Reducing the environmental impact of distribution centres

Designing green-enveloped biobased alternatives to compare with reference steel and concrete designs

MSc Thesis

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Royal HaskoningDHV
Enhancing Society Together

Reducing the environmental impact of distribution centres

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by

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Preface

This project marks the end of my MSc Civil Engineering - Structural Track at TU Delft, with a specialisation in Materials & Environment. After two years at the French engineering school Mines Nancy, I decided to come to the Netherlands to broaden my knowledge in sustainable building and experience life abroad. In my graduation project, I was curious to work on a real-life case study, and investigate possible ways to make a significant positive impact. I chose to focus on biobased materials, and the practical application of green envelopes in large scale projects like distribution centres.

First, I would like to thank Royal HaskoningDHV for providing me with the opportunity for this thesis, on such a inspiring and relevant topic. My gratitude goes to all members of my graduation committee, for their valuable guidance during the past year. In particular, Henk for our insightful discussions on environmental impact calculations and biogenic carbon, Marc for his recommendations on envelope design and greening systems, and Geert for his precious input on timber structural design. A very special thanks to Michiel for helping me stay on track throughout this project, providing me every week with sound advice and challenging me to always keep a critical and open mind.

Lastly, I want to thank my family and friends for their love and continuous support during my studies, even from afar. Thank you to my parents and to my sister Clémence for believing in me at all times, and keeping me motivated during the writing of this thesis. To Lucas, thank you for always being by my side, through the ups and downs.

Justine Ruminy
Delft, January 2023

Summary

The climate emergency calls for a global shift towards a low-carbon economy, to preserve natural resources and biodiversity, while limiting global warming and its detrimental consequences. This project focuses on the impact of distribution centres, intended for the storage of commercial goods before they are sold and shipped to customers. Warehouses are characterised by large flexible spaces and simple structural systems. Steel or concrete are most often selected for the frame, as designers aim at efficiency and low costs rather than adopt a sustainability-driven mindset in the choice of materials. Therefore, warehouse structures show a significant potential for improvement regarding their environmental footprint. As measures are taken to reduce the energy consumption of the building sector and switch to renewable energy sources, reducing the embodied carbon of materials becomes critical to achieve net zero emissions by 2050. The highest share of embodied carbon in the life cycle of a building is associated with the production stage related to immediate impacts, most relevant to short-term sustainability goals than long-term end-of-life scenarios.

The aim of this research is to create a sustainable single-storey warehouse with a significantly lower environmental impact compared to reference designs with steel and concrete frames. Considering a new-build project with no reuse of existing elements, a meaningful target for embodied carbon reduction is set to 50% of the environmental impact score of reference steel and concrete warehouse designs.

A selection of sustainable design strategies are investigated, to give insight into which actions designers shall take in priority to effectively reduce the embodied carbon of a warehouse. Early design measures for sustainability are the most effective to lower the impact of a building project, as environmental hotspots can be identified from the start and tackled in priority. Reducing upfront emissions requires looking into short-term sustainability measures like reducing material use or using low carbon alternatives wherever possible. Additionally, replacing fossil-based products by renewable carbon sequestering materials like timber or other biobased alternatives contributes to lowering the impact of a building, by storing biogenic carbon for the lifespan of these elements. Biogenic carbon storage may be extended beyond the building's end-of-life by designing for long-term reuse or recycling, in line with circularity principles. Greening the envelope of a building takes advantage of ecosystem services provided by vegetation, such as improving air quality, enhancing thermal performance or supporting biodiversity. Many systems can be applied on facades and roofs, depending on the requirements of each project (plant species, layout, load-bearing capacity of the structural frame), to effectively introduce vegetation in the built environment.

In this project, the scope of the warehouse structural design includes the load-bearing frame, foundation pads, ground floor slab, envelope panels and their supports. The environmental impact score refers to the embodied carbon content in kgCO2e, focusing on upfront emissions only (LCA modules A1-A3). Global Warming Potential (GWP) indicators are retrieved from Environmental Product Declarations (EPDs) collected from the OneClick LCA online tool. For each material or product, EPDs representative of the lowest and highest possible impact available on the European market are selected.

