Timber top-ups on Reinforced Concrete structures to increase dwellings area

Master Thesis research

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Aknowledgemement

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Abstract

In recent years, the Netherlands has witnessed a growing housing shortage, prompting an ambitious plan by the government to address the demand through increased construction of new housing until 2050. Moreover, as part of environmental commitments, new dwellings and existing housing stock must align with current energy performance standards and the circular goals by the same target year. Hence, exploring alternative ways of increasing housing capacity is crucial to meet the growing demand while ensuring alignment with the environmental requirements.

This research examines timber top-up's structural feasibility and circular potential to expand existing dwellings' area as an alternative to a common practice: deconstruct followed by building new. Furthermore, provide an approach to the circular principle of Reuse to transition existing Reinforce Concrete structure in dwellings from a linear to a circular economy.

The present thesis consists of a literature review analysis of the potential of mass timber to top up, followed by the analysis of two local case studies that implemented this strategy. Then a dwelling with a relevant RC structure typology in the city of Deflt is selected as a case study to propose a conceptual modular structural design for a Timber Top-up system.

Three scenarios with different area capacities are proposed and tested to analyze the structural feasibility based on four structurally defined criteria (Reaction in the foundations, Utilization and reactions of the main RC structural components & timber components & deflection limit), followed by the analysis of the CO2 footprint of the Top-up scenarios. The tests are performed with the parametric structural analysis plugin Karamaba 3d, followed by the use of the software Granta Edupack with its feature Ecoaduti tool. The result showed that topping up using mass timber is an effective strategy to reduce the Upfront embodied carbon of existing RC structures while increasing the area capacity of the building.

01 Background
Introduction

1.1 Background

According to the Dutch National Agenda, the housing shortage in the Netherlands has been increasing in recent years. In 2020, 325,000 homes were needed to meet the current demand of residents, which is projected to increase to 420,000 by 2024 to continue growing over time (Ollongren, K.H., 2018). As a result, there is an urgent need to rapidly build new homes to address future demand, as the existing homes cannot accommodate the required space (Ollongren, K.H., 2018). Additionally, in the Netherlands, approximately 16,000 dwellings have been demolished annually over the past decade due to their failure to meet the needs of current residents and local governments regulations (Pardo Redondo, 2021), as depicted in Figure 1 Demolition of dwellings in the Netherlands from 2012 to 2020 (Pardo Redondo, 2021).

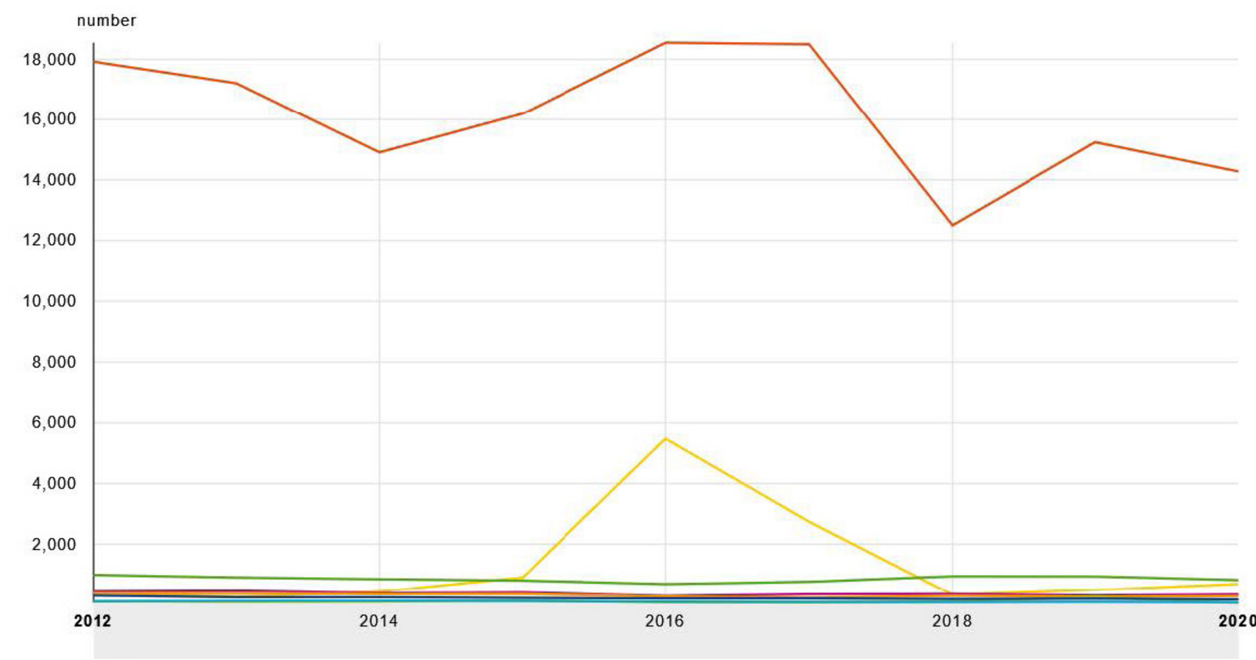

Figure 1. Demolition of dwellings in the Netherlands from 2012 to 2020 (Pardo Redondo, 2021). The orange line represents residential duwellings.

The Dutch Housing National Agenda emphasizes that more than simply accelerating construction projects in the medium term is needed. It is necessary to increase the planning capacity for housing in the most strained regions in the short term and initiate an adequate number of new plans (Ollongren, K.H., 2018). In certain areas, it is evident that the existing built-up areas will need more planning capacity in the long term to meet the substantial housing demand resulting from a growing population, as shown in Figure 2. Therefore, exploring alternatives to increase the planning capacity besides demolishing and building new is essential to face the demand for housing units in the short, medium, and long term.

On a global scale, it is anticipated that over two-thirds of the world's population will reside in urban areas by 2050, leading to a considerable increase in the demand for new housing. Consequently, constructing high-rise buildings in urbanized areas becomes a sensible approach to boost the production of construction materials and meet this increased demand (Žegarac Leskovar & Premrov, 2021).

Furthermore, due to the Paris Agreement's environmental commitments, most new dwellings plus the existing housing stock in the Netherlands must align with the current energy performance standards and the circular goals in 2050. This necessitates responsible utilization of natural resources and effective waste management, as the Construction industry is responsible for consuming over 32% of natural resources and generating a quarter of the global solid waste (Duarte, Maria, et al., 2020).

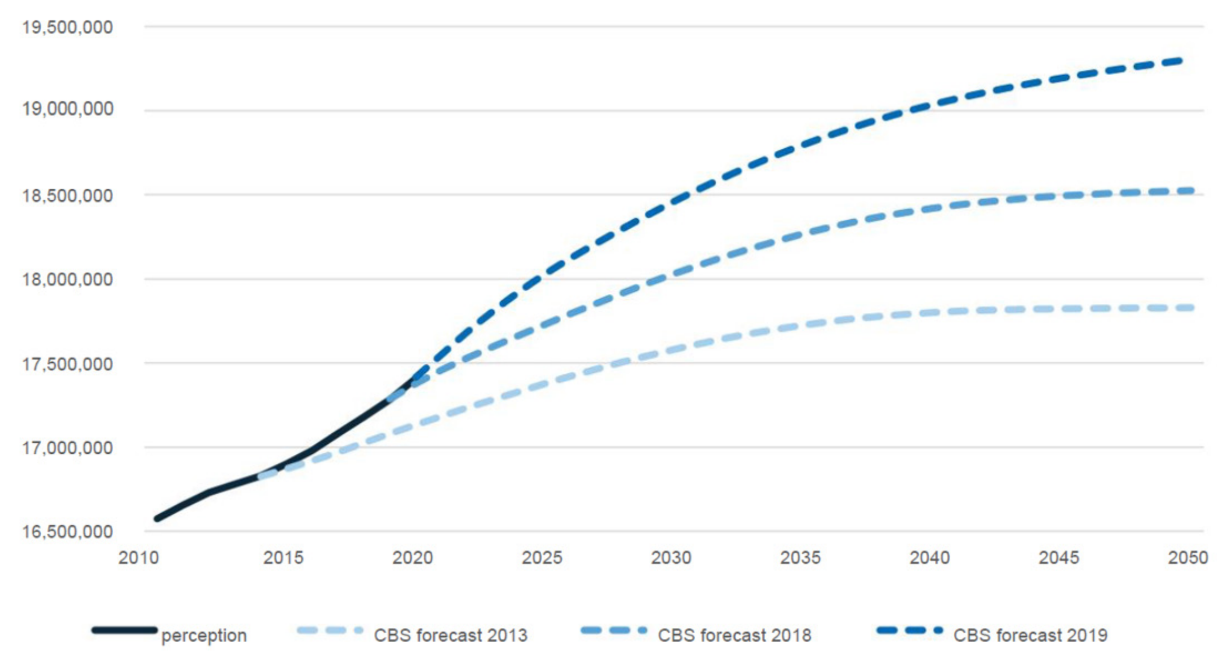

Figure 2. Projected population in the Netherlands in the coming year (PRIMOS 2020).

1.2 Problem statement

From an economic perspective, it is common worldwide to tear down existing buildings and construct new ones with larger capacity. However, this raises concerns about sustainability regarding using resources and managing waste. For example, in the Netherlands, most recently demolished buildings were constructed between 1900 and 1970, and many had Reinforced Concrete (RC) structures. Moreover, many of these buildings were still in their first life cycle (Pardo Redondo, 2021). Similarly, on a global scale, buildings designed to last 100 years or more are being demolished frequently, with an average replacement cycle of around 40 years in the UK and even shorter periods of approximately 20 years in China (Allwood, 2014).

Moreover, as mentioned by Duarte et al. (2020), It is important to note that a significant percentage (around 80%) of the building demolished nowadays have reinforced concrete structures, leading to a large amount of concrete waste. Meaning that the construction industry is not fully utilizing the energy embedded in the building represented by the materials and processes but also is contributing to the generation of excessive concrete waste.

Reduction of the amount of waste at the EoL

Duarte et al. (2020), in their literature review about the Circular economy in the construction industry, bring to attention the fact that reducing the amount of waste generated at the end of the life cycle of a building and managing the resources efficiently considerably reduces the negative impacts on the natural environment compared to other twenty sustainable practices studied in the paper. Therefore, reducing as a strategy might be relevant for RC in the construction of structures, as it is today's most widely used construction material for this purpose. It is estimated that roughly 25 billion tons of concrete are produced globally yearly, or over 3.8 tons per person yearly (WBCSD, 2009). Twice as much concrete is used worldwide as all the other construction materials added together.

Although The specific amount of harmful impacts in a concrete unit is relatively small (in comparison with other construction materials, as seen in Section 5.2.2). Due to the high global production and utilization of concrete, RC structures' final negative environmental impact is significant as it consumes a significant amount of natural resources that generate waste (Marinković et al., 2014).

In recent years, there have been a considerable amount of studies and efforts from academia and industries to research how to reduce RC waste when a building enters its End Of Life, as discussed in the following section.

Concrete recycling as material and as a component.

The final negative environmental impact of RC structures is significant as it consumes a considerable amount of natural resources that generate a large amount of waste.

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Although steel has one of the highest recycling rates compared to other structural material materials) its extraction from the RC matrix can be too costly, make it feasible

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In his research about recycling, Allwood (2014) found that demolished concrete can be recycled. However, it cannot be recycled back into its original constituent materials or complete original form. Moreover, for cement, ceramics, and composite materials, there has yet to be a recycling route by which the material could be returned to its original structure and quality. However, one of the main challenges is that recycling generally involves a loss of quality where it is currently impossible to control the content of recycled material with the same precision applied to virgin material.

If we analyze the composition of RC concrete one by one, we will see that cement cannot currently be recycled for several reasons, as Badraddin et al. (2021) mentioned. To mention some of them:

- Most cement (Portland cement in particular) is supplied as a dry powder, to which water is added to trigger a hydration reaction. This reaction cannot easily be reversed, requiring a considerable amount of energy compared to producing new cement from virgin materials. Moreover, cement is a one-way material, as Worrell & Reuter (2014) mentioned, as there are no commercial processes for cement recycling.

- Furthermore, recycled cement has a lower quality than cement produced from virgin materials (Badraddin et al., 2021)..

- And also today, plenty of natural resources exist worldwide to meet the demand for at least 20 to 10, as seen in Figure 9.

Continuing with the analysis, the steel present in the RC matrix faces a similar challenge as cement. On the one hand, at present is not possible to create the same quality of steel as the one produced from virgin materials, and also (although it has one of the highest recycling rates compared to other materials) its extraction can be too costly to make it feasible as mentioned by Allwood (2014).

Suppose we now analyze the recycling of RC from a component perspective. In that case, concrete can be crushed and transformed into an aggregate called Recycled Concrete Aggregate (RCA) to produce new concrete. However, it is essential to clarify that RCA's most common recycling practice is mainly used in unbound applications for road bases, backfilling, and road pavements, as in The Netherlands (Marinković et al., 2014). In some cases, RCA can be used as a partial or total replacement of coarse natural aggregate in new structural concrete with low-to-medium strength structural concrete, as Marinković et al. (2014) explain in their research. Therefore, instead of being recycled, in reality, it is downcycled, which is even lower in hierarchy in terms of End of life strategies, as seen in Figure 3

Property	RC with recycled aggragated 100%
Compressive strength	lower by 5–20%; commonly there is little effect below 30% replacement.
Tensile strength	Typically lower by 0–30%; commonly there is little effect below 50% replacement.
Modulus of elasticity	Typically reduced by 15–30%, although limit values reported are 5% and 45%; commonly there is little effect below 20% replacement,
Shrinkage	increasedtypically by 10–20%,
Creep	increased by 25–50%,
Water absorption	increased typically by 40–50%,
Freezing and thawing resistance	Same or slightly decreased (up to 10%)

Table 1. Properties of concrete made of 100% RCA. One of the most significant research about RCA, according to (Marinković et al., 2014). Conducted by Pryce-Jenkins (2011)

Moreover, crushed concrete requires a considerable amount of energy to be processed compared to mined aggregates. In turn, this may drive up requirements where concrete might have a higher embodied carbon than the one made of regular aggregates, worsening the overall impacts. Therefore, according to Worrell & Reuter (2014), no evidence that recycling concrete in this form substantially reduces emissions.

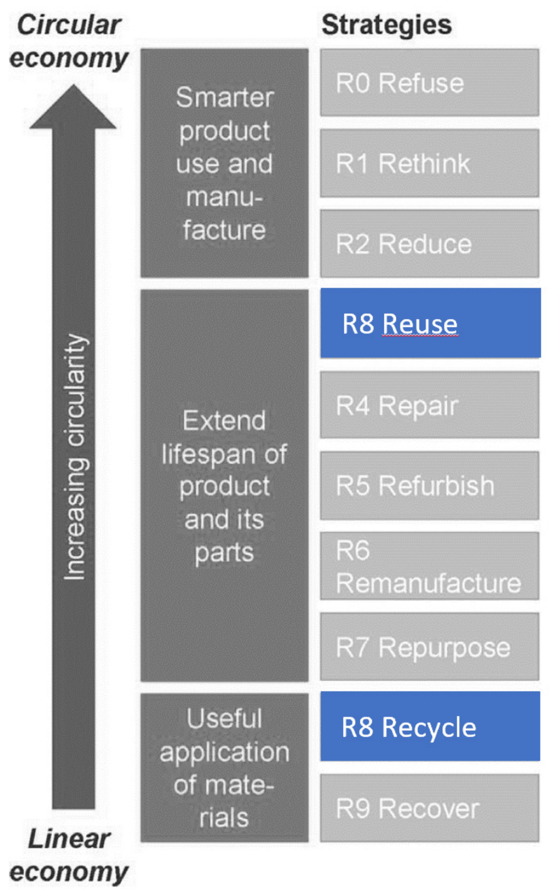

Figure 3. Research about the Lifecycle of RC has been studying the possibilities to reduce concrete waste primarily by reusing, recycling, and Downcycling. The 10 "R" of sustainability framework for transitioning to Circular Economy by Potting et al. (2017) CE strategies.

Furthermore, In their literature review, Marinković et al. (2014), after studying and comparing 103 studies about experimental research on the structural performance of RCA from the last ten years, found that the physical properties are significantly reduced compared to RC made of virgin aggregates as can be seen in Table 1.

Although there has been significant research on recycling concrete, it is essential to clarify that recycling is still one of the lowest strategies in transitioning from a linear to a circular economy compared to Reducing, Reusing, or Refurbishing components and materials. Therefore these strategies might have a more significant potential to reduce unwanted environmental impacts. (Allwood, 2014). Moreover, as Worrell & Reuter (2014) mentioned: "Successful recycling can reduce the demand for new ore or biomass, but successful material efficiency reduces the total demand for material processing."

Furthermore, as seen in Figure 3, a wider set of strategies with a more significant potential impact across all material classes can be found under the umbrella term "material efficiency," which describes the aim to deliver material services with less material input. As discussed in the next section, RC concrete components might be repurposed or remanufactured instead of recycled after demolition.

Repurpose and remanufacture concrete components by deconstruction.

On a component scale, there have been successful attempts to repurpose and reuse concrete elements by conferring them the capacity to be disassembled and reused. For instance, the Finnish company, Peikko, produces extensive bolted mechanical steel connections for concrete elements enabling disassembly for reuse in subsequent buildings, as seen in Image 1 . Thereby, as mentioned by Eberhardt et al. (2019) to prolong the elements' service life by Reusing, and avoiding environmentally burdensome production of new concrete elements.

The principle behind the Reuse of concrete lies in conferring "dry" mechanical joints between prefabricated modules that can be reversed, making the modules capable of being reused. Furthermore, opening the possibilities for RC components at their End of Life (EoL), as mentioned by Allwood (2014), because not only can the components be reused, but these alternative offers concrete companies an attractive new business model that might transition RC from linear to circular Economy: Rather than selling a low-margin powder to constructors, they can sell or lease structural modules whose value can be maintained over several generations of buildings.

"Disassembly Potential 2.0" focuses on evaluating through a series of steps, the disassembly potential of a building based on the ease with which its components can be disassembled to be reused in a following cycle.

On the other hand, the report "Disassembly Potential 2.0" by Van Vliet et al. (2021) proposes a practical method to assess the potential for disassembling components in buildings quantitatively. The report primarily focuses on evaluating, through a series of steps, the disassembly potential of a building based on the ease with which its components can be disassembled to be reused in the following cycle.

Furthermore, as will be discussed in Section 3.1 when it comes to RC structures, this quantitative method analyzes the parameters of existing components, providing stakeholders with the opportunity to assess the feasibility of reusing concrete instead of opting for demolition.

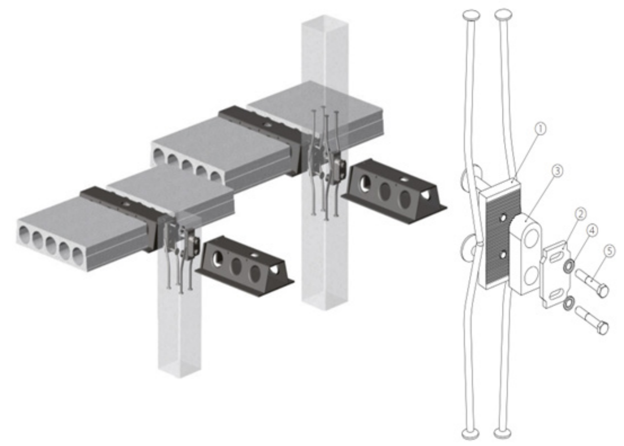

Image 1. Detail of the system PCs® and PCs® UP system with DELTABEAM® by Peikko. Example of a dry connection to conferee assembly and disassembly properties to aerated prefab RC slabs.

Despite the benefits of Reuse, there are still limitations, as Coelho & De Brito (2013) studied in their research about conventional demolition versus deconstruction techniques. This comparative study of traditional demolition versus deconstruction of a standard 100 m2 housing block in different countries was conducted, where it is explained that this strategy faces various challenges compared to conventional demolishing and recycling of RC, such as:

- The lack of experience in deconstruction techniques and the need to implement specific tools that may not always be available.
- The high costs of dismounting compared to demolishing, as low rates for Reinforce Concrete Waste landfilling favors traditional demolition.
- The time it takes to execute dismountability, which can be harder to estimate in terms of cost.

Nevertheless, it was concluded that deconstruction might be a competitive alternative in those cases when local landfill disposal fees were higher than 30 euro/ton, which is currently the case in Germany (213 euro/ton), Canada (64 euro/ton), and Sweden (30 euro/ton).

1.3 Hypothesis

As discussed in the previous section, several circularity strategies have been studied to reduce the consumption of natural resources and minimize waste products in the construction of buildings. Moreover, the framework proposed by Potting et al. (2017) arranges all of them by order of importance and according to different aims to specific objectives, as seen in Figure 3. In this framework, commonly known as "The ten R's of sustainability," are analyzed all of the previously mentioned strategies contribute to the transitioning of products from a linear to a circular economy, in this particular case, the transitioning of buildings. This figure shows that strategies such as Reuse are positioned in a higher hierarchy than recycling. According to this framework, the Lifetime extension of a product or system has a higher impact than just recycling materials through recovery processes.

Furthermore, when examining the literature on the reusability of RC structures, it becomes apparent that most research efforts primarily concentrate on retrofitting methods to prolong the lifespan of RC structures and bring them in line with local structural codes, as discussed by Allwood (2014). However, other scenarios, such as expanding the building's capacity to meet current demand, need more attention in these studies.

Hence, it raises the question: What if it would be possible to increase the number of square meters by using the same amount of RC existing components?, Expanding an existing building's area without compromising its structural load-bearing capacity possible?

The focus of this research aims to analyze an approach to using as much as possible the existing components of RC structures without deconstructing them, not only to extend structures' lifespan through Reuse, as seen in Figure 3 but the possibility to upcycle the structure by increasing its capacity in the area.

Timber as an alternative to RC

Ramage et al. (2017) conducted research highlighting the comparable strength parallel to grain between mass timber components and reinforced concrete (RC). Although mass timber may not match the pure compression strength of RC, its low density presents a competitive advantage in specific scenarios, even when compared to other conventional structural materials, as shown in Chart 1.

Furthermore, in cases where components are subjected to tension and compression, the strength-to-weight or elastic modulus-to-weight ratio of mass timber makes it a suitable choice for structural design.

Chart 1 demonstrates the strength-to-weight and modulus-to-weight ratios of steel, timber, and reinforced concrete, indicating that certain softwood variants perform similarly to steel. Consequently, the potential for timber to possess a higher strength-to-weight ratio than RC implies the possibility of substituting RC components with timber in specific scenarios within existing building structures. Such a substitution would allow for an increase in structural load and expansion of the area without a corresponding increase in mass.

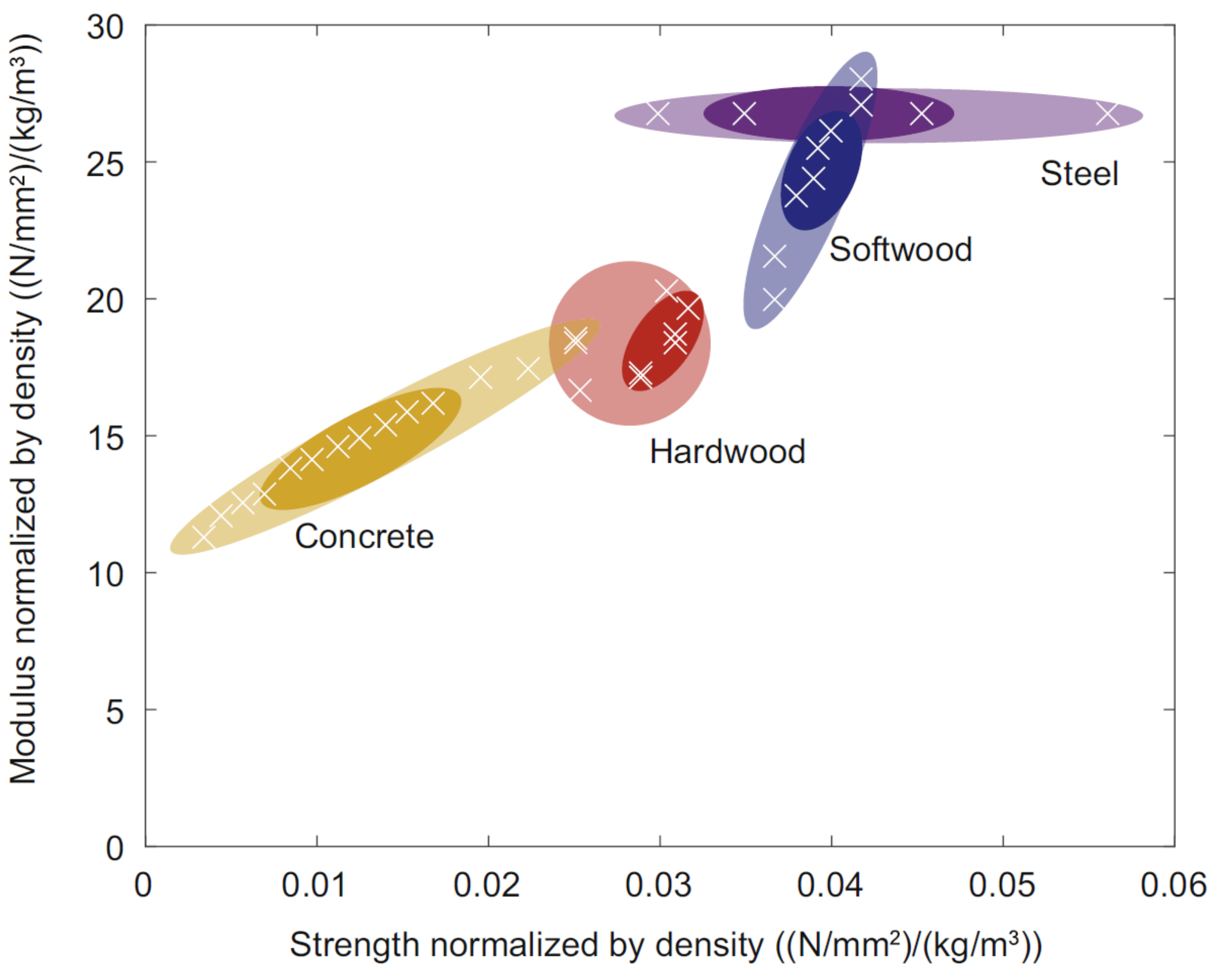

Chart 1. Density and Compression strength for different materials, according to the relevant design standards. (Michael H.Ramage & Henry Burridge, 2017)

What if would be possible to increase the amount of square meters in the buildings with the same amount of materials present in the RC existing structural components?

It is possible to expand an existing building's area without compromising its structural load-bearing capacity?

Therefore, this graduation project aims to investigate the feasibility and limitations of employing timber structures as top-up additions to existing RC buildings to extend their capacity and face the problem described in section 1.2.

By incorporating a Timber top-up to expand the area of an existing RC building, not only can the available space is increased with the integration of new timber components, but also the lifespan of the structure can be extended as it can accommodate additional functionalities due to changing needs, as can be seen in Figure 4.

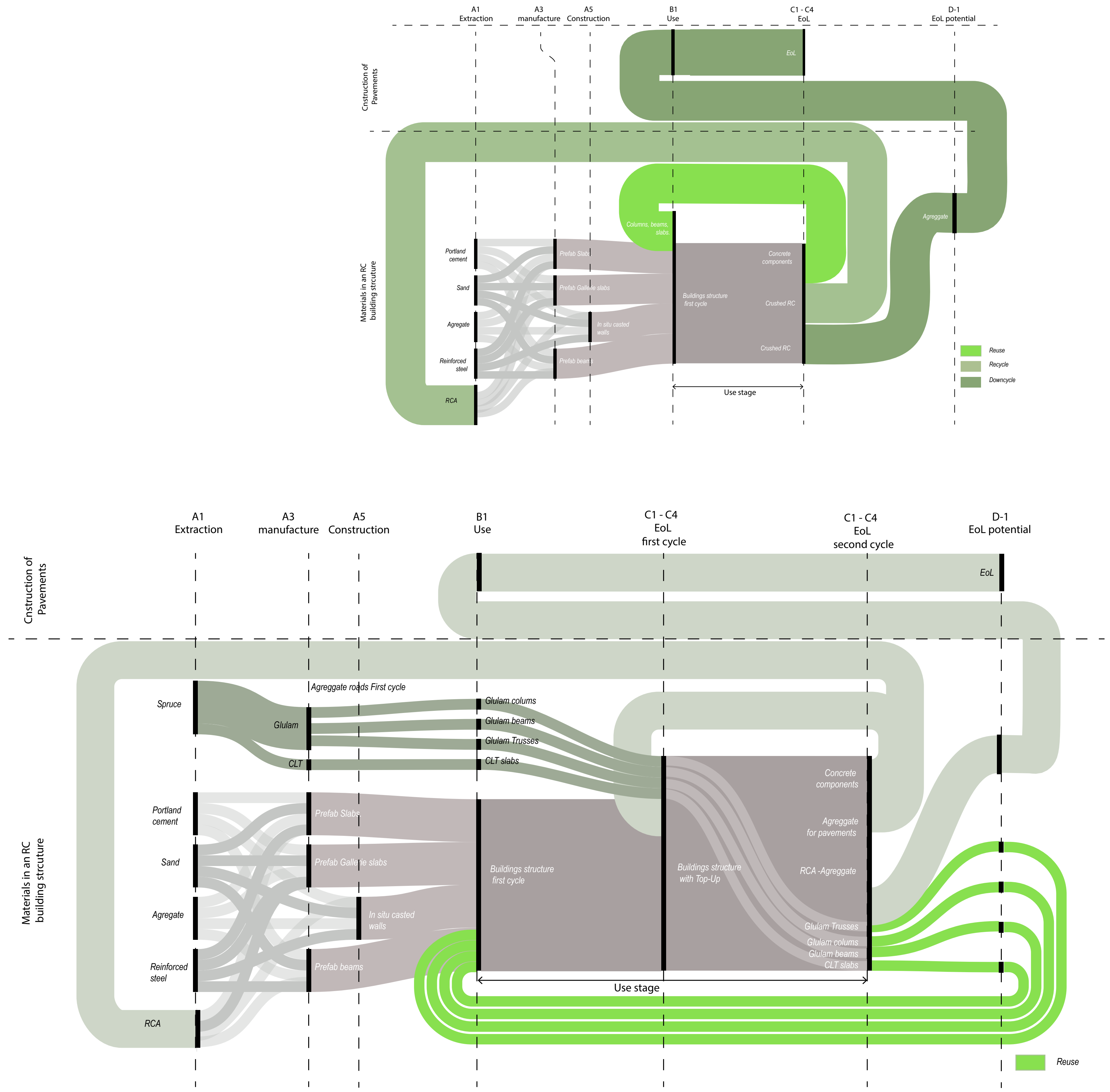

Figure 4. Top: Material flow chart of the materials in an RC structure for a building with different potential End Of Life Scenarios. Bellow: The same situation was described above by adding timber Top-Ups to prolong the lifespan of the RC structure.

Moreover, the project aims to analyze this strategy compared to the scenario where a new building with a higher capacity replaces the existing one. Hence, to offer another alternative to the deck of strategies that might be applied when transitioning an existing building from a linear to a circular economy.

1.4 Research question

How can the area capacity of existing Reinforced Concrete structures be increased and extend their lifespan by using timber top-ups?

1.5 Sub-questions

To address the main research question, the study proposes the following sub-questions. These subquestions serve as a navigational framework within the report's structure. Each subquestion is followed by a description of how it will be addressed in the report:

- **SQ. 1 |** Which structural timber systems can be more effective when topping up existing Reinforced Concrete Structure (RCS) buildings? (literature review: Section 3: Timber: As a material to create top-up structures.).
- **SQ. 2 |** What opportunities and limitations have topping up an existing RC structure building with timber?W (Section 2: Topping Up existing buildings & Section 5: Research through Design).
- **SQ. 3 |** How to make visible topping up in the decision-making for stakeholders when facing demolishing/building new? (Section 5.1: Research through Design).
- **SQ. 4 |** To what extent can the capacity of an existing RC building with timber be increased by reducing the weight of the RC structure? (Section 5.2: Research for Design).

1.6 Approach and Methodology

To answer these questions, the following methodology is proposed, which is structured in five stages: Introduction, Theoretical Framework, Research for Design, Experiment, and Results & Conclusions. Each part consists of sections that will delve into the specific topics of the report, as seen in Figure 5.

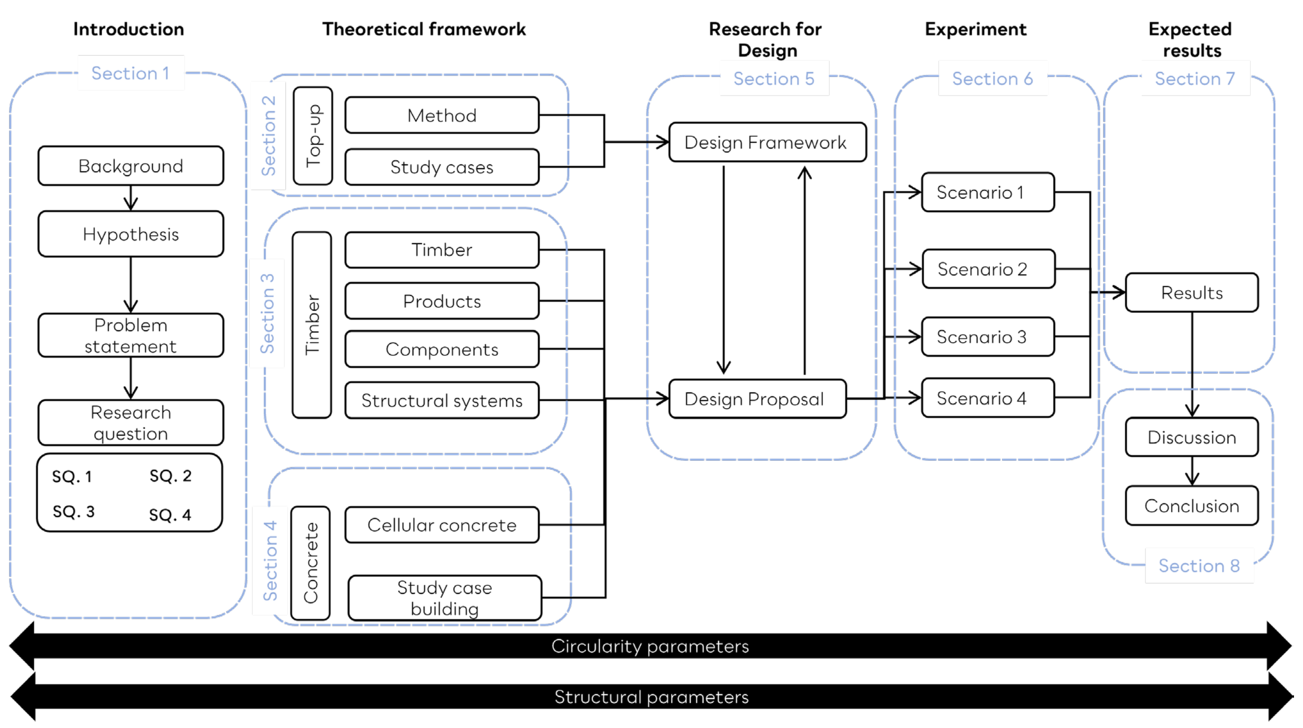

Figure 5. Structure for the methodology of the graduation project. Own elaboration.

The introduction will serve to introduce the topic and provide the necessary context for understanding the research on timber top-ups as has been described in Section 1.

Then a theoretical framework will be developed through a literature review, focusing on relevant local case studies of top-ups developed in the Netherlands. These explained case studies in Section 2 would establish a context for the research question and sub-questions.

Subsequently, Section 3 will analyze the characteristics and properties of timber as a material and its relevance in designing Timber top-ups for RC structures. The technical literature on timber's mechanical and physical properties will be examined, along with an analysis of its environmental impacts on the structural design.

The findings will be organized and analyzed based on different composition scales, starting with timber as a structural material, followed by timber as a product, and then moving on to timber components. The analysis will conclude by exploring timber components in creating timber structural systems according to the European structural codes: Eurocode 0 (EC0) and 5 (EC5).

Following the literature review, Section 4 is proposed a relevant case study of an RC building in the Netherlands with the potential for being topped up for further analysis.

After understanding timber's potential as a structural material for topping up and selecting a Case study, Section 5: "research for design," aims to develop two products. First, an approach for a framework in the decision-making process for Timber Top-ups (Section 5.1), and second, the conceptual modular design of a Top-up timber system for existing RC structures based on the design framework in Section 5.2.

Moreover, in Section 5.2.4, three scenarios with different area capacities will be proposed: Top-up without deconstructing any RC component (5RC+1T), Top-up by removing one layer and adding three layers (4RC+3T), andTop-up by removing one layer and adding four layers (4RC+4T).

Subsequently, the experimental part of the project in Section 6 aims to test relevant structural criteria (bending shear strength and deflection) to understand the preliminary structural feasibility of the top-up scenarios. Additionally, an analysis and comparison of the upfront carbon emissions of each scenario will be conducted, including a theoretical scenario where the entire structure is demolished and replaced with a new one of similar capacity.

A parametric simulation model of the case study and the proposed scenarios will be created using Rhino 7, Grasshopper, and Karamba3D software to assess the structural feasibility results. The analysis will serve two objectives: firstly, to determine the extent to which it is structurally feasible to increase the capacity of the study case by reducing the weight of the RC structure and topping it up with timber, and secondly, to evaluate the CO2 footprint of topping up with timber compared to demolishing and building anew.

To assess the upfront carbon emissions analysis, the CO2 footprint resulting from two cycles will be calculated: the existing building before the Top-up intervention and the building after the Top-up. The volumes of both RC and Mass timber will also be estimated. These analyses will be conducted using Rhino 7, Grasshopper, Ansys Granta Edupack, and Ecoaudit Tool software.

Finally, the results will be plotted and analyzed in Section 7, followed by the discussion.

1.7 Scope and limitations

As explained in Section 2.2, topping up an existing building is a transdisciplinary approach. Therefore, it is essential to acknowledge that this research on the structural feasibility and CO2 footprint of increasing the capacity of a building with timber is just one aspect among many parameters from different fields that should be considered. Future research should explore opportunities and limitations from other relevant fields to provide a more comprehensive analysis.

This graduation studio is focused on the Building Technology field, specifically within the graduation studio of Structural Design for Change, meaning that the project will analyze the structural constraints, possibilities, and feasibility of topping up a specific case study in the Netherlands. Moreover, the research examines timber building top-ups as a circular strategy within the broader context of transforming an existing building's structure to transform using circular design principles.

It is important to note that this research will exclusively focus on analyzing the building's RC (Reinforced Concrete) structure. Further research will be necessary to analyze the implementation of top-ups about other shearing layers, such as façades, partitions, and services. I am topping up existing buildings.

Theoretical framework

Introduction to top-ups 02

2.1 Opportunities in topping up.

2.2 Criteria to be considered when topping up.

2.3 Analysis of study cases.

With an ever-reduced building space worldwide, we have seen many examples in the recent past of densification. In all the cases, different technical approaches have been considered, in some cases, as can be seen in Figure 6, leading to unconventional solutions to preserve as much as possible what is constructed below.

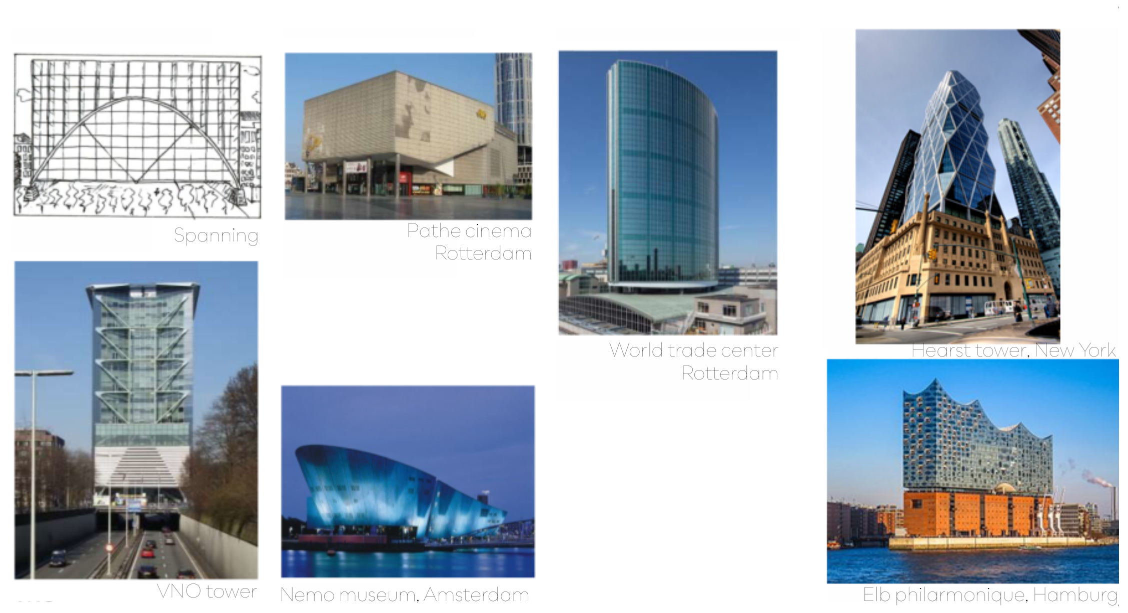

Figure 6.Different approaches to topping up worldwide. Taken from (Hermens et al., 2014) :

Broadgate: A building is constructed around a park to preserve it. VNO: A building is constructed around a highway to avoid relocating it. The NEMO museum in Amsterdam is a specific example of topping up, as it is built on top of a tunnel. The resulting forces in the piles change from tension to compression, allowing the walls, which were previously laterally loaded, to bear the additional weight in compression. No other foundation was necessary.

The Pathé cinema in Rotterdam is constructed above an underground parking garage. The heavy finishing of the original square is removed and replaced with a lightweight structure. A steel and composite structure is utilized to create the cinema theaters. No other foundation was needed.

Through puncture: The Hearst Tower in Manhattan and the Elbe Philharmonic in Hamburg demonstrate how densification can be achieved while preserving relevant construction elements by creating separate structures to support the new construction.

From a circularity perspective, densifying and using as many as possible of the resources that the current building holds also represents a significant benefit from a waste-reduction point of view. Reusing components will always be positioned on a higher hierarchy regarding circular economy transition strategies, as seen in Figure 3 on page 11 (Potting et al., 2017).

In the case of RC structures, The benefits of densification align with the potential opportunities that topping up with timber can offer. As further explained in Section 3.1, Timber construction is known for its lower environmental impact than traditional construction methods, as it has a smaller carbon footprint than other construction materials. Therefore, the combination of densification benefits and the sustainability potential of timber Top-ups in existing construction is a promising alternative to explore in transforming existing buildings to follow circular principles.

2.1 Opportunities in top-up

Topping up with timber instead of demolishing offers various benefits beyond environmental considerations. We can identify additional advantages, as can be seen in Diagram 1 such as:

Social-economic: As mentioned in Section 1, there is an opportunity to top up to address the current housing shortages. However, as analyzed by the TUDelft SUM Team (2022) for the Solar Dechatlon 21/22, in social housing dwellings, increasing the building's area capacity might also lead to creating communal spaces for residents with a cultural and social division to improve their living conditions. Moreover, diversifying the resident group with new economic target groups will allow for more

> Sometimes existing buildings and infrastructure are being demolished, some buildings are built over existing city fabric, and some are built through existing buildings.
>
> In all the cases, these densification strategies have arisen to respond to society's interests for existing buildings facing loss in functionality, damaged structures, and the need to preserve historical and cultural aspects of valuable resources.
>
> Hermes et al.(2014)

> *Increasing the area of an existing building presents an economic incentive to renovate out-of-date buildings. It might provide the necessary economic leverage to execute energy performance improvements.*

harmonic living in places with the lowest socio-economical rates.

Urban Impact: Enabling the building to have multiple uses positively affects the surrounding urban environment. It increases urban flows and creates landmarks within cities, enhancing the overall urban fabric.

Economic: For building owners in dwellings, increasing the area of an existing building presents an economic incentive to renovate out-to-date buildings. This incentive becomes particularly relevant for post-war tenement buildings with outdated facades and services. It provides the necessary economic leverage to transform these buildings towards being zero-energy or, at the very least, improving their energy performance.

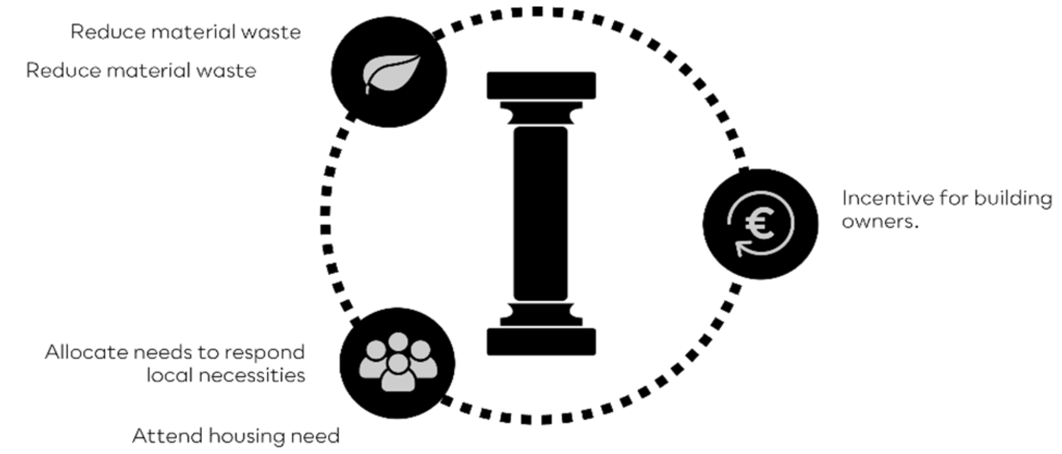

Diagram 1 Analyzing Topping up through the lens of the three pillars of sustainability (also known as "The three Es of economy framework").

Moreover, this economic benefit significantly impacts residents in tenement buildings by allowing them to enhance their living conditions. By improving the performance of systems and facades within the building, they can experience better comfort and energy efficiency without being displaced from their homes. Additionally, most of the construction work takes place on the roof of the existing building, minimizing disruptions to their living spaces.

2.2 Criteria to be considered when topping up.

In their paper, Verburg & Barendsz (2000) analyze the utilization of steel framing structures for top-ups in existing buildings. Although the research topic differs in terms of materiality, the considerations and criteria presented in the research can still be deemed pertinent to the study, as they can be applicable when proposing an approach to address the research question in the graduation project.

This research (Verburg & Barendsz, 2000) analyzes the essential criteria in The Netherlands to consider when proposing a steel top-up in an existing building; the studied criteria are:

Existing Building's lifetime

Pertinent to the research question, before planning to add new levels to an existing building is important to assess the building's expected lifetime carefully. Typically, topping-up is carried out in conjunction with other interventions, such as deep retrofitting, to extend the building's lifespan.

Fitting with the urban plan

Furthermore, if necessary, the intervention should be able to accommodate the new requirements resulting from increasing the area. This includes considerations such as providing sufficient parking space and storage facilities and ensuring proper access to the new spaces. Additionally, it is crucial to assess the potential impacts on the surrounding environment. For example, analyses such as shading assessments are necessary to determine if the increased building volume would unexpectedly block sunlight in certain areas.

Financial feasibility

In terms of costs, adding extra levels can represent a considerable investment, especially for unforeseen interventions and unconventional building methods. In that sense, some buildings are better suited to add extra levels. Therefore, a financial assessment should include costs per unit (old and new), subsidies, and 'land use' costs to validate its feasibility.

Structural feasibility

Described as technical feasibility in this document, this criteria involves assessing the foundation and building structure to determine whether it can support additional levels or if improvements and reinforcements are required before adding extra weight to the building. As a rule of thumb, an additional 12% of the total building mass can be added to existing buildings.

Co-operation from the municipality

Its relevant from a local perspective, the support of the municipality and approval of the project, therefore the vision of the municipality for the neighborhood plays a significant role as increasing height might change land use and costs.

Co-operation from the inhabitants.

A successful plan to top up is only possible if the inhabitants in and around the building agree with the addition and the nuisance it will cause during construction. They can be convinced by possible benefits like an added elevator, renovation of the entire building (lower energy costs), or some expenses for the nuisance.

Architectural conditions and regulations of the top-up

The added volume should complement the overall appearance of the neighborhood. It is important to consider practical options for accessing the new levels. Additionally, it should be determined whether the existing building has storage space that can be repurposed to accommodate the additional dwellings. In the case of lightweight structures, meeting fire regulations can be challenging, as Emergency staircases must be added to the intervention to comply with the code.

Overall, taking an interdisciplinary approach is crucial for successfully implementing a top-up intervention in an existing building. This approach requires analyzing relevant considerations from various fields of knowledge. In the context of this graduation studio: Structural Design for Change, the research will primarily investigate timber top-ups' structural and circular feasibility. However, these explained criteria would be considered when approaching the design of a Decision-making framework for timber Top-ups in section 5.1. It is essential to consider these criteria extensively, as feasibility relies on technical and environmental aspects and the broader considerations outlined.

2.3 Analysis of study cases

As seen in Figure 6, various projects worldwide have implemented different approaches to increase the capacity of existing buildings. Each of these approaches responded to tailored solutions to meet the specific requirements of the context and the interests of the people around each project. To mention some of these requirements: Increasing capacity, avoiding obsolescence through renovation, activating urban areas by creating landmarks, and renovating heritage constructions. Consequently, the strategy of topping up encompasses a wide range of approaches in which the possibilities of structural systems, materials, shapes, textures, and intensities of the increase might be unlimited. Moreover, some projects even involve independent structural foundations and separate structures attached to the existing ones.

Given that the primary objective of this project is to transform an existing building using circular principles, it is imperative to prioritize waste reduction as

Taking an interdisciplinary approach is crucial for the successful implementation of a Top-up intervention in an existing building. This approach requires analyzing relevant considerations from various fields of knowledge.

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a hard constraint. Therefore, the case study analysis will focus on case studies where the existing structure is made of RC utilized to the maximum extent in the new intervention. Moreover, cases where topping up was achieved by utilizing the existing foundation of the building. Since the problem statement is based in The Netherlands, this will define the search for buildings built within this country.

After an extensive search, two well-documented case studies were selected for the analysis: The Karel Doorman building in Rotterdam and the intervention by the SUM team for the Solar Decathlon 2021, which will be described in Sections 2.3.1 & 2.3.2.

2.3.1 Case study 1: De Karel Doorman Building.

Figure 8 . Top: Photo taken in 1946 of the Teruel Building. Bottom: photo of the De Kareel building today.

The information presented in this section is derived from two primary sources. The first source is an interview conducted with architect Mark Van Tilburg, who represents Ibelings Van Tilburg Architecten, the firm responsible for designing and coordinating the intervention in the case study. The second source is the research conducted by Hermens et al. (2014) along with the graduation thesis by Cobelens (Cobelens, 2018) in his graduation Thesis of Civil Engineering.

During World War II, the city center of Rotterdam was almost destroyed. In the following years, during the city reconstruction, The Ter Meulen building was built between the years 1948 and 1951. Designed by the Dutch architect Van den Broek & Bakema, it symbolized the city's reconstruction after the war.

As can be seen in the top image in Figure 8, The main characteristic of this project lies in the strategy used back then to allow future vertical expansion, where the 2nd floor (dedicated in principle to housing offices and a canteen) was conceived during the structural design of the pile foundation to hold an expansion for a future salesroom. In that case, the offices and canteen would be replaced to a new-to-be-built 3rd floor. In addition, the design comprised an open floor plan made possible by a structural system of columns and beams that provide lateral stability avoiding structural walls. Therefore, ultra-light-weight building concepts were implemented, allowing the building to have a five times higher density than similar projects back then, making vertical expansion possible.

In the late 70s, two extra floors were placed on the original building. This was possible by using relatively lightweight floors. However, during the 90s, the retail market changed, and the building deteriorated. In the 00s, an initiative to reactivate the building arose where heritage characteristics played a significant role. After analyzing the existing structure, The architects and engineers concluded that there was potential in the over-dimensioned RC structure of this building to add 16 new lightweight stories with apartments truly on top of the existing building. Using the existing load-bearing system of columns and pile foundation allowed it to keep the original building.

For the new top-up structure, the existing columns and beams were suitable to bare the new loads and provide the lateral stability required through rigid frame action. With a column grid layout of 8 x 10 m and column sections similar in almost all the floors (around 850 mm in the basement and 800 on the 2nd floor) was determined that the compression strength of the columns was 250 kg/cm2, which can be compared to a concrete garde C14/17 strength according to Eurocode 0 (EC 0). The beams were suitable as they had a 600 x 850 mm dimension, capable of bearing a compression strength of 200 kg/cm2. Regarding the health of the concrete, Inspections indicated that the quality of the construction was excellent. After visual and destructive testing of the concrete, it was determined that its strength was around 40,9 N/mm2.

Moreover. the original foundation was designed with reinforced prefabricated concrete piles, with a shaft dimension of square 380 mm and a + shaped pile tip of 760 mm. Making foundation loadbearing capacity much larger than the originally intended 70 tons (or 900 kN according to present codes).

Two strategies were considered: First, Propose a lightweight structure composed of steel and timber to achieve floors that could weigh 1/5th of the weight of standard Dutch codes). And second, to create stability by building two stability cores.

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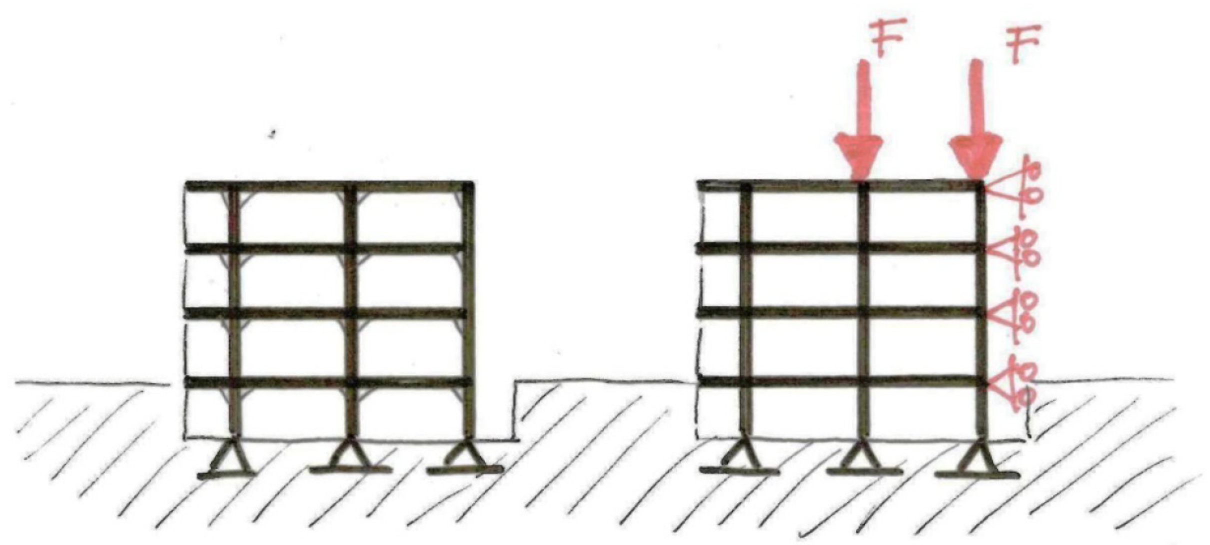

Image 2 Structural stability scheme before and after the intervention Structural stability scheme before and after the intervention.

Timber was chosen as the primary structural material because it was a bio-based renewable resource that encompasses low embodied energy and because of its carbon sequestration and workability.

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The innovative solution to the challenge of adding 16 stories to the existing building lay in separating the horizontal loads derived from the wind pressure that might affect the Top up from the vertical loads (Dead and live loads). Therefore, two strategies were considered: First, Propose a lightweight structure composed of steel and timber to achieve floors that could weigh only 250 kg/m2 (1/5th of the weight of traditional Dutch concrete apartment buildings.). Moreover, to create stability by building two concrete stability cores that might be used for staircases, elevators, and service ducts.

As a result, by using these two strategies, the structural load-bearing system changed from a system with a rigid frame action, with bending moments in the beams and columns caused by horizontal loads, to a system with supported columns, only having to carry vertical loads as can be seen in Image 2. The new building was named then “De Karel Doorman” and demonstrated that it is possible to achieve urban densification by building on top of existing buildings or infrastructure. Understanding the hidden load-bearing capacities of existing structures gives excellent possibilities to add a significant amount of apartments or other functions in city centers.

2.3.2 Solar Decathlon Europe competition 21/22 SUM team

For the following Study case analysis, report # 7 by the TUDelft's SUM Team (TUDelft SUM Team, 2022). Moreover, Section 36: Engineering & Construction Report was used.

Image 3 Picture 1. Render of the proposal of team SUM. The team focused on proposing and analyzing the possibilities of Topping up an existing building in the city of The Hague.

for a renovation project for the post-war Walk apartment building in Dreven, a Southwestern district in The Hague. The objective was to enhance the quality of life and increase the area's capacity by renovating the existing building and adding two additional stories. Moreover, since the buildings in the Dreven neighborhood have a similar typology, featuring an RC cellular concrete structure, this strategy was scalable to all of the 100 hectares complexes.

