	31.55	28.79	28.14	42.06	55.29	264	11.51	28.14
27	**61**	13.8	38.13	34.5	34.03	50.87	69.98	329.4	13.8	34.03
Grade 5.6 Bolts										
10	**28**	2.89	7.98	7.23	7.07	10.73	12	56	2.89	7.07
12	**33**	4	10.96	10	9.73	14.66	17.28	79.2	4	9.73
14	**37**	5.11	14.26	12.78	12.65	19	23.52	103.6	5.11	12.65
16	**42**	6.48	17.86	16.2	15.92	23.81	30.72	134.4	6.48	15.92
18	**47**	7.96	21.75	19.9	19.47	29.02	38.88	169.2	7.96	19.47
20	**51**	9.36	25.86	23.41	23.19	34.55	48	204	9.36	23.19
22	**56**	11.03	30.27	27.57	27.2	40.45	58.08	246.4	11.03	27.2
24	**61**	12.77	34.91	31.93	31.42	46.67	69.12	292.8	12.77	31.42
27	**68**	15.38	42.23	38.46	38.03	56.47	87.48	367.2	15.38	38.03
Grade 6.8 Bolts										
10	**30**	3.09	8.74	7.74	7.73	11.67	14.4	60	3.09	7.73
12	**36**	4.36	11.98	10.91	10.65	15.96	20.73	86.4	4.36	10.65
14	**41**	5.66	15.53	14.16	13.88	20.72	28.22	114.8	5.66	13.88
16	**46**	7.09	19.42	17.74	17.44	25.95	36.86	147.2	7.09	17.44
18	**51**	8.64	23.63	21.6	21.29	31.6	46.65	183.6	8.64	21.29
20	**55**	10.1	28.08	25.25	25.35	37.6	57.6	220	10.1	25.25
22	**61**	12.01	32.91	30.04	29.78	44.06	69.69	268.4	12.01	29.78
24	**66**	13.82	37.93	34.55	34.33	50.8	82.94	316.8	13.82	34.33
27	**74**	16.74	45.91	41.86	41.55	61.51	104.97	399.6	16.74	41.55
Grade 8.8 Bolts										
10	**35**	3.61	10.03	9.04	8.93	13.37	19.2	70	3.61	8.93
12	**41**	4.97	13.66	12.42	12.27	18.26	27.64	98.4	4.97	12.27
14	**47**	6.49	17.73	16.24	16.01	23.72	37.63	131.6	6.49	16.01
16	**52**	8.02	22.14	20.05	20.02	29.67	49.15	166.4	8.02	20.02
18	**58**	9.82	26.96	24.56	24.38	36.16	62.2	208.8	9.82	24.38
20	**64**	11.75	32.12	29.38	29.04	43.11	76.8	256	11.75	29.04
22	**69**	13.59	37.53	33.98	33.87	50.41	92.92	303.6	13.59	33.87
24	**75**	15.7	43.28	39.26	39.06	58.15	110.59	360	15.7	39.06
27	**84**	19	52.38	47.51	47.26	70.39	139.96	453.6	19	47.26
Grade 10.9 Bolts										
10	**40**	4.13	11.15	10.33	10.02	14.89	24	80	4.13	10.02
12	**46**	5.57	15.16	13.94	13.73	20.3	34.56	110.4	5.57	13.73
14	**52**	7.18	19.65	17.96	17.77	26.35	47.04	145.6	7.18	17.77
16	**59**	9.1	24.61	22.75	22.27	33.03	61.44	188.8	9.1	22.27
18	**64**	10.84	29.88	27.11	26.95	40.17	77.76	230.4	10.84	26.95
20	**70**	12.85	35.57	32.14	32.05	47.85	96	280	12.85	32.05
22	**77**	15.16	41.65	37.92	37.55	56.05	116.16	338.8	15.16	37.55
24	**83**	17.37	47.99	43.44	43.22	64.61	138.24	398.4	17.37	43.22
27	**93**	21.04	58.09	52.6	52.3	78.23	174.96	502.2	21.04	52.3

D.3.2 Optimum Bolt Hole Clearances

All formulas in this section are obtained from [34].

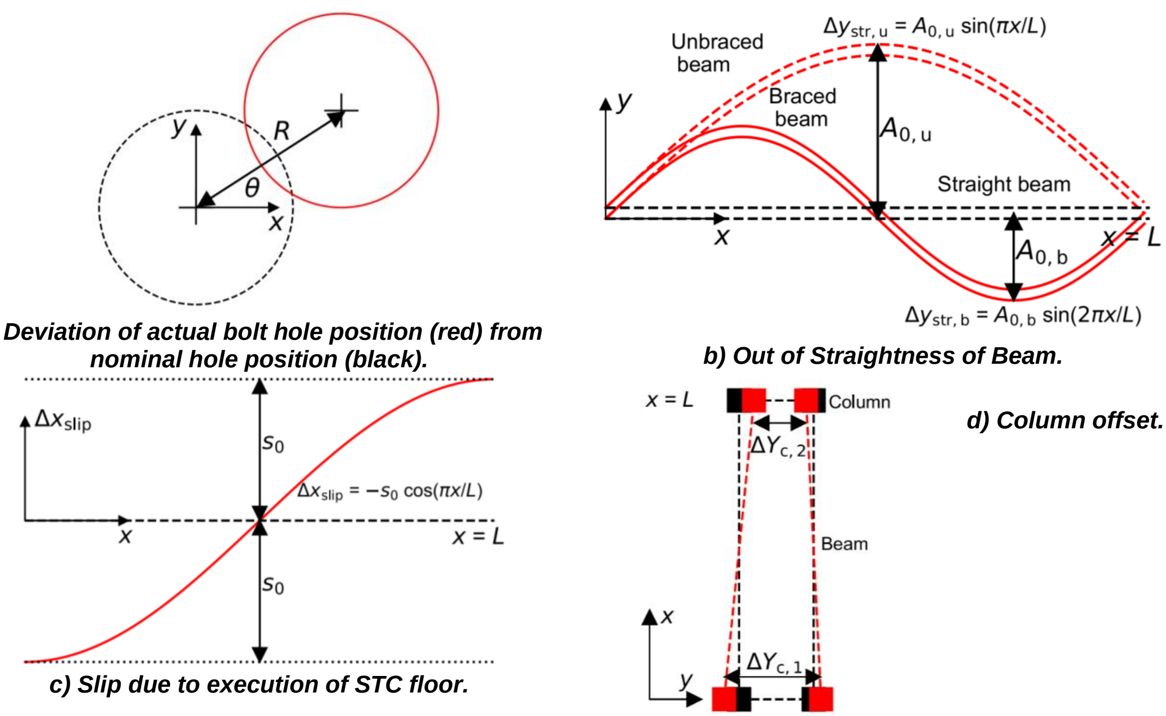
$$\sigma = \frac{|TC|}{1.96} \tag{Eq 52}$$

Where,

$$\sigma \text{ is the standard deviation of the normally distributed variable}$$

$$TC \text{ refers to the tolerance limits associated with each tolerance class}$$

$$\mu \text{ is the average value of the normally distributed variable}$$



a) Deviation of actual bolt hole position (red) from nominal hole position (black).

b) Out of Straightness of Beam.

c) Slip due to execution of STC floor.

d) Column offset.

Figure D.3: Geometrical deviations in prefabricated floor systems, from [34].

Geometrical deviation of the position of bolt hole:

$$(\Delta x_{hole}, \Delta y_{hole}) = (R * cos\theta, R * sin\theta) \qquad \textbf{(Eq 53)}$$

Where,

$$R \text{ is the deviation between centreline of the actual bolt hole to the centreline of the nominal bolt hole}$$

$$R \text{ is a Normally distributed random variable}$$

Table D.5: Statistical values of Normal Random Variable R.

[mm]	TC	σ	μ
Tolerance Class 1	*+-2*	*1.02*	*0*
Tolerance Class 2	*+-1*	*0.51*	*0*

$$\theta \text{ is the angle between the line joining the centrelines and the } x - axis$$

$$\theta \text{ is a Uniformly distributed random variable with limits } [0,2\pi]$$

Out – of Straightness of Beam:

For an unbraced beam,

$$\Delta y_{str,u} = A_0 * sin\left(\frac{\pi * x}{L_{Beam}}\right) \qquad \textbf{(Eq 54)}$$

Where,

$$A_0 \text{ is the magnitude of out } - \text{ of straightness}$$

$$A_0 \text{ is a Normally distributed random variable}$$

$$\sigma_{A_0} = \frac{L_{Beam}}{5700}, \mu_{A_0} = \frac{L_{Beam}}{2800},$$

$$x \text{ is a Uniformly distributed random variable with limits } [0, L_{Beam}]$$

Relative slip due to execution of STC floor:

Relative slip for an unpropped beam,

$$s_0 = \frac{q_{SLS,DL} * {L_{Beam}}^3 * r}{24 * EI_S} \qquad \textbf{(Eq 55)}$$

Where,

$$q_{SLS,DL} \text{ is the SLS dead load on the beam}$$

$$EI_S \text{ is bending stiffness of steel beam only}$$

$$r \text{ is the distance between the neutral axis of the steel and timber section}$$

$$\Delta x_{slip} = -s_0 * cos\left(\frac{\pi * x}{L_{Beam}}\right) \qquad \textbf{(Eq 56)}$$

Position of Shear Connector within Timber Slab:

$$(\Delta x_{sc}, \Delta y_{sc}) = (c_0 * cos\Psi, c_0 * Rsin\Psi) \quad \textbf{\textit{(Eq 57)}}$$

Where,

$$c_0 \ is \ the \ bolt \ to \ slab \ clearance$$

$$c_0 \ is \ a \ Normally \ distributed \ random \ variable$$

Table D.6: Statistical values of Normal Random Variable c_0.