Biogenic carbon uptake in biobased materials occurs during photosynthesis, responsible for biomass growth. When the material is burnt or degraded, carbon is released back to the atmosphere and the biogenic carbon balance ends up being zero over the whole lifecycle of biobased products, hence why it may be referred to as temporary carbon storage. A sustainable use of wood presumes that its design lifespan is at least equal to forest rotation periods to maintain or increase the forest timber stock, combined with sustainable forest management practices. End-of-life scenarios for wood products result either in carbon re-emission to the atmosphere by burning or landfilling, or extending the storage duration beyond 100 years by reusing or recycling. The second option turns waste into a resource and is supported by circularity goals for 2050, therefore affecting all current projects with a design service life over 30 years. Reuse and recycling of biobased products becoming the norm in the building sector would invalidate the notion of ”temporary” storage as it could then be considered permanent. In current LCA methods, biogenic carbon storage is to be credited separately from fossil emissions. Biogenic carbon uptake at the product stage can only be accounted as negative emissions when carbon release is reported as positive at the end-of-life. However, some argue that considering carbon uptake as negative emissions rightly translates the potential for long-term carbon storage in elements with a high probability of reuse at their end-of-life. Moreover, in the context of climate emergency, even temporary storage contributes to reducing carbon levels in the atmosphere immediately, illustrated by this negative value. In this research, final impact score results are expressed as specified by EN 15804, considering fossil carbon only on one hand, and the total score subtracting biogenic carbon storage from fossil emissions on the other hand for clarity.

To answer the main research question, the following design steps are carried out:

- **Design step 1:** Evaluating the impact of material substitution of fossil-based steel and concrete by timber in the warehouse load-bearing frame
- **Design step 2:** Further reducing the environmental impact of a timber warehouse by investigating a selection of sustainable design strategies targeting all parts of the building (thinner floor slab, biobased envelope materials, greening systems, demountable connections...)

The **first design step** investigates the embodied carbon reduction potential of substituting steel or concrete by timber in the load-bearing frame of a warehouse. To do so, reference steel and concrete warehouse designs are selected. Two baseline timber warehouses are then designed with similar geometrical characteristics and stability systems to compare their environmental impact on a fair basis. The bi-directional sway frame of both timber designs have semi-rigid column base connections, using glued-in rods to provide lateral stiffness. Amongst a variety of wood engineered materials, glued laminated timber of strength class GL28h is selected for beams and columns in the load-bearing frame. The floor slab thickness (200mm) and envelope components (Kingspan sandwich panels on steel supports) are identical for all warehouse designs in this first step.

Substituting steel by timber in the frame of a warehouse results in a reduction of approximately 31% of the total embodied carbon. By considering the benefits of biogenic carbon storage in timber elements, the impact score reduction reaches 66%. The embodied carbon reduction from concrete to timber is 33% when only fossil emissions are considered, and up to 64% with biogenic carbon storage. Significant impact reduction is observed in both cases, but the 50% target set in the main research question is only achieved when biogenic carbon is accounted for in the final score.

The **second design step** of the research aims at further reducing the impact of not only the frame, but also other parts of the timber warehouse. Big ticket items responsible for the most impact are identified and tackled in priority: in both baseline timber designs, the floor slab and envelope are responsible for the largest share of embodied carbon, around 40% each, followed by foundation pads and the frame.

First, it is found that every 10mm reduction of the floor slab thickness results in 2% reduction of the total embodied carbon of the warehouse, making it an effective measure towards sustainability.