To increase the lettable floor area of the buildings, the proposal involved adding 30 wooden modules on top, following the Passive House standard. These modules were prefabricated as two 3D units, easily transported and assembled on-site. Timber was chosen as the primary structural material due to its renewable and bio-based nature, low embodied energy, carbon sequestration capability, and workability. Moreover, The flooring consisted of Leveled Veener Lumber (LVL) ribbed decks, known for their lightweight properties, reduced thickness requirements, simplified framing, lower logistical costs, and decreased construction waste. The construction process employed dry connections to ensure circularity and demountability. LVL offered the benefits of low weight, dimensional stability, and high load-bearing capacity.

Given the questionable structural capacity of the existing buildings, lightweight timber framed walls with vertical studs were employed as the primary load-bearing structure. This combination proved efficient, facilitating drilling, fastening, and fitting while ensuring predictability and dimensional stability. To address the span of the beam connecting the two modules, an additional glulam post was introduced to transfer the vertical loads. For stability, modularity, and prefabrication purposes, each module had one column, and the mirrored units were fixed together on-site using fasteners. The structural design of the Top-up embraced circularity and modularity, primarily utilizing 2D and 3D modular elements.

Each prefabricated module consisted of ribbed floor and roof decks, three sides of load-bearing timber framed walls (one opaque and two with openings), and one column. The 2D elements were combined in the factory using dry connections to create 3D modules with a total width of 3m, aligned with transportation, manufacturing, and lifting requirements. This dimension corresponded to the production size, allowing wood panels to be cut and sawed into the necessary LVL beams and panels.

The proposed strategies by the SUM team exemplify how lightweight timber top-ups can effectively achieve vertical expansion in existing buildings. The concept of lightweight and strength was realized using timber as a structural material and incorporating components sharing similar characteristics, such as vertical frame walls and horizontal studs in the flooring systems.

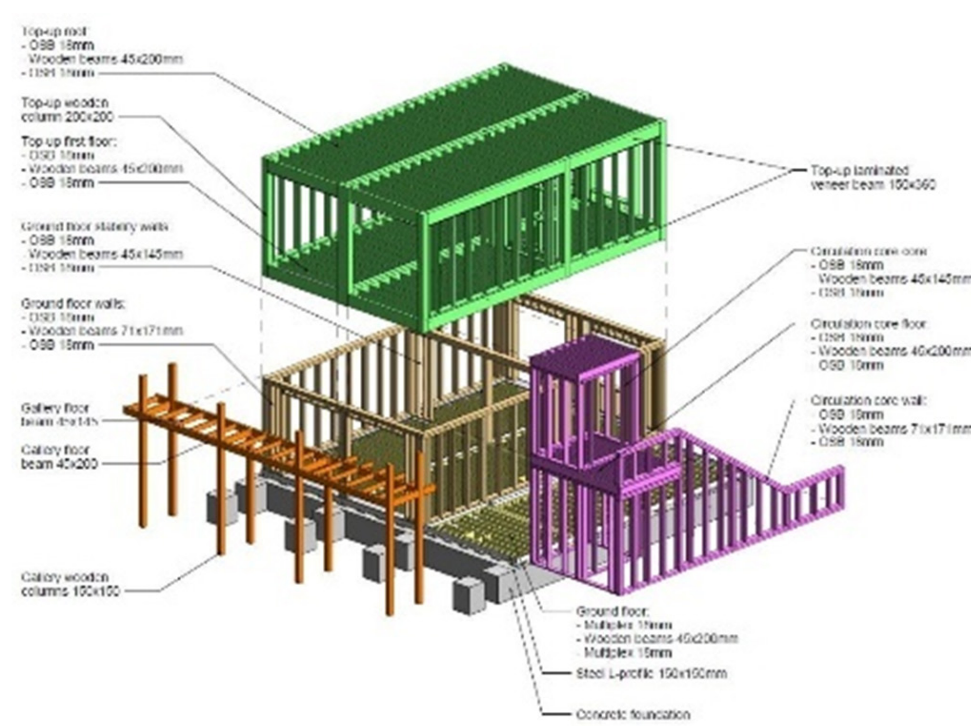

Image 4. Exploded axonoemtric of the pproposed timber top up made of lightweight timber framed walls with vertical studs

03

Theoretical framework

Timber: As a material to build top-up structures.

Regarding environmental sustainability, wood has a long history as an environmentally friendly building material in countries with boreal forests. Therefore, in these countries, wood-based products are highly competitive compared to the rest of the structural materials (Russell & Kumar, 2017). One of the reasons for this is that timber is a renewable material binding carbon during its growth, which leads to a lesser impact on the global warming potential, as compared to other building materials in the construction sector, as mentioned by Žegarac Leskovar & Premrov, (2021).

Therefore, to understand how timber can be used as a structural material for the design of timber Top-ups, this Section will analyze timber from the two main fields of study of the graduation studio to approach o SQ. 1 & SQ. 2:

First, in Section 3.1, from a Circularity perspective, to understand its relevance in the transitioning from a linear model to a circular for existing buildings by topping up using timber structures.

Furthermore, from Sections 3.2 to 3.5, from a structural perspective, to understand which species, products, components, and systems might be effective in the design stage of timber Top-ups. Therefore, its structural properties will be analyzed first, followed by the products used to produce structural components.

Figure 9 Estimated remaining material supplies worldwide (years to the left). Taken from (Lugt & Harsta, 2020)

3.1 Timber to Transform an Existing RC building using circular principles.

Forests cover approximately one-third of the Earth's land surface and provide various ecosystem services essential for human well-being. These services include providing food, fresh water, and raw materials, regulating air quality, climate, and water, and offering cultural and recreational value (Lugt & Harsta, 2020).

As studied by Carle & Holmgren (2009), Out of the total global forest area of 4 billion hectares, around 54% is utilized for timber production, food cultivation, and other product manufacturing. Roughly 24% is dedicated to conservation, with half-grown on legally protected land. Although planted forests constitute only 7% of the total forest area, they contribute significantly to the global supply of industrial roundwood, accounting for 35-40% presently and potentially reaching up to 80% by 2030 if the current expansion trend continues. These newly established forests aim for high species diversity and long rotation cycles, which enhance biodiversity and increase resilience against pests and wildfires.

Moreover, according to (Lugt & Harsta, 2020), Europe has demonstrated a notable increase in the carbon stock of its forests, primarily due to intensified reforestation efforts and improved forest management practices. The region's positive approach presents an opportunity to reduce the reliance on other construction materials with limited reserves, as shown in Figure 9. However, this can only be achieved through careful planning and sustainable harvesting of wood resources, ensuring that the current forest stock is not compromised for future generations.

Deforestation and Certification Schemes.

The increase in demand for timber products and deforestation is a common concern. However, Lugt & Harsta (2020) point out that the leading causes of deforestation are related to land use and the value attributed to the land for other uses due to not considering the environmental and economic value of the forest.

Consequently, in places where conserving forests under pressure from other economic activities becomes challenging as long as ecosystem services are not monetarily valued. In the case of Europe, because of the combination of local protection policies for forests, and the recognition of the added value found in forests, the use of timber is not a factor of deforestation in the present days in this region.

Moreover, forest certification schemes like the Forest Stewardship Council FSC and the Program for the Endorsement of Forest Certification (PEFC) are crucial in combating deforestation, illegal logging and valuing forest ecosystem services. Therefore, certified hardwood derived from sustainably managed tropical forests contribute to preserving a forest that would otherwise be destroyed for other purposes.

Figure 10 illustrates the forest development in the last years, from which certified forests account for 13% of the total forest area (525 million hectares in 2020, with a yearly growth of about 5% over the past decade), according to Vogt et al. (2019). Moreover. 37% of these certified forests are in Europe and 48% in North America. Unforutanelly, the chart shows that tropical regions experience significant forest loss. The total forest coverage in Africa and South America was approximately 1550 million hectares in 2020, with an annual forest loss of 6.5 million hectares (Vogt et al., 2019).

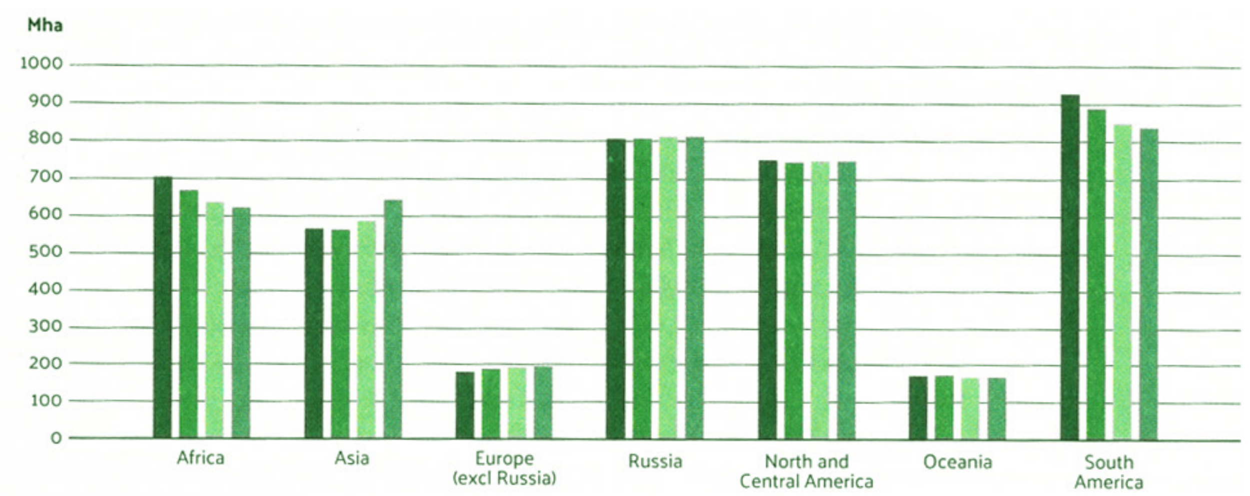

Figure 10. Global forest area in different continents in the past years (Mha).

3.1.1 Biobased materials compared to techno-cycle Materials

A potential for transforming an existing RC structure using circular principles is offered using mass timber products made of biobased materials instead of Technocycle materials.

According to McDonough & Braungart (2010), Bio-cycle materials refer to materials originating from living organisms or biological processes, while techno-cycle

materials are materials derived from non-renewable resources such as minerals, metals, and fossil fuels. As stated by these authors, Bio-cycle materials are typically renewable, biodegradable, and have a lower environmental impact than techno-cycle materials, which undergo extraction, manufacturing, and disposal processes within a linear economy.

Therefore utilizing materials from the bio-cycle, such as Timber, instead of those from the techno-cycle, such as RC, prompts benefits such as they are renewable, exhibit a low or even negative carbon footprint in contrast to the typically high carbon footprint, and possess biodegradability.

Although the discussion regarding what is environmentally more convenient between the use of bio-cycle and techno-cycle materials prompts an extensive discussion with several aspects to be considered, this research sets the stage for the following Sections, which will examine the topics of Upfront Upfront embodied carbon and carbon credits throughout the life cycle of Timber.

3.1.2 Upfront embodied carbon and Carbon credits during the timber's life cycle.

Upfront embodied carbon refers to the emissions associated with timber production, including logging, transportation, and processing processes. It accounts for the carbon dioxide released during these activities, contributing to the timber's overall carbon footprint. According to Olivier et al. (2017), from the total 35% of Greenhouse Gas emissions (GHE) from the construction industry, Upfront carbon emissions correspond to 28% (11% overall), while operational 72% (24%). The concept of upfront embodied carbon in timber construction is closely tied to the calculation of biogenic CO2 absorbed by trees during their growth and its impact on the overall carbon footprint of buildings.

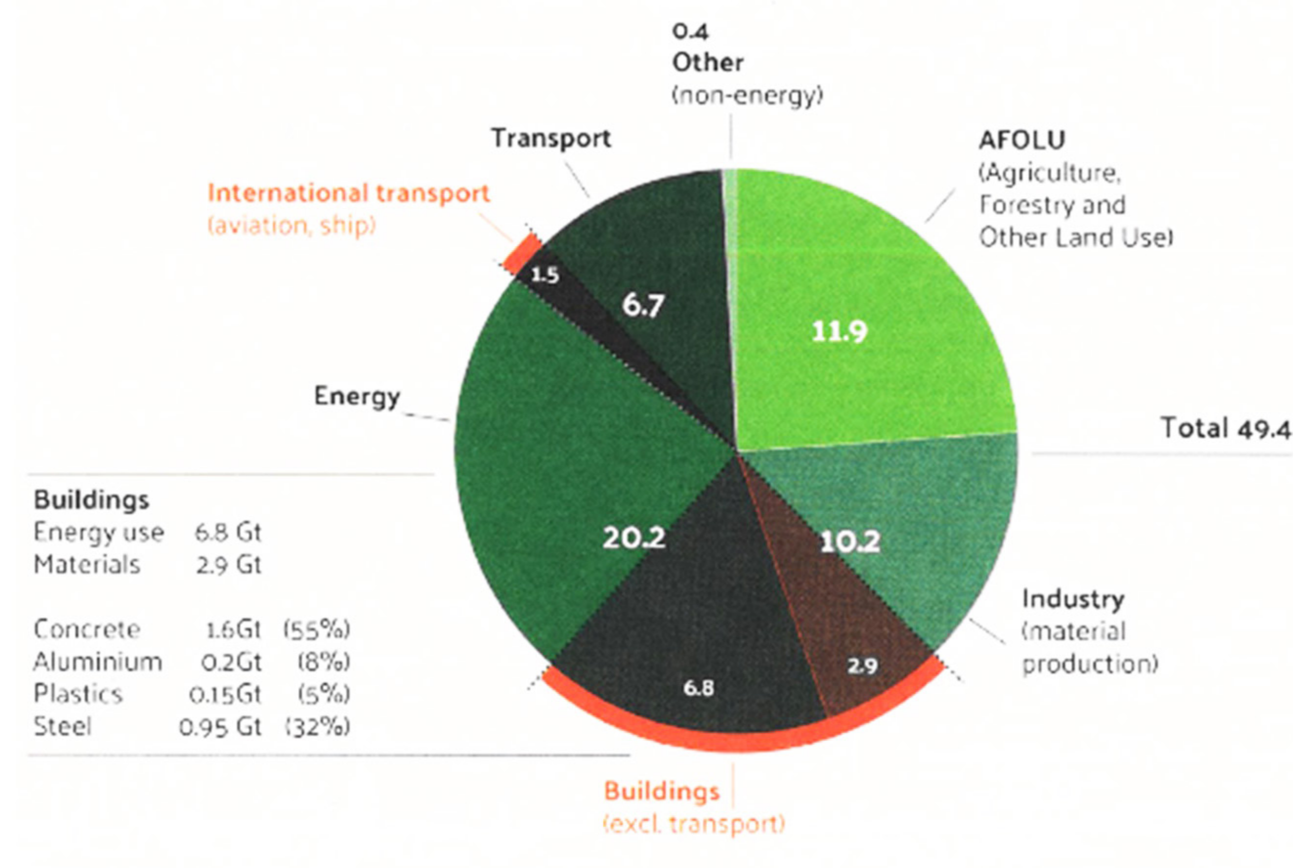

Figure 11 Division of GHG emissions worldwide (billions fo tons of CO2), taken from (Lugt & Harsta, 2020).

Lugt & Harsta (2020) describe how the biogenic CO2 absorbed by trees during their growth can be calculated and how it can affect the upfront embodied carbon in buildings with the concept of "Carbon credits." The calculation of CO2 stored in timber is relatively straightforward. Approximately half of the mass of oven-dry timber consists of carbon, and by using the molecular weight ratio of CO2 to carbon, the amount of CO2 locked in timber products can be determined based on wood density, as seen in Equation 1.

Carbon stotage of timber kg=(ρ wood/2)(44 g/mol/12g/mol).*

Equation 1 Carbon sequestration calculation of timber based on its density.

This calculation of CO2 captured is stored in trees as biomass, which will be processed into timber products. Furthermore, When these timber products end their life, if they are discharged and burnt for energy recovery (unless reused), the carbon captured during the tree's growth is released back into the atmosphere. However, suppose the timber product is burned in an electrical power plant, a common scenario in Western European countries. In that case, the resulting energy generation can replace electricity from fossil fuels. This substitution of fossil fuel-based electricity with renewable energy sources grants the timber product a "carbon credit," contributing to a negative carbon footprint.

In the case of timber components, the temporary sequestration effect of carbon in wood is considered in each producer's Environmental products Declarations (EPDs) following the EN 15804 standard. Moreover, it is accounted for as a negative value during the production phase (modules A-1 to A-3) and a positive value (emission) during its EoL phase in module D2.

3.2 Timber as a structural material.

Porteous and Kermani (2007) explain that the classification of commercial timbers into softwoods and hardwoods is based on their botanical origin (coniferous and deciduous) and not on the actual hardness of the wood. In that sense, Softwoods, which are usually evergreen and have needle-like leaves, are characterized by naked seeds or cone-bearing trees and are made up of single-celled structures called tracheids. These structures serve as conduits and support the tree as fibers that confer rigidity to Softwood timber.

Hardwoods, on the other hand, are generally deciduous and have broad leaves that are shed at the end of each growing season. The cell structure of hardwoods is more intricate than that of softwoods, with thick-walled fibers providing structural support and thin-walled vessels acting as a medium for food conduction, making, in many cases, stronger than softwoods. Although timber from deciduous species might have a higher strength than the one from Softwoods, these last ones are more commonly used due to their greater availability, ease of processing, lighter weight, high strength-to-weight ratio, and faster growth rate than hardwoods. Hence, softwoods are more practical and cost-effective in structural design.

3.2.1 Physical Properties of Timber

As a structural material, timber differs from steel, reinforced concrete or other composites because it is a biological and natural material with an orthotropic disposition, meaning that the mechanical and physical properties differ significantly on the fiber direction.(Fröbel & Godonou, 2022). Therefore a volumetric unit of timber will vary depending on its radial, longitudinal, and tangential direction in sawn timber.

Therefore, In structural design, it is crucial to recognize whether timber is loaded parallel or perpendicular to the grain. In the first case, timber has a superior strength, typically 1/10 compared to perpendicular, often close to zero.

3.2.2 Combustible characteristic

Timber, being a combustible material, has the potential to ignite and burn when exposed to heat or flames. Östman (2017) explains that this is due to the organic composition of wood, which includes cellulose, hemicellulose, and lignin. These organic compounds can decompose and release flammable gases when heated.

Although timber from deciduous species might have a higher strength than the one coming from Softwoods, these last ones are more commonly used due to their greater availability, ease of processing, lighter weight, high strength-to-weight ratio, and faster rate of growth compared to hardwoods.

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Graduation studio Structural design for change

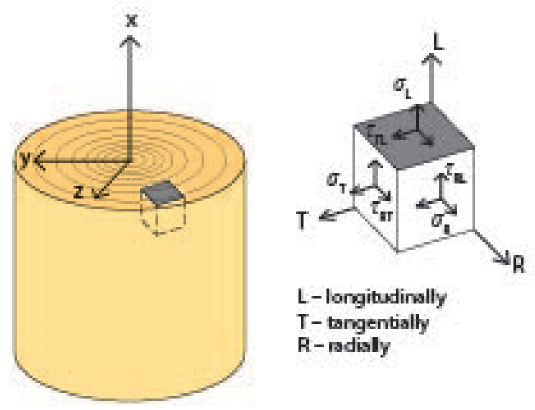

Image 5: shows the definition of the different stress in wood. By disregarding the difference in tangential and radial direction, the number of variables can be reduced to six, often denoted:

The heat from a fire causes the moisture in timber to evaporate and triggers the decomposition process, generating flammable gases like carbon monoxide and hydrogen. When exposed to a flame or spark, these gases can ignite and sustain a fire.

However, when timber is used as a structural material, measures can be taken to ensure that larger structures maintain their structural integrity in case of a fire. Ramage et al. (2017) explain that this involves implementing various strategies, which will be further explored in Section 3.5.3. Examples include using non-combustible materials to enclose smaller timber components and considering the charging rate in mass timber elements.

3.2.3 Density-to-strength ratio

The starting point of this graduation project focuses on this main property. The capacity of timber to have a high density-to-strength ratio compared to other conventional structural materials makes it a suitable candidate to Top-up existing structures. Timber's strength parallel to the grain is comparable to that of reinforced concrete, with hardwood slightly stronger and softwood slightly weaker. The strength-to-weight or elastic modulus-to-weight ratios are measures of the mass of material required for a structure of a given area. However, as seen in Table 2, timber Strenght-to-density ratio might withstand, but it cannot match modern high-strength concrete's compression strength.

Both timber and concrete are less stiff and strong than steel, although timber's lower density makes it more efficient for long-span or tall structures where the load is mainly its own weight, height, or span, and softwood performs similarly to steel in those measures. Therefore, timber is a structurally efficient material for structures or parts of structures where a high proportion of the load is the self-weight. However, as will be further discussed in the following sections, adding more timber mass in the composition of structural components by using Engineered wood products (EWP) can support and transfer a considerable amount of dead loads that might be present in a timber Top-up.

MATERIAL	DENSITY (kg/m^3)	STRENGTH (MPa)	STRENGTH/DENSITY [10^{-3} $MPa.m^3/kg$]
Structural steel	7800	400-1000	50-130
Concrete (compression)	2400	30-120	13-50
Clear softwood (tension)	400-600	40-200	100-300
Clear softwood (compression)	400-600	30-90	70-150
Structural timber	400-600	15-40	30-80

Table 2 . The relation between strength-density of different structural materials includes timber products, according to (Karol et al., 2018). Although the strength and density vary between species and mass timber products, this chart can give an overal

By adding more timber mass in the composition of structural components by using Engineered wood products (EWPs) components can support and transfer a considerable amount of dead loads that might be present in a timber Top-up.

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3.2.4 Moisture content

As a natural material, timber is hygroscopic, meaning it fluctuates in moisture content relative to its surrounding environment. Moreover, it is susceptible to fungal degradation above 20% moisture content. However, European standards for structural timber also specify an upper limit of 20% moisture content for 'dry graded' timber to receive a defined strength grading (Ramage et al., 2017). Furthermore, drier timber also provides a more receptive substrate for gluing and is lighter to transport. On the other hand, equilibrium in the moisture content might be necessary to avoid shrinkage in service.

For structural purposes, moisture content and its variation in timber play a significant role in the strength and stiffness of wood-based products. Therefore, to incorporate this effect in the structural design, three service classes have been defined in EN 1995:

25

- Service class 1 – Average moisture content does not exceed 12 %
- Service class 2 – Average moisture content does not exceed 20 %
- Service class 3 – Average moisture content exceeds 20 %.

Although in mass timber production, it is more common to find Service Class 1 products for the reasons mentioned above, it is relevant to remember that moisture percentage might vary over time, making its physical properties overcome what has been stated in the design codes in the future.

Moreover, as explained in Section 3.1.2, moisture plays a relevant role in Carbon Sequestration. The carbon sequestration potential is reduced when timber has a higher moisture content. Because moisture limits the amount of carbon stored in the wood, moisture acts as a barrier, occupying space within the timber that could otherwise be used to store carbon.

On the other hand, as explained by Lugt & Harsta (2020), the carbon sequestration capacity increases when timber's moisture content decreases through natural drying or specific drying processes. As moisture is removed, the wood becomes denser, allowing a greater carbon concentration to be stored within its structure.

Moisture plays an relevant role in Carbon Sequestration. When timber has a higher moisture content, the carbon sequestration potential is reduced.

3.2.5 Temperature

Besides ensuring constant moisture content through its lifespan, keeping control of the surrounding temperature of wood-based products between -30 °C to +90 °C is also imperative, as the strength and stiffness of wood decrease with increasing temperature). In the case of fire, temperatures above 95 °C (or 65 °C for long-term loads) might result in thermal degradation of the wood material. Moreover, due to higher temperatures, the cellulose chains around the fibers are shortened, and the structure of the hemicelluloses changes.

On the other hand, temperature variations can produce thermal expansion and contraction, which can cause the timber to change dimensions as it absorbs or releases heat. When the temperature of the timber increases, it expands in size, while a decrease in temperature causes it to contract. These changes in dimension can lead to stress on the timber. Therefore the connection design must be considered this effect to avoid potential cracking, warping, and other forms of deformation in the timber component. Maintaining steady temperature conditions is crucial for preserving the stable characteristics of timber and ensuring the long-term durability of the component, enabling its future reuse.

3.2.6 Creep (Sagging)

When utilizing timber as a structural material, it is crucial to consider the direction, magnitude, and nature of forces in designing the structural system and account for its unique properties as a natural material. Unlike other conventional materials, timber's physical properties can be influenced by factors such as the duration of load and size, which can cause variations in its behavior. For instance, timber is susceptible to sagging when subjected to constant loads over time. Although most of the deformation is recovered upon load removal, a small permanent deformation typically remains. Additionally, tests have shown that the bending strength of timber decreases with increased loading time, particularly in bending scenarios.

3.2.7 Timber Grading for Structures

According to Fröbel & Godonou (2022), to ensure that processed timber materials can support anticipated maximum loads as part of a structure in service, it is necessary to strength grade each piece of dimensional timber according to local codes. In the case of Europe, the BS EN 14081 grading standard permits the specification of a chosen strength class of timber and uses the characteristic strength values of that class in their design calculations, as seen in Table 3.

It is essential to acknowledge that Softwoods are susceptible to environmental changes that can impact their physical properties, potentially exceeding the structural capacity limits.

According to this standard, Strength grading consists of visual strength grading (VSG) and machine strength grading (MSG).

VSG is defined by rules describing weakness-related features, such as knots on the timber surface and any splits or related defects that may occur due to drying. On the other hand, MSG tests the characteristic values of stiffness and density for the strength classes.

The relevance of visual and machine grading in structural timber design is their ability to ensure that the timber used in construction meets the required strength and safety standards. By assigning a grade to the timber based on its strength properties, Its possible to decide the appropriate timber composition in the structural project and ensure that it will perform as expected.

Conclusion section 3.2

In conclusion, when designing Timber Top-ups or other lightweight structures, it is important to consider timber's moisture content, temperature, and creep. These considerations become even more significant when dealing with Engineered Wood Products, particularly in larger-scale applications such as long-span beams or columns with a significant section.

Softwoods are often preferred for timber top-ups due to their favorable strength-to-weight ratio. However, it is essential to acknowledge that they are susceptible to changes that can impact their physical properties, potentially exceeding the limits specified by structural codes for their intended lifespan.

Therefore, carefully evaluating the loading conditions and environmental factors is crucial during the construction of the structural system. Additionally, maintaining moisture content and temperature levels is vital to ensure softwood structures' long-term performance and durability. Considering these factors, timber-based constructions' structural integrity and reliability can be effectively preserved.

Strength class		Characteristic strength properties (N/mm²)						Stiffness properties (kN/mm²)				Density (kg/m³)	
		Bending ($f_{m,k}$)	Tension 0 ($f_{t,0,k}$)	Tension 90 ($f_{t,90,k}$)	Compression 0 ($f_{c,0,k}$)	Compression 90 ($f_{c,90,k}$)	Shear ($f_{v,k}$)	Mean modulus of elasticity 0 ($E_{0,mean}$)	5% modulus of elasticity 0 ($E_{0.05}$)	Mean modulus of elasticity 90 ($E_{90,mean}$)	Mean shear modulus (G_{mean})	Density (ρ_k)	Mean density (ρ_{mean})
Softwood and poplar species	C14	14	8	0.4	16	2.0	1.7	7.0	4.7	0.23	0.44	290	350
	C16	16	10	0.5	17	2.2	1.8	8.0	5.4	0.27	0.50	310	370
	C18	18	11	0.5	18	2.2	2.0	9.0	6.0	0.30	0.56	320	380
	C20	20	12	0.5	19	2.3	2.2	9.5	6.4	0.32	0.59	330	390
	C22	22	13	0.5	20	2.4	2.4	10.0	6.7	0.33	0.63	340	410
	C24	24	14	0.5	21	2.5	2.5	11.0	7.4	0.37	0.69	350	420
	C27	27	16	0.6	22	2.6	2.8	11.5	7.7	0.38	0.72	370	450
	C30	30	18	0.6	23	2.7	3.0	12.0	8.0	0.40	0.75	380	460
	C35	35	21	0.6	25	2.8	3.4	13.0	8.7	0.43	0.81	400	480
	C40	40	24	0.6	26	2.9	3.8	14.0	9.4	0.47	0.88	420	500
	C45	45	27	0.6	27	3.1	3.8	15.0	10.0	0.50	0.94	440	520
	C50	50	30	0.6	29	3.2	3.8	16.0	10.7	0.53	1.00	460	550
Hardwood species	D30	30	18	0.6	23	8.0	3.0	10.0	8.0	0.64	0.60	530	640
	D35	35	21	0.6	25	8.4	3.4	10.0	8.7	0.69	0.65	560	670
	D40	40	24	0.6	26	8.8	3.8	11.0	9.4	0.75	0.70	590	700
	D50	50	30	0.6	29	9.7	4.6	14.0	11.8	0.93	0.88	650	780
	D60	60	36	0.6	32	10.5	5.3	17.0	14.3	1.13	1.06	700	840
	D70	70	42	0.6	34	13.5	6.0	20.0	16.8	1.33	1.25	900	1080

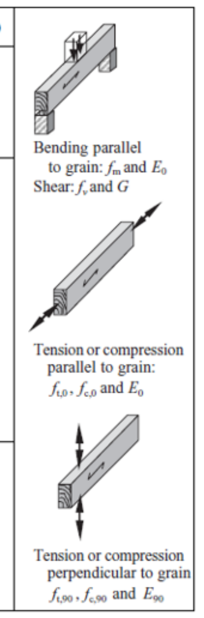

Table 3. Strength grading according to EN 338: 2003, taken from (Porteous & Kermani, 2007). This classification can be observed in the difference in physical properties of conifers and deciduous species. Although Hardwoods can perform better than most the softwoo

3.3 Timber Products for structural components

As explained by Ramage et al. (2017), Structural timber products can be classified based on the primary transformation processes after the logs are harvested from the forest. These processes, as illustrated in Diagram 2, can be organized into three: Sawing processes, Pelling processes, and stranding processes, which produce the raw material that will be converted in a second process into linear and planar components.

After the wood is harvested from certificated woods, the crown is removed, and often, it is debarked in the forest to be transformed into Solid round timber.

- Sawing: corresponds to cutting logs or trees into wood planks, also known as sawn timber or lumber. Logs are then classed and stockpiled underwater sprays to maintain steady moisture conditions and avoid drying out. Some of the better quality ones are sent to peeling plants to manufacture veneers. However, the majority (depending on the quality) are sent to sawmills to convert round logs to sawn timber.

- Peeling: The logs that meet the desired criteria for producing veneers are fed into a la machine that rotates the log against a stationary knife blade, which removes a thin layer of wood from the log's surface. This process is repeated several times, rotating the log slightly each time until the entire log has been peeled into a continuous sheet of veneer. The veneers are then sorted by quality, thickness, and size. The higher-quality veneers are used for high-end products, such as furniture, while lower-quality ones are used for engineered timber products.

- Stranding: The logs suitable for producing engineered timber products after debarked are cut into small pieces that are sorted and graded based on their quality and size. Next, the logs are fed into a machine that uses a large rotary drum to flake the wood into thin strands. This process, called flaking, involves cutting the logs at an angle to produce thin, flexible strands of wood that are then dried to remove any remaining moisture, which helps to improve their strength and durability. Once dried, the strands are sorted by length and thickness, and any defective strands are removed. Finally, the strands will be used to produce boards, as will be further discussed in section 3.21.

The significant negative aspect of round wood poles is that they pose difficulties for systematic building construction. They lack straightness, gradually decreasing width or diameter, and length and cracks.

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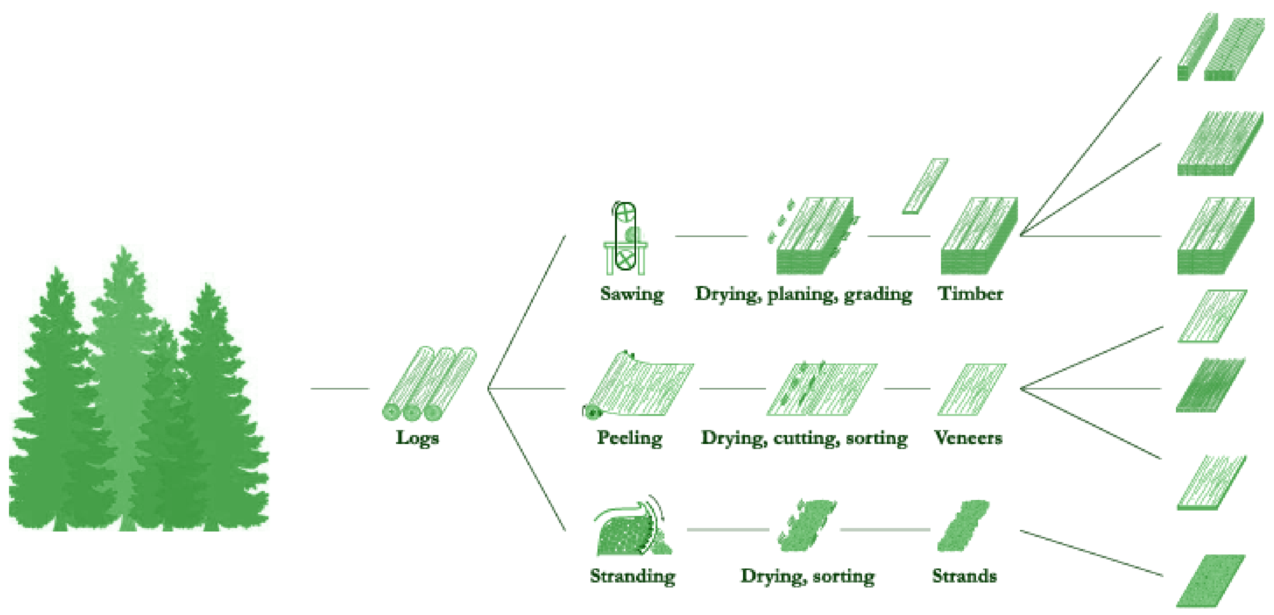

Diagram 2. The processing chain of engineered timber products is taken (Ramage et al., 2017). This diagram illustrates an overall of processing of the most common structural timber products available in the market.

3.3.1 Engineered Wood Products (EWPS)

“The basic element of modern timber construction is, therefore, the plane and no longer the linear member,” declared Swiss architect Andrea Deplazes in 2000, stating that timber construction is developing into a “plate tectonics” system from a “linear” system. (Kaufmann et al., 2018)

As described in the previous section. The readily available sawn sections of softwood are limited in size and quality. Moreover, the most significant section sizes available are 75 mm thick × 225 mm wide and at most 5 meters long (Porteous & Kermani, 2007). In that sense, building structural systems with higher capacities are necessary to increase the amount of timber in components in its section and shape.

Therefore, as seen in Image 7, Engineered Wood Products (EWPs) were developed at the end of the XX century to overcome the limitations of sawn timber for structural components by incorporating a combination of adhesives and dry connections in various forms. These products are engineered and tested to predetermined design specifications and to respond to national or international standards. According to (Porteous & Kermani, 2007) EWPs may be selected over solid-sawn timber in many applications due to a number of advantages such as:

- They can be manufactured to meet application-specific performance requirements.

- Large sections or long-length panels can be made from small logs by removing or dispersing defects.

- They are often stronger and less prone to humidity-induced warping than equivalent solid timbers. However, particle- and fiber-based boards readily soak water unless treated with a sealant or paint.

- EWPs do not swell or shrink, allowing for accurate mechanical properties prediction.

In their study, Michael H. Ramage et al. (2017) extensively classify the most commonly used Engineered Wood Products (EWPs) for structural purposes that might be used in the proposal of timber Top-ups for existing structures. However, since the study case for this graduation project is set in the Netherlands (as will be explained in detail in further sections), the next part of the document will focus on studying just the EWPs validated by the local structural codes, in this case, the Eurocode 5 (EC 5). In that sense, the structural design proposed in section 5.2 will be closer to being feasible. It will bring the possibility of using normative approved components for timber top-up systems closer to reality.

Therefore, after analyzing the most common existing EWPs in the market, the following sections will review the products that, due to their structural and circular capacities, might be suitable for producing timber components in the design of timber top-ups. The analysis of the products not mentioned in this section might be found in Appendix B.

Engineered Wood Products (EWP’s) were developed in the end of the XX century to overcome the limitations of sawn timber for structural components by incorporating a combination of adhesives and dry connections in various forms.

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Glued-laminated timber (Glulam)

As defined in their book Structural timber design to Eurocode 5, Porteous & Kermani (2007), Glued-laminated timber, also known as glulam, is a structural timber product made by bonding small sections of timber boards (called laminates) together with engineered high-performance adhesives (Commonly Polyurethane reactive adhesives or PUR). The laminates are laid up so their grains are parallel to the longitudinal axis, resulting in a strong and rigid product compared to sawn timber. These individual laminates are usually 19-50 mm in thickness and 1.5-5 m in length, and they are end-jointed using a technique called finger jointing that allows the production of components with lengths of almost 16 mts long.

Glulam can be manufactured with laminates of different grades to match the level of design stresses, commonly by arranging the higher-grade laminates in the outer sections of the component (which are highly stressed regions when used as a beam). In comparison, the lower grade laminates are used in the inner parts. This makes it possible to use the timber resource more efficiently.

For its production, High-performing synthetic polyurethane (PUR) is applied to the faces of the laminates to assemble glulam. They are then placed in mechanical or hydraulic jigs of the appropriate shape and size and pressurized at right angles to the glue lines.

For structural purposes, Glulam can produce structural components subjected to combined bending and axial loading, such as beams or columns. Moreover, according to the author, the key design considerations focus on four main strategies:

- Choice of materials for the Glulam component: On one hand, consider the properties of the individual laminates for structural calculations (density, strength, stiffness, and moisture content). And in the other hand, the type of adhesive used to bond the laminates as it affects the mechanical properties of the component. This aspect can be considered by reviewing the technical product sheets of different providers that can ensure the product's mechanical properties.

- Ensure structural capacity of components: The design of glulam members must ensure they can carry the required loads with sufficient strength and stiffness. This involves calculating the design resistance of the member using appropriate partial safety factors in the Ultimate Limit Design method (ULS) and Serviceability Limit State (SLS), further explained in section 0.

- Durability: Ensure conditions to resist deterioration due to environmental factors such as moisture, fungi, and insects (as explained in section 3.1). This can be done after selecting appropriate preservatives and coatings in the production stage and ensuring adequate ventilation and drainage conditions on the intervention site.

- Considering the inner structure of the Glulam component: For the design of connections between glulam components and their loading, the properties of the individual laminates and the adhesive used to bond them must be considered. The connection design should also ensure that the load transfer between the members is efficient and that there is sufficient strength and stiffness to resist the applied loads. Being a product made of laminates, it can present concentrated stresses that might delaminate the component without considering its composition.

Increased strength and stiffness, bigger sizes and shapes, particularly in width, length, and section compared to solid timber. results in greater design flexibility and enables longer spans and bigger laods in big scale structures.

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Using Glulam to create structural components offers several advantages, including increased strength and stiffness compared to solid timber of equivalent size and shape, particularly in width, length, and section. This results in greater design flexibility and enables longer spans without intermediate support. Additionally, modern production methods allow for the manufacture of various shapes and sizes to meet specific design requirements, making it particularly well-suited for applications where high strength in bending and axial loading is required.

Cross-laminated timber (CLT) and Doweled laminated timber (DLT)

In recent decades, one of the most significant developments in timber for structural building design is the introduction of cross-laminated timber (CLT) and Doweled Laminated Timber (DLT). Introducing these materials into the construction of buildings has led to increased use of timber in large-scale construction. This is because it can create planar geometry structural components that minimize the inhomogeneity and anisotropy inherent in wood's fibrous structure (Kaufmann et al., 2018), allowing greater design flexibility and the ability to create larger, more complex structures with timber. Moreover, using different species with different service classes of timber made the form even better than Solida-sawn timber components.

As seen in Image 9, CLT is composed of at least three layers of laminates made from softwood and/or hardwood that varies in thickness from 60 mm to 400 mm or more (depending on the producer and the structural requirements of the building), into panels that can be produced in widths up to 3.5 meters up to 20 meters long, making it a suitable option for planar structural components such as slabs or shear walls. Its main characteristics may vary depending on how it is placed and loaded in a structure. However, this material's most relevant feature is that the constituent laminates' properties determine the CLT panel's properties.

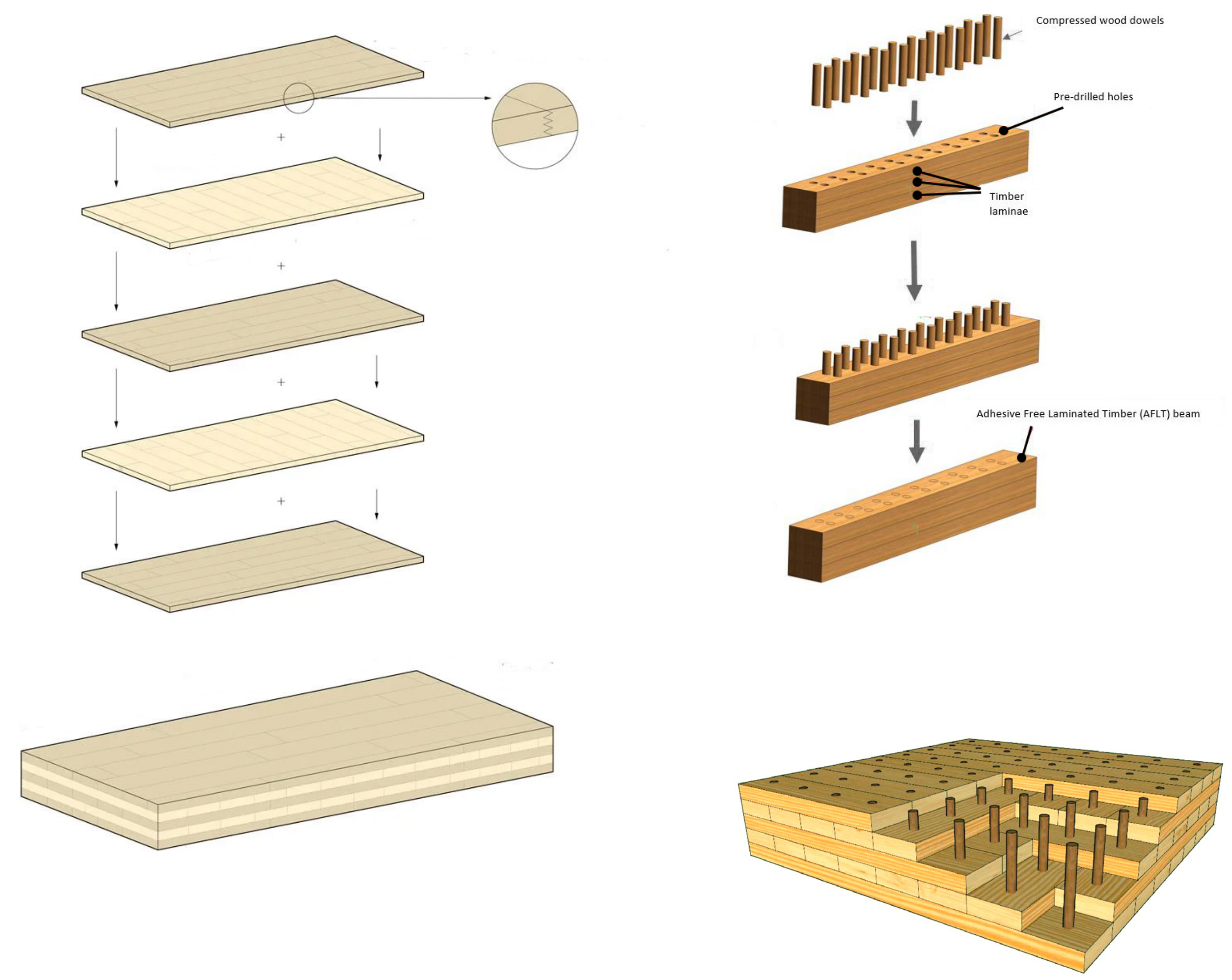

Image 9 Sections of a CLT and DLT laminated used for structural compoentns .

the use of CLT and DLT in Timber Top-ups might present an opportunity to respond to the different structural constraints that might appear during the design process of the timber top-up

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In the book Manual for Multistory Building, Kaufmaan et al. (2018) state the main mechanical properties of this material used as a planar component that might be used in the design of timber Top-ups as well:

- Loads along the plane (wall system): In this structural situation, the planar and homogeneous cross-sectional strength of cross-laminated timber walls makes it easy for them to absorb horizontal loads in the direction of the wall, makings them ideal for bracing multi-story buildings.

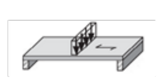
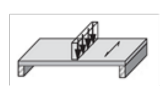
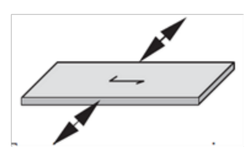
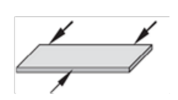
- Loads perpendicular to the edge (Slabs): As a slab, CLT can span different distances depending on their thickness and support configuration. The load-bearing direction of a CLT element depends on the position and number of layers of its boards. The main load-bearing direction runs parallel to the top layers since the layers in the span direction determine its structural performance. Linear support that distributes loads evenly is optimal for CLT elements, although loads can be supported at specific points.

- Overhangs: It is possible for CLT slabs to overhang on two sides, and the degree of overhang can be adjusted according to the load-bearing capacity of the main load-bearing directions. To facilitate the distribution of loads within the element, supports can be positioned from the corners toward the center of a panel. CLT elements can form rigid plates that brace buildings when the ceiling slabs are correctly joined.

Like CLT, DLT is made by stacking multiple softwood laminates together (Image 9). However, instead of cross-laminating the layers, the layers are aligned in the same direction allowing a planar surface as well. Moreover, the layers are then mechanically interlocked using hardwood dowels or timber fasteners, which are inserted through the thickness of the panel at regular intervals to create a strong, rigid, and stable composite structure. This prominent feature limits the amounts of bounding agents, making it perform better regarding environmental impacts in its EoL.

However, according to Sotayo et al. (2020,) unlike CLT, DLT panels have limited span capacity, which means design challenges that may affect, in this case, its use in timber Top-ups. The reason behind this is that, since there is the absence of cross-lamination in DLT laminates, the structural rigidity and strength are reduced, meaning that the ability of slabs and walls to bear weight across larger areas might be limited. Therefore, this may result in the need for additional structural support or alternative materials and the higher mass that will increase the Upfront Upfront embodied carbon in the intervention.

Overall, because of the reasons exposed in this section, the use of CLT and DLT in Timber Top-ups might present an opportunity to respond to the different structural constraints that might appear during the design process of the timber top-ups. Therefore, its application might vary depending on its specific use in the building, and its decision should be carefully analyzed to respond to the structural and circular constraints efficiently.

		Bending 0	Bending 90	Tension 0	Compression 0	Tension 90	Compresion 90	Panel Shear
		fm,0,k	*fm,90,k*	*ft,0,k*	*fc,0,k*	*ft,90,k*	*fc,90,k*	*fv,k*
								
Product		N/mm2	N/mm2	N/mm2	N/mm2	N/mm2	N/mm2	N/mm2
Plywood 12 mm		*23*	*11,4*	*15*	*15*	*12*	*12*	*2,9*
Plywood 24 mm		*21,6*	*12,4*	*15,4*	*15,4*	*11,4*	*11,4*	*2,9*
Laminated Venner lumber Kerto S	***LVL***	*50*	*50*	*35*	*35*	*0,8*	*6*	*4,1*
Laminated Venner lumber Kerto Q	***LVL***	*36*	*36*	*26*	*26*	*6*	*9*	*4,5*
Laminated strand lumber 1.5 E	***LSL***	*36,3*	*36,3*	*24,4*	*25,4*	*-*	*8,9*	*8,6*
Laminated strand lumber 1.7 E	***LSL***	*42*	*42*	*28,9*	*31*	*-*	*10,1*	*8,6*
Oriented Strand Board/4 18-25	***OSB***	*21*	*11,4*	*10,9*	*17*	*8,8*	*13,7*	*6,9*
Cross laminated timber CL 24h	***CLT***	*24*	*24*	*16*	*24*	*16*	*24*	*8,9*
Cross laminated timber CL 28h	***CLT***	*28*	*28*	*18*	*28*	*18*	*28*	*9,5*

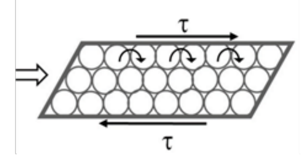
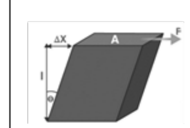
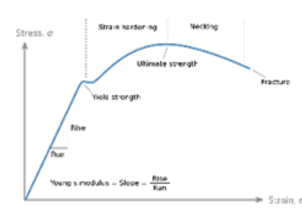
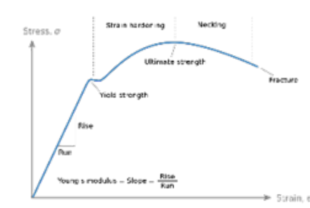
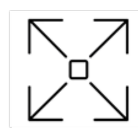
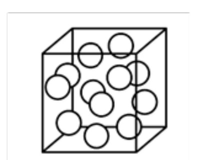
General Conclusion for timber products for structural components

In conclusion, after reviewing all of the available timber products for structural design, we will find that depending on the structural and architectural constraints, Some products are more suitable for specific structural components. However, To create timber components that might respond to the structural demands of topping up existing buildings, EWPs are required instead of natural timber to allow more significant dimensions. However, it is relevant to consider for the structural design that the dimensions of these products are limited, as they depend on the size of the machines that manufacture them and the capacity of the existing transportation methods.

Regarding the structural constraints, as in natural timber, the strength per unit of EWPs will vary depending on how loads are positioned and more over-related to the alignment of the product's materials, which might affect the composition of the structural system and might be something to take into the design phase.

On the other hand, from a circular perspective, The quantity of synthetic binding agents used to create EWP varies for each product and affects their environmental impact: Although the amount of these agents is usually a tiny percentage compared to the amount of timber, its variation might result into impacts due to the additional resources required for manufacturing and disposal of these materials.

Table 4, presents a comprehensive overview of the mechanical properties of various exposed Engineered Wood Products (EWPs) to facilitate comparison. Based on the evaluation, it can be inferred that Cross-Laminated Timber (CLT) and Glulam demonstrate superior performance compared to other components across most of the assessed parameters. This suggests that CLT and Glulam are highly suitable options for designing Timber Top Ups, considering their exceptional performance capabilities.

Planar rolling Shear 0	Planar rolling Shear 90	Panel Shear	Mean modulus of elasticity Bending 0	Mean modulus of elasticity Bending 90	Mean modulus of elasticity tension compresion 0	Mean modulus of elasticity tension compresion 90	max size	Density
fr,0,k	*fr,90,k*	*Gv*	*Em,0, mean*	*Em,90, mean*	*Et/c, 0, mean*	*Et/c,90, mean*	*t*	*Pk*
								
N/mm2	N/mm2	N/mm2	Mpa	Mpa	N/mm2	N/mm2	mm	*Kg/m3*
0,9	*0,9*	*2,9*	*9200*	*4600*	*7200*	*4800*	*1200*2400*	*460*
0,9	*0,9*	*2,9*	*8700*	*5000*	*7400*	*4600*		
2,3	*2,3*				*13800*	*10500*	*600*20000*	*510*
1,3	*1,3*				*13800*	*10500*	*2500*20000*	
8,6	*8,6*				*10300*	*11700*	*2440*14630*	*420*
3,2	*3,2*				*10300*	*11700*		
1,1	*1,1*	*6,9*	*6780*	*2680*	*4300*	*3200*	*2400*4800*	*550*
1,5	*1,5*	*7*	*10000*	*10000*	*9800*	*9800*	*3500*16000*	*460*
1,6	*1,6*	*7,2*	*11000*	*12000*	*9600*	*9600*	*3500*16000*	*500*

Tabble 4 Structural properties of different timber products.

3.4 Structural timber components

The most commonly used structural components in timber construction are prefabricated walls, ceilings, and roofs. Therefore, this section will discuss the different types of components employed in timber structures, specifically focusing on those commonly found in multi-story timber construction. This analysis aims to examine their respective demands on the support of timber structures and explore their potential application in the design of Timber Top-ups.

3.4.1 Vertical structural components

These components will be characterized based on their orientation within the structure: vertical structural components (walls and columns) and horizontal components (ceilings and plates). By conducting a comparative analysis of these various structural elements, we can comprehensively understand their advantages and disadvantages.

Columns

As seen in the previous section, Glulam has an outstanding compression parallel to grain that allows continuous sections of the fibers compared to other EPWs; therefore, it can be an effective alternative to use in the design of columns in timber structures subjected to flexo compression. Moreover, since they can be prefabricated in considerable sizes, they can be produced in many sections.

CLT walls

Because of its interlayer composition, In CLT, vertical layers of boards best support vertical loads in cross-laminated timber walls. As a result, a wall with vertical boards in the top layer is stronger than one with horizontal boards. The walls' planar homogeneous cross-section and strength enable them to easily absorb horizontal loads in the wall's direction, making them well-suited for bracing multi-story buildings.

DLT walls

Dowel laminated timber walls as a component can absorb heavy vertical loads as they are applied in the direction of the wood's grain. Moreover, by stacking the laminates next to each other, buckling can be prevented in the direction of their weaker cross-sectional axes and, therefore, homogeneous planar distribution of forces where the weak points are minimized. However, due to its composition of staking laminates, DLT walls present a relatively low stiffness when subjected to horizontal loads along and transverse; therefore, they might require additional components such as bracings to ensure their stability.

Laminated veneer lumber walls

As explained in section 3.3.4 LVL, Being made of rotary-cut veneers with some layers at 90° angles reduces the wood's anisotropic properties and balances out any inhomogeneities in the wood. In that sense, LVL walls can optimally transfer Vertical loads as far as the material walls are mainly aligned in the vertical direction, enabling these walls to bear heavy loads. Laminated veneer lumber's excellent structural properties mean it is used to reinforce other timber structures around bearings and joints.

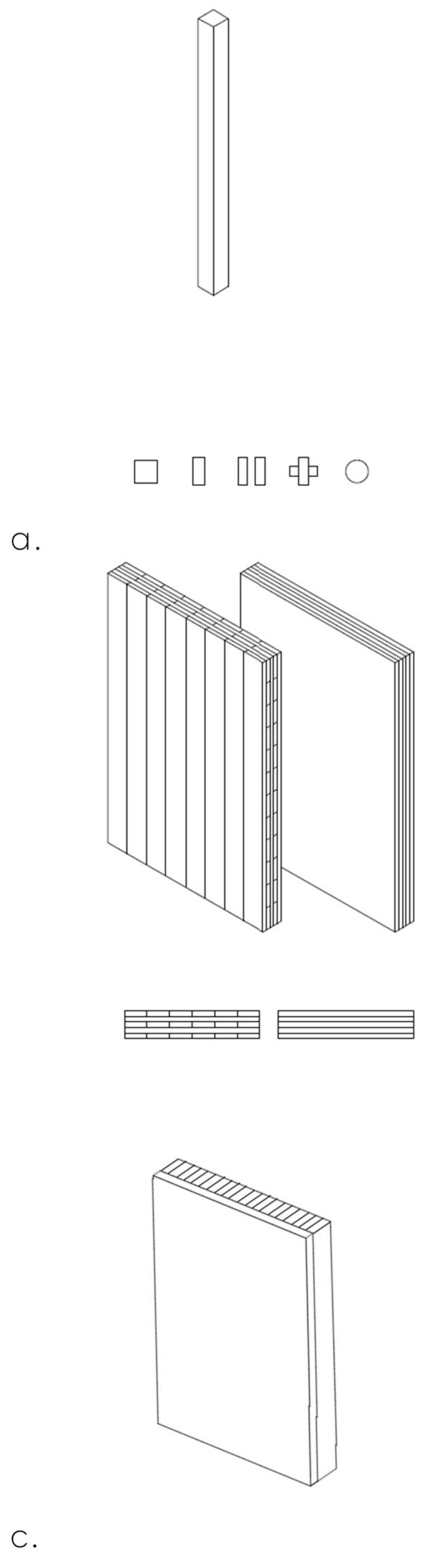


Figure 12. Examples of vertical structural components. a. Glulam & timber columns, b. CLT and DLT walls, c. LVL walls

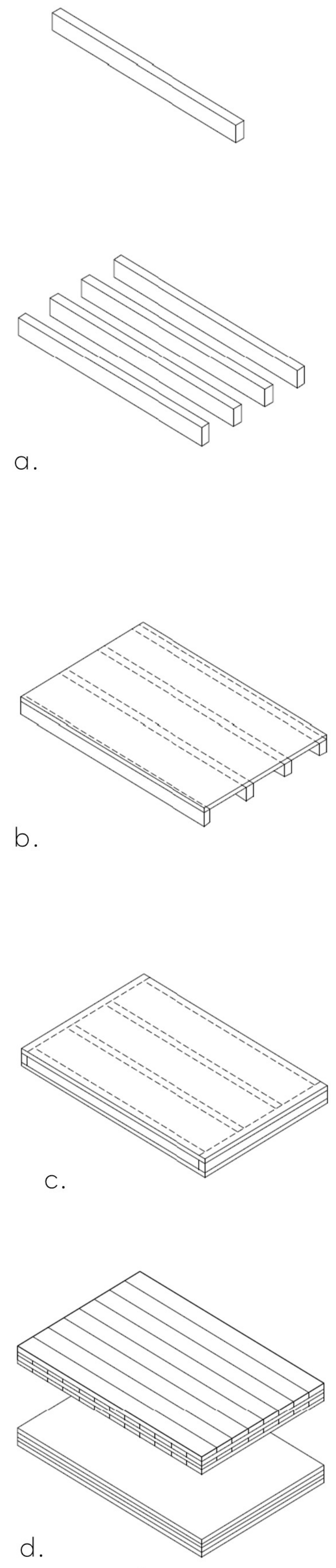


Figure 13. Examples of horizontal structural components. a. Beams made of Glulam and LVL, b. Ribbed box opened slabs, c. Ribbed closed slabs, d. CLT and DLT slabs

3.4.2 Horizontal structural components

EWP Beams

Although beams can be made of solid timber or solid construction lumber, as seen in previous sections, this limits the dimensions of their cross sections. Therefore, EWPs such as Glulam and LVL can create beams with greater sections that can withstand and transfer bigger loads. Moreover, EWPs can also be configured to create variations in geometry in sections that might have a more efficient volume-strength ratio, resembling the geometry sections of steel beams, such as IPE profiles and box sections. These sections are:

Timber I joist sections: I beam made of thin wood-based material webs with solid timber upper and lower flanges that absorb tensile and compressive forces and brace the thinner web against buckling. This section follows the principle where the beam section is mainly subjected to compression in the upper section and tension in the lower part of the beam.