[mm]	TC	σ	μ
All Tolerance Classes	*+-1*	*0.51*	*0*

$$\Psi \ is \ the \ angle \ of \ direction \ of \ offset$$

$$\Psi \ is \ a \ Uniformly \ distributed \ random \ variable \ with \ limits \ [0,2\pi]$$

Column Offset:

$$\begin{gathered} \Delta y_{c,L} = \eta_L * \Delta Y_{c,1} + \frac{x}{L_{Beam}} * (\eta_R * \Delta Y_{c,2} - \eta_L * \Delta Y_{c1}) \\ \Delta x_{c,L} = \Delta X_{c,L}, \Delta x_{c,R} = \Delta X_{c,R} \\ \Delta y_{c,R} = \eta_R * \Delta Y_{c,1} + \frac{x}{L_{Beam}} * (\eta_L * \Delta Y_{c,2} - \eta_R * \Delta Y_{c1}) \end{gathered} \quad \textbf{\textit{(Eq 58)}}$$

Where,

$$\Delta Y_{c,1/2} \ is \ the \ deviation \ between \ centreline \ of \ columns \ in \ row \ 1/2$$

$$\Delta X_{c,L/R} \ is \ the \ deviation \ between \ centreline \ of \ columns \ in \ along \ Left/Right \ Beam$$

$$\Delta Y_{c,1/2}, \Delta X_{c,L/R} \ are \ Normally \ distributed \ random \ variables$$

Table D.7: Statistical values of Normal Random Variables $\Delta Y_{c,1/2}$, $\Delta X_{c,L/R}$.

[mm]	Tolerance	σ	μ
Tolerance Class 1	*+-10*	*5.1*	*0*
Tolerance Class 2	*+-5*	*2.55*	*0*

$$\eta_{L/R} \ is \ the \ Uniformly \ distributed \ random \ variable \ with \ limits \ [0,1]$$

Total deviations,

$$r_H = \sqrt{(\Delta x_{Hole} + \Delta x_c - \Delta x_{slip} - \Delta x_{sc})^2 + (\Delta y_{Hole} + \Delta y_c + \Delta y_{str,u} - \Delta y_{sc})^2} \quad \textbf{\textit{(Eq 59)}}$$

Minimum required hole clearance,

$$d_H > 2 * r_H \quad \textbf{\textit{(Eq 60)}}$$

Where,

$$d \ is \ the \ bolt \ diameter, d_H \ is \ the \ hole \ clearance$$

D.4 Design of STC Beam

D.4.1 General Properties of Sections

Timber Section: *LFE160*

$L_S = 5300\ mm,\ h_T = 160\ mm$

$k_{def} = 0.6, k_{mod} = 0.8,\ E_T = E_{1,T,ULS} = \frac{11000}{1+0.5*0.6} = 8461\ Nmm^{-2},$

Top Flange	***Web***	***Bottom Flange***
$t_{T,tf} = 34\ mm$ $b_{T,tf} = min(760{,}610{,}2725) = 610\ mm$ $k_h = 1.3$ $f_{md} = \frac{1.3*24*0.8}{1.3} = 19.2\ MPa$	$t_{T,w} = 2*40 = 80\ mm$ $h_{T,w} = 73\ mm$ $k_h = 1.3$ $f_{md} = \frac{1.3*24*0.8}{1.3} = 19.2\ MPa$	$t_{T,bf} = 53\ mm$ $b_{T,tf} = 190\ mm$ $k_h = 1.23$ $f_{md} = \frac{1.23*24*0.8}{1.3} = 18.25\ MPa$

$f_{md} = min\ (19.2{,}18.25) = 18.25\ MPa$

$A_T = 42480\ mm^2,\ I_T = 130.5*10^6\ mm^4$

Elastic neutral axis: $x_T = 68\ mm = e_T$

$EI_T = 0.1104*10^{13}\ Nmm^2,\ EA_T = 0.359*10^9\ N$

Steel Section: *HEA360*

$E_S = 200\ GPa,\ f_y = 355\ MPa$
$h_S = 350\ mm,\ b_S = 300\ mm,$
$t_{S,w} = 10\ mm,\ t_{S,f} = 17.5\ mm$
$Cross\ Section\ Class: 2,\ L_B = 10900\ mm$
$A_S = 14276\ mm^2,\ A_{S,v} = 10500\ mm^2$
$I_S = I_{xx} = 316.5*10^6\ mm^4,$

$M_{el,Rd,S} = 642.1\ kNm \rightarrow q_{el,S} = 42.23\ kN/m$
$W_{pl,S} = 1.702*10^6\ mm^3, M_{pl,Rd,S} = 604\ kNm$
Elastic neutral axis:
$x_S = 175\ mm\ ,\ e_S = 160 + 175 = 335\ mm$
$EI_S = 6.33*10^{13}\ Nmm^2,$
$EA_S = 2.855*10^9\ N$
$N_{pl,Rd,S} = 5068\ kN,\ N_{pl,Rd,S,w} = 1118\ kN$

D.4.2 STC with Full Composite Action

Effective Bending Stiffness:

Effective Bending Stiffness for Zero Composite Action

$$EI_0 = EI_S + EI_T = 6.44*10^{13}\ Nmm^2$$

Elastic neutral axis (*NA)* of *STC* :

$$x_{el} = \frac{0.310*57.5 + 1.976*385}{0.310 + 1.976} = 305\ mm$$

Distance to Centroids of steel/timber sections to elastic *NA* of *STC*,

$$a_T = 237\ mm, a_S = 30\ mm$$

Distance of Centroids of stee/timber sections to *STC* interface,

$$r_T = 160 - 68 = 92\,mm, r_S = 175\,mm \rightarrow r = 267\,mm$$

Bending Stiffness (assuming Rigid Connections),

$$EI_{STC} = EI_0 + EA_T * a_T{}^2 + EA_S * a_S{}^2 = 8.712 * 10^{13}\,Nmm^2$$

SLS live loads on beam, $q_Q = 18.98\,kN/m < q_{el,STC} = 43.65\,kN/m$ (Calculated below)

and $q_Q = 18.98\,kN/m < q_{elastic\ limits,S} = 42.23\,kN/m$

Deflection for Steel beam alone, $\Delta_S = \frac{5*q_Q*L_B{}^4}{384*EI_S} = 55.1\,mm$

Total deflection of *STC* beam, $\Delta_{STC} = \frac{5*q_Q*L_B{}^4}{384*EI_{STC}} = 42.12\,mm$

First Moment of area of Timber at *STC* interface,

$$S_T = 10.07 * 10^6\,mm^3 \rightarrow ES_T = 8.525 * 10^{10}\,Nmm$$

Distance of extreme fibres of steel/timber sections from *STC* elastic *NA,*

$$z_{STC,T,t} = 305\,mm, \qquad z_{STC,T,b} = 145\,mm = z_{STC,S,t}, \qquad z_{STC,S,b} = 205\,mm$$

Load Carrying Capacity of STC Section:

Similar to the approach in Appendix D.2.2, we compute the elastic bending moment resistance of the *STC* section $M_{el,Rd,STC}$ from the yielding of steel/timber extreme fibres. Here, there is the difference that the steel beam is precambered to take all the dead loads, as mentioned in Section 6.4.1. All the live loads are taken by the *STC*. Thus, the total load carrying capacity of this system will be the sum of the capacity of the *STC* section added to the dead loads taken by the steel beam alone.

Dead loads taken by steel beam, $q_G = 10.85\,kN/m \rightarrow M_G = 161.2\,kNm$

Stress in steel extreme fibres, $\sigma_{G,S,t/b} = \frac{161.2*175}{316.5} = +-89.1\,Nmm^{-2}$

STC Section Only (Excluding the Effect of Precambering)

From yielding of Steel bottom fibre (in tension):

$$\kappa = \frac{f_y}{E_S * z_{STC,S,b}} = 8.66 * 10^{-6}\,1/mm$$

$$M_{el,Rd,STC} = \kappa * EI_{STC} = 755.3\,kNm$$

From yielding of Timber top fibre (in compression):

$$\kappa = \frac{f_{md}}{E_T * z_{STC,T,t}} = 7.44 * 10^{-6}\,1/mm$$

$$M_{el,Rd,STC} = \kappa * EI_{STC} = 648.3\,kNm$$

Therefore, $\boldsymbol{M_{el,Rd,STC}} = min(755.3, 648.3) = \mathbf{648.3\,kNm}$

$$\rightarrow \boldsymbol{q_{el,STC}} = \frac{M_{el,Rd,STC}}{0.125 * L_B{}^2} = \mathbf{43.65\,kN/m}$$

Including Effect of Precambering:

From yielding of Steel bottom fibre (in tension):

$$\kappa = \frac{(f_y - \sigma_{Ed,S})}{E_S * z_{STC,S,b}} = 6.49 * 10^{-6}\ 1/mm$$

$$M_{el,Rd,STC} = \kappa * EI_{STC} = 565.7\ kNm$$

From yielding of Timber top fibre (in compression):

$$\kappa = \frac{f_{md}}{E_T * z_{STC,T,t}} = 7.44 * 10^{-6}\ 1/mm$$

$$M_{el,Rd,STC} = \kappa * EI_{STC} = 648.3\ kNm$$

Therefore, Total Moment Carrying Capacity of *STC*,

$$\boldsymbol{M_{el,Rd,STC,Total}} = min(565.7{,}648.3) + 161.2 = \mathbf{726.9\ kNm}$$

$$\rightarrow \boldsymbol{q_{el,STC,Total}} = \frac{M_{el,Rd,STC,Total}}{0.125 * {L_B}^2} = \mathbf{54.5\ kN/m}$$

Distribution of Stresses at ULS in STC Section:

ULS Load, $q_{ULS} = 41.5\ kN/m$

For Steel Section alone:

Elastic Load limit at *ULS*, $q_{el,S} = 42.23 > q_{ULS}$

$$\rightarrow M_{ULS} = 616.2\ kNm$$

Stresses in Steel in Extreme fibres, $\sigma_{ULS,S,t/b} = \frac{M_{ULS} * z_{S,t/b}}{I_S} = +-340.7\ MPa$

For STC Section:

All dead loads are taken by steel beam, $\sigma_{G,S,t/b} = +-89.1\ Nmm^{-2}$

Loads taken by *STC*, $\rightarrow q_{STC,ULS} = 41.5 - 10.8 = 30.64\ kN/m$

$$\rightarrow M_{STC,ULS} = 455.02\ kNm, \qquad EI_{STC} = 8.712 * 10^{13}\ Nmm^2$$

$$\rightarrow \kappa = \frac{M_{STC,ULS}}{EI_{STC}} = 5.22 * 10^{-6}\ 1/mm$$

Elastic load limit of *STC* Section $q_{el,STC} = 43.65\ kN/m > q_{STC,ULS}$

Table D.8: Distribution of Normal Strains and Stresses in STC Section.