Then, several design alternatives are created for the building envelope, all complying with similar functional requirements like thermal insulation or protection against external climate. For each alternative, the size of structural elements in the load-bearing frame is adapted to support possible heavier loads in the envelope. Three types of roof and facade sandwich panels are selected, with steel sheathing surrounding different insulation materials (Kingspan Quadcore, polyurethane and mineral wool). The influence of steel or glulam supports on the total impact is investigated. Biobased built-up solutions are also created for the roof and facades, using wood engineered products like glulam mullions, wood fibre insulation, OSB panels, I-joists and hollow core timber boxes. External protection of the biobased envelope is ensured by an EPDM layer on the roof, and external steel sheathing on the facade. Design alternatives involving biobased materials are efficient to reduce fossil carbon, beside the clear advantage of biogenic carbon storage. Using long-spanning elements also allows to do without additional purlins or mullions, resulting in material savings for supports.

A lightweight green envelope option is also considered: vertical green is applied with an indirect climber plants system made with a stainless steel mesh, and horizontal green is implemented with extensive green roof options of saturated weight ranging from 50 to 150kg/m^2. The embodied carbon of additional materials required in the greening process (green systems, larger frame elements) should be weighed against the benefits of ecosystem services provided by vegetation, among which are biodiversity preservation, air quality regulation, improved thermal insulation, and pleasant aesthetics positively impacting people's well-being. Only the environmental costs of materials is evaluated in this project, leaving the quantification of ecosystem services benefits for future research.

The last sustainable design measures apply to the structural timber frame. An alternative design is proposed for the base glued-in rods connection, adding an intermediate baseplate between the rods and foundation pad to facilitate disassembly and reuse of timber columns at the warehouse end-of-life, ensuring long-term carbon storage. More generally, core characteristics of the frame layout and structural system could be modified to reduce material use. These strategies are not only applicable to timber frames, but also steel and concrete. Changing the grid size to reduce the span of roof beams would help reducing design loads, hence the required dimensions of structural elements. Cross-sections of beams and columns may be adapted to best fit design loads along the length. Rethinking the general stability system of the warehouse, for instance by bracing the sway frame in one direction, would remove the need for fixed connections, hence the size of columns. Timber frame variants could be developed using parametric design, integrating environmental impact calculations to the structural design process to evaluate a large number of design alternatives simultaneously. Functionality and flexibility of the warehouse should be kept in mind during this sustainability-driven design process.

Based on the results of design steps 1 and 2, the 50% embodied carbon reduction target can be achieved by substituting steel or concrete in the frame by timber (-31%), replacing sandwich panels on steel supports by biobased materials in the envelope (-13%), and reducing the floor slab thickness by 50mm (-7%), when considering only fossil emissions. If biogenic carbon storage is accounted for, using biobased materials in the frame (-65%) and in the envelope (-58%), without modifying the floor slab, results in a carbon positive warehouse design. These results are only valid under the assumption of long-term carbon storage in biobased materials, supported by circular design measures. Greening the envelope can also contribute to making warehouses more sustainable, provided that benefits from ecosystem services outweigh environmental costs from additional materials.

Contents

List of Figures

List of Tables

1 Introduction

This chapter sets the basis for the research project, explaining the global context behind it in Section 1.1. As this project is conducted with Royal HaskoningDHV, the company is presented in Section 1.2 as well as the case study.

1.1 Context

Climate change is now clearly identified as a threat for populations and the environment, making it a common concern worldwide. The global population doubled over the past five decades and is expected to reach 8.6 billion by 2030, setting new challenges for the construction industry and the use of natural resources. Continuing with current practices would lead to double the actual consumption in natural resources by 2050, already responsible for more than 90% of biodiversity loss and water stress, and approximately half of the observable impacts of climate change. This calls for a significant change in design practices, focusing on renewable resources, minimising material use and the overall environmental impact of buildings (UNEP, 2019).