Timber box sections: Box beams follow the same principle as I-joist, but their geometry helps them to resist lateral buckling better. Here the upper and lower surface layers take on the function of upper and lower flanges.

Trusses: A trussed beam's web minimizes the use of material necessary to form a rigid, shear-resistant connection between its upper and lower flanges. Its members are subject mainly to regular forces, with only the upper flange also subject to bending loads. This system is more commonly used when big spans are required where it is required to reduce the self-weight of the component in order to withstand low loads, such as in open space areas.

Beam and box ceilings (Ribbed box)

Beam ceilings consist of beams covering the primary span of a slab attached with panels or boards extending from beam to beam to form a ceiling. In this case, the direction of the beams follows the bigger span, meaning that they transfer loads unidirectionally. This principle applies to ceiling structures with a wide range of beam spacings. This system cannot work well in deflection compared to other systems due to the lack of an effective section in the lower part of the slab. However, box ceilings correspond to an alternative to face this limitation.

Following the same principle as Beam ceilings, Box ceilings, sometimes called hollow box ceilings, are lighter ceiling slabs and ribs combined with planking to create a very high-performance planar support structure. They enable the ceiling slab's thickness to be minimized and benefit from the fact that.

Although these elements might be efficient regarding volume-strength ratio, their use in timber ups might be limited as they may lack the mass sufficient to respond to fire resistance, as will be discussed in further sections.

CLT and DLT slabs

CLT elements function structurally as relatively homogeneous slabs. Their spans depend on the slab's thickness and support situation as they have a main and ancillary load-bearing direction depending on the positioning and number of their layers of boards. Therefore, it can be used in structural slabs, on the one hand, as the primary load-bearing element of the slab, supported by a grid of beams or columns, and on the other, in conjunction with other materials, such as concrete, create composite structural slabs.

It will perform differently depending on the direction where the top layers determine the span direction to determine its structural performance. Compared to others, one characteristic of these systems is their possibility to overhang on two sides in proportion to their load-bearing capacity in the main and ancillary load-bearing directions and be installed to have a continuous beam effect. Moreover, CLT slabs can form rigid plates and, if the ceiling slabs are appropriately joined, can effectively brace buildings.

On the other hand, as seen in previous sections, DLT, not using synthetic bounding agents but mechanical connections, will perform lower compared to CLT, requiring a larger amount of material to overcome this limitation.

Laminated veneer lumber ceilings.

Laminated veneer lumber (LVL) ceilings are designed to function as uniform slabs with a clear main span direction, following the direction of the fibers in the layers. As a result, linear supports are required. On the other hand, LVL panels with crossed layers can also support an additional load-bearing direction and can be supported at various points. The span of these elements will depend on the thickness of the panel and the support system in place. LVL elements are sturdy and can effectively support buildings, provided the slab elements are correctly connected.

Overall, horizontal and vertical components can be integrated into different contexts to address distinct structural and architectural considerations that present limitations and opportunities, as seen in Image 10. The case study chosen in Section 4.1 investigates the most appropriate conditions for a specific situation involving RC structures.

Subsequently, in the upcoming section, an analysis will be conducted from a structural design perspective to determine the optimal combinations based on EC 5. This analysis will consider various parameters, including fire resistance, modularity, wind loads, and durability, as crucial requirements for timber structures.

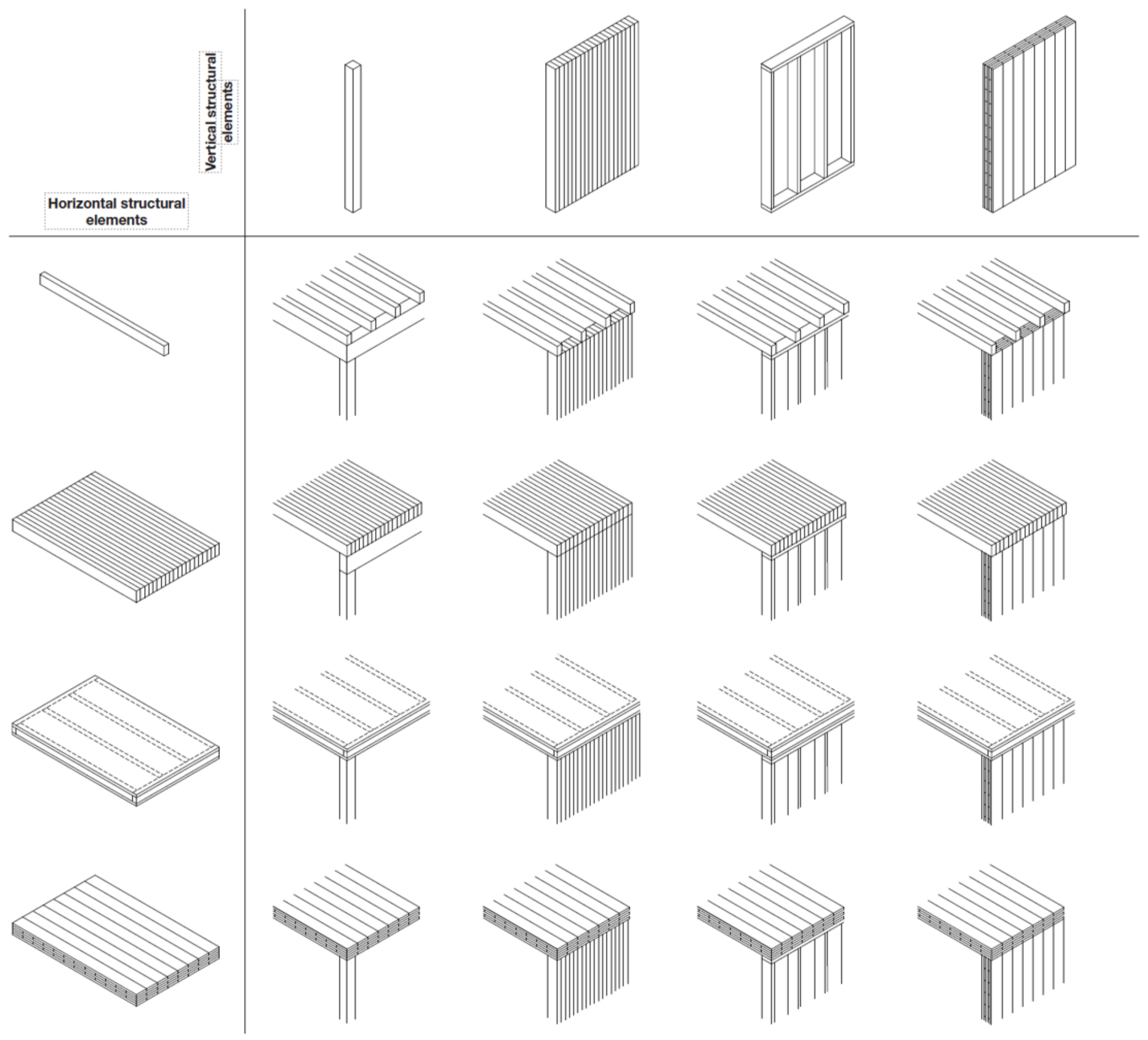


Image 10 Comparison matrix of EPW in the definition of structural systems.

The application of the limit state method in the design of timber top-up structures within existing RC structures offers a systematic approach to assess the structural feasibility of a conceptual structure.

3.5 Structural timber systems

After understanding the main characteristics of timber and how it can be used as a component and product for timber Top-ups, follows the integration of these elements into a structural composition that would host the Top-up. Moreover, this stage involves choosing the dimensions of load-bearing members and modeling the load-bearing structure's requirements for material resistance, performance, and durability during the structure's service life.(Fröbel & Godonou, 2022).

Therefore the following section will describe the most relevant criteria in the proposal for the structural design of timber to set a design framework for the design of Timber Top-ups. To do so, Eurocode 5 works as an extension for Eurocode 0: which uses the Limit design States analysis to define Structural design.

3.5.1 Principles of limit state design for timber components

As defined in EC 0 (ENV 1995–1–2, 1995), the objective of the limit state method is to define the conditions beyond which the structure no longer satisfies the relevant structural performance requirements in two scenarios:

- The Ultimate limit state (ULS): related to the safety and evaluating the states associated with collapse or other forms of structural failure such as rupture, loss of equilibrium, transformation into a mechanism, and fatigue.

- Serviceability limit states (SLS), related to those states in which the structure, although standing, behaves in an unsatisfactory way due to excessive: deformation, vibration, cracks, or damage in a negative way affecting the use.

Its application in the proposal of structures can be described in five main steps:

- Define the relevant limit states for the structural behavior to be checked.
- Define the respective design situations for the structure (Conceptual design).
- Determine the appropriate actions and load combinations to be considered. (Load cases).
- Using appropriate structural models for design and taking account of the inevitable variability of parameters such as material properties and geometrical data. (Calculation and simulations).
- Verify that none of the relevant limit states is exceeded. (Verification).

Overall, applying the limit state method in the design of timber top-up structures within existing RC structures offers a systematic approach to assess the structural feasibility of a conceptual structure. Therefore, as described in Section 5.2.4, following these five steps, the limit state method will facilitate the design of the Timber top-up.

3.5.2 Durability

As explained in EC 5, durability is a crucial parameter to consider when applying timber Top-ups. It refers to the ability of the structure and its elements to remain fit for use throughout the design working life while accounting for potential deteriorating factors that may arise during this period.

According to this code, several key criteria contribute to the durability of timber structures. For the Conceptual design of structures, the most relevant ones are explained:

Firstly, the structure's intended use must be considered during the elements' pre-dimension. For example, mass timber slabs involve considering factors that may impact the composition of the element over its working life and making provisions to accommodate them.

Secondly, the required design criteria should be clearly defined from the outset, identifying any elements needing replacement during the working life. Aligned with the principle of design of Dissasembly, the design should be planned in a way that allows for replacement while ensuring the continued fitness for the use of the structure.

Moreover, careful consideration must be given to the expected environmental conditions for installing the timber components. As seen in Section 3.2, changes in environmental conditions can affect the strength properties of Top-ups, so it is essential to ensure that they are designed to function within the appropriate service class conditions throughout their working life.

Furthermore, choosing a robust structural system is advisable, capable of safely withstanding known design hazards. Ideally, the system should incorporate built-in redundancy beyond the minimum requirements specified by the code, providing extra resilience.

Considering these criteria and implementing measures to address them, applying timber top-ups can result in durable structures that effectively meet their intended purpose over their design working life.

3.5.3 Fire Safety and Resistance

In section 3.1, it is stated that timber is a combustible material. However, as Ramage et al. (2017) explained, .this does not necessarily mean that timber cannot be used for construction due to its combustibility. Timber is often preferred for construction due to its excellent thermal insulation properties. When timber burns, it forms a layer of char that can help protect and maintain the strength and structural integrity of the wood inside. This is why large timber sections can often be used when non-combustible materials like steel require special fire protection.

The requirements for timber structures might vary depending on the type of components and structural system. However, Eurocode 5 looks for minimum fire resistance requirements expressed in terms of the length of time the structural elements must maintain their load-bearing capacity and ability to prevent the spread of fire and smoke.

In the Netherlands, the Fire Building Decree (known in Dutch as the "Bouwbesluit") is the code that regulates fire safety in buildings. Its goal is to prevent casualties and the spread of fire to other areas by maintaining the load-bearing capacity and integrity of a building and its components for a certain amount of time while people evacuate the construction.

Fully understanding the influence of the intended use of in structure, plus addressing the elements that may need replacement during the working life, and ensuring changes in environmental conditions ensures Durabiliy.

Image 11. Due to its combustion properties, timber creates a charring layer that protects the components during a fire.

Therefore, the code provides resistance and regulations based on the function types of buildings, divided into three main categories: housing, utility buildings with sleeping accommodations, and utility buildings without sleeping. The length of the fire resistance required, or the time that the building must maintain its load-bearing capacity and integrity, depends on the function type of the building and its height, as seen in Table 5.

The code states, as well as a design principle, the compartment of fire resistance to avoid a sub-fire or fire compartment collapse, as long as this does not lead to the collapse of structures outside the compartment within a specific time frame. This is to prevent progressive collapse.

In the case of timber top-ups, being a new intervention must be considered the category of a new building, and in many cases reaching the height of 13 meters, therefore a 120-minute resistance must be considered.

Highest floor of accommodation area above measurement level	Fire resistance, expressed in minutes	Reduced fire resistance at a permanent fire load density of ≤ 500 MJ/m^2, expressed in minutes
	60	30
≤ 7 m	60	30
7-13 m	90	90 (no reduction)
> 13 m	120	120 (no reduction)

Table 5. Fire resistance requirements for new housing, according to the Building Decree of 2012. In a building 2.8 mts S.S.L from the fifth floor (14 mts) for new buildings: 120 minutes. Therefore the charring layer or fire resistance should be calculated based on this principle.Taken from (Bouwbesluit 2012)

To employ wood in products with a design lifespan that matches or exceeds the tree growth rotation period. This allows for "sustainable-yield logging" or "cascading" of the material.

3.5.4 Lifespan and EoL in timber structures.

According to Michael H. Ramage et al. (2017), it is important to employ wood in products with a design lifespan that matches or exceeds the tree growth rotation period for sustainable use of wood resources. This allows for "sustainable-yield logging" or "cascading" of the material, as seen in Image 12. Therefore, aiming for a prolonged service lifespan of the structure aligns with the cascade use principle for wood, prioritizing the following order of use: wood-based products, Reuse, Recycling, Energy recovery, and last but not ideal, Disposal.

Furthermore, by analyzing the strategies of the 10 Rs of sustainability framework shown in Figure 3 about the lifespan and end of life of timber for structures, it becomes evident that structural systems have potential as described as follows:

- Reuse: Timber's durability allows its components to be reused in further applications. Design considerations must ensure controlled conditions of moisture, temperature, and ongoing monitoring to ensure the components' physical integrity. The challenge lies in designing long-lasting interface protocols that enable subassemblies to last through multiple generations of products (Allwood, 2014).
- Remanufacture: Mass timber elements can be adapted or repurposed for

different functions or applications within a structure or other projects. For example, timber beams or columns can be reused in different configurations or layouts to accommodate changing spatial requirements. Structural capacities must be within acceptable values according to local codes.

- Repair and Refurbish: Various methods can be employed to repair mass timber elements, restoring their structural integrity and enabling continued use. The specific repair techniques depend on the type of damage or degradation encountered.

- Repurpose: Mass timber elements offer opportunities for repurposing, as their shape and configuration can be altered to serve different functions in future cycles. With their high structural parameters compared to interior layouts or partition walls, structural elements hold great potential for repurposing as they can be reconfigured to serve purposes with a lower load-bearing capacity.

- Energy Recovery: At the end of their service life, the Mass timber structure that cannot be repaired or reused can be utilized as biomass for energy generation through combustion or gasification. This allows for energy extraction from timber while reducing reliance on fossil fuels.

In structural design, developing standardized open architecture and interfaces would greatly enhance the possibility of reusing components and assemblies to maximize their lifespan (Allwood, 2014). By considering these strategies, timber can contribute to a circular economy by extending its usefulness and minimizing waste.

3.5.5 Connections

Design connections between load-bearing members are a crucial aspect of timber structures. According to Michael H. Ramage and Henry Burridge (2017), the size of the structural member can be influenced by the dimensions required to accommodate the connection. This is because edge distances from connectors are established to prevent splitting, which may necessitate additional material in the structure solely to accommodate connections. Therefore, The efficiency of a connection is determined by the ratio of its strength to the strength of the member it connects.

Furthermore, the design of reversible and robust connections in timber top-up structures can enhance their potential for disassembly and reuse. As mentioned by Brancart et al. (2017). Considering potential reconfigurations can significantly increase the reuse potential of components, particularly in kit-of-parts structures.

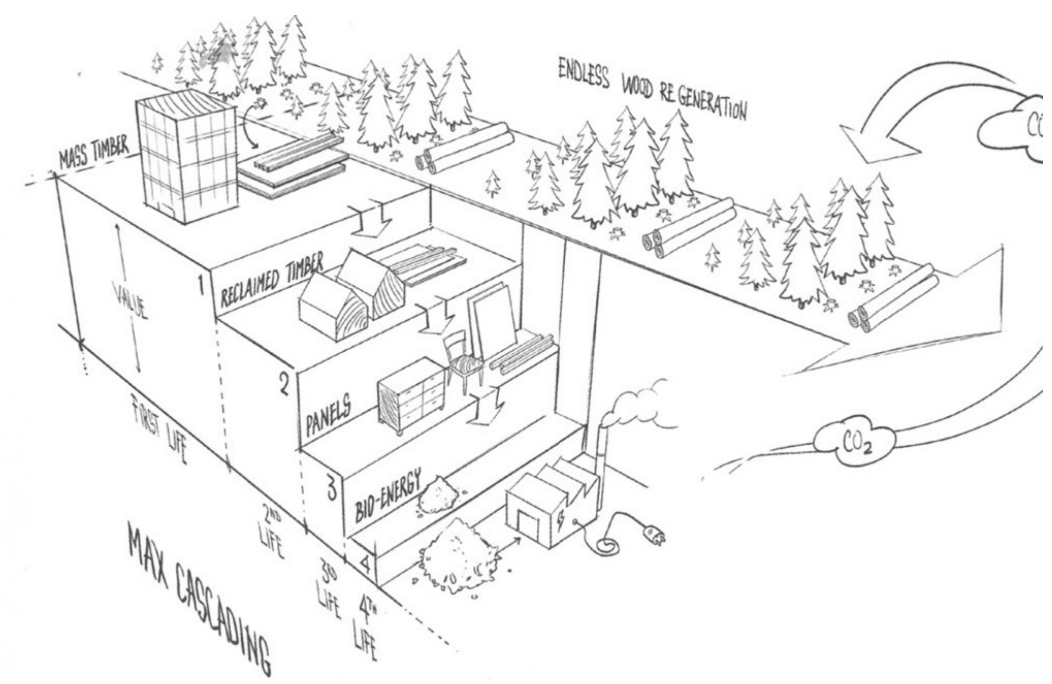


Image 12 Cascade process of mas timber used in buildings, compared to wood growth to create Sustainable yield Logging.

Conclusion Chapter 3

Overall, when considering the design of timber top-ups and structural timber principles, several parameters play a vital role in ensuring the success and sustainability of such structures. It can be concluded from this section the relevance of specific parameters that might help to align the design in the transition of the existing building in a circular process:

First, designing for disassembly is crucial, as it allows for the efficient dismantling and reusing of components, minimizing waste and maximizing resource utilization. Secondly, modularity enhances flexibility and adaptability, enabling easy modifications and reconfigurations as needs change over time. Throd, controlled environmental conditions, such as moisture and temperature, are essential for preserving the integrity and durability of timber elements.

Additionally, incorporating long-span components and structures enhances the efficiency of space utilization and reduces the need for intermediate supports, providing greater design freedom. Lastly, considering fire resistance measures is crucial to ensure the safety and resilience of timber top-ups in the face of potential fire hazards. These concepts applied to the case study will be addressed in Section 5 by incorporating them into the design process of timber top-ups.

04 Case study selection
Research for a building to top-up

4.1 Case study selection

Previous to the Research for the Design stage of the project, the proposal of a case study is required to use as a reference to develop the design of the structural system. Moreover, to respond to the area capacity of housing units stated in Section 1.1, this case study should be a specific residential building in The Netherlands that will be used as an example to Top-up with timber. Based on what has been described so far in the introduction and the literature review, the specific structural criteria defined to select this building to consider the following:

- The selected building should have an RC structure: As explained in the problem statement, the object of study of the graduation project is to analyze how to reduce RC waste.
- Moreover, The selected building should represent a relevant typology of structural systems within the Dutch context. This choice adds significance to the identified problem in the project background and allows for the potential scalability of the proposed design solution during the design stage.
- The structural integrity of the building should be intact and in optimal conditions, ensuring the safety of its current occupants. As outlined in the project's scope and limitations, the primary objective is to explore the potential for increasing the building's capacity. Therefore Retrofitting the existing RC structure will not be considered part of the scope of the graduation project.

4.1.1 PostPost-war Walk-up Apartment Buildings

Post-war Walk-up Apartment buildings in the Netherlands are an important housing solution within the Dutch context, especially for affordable urban housing (Bernard Colenbrander, 2020). These buildings were predominantly constructed between the 1950s and 1970s to respond to the housing shortage following World War II. In today's context, they continue to be popular due to their affordability and convenience.

As of 2016, there were still 409,363 post-war period apartments (1945-1965) in the Netherlands, accounting for approximately 15.1% of the current housing stock. Major cities such as Amsterdam, Rotterdam, The Hague, and Utrecht. It is assumed that many of these buildings consist of walk-up apartments.

According to (Oorschot et al., 2018), these buildings are typically linear structures comprising four to five stories with multiple stairwells, each leading to eight apartments and a shared front door. They feature outdoor galleries for circulation, and their roofs are generally flat or slightly pitched. Additionally, the ground floor often provides storage space.

More specifically, in the 60s, larger European construction companies introduced more sophisticated building methods and construction systems to meet the urgent demand for housing. As a result, various reinforced concrete (RC) industrialized methods with similar structural typologies were employed in Walk-up apartments throughout the country.

Similar structural typologies within specific regions can be attributed to agreements between contractors and local governments. Moreover, this arrangement aimed to minimize transportation costs between production sites and building locations, leading to the typological consistency of RC structures in dwellings constructed during this period.

4.1.2 Importance of Postwar Walk-up Apartments in the Dutch Context.

Walk-up Apartment buildings are often constructed to high standards of quality and durability. They are designed to last for many years, making them a sustainable option for housing. Therefore they are proposed for the case study selection because by prolonging their lifespan instead of demolishing it is possible to preserve and even enhance specific positive characteristics, as mentioned by (Oorschot et al., 2018), such as :

Image 13 Examples of Post-war Walk-Up Dwellings in The Netherlands.Top: Post-war apartment building in the Marco Pololaan in Utrecht & Apartment building in Proosdijerveldweg in Tilburg. Bellow: Robijnhof Utrecht& Post-war apartment building in Gillisbuurt in Delft.

- Affordable Housing: With rising housing costs in urban areas, many people in the Netherlands are turning to these buildings to find affordable and centrally located housing.

- Efficient Use of Space: This typology was designed to make the most efficient use of space. With multiple floors and apartments accessed through shared stairwells, these buildings can accommodate many residents without taking up much space.

- Urbanization: As urbanization continues to be a trend in the Netherlands, These buildings are often located in urban areas, making them an ideal choice for people who want to live close to work, school, and other amenities.

- Community Living: Living in a Walk-up Apartment building can provide a sense of community and shared living. Residents can build relationships and create a sense of belonging with shared spaces and common areas.

For this reason, Postwar Walk-up Apartment buildings remain a relevant and vital housing option for society in the Netherlands even 70 years after construction.

4.1.3 The neighborhood of Gillisbuurt, district of Buitenhof, Delft.

The residential complex in the neighborhood of Gillisbuurt, located in the south of the city of Delf,t is an example of a Walk-in apartment with an ERA-1 system. This neighborhood, located in the district of Buitenhof's, is a residential area surrounded by considerable greenery, composed of 14 Walk-in buildings with five stories distributed along the block and around massive green areas with vegetation.

The document called "Integral exploration of the Buitenhof area-opportunity cards by the Delfts Geemente" set up a plan for this district where several spatial interventions are proposed to improve the socio-economic conditions of the people that live there. Among the most relevant ones can be found: The construction of a heat grid, proper drainage for the green areas, an update of sewage systems, and the demolition and construction of new residential complexes.

For the neighborhood of Gillisbuurt, the document proposes an intervention where the two towers on the Chopinlaan (blocks O and P) are demolished to create space for a new development that will enhance the liveliness and attractiveness of the neighborhood, in this case, Single-family homes with a front garden in order to create visibility from the courtyards on where the Haydnlaan where an urban intervention is proposed as can be seen in Image 15.

The reason behind the decision to demolish and to create more visibility to the inner courtyard follows the goal, as stated in the same document, to tackle specific challenges that this neighborhood has compared to other ones in Delft.

According to data provided by the municipality of Delft, the Gillisbuurt neighborhood exhibits a relatively high concentration of low-income residents compared to other areas within the city. In 2019, Gillisbuurt had a household low-income rate of 39.8%, surpassing the average for the municipality of Delft. This suggests a greater prevalence of economically disadvantaged households in Buitenhof compared to other neighborhoods in Delft.

While the graduation project's main focus does not involve an in-depth analysis of the socio-economic conditions of Gillisbuurt's residents, it is important to highlight these issues in this section. They serve as contextual inputs and aid in decision-making regarding strategies for the architectural proposal within the project. Consequently, the two towers located on Chopinlaan Street, slated for demolition, are chosen as a case study to guide the design phase of the graduation project as these towers are composed of RC structures. The characterization of the building itself will be further discussed in section 5.2

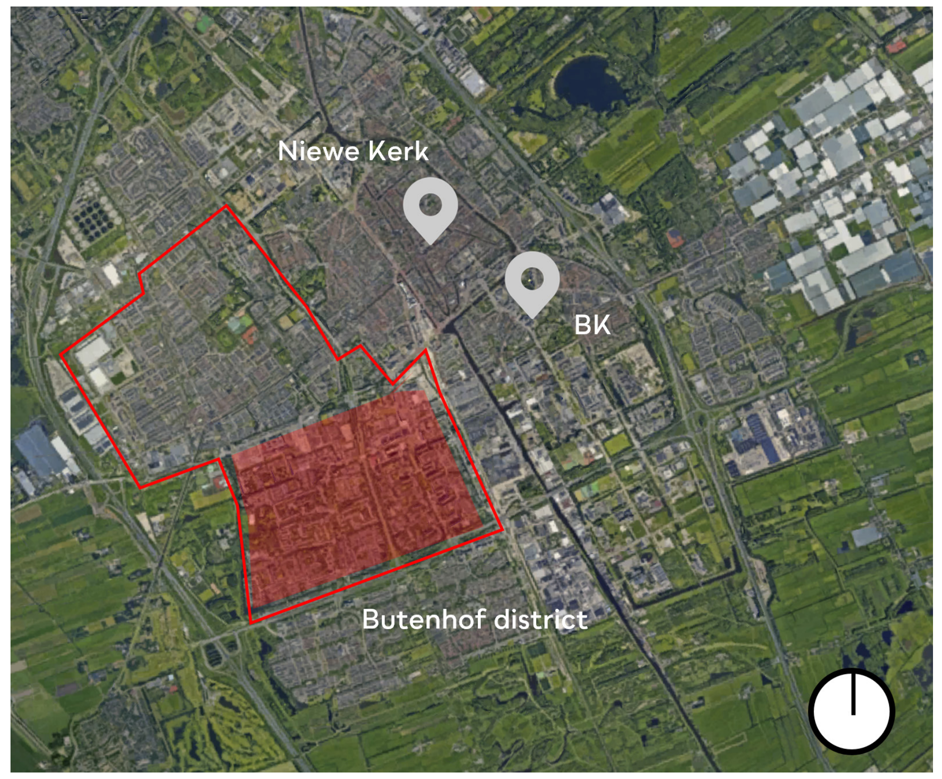
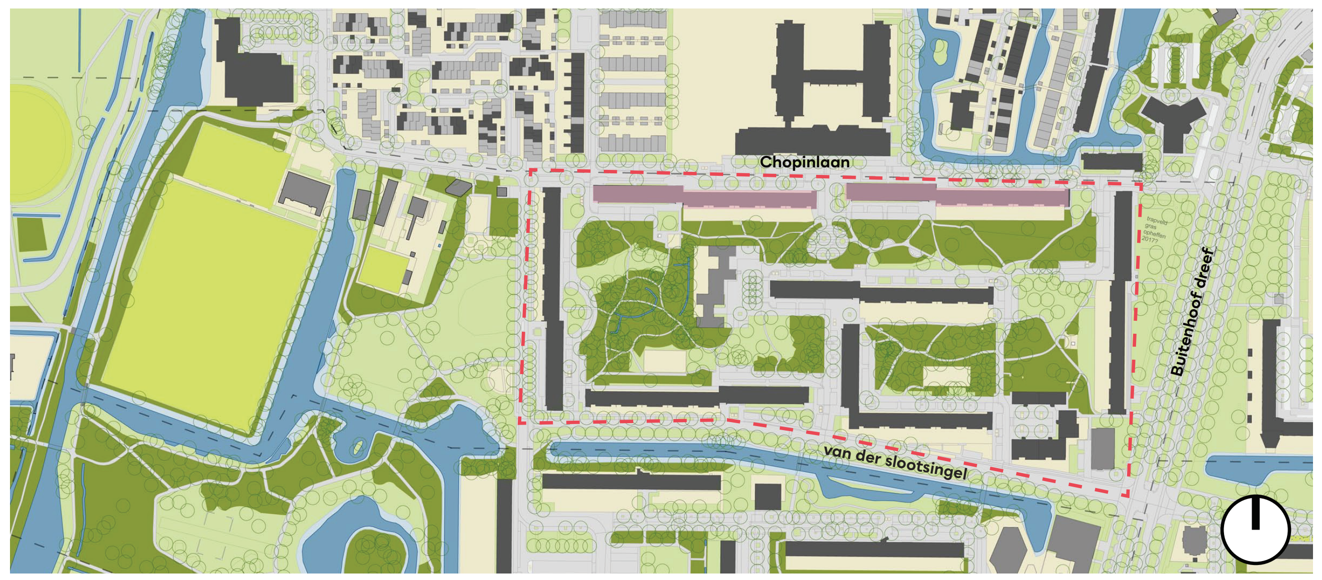


Image 15. Location in Delft of the study case: Gillisbuurt residential complex.

Top: Location in the city of Delft.

Bottom: residential complex of Gillisbuurt in the Buitenhof distric.



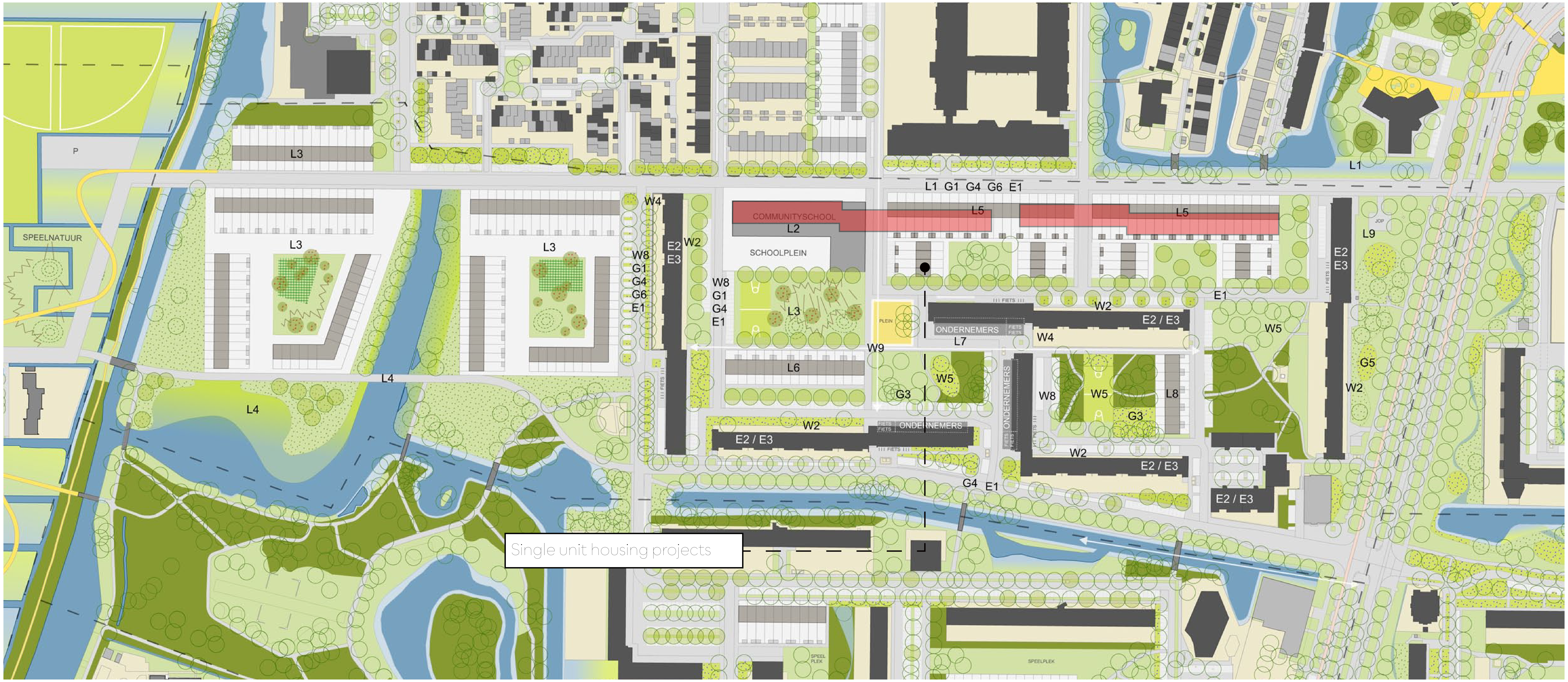


Image 16. Plan for the Gilisburrt neighborhood according to the "Integral exploration of the Buitenhof area-opportunity cards by the Delfts Geemente." In red opacity are marked todays location of the Walk-in apartments that will be subjected to demolition.

05 Proposal **Research for design**

As seen in Section 2, there needs to be more literature about construction methods for timber top-ups in existing RC buildings. As seen in Section 3.5, a reason that might relate to the use of timber in structures for highrise constructions is a relatively recent practice, which means that its application is still experimental. Consequently, this prompts a challenge to approach the research question: How to increase the area capacity of existing Reinforce Concrete structure buildings and extend their lifespan by using timber top-ups? as there are limited documented data and literature that would help to analyze what are its benefits and limitations of Timber Top-ups.

Moreover, insufficient literature regarding topping up timber directly affects SQ.3: How to make topping up in the decision-making for stakeholders when facing demolishing/building new? No theoretical background is available to base the feasibility of a top-up timber structure. Therefore, the graduation project focuses on producing specific approaches aiming to fill this gap :

First, a proposal of a design framework that will document the design process necessary to assess the feasibility of timber Top-up. Furthermore, to define the inputs and hard constraints necessary to evaluate the possibilities and limitations of the strategy.

And second, the design of a timber top-up structure for the study case described in Section 4.1.3. The reason for designing a specific structure for the case study is to include, from a practical perspective add, inputs to the design framework. Moreover, to ensure this framework is coherent.

Therefore, In the case of the graduation project, the design will be conducted in parallel, as relevant findings might appear during the design of the top-up that complements the design framework.

5.1 Approach in the proposal of a Design framework for Timber top-ups

A design framework is a step-by-step definition defined in a diagram that defines the inputs and outcomes necessary to assess an objective. In the case of the graduation project, a diagram that aims to approach SQ. 3 by documenting the steps in the design of a timber top-up.

Therefore, a graphical tool capable of organizing processes is required to construct the design framework. In his book: "Workflow Modeling: Tools for Process Improvement and Application Development," Sharp & McDermott (2009) states that a swimlanes diagram provides a clear and structured view of how various activities are performed by different individuals, teams, or departments involved in the process. In this case, assuming the decision-making of a Timber Top-up requires the analysis of different stakeholders from different fields, it might be an effective graphical tool to organize the process.

As defined by the author, the process starts by listing the steps of the process. To do so, a diagram called the "augmented scope model" (Diagram 3), aims to recognize the success of a process by understanding and addressing the broader context in which the process operates. Moreover, this model allows for a more comprehensive process analysis by considering the interdependencies, inputs, outputs, and interactions between the core process and its surrounding activities. The defined steps for the design framework to analyze the feasibility of timber top-ups are defined as follows:

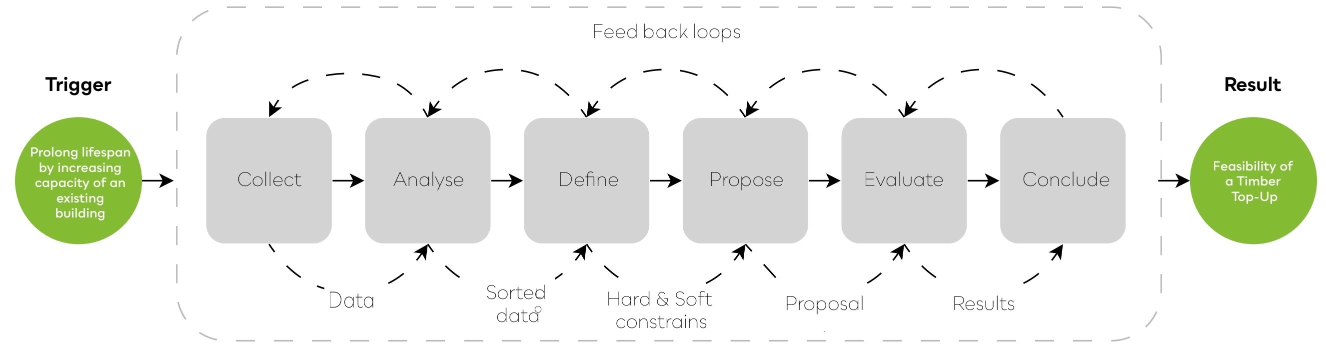


Diagram 3. Proposal of augmented Scope model in the decision-making of TimberTop-Ups. Based on (Sharp & McDermott 2009)

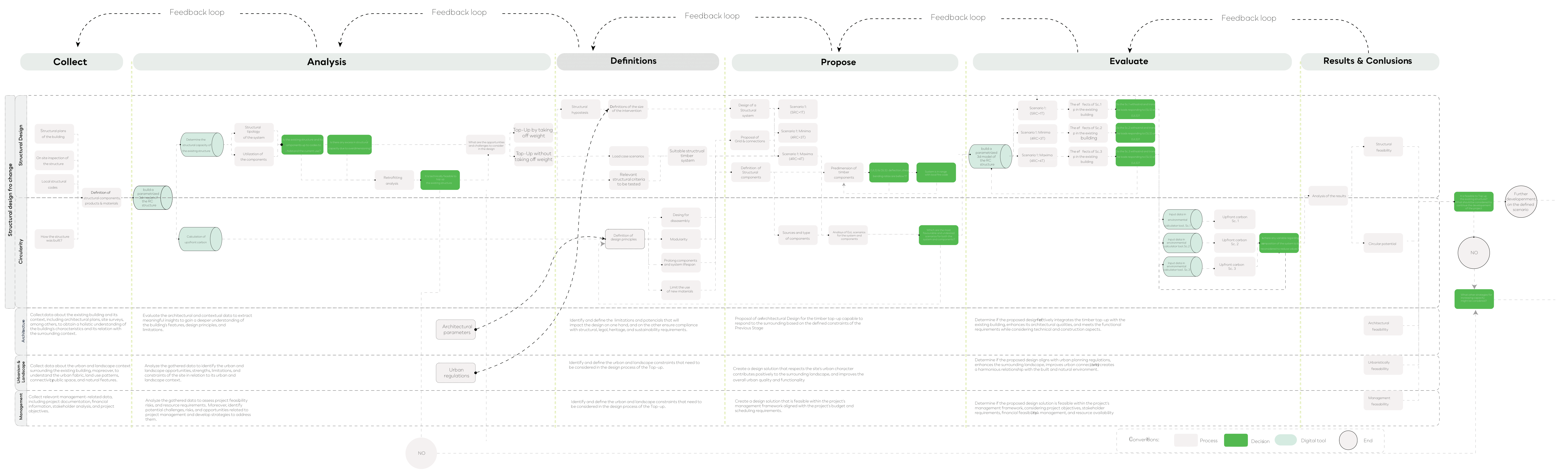


Diagram 16. Swimlane diagram for the Design framework in the desition making for timber top-ups.

- The Collect stage comprises a series of processes to establish the foundation for analysis by developing a contextual understanding of the case study. It involves gathering all available data relevant to the subject matter. Additionally, this stage may generate additional information necessary for subsequent stages but is currently unavailable.

- The Analysis stage involves filtering and analyzing the data collected during the previous stage. This stage aims to organize the data based on its relevance and importance in the decision-making process for Timber Top-ups.

- Definition stage: In this stage, the hard and soft constraints are defined based on the data filtered and organized in the previous analysis stage. These constraints are requirements that the Timber Top-up proposal in the case study must meet. The defined constraints can include quantitative and qualitative aspects, which will be evaluated in the following stages.

- Propose stage: In this stage, the defined hard and soft constraints act as input for designing a Timber Top-up for the case study. Considering a broad range of constraints during the design process enables more accurate results in the subsequent evaluation stage, resulting in a more precise and well-supported proposal. Moreover, this approach allows for exploring multiple architectural or engineering proposals in different scenarios, facilitating a comprehensive evaluation.

- Evaluate stage: The previously proposed scenarios undergo testing in the evaluation stage. This involves assessing the performance of the defined criteria for each iteration using digital tools. Parametric design tools can be employed in this assessment to evaluate different iterations and optimize the results in each proposed scenario.

- Results stage: In this final stage, the evaluation process yields preliminary results for each scenario based on the specified criteria. These results are then compared across the different scenarios to gain insights into their potential and constraints. Through this comparison, a discussion can be facilitated to assess the feasibility of implementing a top-up solution for the specific case study.

Based on the augmented scope model illustrated in Diagram 3, a swimlane diagram is proposed where the are specified necessary activities to fulfill the goal. As can be seen in Diagram XX, each dependency to be analyzed for topping up is assigned a "swimlane" or a horizontal lane. These swimlanes are typically arranged vertically, representing the process's sequence of activities or steps where the responsibilities of the different dependencies are visually separated, allowing for a better understanding of the handoffs and interactions.

It is relevant to mention that the design process for the graduation project will primarily focus on the dependencies of Structural Mechanics and Circular Design. However, it's important to note that analyzing the feasibility of a Timber Top-up is not limited to these two fields alone. Considering additional constraints from other fields of study can lead to a more specific and well-founded proposal. Moreover, the design framework should include the analysis of other relevant dependencies, such as Urbanism & Landscape, Architecture, and Management, which may contribute significant findings to the process. Although these areas of knowledge will be mentioned in this research, their development should be further explored in future studies.

5.2 Processes followed in the design of a timber top-up

The following section describes the processes illustrated in the design framework in diagram XX. Moreover, in parallel, the design process of the timber top-up for the Gillisbuurt residential complex will be differentiated in italic text font.

5.2.1 Collect

The first step in the design Framework focuses on collecting the historical records, drawings, and building details in the case study. This involves researching to locate and obtain relevant historical records, such as construction documents, architectural plans, and engineering drawings concerning the target building to be Topped Up.

Collect historical records of structural drawings and existing detailing.

Besides gathering the architectural data, gathering information related to the concrete elements' composition and dimensions is relevant. These data serve as a valuable resource for understanding the structural characteristics and constraints of the building, which will help in the subsequent stages of the assessment to design the timber top-up structure.

Applied to the case study, The historical archive of the city of Delft (Stadsarchivedelft) keeps all the available documentation related to the city's buildings. Moreover, for the case study building mentioned in Section 4.1.4: Walk-in apartments in Chopinlaan Street (block O&P), there was found an extensive database with the original Architectural drawings (Plans, sections, and facades) as the original structural proposal (composed of structural plans, details, sizes of components, and some specifications of the materials used back then in the construction). All of the relevant data that might be useful for the design of the Timber Top-up will be mentioned in the following section, 5.2.2 Analyse. However, all of the gathered information found in the archive of the city of Delft can be reviewed in the appendix section (Appendix A) at the end of the report.

On-site inspection of the existing structure.

The next activity in the process involves conducting an on-site inspection of the building to gather field information; this activity aims to validate and inspect the building drawing and specification with reality.

Moreover, examine the structural physical characteristics and conditions. Furthermore, if technically possible, as in The case study of the Karel Voorman building described in Section 2.4.1, destructive and non-destructive testing can be performed in the structure to validate the main physical characteristics of the structural components such as beams, columns, and foundations, paying attention to any visible signs of deterioration, damage, or structural weaknesses.

Applied to the case study, an on-site inspection was conducted at the Walk-in apartments on Chopinlaan Street in blocks O & P. The purpose of the inspection was to visually assess the overall project, including one of the housing units. Due to the building being occupied, it is impossible to conduct structural testing to determine the physical characteristics of its components. However, based on the visual inspection during the visit and the fact that the building is functioning normally, it can be inferred that the structure is in good condition. Therefore, for this graduation project, it will be assumed that the processes involved will produce acceptable results regarding the structural integrity of the components.

Analysis of the design codes: Timber and concrete.

The following process involves determining and analyzing the local structural codes and regulations that apply to the design of the timber Top-up structure. Particularly those related to timber construction and structural engineering, to ensure that further steps for the design intervention are aligned with the local safety standards.

In the Netherlands, the design and construction of structures are governed by the structural code Eurocode. Moreover, the NEN-EN 1995-1-1 (EC5) and NEN-EN 1994-1-1 (AC4) standards are proposed to design timber and composite structures.

In addition to Eurocode, the Building Decree (Bouwbesluit) sets out the minimum requirements for building safety, health, usability, and energy efficiency. Moreover, this code states the norms required to ensure fire safety and other aspects relevant to the combination of timber and concrete in structures.

Besides the information provided by the Delfts historical archive and the report: "Integral exploration of the Buitenhof area-opportunity cards" mentioned in Section 4.1.1 by the Delfts Geemente, no other relevant published data was found during the conduct of this research about the case study.

However, some projects conducted by the University of Delft by the S.W.A.T studio from the Building Technology Master track have researched and performed proposals to improve the social conditions of the neighborhood's inhabitants by proposing specific interventions in the public space. These testimonies were included in Section 4.1.1. and might help in further steps as input for the architectural design.

5.2.2 Analyse

Analysis of the existing building, including the structural typology, components, and materials.

Starting the analysis stage, this first process aims to characterize the existing building followed to determine the load-bearing capacity of the whole structure to understand its current state and identify its strengths and limitations. Moreover, to understand the configuration of the existing structure to intervene.

The analysis of the case study building is as follows described:

Walk-in apartments in Chopinlaan Street (block O&P) in Gillisbuurt.

Year of construction: 1968

Construction company: Ir. E.J.A CORSMIT c.i

Use: Residential dwellings.

Number of stories: G+4.

Number of households and typologies: 64 households, 75 m2 each, with a terrace of 11.2 m2 facing South.

Orientation: North-South.

Overall dimensions: 162 m (length), 12,89 m (width), 14 m (Height).

- *Characteristics of the existing structure*

The Gillisbuurt blocks have a structural system that consists of in-situ casted concrete frameworks connected by RC prefab floors, creating a strong and stable framework. The construction is based on a grid pattern with center-to-center distances of 3.90 meters and 5.40 meters. At the same time, the floor-to-floor height is 2.8 meters, with a variable depth between 10.64 meters and 9.74 meters, depending on the type of structural cell. The casted walls and prefab slabs in the floors have a thickness of 180 mm.

Each framework accommodates 80 cells, of which 70 are designated for dwellings. The remaining ten cells are on the ground floor and serve as storage and utility rooms.

- *components and materials:*

As mentioned in the previous section, the main components of the existing concrete

Image 10. Axonometric of the case study: Gillisbuurt dwellings in Delft.



Images 11. Photos taken from the outside, front left to right: 1. View of the facade from the Chopinlaan. 2. View of the west facade of the building. 3. Detail view of the south facade.

structure are two: The concrete prefab slabs that were produced back then by the company E.J.A CORSMIT c.i in their facilities in Bergambacht in Zuid-Holland. And the cast in situ walls with steel reinforcement.

According to the archived structural specifications (Appendix A), the specified concrete for the project was K300. In current standards, specifically NEN-EN 1992 and Eurocode 2, this specification translates to a concrete type known as C19/22. This concrete type primarily consists of Portland cement, sand, aggregates, and water in a mix ratio of 1:2:3.For the reinforcement steel bars, a reference code QRn48 was used, indicating steel with a yield strength of 480 MPa.

In addition to the main structural components, the terraces, gallery access, and supporting cantilever beams are prefabricated elements that adhere to the specifications mentioned above. The comprehensive list of components and their corresponding composition of raw materials can be found in Diagram 5.

Parametrization of the existing model as a base for the analysis

In the following process, all the gathered data (construction drawings, architectural plans, technical reports, literature, and any other available documentation in the previous stage) is sorted and analyzed to Parametrize the existing building into a 3d model using Rhino 7 and Grasshopper. Moreover, define the parameters to be assessed and controlled in the proposal stage.

In the Study case, a script in Grasshopper was developed based on the available planimetry of the structure. One of the main characteristics of the script was the possibility of removing the number of layers of the RC structure with a slider, as seen in Image XX.

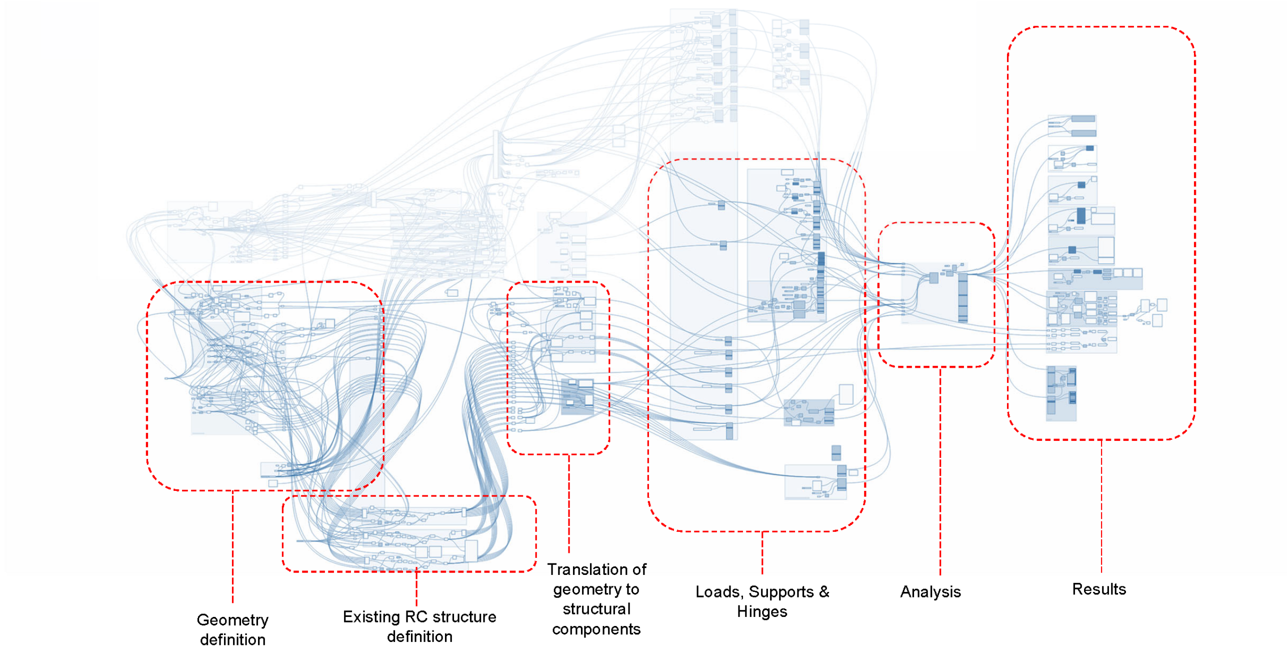


Figure 16 Script definition of the parametrized model. Top: Planimetry from the building extracted from the 3d model

Calculation of embodied energy and embodied carbon of the existing structure.

The subsequent step involves the analysis of the building's life cycle, specifically focusing on the Upfront embodied carbon throughout its entire first cycle and on understanding the environmental impact of the building's construction, operation, and end-of-life potential.

Therefore, factors such as the energy intensity of the materials, the distance traveled during transportation, and the energy required for on-site construction activities are considered.

In the study case, this analysis was achieved by using the Granta Edupack software with the EcoAudit feature, in which, by inputting the components of the existing structure, it was possible to obtain the current CO2 footprint of the project. Moreover, as seen in graph XX, the feature of EoL potential is used to analyze three possible scenarios to process concrete: Downcycle, recycle, and Reuse.

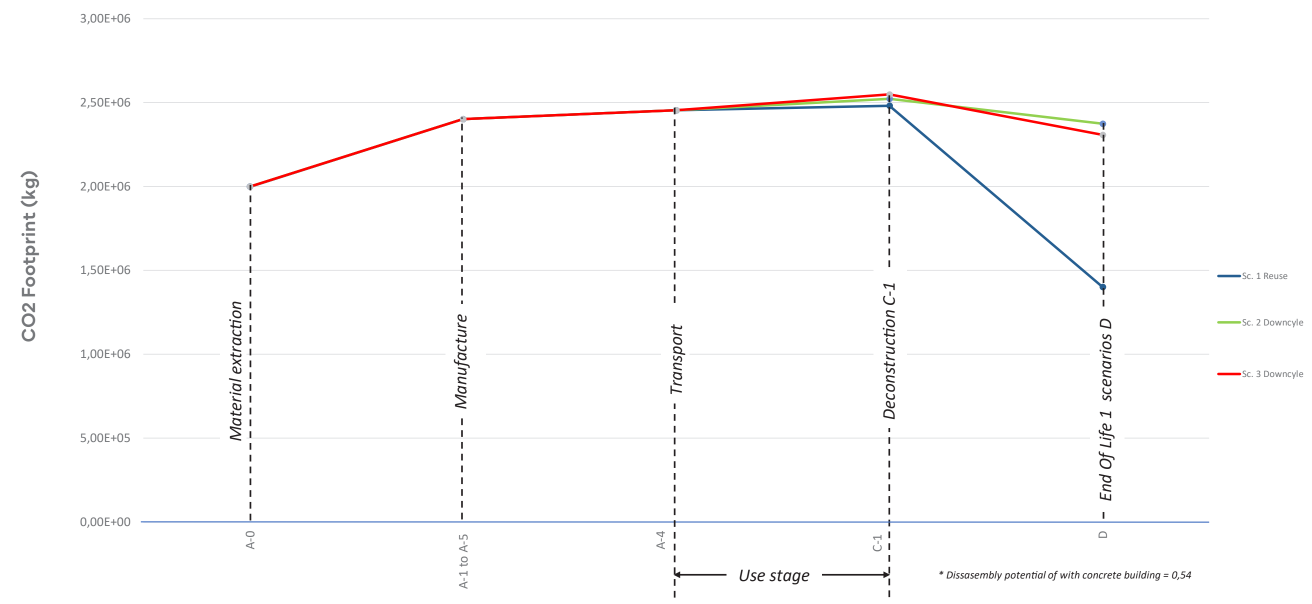


Diagram 16. Upfront carbon of the existing RC structure for the Gillisbuurt residential complex with three possible scenarios to manage the waste if deconstructed.

It is essential to clarify that the values used by Granta Edupack and illustrated in chart XX are based on a database fed with numbers taken from literature review on different research papers about LCA analysis. Therefore, these numbers are reference values that might be further compared with a specific LCA of the project that would consider all the construction variables in the construction of the building.

Analysis of the structural capacity of the existing structure and its components.

This analysis helps determine the load-bearing capacity of the whole structure to understand the building's current state and identify its strengths and limitations. Moreover, this analysis looks to compare the structural capacity of the existing building to support and transfer the expected loads and design requirements of the proposed timber top-up structure.

It's relevant to highlight that in some cases in The Netherlands, RC structures might be over-dimensioned to bare the current load cases to comply with safety factors of out-to-date design codes, conferring the existing structure an additional capacity to withstand and transfer some additional loads.

In the study case, the structural feasibility of top-up relies heavily on the structural capacity of the concrete vertical components of the structure. Therefore, using the software Karamaba 3d, the structural utilization of the RC walls in the existing structure is analyzed, as seen in Image XX.