Depth of STC ($\boldsymbol{z}$) [$\boldsymbol{mm}$]	Normal Strains due to Live Loads ($\boldsymbol{\epsilon_{ULS-G}}$) [$* \mathbf{10^{-3}}$]	Normal Stresses due to Live Loads ($\boldsymbol{\sigma_{ULS-G}}$) [$\boldsymbol{Nmm^{-2}}$]	Normal Stresses due to Dead Loads ($\boldsymbol{\sigma_G}$) [$\boldsymbol{Nmm^{-2}}$]	Total Normal Stresses ($\boldsymbol{\sigma_{ULS}}$) [$\boldsymbol{Nmm^{-2}}$]
0	*1.59*	*13.47*	*0*	*13.47*
160	*0.75*	*6.4*	*0*	*6.4*
160	*0.75*	*151.51*	*89.11*	*240.62*
510	*1.06*	*213.85*	*89.11*	*302.97*

Plastic Bending Moment Capacity of STC Section:

For Steel Section alone,

Plastic Section Modulus, $W_{pl,S} = 2.09 * 10^6\ mm^3 \rightarrow M_{pl,Rd,S} = 741.1\ kNm$

For the *STC* section,

$$N_T = A_T * f_{md} = 800.5\ kN, N_S = A_S * f_y = 5068\ kN, N_{S,w} = A_{S,w} * f_y = 1118\ kN$$

$$\rightarrow Plastic\ NA\ in\ Steel\ Web$$

$M_{pl,Rd,STC} = N_T * (z_{T,b} + z_{S,t}) + M_{pl,Rd,S} - \frac{{N_T}^2}{4 * f_y * t_{S,w}} = 876.4\ kNm$ (from *EC4* [64])

D.4.3 Shear Connectors

2 x Grade 4.6 M14 Bolts,

$t_{T,bf} = 53\ mm, t_{S,tf} = 17.5\ mm, d = 14\ mm \rightarrow$ Connection with Thick steel plates

Size of bolt hole clearance, $d_H = 16\ mm$, from Section 6.3.2.

Design resistance of 2 x Grade 4.6 M14 bolts (per row), $P_{Rd,sc} = 13.1 * 2 = 26.2\ kN$

The edge distances $e_{1,sc}$ and $e_{2,sc}$ provided in the longitudinal and transverse direction conform to the minimum values according to *EC5* [53].

$$e_{1,sc} = e_{1,sc,min} = max(70, 8 * d) = 112\ mm$$

$$e_{2,sc,min} = 4 * d = 56\ mm \qquad \textbf{(Eq 61)}$$

The spacing of the shear connectors, s_{sc} are obtained as follows:

$$s_{sc} = \frac{0.5 * L_B - e_{1,sc}}{N_{sc}} \qquad \textbf{(Eq 62)}$$

Where,

$$N_{sc}\ is\ the\ total\ number\ of\ bolt\ rows\ per\ L_B/2$$

It should be ensured that the spacing of the shear connectors comply with the minimum spacings prescribed by *EC5* [53] for steel-timber connections.

$$p_{1,sc,min} = 5 * d = 70\ mm$$

This sets a limit for the maximum number of shear connectors that can used in the *STC*:

$$N_{sc,max} = \frac{0.5 * L_B - e_{1,sc}}{p_{1,sc,min}} = 76$$

For complete composite action between steel and timber, the shear connectors should be designed to withstand the longitudinal shear flow between the 2 elements.

Total longitudinal shear flow at the steel-timber interface, (from *ULS*)

$$V_{STC,Ls,Ed} = \frac{q_{STC,ULS} * ES_T * L_B{}^2}{EI_{STC} * 4}, \qquad q_{STC,ULS} = 30.64\ kN/m$$

$$\rightarrow V_{STC,Ls,Ed} = 875.2\ kN$$

Number of Shear Connectors required, from Shear Flow,

$$N_{sc} = 875.2/26.2 = 33.4 \sim 2\ x\ 34 \quad per\ L_B/2$$

D.4.4 Composite Action in Steel – Concrete

The properties of the steel and concrete sections are given in Table C.15, in Appendix C.3.4.

Bending Stiffness of steel beam (*HEA)* $\rightarrow EI_S = 3.652 * 10^{13}\ Nmm^2$

Bending Stiffness of Concrete $\rightarrow EI_C = 0.128 * 10^{13}\ Nmm^2$

Bending Stiffness of Composite Beam $\rightarrow EI_{Comp} = 11.33 * 10^{13}\ Nmm^2$

Elastic *NA* of Composite beam, $x_{el} = 135\ mm$

Distance to top and bottom of concrete from *NA* of Composite beam,

$$\rightarrow z_{Comp,C,t} = 135\ mm, z_{Comp,C,b} = 65\ mm$$

Distance to top and bottom of steel from *NA* of Composite beam,

$$\rightarrow z_{Comp,S,t} = 5\ mm, z_{Comp,S,b} = 285\ mm$$

Loads, $q_G = 13.1\ kN/m, q_Q = 12.7\ kN/m, q_{ULS} = 34.8\ kN/m, q_{ULS,Comp} = 21.7\ kN/m$

Like in the case of *STC,* here also, the steel beams are precambered to take the dead loads.

$M_G = 194.8\ kNm$, Stress in steel due to dead loads, $\sigma_{G,S,t/b} = \frac{194.8*145}{566.9} = +-154.7\ Nmm^{-2}$

Elastic Yield limit of Steel Beam, $M_{el,Rd,Comp} = \frac{f_y * z_{S,t/b}}{I_S} = 447\ kNm$, $q_{el,S} = 30.1\ kN/m$

Elastic Yield Limit of Composite Section

From yielding of Steel bottom fibre (in tension):

$$M_{el,Rd,Comp} = \frac{(f_y - 154.7) * EI_{Comp}}{E_S * z_{Comp,S,b}} = 398.6\ kNm$$

From yielding of Concrete top fibre (in compression):

$$M_{el,Rd,Comp} = \frac{f_{cd} * EI_{Comp}}{E_C * z_{Comp,C,t}} = 648.3\ kNm$$

$$\rightarrow q_{el,Comp} = \frac{min(M_{el,Rd,Comp})}{0.125 * L_B{}^2} = 39.9\ kN/m$$

$$\rightarrow q_{el,Comp,Total} = q_G + q_{el,Comp} = 53.1\ kN/m, M_{el,Rd,Comp,Total} = 593.4\ kNm$$

$$q_{ULS,Comp} = 21.7\ kN/m < q_{el,Comp} = 39.9\ kN/m \rightarrow M_{ULS,Comp} = 322.3\ kNm$$

Stresses in Steel Bottom, $\sigma_{ULS,S,b} = \sigma_{G,S,b} + \frac{M_{ULS,Comp} * z_{Comp,S,b} * E_S}{EI_{Comp}} = 316.6\ Nmm^{-2}$

Considering steel beam only, $q_{ULS} = 34.8\ kN/m > q_{el,S} = 30.1\ kN/m \rightarrow Steel\ is\ yielding$

Plastic Section modulus of Steel, $W_{pl,S} = 1.383 * 10^6\ mm^3 \rightarrow M_{pl,Rd,S} = 491\ kNm$

Plastic bending moment resistance of Composite Section (from *EC4* [64]),

$$N_C = 0.85 * A_C * f_{cd} = 3242\ kN, N_S = A_S * f_y = 3994\ kN \rightarrow NA\ is\ Steel\ Flange$$

$$N_A = N_S - N_C = 752\ kN, a = \frac{N_A}{2 * f_y * b_{S,f}} = 3.53\ mm$$

$$\rightarrow M_{pl,Rd,Comp} = N_C * (0.5 * h_S + h_p + 0.5 * h_C) + N_A * 0.5 * (h_S - a) = 886\ kNm$$

The comparison of mechanical properties for composite action in steel – concrete is summarised below in Table D.9.

Table D.9: Summary of Properties for Composite Action in Steel – Concrete.

Property	Steel Beam	Composite Beam	Percentage Change
Bending Stiffness $(\boldsymbol{EI})\ [\boldsymbol{x10^{13}\ Nmm^2}]$	*3.652*	*11.33*	***+210.24%***
Peak Stresses in Steel $(\boldsymbol{\sigma_{S,b}})\ [\boldsymbol{MPa}]$	*Yielding*	*316.6*	***At least -10.8%***
Elastic Bending Moment Capacity $(\boldsymbol{M_{el,Rd}})\ [\boldsymbol{kNm}]$	*447*	*593.4*	***+32.7***
Plastic Bending Moment Capacity $(\boldsymbol{M_{pl,Rd}})\ [\boldsymbol{kNm}]$	*491*	*886*	***+80.4%***

Shear Connectors used → 2 x Grade 4.6 M14 Bolts,

$$P_{Rd,Bolt} = \frac{0.8 * f_{ub} * \pi * d^2}{4 * 1.25} = 39.38\ kN \rightarrow From\ Shear\ failure\ of\ Bolt$$

$$P_{Rd,Bolt} = \frac{0.29 * 1 * d^2 * (f_{ck} * E_{cm})^{0.5}}{1.25} = 45.2\ kN \rightarrow From\ Crushing\ of\ Concrete$$

Therefore, $P_{Rd,sc} = 2 * min(39.4, 45.2) = 78.8\ kN$ per Bolt Row

Total longitudinal shear flow at the Steel – Concrete interface, (from *ULS*)

$$V_{Comp,Ls,Ed} = \frac{q_{Comp,ULS} * ES_C * {L_B}^2}{EI_{Comp} * 4}, \qquad q_{STC,ULS} = 30.64\ kN/m$$

$$\rightarrow V_{STC,Ls,Ed} = 1794\ kN$$

Number of Shear Connectors required, from Shear Flow,

$$N_{sc} = 1794/78.8 = 22.7 \sim 2\ x\ 23\ \ per\ L_B/2$$

For plastic distribution of stresses,

$$N_{sc} = N_C/P_{Rd,sc} = 41.1 \sim 2x42\ Bolts\ per\ L_B/2$$

D.4.5 Composite Action in STC with CLT slabs

For comparison of composite action in an *STC* floor using *CLT* slabs, which offer the advantage of bidirectional bending, the slab is chosen such that it is comparable to ***DA1_STC***. Thus, the same structural grid is maintained, and the slab span of 5.3 m is assumed. From Table A.5, the required section is adopted, as shown in Figure D.4 (Right). The properties of *CLT* slabs are as given in Table A.1.