The Intergovernmental Panel on Climate Change (IPCC) released in 2022 its Sixth Assessment Report, gathering current scientific findings on climate change, its impacts and mitigation measures for the future. It is clear that the climate emergency is endangering nature, people and infrastructures all over the world. Anthropogenic driven global warming affects global ocean currents, strongly modifies precipitation patterns and leads to recurring extreme weather events like storms, flooding or hurricanes on a local scale. As a result, many regions shall witness substantial biodiversity loss, food and water insecurity. Action should be taken on the short term to avoid the most extreme scenarios for vulnerable areas. Therefore, a drastic cut in human-made Greenhouse Gas (GHG) emissions is required to limit global warming in the upcoming years (IPCC, 2022). Following the COP21, the 2015 Paris Agreement set targets for cutting down 50% of GHG emissions by 2050 relative to 1990, to keep global temperature rise below 2^{o}C. The EU should also aim at achieving carbon neutrality by 2100, using 100% renewable energy (Jonkers, 2020).

The building sector as a whole is responsible for a large part of greenhouse gases emissions, water and material consumption, waste generation, and energy use. In 2018, the industry was responsible for approximately 36% of final energy use and 39% of energy and process-related carbon emissions. Furthermore, emissions linked to the building stock are increasing over the years, driven by population growth. It is therefore a primary target for mitigating climate change (GlobalABC & IEA, 2019). In 2010, the EU required all buildings to be nearly-zero energy (nZEB) by 2020. A revised version of the Energy Performance of Buildings Directive set goals for decarbonizing the building stock by 2050 (European Parliament & Council, 2018). On the national scale, European countries also launched national sustainability policies for the construction sector, encouraging thermal renovation of the existing building stock, and reduction of primary resource consumption for new projects. In the Netherlands, 50% of raw primary material resources are used by building activities, also responsible for generating consequent amounts of waste products. The Dutch government aims at 50% reduction of primary resource consumption by 2030, and at least 90% by 2050 (Jonkers, 2020).

Building-related GHG emissions are divided between operational energy during the use phase for cooling and heating, and embodied emissions linked to material use during the construction process (CE Delft, 2020). Both of these should be considered for achieving Net Zero Carbon buildings by 2050, in the context of whole life carbon. The primary focus of the European Energy Performance of Buildings Directive is to reduce final energy consumption of buildings by deep renovation of the ageing building stock, while decarbonising energy sources globally (European Parliament & Council, 2018). When the energy use is effectively reduced, the remaining fraction of the environmental impact is governed by embodied emissions. Strategies to reduce these include adopting a circular approach, using biobased materials, or finding innovative ways to design, build and deal with end-of-life of construction works.

1.2 Case study

1.2.1 Royal HaskoningDHV

Royal HaskoningDHV
Enhancing Society Together

Figure 1.1: RHDHV logo

Royal HaskoningDHV is an independant consultancy firm based in the Netherlands, operating in more than 25 countries across the world on engineering projects, with a common mission: *"Enhancing Society Together"*. The company aims at having a positive impact on the world through innovation and technology, by developing sustainable solutions to local and global issues related to the built environment. Their clients come from public and private sectors in the fields of aviation, buildings, energy, industry, infrastructure, maritime, urban development and water.

1.2.2 The project - Sustainable distribution centres

This research project will focus on distribution centres. Standard designs are generally implemented across many countries for commercial activities. The client of Royal HaskoningDHV aims at having only net zero carbon buildings by 2040, hence the need to develop sustainable alternatives to the current steel and concrete reference designs. Only focusing on the load-bearing structure, responsible for 50 to 80% of total material related emissions, the environmental impact could be reduced greatly. Apart from the structural frame, elements in the envelope or floor slab also constitute a considerable share of all building materials used in such projects. However, design choices are often driven by economic costs rather than sustainability. The climate emergency requires taking deep action to rethink the whole design and building process, shifting the focus to low impact solutions in priority. In the building industry, a sustainability driven mindset should be adopted from the early design stage to drastically reduce carbon emissions.

2
Research approach

This chapter presents the main objective and research questions to be answered in the research project. The general approach is summarised, followed by the scope definition and an overview of the thesis outline.