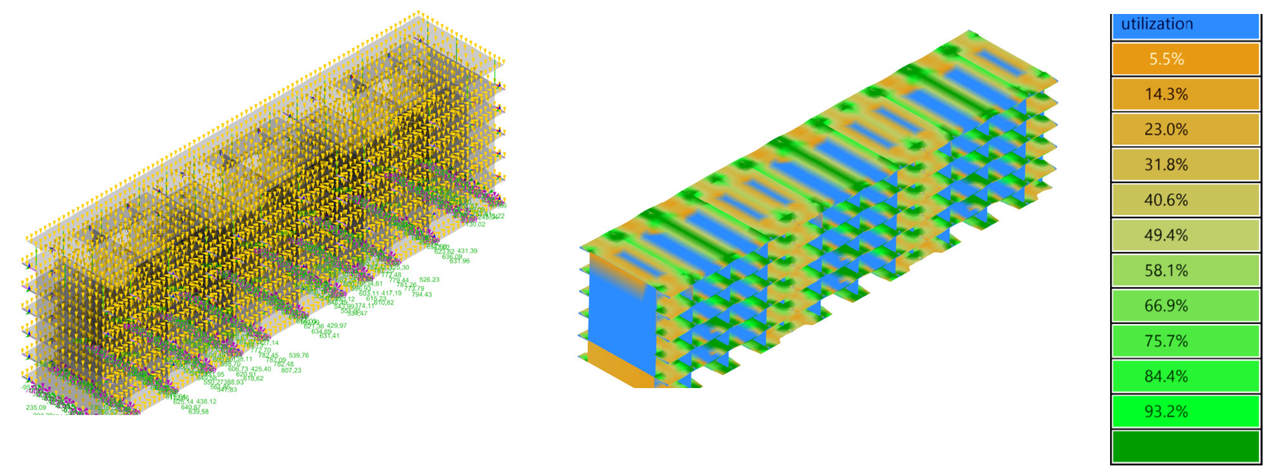


Image XX. To the right: Render showing the existing utilization of structural componentes of the Parametric model of the Gillisbuurt RC structure today. To the left: reactions in the foundations.

As seen in the graph, walls have an overutilization capacity in those elements allowing them to withstand a major load case. Further research is needed to determine if this over-dimensioned capacity is also present in the foundation piles. It's worth noticing that the loads can increase by 40% in the largest load cases.

5.2.3 Define

Definition of structural assumptions.

The next stage in the process focuses on defining the structural assumptions based on hypothetical scenarios of building Timber Top Ups in the existing structure. Moreover, to make a guess based on previous knowledge in structural design about how the entire structural system will work, especially considering the impact of the timber top-up on the existing reinforced concrete structure.

In the study case, these assumptions, along with predefined structural parameters, include as seen in Figure 16:

a. Constructing a lightweight structure on a rigid concrete base may result in concentrated tension stresses, particularly in the reinforced concrete walls beneath the timber top-up.

b. Timber top-ups at certain heights are exposed to significant wind loads compared to the original building, necessitating stabilizing elements to withstand and transfer horizontal loads.

c. To avoid creating differential load reactions in the foundation that could destabilize the structure, the weight and loads of the top-up should be uniformly distributed, ensuring even loading of the existing structure.

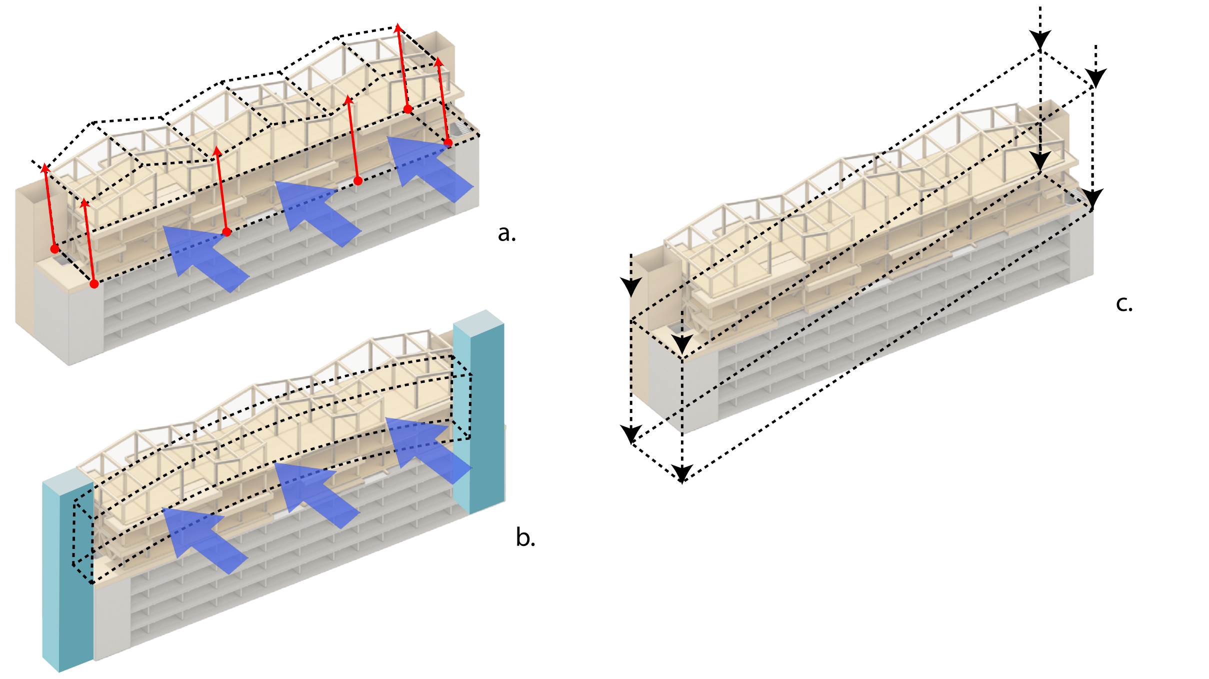


Figure 16 Definition of structural assumptions.

Definition of load case scenarios and structural situations.

This stage focuses on determining the specific load cases that will be considered during the design of the timber top-up structure. Moreover, these load cases are defined based on factors such as the structure's intended use, location, and architectural constraints. Therefore, various load scenarios can be proposed throughout the lifespan of the second cycle of the building.

The defined load cases are also utilized in the preliminary dimensioning for the structural elements. This means that the loads determined in this stage are also used to estimate the initial sizes and capacities of the timber components.

Figure 17 illustrates the load case scenarios used in the project based on the defined Live loads, dead loads, snow loads, and wind loads for the intervention. The calculation for the wind load is explained in detail in Appendix D.

Load case		Load
LC1	1.35 g_k	2,7 kN/m2
LC2	1.35 g_k + 1.5 q_k	11,7 kN/m2
LC3	1.35 g_k + 1.5 s_k	4,0 kN/m2
LC4	1.0 g_k + 1.5 w_k	4,9 kN/m2
LC5	1.35 g_k + 1.5 w_k	5,6 kN/m2
LC6	1.35 g_k + 1.5 q_k + 1.5 * 0.6 * s_k	12,5 kN/m2
LC7	1.35 g_k + 1.5 q_k + 1.5 * 0.5 * w_k	13,1 kN/m2
LC8	1.35 g_k + 1.5 * 0.0 * q_k + 1.5 * s_k	4,2 kN/m2
LC9	1.35 g_k + 1.5 * 0.0 * q_k + 1.5 * w_k	5,6 kN/m2
LC10	1.35 g_k + 1.5 * 0.6 * s_k + 1.5 * w_k	6,3 kN/m2
LC11	1.35 g_k + 1.5 * 0.5 * w_k + 1.5 * s_k	5,4 kN/m2
LC12	1.35 g_k + 1.5 * 0.0 * q_k + 1.5 * s_k + 1.5 * 0.5 * w_k	5,4 kN/m2
LC13	1.35 g_k + 1.5 * q_k + 1.5 * 0.6 * s_k + 1.5 * 0.5 * w_k	13,9 kN/m2
LC14	1.35 g_k + 1.5 * 0.0 * q_k + 1.5 * 0.6 * s_k + 1.5 * w_k	6,3 kN/m2

Live load	g_k	2,00 kN/m2
2	kN/m2	Internal: Class A Residential EC 0
3	kN/m2	Balconies: Class A: Residential EC 0
Dead load	**q_k**	**6,00 kN/m2**
3	kN/m2	Partitions in lightweight CLT walls.
1,5	kN/m2	Mechanical services (HVAC, plumbing, etc.).
1,5	kN/m2	Facade system (windows, doors, etc.)
Snow load	**s_k**	**0,85 kN/m2**
0,85	kN/m2	Zone 1 of the Netherlands
Wind load	**w_k (h existing)**	**1,90 kN/m2**
1,9	kN/m2	
Wind load	**w_k (h Top up)**	**2,45 kN/m2**
2,4	kN/m2	

Figure 17 To the left: Definition of load case scenarios in the project. To the right: Values of dead, live, snow and wind loads.

Definition of structural criteria based on the Limit state design method (LSD).

This stage aims to select and analyze structural criteria derived from the Limit State Design method, commonly used in structural codes like Eurocode. This method encompasses structural criteria such as Yield strength, buckling, stability, fracture due to fatigue, deflection, vibration, fatigue, and fire resistance in the Ultimate limit state (ULS) and service limit state (SLS).

The aim is to identify the most relevant criteria that will enable a preliminary structural assessment for evaluating the feasibility of timber top-ups. By examining and sorting these criteria, the design team can establish a comprehensive framework for assessing the structural viability and performance of the proposed timber top-up, ensuring compliance with safety and design standards.

For the case study, five criteria are decided to be evaluated, the first three for the RC existing structure related to Ultimate Limit State (ULS):

Reactions in the foundations after Toping-up: To evaluate if there are differences in the reactions of the foundation after and before topping up.

Utilization of the components in the concrete structure: To evaluate if the utilization of the concrete elements is in admissible ranges after the top-up intervention.

Analysis of concentrated stresses in the RC walls: To evaluate if affectation is due after the top-up placement.

And there regarding the structure of the timber top-up:

ULS - Utilization of the Timber top-up components (maximum shear and moment): To determine the component's response to the structural requirements in the load case scenarios.

ULS – Fire resistance for the top-up to withstand 120 minutes. As can be seen in Graph XX

SLS - Deflection limit: To control the displacement of structural components of the top-up are in admissible values.

Charring layer calculation					
Notional charing depth	def=dchar+ko*do	72,0	mm		
Fire resistance 120 min	R120	120	min		
Charring rate C24 - Spruce		0,65	mm/min		
Charring rate PUR		1,3	mm/min		
Coeficcient of duration	k0	1			
line) with zero strenght and stiffness	d0	7	mm		
Charing layer depth	dchar= sum layers	65,0	mm		
Components Layer analysis					
Charin layer		d [mm]	t [min]	ttotal [min]	
Gipsum board	tch=2,8 hp-14	20,0	42	0	
Layer 1		30,0	46,2	88,2	
Layer 2		30,0	26,9	115,1	
Layer 3		5,0	7,7	122,8	

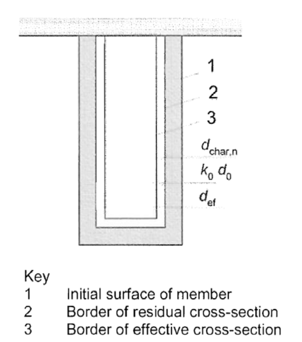


Figure 17 -Calculation of the charring layer for beams.

Definition of circular design principles

The next step in the process focuses on determining criteria that will be utilized during the proposal stage to facilitate the transformation of an existing building using circular principles for the intervened building. To establish these criteria, two specific frameworks from two researchers are proposed and used as a reference:

First, the 10 "R" framework of sustainability of Potting et al. (2017) (as seen in Figure 3 on page 10), where the authors characterize key principles of circular design to transform an existing building using circular principles such as: Reduce, Reuse, Recycle, Repair, and Recover of the components.

Second, the work of Ljungberg, L. Y. on Materials Selection and Design for the Development of Sustainable Products (as seen in Table 6 shows a brief theory recap of LSD focusing on the main relevant criteria assessed in the Ultimate limit design (ULS) and Service limit state (SLS).), where are provided valuable insights into sustainable material choices and design strategies that can be applied to enhance the circularity of the building intervention such as material life cycle assessments, the use of renewable materials, and eco-friendly manufacturing processes.

Concept	Characteristics
Eco-design	This is also known as Design For the Environment (DFE).
Modular design	Easy repair and change of components are important. *e.g.*, parts in copying machines and computers.
Design for material substitution	Substitution of materials with high environmental impact to more superior materials in terms of sustainability.
Waste source reduction design	Reduce the amount of material both in terms of the product itself and packaging.
Design for disassembly (DFDA)	A product should be easy to disassemble, *e.g.*, snap fits, mechanical locks, *etc.* in order to recycle the materials.
Design for recycling (DFR)	DFR focuses on maximum recycle-ability and a high content of recycled material in the product. Different materials should not be mixed if not necessary and different parts should be labelled for easy materials separation.
Design for disposability	Assures that non-recyclable parts or materials can be disposed in an ecological way.
Design for reusability	Focuses on possible reuse of different components in a product. The reused parts could be freshened up and reused.
Design for service (DFS)	The design of a product is made here in order to obtain easy service from the outer regions.
Design for substance reduction	Undesirable substances, which are used during the products' life cycle, should be minimized.
Design for energy recovery	The design here is made with materials suitable for burning with a minimum of toxic or harmful emissions.
Design for life extension	Reduced waste through prolonged life for components or products is the aim of this strategy.

Table 6. *Materials Selection and Design for the Development of Sustainable Products by Ljungberg, L. Y.*

Based on the these approachs to circular design principles,, hard design constraints are proposed to be incorporated in the design:

Design for disassembly that facilitates the application of reuse, repair, remanufacture, and recycling strategies into the Top-up, moreover, on conferring the capacity to structural components and products to be easily assembled and detached by limiting the amount of bounding agents by using as many as mechanical and dry connections as possible. In that sense, it allows the easy recovery of products, parts, and materials when a building is disassembled or renovated.

The possibility of configuring different space plan layouts by creating open floor plans. Reducing the section of structural elements as much as possible is possible to create different architectural program arrangements. Therefore, prolonging the lifespan of the building by allowing different space plan configurations to host different uses through its life cycle.

Efficiency in the use of material: Limiting the volume of material waste in the production of the components of the structural systems and responding to the structural requirements without over-dimensioning elements.

Prolong the lifespan of the components by protecting them from degradation. Moreover, in mass timber structural components, how to protect them from external agents that might affect their physical and mechanical properties. As timber is an organic material, these properties can significantly change if exposed to differential moisture and temperature conditions over time. Additionally, it's necessary to control biological risks during the use period for the elements to be in the structural ranges that Eurocode 5 states.

To limit materials with a high Upfront embodied carbon in terms of CO2 emissions and to enhance the use of one's energy compared to the traditional ones. Although the core of the thesis focuses on this statement, the design can be applied by opting for structural systems, components, and connections that limit the amount of these materials. As seen in Table XX, timber and organic materials (on the bottom of the pyramid) have a lower environmental impact than steel and concrete (on top).

5.2.4 Propose

This process focuses on the design of the entire timber top-up structure. It integrates the pre-dimensioned structural components into a cohesive system that fulfills the project requirements. The proposal encompasses the arrangement and connection of the timber elements, considering factors such as load distribution, stability, and overall structural integrity.

Definition of the grid and supports

This process involves designing the structural grid distribution and support system for the timber top-up in the existing structure. The objective is to ensure that the new loads imposed by the top-up are effectively transferred to the underlying structure. The design of the structural grid and supports significantly influences the overall structural system and the selection of appropriate structural components.

The existing grid of the Gillisbuurt building of concrete walls separated 5.4 mts and 3.9 represents an opportunity to create a lightweight system. For the case study, the grid of the existing concrete structure described in Section 5.2.2 is used as a base to propose the timber top-up grid, as seen in diagram XX.

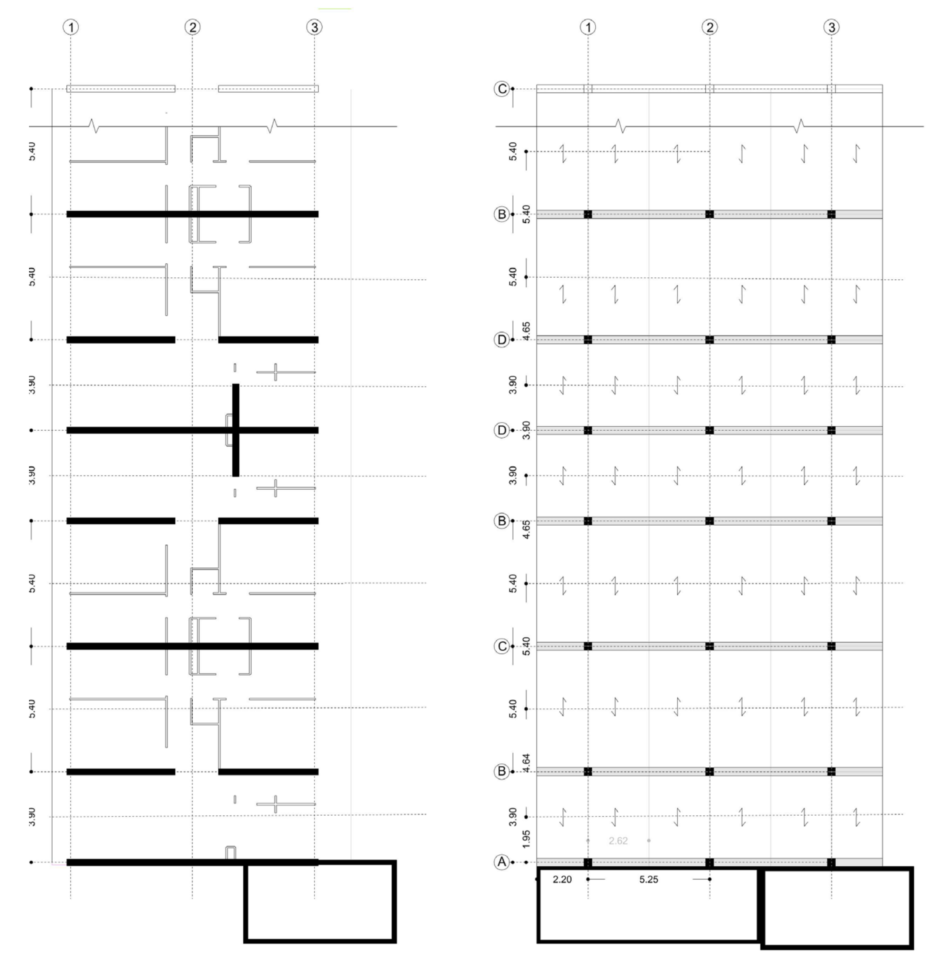


Image XX. To the left: Floor plan of the existing building with the existing grid. o the right: Proposed grid for the top up.

Definition of Structural Components.

This stage involves characterizing all the timber structural elements that could be used for the timber top-up. The aim is to create a toolbox or inventory of suitable timber components that can be utilized in the following proposal stage. This includes identifying and categorizing different timber elements such as beams, columns, trusses, connectors, and panels to understand the options for selecting and integrating timber elements in the overall design.

For the definition of vertical structural components, the design principle of The open space floor plan allows for different configurations. Glulam columns are designated as horizontal elements to provide maximum flexibility in floor plan arrangement and concentrate the necessary structural area in an open grid, enabling flexibility. The elements need to be aligned with the existing walls of the structure, following a predefined grid.

For the proposal of slabs, the grid size represents a hard constraint as, for instance, for Cross-laminated timber production, the maximum width is 3.5 with a length of up to 16 m, which restricts the possibility of directly connecting the slab to the columns. Therefore, it is required to include a beam system to support it, which will

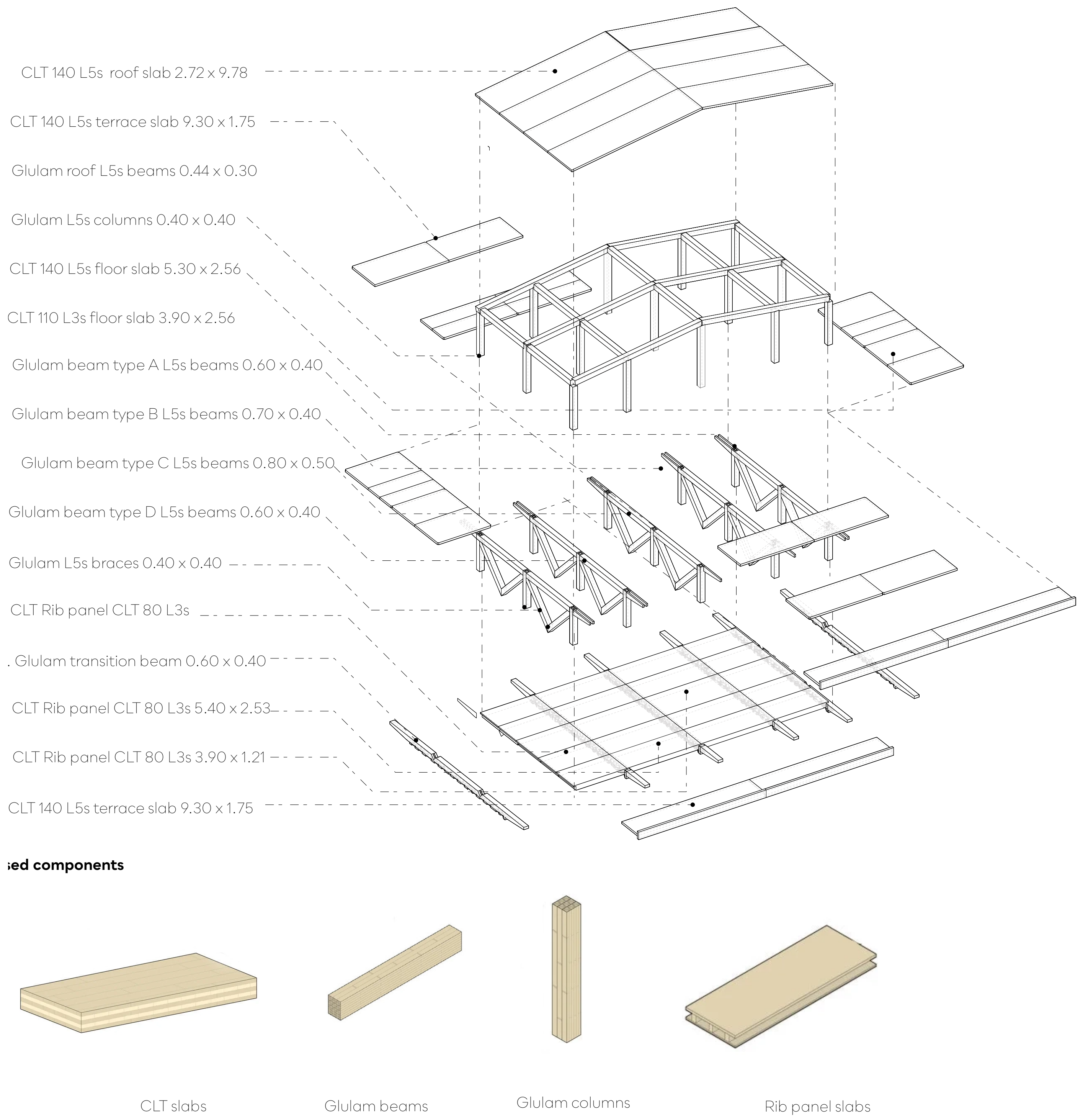


Figure 21 Exploded schematic view of the Timber Top-Up and its components.

connect the columns described in the next section for the slab. Then is a proposed two-directional system that can fit in spans of 3.9 and 5.4. The proposed elements for the study are CLT. Based on the grid design, the structural components can be identified in Figure 21.

Predimension of the structural components.

In this process, the inventory of structural components proposed in the previous stage is utilized to determine the initial dimensions of the system components. The pre-dimensioning process involves evaluating the load requirements, structural performance criteria, and other relevant factors to assign appropriate dimensions to the timber components. This step helps in ensuring that the structural.

Skyciv is used to determine the minimum size based on the loads defined in Section 5.2.3 and the structural requirements stated in EC 5 for calculating beams, columns, and slabs. The results of this dimension can be seen in Chart XX.

5.2.5 Evaluate

The next stage of the process is the evaluation of different structural scenarios for the timber top-up proposed in the previous stage. Moreover, to demonstrate compliance with the limited state design values specified in the structural codes.

Parametrization of the 3d model and the creation of the different scenarios.

This process focuses on adding to the model in Section 5.1.2 the structural proposals defined in the previous section of the timber Top-ups into the digital environment. Moreover, to input into de components of Grasshopper and Rhino gathered so far in the process: Materials, preliminary dimensions, geometries, load cases, and supports.

Structural evaluation of the scenarios.

By having the parametrized model in Grasshopper, Karamba 3D can evaluate the defined criteria from the previous stages and ensure that the timber top-up structure meets the required structural values. The evaluation process in Karamba 3D includes analyzing factors such as structural stability, load-bearing capacity, deflection limits, and the other relevant performance indicators previously stated in the "define" stage.

For the Study case, three scenarios are defined to be evaluated in the Evaluate stage.

- Scenario # 1 Don't Remove: Top-up with timber without demolishing.

This first scenario aims to understand the maximum capacity to Top-up by utilizing the remaining structural capacity of the existing components in the RC structure. Top up the existing structure to increase the capacity of the building as much as structurally possible without retrofitting the existing RC structure.

- Scenario # 3 Maxima: Remove one story and add four.

The purpose of this scenario is to evaluate the possibilities of height increase after removing a fraction of the structural mass of the RC components in the structure, more specifically, a fifth of the total mass in this case.

- Scenario # 2 Minima: Remove one story and add three.

This scenario intends to analyze the results based on increasing the minimum amount of timber layers, moreover by removing one layer and adding two additional ones. This experiment Will set a minimum value of the area. The idea is to create a lower value for a numeric where the range is between minimum and maximum.

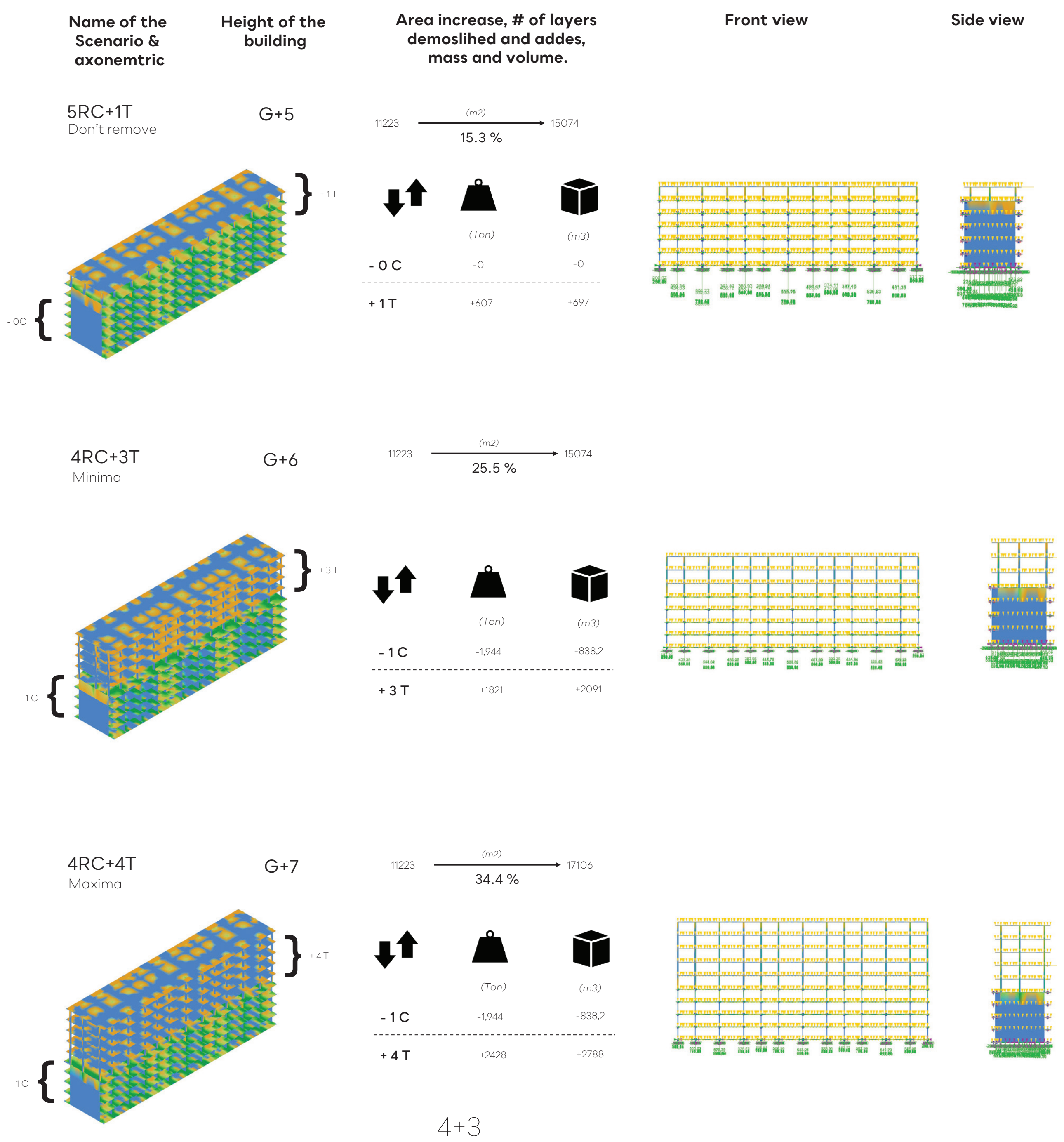


Figure 35 Scenarios to be evaluated in the next stage, from top to bottom: Dont remove, Maxima, and Minima.

Environmental impact analysis of the scenarios with the Ansys Ecoadut tool.

The following process involves collecting data from two sources to assess the environmental impact of the timber top-up design. Moreover, the analysis focuses on analyzing the embodied carbon (measured in CO2 kg per m3)

Firstly, information about the mass and composition of the different design iterations is gathered from the "Structural evaluation of the scenarios" stage. And secondly, data related to manufacturing the structure's components are collected from the earlier "Define" stage.

All of this information is input into the Ansys Ecoaudit tool, which facilitates the assessment of environmental performance by analyzing the embodied carbon and Upfront embodied energy of the project during its lifespan supported on databases and assumptions from academic research.

The environmental analysis carried out with the Ansys Ecoaudit will provide valuable results on the carbon footprint and energy consumption associated with the timber top-up that will be discussed in the following stage.

5.2.5 Conclude

The final stage of the framework involves comparing the structural results obtained from the evaluation stage, including the structural and environmental impact assessments, among the proposed scenarios.

In this stage, the outcomes of the different scenarios assessed throughout the methodology are analyzed and compared. Moreover, to examine the structural performance regarding stability, load-bearing capacity, deflection, and other relevant criteria of the overall structure previously established in the earlier stages.

Furthermore, the environmental impact assessment results are also evaluated among the scenarios. This includes evaluating the embodied carbon and energy values obtained from the environmental analysis stage to identify which design options have a lower carbon footprint and energy consumption, contributing to a more sustainable solution.

Finally, by comparing the scenarios' structural and environmental results, it is possible to make informed decisions to select a final design proposal that meets the required structural criteria but also demonstrates a reduced environmental impact.

The outcome of the whole framework ensures that in other processes, the Top-up timber design will align with structural integrity and sustainability objectives and therefore give insights for the decision-making about continuing with the process or considering a different approach.

For the conclusion and result part of the study case, Chapter 6 will describe them and will explain the obtained results.

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5.2.6 Architectural design

The objective outlined in Chapter 4.1.3 was the demolition of the buildings on Chopinlaan in order to replace them with single-unit houses of reduced height compared to the current Gillisbuurt buildings. This initiative was driven by the Delfts Geemente's aim to enhance transparency from Chopinlaan Street to the interior of the residential complex, which houses two parks. The intention was to improve the urban conditions in the neighborhood.

To achieve this, the ground floor is opened up to allow for transparency and improved visual conditions from Chopinlaan to the park. The relocation of ground-floor apartments to the top level frees up the ground-floor space. Stores and communal activities are added on the ground floor to diversify the building's activities and make the neighborhood more dynamic throughout the day and night.

Regarding the height of the intervention, the top-up involves adding two stories of timber while maintaining the existing RC structural components as can be seen in the rendered image 8 in the next page. This decision is made to preserve the uniform building height in the surrounding area and prevent overshadowing of the buildings below. It also avoids obstructing natural light and views in the neighboring structures.

In terms of energy efficiency, the building's facade is being intervened to reduce energy demand. This includes measures such as insulation, improved air tightness, temperature-controlled heating systems, and improved ventilation.

Rendered image 8. Street view of the top-up intervention in the Gillisbuurt residential complex

These interventions aim to offset 72% of the building's total embodied carbon represented in the use stage of the dwellings. The remaining 38% corresponding to the upfront carbon energy, is intended to be reduced by constructing the timber top-up.

As part of the intervention, some of the existing concrete galleries are being replaced with timber slabs. This allows for the expansion of corridors in front of existing apartments and the creation of sun terraces/gathering spaces for communal interactions. It also breaks the monotony of the current gallery facades and reduces the weight of the structure. These terraces integrate the top-up design with the existing structure, creating a unified architectural typology.

Rendered image 9.Street level intervention.

From an urban perspective, the introduction of these terraces enhances outdoor activities and improves the relationship between residents and the street environment. This contributes to an improved sense of security in the sector as can be seen in image 9.

The desition of having tilted roofs was taken as it offer numerous advantages for the Netherlands.Having a high rainfall, the sloped roofs enables efficient rainwater drainage, minimizing water damage in the interior. Additionally, during snowy periods, tilted roofs prevent snow accumulation, reducing this load in the loadcase. Moreover, these roofs also facilitate the installation of solar panels at optimal angles, maximizing energy generatio. Furthermore, from an architectural perspective the angled roofs allow for better natural lighting, reducing the use of artificial light in the interior. Lastly, tilted roofs adds and architectural interest to the the urban landscape where flat roofs are predominant.

Rendered image 10. Top facade of the top-up.

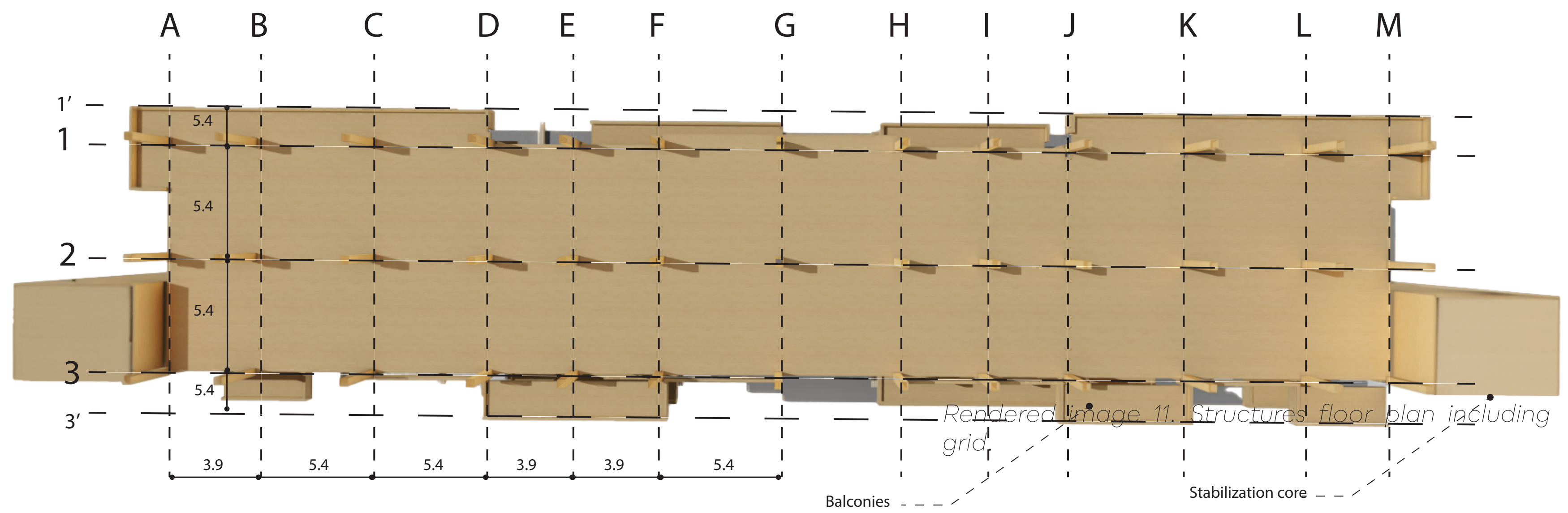
Rendered image 2. Facade of the top-up intervention in the Gillisbuurt residential complex

Grid and structure

Suspendisse eget tincidunt sapien. Ut ornare sem a consectetur aliquet. Sed Section 5.2.4 emphasizes that the top-up grid's definition relies on the existing grid of the building's RC structure. To align with the concrete walls of the existing structure, a proposed grid measuring 3.9 meters and 5.4 meters by 10.8 meters is suggested, as depicted in image 11. This grid arrangement serves to organize the placement of glulam columns responsible for transferring loads to the unidirectional CLT slabs.

Furthermore, there is a proposal to extend the CLT slabs to create balconies on the northern and southern sides. These extended slabs would be supported by the CLT stabilization cores, enabling the structural system to gain rigidity and withstand wind loads effectively.

Rendered image 13. Top: Floor plan of the existing building with the existing grid. Bottom: Proposed grid for the top up.



Rendered image 11. Structures floor plan including grid

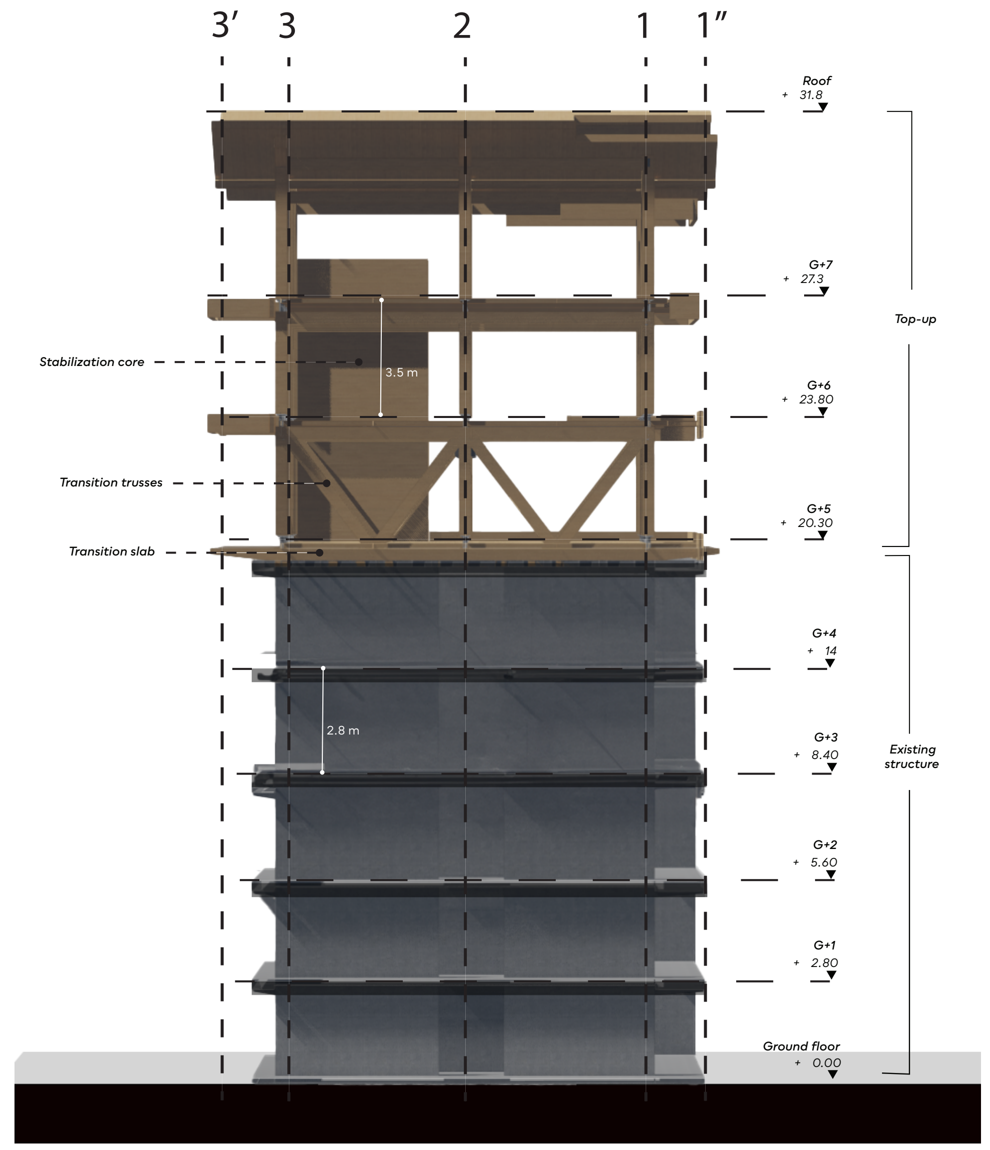
Additionally, a bracing system composed of glulam beams is recommended for the transition floor between the top-up and the existing RC structure. This bracing system aims to distribute loads evenly between the top-up and the existing concrete walls. Its purpose is to ensure structural integrity and alleviate concentrated loads on top of the RC walls.

Architectural program

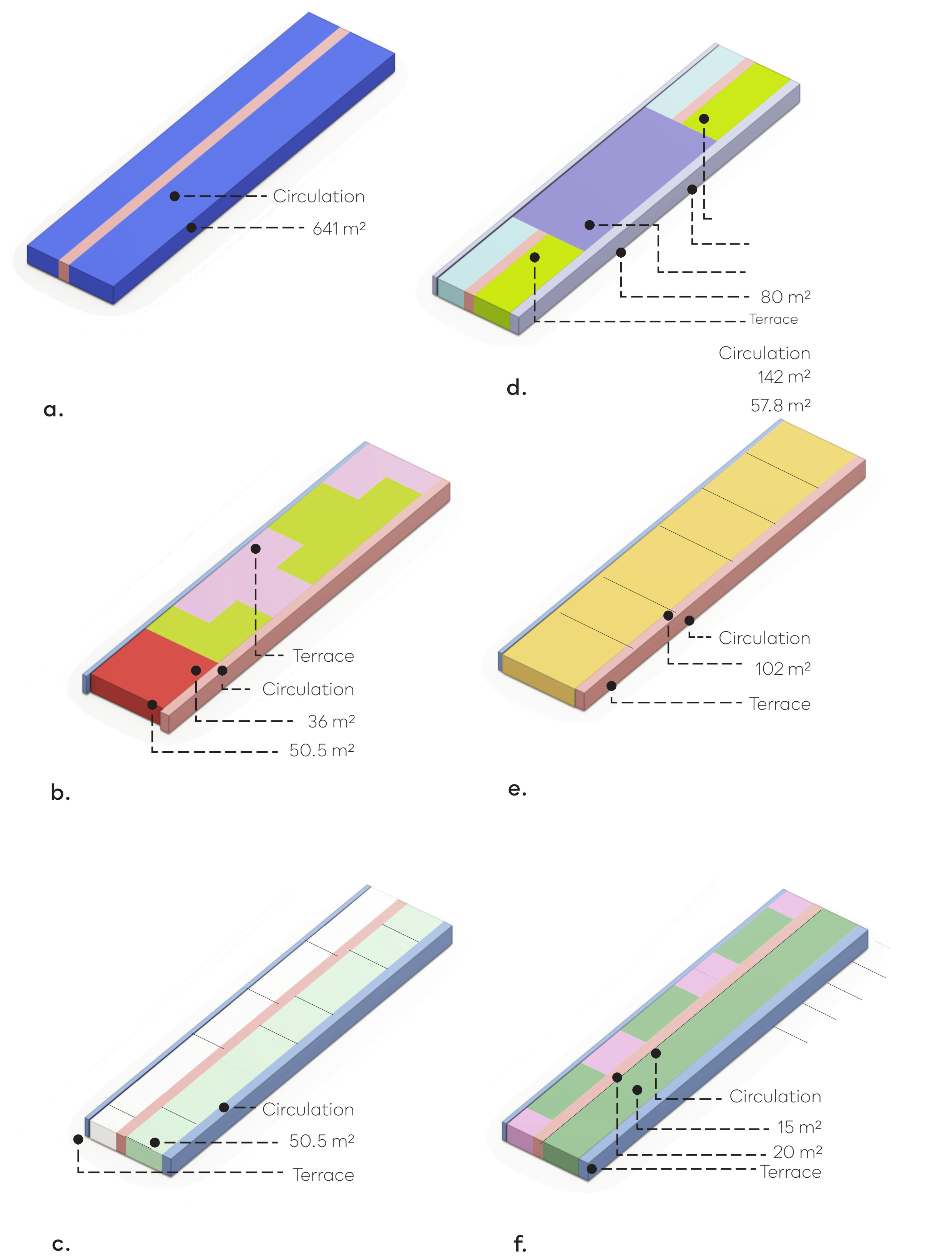
The structural disposition of columns in the proposed floor plan configurations prioritizes flexibility, aiming to create an open floor plan that allows for adaptable space utilization. Unlike the existing RC structure, the top-up design's absence of fixed structural walls facilitates easy reconfiguration and layout adjustments according to changing needs over time. This flexibility allows the building to accommodate diverse uses and ensures that the lifespan of the building is extended.

Considering the predominant use in the Gillisbuurt neighborhood is currently housing, the intervention proposes a mixed-use approach with commercial spaces on the ground floor. However, considering the potential future shifts in urban dynamics, it is important to design the building to accommodate different housing typologies, as can be seen in image 15. This circular approach ensures the building's ability to adapt to changing demands and extend its lifespan.

Furthermore, the proposed structural design allows for potential future uses beyond housing. It could be repurposed to serve as student housing, a hotel, or even retrofitted to comply with codes and accommodate office spaces in the future. This versatility in potential uses enhances the building's adaptability and ensures its relevance to evolve and respond to the changing market demands.



Rendered image 12. Strucures section.



Rendered image 15. Proposal for different architectural programs in the top-up: a. Circulations trough the middle. Open space floor plan. b. Circulations trough the side with balconies. c. Circulation trough a corridor in the middle. d. Circulation by the sides - Open space in the middle. . e. Open space in the middle with terraces towards one side with balconies. f. L shape typology apartments.

Circulations

The top-up design proposes two types of circulations for efficient movement within the building. Firstly, there is the vertical circulation, which includes the elevator shaft, ducts, and staircase. These elements are located within the stabilization timber core, as illustrated in image 16. This vertical circulation system allows easy access to both the existing building and also the top-up. Moreover, The proposed circulations also serve to comply with fire safety requirements, which mandate the placement of staircases accessible to evacuate the building in 120 minutes.

In terms of horizontal circulation, two options are presented. The first option suggests a central circulation route that provides lateral access to households and rooms. This option allows for a seamless flow between different areas on each floor. The second option is to maintain the existing circulation pattern of the building, utilizing side corridors similar to the current layout. This choice would maintain familiarity and continuity for occupants.

By incorporating these different types of circulations, the design caters to various functions and requirements over time. It offers flexibility and adaptability as the building's needs may change.

Connections and assembly

For the connections of the structure, is opted for steel connections as seen in Table 7. Since timber construction has been popular in recent years, there has been an increase in systems capable of responding to highrise situations. The reasons behind using steel respond to the benefits that they present in the present as follows:

From a structural perspective, allow:

- A higher structural strength compared to others: As they provide robustness and strength to the timber structure, allowing it to withstand the vertical and lateral loads experienced in high-rise buildings. The steel connections enhance the overall structural integrity and ensure the stability of the timber system.

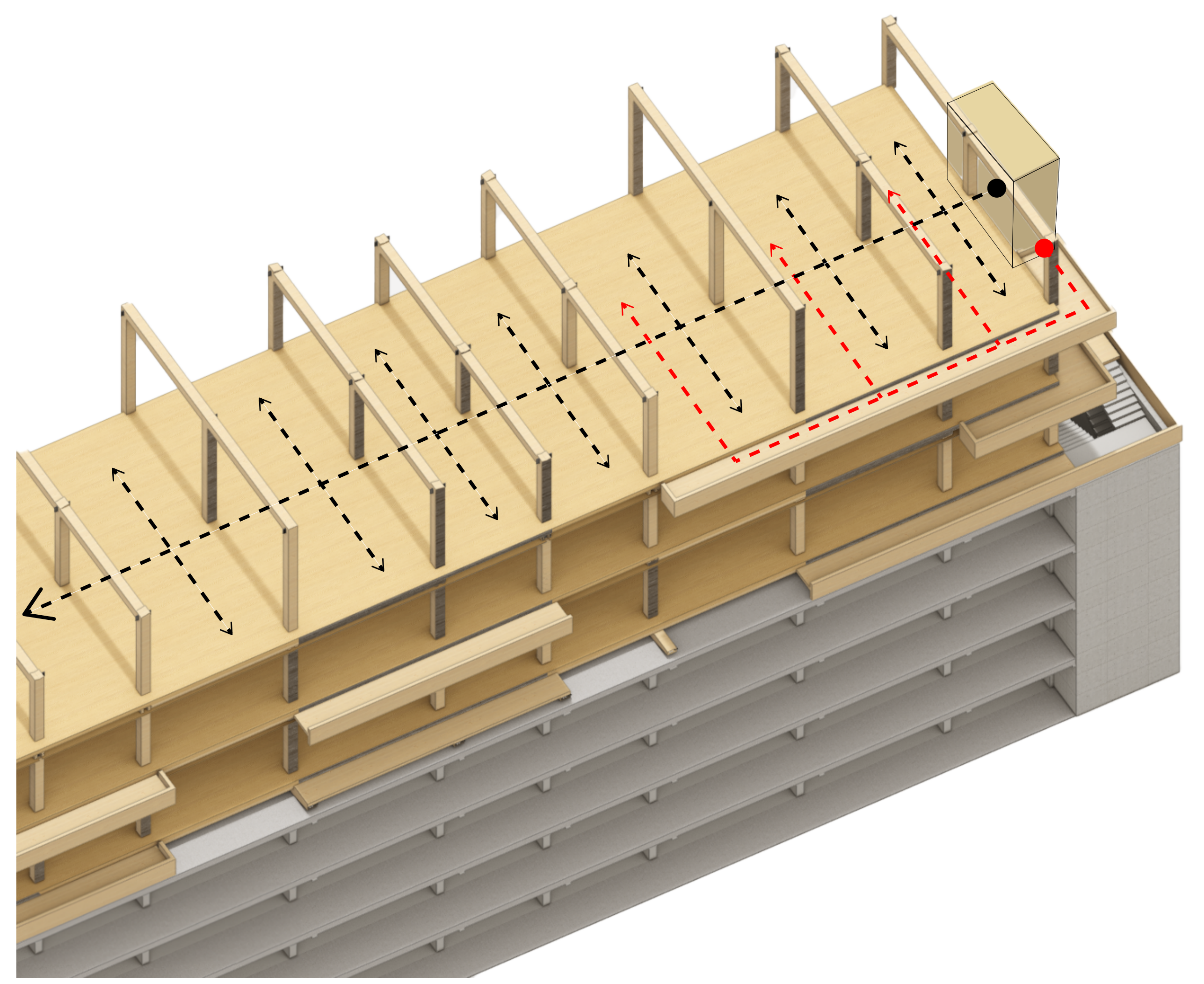
- Durability: Steel connections provide durability and longevity to high-rise timber structures, ensuring the long-term performance and reliability of the connections..

From a circular perspective:

- Reversibility and Reusability: Steel connectors allow for the disassembly of timber elements without causing damage, providing a reversible construction approach. The ability to disconnect and reuse timber members facilitates the Reuse, Refurbish, and Repair of timber components in the structure.

- EoL sorting: Steel connectors facilitate the separation of timber and steel components during the disassembly process at its EoL. This segregation makes the sorting of materials efficient for recycling or proper disposal, contributing to waste reduction and improved environmental outcomes.

The construction process of a timber structure in a top-up offers the benefit of



Rendered Image 16. Two proposal for circulations in the proposal: Black, cirulation trough the middle of the floor plan. red, circulation trough the side.

being performed without interfering with the use stage of the existing structure. This means that the construction works can take place while the building below remains operational and occupied. Moreover, the process can be resumed once the final drawings are finished, as seen in Image 17 in the following steps:

Site Preparation: (step a): Consist of installing a crane next to the existing building that will lift to the top of the building all of the prefab components.

Timber Fabrication: The timber components, such as glulam beams, columns, and CLT panels, are fabricated off-site. The timber is precisely cut and shaped according to the design specifications, ensuring accurate dimensions and connections. Moreover, Steel connectors, such as plates, brackets, and fasteners, are positioned and installed in the timber components.

Assembly and Erection (steps b to g): The prefabricated timber components are transported to the construction site and lifted into place using cranes or other lifting equipment. The components are carefully positioned and connected according to the construction drawings. This includes aligning and bolting the steel connectors to ensure secure and rigid connections.

Installation of Stabilization Cores and roofs (step h): Stabilization cores, typically made of timber, are incorporated into the structure to enhance stability and resist lateral forces. Once the main structure is in place, the construction of floors and roofs commences. The roof made of CLT panels is assembled to create a weather-tight and durable covering.

Finishing and Services Installation: Interior finishes, such as wall cladding, ceilings, and flooring, are installed. Electrical, plumbing, and heating.

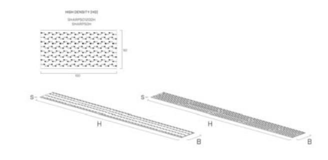
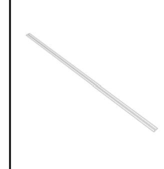
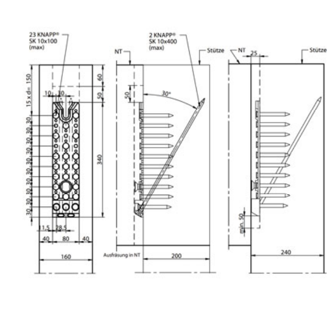
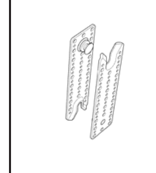
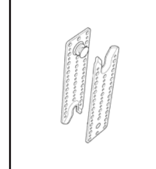
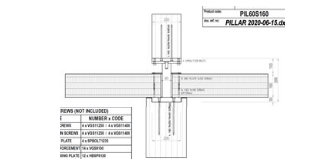
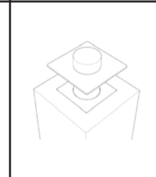
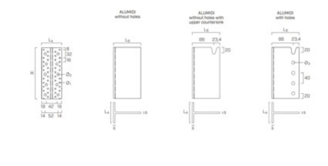
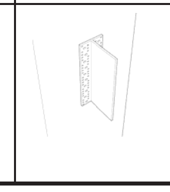
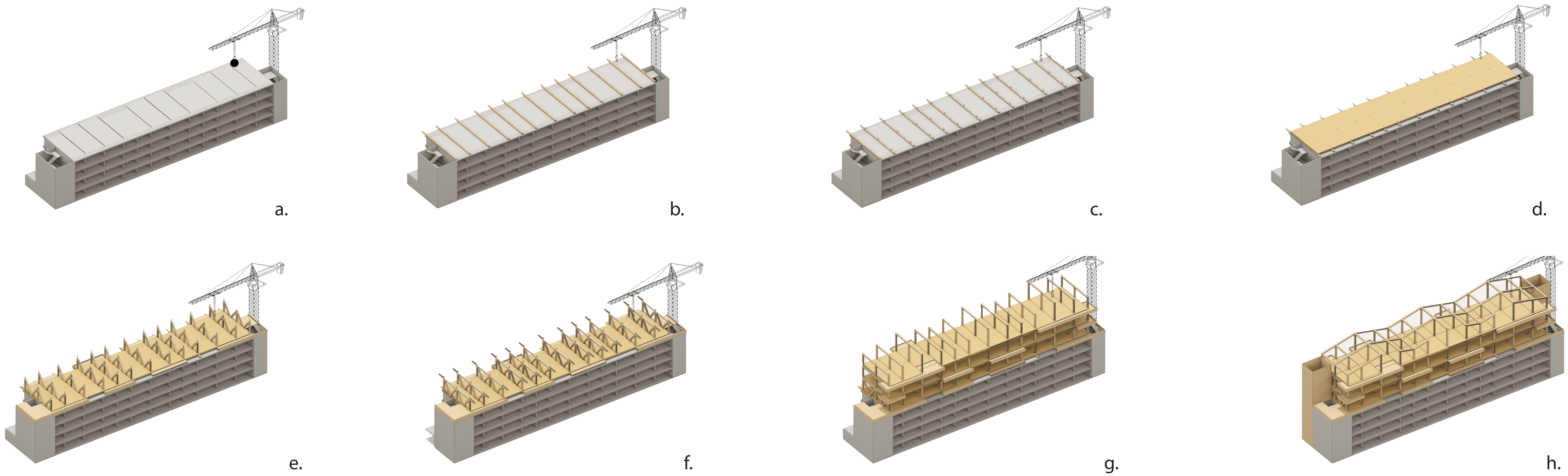
Connection	Type	Dimensions	R[ax,k]	Front/side view	Image	Producer	
pä~ÄJíçJÄÉ~ã =ÅçååÉÅíáçåK	` ~êÄçå=ëíÉÉä=é ä~íÉ	RM ñ NOMM ã ã ñ NM å~á äë é Éê=NKO=ã íë	OOMM= âk Gã O			Rothoblas	Sharp metal hooked plate
_É~ã =íç=Åçäìã å=Åçåå ÉÅíáçåK	` çååÉÅíçêë=Ñçê=ÜáÖÜ= Å~é~Åáíó=ëí~åÇ~êÇ=ÅçååÉÅíáçåë	PQMGNSM=ã ã	NVM=âk			Knapp	RICON® S
_ê~ÅáåÖ=íç=ÄÉ~ã		PQMGNSM=ã ã =G=O				Knapp	RICON® S 390/80 VS+ZP
` çäìã åJíçJÅçäìã å=ÅçååÉÅíáçåK	` çäìã å=íç=Åçäìã å=ÅçååÉÅíáçå= ëóëíÉã	OUMGOUM=ã ã	OPRM=âk			Rothoblas	PIL80S160
_ê~ÅáåÖ=íç=Åçäìã å=ÅçååÉÅíáçåK	` çåÅÉ~äÉÇ=Äê~ÅâÉí=ï áíÜ=~åÇ= ï áíÜçìí=ÜçäÉë	NQS G QUM ã ã ï áíÜ SM å~áäë	NQSIS=âk			Rothoblas	Concealled s

Table 7. Proposed connections for the structural system



Rendered image 17. Assembly process of the top up; a. (If applicable) deconstruction of RC components. b. Installation of transition beams. c. Instalation of steel column conectors in the transition floor. d. Instalation of Ribbed box slab in the trasnsition floor. e.Instalation of transition trusses. f. Instalation of beams, g. Steps f and g are repeated until the desired height is reached. h. Instalation of roof berams and roof slab.