From Figure D.4, it can be observed that in the longitudinal section, layers 1,3 and 5 of the *CLT* slab are oriented parallel to the y direction i.e., they are oriented optimally to transfer loads in the y-direction. In the transverse section below, it can be observed that the grain direction of these layers is perpendicular to the main load carrying direction (x-direction in this case). Consequently, the bending stiffness of the *CLT* slab is less, and more importantly, less than that for the *LFE* slabs, as given in Table D.10. Since we use reoriented timber flanges near the supports for the *LFE* slabs, we get a higher value of bending stiffness.

Table D.10: Properties of Timber Slabs active in Composite Action. (Left) LFE Slabs. (Right) CLT Solid Slabs.

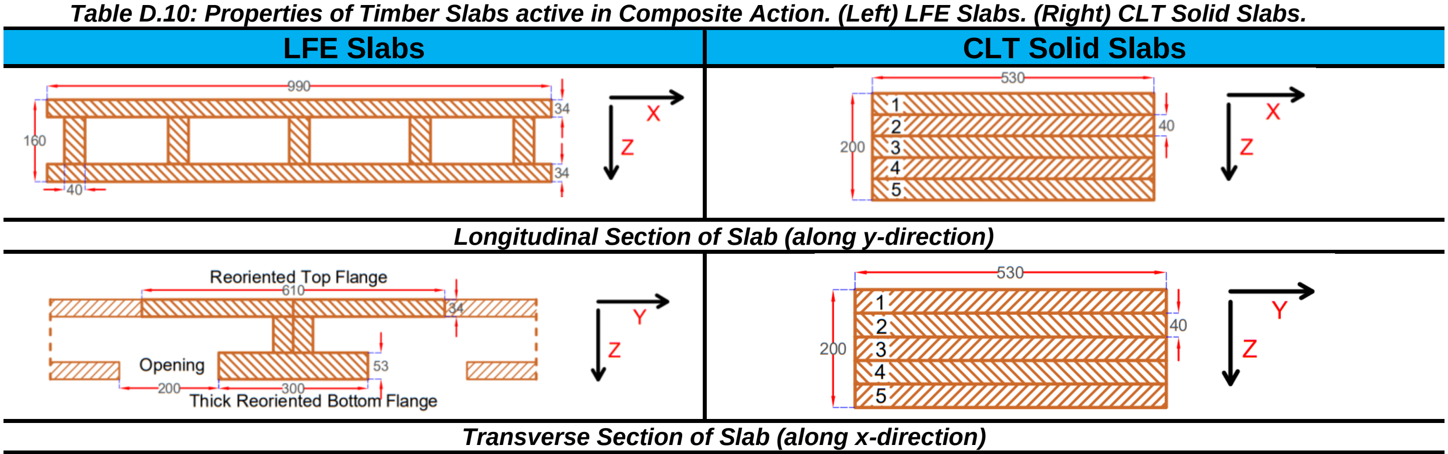


Figure D.4: Sections of Timber Slabs for DA1_STC. (Left) LFE Slabs. (Right) CLT Slabs.

LFE Slabs	CLT Solid Slabs
$A_{T,x} = 42480\, mm^2 = A_{T,eff,y}$	$A_{T,x} = 106000\, mm^2,\ A_{T,eff,x} = 44539\, mm^2$
$I_{T,x} = 130.5 * 10^6\, mm^4$	$I_{T,x} = 82.63 * 10^6\, mm^4$
$EA_{T,x} = 0.359 * 10^9\, N$	$EA_{T,x} = 0.376 * 10^9\, N$
$EI_{T,x} = 0.1104 * 10^{13}\, Nmm^2$	$EI_{T,x} = 0.0699 * 10^{13}\, Nmm^2$

D.5 Span Tables for STC

In calculating the span tables for *STC* floors for the different office layouts, the following assumptions were used:

- **Live Loads:** $4\ kNm^{-2}$ (including partitions), same as used in [132] for other floor systems.
- **Dead Loads:** $1.5\ kNm^{-2}$ (including floor finish, fire and sound additions, and ceilings/services) + Self weight of slab and beam.
- **Structural Scheme:** All slabs and beams are assumed to be simply supported, for ease of calculation, and as a conservative appraoch for design.
- **Timber Slabs:** All design checks done, as given in Section 6.2.
- **Steel Beams:** *HEA* beams are used. Design checks for Deflection, Shear and Bending Moments. Beam is precambered to negate the deflections due to the dead loads.
- **Composite Action:** Same appraoch as in Section 6.4.
- **Floor Height:** For *STC,* floor height is equal to the construction height (height of the structural elements) as the pipes for integration of services is provided inside the floor. For Composite slab, same appraoch as for *STC*.

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Table D.11 below shows the timber sections required for different spans.

Table D.11: Timber sections required for different spans.

Span $[m]$	Section	Depth $[mm]$	Thickness of Flange $[mm]$	Thickness of Web $[mm]$	Spacing of Transverse Stiffeners $[mm]$	Slab Dead Load $[kg/m^2]$
3.6	***LFE120***	*120*	*27*	*42*	*1200*	*28*
5.4	***LFE160***	*160*	*36*	*41*	*1200*	*39*
7.2	***LFE240***	*240*	*33*	*33*	*1200*	*40*
9	***LFE320***	*320*	*32*	*34*	*1200*	*46*

Table D.12 below shows the sections of steel and timber required for different sizes of column grids, for *STC* floor systems. As mentioned earlier, all the slabs and beams are simply supported. Moreover, for the column grids considered below, the slabs are spanning transversely to the beams. The beams are symmetrically loaded on either side, from the slabs. This is shown below in Figure D.4. When *HEA* beams were not sufficient, *HEB* and *HEM* beams were used.

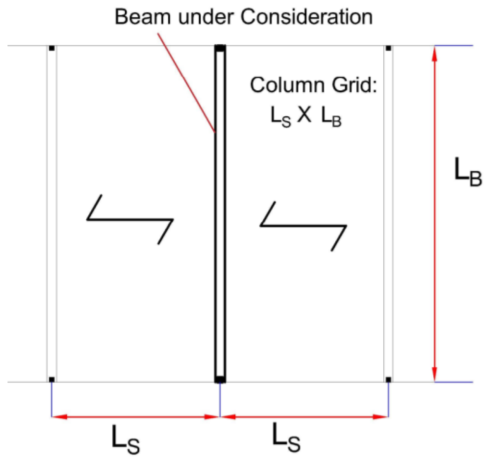

Figure D.5: STC Floor structural scheme.

Table D.12: Span tables for STC Floors for different column grids.

Timber Slab (LFE)		I – Beam (HEA)						
Span of Slab (L_S) $[m]$	Depth of Slab $[mm]$	Span of Beam (L_B) $[m]$	With Composite Action: Section	With Composite Action: Depth of Beam $[mm]$	With Composite Action: Depth of Floor $[mm]$	Without Composite Action: Section	Without Composite Action: Depth of Beam $[mm]$	Without Composite Action: Depth of Floor $[mm]$
3.6	120	5.4	HEA140	133	313	HEA200	190	370
3.6	120	7.2	HEA220	210	390	HEA240	230	410
3.6	120	9	HEA260	250	430	HEA280	270	450
3.6	120	10.8	HEA300	290	470	HEA340	330	510
3.6	120	12.6	HEA360	350	530	HEA400	390	570
3.6	120	14.4	HEA450	440	620	HEA450	440	620
3.6	120	16.2	HEA500	490	670	HEA500	490	670
5.4	160	5.4	HEA140	133	353	HEA220	210	430
5.4	160	7.2	HEA220	210	430	HEA280	270	490
5.4	160	9	HEA280	270	490	HEA320	310	530
5.4	160	10.8	HEA320	310	530	HEA400	390	610
5.4	160	12.6	HEA400	390	610	HEA450	440	660
5.4	160	14.4	HEA450	440	660	HEA500	490	710
5.4	160	16.2	HEA550	540	760	HEA600	590	810
7.2	240	3.6	HEA100	96	396	HEA180	171	471
7.2	240	5.4	HEA120	114	414	HEA240	230	530
7.2	240	7.2	HEA180	171	471	HEA300	290	590
7.2	240	9	HEA260	250	550	HEA340	330	630
7.2	240	10.8	HEA320	310	610	HEA450	440	740
7.2	240	12.6	HEA400	390	690	HEA500	490	790
7.2	240	14.4	HEA450	440	740	HEA600	590	890
7.2	240	16.2	HEA550	540	840	HEB600	600	900
9	320	3.6	HEA100	96	476	HEA200	190	570
9	320	5.4	HEA120	114	494	HEA260	250	630
9	320	7.2	HEA180	171	551	HEA320	310	690
9	320	9	HEA240	230	610	HEA400	390	770
9	320	10.8	HEA300	290	670	HEA450	440	820
9	320	12.6	HEA400	390	770	HEA550	540	920
9	320	14.4	HEA500	490	870	HEA600	590	970
9	320	16.2	HEA550	540	920	HEM600	620	1000

Table D.13 below shows the height of the floors for *HCS* and Composite Slab floor systems. For *HCS*, the services cannot be integrated into the height of the floor system and are provided beneath the steel beams. Thus, the total height of the floor is assumed to be 50 mm greater than the height of construction. For Composite slabs, as mentioned in Section 6.5, whenever a span greater than 3.6 m is required, secondary beams are used.

Table D.13 Span tables for Composite Slab and HCS floor systems for different column grids. a) With Cocoon/Combination Office.

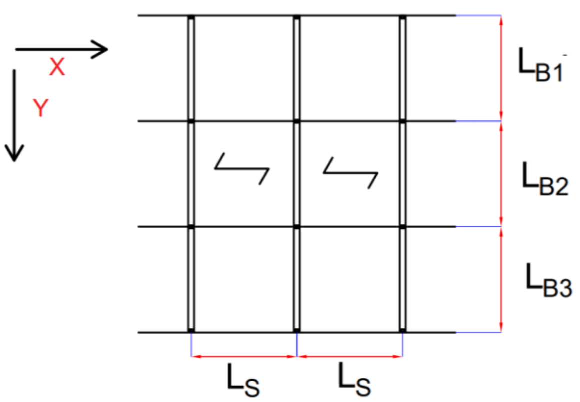

Figure D.6: Structural scheme for Cocoon/Combination Office with HCS floor system.