2.1 Objective

The goal is to reduce the environmental impact of a reference distribution centre by at least half. In this research project, the objective is to create a sustainable warehouse design so as to significantly reduce the environmental impact compared to steel or concrete references. To achieve the target of ”at least half”, several alternatives shall be investigated, including the use of biobased materials in the load-bearing frame and the envelope, or applying a living green envelope if beneficial.

2.2 Research questions

Main research question

How to reduce the environmental impact score of reference concrete and steel distribution centre designs by at least half?

Subquestions

The main research question is divided into 3 specific subquestions structuring the project:

1. **What are the most effective strategies to lower the environmental impact of warehouse structures?**
    - What are the benefits of including sustainability from an early design stage?
    - What is the advantage of using biobased building materials over steel or concrete regarding sustainability?
    - What circular building strategies can be implemented to reduce the impact of a warehouse structure?

- How does integrating nature in the design of buildings contribute to lowering their environmental impact?

2. **How much environmental impact reduction can be achieved by substituting steel or concrete by timber in the load-bearing frame of a warehouse?**
   - What type of structural system is appropriate for a baseline timber warehouse?
   - What is the environmental impact of steel and concrete warehouse structures?

3. **What design strategies should be considered in priority to effectively reduce the environmental impact of a green enveloped biobased warehouse?**
   - Which structural elements of the baseline timber warehouse have the most influence on its total environmental impact?
   - How much environmental impact reduction can be achieved by choosing different design options for the structural elements of a timber warehouse, compared to the baseline design?
   - When does a green envelope contribute to reducing the environmental impact of a warehouse?

2.3 General approach

The goal of this research project is to develop a structural design for distribution centres with less than half the impact score of reference steel and concrete designs. In order to fulfil this objective, a first part investigates sustainable design strategies applicable to warehouses, after which two steps are taken putting these strategies into practice. Carrying out two separate design steps enables a fair comparison of results.

2.3.1 Literature study - Sustainable design strategies

The literature study presented in Chapter 3 sets the foundation for this research project and aims at answering the first research question:

> *1. What are the most effective strategies to lower the environmental impact of warehouse structures?*

2.3.2 Design step 1 - Substituting steel/concrete by timber in the load-bearing frame of a warehouse

First, the influence of choosing timber over steel or concrete as a structural building material is investigated to answer the second research question:

> *2. How much environmental impact reduction can be achieved by substituting concrete or steel by timber in the load-bearing frame of a warehouse?*

Steel and concrete reference designs are selected from existing projects at RHDHV, answering client specific requirements. As these references are meant to be implemented all over Europe and adapted to each location, their layout and the size of structural elements were not designed to satisfy sustainability related matters like material use or demountability. To ensure a fair comparison, timber baseline designs are developed with a similar geometry as the reference SC16 and IXD models in a first design iteration. The embodied carbon of each warehouse design is calculated based on materials quantities and environmental data from EPDs. The contribution of different parts of the design (foundations, ground floor, frame, envelope elements) to the final impact can be analysed individually to determine leads for sustainability improvement. The goal of this first stage is also to explore the basics of designing with timber, resulting in a simple structural design to serve as a base for the second design step.

Scope:

- **Frame:** material substitution from steel or concrete to timber, connection design
- **Foundation pads:** size adapted to design loads at the base of columns
- **Floor slab:** unchanged
- **Building envelope:** out of the scope

2.3.3 Design step 2 - Further reducing the impact of a timber warehouse focusing on environmental hotspots

After designing timber warehouses, a selection of sustainable design strategies is investigated to answer the third research question:

> *3. What is the most optimal design of a green enveloped biobased warehouse for minimizing its environmental impact?*

After comparing the differences between using timber and fossil-based materials in the load-bearing structure of a distribution centre, the second phase of the research focuses on timber frames only. Big ticket items responsible for the most environmental impact identified during the first design iterations are tackled in priority. They are the starting point for applying impact reduction strategies among those identified in the literature review: for instance reducing material use where possible in the warehouse, or substituting carbon intensive materials by low impact or biobased alternatives. A sensitivity analysis is conducted to compare design alternatives on the basis of their embodied carbon.