06 Experiment
Evaluation & Results

For the experiment setup, the defined scenarios are analyzed based on the following criteria; they will be explained in the further section based on the software used to perform the simulations:

Rhino 7 + Grashopper +Karamba 3d to analize:

- Reaction in the foundations after Toping-up.
- Utilization of the Timber top-up components (maximum shear and moment).
- SLS - Deflection limit (Nodal displacements).
- Reactions of the concrete structure (Nodal displacements).
- Utilization in the concrete structure.

Rhino 7 + Grasshopper + Granta Edupack + Ansys Ecoaudit Tool to analyze:

- Upfront embodied carbon.

6.1 Experiment Set-up

Scenarios set-up: RC elements remove, timber added, mass and volume

Input values to be used in the software Granta Edupack to calculate the upfront

		Set up for the scenarios							
		Intervention		Quantities					
		Stories of concrete removed	Stories of timber added	Volume of concrete removed	Volume of timber added	Mass of concrete removed	Mass of timber added C24	Total Area	Increase in area capacity
Scenarios		Stories	Stories	M3	M3	kg	Kg	M2	%
0	Structure today	0	0	0	0	0	0	11223,0	0,0
1	(5RC+1T)	0	1	0	697	0	607042	13255,0	15,3
2	(4RC+3T)	1	3	838,2	2091	1944232	1821126	15074,4	25,5
3	(4RC+4T)	1	4	838,2	2788	1944232	2428168	17106,4	34,4

Table 10 Table with the Set-Up for evaluating the experiments

carbon emissons of the stucture.

Parameters	Value	Unit
Existing building area	11223	m2
Area of one layer of a existing	2244,6	
Area of one layer of a Top-Up	2032	m2
Exixting volume of concrete in the existing building	4191	m3
Existing mass concrete in the existing building	9721162	kg
Existing mass timber in one layer	607042	kg/m3
Existing volume of concrete in one layer	838,2	m3
density of scpurce C24	420,0	kg/m3
ratio of molecular weight Co2	3,7	
Moisture of timber	12	%
Secuestred carbon of Spruce C24	-0,69	kg*m3
Dissasembly potential of with concrete building	0,54	
Dissasembly potential of timber top up building	1,00	

Table 11 Parameters the will be used in the simulation in Granta Edupack

Definition of disassembly potential for the reuse of RC components in the existing structure and for the timber components in the top-up

Parameter		Prefab existing RC Slabs	In cast existing RC walls	Timber Top up components
Environemtal Cost indicator	*ECI*	16,59	16,59	16,59
Connection type	*CT*	0,60	0,10	1,00
Connection accesibility	*CA*	0,80	1,00	1,00
Dissasembly potential of the connection	*DPc*	0,69	0,18	1,00
Independency	*ID*	1,00	0,40	1,00
Geometry of product edge	*GPE*	1,00	1,00	1,00
Dissasembly potential of the compoisition	*DPcp*	1,00	0,57	1,00
Dissasembly potential of the product or element	*DPp*	0,81	0,28	1,00
Dissasembly potential of building	*DPb*	0,54		1,00

Table 12 Calculation for the disassembly potential of the components in the project based on the DISASSEMBLY POTENTIAL MEASUREMENT METHOD VERSION 2.0.

Definition of the sequestration carbon of Spruce C 24 Timber 12% humid content used in the timber top up components.

$$\text{captured } CO_2 \text{ (in kg/m}^3\text{)} = \frac{\text{quantity of wood in m}^3 \times \text{density of wood (at 12\% wood moist content per kg/m}^3\text{)}}{1 + (12 / 100) \text{ (at 12\% wood moist content)}} \times \frac{3.67}{2}$$

Figure 36 Equation to estimate carbon sequestration of wood. As seen in Section 3.1.2, woods have the potential to sequestrate carbon; based on (Lugt & Harsta 2020), the carbon sequestration of C24 timber is calculated.

6.2 Results

6.2.1 Reaction in foundations after Toping Up.

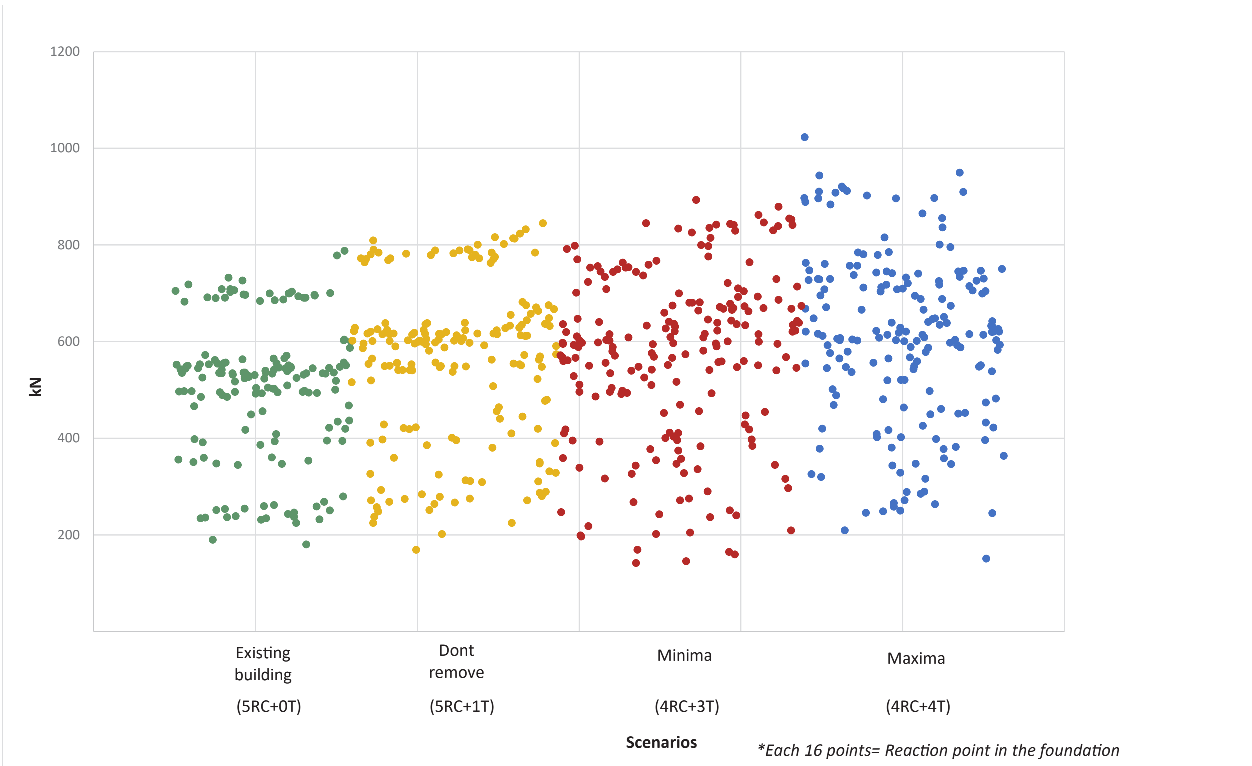


Figure 37. Reactions in the foundations after toping up

Figure 37 illustrates the reactions in the structure's supports (measured in kN in the Y axis) compared to the analyzed scenarios in the X axis. Consequently, these reactions will be the forces that will be transferred into the existing foundations of the building in each scenario.

Scenario 0: Existing building (Sc. 1) shows the current reactions in the supports today without any intervention, where each point corresponds to support in the structure's base.

When comparing Sc.0 with the following four scenarios, the following conclusions can be drawn:

Scenarios 1 to 3 progressively increase when more Timber Top-up layers are added. Among these three, Scenario 3: Maxima (Sc.3) demonstrates the highest reaction increase, reaching approximately 400 kN in each support. Scenario 4:Variation (Sc.4) shows a similar but lower result than Sc. 3.

Scenario 2:Minima (Sc.2) and Scenario 1: Not remove (Sc.1) exhibit a comparable load increase in the supports of approximately 200 kN compared to Sc.0.

Furthermore, the hypothesis stated in Section 1.3 at the beginning of the document, suggesting that replacing RC components with timber could lighten the structure, is partially validated. Although timber is lighter than RC, replacing RC components with timber components (as tested in Sc. 2, 3 & 4) does not necessarily reduce the reactions in the existing building foundations.

One possible xplanation for these results is that while the weight of the removed RC components might be replaced with a similar one of timber (representing more mass and hence more area), there is an increase in the load cases due to the expanded area of the building, which include new wind and live loads. This hypothesis will be further investigated for the final report of the graduation project in P5, considering its implications in the design process.

6.2.2ULS - Utilization of the Timber components used in the top-up (maximum shear and moment).

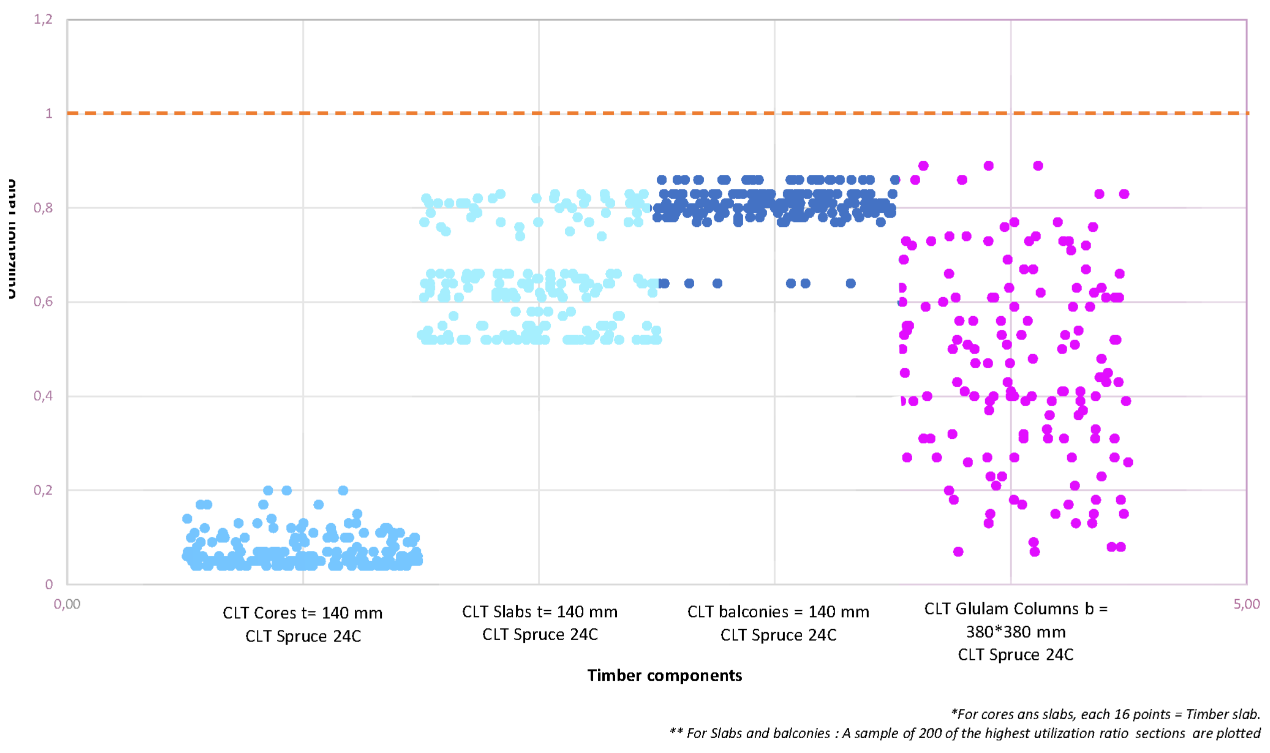


Figure 38. Utilization of the Timber components (maximum shear and moment).

Figure 38 illustrates the utilization ratios of various timber components within the timber top-up in Scenario 3.

On the Y-axis, the utilization ratio (the applied load divided by the design capacity of the component) is shown, while the X-axis represents the evaluated components.

Consequently, four timber components were plotted from the model: CLT cores, CLT slabs, CLT balcony slabs, and CLT glulam columns. Plotting the following results:

The 140mm thick CLT cores exhibit an 80% excess capacity. This excess may be attributed to two factors, which will be further analyzed in the P5 report: The cores solely transfer tension loads resulting from wind loads on the slabs, and the charring layer tends to increase the dimension of the element.

The CLT slabs show a utilization ranging between 50% and 80% depending on the specific structural situation, such as the 3.9 or 5.4 meters span. The remaining excess capacity is due to the over-dimensioning of the section caused by the charring layer.

The utilization of CLT balconies ranges from 78% to 85%—the excess utilization results from the over-dimensioning of the section due to the charring layer.

For the glulam columns, it is evident that some columns have higher utilization than others. This discrepancy arises because all columns currently have the same dimensions in their section, which means that those on the lower levels of the Top-up experience higher load cases compared to those on the upper levels, such as the ones on the last floor, which only bear the loads from the roof.

All components are below the limits, meaning that ULS (bending, normal, and shear) can bear and transfer the proposed load case. A further process will focus on optimizing their section to reduce the mass of timber of the Top-up.

6.2.3 SLS - Deflection limit (Nodal displacements).

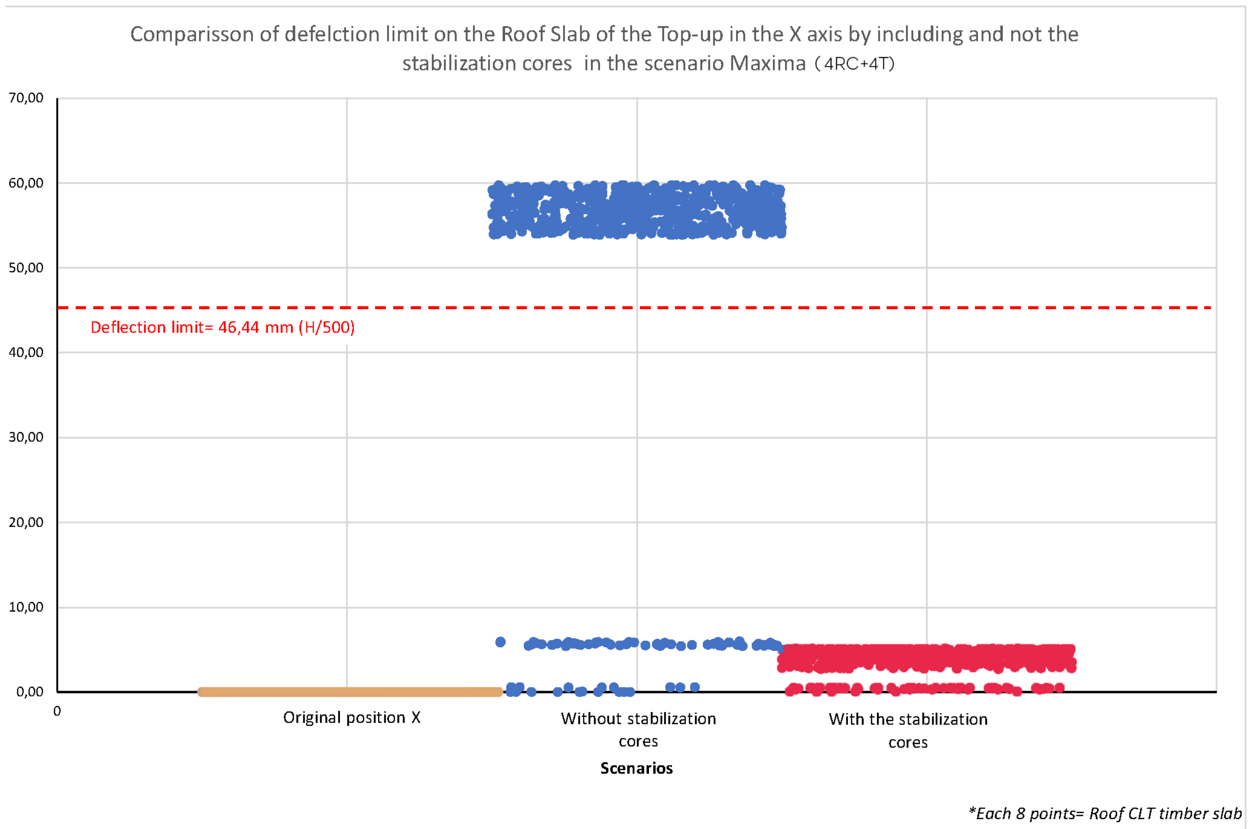


Figure 39. Utilization of the Timber components (maximum shear and moment).

Figure 39 compares the displacement of nodes in the CLT roof slab in Scenario 3 under two strategies: utilizing stabilization cores and not using them at all. The Y-axis represents the numerical values of node displacement caused by wind loads. The maximum deflection limit for the overall structure, as defined by EC 0, is stated as max. deflection = H/500 (0.046 meters = 23.22 meters/500).

The chart demonstrates that if the structure consists solely of beams, columns, and slabs, the roof will experience a displacement of over 6 centimeters due to structural instability, surpassing the acceptable values. However, in the following scenario, it can be observed that with the inclusion of stabilization cores, the displacement values can be significantly reduced to values that fall within acceptable limits.

Stabilization cores represent one potential strategy to address excessive deflection in the CLT roof slab. However, it is important to note that they are not the only solution available. In the next stage, other optimization processes will be employed to explore alternative approaches, such as introducing bracing components to reduce the mass of timber within the cores.

6.2.5 Reactions of the RC walls in the top-up connection (Nodal displacements).

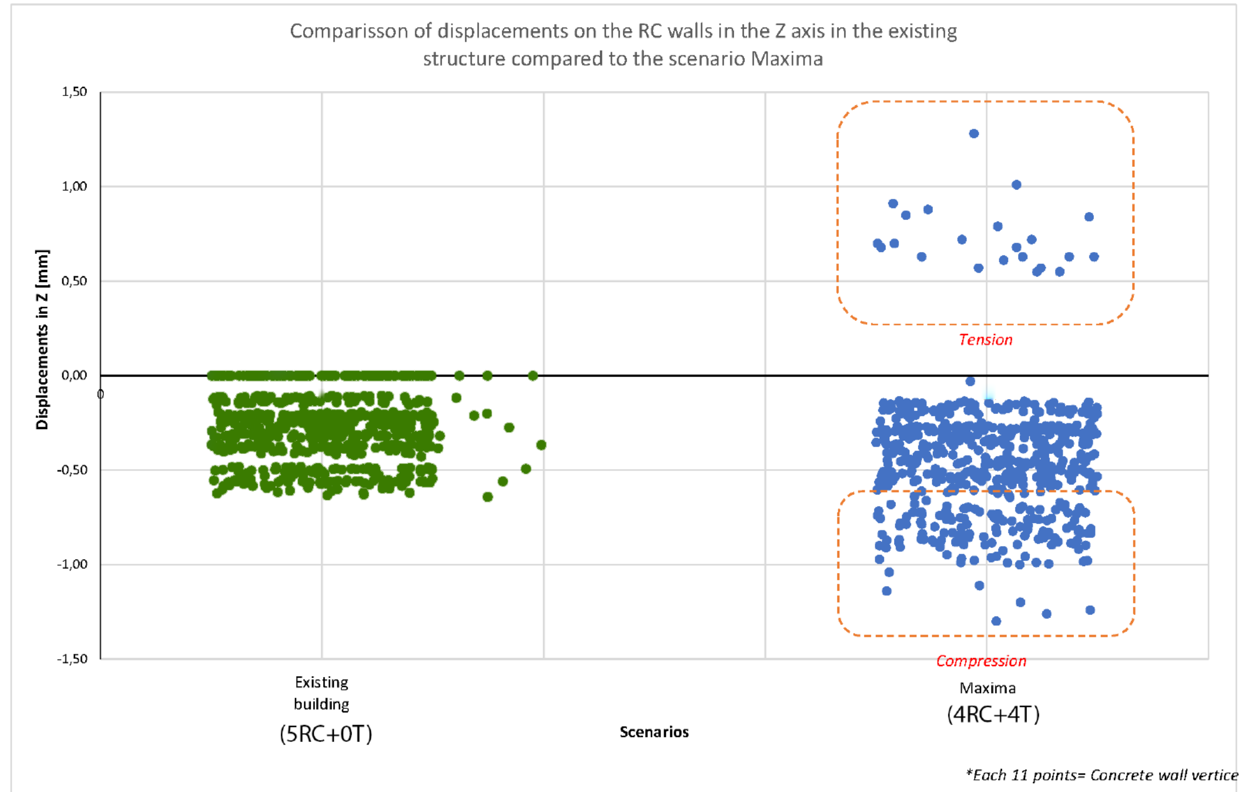


Figure 40. Reactions of the concrete structure (Nodal displacements).

The following results aim to assess the impact of the top-up on the existing concrete walls in terms of vertical displacements, as previously stated in the purpose stage as one of the structural assumptions.

The graph illustrates the displacements in the Z-axis of the concrete walls located in the first four stories of the existing structure. To the left, It can be observed that the existing structure today experiences a maximum displacement in the Z axis of only half a millimeter due to the current load case scenario, which can be considered negligible.

6.2.6 ULS - Utilization of the RC walls in the structure after Toping-up.

The purpose of thye analizis iun figure 41 is to evaluate the structural capacity of the existing RC walls in terms of the Utilization ratio when subjected to different timber top-up scenarios. The focus on evaluating the walls is because they are the primary components responsible for transferring and supporting loads of the top-up.

In Scenario 0, it is evident that the existing structure has an overcapacity of 80%, indicating the presence of unused potential capacity that could accommodate additional weight.

Moving to Scenario 1, where a complete layer of timber is added, there is a 20% increase in capacity, resulting in a total overcapacity of 60%.

Scenarios 2 and 4 demonstrate a 30% increase in loads due to the added weight compared to Scenario 0. Despite this increase, a remaining capacity of 50% can potentially be utilized to accommodate an overload.

The most demanding scenario, Scenario 3, exhibits an overcapacity of 40%, indicating that it can withstand even higher loads without exceeding the utilization limit.

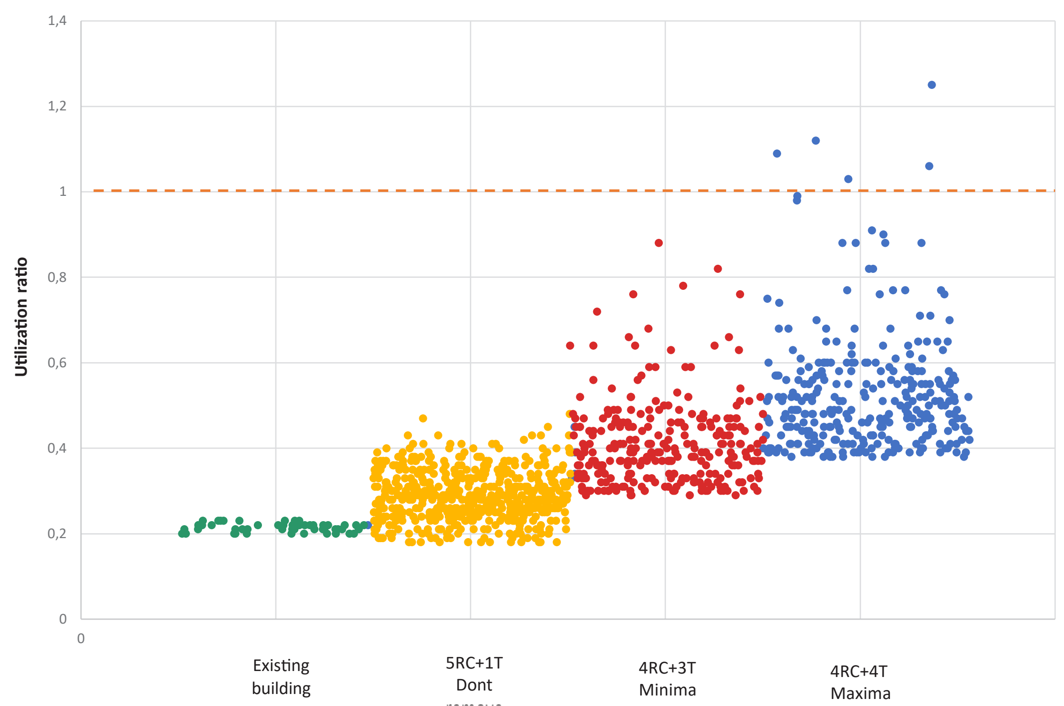


Figure 41. Utilization of the RC walls in the RC structure after Toping-up.

6.2.7 Upfront carbon

A roadmap was created to determine the top-up intervention's upfront energy, illustrating 18combinations of End-of-Life (EoL) scenarios for the structure's first and second cycles.

The project has generated 2454 tons of CO2 emissions after analyzing the extraction, manufacturing, and transportation processes since its construction in 1968. Without any interventions, this emission level will remain unchanged until a decision is made.

After comparing the previous iterations, the following conclusions can be drawn:

Reusing materials in the first and second cycles is the most effective strategy for reducing CO2 emissions across all scenarios.

Downcycling and recycling reinforced concrete (RC) components in the first cycle, followed by energy recovery of the timber in the second cycle, resulting in the highest overall CO2 emissions. This strategy emits even more CO2 than leaving the structure untouched without interventions.

Downcycling and recycling RC components in the first cycle, followed by reusing the timber, reduce CO2 emissions compared to leaving the structure without any interventions. However, this difference is more noticeable when the entire structure is dismantled and replaced with a new timber structure.

		Existing building (1st. cycle)												
		Upfront carbon - Existing RC structure				End of life Scenarios								
		Material	manufacture	Transport	Total	Downcycle		Recycle		Reuse		Total Upfront C. first cycle EoL scenario		
		A-0	A-1 to A-5 (whiout A-4)	A-4	A-0 to A-5	C-1 (Preparation)	D (Downcyle)	C-1 (Process)	D (Recycle)	C-1 (Preparation)	D (Reuse)*	D (Downcyle)	D (Recycle)	D (Reuse)
Scenario		Kg	Kg	Kg	kg	Kg	Kg	Kg	Kg	Kg	Kg	Ton	Ton	Ton
0 1	Structure today 5RC+1T	2,00E+06	4,02E+05	5,25E+04	2,45E+06	0	0	0	0	0	0	2454	2454	1374
2 3	4RC+3T 4RC+4T	2,00E+06	4,02E+05	5,25E+04	2,45E+06	6,88E+04	-1,49E+05	9,53E+04	-2,41E+05	2,72E+04	-1,08E+06	2374	2309	1402

* Includes a Dissasembly potential of building of 0,54

Figure 42. Upfront carbon results in the first cycle for the RC structure

		Top-Up (2nd. Cycle)									
		Upfront carbon Top-Up				End of life Scenarios					
		Seq. carbon**	Manufacture /material	Transport	Total	Combustion		Reuse		Total Upfront C second cycle timber	
		NA	A-1 to A-5 (whitout A-4)	A-4	A-0 to A-5	C-1 (Preparation)	D (Combustion)	C-1 (Process)	D (Reuse)*	D (Energy recovery)	D (reuse)
Scenarios		Kg	Kg	Kg	kg	Kg	Kg	Kg	Kg	Ton	Ton
0	Structure today	0	0	0	0	0	0	0	0	0	0
1	5RC+1T	-4,80E+04	2,21E+05	6,37E+04	2,37E+05	2,12E+04	4,80E+04	3,01E+04	-2,21E+05	306	46
2	4RC+3T	-4,32E+05	6,64E+05	1,91E+05	4,23E+05	6,37E+04	4,32E+05	9,04E+04	-6,64E+05	919	-150
3	4RC+4T	-7,67E+05	8,85E+05	2,55E+05	3,72E+05	8,50E+04	7,67E+05	1,21E+05	-8,85E+05	1225	-392

** Calculated using the equation Includes a Dissasembly po * Includes a Dissasembly potential of building of 1.0

Figure 43 Upfront carbon results in the second cycle for the Timber Top-up.

		Upfront Embodied carbon in all of the scenarios					
Scenario		Downcycle Energy recovery	Recyle+ Energy recovery	Reuse+ Energy recovery	Downcylce+ Reuse	Recyle+ Reuse	Reuse+ Reuse
		Ton	Ton	Ton	Ton	Ton	Ton
0		2454					
1	5RC+1T	2760	2760	1681	2500	2500	1420
2	4RC+3T	3293	3227	2320	2224	2158	1251
3	4RC+4T	3599	3534	2626	1982	1916	1009

0

Figure 44 Total values of the whole Upfront Upfront embodied carbon in two lifecycle of the building

82

83

Graduation studio Structural design for change

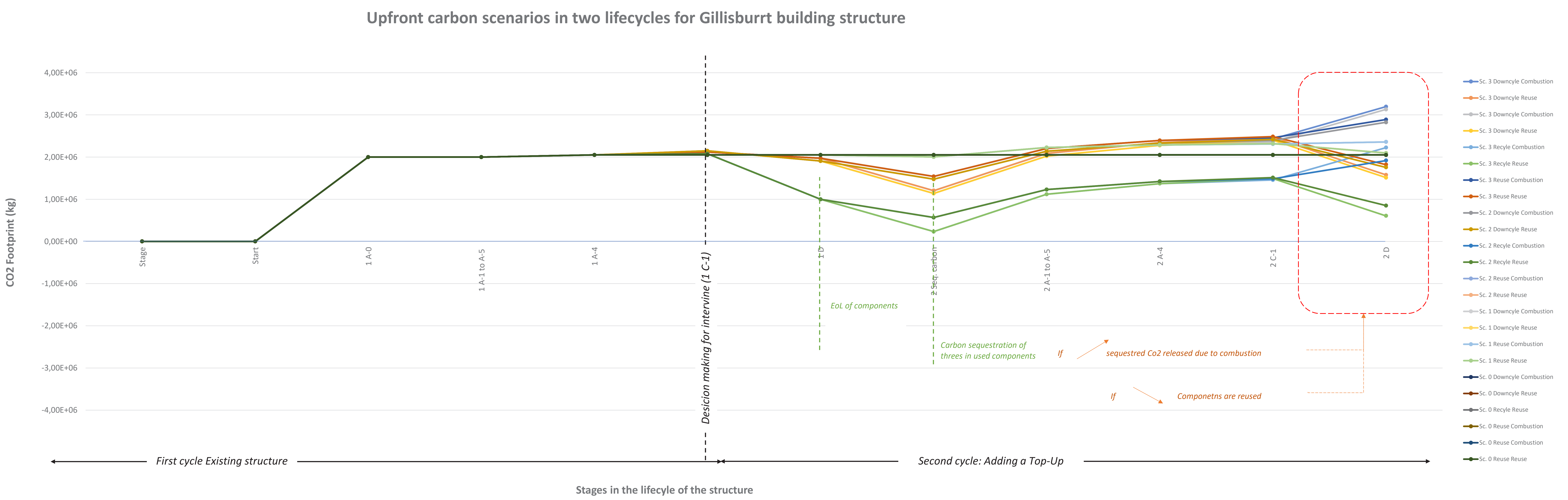


Figure 45 Graphical representation of the whole Upfront Upfront embodied carbon in two lifecycles of the building

07 Conclusions & **Discussion**

7.1 Discussion

The findings of this study provide information on the possibility of using timber top-ups in existing reinforced concrete structures to increase the building's area capacity. By analyzing different scenarios. Moreover, regarding the research question:

How can the area capacity of existing Reinforced Concrete structures be increased and extend their lifespan by using timber top-ups?

The document states that the area of the structure might be increased by having two approaches:

On the one hand, by constructing the top-up on the existing RC structure as seen in scenario 1 (5RC+1T), and on the other by deconstructing part of the components to reduce the weight of the structure as seen in scenarios 2 (4RC+3T) and 3 (4RC+4T).

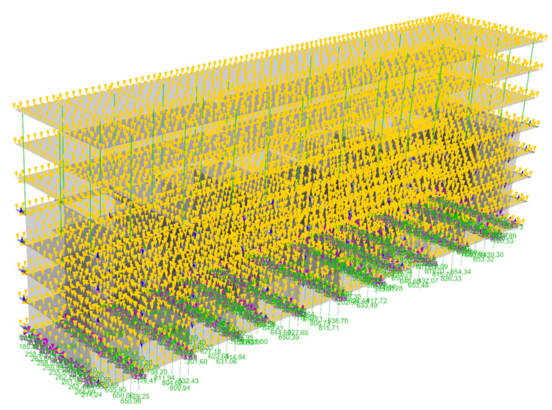
In both approaches, the results seen in Figure 41 (RC wall utilization) show an opportunity in the case study's structure, represented by the excess capacity in the RC walls. This capacity can load them until their utilization limits, allowing components to withstand even the highest scenario.

Regarding reactions in the foundation system, The results exposed in Figure 37 shows that although the deconstruction of one layer of RC components might reduce the structure's weight in scenario 2 when topping up, these reactions increase, overpassing the values of the original structure.

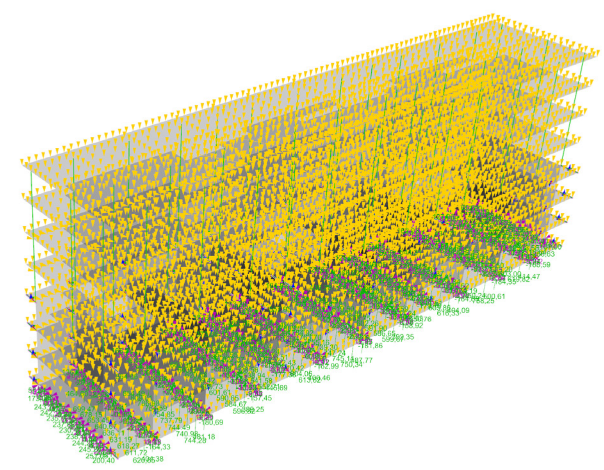
However, if we compared scenarios 1 & 2 (W & 4RC+3T) can be concluded that removing one layer of RC components and adding three of timber will have a similar increase in the reactions in the foundations as not deconstructing and adding one layout, which means that deconstructing would allow more capacity with a similar load increase. In the case of scenario 3 (4RC+4T), there will be an exponential increase in the foundation system.

Therefore, it is essential to evaluate the structural capacity of the foundation system to withstand the new load-case scenarios derived from topping up, even if there might be an opportunity to use the excess capacity in the utilization of existing RC components.

However, having the base of the parametric model, it would be relevant for further research to explore different deconstruction/adding situations that would overload



(4RC+3T)



(3RC+4T)

Different tructural iterations of top up whit the same height. To the right, the scenario Maxima (4RC+3T).

the existing foundations to limit the size of their intervention. For example, analyzing a scenario where two layers of concrete and four of timber are added (3RC+4T):

To approach the subquestion: *How to make visible topping up in the decision-making for stakeholders when facing demolishing/building new?*

The step-by-step exposed in the exploration framework proposed in Chapter 5 was intended to document the design process of the timber top-up for the study case. Therefore approaching top-ups from an architectural domain contributes to visible this strategy as an alternative to demolishing.

To approach *the subquestion: To what extent can the capacity of an existing RC building with timber be increased by reducing the weight of the RC structure?* The research shows that it can be increased as much as possible considering the following criteria:

The height of the building is determined by the capacity of the existing structure and its main structural components to withstand the new load case scenarios. As seen in Figure 41 with scenario 3 (4RC+4T), it's even possible to double the height of the building. However, since the research focused mainly on analyzing the components' utilization and the system's load-bearing capacity, further research must analyze other relevant structural requirements, such as connections to the existing building and foundations' capacity to withstand and transfer loads, that would determine structural feasibility.

However, as illustrated in the research, the top-up's area capacity must also be analyzed from other relevant domains of knowledge that might determine the total height. In the case study, doubling the height of the existing building with a top-up represents a massive aesthetical result that affects the building's aesthetics. Therefore in this particular case, as seen in the proposal section, it opted to increase the height of two stories.

Which structural timber systems can be more effective when topping up existing Reinforced Concrete Structure (RCS) buildings?

As seen in the case study proposal, some structural products might be more suitable to comply with the local fire safety regulations due to the mass required to generate the charging layer during a fire. CLT and Glulam being mass timber components might work to increase the height of the building considerably. Furthermore, these two products can perform better in terms of overall structural stability necessary to withstand the wind force in the new load case scenario. Therefore are recommended in cases where the top-up overpass the fire safety height and is subjected to higher wind loads. Moreover, a system based on timber columns and beams proved effective as it confers the top-up possibility to house different plan layouts during its lifespan.

Also, components that would rigidize the system are mandatory to create structural stability in the top-up, as seen in Figure 39, where it was demonstrated that not having CLT cores would surpass the allowable deflection limit of the building with the top-up.

On the other hand, the research also focused on limiting the amount of timber entering the chain as much as possible. Therefore although the pre-dimension of the components was proposed with the principle of using as less as possible necessary to comply with the safety factors, there are still opportunities to reduce the volume used in the intervention by following strategies such as:

Reducing the section of timber Glulam columns in the upper levels: They are not required to withstand and transfer as many loads as the ones in the bottom used for the dimension.

Replacing the CLT cores for stabilization of the structure with another system: Although CLT cores were used to house the elevators, shafts, and staircases, by using

other stabilization systems in the structure, such as bracings, it might be possible to rigidize the Structure with less use of timber.

What opportunities and limitations have topping up an existing RC structure building?

Opportunities

From an environmental impact perspective, as seen in Figure 45, due to the capacity of timber to sequester carbon, adding mass to increase the capacity would reduce the upfront energy of the building considerably in all the scenarios. However, as seen in scenarios 1 & 2, adding a small timber volume might not reduce the upfront carbon enough to compensate for the emissions produced in constructing the structure.

However, Scenario 3 (4RC+4T) proves that adding more volume to the intervention by increasing the height would lead to higher carbon sequestration than the other two scenarios, which means even a reduction of almost half of the upfront carbon produced in 1968.

Moreover, it is relevant to consider that the upfront carbon emissions of the structure represent just a percentage of the overall CO_2 emissions of the building. As mentioned by Moazzen & Ashrafian (2022), 72% of the embodied carbon of the building Life cycle is attributed to the building's use stage, while upfront energy represents 28%. Therefore, to effectively recover part of the CO_2 emissions of the whole building, topping up should not only be an intervention limited to increasing the capacity of the building but also should aim to incorporate interventions that may reduce CO_2 emissions in operational use.

On the other hand, it is relevant to consider that In all the scenarios, if this sequestered CO_2 of timber is released into the environment at the EoL when burning for energy recovery, it would increase the upfront emissions of the whole structure. Therefore, prolonging the life cycle of the timber structure as much as possible will be an effective strategy to reduce the overall upfront carbon emissions, as can be seen in Scenario 3 when reusing all of the components. Furthermore, it is relevant to apply in the construction sector practices that would ensure the reuse of the structure and/or its components as much as possible.

Limitations

Due to the relative novelty of using mass timber in buildings, there is considerable uncertainty regarding the fate of timber components at the end of their life cycle, whether they will be recycled, landfilled, or incinerated.

Moreover, regarding reuse as a strategy, timber being an organic material, ensuring its durabillity and long-term steady performance of its mechanical properties through its life cycle might be challenging as it is susceptible to be deteriorated by external conditions such as biological agents, moisture differences, temperature differences or deformation due sagging.

In the case of concrete, RC structures can face several common problems throughout

To the left, Deterioration of concrete due to corrosion (structuralguide.com). To the right: images of a test for a CLT connection under wet conditions for a prolonged period of time (Cappellazzi et. al, 2020).

their life cycle, including Corrosion, cracking, and degradation, which might affect the possibility of being reused.

The potential disassembly factor Van Vliet et al. (2021) proposed for estimating the disassembly potential of RC and timber components needs to consider the structural components' physical integration when disassembling. Therefore, cascading both timber and concrete in multiple cycles might have limitations. Therefore, to reuse them, evaluating their physical integrity is relevant to determining whether they are between admissible utilization values.

In conclusion, this research thesis aimed to analyze the structural feasibility of building timber top-ups in existing reinforced concrete structures to increase the building's area capacity.

The findings demonstrated that although there might be different approaches to constructing them, there are opportunities to utilize the excess capacity of RC walls by increasing height using lightweight material. This research corresponds just to a small piece of findings in what might be a strategy that should be analyzed from different perspectives to make it a massive construction system.

7.2 Reflection.

The graduation project: "Timber Top-ups for existing reinforced concrete structure Buildings" is part of the "Structural Design for Change" graduation studio of the "Building Technology master track" in the Msc in Architecture at TUDelft. This graduation studio is relevant to the construction industry's transition from a linear to a circular economy. Because by analyzing and optimizing the lifespan of existing and new structural systems and components, it is possible to limit negative environmental impacts by managing natural resources efficiently. Overall, the studio is relevant as research about one of the building's most complex and intensive systems (material-wise speaking): the structure.

My interest in the topic started after observing that demolishing existing Reinforced Concrete structures to construct new buildings with a higher capacity was the most common practice to respond to the increasing demand for new houses worldwide. This prompts a sustainability discussion as, in several cases, these demolished building structures are still in their service life phase, producing a large amount of waste. Moreover, although significant literature has focused on studying Reinforced Concrete recycling and reuse, it still needs to improve its practice.

Therefore, this project aims to analyze the strategy of adding a substructure made of mass timber components to increase an existing RC structure building area when it reaches its End of Life. Moreover, the research proposes an alternative approach to demolishing and building a new building. Thus, the transformation of the existing building using design principles by reusing as much as possible the existing structure system, increasing the building's area and extending its life span.

Therefore the research question for the project was proposed:

How to increase the capacity of existing Reinforce Concrete structures and extend their lifespan by using timber top-ups?

After researching the state of the art of timber top-ups, in P2, I found that there needs to be a methodology to develop Timber Top-ups on RC structures. This leads to projects of this kind, but everyone is singular and unique depending on the specific context where applied. Therefore I found it imperative not only to design the system itself but to create a framework that would allow to document the process and feed it to structure it and make a mechanism in the decision-making for future interested stakeholders.

This finding represented a considerable income in my process because I managed to redefine my sub-questions more to include in my outcomes the structure that would allow me to document and follow the design of a timber Top up. Therefore, the product of the project became two:

- The proposal of a design framework for the decision-making of timber Top-ups in existing RC structures and
- The conceptual structural design of a Top-up for a specific case study.

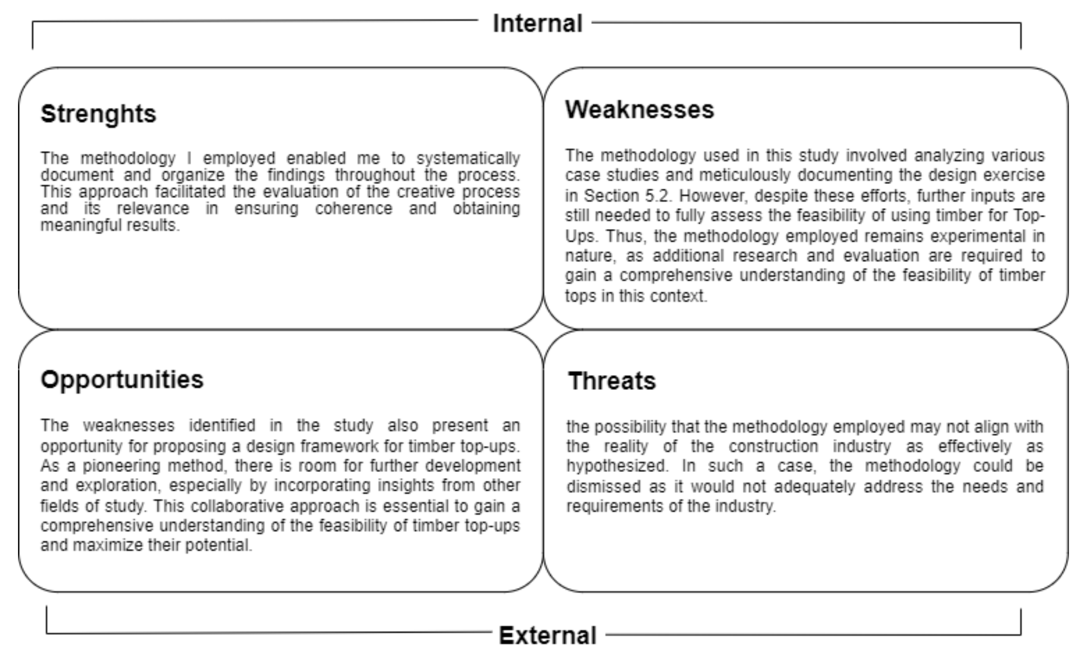
Moreover, the Subquestions that would help to approach the research question became four:

- Which structural timber systems can be more effective when topping up existing Reinforced Concrete Structure (RCS) buildings? (literature review: Section 3: Timber: As a material to create top-up structures.).
- What opportunities and limitations have topping up an existing RC structure building compared to demolishing and constructing a new one? (Section 2: Topping Up existing buildings & Section 5: Research through Design).
- How to make visible topping up in the decision-making for stakeholders when facing demolishing/building new? (Section 5.1: Research through Design).
- To what extent can the capacity of an existing RC building with timber be increased by reducing the weight of the RC structure? (Section 5.2: Research through Design).

The methodology was structured as follows:

- Introduction (including the background) as a starting point for the research.
- Construction of a theoretical framework based on a literature review.
- Selection of a case study for top-up implementation.
- research design, focusing on developing both products while applying circular design principles to structures.
- Experimentation to evaluate quantitative criteria for the structural conceptual design, leading to results for analysis and discussion.

The process structure followed two principles: applying the scientific method to the research and intuitively approaching the problem from different angles through trial and error. It involved perspectives from both an architect and a building technologist. Moreover, The feedback and inputs from my mentors, who had backgrounds in sustainability, and focused on circular design and structural design, played an immense role in shaping the process and its sequence to obtain a reliable result. If the



methodology used for the project is analyzed with a SWOT analysis, the following findings emerge:

The project's contribution to sustainable development can be seen in the results achieved, where it analyzed the performance in terms of CO2 emissions for Timber Top-ups as an alternative to demolishing and rebuilding. Additionally, it explored the concept of Reuse as another viable option for renovating existing buildings constructed with RC structures in transforming an existing building using circular principles, emphasizing the importance of sustainability and resource efficiency.

Regarding the project's sociocultural impact, it responds to this aspect by offering the residents of the intervened building an alternative to reconfigure their current living conditions by expanding the capacity of their existing buildings. By doing so, they can gain additional space and explore different possibilities for rearrangement. Furthermore, these newly acquired areas can be utilized for various purposes that promote social welfare and cultural exchanges within the local communities. Thus, the project aims to provide a financial advantage by facilitating renovation works that can enhance the housing units' energy performance and indoor comfort. By aligning with the principles of sustainable development, the project seeks to improve both the living conditions and overall sustainability of the housing units.

When considering the applicability of using timber tops to top up existing reinforced structures, a practical approach is essential rather than relying solely on scientific analysis. Practical considerations are significant because they consider real-world scenarios that can directly impact the feasibility of using timber tops. Therefore, observing the practical application of timber tops in real-life situations becomes necessary to determine how effectively they can be implemented in different contexts.

In the context of assessing the extent to which the projected innovation has been achieved, it is essential to consider the following aspects of the project:

Firstly, within the Professional framework, the project aims to raise awareness of Topping Up among stakeholders in the architecture and construction field. This involves making the concept visible and understood by these stakeholders, allowing them to recognize its value and potential benefits.

Secondly, the project introduces a Design framework that provides a valuable tool for decision-making in renovation projects. This tool offers guidance and assistance in evaluating the feasibility and effectiveness of Topping Up as a viable option.

Finally, from a scientific standpoint, the project's significance lies in its practical component. The experimental method analyzes specific criteria to assess Top-ups' structural and environmental performance. This rigorous evaluation helps to determine the project's efficacy and contributes to the overall understanding of its potential advantages and limitations.

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09 Appendices

Appendix B – Research about timber products

Solid round timber

Solid round timber corresponds to the lumber before the sawing process that will transform it into timber as can be seen in Image 6 . It has a long history of use in construction, dating back thousands of years,

As it was one of the earliest and most commonly used building materials by humans, it was utilized in the construction of a wide range of structures, including simple dwellings, farm buildings, bridges, temples, and churches (Chilton & Mungan, 2009).

The reasons are due to its abundance and ease of use, as well as its favorable mechanical properties, which have made it suitable for a variety of applications in structures as framing material.

Nowadays, compared to the rest of the structural timber components, round wood poles corresponds to the most basic form of a structural element as, for its obtention, it requires fewer transformation processes. Moreover, removing logs branches, peeling or shaving the bark off tree stems, and cutting the trunks to a suitable length is possible to obtain framing elements such as beams and columns.

It results in minimal loss of timber fibers contributing to round timber's greater strength, as the fibers remain uninterrupted even around natural defects on the log's surface. (Michael H. Ramage & Henry Burridge, 2017).

Nevertheless, the significant negative aspect of round wood poles is that they pose difficulties for systematic building construction. They lack straightness, gradually decreasing width or diameter, and length and cracks. Poles can have a 'sweep' or a 'crook,' which affects straightness. (Chapman, John, 2004).

Therefore it becomes a component challenging to scale up in multistory residential dwellings for two reasons: It has unpredictable behavior when subjected to higher loads due to its natural characteristics on the one hand, and on the other, its section lacks the amount of mass necessary to withstand and transfer a considerable amount of loads in this type of buildings.

Solid Sawn Timber

Structural timber, or "solid sawn timber," is made from rounded timber poles cut from the tree trunk. As a result, its structural characteristics closely resemble those of the tree trunk, which provides rigidity, mechanical strength, and height to support the tree crown and branches (Porteous & Kermani, 2007).

In structural design, sawmills produce linear timber components from the tree trunk's geometrical shape and fiber composition. The resulting shape of the linear timber components often depends on the size and shape of the processed trunk. As a result, the most common structural components produced with solid-sawn timber are linear elements with rectangular, squared, and rounded sections, including beams, columns, trusses, and bracing systems.
In the book Manual of multi-storey timber Construction by Kauffan et Al. (2018), the authors give a complete explanation of strategies to use Solid sawn timber for structural design in buildings according to the component arrangement these are:

- Post and beam structures: This system uses solid-sawn timber columns and beams to support the roof and upper floors, which can be designed in various shapes and sizes to provide specific structural properties by creating an open interior space that can be used for a variety of purposes.

• Timber frame structures: These structures use solid-sawn timber elements to create a load-bearing framework for the building. The timber frame can be designed to support a range of roof and floor systems and can be finished with various exterior and interior finishes.

• Hybrid structures: Hybrid structures combine solid-sawn timber with other building materials, such as steel or concrete, to create a hybrid structural system that takes advantage of the benefits of each material.

• Timber shear wall structures: These structures use solid-sawn timber shear walls to resist lateral loads such as wind or earthquake forces. The timber shear walls can be designed in various shapes and sizes to provide specific structural properties.

In short, solid-sawn timber can be more versatile in shape production compared to its predecessor (the solid rounded timber). However, it prompts a limitation in the design of structures when used as a component as it can be limited to the dimensions of the original tree trunk and its drying process, which might result in restrictions in their use in large-scale building structures.

Plywood and Laminated Veneer Lumber (LVL)

Plywood was the first type of EWP created at the end of the XIX century and consists of a flat panel made by bonding together and, under pressure, a number of thin layers of veneer, often referred to as plies or laminates.

As illustrated in Diagram 2, for its production, trough rotary-peeled process a primary process transforms logs into 2–4 mm thick veneers and clipped into sheets of some 2 m wide. After kiln-drying and gluing, the veneers are laid up with the grain perpendicular to one another and bonded under pressure in an odd number of at least three laminates in sheet sizes of 1200 mm × 2400 mm or 1220 mm ×2440 mm, where the face veneer is generally oriented with the longer side of the sheet in most cases.

On the other hand, Laminated veneer lumber (LVL) was invented in the 60s with the need for a strong, lightweight, and dimensionally stable alternative to traditional solid-sawn lumber, which was becoming scarce and expensive at the time. By using the same principle of Plywood, but alternating the Veener 90s in each layer, LVL could be engineered to have superior strength and durability compared to Plywood.

In timber structural design, Porteous & Kermani (2007) state that these materials' structural properties and strength depend mainly on the number and thickness of each ply, the species and grade, and the arrangement of the individual plies.

As with timber, the structural properties of plywood are functions of the type of applied stresses, their direction concerning the grain direction of the face ply, and the load duration. Therefore being a planar material, Plywood may be subjected to bending in two different planes, depending on its intended use and the direction of the applied stress :

• Loads along the plane slab system): in the plane of the board, as shown in Image 8 for example, in situations where it is used as shelving or floorboard.

• Loads perpendicular to the edge (beam or wall): As shown in Figure XX for example, when it is acting as a web of a flexural member such as in ply-webbed beams.

The use of plywood and LVL as a structural material can be extended as far as its nominal dimensions and thickness allow it to respond to the loading system the component will be subjected to; Unfurtanelly, as we will see in further sections, its capacities can be limited in the design of buildings in a big scale due to its low strength compared to other EPW. Therefore, it's common in the structural design of framed components such as walls and slabs in low-high constructions in single housing projects and small dwellings.

Laminated boards from strand lumber (LSL®) (PSL) and (OSB).

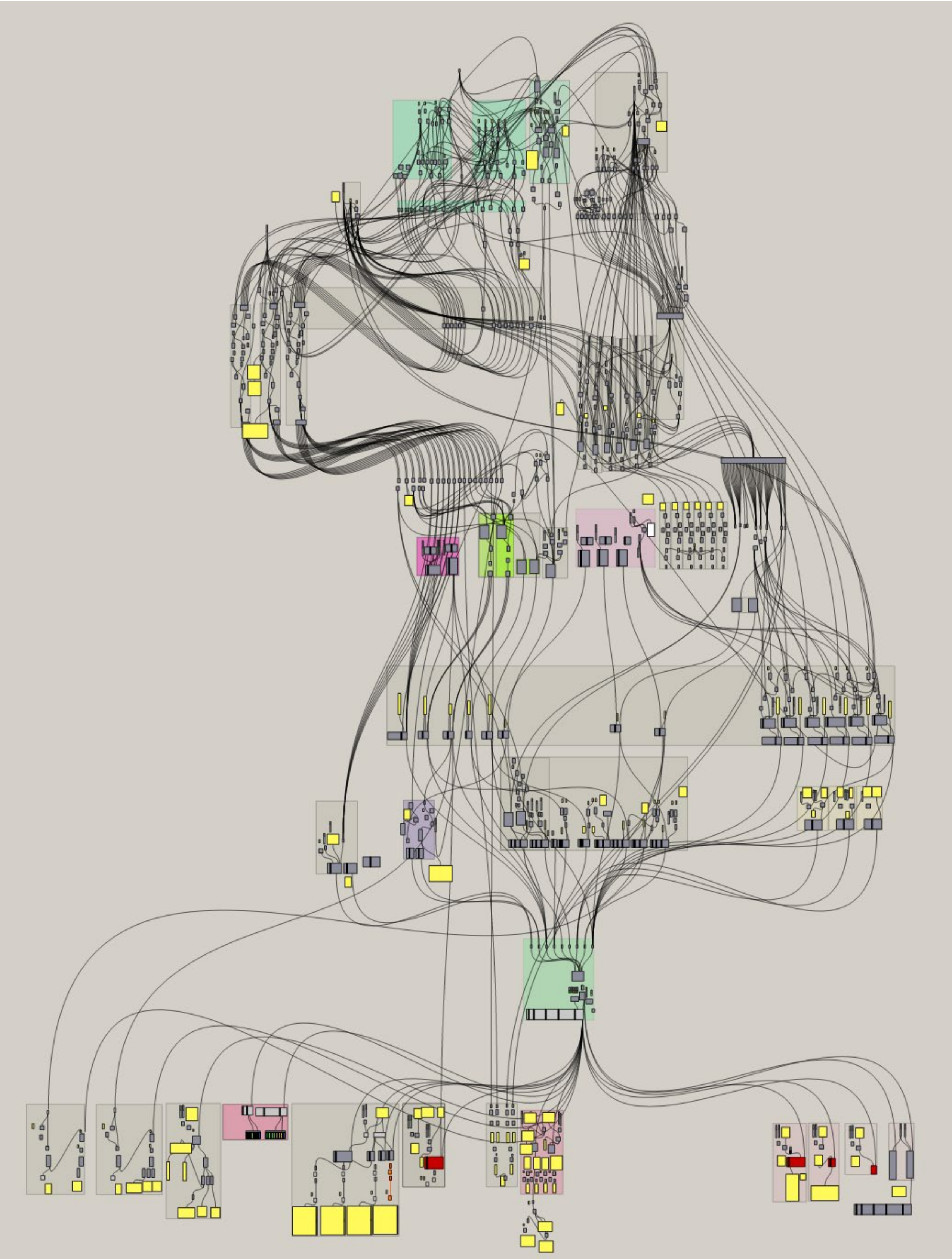
according to Porteous & Kermani (2007), Laminated boards from strand Lumber (LSL) and Parallel Strand Laminated (PSL) are produced from strands of wood species of different species, up to 300 mm in length and 30 mm in width, where different species and combinations are blended with a polyurethane-based adhesive. In the case of PSL, the strands are oriented in a parallel direction, while LSL is made by orienting the strands in layers and then bonding them together with an adhesive under heat and pressure. Due to their production processes, PSL forms mats of 4.75 20 meters long, while LSL forms mats with the same width but with a maximum length of up to 14.63 m long. Various thicknesses can be achieved after pressing the mats by steam injection to achieve the required thickness of up to 140 mm.