Primary Beams spanning between columns in y-direction. HCS spanning in x-direction, between the primary beams.

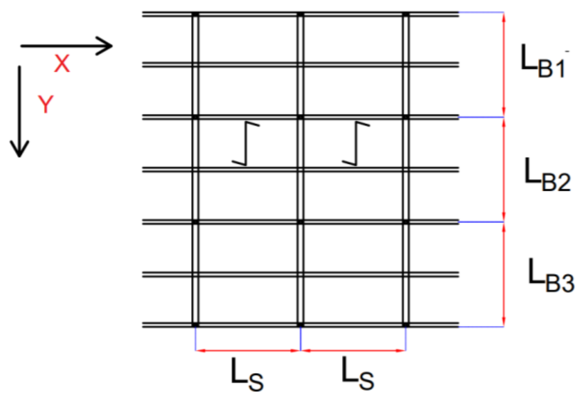

Figure D.7: Structural Scheme for Cocoon/Combination Office with Composite Slab floor system.

Primary beams spanning between columns in y-direction. Secondary beams spanning in x-direction. Composite slabs spanning in y-direction between the secondary beams.

Building Depth (Façade to Façade) $(L_B = L_{B1} + L_{B2} + L_{B3})$ $[m]$	**Maximum Column Grid**		**HCS Floor System**		**Composite Slab Floor System**
	Along x (L_S) $[m]$	**Along x** (L_B) $[m]$	**Construction Height for HCS** $[mm]$	**Floor Height for HCS** $[mm]$	**Floor Height for Composite Slab** $[mm]$
16.2 (5.4+5.4+5.4)	*3.6*	*5.4*	*NA*	*NA*	*370*
	5.4	*5.4*	*245*	*295*	*390*
	7.2	*5.4*	*265*	*315*	*NA*
	9	*5.4*	*325*	*375*	*NA*
18 (5.4+7.2+5.4)	*3.6*	*7.2*	*NA*	*NA*	*440*
	5.4	*7.2*	*NA*	*NA*	*480*
	7.2	*7.2*	*325*	*375*	*540*
	9	*7.2*	*335*	*385*	*NA*
19.8 (5.4+9+5.4)	*3.6*	*9*	*NA*	*NA*	*490*
	5.4	*9*	*NA*	*NA*	*580*
	7.2	*9*	*NA*	*NA*	*630*
	9	*9*	*375*	*425*	*630*
19.8 (7.2+5.4+7.2)	*3.6*	*7.2*	*NA*	*NA*	*440*
	5.4	*7.2*	*NA*	*NA*	*480*
	7.2	*7.2*	*325*	*375*	*540*
	9	*7.2*	*335*	*385*	*NA*

b) With Cell Office.

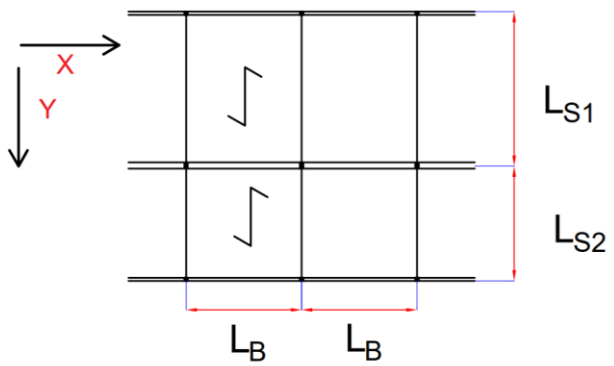

Figure D.8: Structural scheme for Cell Office with HCS floor system.

Primary Beams spanning between columns in x-direction. HCS spanning in y-direction, between the primary beams.

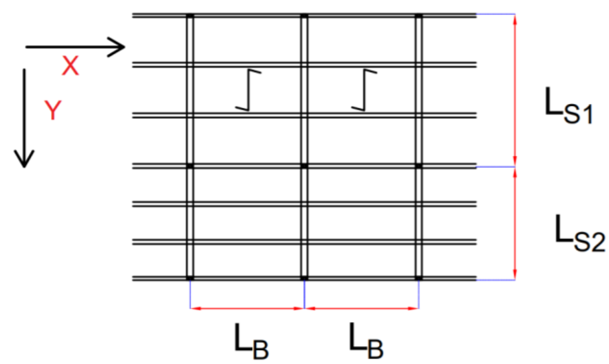

Figure D.9: Structural Scheme for Cell Office with Composite Slab floor system.

Primary beams spanning between columns in y-direction. Secondary beams spanning in x-direction. Composite slabs spanning in y-direction between the secondary beams.

Building Depth (Façade to Façade) ($L_B = L_{B1} + L_{B2}$) $[m]$	**Maximum Column Grid**		**HCS Floor System**		**Composite Slab Floor System**
	Along x (L_S) $[m]$	**Along x** (L_B) $[m]$	**Construction Height for HCS** $[mm]$	**Floor Height for HCS** $[mm]$	**Floor Height for Composite Slab** $[mm]$
12.4 (5.4+7.2)	*7.2*	*3.6*	*265*	*315*	*440*
	7.2	*5.4*	*265*	*315*	*480*
	7.2	*7.2*	*325*	*375*	*540*
14.4 (5.4+9)	*9*	*3.6*	*325*	*375*	*490*
	9	*5.4*	*325*	*375*	*580*
	9	*7.2*	*325*	*375*	*630*
14.4 (7.2+7.2)	*7.2*	*3.6*	*265*	*315*	*440*
	7.2	*5.4*	*265*	*315*	*480*
	7.2	*7.2*	*325*	*375*	*540*
16.2 (7.2+9)	*9*	*3.6*	*325*	*375*	*490*
	9	*5.4*	*325*	*375*	*580*
	9	*7.2*	*325*	*375*	*630*

c) With Group Office.

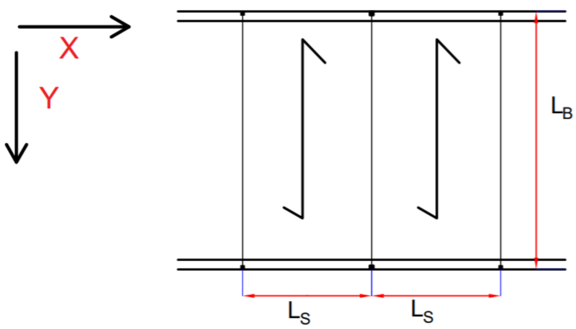

Figure D.10: Structural scheme for Group Office with HCS floor system.

Primary Beams spanning between columns in x-direction, as edge beams. HCS spanning in y-direction, between the primary beams.

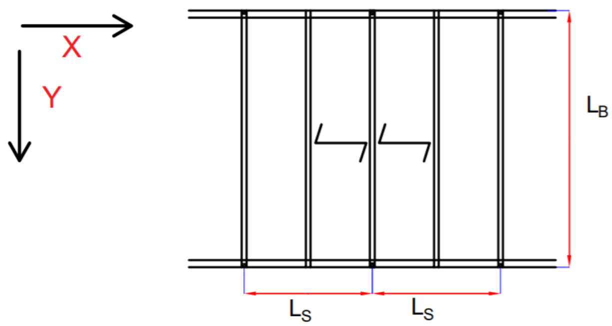

Figure D.11: Structural Scheme for Group Office with Composite Slab floor system.

Primary beams spanning between columns in x-direction. Secondary beams spanning in y-direction. Composite slabs spanning in x-direction between the secondary beams.

Building Depth (Façade to Façade) (L_B) $[m]$	**Maximum Column Grid**		**HCS Floor System**		**Composite Slab Floor System**
	Along x (L_S) $[m]$	**Along x** (L_B) $[m]$	**Construction Height for HCS** $[mm]$	**Floor Height for HCS** $[mm]$	**Floor Height for Composite Slab** $[mm]$
10.8	*3.6*	*10.8*	*200*	*250*	*650*
	5.4	*10.8*	*200*	*250*	*610*
	7.2	*10.8*	*265*	*315*	*650*
	9	*10.8*	*NA*	*NA*	*NA*
12.6	*3.6*	*12.6*	*200*	*250*	*740*
	5.4	*12.6*	*200*	*250*	*670*
	7.2	*12.6*	*265*	*315*	*740*
	9	*12.6*	*NA*	*NA*	*NA*
14.4	*3.6*	*14.4*	*385*	*435*	*810*
	5.4	*14.4*	*385*	*435*	*730*
	7.2	*14.4*	*385*	*435*	*810*
	9	*14.4*	*NA*	*NA*	*NA*
16.2	*3.6*	*16.2*	*465*	*515*	*890*
	5.4	*16.2*	*465*	*515*	*800*
	7.2	*16.2*	*465*	*515*	*890*
	9	*16.2*	*NA*	*NA*	*NA*

E. Life Cycle Analysis

E.1 Quantity of Materials

The assumptions used to calculate the materials, and the calculation of quantity of each is given below in Table E.1. The design and dimensioning of all the elements considered is done in Chapter 5.

Table E.1: Quantity of Materials.

Assumptions:

STC Floors Number of beams: 2 Side Cross Beams, 3 Cross Beams, 2 Edge Beams Number of Columns: 10 HCS Floors Number of beams: 2 Edge Beams Number of Columns: 6	CS Floors Rebars: 2% by area of concrete for unpropped construction [99] Number of beams: 7 Cross Beams, 2 Edge Beams Number of Columns: 6

Design Alternative	Component	Material	Weight per GFA $[kg/m^2]$	Remark
Common for all STCs	*Sound Insulation*	*Chipboard*	*19.02*	*Obtained from Appendix A.3.*
	Sound Insulation	*Insulating Mineral Wool*	*4.8*	
	Fire Protection	*Gypsum Plasterboard*	*27.2*	
DA1_STC With Composite Action	*Slabs*	*Timber*	*36.73*	*Calculations in Appendix C.2.*
	Beams	*Steel*	*26.3*	
	Columns	*Steel*	*3.5*	
DA1_STC No Composite Action	*Slabs*	*Timber*	*36.73*	
	Beams	*Steel*	*28.1*	
	Columns	*Steel*	*3.5*	
DA2_HCS	*Slabs*	*HCS*	*383*	*From [98]*
	Beams	*Steel*	*43.6*	*Calculations in Appendix C.3.*
	Columns	*Steel*	*5.8*	
DA3_CS With Composite Action	*Composite Slab*	*ComFlor60*	*11.2*	*From [99]*
	Composite Slab	*Concrete*	*240.6*	
	Composite Slab	*Rebars*	*1.1*	
	Beams	*Steel*	*52.1*	*Calculations in Appendix C.4.*
	Columns	Steel	5.1	

E.2 Comparison of GWP

E.2.1 EPD Data

For sawn timber, the *GWP* data from 7 different *EPDs* are used, with different *EoL* scenarios as discussed in Section 7.3.5. The *GWP* data used here is the total *GWP* which includes the biogenic carbon, and is applicable to EN 15804+A1 and EN15804+A2 *EPDs* alike (total *GWP* for ***A2 EPDs***). For further analysis, the average value of the 7 *EPDs* with Storage as the *EoL* scenario is taken. The declared unit is 1m^3 for all timber *EPDs.* The corresponding environment impact values for 1 kg is obtained by dividing the data with the densities of sawn timber. *GWP* due to biogenic carbon (carbon sequestration effects) is either calculated using Eq. 6 in Section 7.3.3 or obtained directly from the *EPD*. The environment impact data for sawn timber is summarized below in Table E.2.

Table E.2a: Summary of EPDs for Sawn Timber.

EPD	EPD Number	Density $[kg/m^3]$	Biogenic GWP $[kg\ CO_2\ Equivalents/m^3]$	Module A4 Distance $[km]$	*Module C2 Distance $[km]$
BinderHolz	***1***	*460*	*771*	*NA*	*NA, NA, NA, 20, NA*
BergenHolme	***2***	*469*	*715*	*75*	*NA, NA, NA, 85, NA*
Mobilindustrien	***3***	*456*	*715*	*650*	*NA, NA, 150, 100, NA*
Finnish Sawn Timber	***4***	*474*	*728*	*191*	*NA, NA, NA, 100, NA*
Eggert	***5***	*503*	*802*	*NA*	*NA, NA, NA, 50, NA*
Swedish Wood	***6***	*489*	*773*	*NA*	*NA*
Stora Enso	***7***	*460*	*733*	*NA*	*50,50,50,50,50*

*** Distances for Module C2 are given for different EoL scenarios in the order Storage, Reuse, Recycle, Incineration and Landfill respectively.**

Table E.2b: GWP Data for Sawn Timber from EPDs.

Stage of LCA		Total GWP $[CO_2\ equivalents/kg]$							
		1	2	3	4	5	6	7	Average
Module A	***A1-A3***	-1.4E+00	-1.5E+00	-1.5E+00	-1.4E+00	-1.4E+00	-1.5E+00	-1.5E+00	-1.5E+00
	A4	0.0E+00	9.4E-03	4.2E-02	1.9E-02	0.0E+00	0.0E+00	0.0E+00	1.0E-02
	A5	3.3E-03	6.8E-03	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	1.4E-03
	Total	-1.4E+00	-1.5E+00	-1.4E+00	-1.4E+00	-1.4E+00	-1.5E+00	2.5E-08	-1.2E+00
EoL: Storage	***C1***	0.0E+00	1.9E-05	0.0E+00	1.1E-03	0.0E+00	5.0E-04	2.5E-08	2.4E-04
	C2	1.2E-03	1.4E-02	1.0E-02	9.0E-03	2.9E-03	1.4E-02	4.2E-03	7.9E-03
	C3	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
	C4	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
	D	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
	Total (A+C+D)	-1.4E+00	-1.5E+00	-1.4E+00	-1.4E+00	-1.4E+00	-1.5E+00	-1.5E+00	-1.4E+00
EoL: Reuse	***C1***	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	2.5E-08	3.6E-09
	C2	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	4.2E-03	6.1E-04
	C3	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	1.6E+00	2.3E-01
	C4	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
	D	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	-1.7E+00	-2.4E-01
	Total (A+C+D)	-1.4E+00	-1.5E+00	-1.4E+00	-1.4E+00	-1.4E+00	-1.5E+00	-1.6E+00	-1.4E+00
EoL: Recycle	***C1***	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	2.5E-08	3.6E-09
	C2	0.0E+00	0.0E+00	1.2E-02	0.0E+00	0.0E+00	0.0E+00	4.2E-03	2.3E-03
	C3	0.0E+00	0.0E+00	1.6E+00	0.0E+00	0.0E+00	0.0E+00	1.6E+00	4.5E-01
	C4	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00

	D	*0.0E+00*	*0.0E+00*	*-1.7E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*-1.7E+00*	*-4.9E-01*
	Total (A+C+D)	*-1.4E+00*	*-1.5E+00*	*-1.5E+00*	*-1.4E+00*	*-1.4E+00*	*-1.5E+00*	*-1.6E+00*	*-1.5E+00*
EoL: Storage	***C1***	*0.0E+00*	*1.9E-05*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*5.0E-04*	*2.5E-08*	*7.4E-05*
	C2	*1.2E-03*	*1.4E-02*	*8.1E-03*	*0.0E+00*	*2.9E-03*	*1.4E-02*	*4.2E-03*	*6.3E-03*
	C3	*1.7E+00*	*1.6E+00*	*1.6E+00*	*0.0E+00*	*1.6E+00*	*1.6E+00*	*1.6E+00*	*1.4E+00*
	C4	*0.0E+00*	*4.2E-05*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*5.9E-06*
	D	*-8.6E-01*	*-8.6E-02*	*-8.1E-01*	*0.0E+00*	*-8.1E-01*	*-2.4E-01*	*-8.1E-01*	*-5.2E-01*
	Total (A+C+D)	*-5.5E-01*	*5.6E-02*	*-6.6E-01*	*-1.4E+00*	*-5.8E-01*	*-1.6E-01*	*-6.9E-01*	*-5.6E-01*
EoL: Landfill	***C1***	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*2.5E-08*	*3.6E-09*
	C2	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*4.2E-03*	*6.1E-04*
	C3	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*
	C4	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*2.3E+00*	*3.3E-01*
	D	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*-8.3E-03*	*-1.2E-03*
	Total (A+C+D)	*-1.4E+00*	*-1.5E+00*	*-1.4E+00*	*-1.4E+00*	*-1.4E+00*	*-1.5E+00*	*8.0E-01*	*-1.1E+00*

For steel, 5 *EPDs* are considered. All *EPDs* are declared for 1 tonne (1000 kg), except for *EPD12* (BE Group Sverige AB) which is declared for 1 kg. Thus, all the declared data is converted for 1 kg of steel with the relevant conversion factors, and is shown below in Table E.3. All *EPDs* consider a similar *EoL* scenario where most of the steel is reused or recycled (approximately 90%), with a small percentage sent for landfill.

Table E.3a: Summary of EPDs for Structural Steel.

EPD	EPD Number	Declared Unit
Give Steel A/S	***8***	***1000 kg***
ArcelorMittal	***9***	***1000 kg***
Bauforuhmstahl	***10***	***1000 kg***
DS Steel Construction A/S	***11***	***1000 kg***
BE Group Sverige AB	***12***	***1 kg***

Table E.3b: GWP Data for Structural Steel from EPDs.

Stage of LCA		Total GWP $[CO_2\ equivalents/kg]$					
		8	9	10	11	12	Average
Module A	***A1-A3***	*1.0E+00*	*8.4E-01*	*1.3E+00*	*1.3E+00*	*7.2E-01*	*1.1E+00*
	A4	*8.4E-03*	*0.0E+00*	*0.0E+00*	*1.2E-02*	*6.4E-02*	*1.7E-02*
	A5	*6.6E-04*	*0.0E+00*	*0.0E+00*	*6.2E-04*	*0.0E+00*	*2.5E-04*
	Total (A)	*1.0E+00*	*8.4E-01*	*1.3E+00*	*1.3E+00*	*7.8E-01*	*1.1E+00*
EoL: Mixed Scenario for Reuse, Recycle and Landfill	***C1***	*6.6E-04*	*0.0E+00*	*0.0E+00*	*6.2E-04*	*3.3E-03*	*9.1E-04*
	C2	*8.5E-03*	*1.8E-03*	*0.0E+00*	*8.5E-03*	*8.3E-03*	*5.4E-03*
	C3	*2.3E-02*	*0.0E+00*	*2.2E-03*	*4.8E-02*	*2.2E-02*	*1.9E-02*
	C4	*0.0E+00*	*0.0E+00*	*0.0E+00*	*1.9E-04*	*1.9E-01*	*3.8E-02*
	D	*-3.2E-01*	*-9.8E-02*	*-5.0E-01*	*-6.6E-01*	*-1.2E-01*	*-3.4E-01*
	Total (A+C+D)	*7.7E-01*	*7.5E-01*	*8.3E-01*	*7.4E-01*	*8.9E-01*	*7.9E-01*

Apart from steel and timber, concrete is also one of the main building materials used in the floor systems being compared. This includes the cast in-situ ready-mix concrete used for composite slabs, as well as the precast hollow core slabs. Thus, multiple *EPDs* are referred to for these materials as well. The details of the concrete *EPDs,* and their *GWP* data are given below in Table E.4. The declared unit for concrete is 1 m^3, and the data is converted to that corresponding to 1 kg of material using the bulk density.

Table E.4a: Summary of EPDs of Concrete.

EPD Number	Producer	EPD Operator	Modules Not Declared	Region	Validity	Density $[kg/m^3]$
14	*Information Zentrum Beton GmbH* [111]	*IBU*	*C4*	*Germany*	*2018 – 2023*	2400
15	*British Ready-Mix Concrete Association* [110]	*IBU*	*D*	*UK*	*2018 – 2023*	2380
16	*Holcim Romania* [137]	*EPD International*	*A4, A5, Module C and D*	*Romania*	*2020 – 2025*	2329
17	Åkra Sementstøperi AS [140]	*EPD Norway*	*A5, Module C and D*	*Norway*	*2021 – 2026*	2380

Table E.4b: GWP Data for Concrete from EPDs.