On the other hand, Oriented strand Boards (OSB) is an engineered structural board manufactured from thin wood strands, flakes, or wafers sliced from small-diameter round timber logs and bonded with an exterior type adhesive (comprising 95% wood, 5% resin, and wax) under heat and pressure.

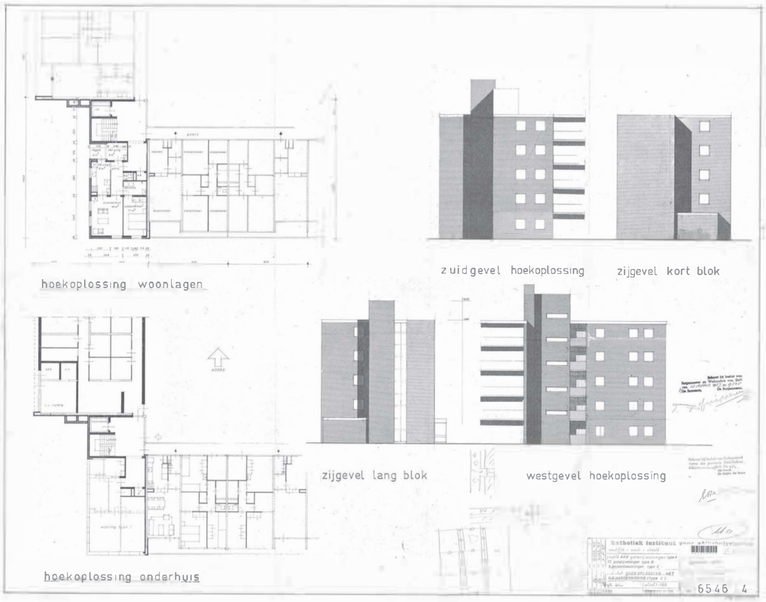
OSB panels comprise exterior or surface layers that are composed of strands oriented in the long panel direction, with inner layers comprising randomly oriented strands. Their strength is mainly due to their multi-layered makeup and the cross-orientation of the strands. The use of water and boil-proof resins/adhesives provides strength, stiffness, and moisture resistance.

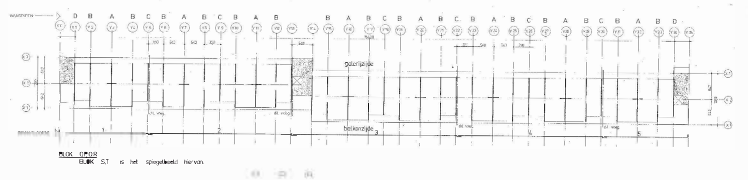
Although it can be produced in a considerably extended length, its structural performance is similar but lower compared with LVL and plywood, making it for structural purposes a material suitable to be used in combined components such as framing for beams and slabs or I-joists beams where its strength capacity can be enhanced by using its considerable dimensional characteristics.

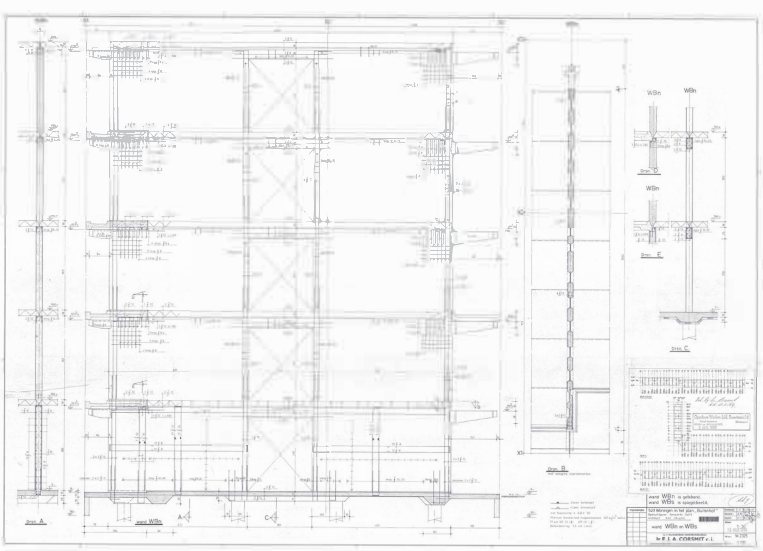
Appendix C – Grashopper defintion script for the whole parametrized 3d models.

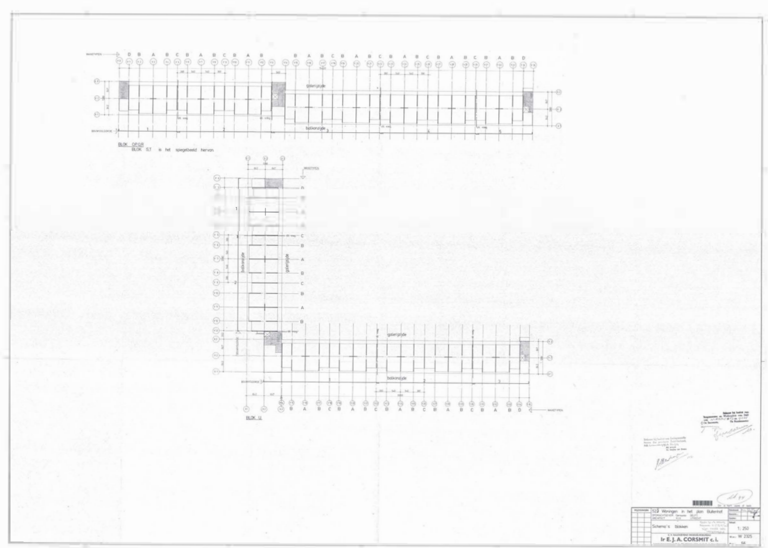


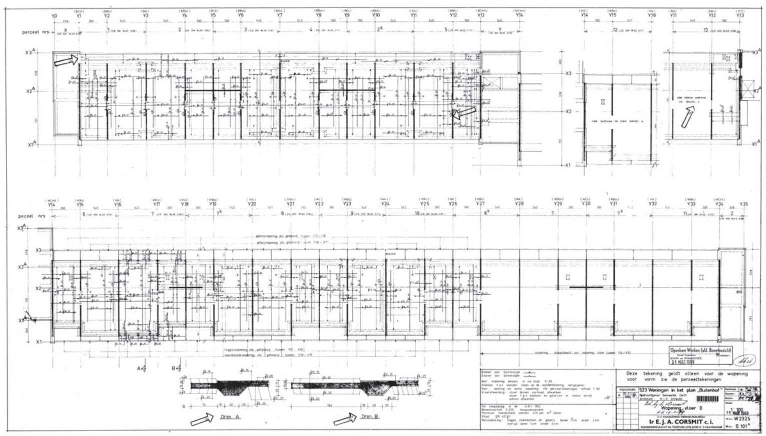
Appendix C – Planimetry of the Gillisbuurt building in Delfts archive

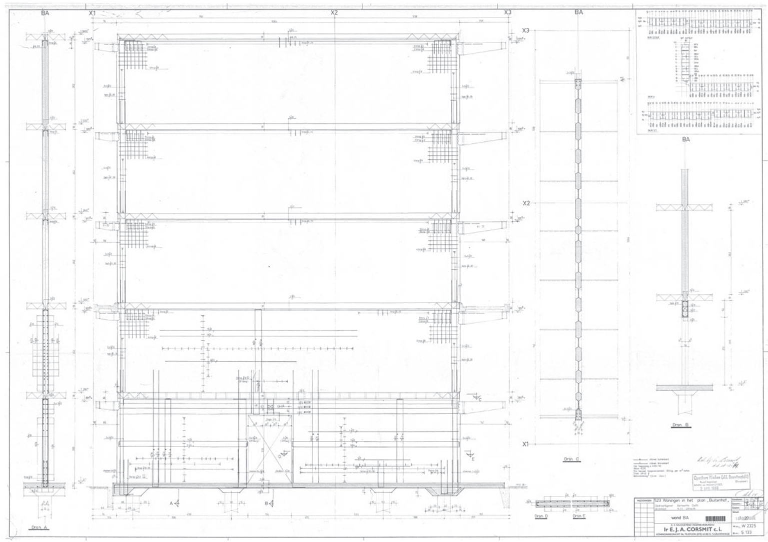


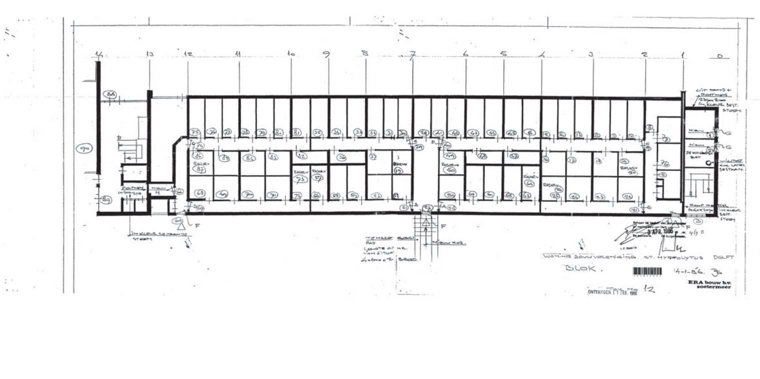







perceel nrs
Deze tekening geldt alleen voor de wapening voor vorm zie de perceeltekeningen
Ir E.J.A. CORSMIT c.i.
Doorsn. A
Doorsn. B


Ir E.J.A. CORSMIT c.i.


BLOK
ERA bouw b.v.
zoetermeer

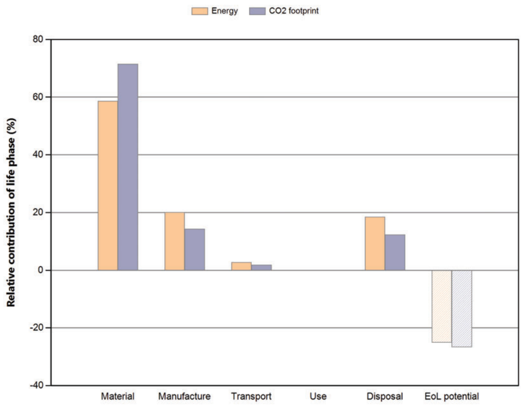
Appendix D – Ecoaudit reports of the existing concrete structure considering three EoL scenarios: Downcycle, Recycle and Reuse.

Ansys GRANTA EDUPACK

Eco Audit Report

Product name	Existing building Downcycle
Country of use	Europe
Product life (years)	56

Summary:



Energy details   CO2 footprint details

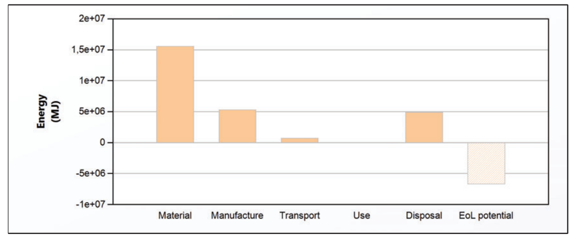
Phase	Energy (MJ)	Energy (%)	CO2 footprint (kg)	CO2 footprint (%)
Material	1,56e+07	58,7	2e+06	71,5
Manufacture	5,34e+06	20,1	4,02e+05	14,4
Transport	7,29e+05	2,7	5,25e+04	1,9
Use	0	0,0	0	0,0
Disposal	4,92e+06	18,5	3,44e+05	12,3
Total (for first life)	**2,66e+07**	**100**	**2,8e+06**	**100**
End of life potential	-6,67e+06		-7,46e+05	

Ansys GRANTA EDUPACK

Eco Audit Report

Energy Analysis

Summary



	Energy (MJ/year)
Equivalent annual environmental burden (averaged over 56 year product life):	4,75e+05

Detailed breakdown of individual life phases

Material:

Summary

Component	Material	Recycled content* (%)	Part mass (kg)	Qty.	Total mass (kg)	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	4,7e+06	1	4,7e+06	3,8e+06	24,4
Prefab. concrete gallery 630 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	1,2e+06	1	1,2e+06	9,6e+05	6,1
Prefab. concrete gallery beams	Concrete (normal, Portland cement)	Virgin (0%)	7,2e+04	1	7,2e+04	5,9e+04	0,4
In situ concrete walls 180 mm	Concrete (normal, Portland cement)	Virgin (0%)	3,5e+06	1	3,5e+06	2,9e+06	18,6
Reinforced steel for components	Structural steel, ASTM A500 Grade A	Virgin (0%)	2,8e+05	1	2,8e+05	7,9e+06	50,5
Total				**5**	**9,7e+06**	**1,6e+07**	**100**

*Typical: Includes 'recycle fraction in current supply'

Manufacture:

Summary

Component	Process	Amount processed	Energy (MJ)	%
Reinforced steel for components	Wire drawing	2,8e+05 kg	5,3e+06	100,0
Total			**5,3e+06**	**100**

Transport:

Summary

Breakdown by transport stage

Stage name	Transport type	Distance (km)	Energy (MJ)	%
Truck	14 tonne (2 axle) truck	50	7,3e+05	100,0
Total		**50**	**7,3e+05**	**100**

Breakdown by components

Component	Mass (kg)	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	4,7e+06	3,5e+05	47,9
Prefab. concrete gallery 630 mm thickness	1,2e+06	8,8e+04	12,1
Prefab. concrete gallery beams	7,2e+04	5,4e+03	0,7
In situ concrete walls 180 mm	3,5e+06	2,7e+05	36,4
Reinforced steel for components	2,8e+05	2,1e+04	2,9
Total	**9,7e+06**	**7,3e+05**	**100**

Use:

Summary

Relative contribution of static and mobile modes

Mode	Energy (MJ)	%
Static	0	
Mobile	0	
Total	**0**	**100**

Disposal:

Summary

Component	End of life option	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Downcycle	2,3e+06	47,3
Prefab. concrete gallery 630 mm thickness	Downcycle	5,9e+05	11,9
Prefab. concrete gallery beams	Downcycle	3,6e+04	0,7
In situ concrete walls 180 mm	Downcycle	1,8e+06	36,0
Reinforced steel for components	Recycle	2e+05	4,0
Total		**4,9e+06**	**100**

EoL potential:

Component	End of life option	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Downcycle	-4,7e+05	7,0
Prefab. concrete gallery 630 mm thickness	Downcycle	-1,2e+05	1,8
Prefab. concrete gallery beams	Downcycle	-7,2e+03	0,1
In situ concrete walls 180 mm	Downcycle	-3,5e+05	5,3
Reinforced steel for components	Recycle	-5,7e+06	85,9
Total		**-6,7e+06**	**100**

Notes:

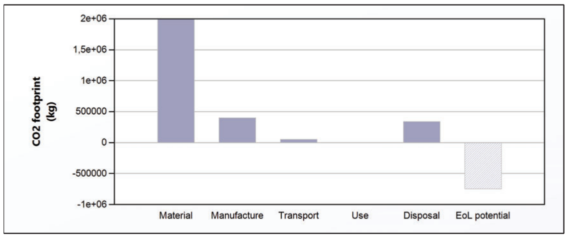
Summary

Ansys
GRANTA EDUPACK

Eco Audit Report

CO2 Footprint Analysis

Summary



	CO2 (kg/year)
Equivalent annual environmental burden (averaged over 56 year product life):	5e+04

Detailed breakdown of individual life phases

Material:

Summary

Component	Material	Recycled content* (%)	Part mass (kg)	Qty.	Total mass (kg)	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	4,7e+06	1	4,7e+06	5,7e+05	28,4
Prefab. concrete gallery 630 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	1,2e+06	1	1,2e+06	1,4e+05	7,1
Prefab. concrete gallery beams	Concrete (normal, Portland cement)	Virgin (0%)	7,2e+04	1	7,2e+04	8,8e+03	0,4
In situ concrete walls 180 mm	Concrete (normal, Portland cement)	Virgin (0%)	3,5e+06	1	3,5e+06	4,3e+05	21,6
Reinforced steel for components	Structural steel, ASTM A500 Grade A	Virgin (0%)	2,8e+05	1	2,8e+05	8,5e+05	42,5
Total				5	9,7e+06	2e+06	100

*Typical: Includes 'recycle fraction in current supply'

Manufacture:

Summary

Component	Process	Amount processed	CO2 footprint (kg)	%
Reinforced steel for components	Wire drawing	2,8e+05 kg	4e+05	100,0
Total			4e+05	100

Transport:

Summary

Breakdown by transport stage

Stage name	Transport type	Distance (km)	CO2 footprint (kg)	%
Truck	14 tonne (2 axle) truck	50	5,2e+04	100,0
Total		50	5,2e+04	100

Breakdown by components

Component	Mass (kg)	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	4,7e+06	2,5e+04	47,9
Prefab. concrete gallery 630 mm thickness	1,2e+06	6,3e+03	12,1
Prefab. concrete gallery beams	7,2e+04	3,9e+02	0,7
In situ concrete walls 180 mm	3,5e+06	1,9e+04	36,4
Reinforced steel for components	2,8e+05	1,5e+03	2,9
Total	9,7e+06	5,2e+04	100

Use:

Summary

Relative contribution of static and mobile modes

Mode	CO2 footprint (kg)	%
Static	0	
Mobile	0	
Total	0	100

Disposal:

Summary

Component	End of life option	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Downcycle	1,6e+05	47,3
Prefab. concrete gallery 630 mm thickness	Downcycle	4,1e+04	11,9
Prefab. concrete gallery beams	Downcycle	2,5e+03	0,7
In situ concrete walls 180 mm	Downcycle	1,2e+05	36,0
Reinforced steel for components	Recycle	1,4e+04	4,0
Total		3,4e+05	100

EoL potential:

Component	End of life option	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Downcycle	-3,3e+04	4,4
Prefab. concrete gallery 630 mm thickness	Downcycle	-8,2e+03	1,1
Prefab. concrete gallery beams	Downcycle	-5e+02	0,1
In situ concrete walls 180 mm	Downcycle	-2,5e+04	3,3
Reinforced steel for components	Recycle	-6,8e+05	91,1
Total		-7,5e+05	100

Notes:

Summary

Ansys
GRANTA EDUPACK

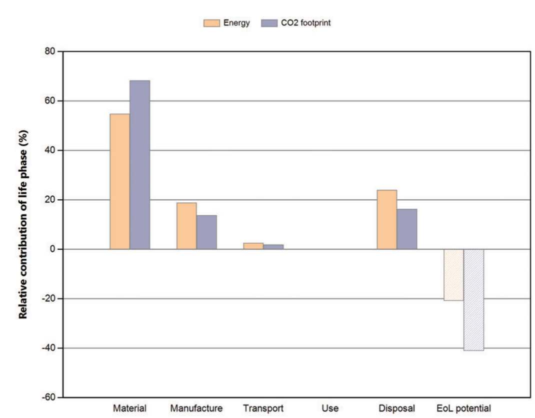
Eco Audit Report

Product name: Existing building recycle

Country of use: Europe

Product life (years): 56

Summary:



Energy details   CO2 footprint details

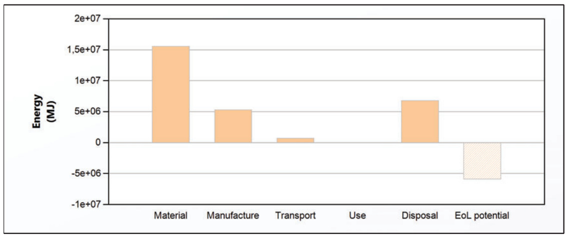
Phase	Energy (MJ)	Energy (%)	CO2 footprint (kg)	CO2 footprint (%)
Material	1,56e+07	54,8	2e+06	68,2
Manufacture	5,35e+06	18,8	4,02e+05	13,7
Transport	7,29e+05	2,6	5,25e+04	1,8
Use	0	0,0	0	0,0
Disposal	6,8e+06	23,9	4,76e+05	16,3
Total (for first life)	2,85e+07	100	2,93e+06	100
End of life potential	-5,93e+06		-1,2e+06	

NOTE: Differences of less than 20% are not usually significant.
See notes on precision and data sources.

Ansys GRANTA EDUPACK — Eco Audit Report

Energy Analysis

Summary



	Energy (MJ/year)
Equivalent annual environmental burden (averaged over 56 year product life):	5,09e+05

Detailed breakdown of individual life phases

Material:

Summary

Component	Material	Recycled content* (%)	Part mass (kg)	Qty.	Total mass (kg)	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	4,7e+06	1	4,7e+06	3,8e+06	24,4
Prefab. concrete gallery 630 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	1,2e+06	1	1,2e+06	9,6e+05	6,1
Prefab. concrete gallery beams	Concrete (normal, Portland cement)	Virgin (0%)	7,2e+04	1	7,2e+04	5,9e+04	0,4
In situ concrete walls 180 mm	Concrete (normal, Portland cement)	Virgin (0%)	3,5e+06	1	3,5e+06	2,9e+06	18,6
Reinforced steel for components	Structural steel, ASTM A500 Grade A	Virgin (0%)	2,8e+05	1	2,8e+05	7,9e+06	50,5
Total				5	9,7e+06	1,6e+07	100

*Typical: Includes 'recycle fraction in current supply'

Manufacture:

Summary

Component	Process	Amount processed	Energy (MJ)	%
Reinforced steel for components	Wire drawing	2,8e+05 kg	5,3e+06	100,0
Total			5,3e+06	100

Transport:

Summary

Breakdown by transport stage

Stage name	Transport type	Distance (km)	Energy (MJ)	%
Truck	14 tonne (2 axle) truck	50	7,3e+05	100,0
Total		50	7,3e+05	100

Breakdown by components

Component	Mass (kg)	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	4,7e+06	3,5e+05	47,9
Prefab. concrete gallery 630 mm thickness	1,2e+06	8,8e+04	12,1
Prefab. concrete gallery beams	7,2e+04	5,4e+03	0,7
In situ concrete walls 180 mm	3,5e+06	2,7e+05	36,4
Reinforced steel for components	2,8e+05	2,1e+04	2,9
Total	9,7e+06	7,3e+05	100

Use:

Summary

Relative contribution of static and mobile modes

Mode	Energy (MJ)	%
Static	0	
Mobile	0	
Total	0	100

Disposal:

Summary

Component	End of life option	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Recycle	3,3e+06	47,9
Prefab. concrete gallery 630 mm thickness	Recycle	8,2e+05	12,1
Prefab. concrete gallery beams	Recycle	5e+04	0,7
In situ concrete walls 180 mm	Recycle	2,5e+06	36,4
Reinforced steel for components	Recycle	2e+05	2,9
Total		6,8e+06	100

EoL potential:

Component	End of life option	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Recycle	-9,8e+04	1,7
Prefab. concrete gallery 630 mm thickness	Recycle	-2,5e+04	0,4
Prefab. concrete gallery beams	Recycle	-1,5e+03	0,0
In situ concrete walls 180 mm	Recycle	-7,4e+04	1,3
Reinforced steel for components	Recycle	-5,7e+06	96,7
Total		-5,9e+06	100

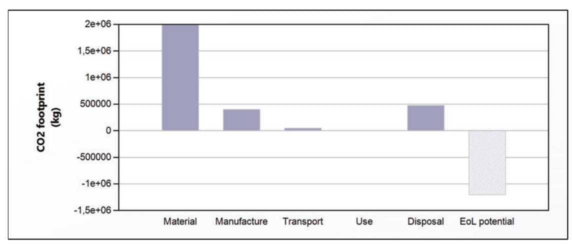
Notes:

Summary

Ansys GRANTA EDUPACK — Eco Audit Report

CO2 Footprint Analysis

Summary



	CO2 (kg/year)
Equivalent annual environmental burden (averaged over 56 year product life):	5,23e+04

Detailed breakdown of individual life phases

Material:

Summary

Component	Material	Recycled content* (%)	Part mass (kg)	Qty.	Total mass (kg)	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	4,7e+06	1	4,7e+06	5,7e+05	28,4
Prefab. concrete gallery 630 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	1,2e+06	1	1,2e+06	1,4e+05	7,1
Prefab. concrete gallery beams	Concrete (normal, Portland cement)	Virgin (0%)	7,2e+04	1	7,2e+04	8,8e+03	0,4
In situ concrete walls 180 mm	Concrete (normal, Portland cement)	Virgin (0%)	3,5e+06	1	3,5e+06	4,3e+05	21,6
Reinforced steel for components	Structural steel, ASTM A500 Grade A	Virgin (0%)	2,8e+05	1	2,8e+05	8,5e+05	42,5
Total				5	9,7e+06	2e+06	100

*Typical: Includes 'recycle fraction in current supply'

Manufacture:

Summary

Component	Process	Amount processed	CO2 footprint (kg)	%
Reinforced steel for components	Wire drawing	2,8e+05 kg	4e+05	100,0
Total			4e+05	100

Transport:

Summary

Breakdown by transport stage

Stage name	Transport type	Distance (km)	CO2 footprint (kg)	%
Truck	14 tonne (2 axle) truck	50	5,2e+04	100,0
Total		50	5,2e+04	100

Breakdown by components

Component	Mass (kg)	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	4,7e+06	2,5e+04	47,9
Prefab. concrete gallery 630 mm thickness	1,2e+06	6,3e+03	12,1
Prefab. concrete gallery beams	7,2e+04	3,9e+02	0,7
In situ concrete walls 180 mm	3,5e+06	1,9e+04	36,4
Reinforced steel for components	2,8e+05	1,5e+03	2,9
Total	9,7e+06	5,2e+04	100

Use:

Summary

Relative contribution of static and mobile modes

Mode	CO2 footprint (kg)	%
Static	0	
Mobile	0	
Total	0	100

Disposal:

Summary

Component	End of life option	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Recycle	2,3e+05	47,9
Prefab. concrete gallery 630 mm thickness	Recycle	5,7e+04	12,1
Prefab. concrete gallery beams	Recycle	3,5e+03	0,7
In situ concrete walls 180 mm	Recycle	1,7e+05	36,4
Reinforced steel for components	Recycle	1,4e+04	2,9
Total		4,8e+05	100

EoL potential:

Component	End of life option	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Recycle	-2,6e+05	21,4
Prefab. concrete gallery 630 mm thickness	Recycle	-6,5e+04	5,4
Prefab. concrete gallery beams	Recycle	-4e+03	0,3
In situ concrete walls 180 mm	Recycle	-2e+05	16,3
Reinforced steel for components	Recycle	-6,8e+05	56,5
Total		-1,2e+06	100

Notes:

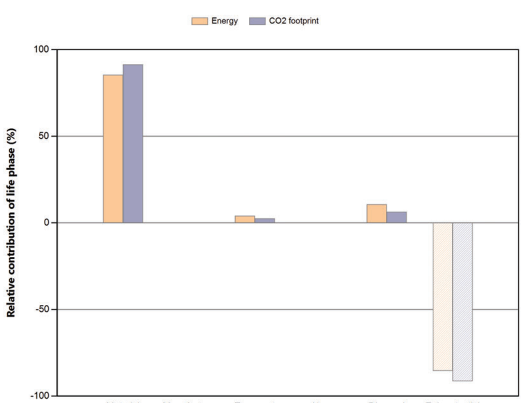
Summary

Eco Audit Report

Product name: Existing building Reuse
Country of use: Europe

Product life (years): 56

Summary:



Energy details   CO2 footprint details

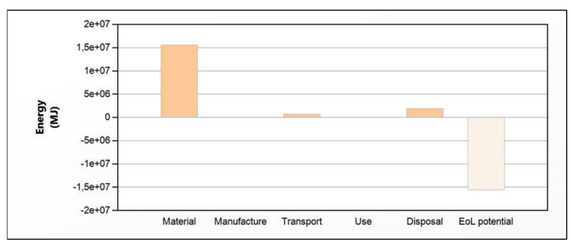
Phase	Energy (MJ)	Energy (%)	CO2 footprint (kg)	CO2 footprint (%)
Material	1,56e+07	85,4	2e+06	91,4
Manufacture	0	0,0	0	0,0
Transport	7,29e+05	4,0	5,25e+04	2,4
Use	0	0,0	0	0,0
Disposal	1,94e+06	10,6	1,36e+05	6,2
Total (for first life)	1,83e+07	100	2,19e+06	100
End of life potential	-1,56e+07		-2e+06	

NOTE: Differences of less than 20% are not usually significant.
See notes on precision and data sources.

Eco Audit Report

Energy Analysis

Summary



	Energy (MJ/year)
Equivalent annual environmental burden (averaged over 56 year product life):	3,26e+05

Detailed breakdown of individual life phases

Material:

Summary

Component	Material	Recycled content* (%)	Part mass (kg)	Qty.	Total mass (kg)	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	4,7e+06	1	4,7e+06	3,8e+06	24,4
Prefab. concrete gallery 630 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	1,2e+06	1	1,2e+06	9,6e+05	6,1
Prefab. concrete gallery beams	Concrete (normal, Portland cement)	Virgin (0%)	7,2e+04	1	7,2e+04	5,9e+04	0,4
In situ concrete walls 180 mm	Concrete (normal, Portland cement)	Virgin (0%)	3,5e+06	1	3,5e+06	2,9e+06	18,6
Reinforced steel for components	Structural steel, ASTM A500 Grade A	Virgin (0%)	2,8e+05	1	2,8e+05	7,9e+06	50,5
Total				5	9,7e+06	1,6e+07	100

*Typical: Includes 'recycle fraction in current supply'

Manufacture:

Summary

Component	Process	Amount processed	Energy (MJ)	%
Total				100

Transport:

Summary

Breakdown by transport stage

Stage name	Transport type	Distance (km)	Energy (MJ)	%
Truck	14 tonne (2 axle) truck	50	7,3e+05	100,0
Total		**50**	**7,3e+05**	**100**

Breakdown by components

Component	Mass (kg)	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	4,7e+06	3,5e+05	47,9
Prefab. concrete gallery 630 mm thickness	1,2e+06	8,8e+04	12,1
Prefab. concrete gallery beams	7,2e+04	5,4e+03	0,7
In situ concrete walls 180 mm	3,5e+06	2,7e+05	36,4
Reinforced steel for components	2,8e+05	2,1e+04	2,9
Total	**9,7e+06**	**7,3e+05**	**100**

Use:

Summary

Relative contribution of static and mobile modes

Mode	Energy (MJ)	%
Static	0	
Mobile	0	
Total	**0**	**100**

Disposal:

Summary

Component	End of life option	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Reuse	9,3e+05	47,9
Prefab. concrete gallery 630 mm thickness	Reuse	2,3e+05	12,1
Prefab. concrete gallery beams	Reuse	1,4e+04	0,7
In situ concrete walls 180 mm	Reuse	7,1e+05	36,4
Reinforced steel for components	Reuse	5,7e+04	2,9
Total		**1,9e+06**	**100**

EoL potential:

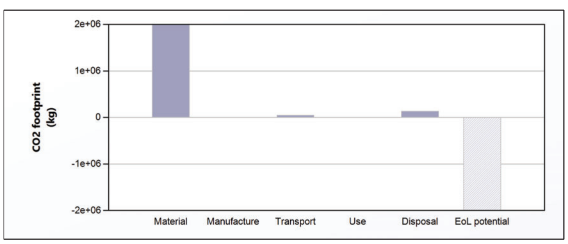
Component	End of life option	Energy (MJ)	%
Prefab. concrete Slabs 180 mm thickness	Reuse	-3,8e+06	24,4
Prefab. concrete gallery 630 mm thickness	Reuse	-9,6e+05	6,1
Prefab. concrete gallery beams	Reuse	-5,9e+04	0,4
In situ concrete walls 180 mm	Reuse	-2,9e+06	18,6
Reinforced steel for components	Reuse	-7,9e+06	50,5
Total		**-1,6e+07**	**100**

Notes:

Summary

CO2 Footprint Analysis

Summary



	CO2 (kg/year)
Equivalent annual environmental burden (averaged over 56 year product life):	3,91e+04

Detailed breakdown of individual life phases

Material:

Summary

Component	Material	Recycled content* (%)	Part mass (kg)	Qty.	Total mass (kg)	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	4,7e+06	1	4,7e+06	5,7e+05	28,4
Prefab. concrete gallery 630 mm thickness	Concrete (normal, Portland cement)	Virgin (0%)	1,2e+06	1	1,2e+06	1,4e+05	7,1
Prefab. concrete gallery beams	Concrete (normal, Portland cement)	Virgin (0%)	7,2e+04	1	7,2e+04	8,8e+03	0,4
In situ concrete walls 180 mm	Concrete (normal, Portland cement)	Virgin (0%)	3,5e+06	1	3,5e+06	4,3e+05	21,6
Reinforced steel for components	Structural steel, ASTM A500 Grade A	Virgin (0%)	2,8e+05	1	2,8e+05	8,5e+05	42,5
Total				**5**	**9,7e+06**	**2e+06**	**100**

*Typical: Includes 'recycle fraction in current supply'

Manufacture:

Summary

Component	Process	Amount processed	CO2 footprint (kg)	%
Total				**100**

Transport:

Summary

Breakdown by transport stage

Stage name	Transport type	Distance (km)	CO2 footprint (kg)	%
Truck	14 tonne (2 axle) truck	50	5,2e+04	100,0
Total		**50**	**5,2e+04**	**100**

Breakdown by components

Component	Mass (kg)	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	4,7e+06	2,5e+04	47,9
Prefab. concrete gallery 630 mm thickness	1,2e+06	6,3e+03	12,1
Prefab. concrete gallery beams	7,2e+04	3,9e+02	0,7
In situ concrete walls 180 mm	3,5e+06	1,9e+04	36,4
Reinforced steel for components	2,8e+05	1,5e+03	2,9
Total	**9,7e+06**	**5,2e+04**	**100**

Use:

Summary

Relative contribution of static and mobile modes

Mode	CO2 footprint (kg)	%
Static	0	
Mobile	0	
Total	**0**	**100**

Disposal:

Summary

Component	End of life option	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Reuse	6,5e+04	47,9
Prefab. concrete gallery 630 mm thickness	Reuse	1,6e+04	12,1
Prefab. concrete gallery beams	Reuse	1e+03	0,7
In situ concrete walls 180 mm	Reuse	5e+04	36,4
Reinforced steel for components	Reuse	4e+03	2,9
Total		**1,4e+05**	**100**

EoL potential:

Component	End of life option	CO2 footprint (kg)	%
Prefab. concrete Slabs 180 mm thickness	Reuse	-5,7e+05	28,4
Prefab. concrete gallery 630 mm thickness	Reuse	-1,4e+05	7,1
Prefab. concrete gallery beams	Reuse	-8,8e+03	0,4
In situ concrete walls 180 mm	Reuse	-4,3e+05	21,6
Reinforced steel for components	Reuse	-8,5e+05	42,5
Total		**-2e+06**	**100**

Notes:

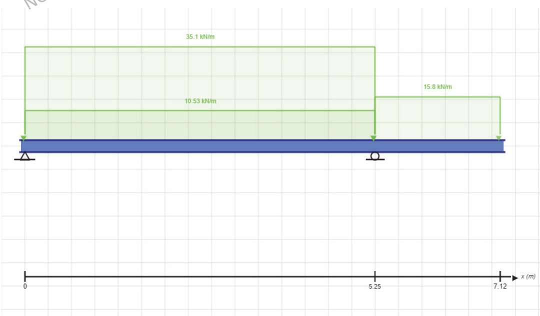
Summary

Appendix E – Predimension of glulam Column using the software SkyCiv

SkyCiv
STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:54:18 GMT+0200 (Central European Summer Time)

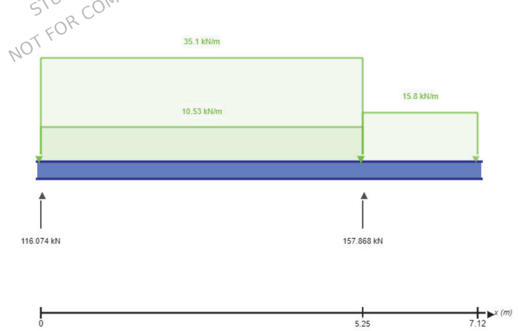
Project Info

File Name: Beam_D (Fire)
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

Free Body Diagram (FBD)
Analysis Summary
Analysis Results
Bending Moment Diagram (BMD)
Shear Force Diagram (SFD)
Deflection Results
Stress Results
Beam Section

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.909	L/275
Custom Stress Limit	PASS	250 MPa	0.032	8.063 MPa

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	116.074 kN	0 kN-m
5.25	0 kN	157.868 kN	0 kN-m

Force Extremes

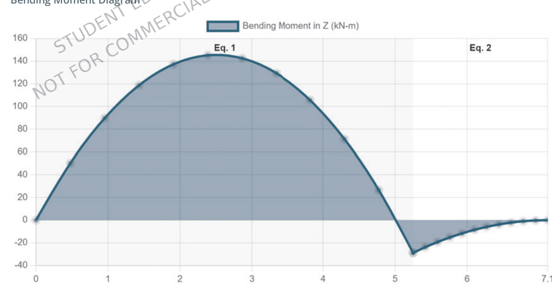
Result	Max	Min
Bending Moment	145.135 kN-m	-28.814 kN-m
Shear	116.074 kN	-127.051 kN
Displacement	6.779 mm	-6.832 mm

Stress Extremes

Result	Max	Min
Bending Stress	8.063 MPa	-8.063 MPa
Shear Stress Total	1.061 MPa	0 MPa
Max Combined Normal Stress	8.063 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-8.063 MPa

DIAGRAMS

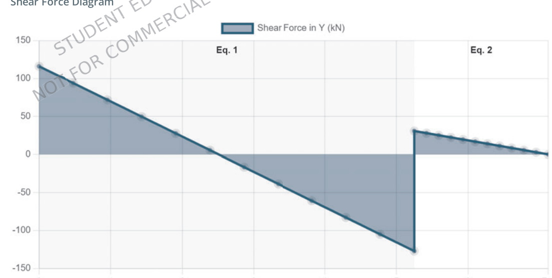
Bending Moment Diagram



Bending Moment Equations

Eq. 1 $M_1(x) = -23.244x^2 + 116.485x$ for $0 \le x \le 5.25$
Eq. 2 $M_2(x) = -8.329x^2 + 118.612x - 422.259$ for $5.25 \le x \le 7.12$

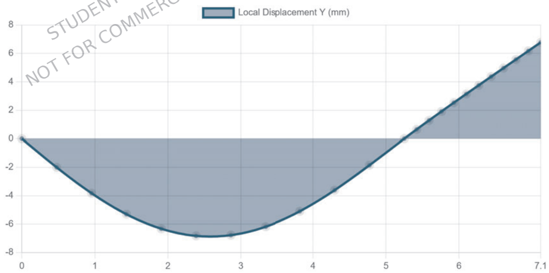
Shear Force Diagram



Shear Force Equations

Eq. 1 $V_1(x) = -46.489x + 116.485$ for $0 \le x \le 5.25$
Eq. 2 $V_2(x) = -16.659x + 118.612$ for $5.25 \le x \le 7.12$

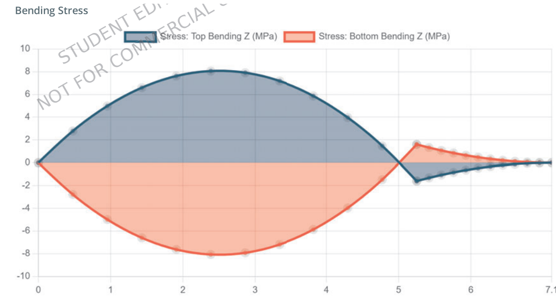
Displacement



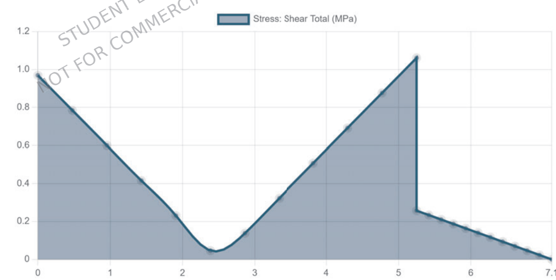
Location (m)	Total Deflection (mm)	Span
0	0 mm	-
2.386	6.832 mm	L/768
5.25	0 mm	-
7.12	6.779 mm	L/275

The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the **model settings**.

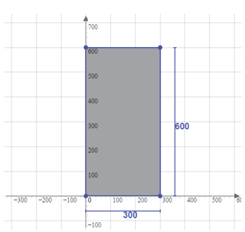
Bending Stress



Shear Stress



Beam Section



Geometric Properties		
A	180000	mm^2
C_z	150	mm
C_y	300	mm

Bending Properties		
I_z	5400000000	mm^4
I_y	1350000000	mm^4

Shear Properties		
A_z	150002.869	mm^2
A_y	150000.16	mm^2

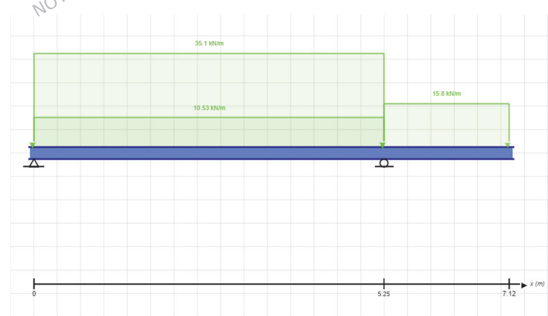
Torsion Properties		
J	3704782192.989	mm^4
r	278.974	mm

Shape	Material	E (MPa)	v	ρ (kg/m^3)
Rectangular	Glulam C24 Spruce	11000	0.4	385

SkyCiv
STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:55:44 GMT+0200 (Central European Summer Time)

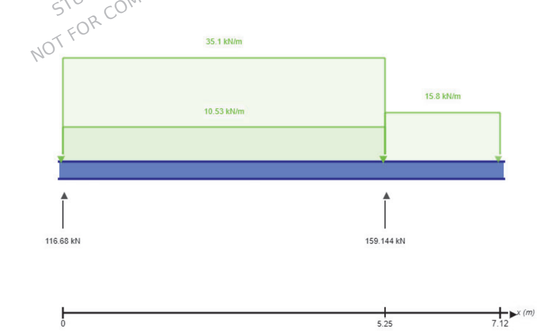
Project Info

File Name: Beam_D (Fire)
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

Free Body Diagram (FBD)
Analysis Summary
Analysis Results
Bending Moment Diagram (BMD)
Shear Force Diagram (SFD)
Deflection Results
Stress Results
Beam Section

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.522	L/479
Custom Stress Limit	PASS	250 MPa	0.021	5.208 MPa

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	116.68 kN	0 kN-m
5.25	0 kN	159.144 kN	0 kN-m

Force Extremes

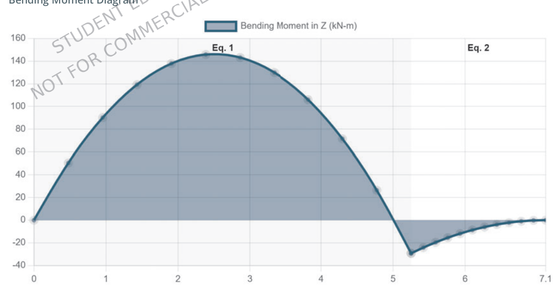
Result	Max	Min
Bending Moment	145.828 kN-m	-29.276 kN-m
Shear	116.68 kN	-127.833 kN
Displacement	3.901 mm	-3.939 mm

Stress Extremes

Result	Max	Min
Bending Stress	5.208 MPa	-5.208 MPa
Shear Stress Total	0.768 MPa	0 MPa
Max Combined Normal Stress	5.208 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-5.208 MPa

DIAGRAMS

Bending Moment Diagram

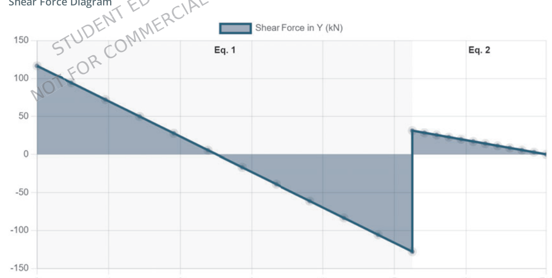


Bending Moment Equations

Eq. 1 $M_1(x) = -23.244x^2 + 116.485x$ for $0 \le x \le 5.25$

Eq. 2 $M_2(x) = -8.329x^2 + 118.612x - 422.259$ for $5.25 \le x \le 7.12$

Shear Force Diagram

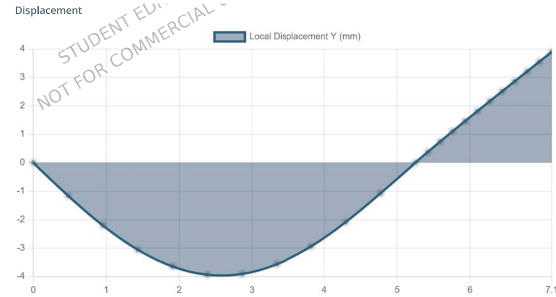


Shear Force Equations

Eq. 1 $V_1(x) = -46.489x + 116.485$ for $0 \le x \le 5.25$

Eq. 2 $V_2(x) = -16.659x + 118.612$ for $5.25 \le x \le 7.12$

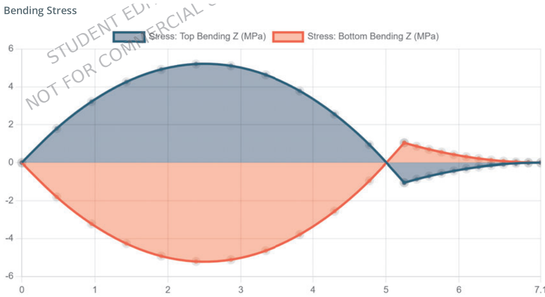
Displacement



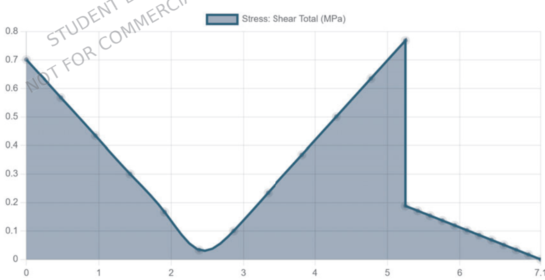
Location (m)	Total Deflection (mm)	Span
0	0 mm	-
2.386	3.939 mm	L/1332
5.25	0 mm	-
7.12	3.901 mm	L/479

The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the **model settings**.

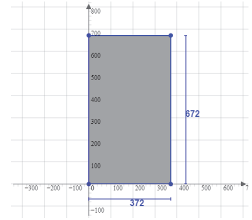
Bending Stress



Shear Stress



Beam Section



Geometric Properties		
A	249984	mm^2
C_z	186	mm
C_y	336	mm

Bending Properties		
I_z	9407397888	mm^4
I_y	2882815488	mm^4

Shear Properties		
A_z	208322.905	mm^2
A_y	208320.237	mm^2

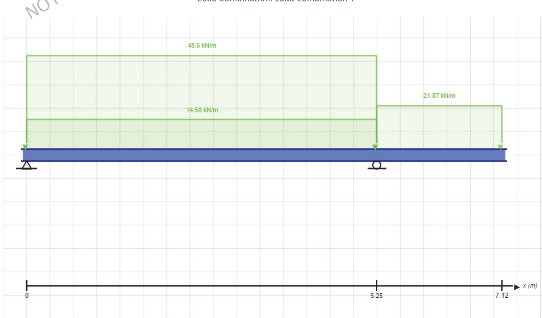
Torsion Properties		
J	7535793623.698	mm^4
r	336.662	mm

Shape	Material	E (MPa)	v	ρ (kg/m^3)
Rectangular	Glulam C24 Spruce	11000	0.4	385

SkyCiv
STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:37:19 GMT+0200 (Central European Summer Time)

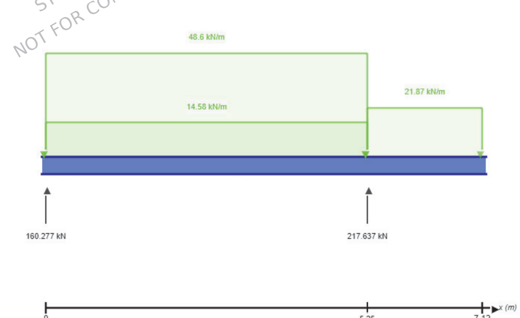
Project Info

File Name: Beam_C (Fire)
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

Free Body Diagram (FBD)
Analysis Summary
Analysis Results
Bending Moment Diagram (BMD)
Shear Force Diagram (SFD)
Deflection Results
Stress Results
Beam Section

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.943	L/265
Custom Stress Limit	PASS	250 MPa	0.037	9.204 MPa

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	160.277 kN	0 kN-m
5.25	0 kN	217.637 kN	0 kN-m

Force Extremes

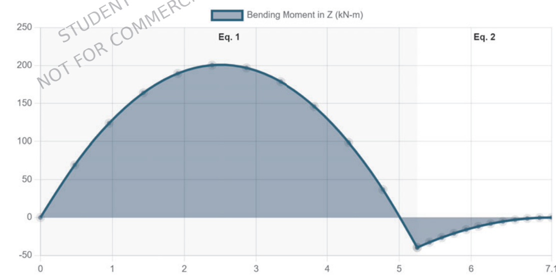
Result	Max	Min
Bending Moment	200.455 kN-m	-39.546 kN-m
Shear	160.277 kN	-175.342 kN
Displacement	7.042 mm	-7.09 mm

Stress Extremes

Result	Max	Min
Bending Stress	9.204 MPa	-9.204 MPa
Shear Stress Total	1.331 MPa	0 MPa
Max Combined Normal Stress	9.204 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-9.204 MPa

DIAGRAMS

Bending Moment Diagram

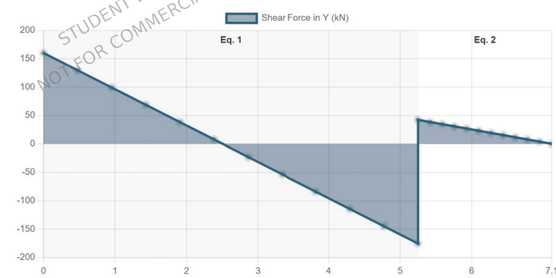


Bending Moment Equations

Eq. 1 $M_1(x) = -32.019x^2 + 160.533x$ for $0 \le x \le 5.25$

Eq. 2 $M_2(x) = -11.364x^2 + 161.83x - 576.115$ for $5.25 \le x \le 7.12$

Shear Force Diagram

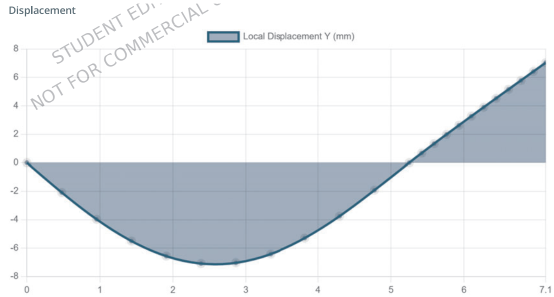


Shear Force Equations

Eq. 1 $V_1(x) = -64.039x + 160.533$ for $0 \le x \le 5.25$

Eq. 2 $V_2(x) = -22.729x + 161.83$ for $5.25 \le x \le 7.12$

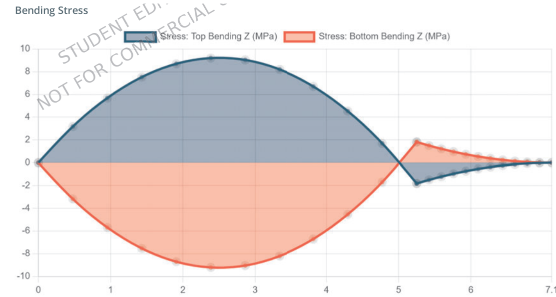
Displacement



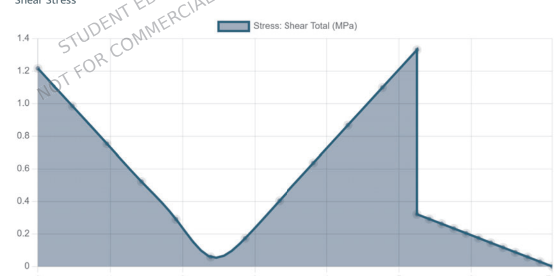
Location (m)	Total Deflection (mm)	Span
0	0 mm	-
2.386	7.09 mm	L/740
5.25	0 mm	-
7.12	7.042 mm	L/265

The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the **model settings.**

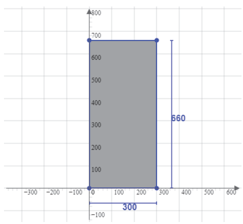
Bending Stress



Shear Stress



Beam Section



Geometric Properties		
A	198000	mm^2
C_z	150	mm
C_y	330	mm

Bending Properties		
I_z	7187400000	mm^4
I_y	1485000000	mm^4

Shear Properties		
A_z	165003.491	mm^2
A_y	165000.151	mm^2

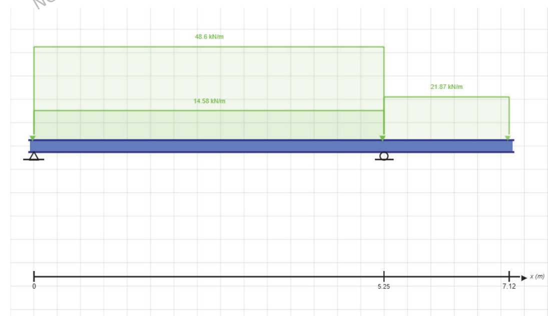
Torsion Properties		
J	4241839098.627	mm^4
r	284.587	mm

Shape	Material	E (MPa)	v	ρ (kg/m^3)
Rectangular	Glulam C24 Spruce	11000	0.4	385

SkyCiv
STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



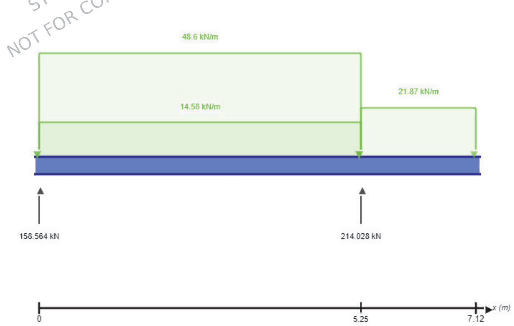
Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:36:31 GMT+0200 (Central European Summer Time)

Project Info

File Name: Beam_C
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.473	L/529
Custom Stress Limit	PASS	250 MPa	0.02	5.079 MPa

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	158.564 kN	0 kN-m
5.25	0 kN	214.028 kN	0 kN-m

Force Extremes

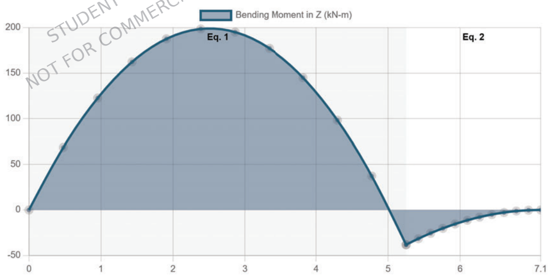
Result	Max	Min
Bending Moment	198.495 kN-m	-38.239 kN-m
Shear	158.564 kN	-173.131 kN
Displacement	3.529 mm	-3.54 mm

Stress Extremes

Result	Max	Min
Bending Stress	5.079 MPa	-5.079 MPa
Shear Stress Total	0.81 MPa	0 MPa
Max Combined Normal Stress	5.079 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-5.079 MPa

DIAGRAMS

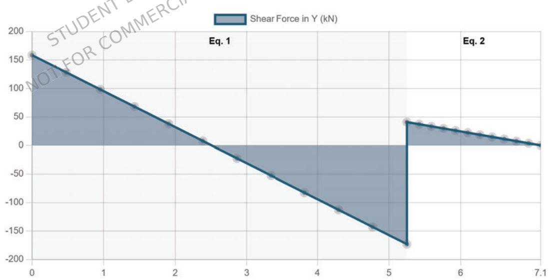
Bending Moment Diagram



Bending Moment Equations

Eq. 1 $M_1(x) = -31.59x^2 + 158.564x$ for $0 \le x \le 5.25$
Eq. 2 $M_2(x) = -10.935x^2 + 155.714x - 554.344$ for $5.25 \le x \le 7.12$

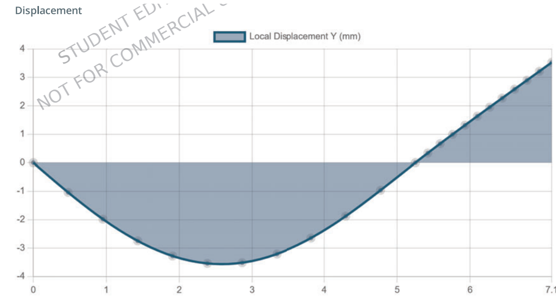
Shear Force Diagram



Shear Force Equations

Eq. 1 $V_1(x) = -63.18x + 158.564$ for $0 \le x \le 5.25$
Eq. 2 $V_2(x) = -21.87x + 155.714$ for $5.25 \le x \le 7.12$

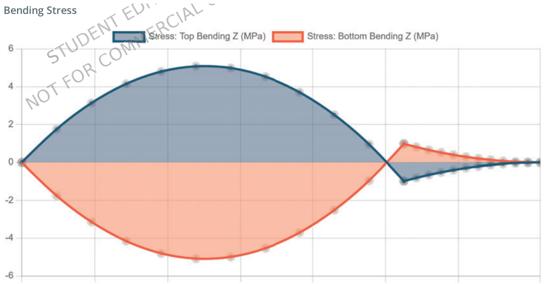
Displacement



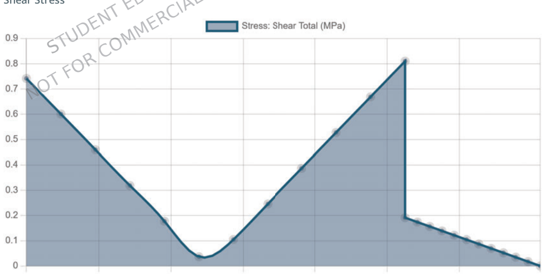
Location (m)	Total Deflection (mm)	Span
0	0 mm	-
2.386	3.54 mm	L/1483
5.25	0 mm	-
7.12	3.529 mm	L/529

The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the **model settings.**

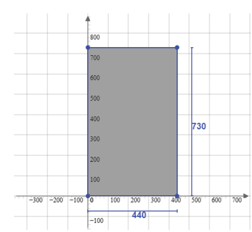
Bending Stress



Shear Stress



Beam Section



Geometric Properties		
A	321200	mm^2
C_z	220	mm
C_y	365	mm

Bending Properties		
I_z	14263956666.667	mm^4
I_y	5182026666.667	mm^4

Shear Properties		
A_z	267668.785	mm^2
A_y	267666.947	mm^2

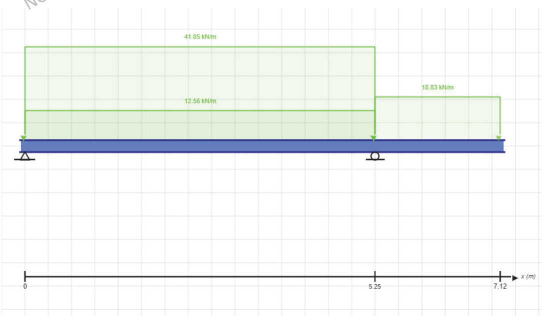
Torsion Properties		
J	12939266587.874	mm^4
r	387.561	mm

Shape	Material	E (MPa)	v	ρ (kg/m^3)
Rectangular	Glulam C24 Spruce	11000	0.4	385

SkyCiv
STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:34:18 GMT+0200 (Central European Summer Time)

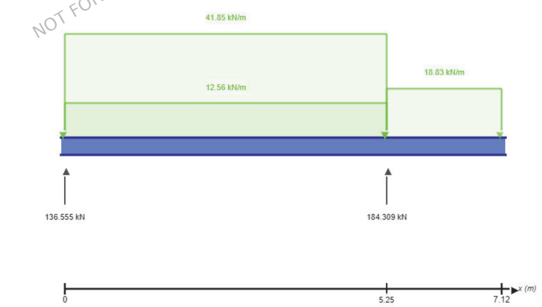
Project Info

File Name: Beam_B (Fire)
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

Free Body Diagram (FBD)
Analysis Summary
Analysis Results
Bending Moment Diagram (BMD)
Shear Force Diagram (SFD)
Deflection Results
Stress Results
Beam Section

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.977	L/256
Custom Stress Limit	PASS	250 MPa	0.036	8.894 MPa

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	136.555 kN	0 kN-m
5.25	0 kN	184.309 kN	0 kN-m

Force Extremes

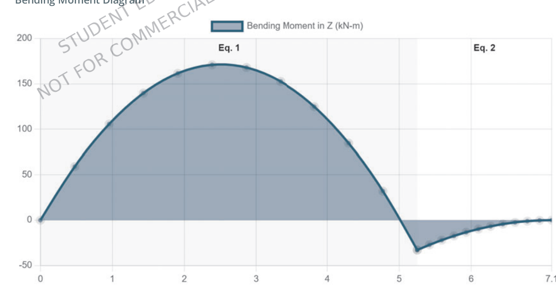
Result	Max	Min
Bending Moment	170.945 kN-m	-32.923 kN-m
Shear	136.555 kN	-149.097 kN
Displacement	7.277 mm	-7.298 mm

Stress Extremes

Result	Max	Min
Bending Stress	8.894 MPa	-8.894 MPa
Shear Stress Total	1.205 MPa	0 MPa
Max Combined Normal Stress	8.894 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-8.894 MPa

DIAGRAMS

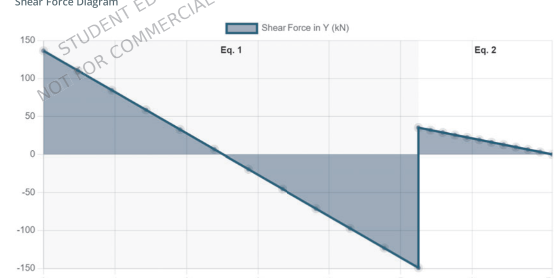
Bending Moment Diagram



Bending Moment Equations

Eq. 1 $M_1(x) = -27.205x^2 + 136.555x$ for $0 \le x \le 5.25$
Eq. 2 $M_2(x) = -9.415x^2 + 134.069x - 477.285$ for $5.25 \le x \le 7.12$

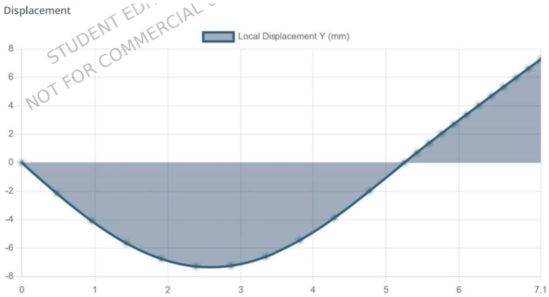
Shear Force Diagram



Shear Force Equations

Eq. 1 $V_1(x) = -54.41x + 136.555$ for $0 \le x \le 5.25$
Eq. 2 $V_2(x) = -18.83x + 134.069$ for $5.25 \le x \le 7.12$

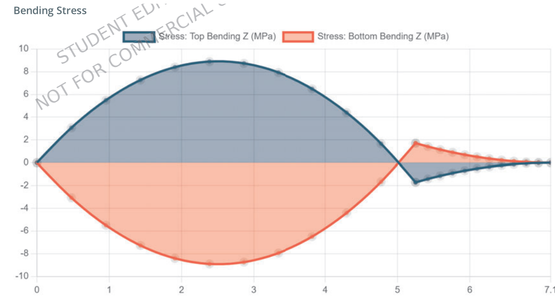
Displacement



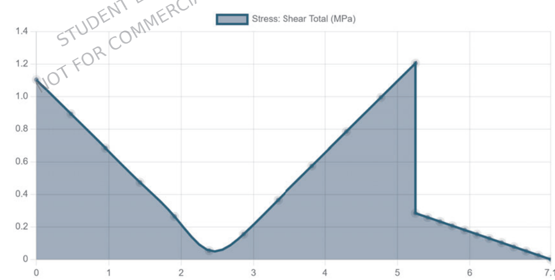
Location (m)	Total Deflection (mm)	Span ℹ
0	0 mm	-
2.386	7.298 mm	L/719
5.25	0 mm	-
7.12	7.277 mm	L/256

ℹ The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the **model settings.**

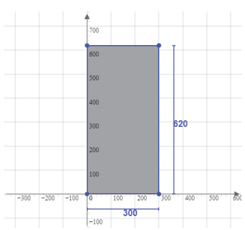
Bending Stress



Shear Stress



Beam Section



Geometric Properties		
A	186000	mm^2
C_z	150	mm
C_y	310	mm

Bending Properties		
I_z	5958200000	mm^4
I_y	1395000000	mm^4

Shear Properties		
A_z	155003.094	mm^2
A_y	155000.187	mm^2

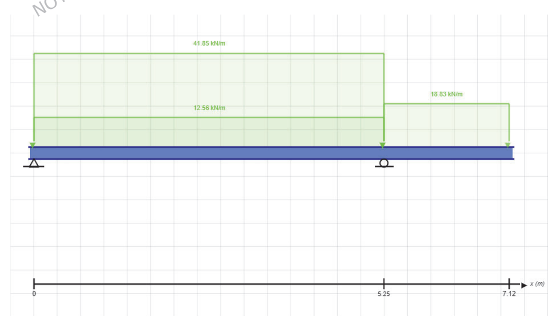
Torsion Properties		
J	3883608731.216	mm^4
r	281.145	mm

Shape	Material	E (MPa)	v	ρ (kg/m^3)
Rectangular	Glulam C24 Spruce	11000	0.4	385

SkyCiv
STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:33:24 GMT+0200 (Central European Summer Time)

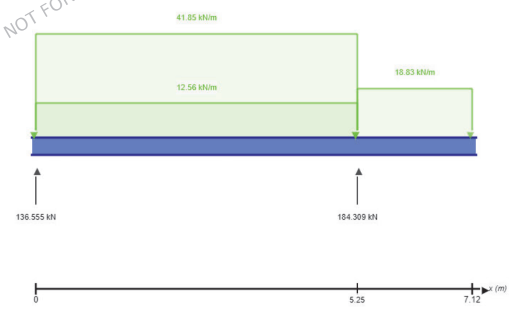
Project Info

File Name: Beam_B
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

Free Body Diagram (FBD)
Analysis Summary
Analysis Results
Bending Moment Diagram (BMD)
Shear Force Diagram (SFD)
Deflection Results
Stress Results
Beam Section

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.564	L/443
Custom Stress Limit	PASS	250 MPa	0.023	5.758 MPa

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	136.555 kN	0 kN-m
5.25	0 kN	184.309 kN	0 kN-m

Force Extremes

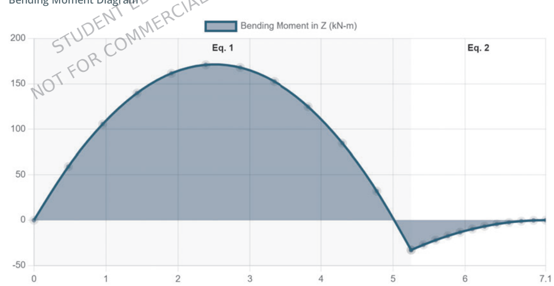
Result	Max	Min
Bending Moment	170.945 kN-m	-32.923 kN-m
Shear	136.555 kN	-149.097 kN
Displacement	4.221 mm	-4.233 mm

Stress Extremes

Result	Max	Min
Bending Stress	5.758 MPa	-5.758 MPa
Shear Stress Total	0.87 MPa	0 MPa
Max Combined Normal Stress	5.758 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-5.758 MPa

DIAGRAMS

Bending Moment Diagram

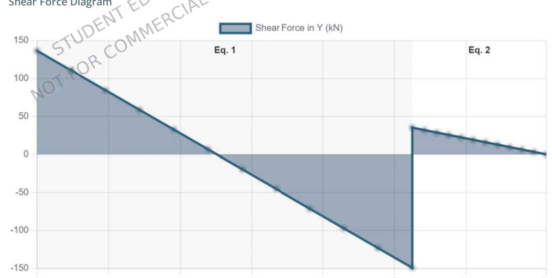


Bending Moment Equations

Eq. 1 $M_1(x) = -27.205x^2 + 136.555x$ for $0 \le x \le 5.25$

Eq. 2 $M_2(x) = -9.415x^2 + 134.069x - 477.285$ for $5.25 \le x \le 7.12$

Shear Force Diagram

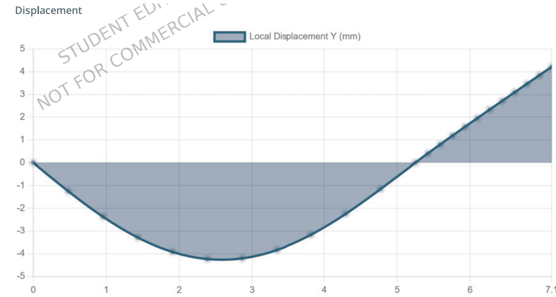


Shear Force Equations

Eq. 1 $V_1(x) = -54.41x + 136.555$ for $0 \le x \le 5.25$

Eq. 2 $V_2(x) = -18.83x + 134.069$ for $5.25 \le x \le 7.12$

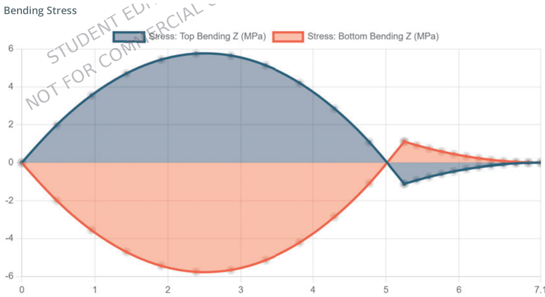
Displacement



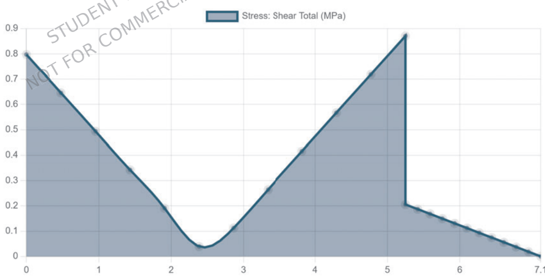
Location (m)	Total Deflection (mm)	Span
0	0 mm	-
2.386	4.233 mm	L/1240
5.25	0 mm	-
7.12	4.221 mm	L/443

The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the **model settings.**

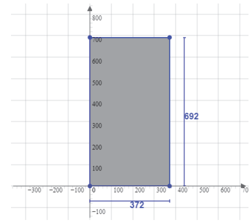
Bending Stress



Shear Stress



Beam Section



Geometric Properties		
A	257424	mm^2
C_z	186	mm
C_y	346	mm

Bending Properties		
I_z	10272590528	mm^4
I_y	2968613568	mm^4

Shear Properties		
A_z	214523.137	mm^2
A_y	214520.234	mm^2

Torsion Properties		
J	7874682386.648	mm^4
r	339.51	mm

Shape	Material	E (MPa)	v	ρ (kg/m^3)
Rectangular	Glulam C24 Spruce	11000	0.4	385

SkyCiv
STUDENT VERSION

Beam F (fire) SkyCiv Report

Wed May 03 2023 18:43:05 GMT+0200 (Central European Summer Time)

File Name: Beam F (fire)
Software: SkyCiv Structural 3D v6.0.1 (Lic. No.: 1N2hF9H4tY4w7B6tUBX9o43u)
Analysis Type: Linear Static & Buckling Analysis

Included in this Report:
Job Setup
Bill of Materials
Nodal Results

Table of Contents

Job Setup

Beam F (fire) SkyCiv Report
SkyCiv Structural 3D v6.0.1
Licence Number: 1N2hF9H4tY4w7B6tUBX9o43u
Date: Wed May 03 2023 18:43:05 GMT+0200 (Central European Summer Time)
Analysis Type: LINEAR STATIC & BUCKLING ANALYSIS

File Name	Beam F (fire)
Job Name	Empty
Designer	Empty
Job Description	Empty

Length Units	m
Section Length Units	mm
Force Units	kN
Moment and Torsion Units	kN-m
Pressure Units	kPa
Material Strength Units	MPa
Material Density Units	kg/m³
Mass Units	kg
Temperature Units	degC
Translation Units	mm
Stress Units	MPa
Nodes	3
Members	2
Plates	0
Meshed Plates	0
Supports	3
Sections	1
Point Loads	0
Distributed Loads	8
Moments	0
Member Prestress Loads	0
Thermal Loads	0
Pressures	0
Area Loads	0
Self Weight	ON
User Defined Nodal Masses	0
Auto Defined Nodal Masses	0
Spectral Loads	0
Member Evaluation Points	9
Extrapolate Plate Results From Gauss Points	YES
General Constraint	RRRRRR
Total Degrees of Freedom	8

NODE COORDINATES (m)

ID	X Coordinate	Y Coordinate	Z Coordinate
1	0.000	0.000	0.000
2	3.900	0.680	0.000
3	9.300	1.630	0.000

MEMBERS (deg, m, mm, m, kN)

F=Fixed, R=Released, S=Spring

ID	Node A	Node B	Length	Type	Section ID	Rotation Angle	Node A Fixity	Node B Fixity
1	1	2	3.959	Normal	4	0.000	FFFFFF	FFFFFF
2	2	3	5.483	Normal	4	0.000	FFFFFF	FFFFFF

ID	Node A Offsets	Node B Offsets	Offset Axis	Node A Rot Stiff Y	Node A Rot Stiff Z	Node B Rot Stiff Y	Node B Rot Stiff Z	Cable Length	T/C Limit	Mirror	Disable Non-Linear
1	0, 0, 0	0, 0, 0	Local	-	-	-	-	-	-	No	No
2	0, 0, 0	0, 0, 0	Local	-	-	-	-	-	-	No	No

SUPPORTS (kN/m, kN-m/rad)

F=Fixed, R=Released, S=Spring

B=Both Axes Restraint, N=Negative-Axis Restraint Only, P=Positive-Axis Restraint Only

ID	Node ID	Restraint Code	Direction Code	X Trans Stiffness	Y Trans Stiffness	Z Trans Stiffness	X Rot Stiffness	Y Rot Stiffness	Z Rot Stiffness	Source
1	3	RFFRRR	BBBBBB	-	-	-	-	-	-	User Defined Nodal Support
2	2	RFFRRR	BBBBBB	-	-	-	-	-	-	User Defined Nodal Support
3	1	FFFFFF	BBBBBB	-	-	-	-	-	-	User Defined Nodal Support

MATERIALS (MPa, kg/m³, 10⁻⁶/degC)

Shear Modulus is used for members only.
Ex, Ey, Gxy, Gxz, Gyz are used for orthotropic plates only.

ID	Name	Young's Modulus	Shear Modulus	Density	Poisson's Ratio	Thermal Exp. Coeff.	Young's Modulus x	Young's Modulus y	Shear Modulus xy	Shear Modulus xz	Shear Modulus yz	Poisson's Ratio xy
8	Glulam C24 Spruce	11000.000	Auto	385.000	0.400	54.000	-	-	-	-	-	Auto

SECTIONS (mm, mm², mm⁴, deg)

ID	Name	Shape	Material ID	Depth	Width	Shear Area z (STRESS)	Shear Area y (STRESS)	Shear Area z (TIMO)	Shear Area y (TIMO)	Torsion Radius
4	Glulam beam - 440 x 300	Rectangular	8	440.000	300.000	110000.606	110000.141	-	-	251.717

ID	Centroid y	Centroid z	Area (A)	y-Axis MoI (Iy)	z-Axis MoI (Iz)	Torsion Constant (J)	Principal Angle	Non Prismatic
4	220.000	150.000	132000.000	990000000.000	2129600000.000	2291833964.074	0.000	N

ID	Area Red. Factor	Iy Red. Factor	Iz Red. Factor	Torsion Constant Red. Factor	Shear Area Y Red. Factor	Shear Area Z Red. Factor
4	-	-	-	-	-	-

MEMBER DISTRIBUTED LOADS (kN/m)

ID	Load Group	Start Position (%)	End Position (%)	Member	Axes	X Start/ End	Y Start/ End	Z Start/ End
1	LG	0%	100%	1	Global Proj.	0.000	-7.880	0.000
						0.000	-7.880	0.000
2	LG	0%	100%	2	Global Proj.	0.000	-7.880	0.000
						0.000	-7.880	0.000
3	LG	0%	100%	1	Global	0.000	-3.540	0.000
						0.000	-3.540	0.000
4	LG	0%	100%	2	Global	0.000	-3.540	0.000
						0.000	-3.540	0.000
5	LG	0%	100%	1	Global	0.000	-4.020	0.000
						0.000	-4.020	0.000
6	LG	0%	100%	2	Global	0.000	-4.020	0.000
						0.000	-4.020	0.000
7	LG	0%	100%	1	Global	0.000	0.000	17.500
						0.000	0.000	17.500
8	LG	0%	100%	2	Global	0.000	0.000	17.500
						0.000	0.000	17.500

SELF WEIGHT (g's)

Load Group	X Gravity	Y Gravity	Z Gravity
SW1	0.000	-1.000	0.000

Bill of Materials

BILL OF MATERIALS FOR MEMBERS (m, kg)

Section	Material	Quantity	Unit Length	Total Length	Unit Mass	Total Mass
4: Glulam beam - 440 x 300	8: Glulam C24 Spruce	1	3.959	3.959	201.188	201.188
4: Glulam beam - 440 x 300	8: Glulam C24 Spruce	1	5.483	5.483	278.643	278.643
						479.831

Nodal Results

Default Load Combo

NODE REACTIONS (kN, kN-m)

Support ID	Node	X Force	Y Force	Z Force	X Moment	Y Moment	Z Moment
3	1	0.000	21.791	-24.094	-1.611	8.793	7.974
2	2	0.000	92.568	-102.406	0.000	0.000	0.000
1	3	0.000	35.011	-38.731	0.000	0.000	0.000
	Reaction Sum	0.000	149.369	-165.231			
	Load Sum	-0.000	-149.369	165.231			
	Equilibrium	0.000	0.000	0.000			

MEMBER END FORCES AND MOMENTS (kN, kN-m)

Member	Node	Axial Force	Y Shear	Z Shear	X Torsion	Y Moment	Z Moment
1	1	3.743	21.467	-24.094	-0.077	8.939	-7.974
	2	-7.016	-40.236	45.186	-0.077	50.689	-45.127
2	2	8.962	50.942	-57.221	0.000	50.689	-45.127
	3	-6.066	-34.481	38.731	0.000	0.000	0.000

NODAL DISPLACEMENTS (mm)

Node	X Translation	Y Translation	Z Translation	Total Translation
1	0.000	0.000	0.000	0.000
2	0.005	0.000	0.000	0.005
3	-0.001	0.000	0.000	0.001

SkyCiv
STUDENT VERSION

Beam F (fire) SkyCiv Report

Wed May 03 2023 18:32:32 4M5G0200 (CeTtra+FnropeaT Snmmer 5ime)

l i+e Eame: Beam F (fire)
Softu are: SkyCiv StrnNtnra+3w v6c0c1 (DiNcEoc 1E2. l Lh 9tH9u YB6t7 BULo93n)
XTa+yAiA 5ype: DiTear StatiNs BnNk+iTg XTa+yAiA

Included in this Report:

&J Setnp
Bi++of Materia+A
Eoda+ReAn+tA

5aJ +e of CoTteTtA

&J Setnp

Beam F (fire) SkyCiv Report
SkyCiv StrnNtnra+3w v6c0c1
DiNeTNe EnmJ er: 1E2. l Lh 9tH9u YB6t7 BULo93n
wate: Wed May 03 2023 18:32:32 4M5G0200 (CeTtra+FnropeaT Snmmer 5ime)
XTa+yAiA 5ype: DiEFXR S5X5bC s B7CODiE4 XEXDHSbS

File Name	Beam (fri)eJ
obE Name	(mpty
Designe)	(mpty
obE Desc)iptibn	(mpty

Length Units	m
Sectibn Length Units	mm
Fb)ce Units	kN
Mbment and Tb)sibn Units	kN-m
P)essu)e Units	kPa
Mate)ial St)ength Units	MPa
Mate)ial Density Units	kg/m³
Mass Units	kg
Tempe)atu)e Units	degC
T)anslatibn Units	mm
St)ess Units	MPa
Nbdes	3
MemEe)s	2
Plates	0
Meshed Plates	0
Suppb)ts	3
Sectibns	1
Pbint Lbads	0
Dist)iEuted Lbads	8
Mbments	0
MemEe) P)est)ess Lbads	0
The)mal Lbads	0
P)essu)es	0
A)ea Lbads	0
Selr Weight	ON
Use) Derined Nbdal Masses	0
Autb Derined Nbdal Masses	0
Spect)al Lbads	0
MemEe) (valuatibn Pbints	9
(xt)apblate Plate Results F)bm Gauss Pbints	Y(S
Gene)al Cbnst)aint	RRRRRR
Tbtal Deg)ees br F)eedbm	8

NODF COORDINEAFS (m)

ID	T Coordinate	X Coordinate	Y Coordinate
1	0.000	0.000	0.000
2	3.900	0.680	0.000
3	9.300	1.630	0.000

Z FZ BFRS (deMgmgmmgmgkN)

I K I i=edxRK Re+eaAedxSK SpriTg

ID	Node E	Node B	, enMth	Aype	Section ID	Rotation EnMe	Node E Lixity	Node B Lixity
1	1	2	3.959	Nb)mal	4	0.000	FFFFFF	FFFFFF
2	2	3	5.483	Nb)mal	4	0.000	FFFFFF	FFFFFF

ID	Node E Offsets	Node B Offsets	Offset Exis	Node E Rot Stiff X	Node E Rot Stiff Y	Node B Rot Stiff X	Node B Rot Stiff Y	Cable , enMth	A/C , imit	Z irror	Disable Non-, inear
1	0, 0, 0	0, 0, 0	Lbcal	-	-	-	-	-	-	Nb	Nb
2	0, 0, 0	0, 0, 0	Lbcal	-	-	-	-	-	-	Nb	Nb

SUPPORAS (kN/mgkN-m/rad)

I K I i=edxRK Re+eaAedxSK SpriTg

BK Bot. X=eA ReAtraiTtxEK Eegative,X=iA ReAtraiTt I T+yxPK PoAitive,X=iA ReAtraiTt I T+y

ID	Node ID	Restraint Code	Direction Code	T Arans Stiffness	X Arans Stiffness	Y Arans Stiffness	T Rot Stiffness	X Rot Stiffness	Y Rot Stiffness	Source
1	3	RFFRRR	BBBBBB	-	-	-	-	-	-	Use) Derined Nbdal Suppb)t
2	2	RFFRRR	BBBBBB	-	-	-	-	-	-	Use) Derined Nbdal Suppb)t
3	1	FFFFFF	BBBBBB	-	-	-	-	-	-	Use) Derined Nbdal Suppb)t

Z EAFRIE, S (Z PagkM/m³g10⁻⁶/deMC)

S. ear Modn+nA iA nAed for memJ erA oT+yc
F=xFyx4 =yx4 =- x4 y- are nAed for ort. otropiN p+ateA oT+yc

ID	Name	XounMs Z odulus	Shear Z odulus	Density	Poisson's Ratio	Thermal Fxp. Coeff.	Young's Modulus x	XounMs Z odulus y	Shear Z odulus xy	Shear Z odulus xz	Shear Z odulus yz	Poisson's Ratio xy
8	Glulam C24 Sp)uce	11000.000	Autb	385.000	0.400	54.000	-	-	-	-	-	Autb

SFCAIONS (mmgmm²gmm⁴gdeM)

ID	Name	Shape	Z aterial ID	Depth	Width	Shear Erea z (SARFSS)	Shear Erea y (SARFSS)	Shear Erea z (AIZ O)	Shear Erea y (AIZ O)	Aorsion Radius
4	Glulam Eeam - 440 x 300	Rectangula)	8	440.000	300.000	110000.606	110000.141	-	-	251.717

ID	Centroid y	Centroid z	Erea (E)	y-Exis Z ol (Iy)	z-Exis Z ol (Iz)	Aorsion Constant (J)	Principal EnMe	Non Prismatic
4	220.000	150.000	132000.000	990000000.000	2129600000.000	2291833964.074	0.000	N

ID	Erea Red. Lactor	Iy Red. Lactor	Iz Red. Lactor	Aorsion Constant Red. Lactor	Shear Erea X Red. Lactor	Shear Erea Y Red. Lactor
4	-	-	-	-	-	-

Z FZ BFR DISARIBUAFD , OEDS (kN/m)

ID	, oad Group	Start Position (%)	Fnd Position (%)	Z ember	Exes	T Start/ Fnd	X Start/ Fnd	Y Start/ Fnd
1	LG	0%	100%	1	GlbEal P)bj.	0.000	-3.900	0.000
						0.000	-3.900	0.000
2	LG	0%	100%	2	GlbEal P)bj.	0.000	-3.900	0.000
						0.000	-3.900	0.000
3	LG	0%	100%	1	GlbEal	0.000	-1.760	0.000
						0.000	-1.760	0.000
4	LG	0%	100%	2	GlbEal	0.000	-1.760	0.000
						0.000	-1.760	0.000
5	LG	0%	100%	1	GlbEal	0.000	-1.990	0.000
						0.000	-1.990	0.000
6	LG	0%	100%	2	GlbEal	0.000	-1.990	0.000
						0.000	-1.990	0.000
7	LG	0%	100%	1	GlbEal	0.000	0.000	17.500
						0.000	0.000	17.500
8	LG	0%	100%	2	GlbEal	0.000	0.000	17.500
						0.000	0.000	17.500

SF, L WFIGHA (Ms)

, oad Group	T Gravity	X Gravity	Y Gravity
SW1	0.000	-1.000	0.000

Bi+of Materia+A

BI, , OL Z EAFRIE, S LOR Z FZ BFRS (mgkM)

Section	Z aterial	Quantity	Unit , enMth	Aotal , enMth	Unit Z ass	Aotal Z ass
4: Glulam Eeam - 440 x 300	8: Glulam C24 Sp)uce	1	3.959	3.959	201.188	201.188
4: Glulam Eeam - 440 x 300	8: Glulam C24 Sp)uce	1	5.483	5.483	278.643	278.643
						479.831

Eoda+ReAn+A

Default , oad Combo

NODF RFECAIONS (kNgkN-m)

Support ID	Node	T Lorce	X Lorce	Y Lorce	T Z oment	X Z oment	Y Z oment
3	1	0.000	11.143	-24.094	-1.611	8.793	4.078
2	2	0.000	47.336	-102.406	0.000	0.000	0.000
1	3	0.000	17.903	-38.731	0.000	0.000	0.000
	Reactibn Sum	0.000	76.382	-165.231			
	Lbad Sum	-0.000	-76.382	165.231			
	(quiliE)ium	0.000	0.000	0.000			

Z FZ BFR FND LORCFS END Z OZ FNAS (kNgkN-m)

Z ember	Node	Exial Lorce	X Shear	Y Shear	T Aorsion	X Z oment	Y Z oment
1	1	1.914	10.977	-24.094	-0.077	8.939	-4.078
	2	-3.587	-20.575	45.186	-0.077	50.689	-23.076
2	2	4.583	26.050	-57.221	0.000	50.689	-23.076
	3	-3.102	-17.633	38.731	0.000	0.000	0.000

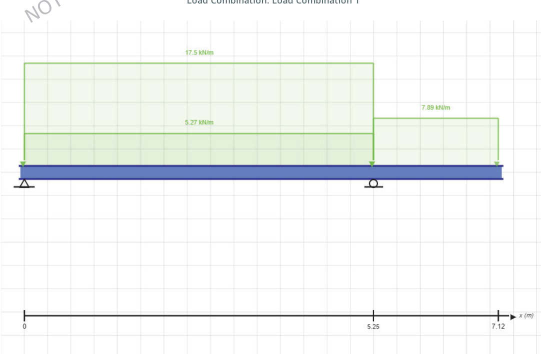
NODE, DISP, ECFZ FNAS (mm)

Node	T Aranslation	X Aranslation	Y Aranslation	Aotal Aranslation
1	0.000	0.000	0.000	0.000
2	0.002	0.000	0.000	0.002
3	-0.001	0.000	0.000	0.001

SkyCiv STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:29:29 GMT+0200 (Central European Summer Time)

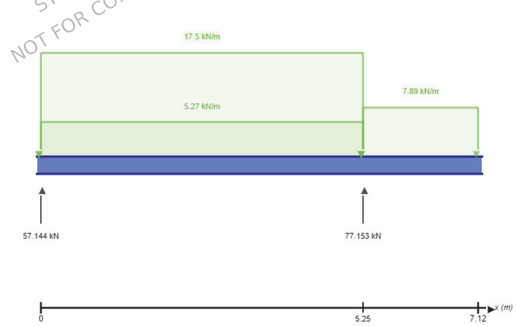
Project Info

File Name: Beam_A (Fire)
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

Free Body Diagram (FBD)
Analysis Summary
Analysis Results
Load Combinations Table
Bending Moment Diagram (BMD)
Shear Force Diagram (SFD)
Deflection Results
Stress Results
Beam Section

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.877	L/285
Custom Stress Limit	PASS	250 MPa	0.025	6.209 MPa

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	57.144 kN	0 kN-m
5.25	0 kN	77.153 kN	0 kN-m

Force Extremes

Result	Max	Min
Bending Moment	71.531 kN-m	-13.795 kN-m
Shear	57.144 kN	-62.399 kN
Displacement	6.56 mm	-6.581 mm

Stress Extremes

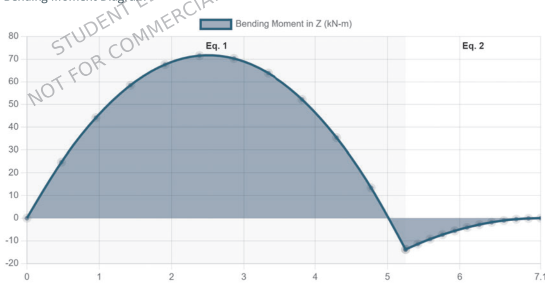
Result	Max	Min
Bending Stress	6.209 MPa	-6.209 MPa
Shear Stress Total	0.651 MPa	0 MPa
Max Combined Normal Stress	6.209 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-6.209 MPa

LOAD COMBINATIONS

Name	Dead Load	Live Load	Wind Load	Roof Load	Rain Load	Snow Load	Earthquake Load	Criteria
Load Combination 1	1	1	1	1	1	1	1	Strength

DIAGRAMS

Bending Moment Diagram

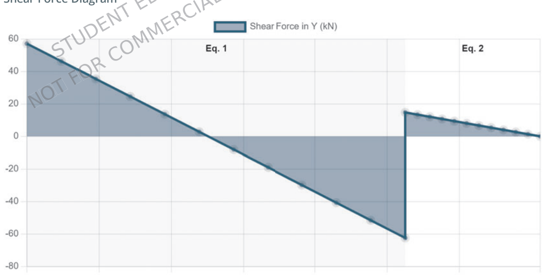


Bending Moment Equations

Eq. 1 $M_1(x) = -11.385x^2 + 57.144x$ for $0 \le x \le 5.25$

Eq. 2 $M_2(x) = -3.945x^2 + 56.177x - 199.988$ for $5.25 \le x \le 7.12$

Shear Force Diagram

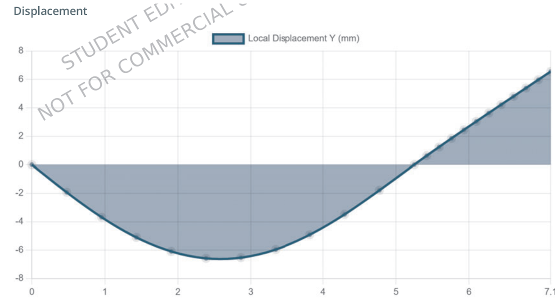


Shear Force Equations

Eq. 1 $V_1(x) = -22.77x + 57.144$ for $0 \le x \le 5.25$

Eq. 2 $V_2(x) = -7.89x + 56.177$ for $5.25 \le x \le 7.12$

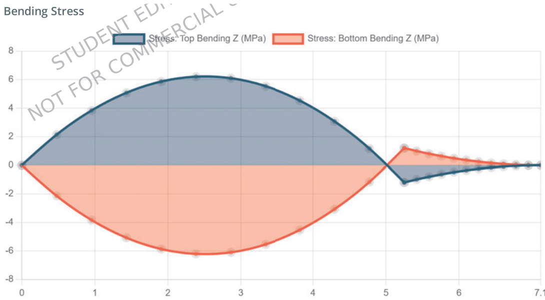
Displacement



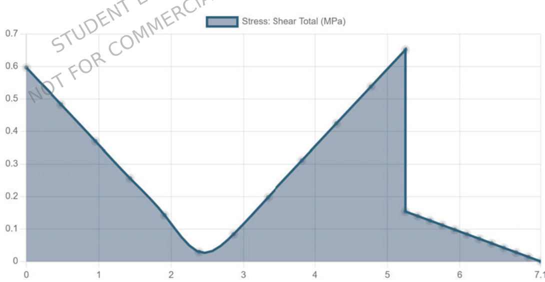
Location (m)	Total Deflection (mm)	Span
0	0 mm	-
2.386	6.581 mm	L/797
5.25	0 mm	-
7.12	6.56 mm	L/285

The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the **model settings**.

Bending Stress

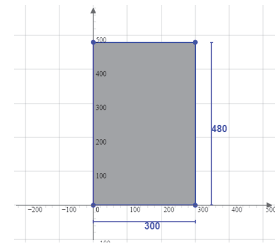


Shear Stress



SkyCiv

Beam Section



Geometric Properties		
A	144000	mm^2
C_z	150	mm
C_y	240	mm

Bending Properties		
I_z	2764800000	mm^4
I_y	1080000000	mm^4

Shear Properties		
A_z	120000.981	mm^2
A_y	120000.163	mm^2

Torsion Properties		
J	2640476653.015	mm^4
r	260.684	mm

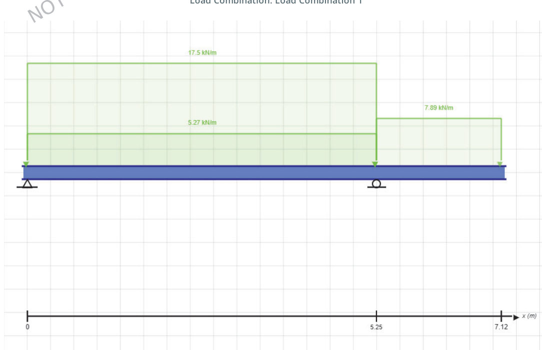
Shape	Material	E (MPa)	v	ρ (kg/m³)
Rectangular	Glulam C24 Spruce	11000	0.4	385

SkyCiv

SkyCiv
STUDENT VERSION

SKYCIV BEAM ANALYSIS REPORT

Load Combination: Load Combination 1



Software: SkyCiv Beam v3.2.2
Wed May 03 2023 14:31:49 GMT+0200 (Central European Summer Time)

Project Info

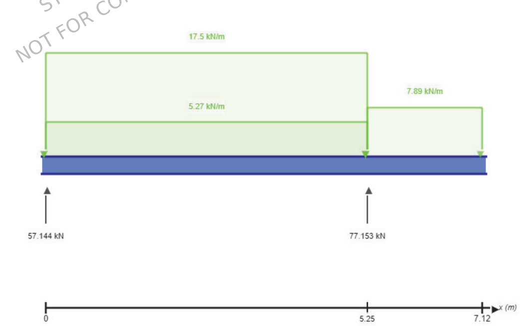
File Name: Beam_A
Engineer: Juan Gomez (juangoserrano@gmail.com)

Included in this Report:

Free Body Diagram (FBD)
Analysis Summary
Analysis Results
Bending Moment Diagram (BMD)
Shear Force Diagram (SFD)
Deflection Results
Stress Results
Beam Section

SkyCiv

FREE BODY DIAGRAM



RESULT SUMMARY

Check	Status	Limit	Ratio	Max
Deflection	PASS	L/250	0.466	L/537
Custom Stress Limit	PASS	250 MPa	0.015	3.786 MPa

SkyCiv

ANALYSIS RESULTS

Reactions

Support at	X	Y	Mx
0	0 kN	57.144 kN	0 kN-m
5.25	0 kN	77.153 kN	0 kN-m

Force Extremes

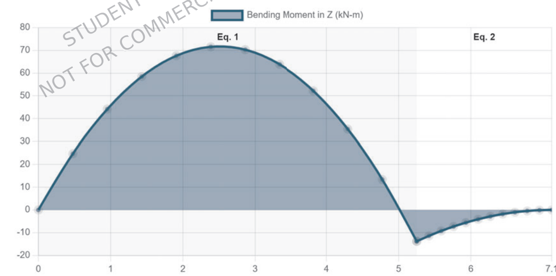
Result	Max	Min
Bending Moment	71.531 kN-m	-13.795 kN-m
Shear	57.144 kN	-62.399 kN
Displacement	3.479 mm	-3.49 mm

Stress Extremes

Result	Max	Min
Bending Stress	3.786 MPa	-3.786 MPa
Shear Stress Total	0.457 MPa	0 MPa
Max Combined Normal Stress	3.786 MPa	0 MPa
Min Combined Normal Stress	0 MPa	-3.786 MPa

DIAGRAMS

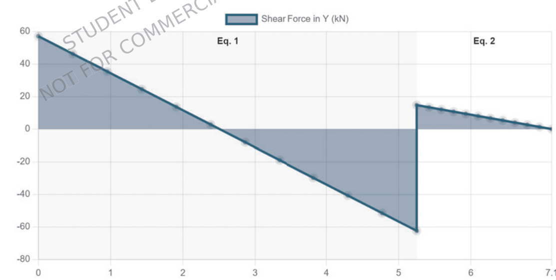
Bending Moment Diagram



Bending Moment Equations

Eq. 1 $M_1(x) = -11.385x^2 + 57.144x$ for $0 \leq x \leq 5.25$
Eq. 2 $M_2(x) = -3.945x^2 + 56.177x - 199.988$ for $5.25 \leq x \leq 7.12$

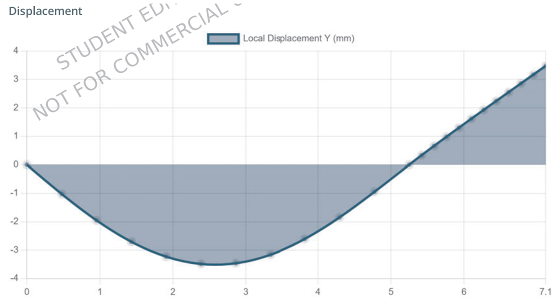
Shear Force Diagram



Shear Force Equations

Eq. 1 $V_1(x) = -22.77x + 57.144$ for $0 \leq x \leq 5.25$
Eq. 2 $V_2(x) = -7.89x + 56.177$ for $5.25 \leq x \leq 7.12$

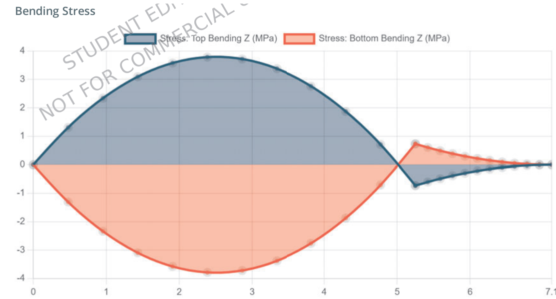
Displacement



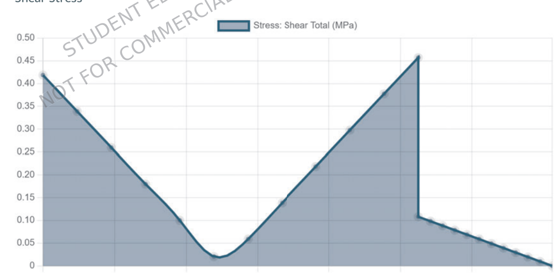
Location (m)	Total Deflection (mm)	Span
0	0 mm	-
2.386	3.49 mm	L/1504
5.25	0 mm	-
7.12	3.479 mm	L/537

The Deflection/Span results are calculated using the analysis results and the Deflection Limit of L/250 set in the model settings.

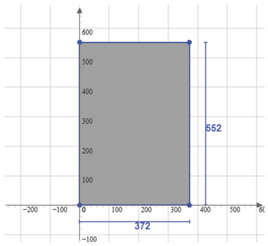
Bending Stress



Shear Stress



Beam Section



Geometric Properties		
A	205344	mm^2
C_z	186	mm
C_y	276	mm

Bending Properties		
I_z	5214094848	mm^4
I_y	2368027008	mm^4

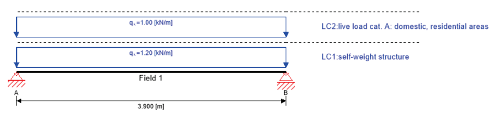
Shear Properties		
A_z	171121.091	mm^2
A_y	171120.248	mm^2

Torsion Properties		
J	5524111810.849	mm^4
r	313.674	mm

Shape	Material	E (MPa)	v	ρ (kg/m^3)
Rectangular	Glulam C24 Spruce	11000	0.4	385

Appendix F – Predimension of CLT slabs sing the Stora Enso tool.

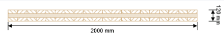
System



Global utilization ratio 95 %

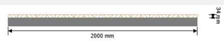
ULS	8 %	ULS Fire	24 %	SLS	23 %	SLS Vibration	95 %	Support	-1 %

Section: CLT 120 L3s



Layer	Thickness	Orientation	Material
1	40.0 mm	0°	C24 spruce ETA (2019)
2	40.0 mm	90°	C24 spruce ETA (2019)
3	40.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**120.0 mm**		

Section Fire: CLT 120 L3s



Layer	Thickness	Orientation	Material
1	34.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**34.0 mm**		

Fire resistance class:R 120
Fire protection layering : 2 x 12.5 mm gypsum plasterboard Type F
gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)

Time **120 min**

$t_{ch,h}$ [min]	$t_{f,h}$ [min]	$t_{a,h}$ [min]	$d_{ta,h}$ [mm]	k_0 [-]	d_0 [mm]	$d_{char,0,h}$ [mm]	$d_{ef,h}$ [mm]
38	38	57	25	1	7	79.0	86.0

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\,min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	live load cat. A: domestic, residential areas	Q	medium term	0.8	0	1.5	0.7	0.5	0.3

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	1.20

LC2:live load cat. A: domestic, residential areas

continuous load

Field	Load at start [kN/m]
1	1.00

ULS Combinations

	Combination rule
LCO1	1.35/1.00 * LC1
LCO2	1.35/1.00 * LC1 + 1.50/0.00 * LC2

ULS Combinations Fire

	Combination rule
LCO3	1.00/1.00 * LC1
LCO4	1.00/1.00 * LC1 + 1.00/0.00 * 0.30 * LC2

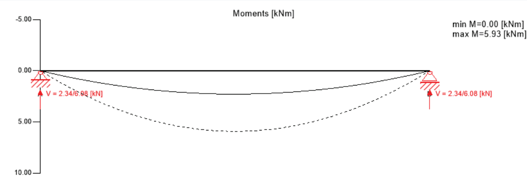
SLS Characteristic Combination

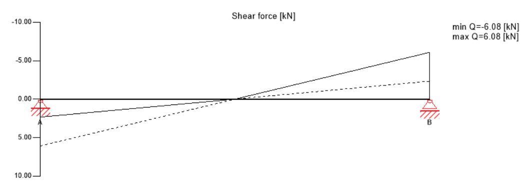
	Combination rule
LCO5	1.00/1.00 * LC1
LCO6	1.00/1.00 * LC1 + 1.00/0.00 * LC2

SLS Quasi-permanent Combination

	Combination rule
LCO7	1.00/1.00 * LC1
LCO8	1.00/1.00 * LC1 + 1.00/0.00 * 0.30 * LC2

Ultimate limit state (ULS) - design results





ULS Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,y,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$\sigma_{m,y,d}$ [N/mm²]	Ratio	
1	1.95	24.00	1.25	0.80	1.10	16.90	5.93	-1.28	8 %	LCO2

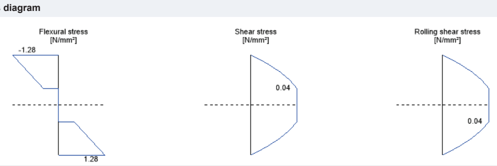
ULS Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	4.00	1.25	0.80	2.56	6.08	0.04	1 %	LCO2

ULS Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.05	1.25	0.80	0.67	6.08	0.04	5 %	LCO2

Stress diagram



Flexural stress analysis

$M_{y,d}$ = 5.93 kNm
$M_{z,d}$ = 0.00 kNm
$N_{t,d}$ = 0.00 kN
$f_{m,k}$ = 24.00 N/mm²
$f_{m,k,z}$ = 24.00 N/mm²
γ_m = 1.25 -
k_{mod} = 0.80 -
$k_{sys,y}$ = 1.10 -
$k_{h,m,y}$ = 1.00 -
$k_{h,m,z}$ = 1.00 -
k_l = 1.00 -
$\sigma_{t,d}$ = 0.00 N/mm² $f_{t,0,d}$ = 8.96 N/mm²
$\sigma_{m,y,d}$ = -1.28 N/mm² $f_{m,y,d}$ = 16.90 N/mm²
$\sigma_{m,z,d}$ = 0.00 N/mm² < $f_{m,z,d}$ = 0.00 N/mm² ✓

Utilization ratio 8 %

Shear stress analysis

V_d = 6.08 kN
$f_{v,k}$ = 4.00 N/mm²
γ_m = 1.25 -
k_{mod} = 0.80 -
$k_{h,v}$ = 0.00 -
$\tau_{v,d}$ = 0.04 N/mm² < $f_{v,d}$ = 2.56 N/mm² ✓

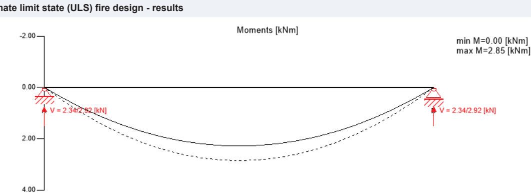
Utilization ratio 1 %

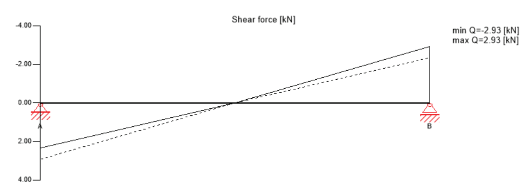
Rolling shear analysis

V_d = 6.08 kN
$f_{r,k}$ = 1.05 N/mm²
γ_m = 1.25 -
k_{mod} = 0.80 -
$\tau_{r,d}$ = 0.04 N/mm² < $f_{r,d}$ = 0.67 N/mm² ✓

Utilization ratio 5 %

Ultimate limit state (ULS) fire design - results





ULS Fire Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	k_{fi} [-]	$f_{m,y,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$\sigma_{m,y,d}$ [N/mm²]	Ratio	
1	1.95	24.00	1.00	1.00	1.10	1.15	30.36	2.85	7.40	24 %	LCO4

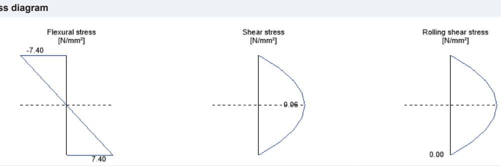
ULS Fire Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	3.9	4.00	1.00	1.00	1.15	4.60	-2.93	0.06	1 %	LCO4

ULS Fire Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	3.9	1.25	1.00	1.00	1.15	1.44	-2.93	0.00	0 %	LCO4

Stress diagram



Flexural stress analysis Fire

$M_{y,d}$ =	2.85	kNm		$f_{m,k}$ =	24.00	N/mm²
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-
				k_{mod} =	1.00	-
				$k_{sys,y}$ =	1.10	-
				$k_{h,m,y}$ =	1.00	-
				$k_{h,m,z}$ =	1.00	-
				k_l =	1.00	-
				k_{fi} =	1.15	-
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	16.10	N/mm²
$\sigma_{m,y,d}$ =	7.40	N/mm²		$f_{m,y,d}$ =	30.36	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm² ✓

Utilization ratio 24 %

Shear stress analysis Fire

V_d =	-2.93	kN		$f_{v,k}$ =	4.00	N/mm²
				γ_m =	1.00	-
				k_{mod} =	1.00	-
				$k_{h,v}$ =	0.00	-
				k_{fi} =	1.15	-
$\tau_{v,d}$ =	0.06	N/mm²	<	$f_{v,d}$ =	4.60	N/mm² ✓

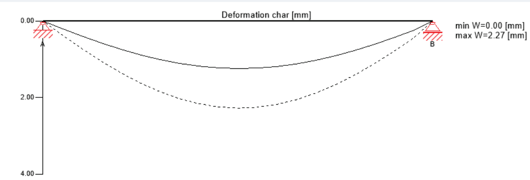
Utilization ratio 1 %

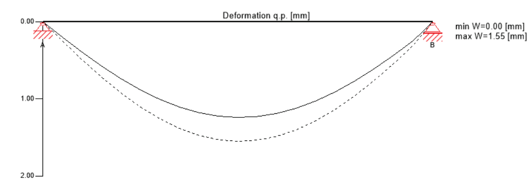
Rolling shear analysis Fire

V_d =	-2.93	kN		$f_{r,k}$ =	1.25	N/mm²
				γ_m =	1.00	-
				k_{mod} =	1.00	-
				k_{fi} =	1.15	-
$\tau_{r,d}$ =	0.00	N/mm²	<	$f_{r,d}$ =	1.44	N/mm² ✓

Utilization ratio 0 %

Service limit state design (SLS) - design results





w_{inst} = w[char]

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/300	13.0	2.3	17 %

w_{fin} = w[char] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/250	15.6	3.5	23 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/300	13.0	2.8	21 %

Vibration analysis

General		
Total mass	1.19	[t]
Tributary width	1.6	[m]
Stiffness Longitudinal direction	3328.0	[kNm²]
Stiffness Cross direction	128.0	[kNm²]
Modal damping	1.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	187.1	[kg]

Analysis

Criterion	Calc.	Class I	Class II	Class I	Class II	Cl. I	Cl. II
Frequency criterion min	17.039 [Hz]	4.5 [Hz]	4.5 [Hz]	26 %	26 %	✓	✓
Frequency criterion	17.039 [Hz]	8.0 [Hz]	6.0 [Hz]	47 %	35 %		
Acceleration criterion	0.082 [m/s²]	0.05 [m/s²]	0.1 [m/s²]	164 %	82 %		
Stiffness criterion	0.237 [mm]	0.25 [mm]	0.5 [mm]	95 %	47 %	✓	✓

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V
self-weight structure	0.6	2.34	2.34
		2.34	2.34
live load cat. A: domestic, residential areas	0.8	1.95	1.95
		0.00	0.00

Reference documents for this analysis

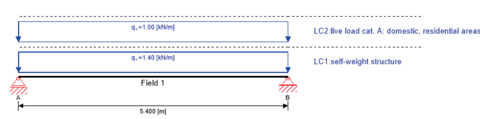
English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
Expertise Rolling shear - no edge gluing, H.J. Blass	Expertise on Rolling shear for CLT
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
Technical expertise 122/2011/02: analysis of load bearing capacity and separation performance of CLT elements	Verification of the load bearing capacity and the insulation criterion of CLT structures with Stora Enso CLT
Technical expertise 2434/2012 - BB: failure time tf of gypsum fire boards (GKF) according to ON B 3410	Expertise on failure time tf of gypsum wall fire boards according to ON B3410 and gypsum wall boards type DF according to EN 520
EN 1990	EN 1990 - Eurocode ? Basis of structural design
BS EN 1995-1-1 NA	BS EN 1995-1-1 - UK - National Annex to Eurocode 5: Design of timber structures - Part 1-1: General- Common rules and rules for buildings
BS EN 1995-1-2 NA	BS EN 1995-1-2 - UK National Annex to Eurocode 5: Design of timber structures - Part 1-2: General - Structural fire design
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guideline for Europe; publishes by SP Technical Research Institute of Sweden

Reference documents for this analysis

English title	Description
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
BS EN 1995-1-2_NA	BS EN 1995-1-2 - United Kingdom - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning BS EN 1995-1-2, national comments and national supplements
Expertise Rolling shear, H.J. Blass	Expertise on rolling shear strength and rolling shear modulus of CLT panels
ÖNORM EN 1995-1-1_NA, chapter 7.3	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings; chapter 7.3

Disclaimer

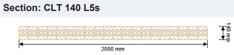
System



Global utilization ratio 98 %

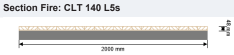
ULS	12 %	ULS Fire	38 %	SLS	41 %	SLS Vibration	98 %	Support	-1 %