Stage of LCA		Total GWP $[CO_2\ equivalents/kg]$				
		8	9	10	11	Average
Module A	***A1-A3***	*9.13E-02*	*9.24E-02*	*8.63E-02*	*1.05E-01*	*9.37E-02*
	A4	*1.88E-03*	*8.45E-04*	*0.00E+00*	*1.53E-03*	*1.06E-03*
	A5	*4.50E-04*	*7.98E-05*	*0.00E+00*	*0.00E+00*	*1.32E-04*
	Total (A)	*9.36E-02*	*9.34E-02*	*8.63E-02*	*1.06E-01*	*9.49E-02*
EoL: Recycling	***C1***	*1.29E-03*	*-3.57E-04*	*0.00E+00*	*0.00E+00*	*2.34E-04*
	C2	*5.00E-03*	*3.47E-03*	*0.00E+00*	*0.00E+00*	*2.12E-03*
	C3	*2.50E-03*	*-7.98E-03*	*0.00E+00*	*0.00E+00*	*-1.37E-03*
	C4	*0.00E+00*	*8.32E-04*	*0.00E+00*	*0.00E+00*	*2.08E-04*
	D	*-8.92E-03*	*0.00E+00*	*0.00E+00*	*0.00E+00*	*-2.23E-03*
	Total (A+C+D)	*9.35E-02*	*8.93E-02*	*8.63E-02*	*1.06E-01*	*9.38E-02*

Figure E.1 below graphically shows the comparison of total *GWP* for concrete. The values for module A show very less variation except for *EPD#17* which shows a slightly higher value. This can be attributed due to the fact that the other *EPDs* are of similar strength grade (C30/37), whereas the former is of strength grade M45/40 (according to Norwegian standards). This is the reason that it has a larger *GWP* value. The main issue with the *EPDs* compared is that only 2 have declared the data for the *EoL* scenario (*EPD#15* has not declared module D, but has given credit in module C3 for waste processing).

Recycling is the most commonly declared *EoL* scenario, wherein the concrete is crushed to produce aggregate. Thus, the total benefits for concrete in the module D is very less. Taking the sum of modules C and D, it results in a very small value in comparison to that of module A. Hence, it can be concluded that including the benefits of *EoL* in the *LCA* does not make much of a difference, and that we can make do with data for the production stage for concrete. The total *GWP* across all modules thus produces a similar trend (and value) to that of module A for concrete *EPDs*. The average, maximum and minimum values $(+0.094, +0.106\ and\ +0.086\ CO_2\ equivalents/kg$ respectively) of total *GWP* have been taken for further analysis.

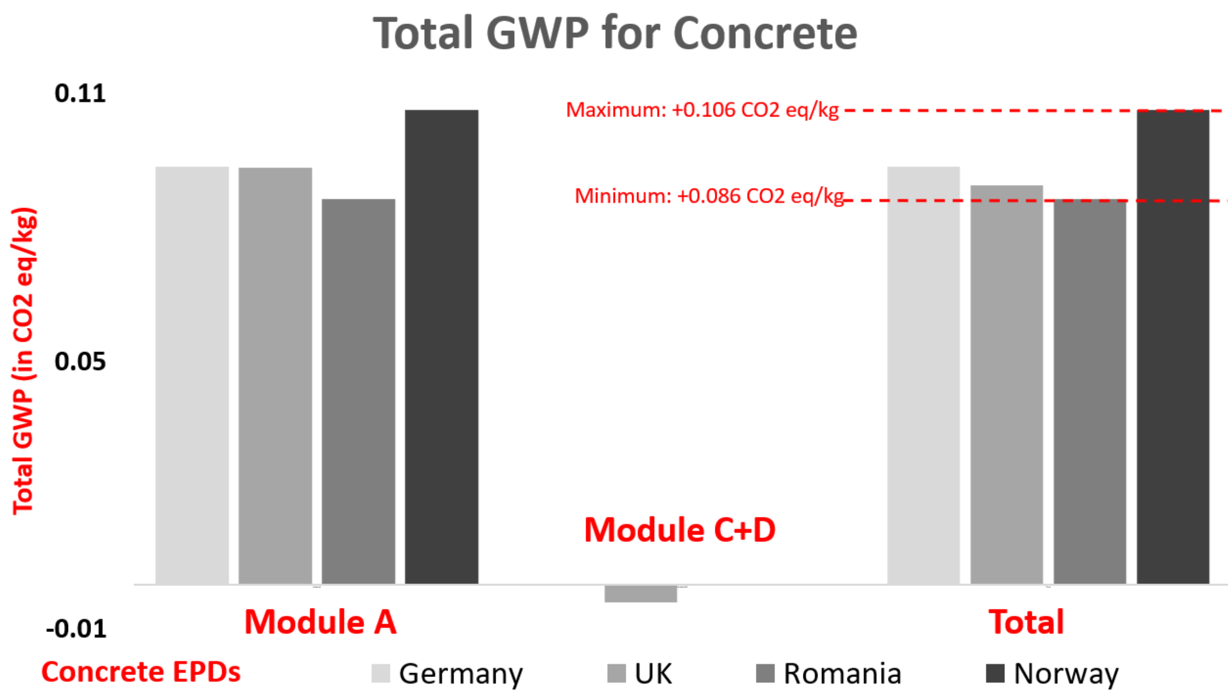

Figure E.1: Comparison of total GWP for concrete. (The EPDs can be identified based on geographical scope, as given in Table E.3a).

The details of the *EPDs* for hollow core slabs, and their *GWP* data are given in Table E.5. The declared units are different in each case, and the data is converted to that corresponding to 1 kg of material using the relevant conversion factor.

Table E.5a: Summary of EPDs of Hollow Core Slabs.

EPD Number	Producer	EPD Operator	Modules Not Declared	Region	Validity	Declared Unit	Conversion Factor (for 1 kg)
18	*CRH Concrete A/S* [138]	*EPD Danmark*	*A5*	*Denmark*	*2021 – 2026*	1 m^2	354
19	*VBI Consolis* [117]	*VBI Consolis*	*NA*	*Netherlands*	*2020 – 2025*	1 m^2	383
20	*Perdanga UAB* [139]	*EPD International*	*A5*	*Lithuania, Sweden and Norway*	*2021 – 2026*	1 Tonne	1000
21	*INHUS Prefab UAB* [141]	*EPD International*	*A5*	*Lithuania, Sweden and UK*	*2021 – 2026*	1 Tonne	1000
22	*Skandinaviska Byggelement* [142]	*EPD International*	*A5, Modules C and D.*	*Norway, Sweden and Finland*	*2019 – 2024*	1 Tonne	1000

For hollow core slabs also, recycling is the *EoL* scenario declared i.e., recycling of concrete to produce aggregates, as well as recycling of the reinforcing steel. Thus, very less credits are assigned for the *EoL* scenario, similar to the case of concrete. Figure E.2 below shows the comparison between different *EPDs* for hollow core slabs. It can be observed that in this case, there is some variation, owing to the difference in manufacturing procedures. Only *EPD#20* shows a much higher value. This can be partly explained due to the higher environmental impact for transportation. The average, maximum and minimum values ($+0.167, +0.185\ and\ + 0.147\ CO_2\ equivalents/kg$ respectively) of total *GWP* have been taken for further analysis. It should be noted here that the corresponding values are slightly higher than that of concrete. This will be more than compensated as less amount of material is required for hollow core slabs, owing to its optimised geometry.

Table E.5b: GWP Data for Hollow Core Slabs from EPDs.

Stage of LCA		Total GWP $[CO_2\ equivalents/kg]$					
		8	9	10	11	12	Average
Module A	*A1-A3*	*1.53E-01*	*1.47E-01*	*1.69E-01*	*1.55E-01*	*1.53E-01*	*1.55E-01*
	A4	*3.36E-03*	*4.39E-03*	*2.25E-02*	*9.11E-03*	*1.20E-02*	*1.03E-02*
	A5	*0.00E+00*	*4.10E-03*	*0.00E+00*	*0.00E+00*	*0.00E+00*	*8.20E-04*
	Total (A)	*1.56E-01*	*1.56E-01*	*1.92E-01*	*1.64E-01*	*1.65E-01*	*1.66E-01*
EoL: Recycling	*C1*	*5.42E-03*	*4.28E-04*	*0.00E+00*	*3.30E-03*	*0.00E+00*	*1.83E-03*
	C2	*2.94E-03*	*5.87E-03*	*0.00E+00*	*4.55E-03*	*0.00E+00*	*2.67E-03*
	C3	*2.99E-03*	*1.46E-03*	*0.00E+00*	*3.06E-03*	*0.00E+00*	*1.50E-03*
	C4	*2.20E-03*	*5.09E-05*	*0.00E+00*	*1.57E-03*	*0.00E+00*	*7.65E-04*
	D	*-2.24E-03*	*-1.65E-02*	*0.00E+00*	*-5.70E-03*	*0.00E+00*	*-4.88E-03*
	Total (A+C+D)	*1.67E-01*	*1.47E-01*	*1.92E-01*	*1.71E-01*	*1.65E-01*	*1.68E-01*