Section: CLT 140 L5s



Layer	Thickness	Orientation	Material
1	40.0 mm	0°	C24 spruce ETA (2019)
2	20.0 mm	90°	C24 spruce ETA (2019)
3	20.0 mm	0°	C24 spruce ETA (2019)
4	20.0 mm	90°	C24 spruce ETA (2019)
5	40.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**140.0 mm**		

Section Fire: CLT 140 L5s



Layer	Thickness	Orientation	Material
1	40.0 mm	0°	C24 spruce ETA (2019)
2	8.0 mm	90°	C24 spruce ETA (2019)
t_{CLT}	**48.0 mm**		

Fire resistance class:R 120
Fire protection layering : 2 x 15.0 mm gypsum plasterboard Type F acc. HFA Report
gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)

Time **120 min**

$t_{ch,h}$ [min]	$t_{f,h}$ [min]	$t_{a,h}$ [min]	$d_{ta,h}$ [mm]	k_0 [-]	d_0 [mm]	$d_{char,0,h}$ [mm]	$d_{ef,h}$ [mm]
44	44	63	25	1	7	85.0	92.0

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	live load cat. A: domestic, residential areas	Q	medium term	0.8	0	1.5	0.7	0.5	0.3

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	1.40

LC2:live load cat. A: domestic, residential areas

continuous load

Field	Load at start [kN/m]
1	1.00

ULS Combinations

	Combination rule
LCO1	1.35/1.00 * LC1
LCO2	1.35/1.00 * LC1 + 1.50/0.00 * LC2

ULS Combinations Fire

	Combination rule
LCO3	1.00/1.00 * LC1
LCO4	1.00/1.00 * LC1 + 1.00/0.00 * 0.30 * LC2

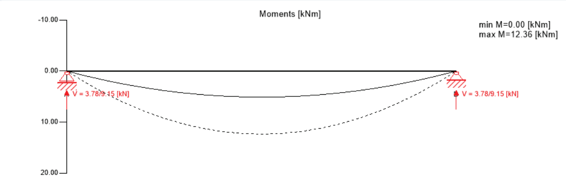
SLS Characteristic Combination

	Combination rule
LCO5	1.00/1.00 * LC1
LCO6	1.00/1.00 * LC1 + 1.00/0.00 * LC2

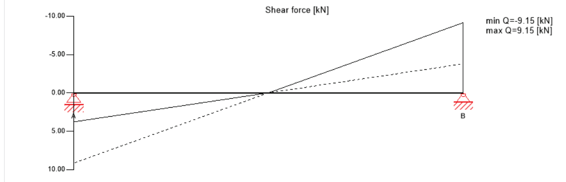
SLS Quasi-permanent Combination

	Combination rule
LCO7	1.00/1.00 * LC1
LCO8	1.00/1.00 * LC1 + 1.00/0.00 * 0.30 * LC2

Ultimate limit state (ULS) - design results



Ultimate limit state (ULS) - design results



ULS Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,y,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$\sigma_{m,y,d}$ [N/mm²]	Ratio	
1	2.7	24.00	1.25	0.80	1.10	16.90	12.36	-2.05	12 %	LCO2

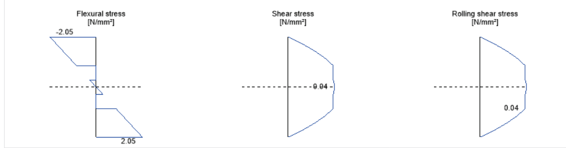
ULS Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	5.4	4.00	1.25	0.80	2.56	-9.15	0.04	2 %	LCO2

ULS Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	5.4	1.25	1.25	0.80	0.80	-9.15	0.04	5 %	LCO2

Stress diagram



Flexural stress analysis

$M_{y,d}$ =	12.36	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	8.96	N/mm²	
$\sigma_{m,y,d}$ =	-2.05	N/mm²		$f_{m,y,d}$ =	16.90	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓

Utilization ratio 12 %

Shear stress analysis

V_d =	-9.15	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.04	N/mm²	<	$f_{v,d}$ =	2.56	N/mm²	✓

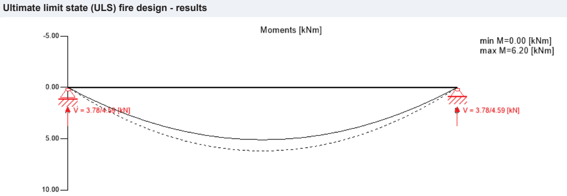
Utilization ratio 2 %

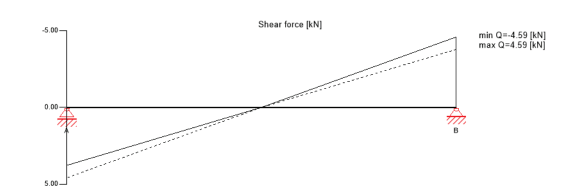
Rolling shear analysis

V_d =	-9.15	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.04	N/mm²	<	$f_{r,d}$ =	0.80	N/mm²	✓

Utilization ratio 5 %

Ultimate limit state (ULS) fire design - results





ULS Fire Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	k_{fi} [-]	$f_{m,y,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$\sigma_{m,y,d}$ [N/mm²]	Ratio	
1	2.7	24.00	1.00	1.00	1.10	1.15	30.36	6.20	-11.62	38 %	LCO4

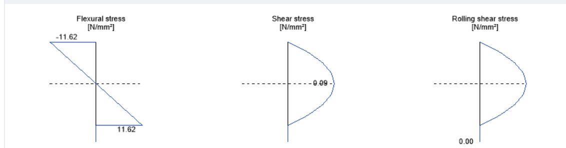
ULS Fire Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	5.4	4.00	1.00	1.00	1.15	4.60	-4.59	0.09	2 %	LCO4

ULS Fire Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	5.4	1.25	1.00	1.00	1.15	1.44	-4.59	0.00	0 %	LCO4

Stress diagram



Flexural stress analysis Fire

$M_{y,d}$ =	6.20	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
				k_{fi} =	1.15	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	16.10	N/mm²	
$\sigma_{m,y,d}$ =	-11.62	N/mm²		$f_{m,y,d}$ =	30.36	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓

Utilization ratio 38 %

Shear stress analysis Fire

V_d =	-4.59	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{h,v}$ =	0.00	-	
				k_{fi} =	1.15	-	
$\tau_{v,d}$ =	0.09	N/mm²	<	$f_{v,d}$ =	4.60	N/mm²	✓

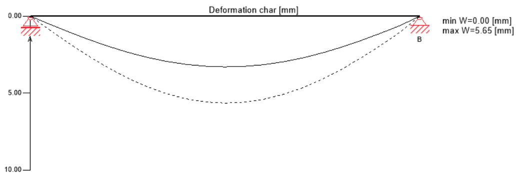
Utilization ratio 2 %

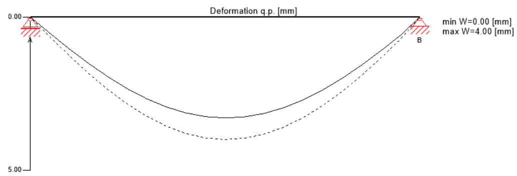
Rolling shear analysis Fire

V_d =	-4.59	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				k_{fi} =	1.15	-	
$\tau_{r,d}$ =	0.00	N/mm²	<	$f_{r,d}$ =	1.44	N/mm²	✓

Utilization ratio 0 %

Service limit state design (SLS) - design results





w_{inst} = w[char]

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/300	18.0	5.6	31 %

w_{fin} = w[char] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/250	21.6	8.9	41 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/300	18.0	7.2	40 %

Vibration analysis

General		
Total mass	1.93	[t]
Tributary width	2.6	[m]
Stiffness Longitudinal direction	5072.0	[kNm²]
Stiffness Cross direction	416.0	[kNm²]
Modal damping	1.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	505.6	[kg]

Vibration analysis

Analysis							
Criterion	Calc.	Class I	Class II	Class I	Class II	Cl. I	Cl. II
Frequency criterion min	10.158 [Hz]	4.5 [Hz]	4.5 [Hz]	44 %	44 %	✓	✓
Frequency criterion	10.158 [Hz]	8.0 [Hz]	6.0 [Hz]	79 %	59 %		
Acceleration criterion	0.476 [m/s²]	0.05 [m/s²]	0.1 [m/s²]	952 %	476 %		
Stiffness criterion	0.246 [mm]	0.25 [mm]	0.5 [mm]	98 %	49 %	✓	✓

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V [kN]
self-weight structure	0.6	3.78	3.78
		3.78	3.78
live load cat. A: domestic, residential areas	0.8	2.70	2.70
		0.00	0.00

Note

CLT slab 5.4 spam simply supported slab. Interior

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
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BS EN 1995-1-1 NA	BS EN 1995-1-1 - UK - National Annex to Eurocode 5: Design of timber structures - Part 1-1: General- Common rules and rules for buildings
BS EN 1995-1-2 NA	BS EN 1995-1-2 - UK National Annex to Eurocode 5: Design of timber structures - Part 1-2: General - Structural fire design
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guideline for Europe; publishes by SP Technical Research Institute of Sweden
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
BS EN 1995-1-2_NA	BS EN 1995-1-2 - United Kingdom - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning BS EN 1995-1-2, national comments and national supplements
Expertise Rolling shear, H.J. Blass	Expertise on rolling shear strength and rolling shear modulus of CLT panels
ÖNORM EN 1995-1-1_NA, chapter 7.3	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General-Common rules and rules for buildings; chapter 7.3

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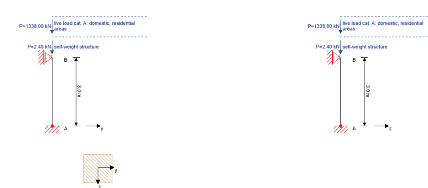
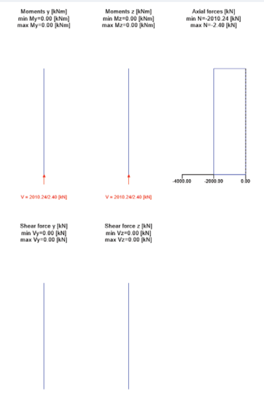
System



Section: Wooden beam 40/40; **Material**: GL 24h; **Service class**: service class 1; **Fire resistance class**: R 120

Appendix G – Predimension of glulam Column using the Stora Enso tool.

Utilization 82 %



Flexural stress analysis 82 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	24.00	N/mm²
$M_{z,d}$ =	0.00	kNm	$f_{m,k,z}$ =	24.00	N/mm²
$N_{c,d}$ =	-2010.24	kN	$f_{c,0,k}$ =	24.00	N/mm²
$\sigma_{c,d}$ =	12.56	N/mm²	$f_{c,0,d}$ =	15.36	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	16.00	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	16.00	N/mm² ✓

Shear stress analysis Y 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.50	N/mm² ✓

Shear stress analysis Z 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	1.50	N/mm² ✓

Shear stress analysis Combined 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			Ratio =	0 %	✓

Lateral torsional buckling analysis 0 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	0.00	N/mm²
$M_{z,d}$ =	0.00	kNm			
$N_{c,d}$ =	0.00	kN	$f_{c,0,k}$ =	0.00	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,0,d}$ =	0.00	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	0.00	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	0.00	N/mm² ✓

Buckling analysis 82 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	24.00	N/mm²
$M_{z,d}$ =	0.00	kNm			
$N_{c,d}$ =	-2010.24	kN	$f_{c,0,k}$ =	24.00	N/mm²
$\sigma_{c,d}$ =	12.56	N/mm²	$f_{c,0,d}$ =	15.36	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	16.00	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	16.00	N/mm² ✓

Flexural stress analysis Fire 13 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	24.00	N/mm²
$M_{z,d}$ =	0.00	kNm	$f_{m,k,z}$ =	24.00	N/mm²
$N_{c,d}$ =	-403.80	kN	$f_{c,0,k}$ =	24.00	N/mm²
$\sigma_{c,d}$ =	3.65	N/mm²	$f_{c,0,d}$ =	27.60	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	29.28	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	29.28	N/mm² ✓

Shear stress analysis Y Fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

Shear stress analysis Z Fire 0 %

V_d =	0.00	kN	$f_{v,k}$ =	3.50	N/mm²
$\tau_{v,d}$ =	0.00	N/mm² <	$f_{v,d}$ =	2.70	N/mm² ✓

Shear stress analysis Combined Fire 0 %

$V_{y,d}$ =	0.00	kN	$V_{z,d}$ =	0.00	kN
$\tau_{v,y,d}$ =	0.00	N/mm²	$\tau_{v,z,d}$ =	0.00	N/mm²
			Ratio =	0 %	✓

Lateral torsional buckling analysis Fire 0 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	0.00	N/mm²
$M_{z,d}$ =	0.00	kNm			
$N_{c,d}$ =	0.00	kN	$f_{c,0,k}$ =	0.00	N/mm²
$\sigma_{c,d}$ =	0.00	N/mm²	$f_{c,0,d}$ =	0.00	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	0.00	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm² <	$f_{m,z,d}$ =	0.00	N/mm² ✓

Buckling analysis Fire 13 %

$M_{y,d}$ =	0.00	kNm	$f_{m,k}$ =	24.00	N/mm²
$M_{z,d}$ =	0.00	kNm			
$N_{c,d}$ =	-403.80	kN	$f_{c,0,k}$ =	24.00	N/mm²
$\sigma_{c,d}$ =	3.65	N/mm²	$f_{c,0,d}$ =	27.60	N/mm²
$\sigma_{m,y,d}$ =	0.00	N/mm²	$f_{m,y,d}$ =	29.28	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm²	$f_{m,z,d}$ =	29.28	N/mm² ✓

Support reaction

Load case category	k_{mod}	A_y [kN]	A_z [kN]	B_x [kN]	B_y [kN]	B_z [kN]	B_{my} [kNm]	B_{mz} [kNm]
self-weight structure	0.6	0.00	0.00	2.40	0.00	0.00	0.00	0.00
		0.00	0.00	2.40	0.00	0.00	0.00	0.00
live load cat. A: domestic, residential areas	0.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	1338.00	0.00	0.00	0.00	0.00

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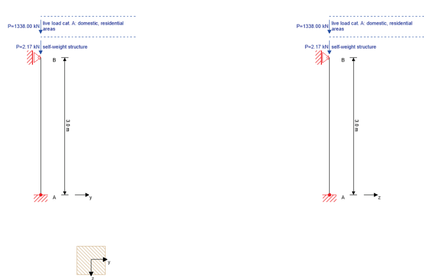
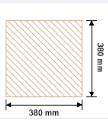
System



Global utilization ratio 91 %

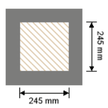
ULS	91 %	ULS Fire	25 %

Section: Wooden beam 38/38



Section width	Section height	Area	Iy	Iz
[cm]	[cm]	[mm²]	[mm⁴]	[mm⁴]
38	38	144,400	1,737,613,000	1,737,613,000

Section Fire: Wooden beam 38/38



Section width	Section height	Area	Iy	Iz
[cm]	[cm]	[mm²]	[mm⁴]	[mm⁴]
24.5	24.5	60,250	302,505,100	302,505,100

Fire resistance class:R 120
Fire protection layering : 2 x 12.5 mm gypsum plasterboard Type F
gypsum plasterboard Type A (acc. to EN 520)gypsum plasterboard Type F (acc. to EN 520)

Time **120 min**

$t_{ch,h}$	$t_{f,h}$	$t_{a,h}$	$d_{ta,h}$	k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$
[min]	[min]	[min]	[mm]	[-]	[mm]	[mm]	[mm]
49	54	70	26	1	7	60.3	134.5

Material values

Material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$E_{0,5}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
GL 24h	24.00	19.20	0.50	24.00	2.50	3.50	1.20	11,500.00	650.00	9,600.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	live load cat. A: domestic, residential areas	Q	medium term	0.8	0	1.5	0.7	0.5	0.3

LC1:self-weight structure

vertical load

P_k	ex. y	ex. z
[kN]	[m]	[m]
2.166	0.00	0.00

LC2:live load cat. A: domestic, residential areas

vertical load

P_k	ex. y	ex. z
[kN]	[m]	[m]
1338	0.00	0.00

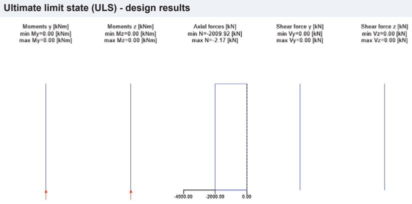
ULS Combinations

	Combination rule
LCO1	1.35/1.00 * LC1
LCO2	1.35/1.00 * LC1 + 1.50/0.00 * LC2

ULS Combinations Fire

	Combination rule
LCO1	1.00/1.00 * LC1
LCO2	1.00/1.00 * LC1 + 1.00/0.00 * 0.30 * LC2

Ultimate limit state (ULS) - design results



ULS Flexural design

Dist. [m]	$f_{m,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,z}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]	$f_{m,z,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
0.0	24.00	24.00	19.20	1.25	0.80	1.00	1.05	1.05	1.00	16.08	16.08	12.29	15.36

$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{m,z,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
0.00	0.00	-2009.92	0.00	0.00	0.00	13.92	0.00	91 %	LCO2

ULS Shear analysis Y

Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{h,v}$ [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
3.0	3.50	1.25	0.80	1.00	1.50	0.00	0.00	0 %	LCO2

ULS Shear analysis Z

Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{h,v}$ [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
3.0	3.50	1.25	0.80	1.00	1.50	0.00	0.00	0 %	LCO2

Flexural stress analysis

$M_{y,d}$ = 0.00 kNm; $M_{z,d}$ = 0.00 kNm; $N_{c,d}$ = -2009.92 kN
$f_{m,k}$ = 24.00 N/mm²; $f_{m,k,z}$ = 24.00 N/mm²; γ_m = 1.25 -; k_{mod} = 0.80 -; $k_{sys,y}$ = 1.00 -; $k_{h,m,y}$ = 1.05 -; $k_{h,m,z}$ = 1.05 -; k_l = 1.00 -
$\sigma_{c,d}$ = 13.92 N/mm² — $f_{c,0,d}$ = 15.36 N/mm²
$\sigma_{m,y,d}$ = 0.00 N/mm² — $f_{m,y,d}$ = 16.08 N/mm²
$\sigma_{m,z,d}$ = 0.00 N/mm² < $f_{m,z,d}$ = 16.08 N/mm² ✓
Utilization ratio 91 %

Shear stress analysis Y

V_d = 0.00 kN
$f_{v,k}$ = 3.50 N/mm²; γ_m = 1.25 -; k_{mod} = 0.80 -; $k_{h,v}$ = 1.00 -
$\tau_{v,d}$ = 0.00 N/mm² < $f_{v,d}$ = 1.50 N/mm² ✓
Utilization ratio 0 %

Shear stress analysis Z

V_d = 0.00 kN
$f_{v,k}$ = 3.50 N/mm²; γ_m = 1.25 -; k_{mod} = 0.80 -; $k_{h,v}$ = 1.00 -
$\tau_{v,d}$ = 0.00 N/mm² < $f_{v,d}$ = 1.50 N/mm² ✓
Utilization ratio 0 %

Shear stress analysis

$V_{y,d}$ = 0.00 kN; $f_{v,k}$ = 0.00 N/mm²; $f_{v,d}$ = 0.00 N/mm²; $\tau_{v,y,d}$ = 0.00 N/mm²
$V_{z,d}$ = 0.00 kN; γ_m = 1.25 -; k_{mod} = 0.00 -; $\tau_{v,z,d}$ = 0.00 N/mm²
Utilization ratio 0 %

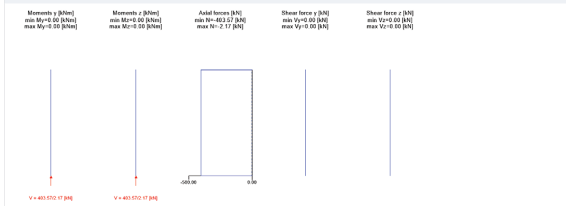
Buckling analysis

$M_{y,d}$ = 0.00 kNm; $M_{z,d}$ = 0.00 kNm; $N_{c,d}$ = -2009.92 kN
$f_{m,k}$ = 24.00 N/mm²; γ_m = 1.25 -; k_{mod} = 0.80 -; $k_{sys,y}$ = 1.00 -; $k_{sys,z}$ = 1.00 -; $k_{h,m,y}$ = 1.05 -; $k_{h,m,z}$ = 1.05 -
$\sigma_{c,d}$ = 13.92 N/mm² — $f_{c,0,d}$ = 15.36 N/mm²
$\sigma_{m,y,d}$ = 0.00 N/mm² — $f_{m,y,d}$ = 16.08 N/mm²
$\sigma_{m,z,d}$ = 0.00 N/mm² < $f_{m,z,d}$ = 16.08 N/mm² ✓
Utilization ratio 91 %

Lateral torsional buckling analysis

$M_{y,d}$ = 0.00 kNm; $M_{z,d}$ = 0.00 kNm; $N_{c,d}$ = 0.00 kN
$f_{m,k}$ = 0.00 N/mm²; γ_m = 1.25 -; k_{mod} = 0.00 -; $k_{sys,y}$ = 1.00 -; $k_{h,m,y}$ = 1.05 -; $k_{h,m,z}$ = 1.05 -; k_l = 1.00 -
$\sigma_{c,d}$ = 0.00 N/mm² — $f_{c,0,d}$ = 0.00 N/mm²
$\sigma_{m,y,d}$ = 0.00 N/mm² — $f_{m,y,d}$ = 0.00 N/mm²
$\sigma_{m,z,d}$ = 0.00 N/mm² < $f_{m,z,d}$ = 0.00 N/mm² ✓
Utilization ratio 0 %

Ultimate limit state (ULS) fire design - results



ULS Fire Flexural design

Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	k_{fi} [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
0.0	1.25	0.80	1.00	1.00	24.00	16.08	12.29	15.36

$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
0.00	-403.57	0.00	0.00	6.70	0.00	24 %	LCO2

ULS Fire Shear analysis Y

Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	k_{fi} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
3.0	3.50	1.00	1.00	1.00	1.15	2.70	0.00	0.00	0 %	LCO2

ULS Fire Shear analysis Z

Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	k_{fi} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
3.0	3.50	1.00	1.00	1.00	1.15	2.70	0.00	0.00	0 %	LCO2

Flexural stress analysis Fire

$M_{y,d}$ = 0.00 kNm; $M_{z,d}$ = 0.00 kNm; $N_{c,d}$ = -403.57 kN
$f_{m,k}$ = 24.00 N/mm²; $f_{m,k,z}$ = 24.00 N/mm²; γ_m = 1.00 -; k_{mod} = 1.00 -; $k_{sys,y}$ = 1.00 -; $k_{h,m,y}$ = 1.09 -; $k_{h,m,z}$ = 1.09 -; k_l = 1.00 -; k_{fi} = 1.15 -
$\sigma_{c,d}$ = 6.70 N/mm² — $f_{c,0,d}$ = 27.60 N/mm²
$\sigma_{m,y,d}$ = 0.00 N/mm² — $f_{m,y,d}$ = 30.18 N/mm²
$\sigma_{m,z,d}$ = 0.00 N/mm² < $f_{m,z,d}$ = 30.18 N/mm² ✓
Utilization ratio 24 %

Shear stress analysis Y Fire

V_d = 0.00 kN
$f_{v,k}$ = 3.50 N/mm²; γ_m = 1.00 -; k_{mod} = 1.00 -; $k_{h,v}$ = 1.00 -; k_{fi} = 1.15 -
$\tau_{v,d}$ = 0.00 N/mm² < $f_{v,d}$ = 2.70 N/mm² ✓
Utilization ratio 0 %

Shear stress analysis Z Fire

V_d = 0.00 kN
$f_{v,k}$ = 3.50 N/mm²; γ_m = 1.00 -; k_{mod} = 1.00 -; $k_{h,v}$ = 1.00 -; k_{fi} = 1.15 -
$\tau_{v,d}$ = 0.00 N/mm² < $f_{v,d}$ = 2.70 N/mm² ✓
Utilization ratio 0 %

Shear stress analysis Fire

$V_{y,d}$ = 0.00 kN; $f_{v,k}$ = 0.00 N/mm²; $f_{v,d}$ = 0.00 N/mm²; $\tau_{v,y,d}$ = 0.00 N/mm²
$V_{z,d}$ = 0.00 kN; γ_m = 1.00 -; k_{fi} = 1.15 -; k_{mod} = 0.00 -; $\tau_{v,z,d}$ = 0.00 N/mm²
Utilization ratio 0 %

Buckling analysis Fire

$M_{y,d}$ = 0.00 kNm; $M_{z,d}$ = 0.00 kNm; $N_{c,d}$ = -403.57 kN
$f_{m,k}$ = 24.00 N/mm²; γ_m = 1.00 -; k_{mod} = 1.00 -; $k_{sys,y}$ = 1.00 -; $k_{sys,z}$ = 1.00 -; $k_{h,m,y}$ = 1.09 -; $k_{h,m,z}$ = 1.09 -; k_{fi} = 1.15 -
$\sigma_{c,d}$ = 6.70 N/mm² — $f_{c,0,d}$ = 27.60 N/mm²
$\sigma_{m,y,d}$ = 0.00 N/mm² — $f_{m,y,d}$ = 30.18 N/mm²
$\sigma_{m,z,d}$ = 0.00 N/mm² < $f_{m,z,d}$ = 30.18 N/mm² ✓
Utilization ratio 25 %

Lateral torsional buckling analysis Fire

$M_{y,d}$ = 0.00 kNm; $M_{z,d}$ = 0.00 kNm; $N_{c,d}$ = 0.00 kN
$f_{m,k}$ = 0.00 N/mm²; γ_m = 1.00 -; k_{mod} = 0.00 -; $k_{sys,y}$ = 1.00 -; $k_{h,m,y}$ = 1.09 -; $k_{h,m,z}$ = 1.09 -; k_l = 1.00 -; k_{fi} = 1.15 -
$\sigma_{c,d}$ = 0.00 N/mm² — $f_{c,0,d}$ = 0.00 N/mm²
$\sigma_{m,y,d}$ = 0.00 N/mm² — $f_{m,y,d}$ = 0.00 N/mm²
$\sigma_{m,z,d}$ = 0.00 N/mm² < $f_{m,z,d}$ = 0.00 N/mm² ✓
Utilization ratio 0 %

Support reaction

Load case category	k_{mod}	A_y [kN]	A_z [kN]	B_x [kN]	B_y [kN]	B_z [kN]	B_{my} [kNm]	B_{mz} [kNm]
self-weight structure	0.6	0.00	0.00	2.17	0.00	0.00	0.00	0.00
		0.00	0.00	2.17	0.00	0.00	0.00	0.00
live load cat. A: domestic, residential areas	0.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		0.00	0.00	1338.00	0.00	0.00	0.00	0.00

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
EN 14080	EN 14080 - Timber Structures - Glued laminated timber and glued solid timber - Requirements
BS EN 1995-1-1 NA	BS EN 1995-1-1 - UK - National Annex to Eurocode 5: Design of timber structures - Part 1-1: General- Common rules and rules for buildings
BS EN 1995-1-2 NA	BS EN 1995-1-2 - UK National Annex to Eurocode 5: Design of timber structures - Part 1-2: General - Structural fire design
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guideline for Europe; publishes by SP Technical Research Institute of Sweden
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
BS EN 1995-1-2_NA	BS EN 1995-1-2 - United Kingdom - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning BS EN 1995-1-2, national comments and national supplements
CERTIFICATE NO. EUFI29-20000564-C	Product certificate
LVL G by Stora Enso_Structural design manual column&beam_ V01	Design manual
ETA 20_0291 LVL G by Stora Enso	ETA

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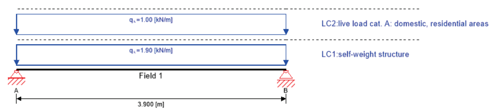
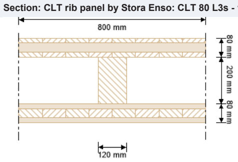
System



Global utilization ratio 37 %

ULS	ULS Fire	SLS	SLS Vibration	Support
12 %	-1 %	5 %	37 %	0 %

Section: CLT rib panel by Stora Enso: CLT 80 L3s - 12/20 - CLT 80 C3s



Ribs width are reduced by 20 mm for the design

Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	20.0	550.0	0°	C24 spruce ETA (2019)
2	40.0	550.0	90°	C24 spruce ETA (2019)
3	20.0	550.0	0°	C24 spruce ETA (2019)
4	200.0	120.0	0°	GL 24h
5	20.0	550.0	90°	C24 spruce ETA (2019)
6	40.0	550.0	0°	C24 spruce ETA (2019)
7	20.0	550.0	90°	C24 spruce ETA (2019)
t_{CLT}	360.0 mm			

ULS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0.8	35.90	523,916,900	41,376	29482.24	0.4928	6287.003
1	Center	0.8	3.1	55.00	770,026,800	53,600	38528	0.3743	9240.322
1	End	3.1	3.9	35.90	523,916,900	41,376	29482.24	0.4928	6287.003

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	9,600	552
2	40.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	40
3	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	9,600	552
4	200.0 mm	120.0 mm	L	GL 24h	9,200	520
5	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	40
6	40.0 mm	550.0 mm	L	C24 spruce ETA (2019)	9,600	552
7	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	40

ULS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0.8	35.90	425,028,600	34,122	24287.75	0.4928	5100.344
1	Center	0.8	3.1	55.00	623,504,300	43,980	31582.72	0.3743	7482.052
1	End	3.1	3.9	35.90	425,028,600	34,122	24287.75	0.4928	5100.344

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	7,742	445
2	40.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	32
3	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	7,742	445
4	200.0 mm	120.0 mm	L	GL 24h	7,797	441
5	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	32
6	40.0 mm	550.0 mm	L	C24 spruce ETA (2019)	7,742	445
7	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	32

SLS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	35.90	654,896,100	51,720	36852.8	0.4928	7858.754
1	Center	0	3.9	55.00	962,533,400	67,000	48160	0.3743	11550.4
1	End	3.9	3.9	35.90	654,896,100	51,720	36852.8	0.4928	7858.754

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	40.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	50
3	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	12,000	690
4	200.0 mm	120.0 mm	L	GL 24h	11,500	650
5	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	50
6	40.0 mm	550.0 mm	L	C24 spruce ETA (2019)	12,000	690
7	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	50

SLS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	35.90	850,564,500	74,233	52566	0.4928	10206.77
1	Center	0	3.9	55.00	1,235,111,000	93,333	66700	0.3743	14821.33
1	End	3.9	3.9	35.90	850,564,500	74,233	52566	0.4928	10206.77

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	15,000	863
2	40.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	63
3	20.0 mm	550.0 mm	L	C24 spruce ETA (2019)	15,000	863
4	200.0 mm	120.0 mm	L	GL 24h	19,167	1,083
5	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	63
6	40.0 mm	550.0 mm	L	C24 spruce ETA (2019)	15,000	863
7	20.0 mm	550.0 mm	C	C24 spruce ETA (2019)	0	63

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00
GL 24h	24.00	19.20	0.50	24.00	2.50	3.50	1.20	11,500.00	650.00	65.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	live load cat. A: domestic, residential areas	Q	medium term	0.8	0	1.5	0.7	0.5	0.3

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	1.90

LC2:live load cat. A: domestic, residential areas

continuous load

Field	Load at start [kN/m]
1	1.00

ULS Combinations

	Combination rule
LCO1	1.35/1.00 * LC1
LCO2	1.35/1.00 * LC1 + 1.50/0.00 * LC2

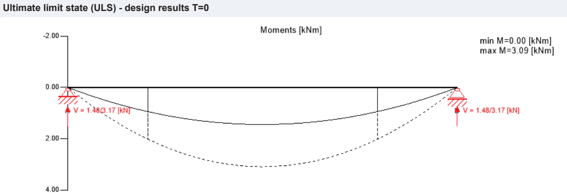
SLS Characteristic Combination

	Combination rule
LCO3	1.00/1.00 * LC1
LCO4	1.00/1.00 * LC1 + 1.00/0.00 * LC2

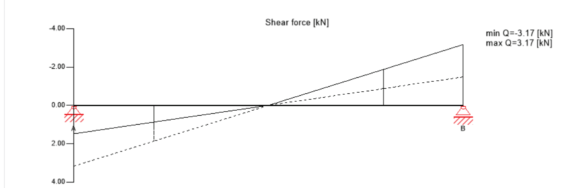
SLS Quasi-permanent Combination

	Combination rule
LCO5	1.00/1.00 * LC1
LCO6	1.00/1.00 * LC1 + 1.00/0.00 * 0.30 * LC2

Ultimate limit state (ULS) - design results T=0



Ultimate limit state (ULS) - design results T=0



ULS Flexural design T=0

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]
1	1.15	1.25	0.80	1.10	1.00	1.00	24.00	16.90	8.96
Field	$f_{c,0,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	13.44	3.09	0.00	0.00	-0.58	0.00	0.00	3 %	LCO2

ULS Axial force design T=0

Field	Dist. [m]	$f_{t,0,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]
1	1.15	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90
Field	Dist. [m]	$M_{y,d}$ [kNm]	$f_{c,0,d}$ [N/mm²]	$N_{c,d}$ [kN]	$f_{t,0,d}$ [N/mm²]	$N_{t,d}$ [kN]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Utilization	
1	1.15	3.09	13.44	0.00	8.96	0.00	0.00	0.45	5 %	LCO2

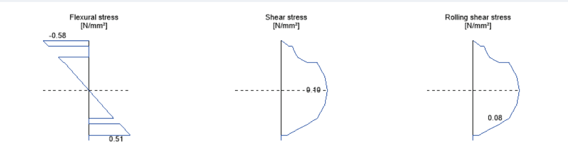
ULS Shear analysis T=0

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	3.50	1.25	0.80	0.67	2.24	3.17	0.10	5 %	LCO2

ULS Rolling shear T=0

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.05	1.25	0.80	0.67	3.17	0.08	12 %	LCO2

Stress diagram T=0



Flexural stress analysis T=0

$M_{y,d}$ =	3.09	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	8.96	N/mm²	
$\sigma_{m,y,d}$ =	-0.58	N/mm²		$f_{m,y,d}$ =	16.90	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						3 %	

Shear stress analysis T=0

V_d =	3.17	kN		$f_{v,k}$ =	3.50	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.10	N/mm²	<	$f_{v,d}$ =	2.24	N/mm²	✓
Utilization ratio						5 %	

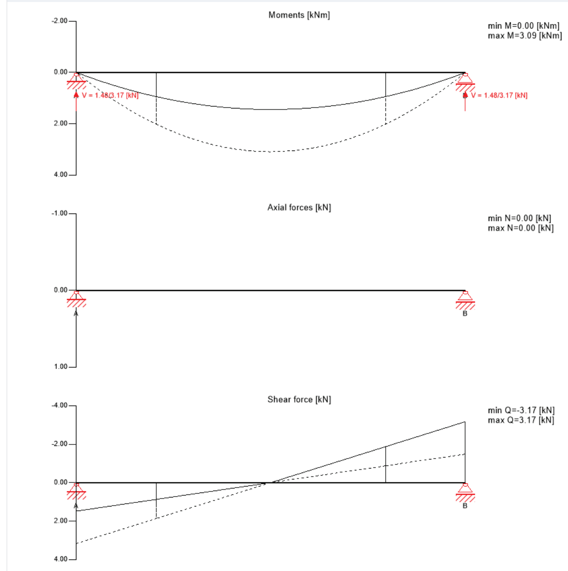
Rolling shear analysis T=0

V_d =	3.17	kN		$f_{r,k}$ =	1.05	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.08	N/mm²	<	$f_{r,d}$ =	0.67	N/mm²	✓
Utilization ratio						12 %	

Buckling analysis T=0

$M_{y,d}$ =	3.09	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	15.36	N/mm²	
$\sigma_{m,y,d}$ =	0.45	N/mm²		$f_{m,y,d}$ =	15.36	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						3 %	

Ultimate limit state (ULS) - design results T=∞



ULS Flexural design T=∞

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]
1	1.15	1.25	0.80	1.10	1.00	1.00	24.00	16.90	8.96

Field	$f_{c,0,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	13.44	3.09	0.00	0.00	-0.58	0.00	0.00	3 %	LCO8

ULS Axial force design T=∞

Field	Dist. [m]	$f_{t,0,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]
1	1.15	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90

Field	Dist. [m]	$M_{y,d}$ [kNm]	$f_{c,0,d}$ [N/mm²]	$N_{c,d}$ [kN]	$f_{t,0,d}$ [N/mm²]	$N_{t,d}$ [kN]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Utilization	
1	1.15	3.09	13.44	0.00	8.96	0.00	0.00	0.45	5 %	LCO8

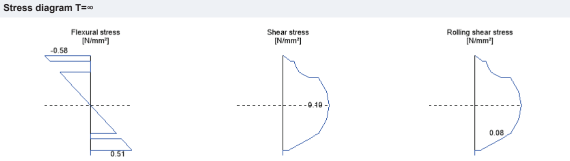
ULS Shear analysis T=∞

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	3.50	1.25	0.80	0.67	2.24	3.17	0.10	5 %	LCO8

ULS Rolling shear T=∞

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.05	1.25	0.80	0.67	3.17	0.08	12 %	LCO8

Stress diagram T=∞



Flexural stress analysis T=∞

$M_{y,d}$ =	3.09	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	8.96	N/mm²	
$\sigma_{m,y,d}$ =	-0.58	N/mm²		$f_{m,y,d}$ =	16.90	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						3 %	

Shear stress analysis T=∞

V_d =	3.17	kN		$f_{v,k}$ =	3.50	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.10	N/mm²	<	$f_{v,d}$ =	2.24	N/mm²	✓
Utilization ratio						5 %	

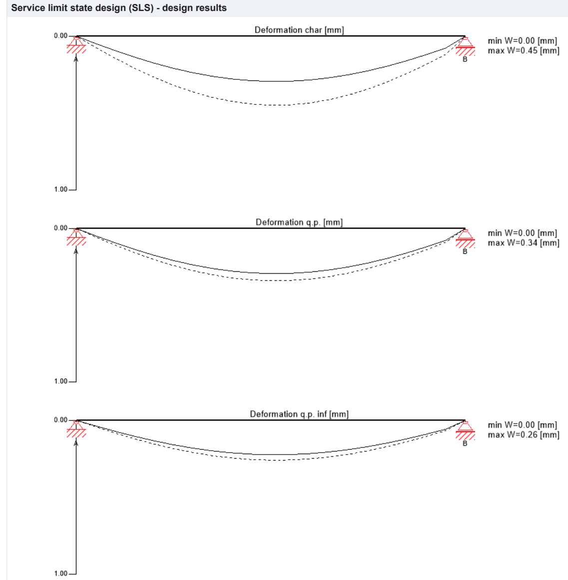
Rolling shear analysis T=∞

V_d =	3.17	kN		$f_{r,k}$ =	1.05	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.08	N/mm²	<	$f_{r,d}$ =	0.67	N/mm²	✓
Utilization ratio						12 %	

Buckling analysis T=∞

$M_{y,d}$ =	3.09	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	15.36	N/mm²	
$\sigma_{m,y,d}$ =	0.47	N/mm²		$f_{m,y,d}$ =	15.36	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						3 %	

Service limit state design (SLS) - design results



w_{inst} = w[char]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	0.6	0.8	L/300	13.0	0.4	3 %

w_{fin} = w[char] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	0.6	0.8	L/250	15.6	0.7	5 %

$w_{net,fin}$ = w[q.p.] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	0.6	0.8	L/300	13.0	0.6	5 %

Vibration analysis

General		
Total mass	1.89	[t]
Tributary width	1.2	[m]
Stiffness Longitudinal direction	11550.4	[kNm²]
Stiffness Cross direction	128.0	[kNm²]
Modal damping	1.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	217.0	[kg]

Analysis

Criterion	Calc.	Class I	Class II	Class I	Class II	Cl. I	Cl. II
Frequency criterion min	39.887 [Hz]	4.5 [Hz]	4.5 [Hz]	11 %	11 %	✓	✓
Frequency criterion	39.887 [Hz]	8.0 [Hz]	6.0 [Hz]	20 %	15 %		
Acceleration criterion	0.0 [m/s²]	0.05 [m/s²]	0.1 [m/s²]	0 %	0 %		
Stiffness criterion	0.093 [mm]	0.25 [mm]	0.5 [mm]	37 %	19 %	✓	✓

Support design

Nr.	Type	Width [mm]	Area [cm²]	k_{mod} [-]	γ_m [-]	$k_{c,90}$ [-]	$f_{c,k}$ [N/mm²]	$f_{c,d}$ [N/mm²]	V_{max} [kN]	V_{min} [kN]	$\sigma_{c,90,d}$ [N/mm²]		Ratio
A	Rigid plate	350	3040.00	0.80	1.25	1.50	2.50	2.40	3.17	0.00	0.01	LCO2	0 %
B	Rigid plate	350	3040.00	0.80	1.25	1.50	2.50	2.40	3.17	0.00	0.01	LCO2	0 %

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V [kN]
self-weight structure	0.6	1.48	1.48
		1.48	1.48
live load cat. A: domestic, residential areas	0.8	0.78	0.78
		0.00	0.00

Note

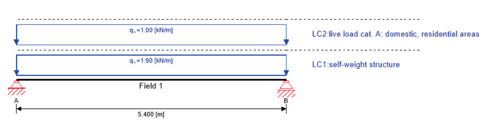
Load case with whole support of the slab, CLT deck on top

Reference documents for this analysis

English title	Description
ETA-11/0190	selftaping screw by Würth

Disclaimer

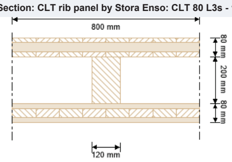
System



Global utilization ratio 64 %

ULS		ULS Fire		SLS		SLS Vibration		Support	
ULS	17 %	ULS Fire	-1 %	SLS	9 %	SLS Vibration	64 %	Support	1 %

Section: CLT rib panel by Stora Enso: CLT 80 L3s - 12/20 - CLT 80 C3s



Ribs width are reduced by 20 mm for the design

Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	20.0	644.0	0°	C24 spruce ETA (2019)
2	40.0	644.0	90°	C24 spruce ETA (2019)
3	20.0	644.0	0°	C24 spruce ETA (2019)
4	200.0	120.0	0°	GL 24h
5	20.0	644.0	90°	C24 spruce ETA (2019)
6	40.0	644.0	0°	C24 spruce ETA (2019)
7	20.0	644.0	90°	C24 spruce ETA (2019)
t_{CLT}	360.0 mm			

ULS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0.8	43.10	616,691,200	45,984	32892.16	0.4464	7400.294
1	Center	0.8	4.6	64.40	891,148,900	59,616	42979.84	0.3208	10693.79
1	End	4.6	5.4	43.10	616,691,200	45,984	32892.16	0.4464	7400.294

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	9,600	552
2	40.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	40
3	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	9,600	552
4	200.0 mm	120.0 mm	L	GL 24h	9,200	520
5	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	40
6	40.0 mm	644.0 mm	L	C24 spruce ETA (2019)	9,600	552
7	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	40

ULS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0.8	43.10	499,846,700	37,838	27037.69	0.4464	5998.16
1	Center	0.8	4.6	64.40	721,183,500	48,832	35172.92	0.3208	8654.202
1	End	4.6	5.4	43.10	499,846,700	37,838	27037.69	0.4464	5998.16

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	7,742	445
2	40.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	32
3	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	7,742	445
4	200.0 mm	120.0 mm	L	GL 24h	7,797	441
5	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	32
6	40.0 mm	644.0 mm	L	C24 spruce ETA (2019)	7,742	445
7	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	32

SLS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	43.10	770,864,100	57,480	41115.2	0.4464	9250.369
1	Center	0	5.4	64.40	1,113,936,000	74,520	53724.8	0.3208	13367.23
1	End	5.4	5.4	43.10	770,864,100	57,480	41115.2	0.4464	9250.369

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	40.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	50
3	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	12,000	690
4	200.0 mm	120.0 mm	L	GL 24h	11,500	650
5	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	50
6	40.0 mm	644.0 mm	L	C24 spruce ETA (2019)	12,000	690
7	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	50

SLS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	43.10	995,524,500	81,433	57894	0.4464	11946.29
1	Center	0	5.4	64.40	1,424,365,000	102,733	73655.99	0.3208	17092.38
1	End	5.4	5.4	43.10	995,524,500	81,433	57894	0.4464	11946.29

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	15,000	863
2	40.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	63
3	20.0 mm	644.0 mm	L	C24 spruce ETA (2019)	15,000	863
4	200.0 mm	120.0 mm	L	GL 24h	19,167	1,083
5	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	63
6	40.0 mm	644.0 mm	L	C24 spruce ETA (2019)	15,000	863
7	20.0 mm	644.0 mm	C	C24 spruce ETA (2019)	0	63

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00
GL 24h	24.00	19.20	0.50	24.00	2.50	3.50	1.20	11,500.00	650.00	65.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	live load cat. A: domestic, residential areas	Q	medium term	0.8	0	1.5	0.7	0.5	0.3

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	1.90

LC2:live load cat. A: domestic, residential areas

continuous load

Field	Load at start [kN/m]
1	1.00

ULS Combinations

	Combination rule
LCO1	1.35/1.00 * LC1
LCO2	1.35/1.00 * LC1 + 1.50/0.00 * LC2

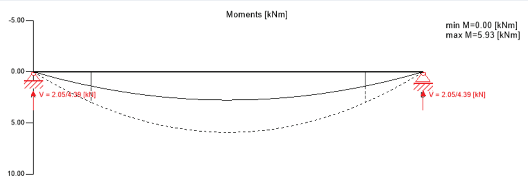
SLS Characteristic Combination

	Combination rule
LCO3	1.00/1.00 * LC1
LCO4	1.00/1.00 * LC1 + 1.00/0.00 * LC2

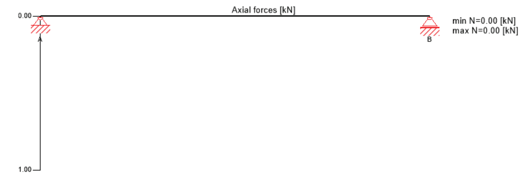
SLS Quasi-permanent Combination

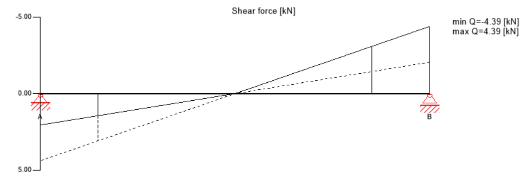
	Combination rule
LCO5	1.00/1.00 * LC1
LCO6	1.00/1.00 * LC1 + 1.00/0.00 * 0.30 * LC2

Ultimate limit state (ULS) - design results T=0



Ultimate limit state (ULS) - design results T=0





ULS Flexural design T=0

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]
1	1.9	1.25	0.80	1.10	1.00	1.00	24.00	16.90	8.96

Field	$f_{c,0,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	13.44	5.93	0.00	0.00	-0.96	0.00	0.00	6 %	LCO2

ULS Axial force design T=0

Field	Dist. [m]	$f_{t,0,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]
1	1.9	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90

Field	Dist. [m]	$M_{y,d}$ [kNm]	$f_{c,0,d}$ [N/mm²]	$N_{c,d}$ [kN]	$f_{t,0,d}$ [N/mm²]	$N_{t,d}$ [kN]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Utilization	
1	1.9	5.93	13.44	0.00	8.96	0.00	0.00	0.74	8 %	LCO2

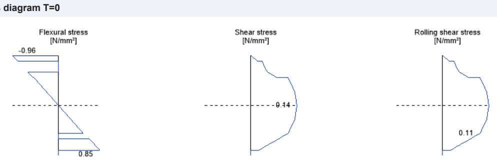
ULS Shear analysis T=0

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	3.50	1.25	0.80	0.67	2.24	4.39	0.14	6 %	LCO2

ULS Rolling shear T=0

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.05	1.25	0.80	0.67	4.39	0.11	17 %	LCO2

Stress diagram T=0



Flexural stress analysis T=0

$M_{y,d}$ =	5.93 kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00 kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00 kN		γ_m =	1.25	-	
			k_{mod} =	0.80	-	
			$k_{sys,y}$ =	1.10	-	
			$k_{h,m,y}$ =	1.00	-	
			$k_{h,m,z}$ =	1.00	-	
			k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00 N/mm²		$f_{t,0,d}$ =	8.96	N/mm²	
$\sigma_{m,y,d}$ =	-0.96 N/mm²		$f_{m,y,d}$ =	16.90	N/mm²	
$\sigma_{m,z,d}$ =	0.00 N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓

Utilization ratio 6 %

Shear stress analysis T=0

V_d =	4.39 kN		$f_{v,k}$ =	3.50	N/mm²	
			γ_m =	1.25	-	
			k_{mod} =	0.80	-	
			$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.14 N/mm²	<	$f_{v,d}$ =	2.24	N/mm²	✓

Utilization ratio 6 %

Rolling shear analysis T=0

V_d =	4.39 kN		$f_{r,k}$ =	1.05	N/mm²	
			γ_m =	1.25	-	
			k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.11 N/mm²	<	$f_{r,d}$ =	0.67	N/mm²	✓

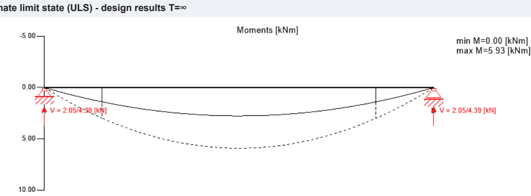
Utilization ratio 17 %

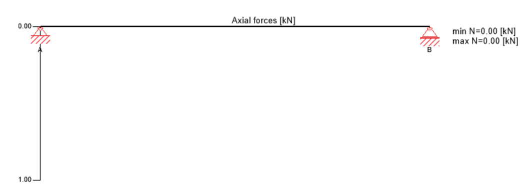
Buckling analysis T=0

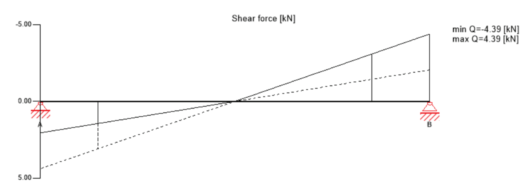
$M_{y,d}$ =	5.93 kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00 kNm					
$N_{c,d}$ =	0.00 kN		γ_m =	1.25	-	
			k_{mod} =	0.80	-	
			$k_{sys,y}$ =	1.00	-	
			$k_{sys,z}$ =	1.00	-	
			$k_{h,m,y}$ =	1.00	-	
			$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00 N/mm²		$f_{c,0,d}$ =	15.36	N/mm²	
$\sigma_{m,y,d}$ =	0.74 N/mm²		$f_{m,y,d}$ =	15.36	N/mm²	
$\sigma_{m,z,d}$ =	0.00 N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓

Utilization ratio 5 %

Ultimate limit state (ULS) - design results T=∞







ULS Flexural design T=∞

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]
1	1.9	1.25	0.80	1.10	1.00	1.00	24.00	16.90	8.96

Field	$f_{c,0,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	13.44	5.93	0.00	0.00	-0.95	0.00	0.00	6 %	LCO8

ULS Axial force design T=∞

Field	Dist. [m]	$f_{t,0,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]
1	1.9	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90

Field	Dist. [m]	$M_{y,d}$ [kNm]	$f_{c,0,d}$ [N/mm²]	$N_{c,d}$ [kN]	$f_{t,0,d}$ [N/mm²]	$N_{t,d}$ [kN]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Utilization	
1	1.9	5.93	13.44	0.00	8.96	0.00	0.00	0.74	8 %	LCO8

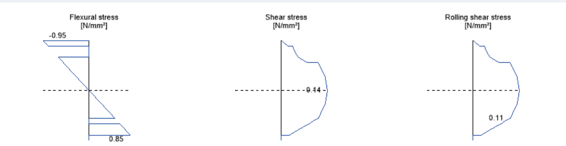
ULS Shear analysis T=∞

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	3.50	1.25	0.80	0.67	2.24	4.39	0.14	6 %	LCO8

ULS Rolling shear T=∞

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.05	1.25	0.80	0.67	4.39	0.11	17 %	LCO8

Stress diagram T=∞



Flexural stress analysis T=∞

$M_{y,d}$ =	5.93	kNm		$f_{m,k}$ =	24.00	N/mm²
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-
				k_{mod} =	0.80	-
				$k_{sys,y}$ =	1.10	-
				$k_{h,m,y}$ =	1.00	-
				$k_{h,m,z}$ =	1.00	-
				k_l =	1.00	-
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	8.96	N/mm²
$\sigma_{m,y,d}$ =	-0.95	N/mm²		$f_{m,y,d}$ =	16.90	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm² ✓

Utilization ratio 6 %

Shear stress analysis T=∞

V_d =	4.39	kN		$f_{v,k}$ =	3.50	N/mm²
				γ_m =	1.25	-
				k_{mod} =	0.80	-
				$k_{h,v}$ =	0.00	-
$\tau_{v,d}$ =	0.14	N/mm²	<	$f_{v,d}$ =	2.24	N/mm² ✓

Utilization ratio 6 %

Rolling shear analysis T=∞

V_d =	4.39	kN		$f_{r,k}$ =	1.05	N/mm²
				γ_m =	1.25	-
				k_{mod} =	0.80	-
$\tau_{r,d}$ =	0.11	N/mm²	<	$f_{r,d}$ =	0.67	N/mm² ✓

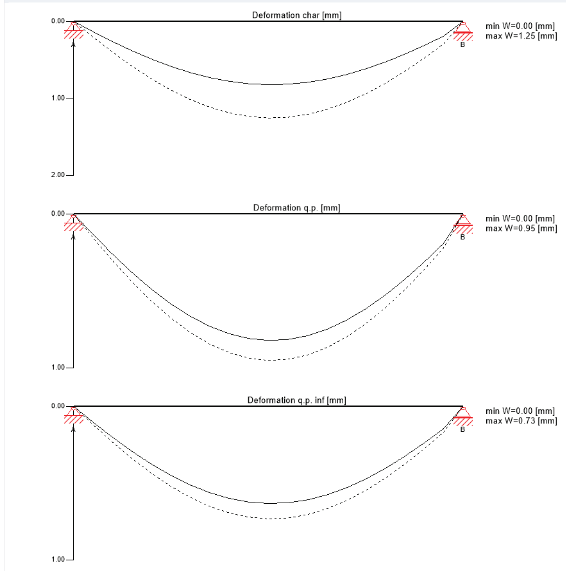
Utilization ratio 17 %

Buckling analysis T=∞

$M_{y,d}$ =	5.93	kNm		$f_{m,k}$ =	24.00	N/mm²
$M_{z,d}$ =	0.00	kNm				
$N_{c,d}$ =	0.00	kN		γ_m =	1.25	-
				k_{mod} =	0.80	-
				$k_{sys,y}$ =	1.00	-
				$k_{sys,z}$ =	1.00	-
				$k_{h,m,y}$ =	1.00	-
				$k_{h,m,z}$ =	1.00	-
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	15.36	N/mm²
$\sigma_{m,y,d}$ =	0.77	N/mm²		$f_{m,y,d}$ =	15.36	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm² ✓

Utilization ratio 5 %

Service limit state design (SLS) - design results



w_{inst} = w[char]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	0.6	0.8	L/300	18.0	1.3	7 %

w_{fin} = w[char] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	0.6	0.8	L/250	21.6	2.0	9 %

$w_{net,fin}$ = w[q.p.] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	0.6	0.8	L/300	18.0	1.7	9 %

Vibration analysis

General		
Total mass	2.61	[t]
Tributary width	1.5	[m]
Stiffness Longitudinal direction	13367.2	[kNm²]
Stiffness Cross direction	128.0	[kNm²]
Modal damping	1.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	401.1	[kg]

Analysis

Criterion	Calc.	Class I	Class II	Class I	Class II	Cl. I	Cl. II
Frequency criterion min	22.382 [Hz]	4.5 [Hz]	4.5 [Hz]	20 %	20 %	✓	✓
Frequency criterion	22.382 [Hz]	8.0 [Hz]	6.0 [Hz]	36 %	27 %		
Acceleration criterion	0.005 [m/s²]	0.05 [m/s²]	0.1 [m/s²]	9 %	5 %		
Stiffness criterion	0.16 [mm]	0.25 [mm]	0.5 [mm]	64 %	32 %	✓	✓

Support design

Nr.	Type	Width [mm]	Area [cm²]	k_{mod} [-]	γ_m [-]	$k_{c,90}$ [-]	$f_{c,k}$ [N/mm²]	$f_{c,d}$ [N/mm²]	V_{max} [kN]	V_{min} [kN]	$\sigma_{c,90,d}$ [N/mm²]		Ratio
A	Rigid plate	350	3040.00	0.80	1.25	1.50	2.50	2.40	4.39	0.00	0.01	LCO2	1 %
B	Rigid plate	350	3040.00	0.80	1.25	1.50	2.50	2.40	4.39	0.00	0.01	LCO2	1 %

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V [kN]
self-weight structure	0.6	2.05	2.05
		2.05	2.05
live load cat. A: domestic, residential areas	0.8	1.08	1.08
		0.00	0.00

Note

Load case with whole support of the slab, CLT deck on top

Reference documents for this analysis

English title	Description
ETA-11/0190	selftaping screw by Würth

Disclaimer

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Master Thesis Research

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