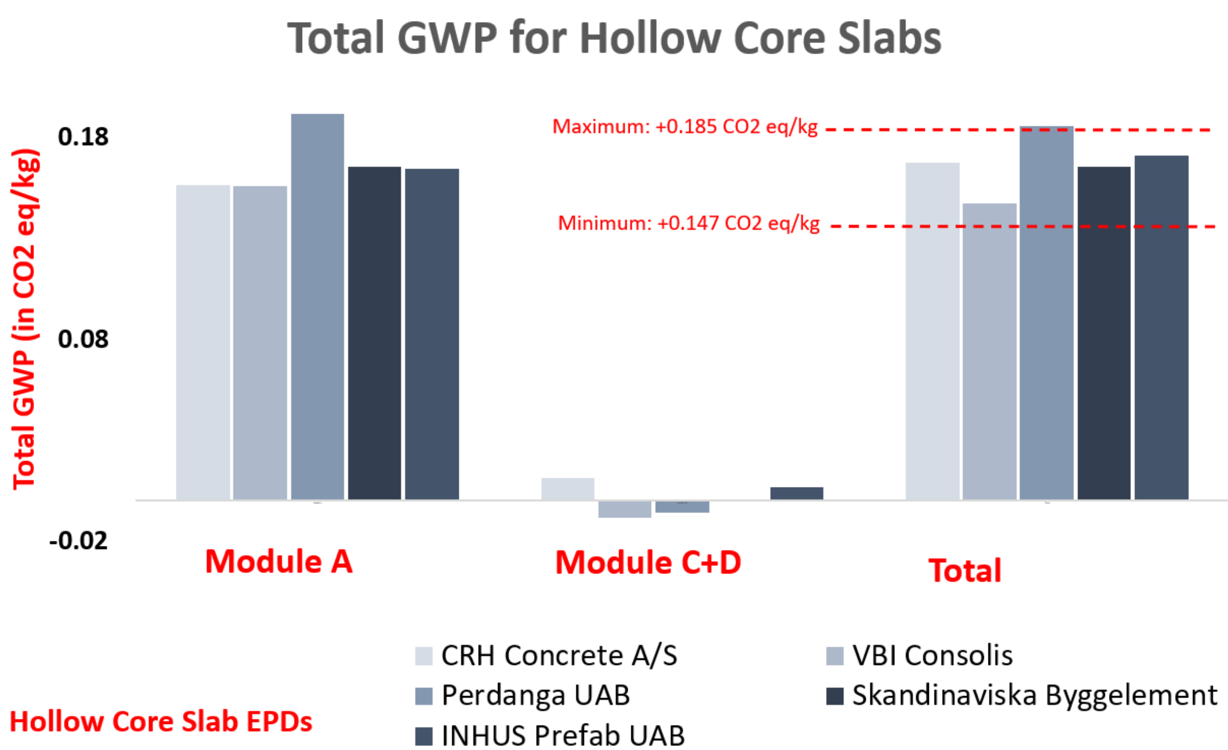

Figure E.2: Comparison of GWP for Hollow Core Slabs.

For the remaining materials, one *EPD* has been used corresponding to each. The composite deck and rebars are essential elements of the slab. However, their quantities are very small, and hence a sensitivity analysis of these elements would not result in a significant change in the total values. The details of the *EPDs* are given in Table E.6a and the *GWP* data is given in Table E.6c. All the *GWP* data has been converted into the corresponding values for 1 kg material, based on the declared unit and densities (Table E.6b). It should be noted here that the fill used for sound insulation (from Table A.7). is not considered here. This is basically gravel. These can be used, removed and reused as many times as possible, without any additions. The only environmental impact that will be reflected in a cradle to grave analysis will be its impact for transportation, which is minimal. Thus, the total *GWP* for gravel for this analysis can be assumed to be zero.

Table E.6a: Summary of EPDs of other Materials.

Other Products					
Products	**Producer**	**EPD Operator**	**Modules Not Declared**	**Region**	**Validity**
Chipboard [112]	*Fritz Egger GmbH*	*IBU*	*A4, A5*	*Europe*	*2021 – 2022*
Insulating Mineral Wool [113]	*FMI Association*	*IBU*	*C1, C3*	*Europe*	*2021 – 2026*
Gypsum [114]	*Knauf*	*IBU*	*A4, A5, C, D*	*Europe*	*2019 –2024*
Rebars [116]	*ArcelorMittal*	*IBU*	*A4, A5, C1, C2, C4*	*Europe*	*2016 –2022*
ComFlor60 [115]	*Tata Steel*	*Tata Steel UK*	*A4, A5, C1*	*UK*	*2021 –2025*

****EPDs according to EN 15804+A2.***

Table E.6b: Summary of EPDs for other Materials.

EPD	EPD Number	Declared Unit	Density $[kg/m^3]$
Chipboard	***23***	$1\ m^3$	*655*
Insulating Mineral Wool	***24***	$1\ m^3$	*120*
Gypsum Plasterboard	***25***	$1\ m^3, 12.5\ mm\ thickness$	**8.31*
Rebars	***26***	$1000\ kg$	*7850*
ComFlor60	***27***	$1\ m^2$	**11.2*

****Density in** kg/m^2*

Table E.6c: GWP Data for other Materials from EPDs.

Stage of LCA		Total GWP $[CO_2\ equivalents/kg]$				
		23	**24**	**25**	**26**	**27**
Module A	***A1-A3***	*-1.3E+00*	*1.4E+00*	*1.8E-01*	*1.2E+00*	*3.0E+00*
	A4	*0.0E+00*	*2.7E-02*	*0.0E+00*	*0.0E+00*	*0.0E+00*
	A5	*0.0E+00*	*1.8E-01*	*0.0E+00*	*0.0E+00*	*0.0E+00*
	Total (A)	*-1.3E+00*	*1.7E+00*	*1.8E-01*	*1.2E+00*	*3.0E+00*
Module C+D	***C1***	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*	*0.0E+00*
	C2	*3.2E-03*	*3.8E-03*	*0.0E+00*	*0.0E+00*	*2.2E-02*
	C3	*1.6E+00*	*0.0E+00*	*0.0E+00*	*4.3E-03*	*1.0E-02*
	C4	*0.0E+00*	*3.2E-02*	*0.0E+00*	*0.0E+00*	*2.3E-03*
	D	*-7.8E-01*	*-5.7E-02*	*0.0E+00*	*-1.8E-02*	*-1.3E+00*
	Total (A+C+D)	*-5.3E-01*	*1.6E+00*	*1.8E-01*	*1.2E+00*	*1.6E+00*

The total values of *GWP* for all the materials used are summarised below in Table E.7. For the main constituent elements (related to the slabs), the average, minimum and maximum values are given. The average values are used for further analysis. The maximum and minimum values are used for the sensitivity analysis. For all the remaining materials, only one value has been used.

Table E.7: Summary of total GWP for all materials. All values in CO_2 $equivalents/kg$.

	Main Materials	Average	Maximum	Minimum		Other Materials	Average
1	***Timber***	*-1.429*	*0.800*	*-1.589*	***5***	***Chipboard***	*-0.533*
2	***Steel***	*0.792*	*0.885*	*0.735*	***6***	***MW***	*1.632*
3	***Concrete***	*0.094*	*0.106*	*0.086*	***7***	***Gypsum***	*0.189*
4	***Hollow Core Slabs***	*0.167*	*0.185*	*0.147*	***8***	***ComFLor60***	*1.645*
					9	***Rebar***	*1.216*

E.2.2 Effect of Transport Distances on analysis of Timber

The data given in Table E.2b is with the assumed distances for the Modules related to transport (Module A4 and Module C2). The value representing the assumed transport distances is the average value for all 7 *EPDs* considered. For studying the effect of transport distances, it can be seen from Table E.2a that all *EPDs* except Swedish wood have provided some information related to the transport distances (either for Module A4 or for Module C2). Thus, for representing the actual transport distances, the average value for all these 6 *EPDs* have been considered. All the timber data considered below is with Storage as the *EoL* scenario, which is the one chosen for further analysis of the floor systems. Further, to exclude the effect of stored carbon in the analysis, the average value for biogenic carbon from all the 7 *EPDs* is taken. This was calculated to be $-1.58\ CO_2\ eq/kg$. This has been omitted from stages A1-A3, to which it was earlier assigned. Table E.8 below shows the *GWP* data for timber, obtained using the procedure explained above.

Table E.8: Data for Effect of Transport Distances on Timber.

Life Cycle Stages	$[CO_2\ eq/kg]$ Assumed Transport Distances	$[CO_2\ eq/kg]$ Actual Transport Distances
A1-A3 (Including Biogenic Carbon)	*-1.45E+00*	*-1.45E+00*
A1-A3 (Excluding Biogenic Carbon)	*1.31E-01*	*1.29E-01*
A4	*1.00E-02*	*1.78E-02*
A5	*1.44E-03*	*1.37E-03*
C1	*2.37E-04*	*2.32E-04*
C2	*7.92E-03*	*1.17E-02*
C3	*0.00E+00*	*0.00E+00*
C4	*0.00E+00*	*0.00E+00*
D	*0.00E+00*	*0.00E+00*
Total (A+C+D, Including Biogenic Carbon)	*-1.43E+00*	*-1.42E+00*
Total (A+C+D, Excluding Biogenic Carbon)	*1.50E-01*	*1.61E-01*

Using the total values across all life stages (including biogenic carbon), the total *GWP* for the functional equivalents is calculated, for the for ***DA1_STC***. This is given below in Table E.9.

Table E.9: Total GWP for DA1_STC (Effect of Transport Distances).

Total GWP for DA1_STC $[CO_2\ eq/m^2]$		
	Assumed Transport Distances	Actual Transport Distances
With Composite Action	-26.06	-26.16
Without Composite Action	-24.63	-24.73

E.2.3 Total GWP for Floor Systems

The total *GWP* for the different design alternatives is given below in Table E.10.

Table E.10: Total GWP for Floors Systems. a) For STC Design Alternatives (DA1_STC).

DA1_STC				
Materials			Quantity $[CO_2\ eq/m^2]$ With Composite Action	Quantity $[CO_2\ eq/m^2]$ No Composite Action
Floor	Slab	Timber	-52.5	-52.5
	Sound	Chipboard	-10.1	-10.1
	Insulation	Mineral Wool	7.8	7.8
	Fire Protection	Gypsum	5.1	5.1
Beams		Steel	20.8	22.2
Columns		Steel	2.8	2.8
		Total	-26.1	-24.6

b) For HCS Design Alternative (DA2_HCS) and

DA2_HCS			
Materials			Quantity $[CO_2\ eq/m^2]$
Floor	Slab	HCS260	64
	Floor Finish	Concrete	6.7
Beams		Steel	34.5
Columns		Steel	4.6
		Total	109.8

c) Composite Slab Design Alternative (DA3_CS)

DA3_CS			
Materials			Quantity $[CO_2\ eq/m^2]$
Floor	Composite Deck	ComFlor60	18.4
		Concrete	22.5
		Rebars	1.3
Beams		Steel	41.2
Columns		Steel	4.1
		Total	92.3

The above values have been calculated using the average values for all materials. Using the minimum and maximum values obtained from the sensitivity analysis, the upper and lower bounds for all the design variants have been computed. These are given below in Table E.11.

Table E.11: Sensitivity analysis for all design variants.

	Design Variant	$[CO_2\ eq/m^2]$ Average	Minimum	Maximum
1	DA1_STC (With Composite Action)	-26.06	-33.62	58.62
	DA1_STC (Without Composite Action)	-24.63	-32.3	60.21
2	DA2_HCS	109.86	98.8	122.38
3	DA3_CS	92.41	86.9	101.36