Scope:

- **Frame:** timber elements, size adapted to design loads from envelope variants, connection design
- **Foundation pads:** size adapted to design loads at the base of columns
- **Floor slab:** variations in thickness and materials
- **Building envelope:** variations in materials, adding a green layer

2.3.4 Conclusion and recommendations

The final chapter concludes the research project by answering the main research question:

> *How to reduce the environmental impact score of reference concrete or steel distribution centres designs by at least half?*

An overview is given of results from the two design steps conducted, and keys designers may retrieve from this thesis to effectively reduce the environmental impact of distribution centre designs.

2.4 Scope

Due to the restricted time allowed for the thesis, the research activities should be confined to certain limits to fit within the planned duration.

- **Distribution centres:** This study focuses on single-storey warehouse structures, considering no bumpouts.
- **Reference designs:** The study of concrete and steel designs is restricted to two reference projects developed by RHDHV, presented extensively in Section 4.1.
- **Structural elements:** The typical warehouse structure considered in this research comprises the load-bearing frame (columns and beams), foundation pads, ground floor slab and building envelope (roof and facade panels, supporting purlins and mullions).
- **Roof bracings:** The roof structure of timber warehouse designs is assumed braced, acting as a horizontal diaphragm. The specific design of bracing elements is left out of the scope, and they are not considered either in environmental impact calculations.
- **Impact reduction target:** The initial target is to divide the reference impact to at least half. The study of design variants in step 2 does not aim at strictly minimising the impact, but only achieving this target in the first place.
- **Environmental impact score:** Upfront carbon emissions only, associated with the product stage (LCA modules A1-A3). Most certainty and highest contribution to the total embodied carbon of a building over its life cycle.

2.5 Thesis outline

RESEARCH DEFINITION

Chapter 1 – Introduction

Chapter 2 – Research approach

LITERATURE REVIEW

Chapter 3 – Sustainable design strategies

(1) What are the most effective strategies to lower the environmental impact of warehouse structures?

- *What are the benefits of including sustainability from an early design stage?*
- *What is the advantage of using biobased building materials over steel or concrete regarding sustainability?*
- *What circular building strategies can be implemented to reduce the impact of a warehouse structure?*
- *How does integrating nature in the design of buildings contribute to lowering their environmental impact?*

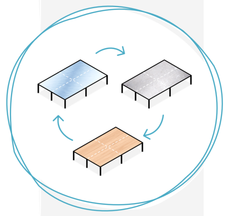
DESIGN PROCESS

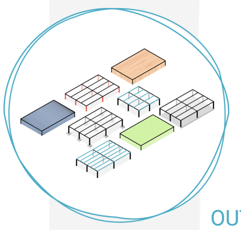
Chapter 4 – Design step 1

(2) How much environmental impact reduction can be achieved by substituting steel or concrete by timber in the load-bearing frame of a warehouse?

- *What type of structural system is appropriate for a baseline timber warehouse?*
- *What is the environmental impact of steel and concrete warehouse structures?*

OUTPUT: Baseline timber designs and their environmental impact compared to steel and concrete reference warehouses





Chapter 5 – Design step 2

(3) What design strategies should be considered in priority to effectively reduce the environmental impact of a green enveloped biobased warehouse?

- *Which structural elements of the baseline timber warehouse have the most influence on its total environmental impact?*
- *How much environmental impact reduction can be achieved by choosing different design options for the structural elements of a timber warehouse, compared to the baseline design?*
- *When does a green envelope contribute to reducing the environmental impact of a warehouse?*

OUTPUT: Comparison of design strategies reducing the impact of a timber warehouse

DISCUSSING THE RESULTS

Chapter 6 – Conclusions and recommendations

<u>Main research question</u> – How to reduce the environmental impact score of reference concrete and steel distribution centre designs by at least half?

Figure 2.1: Thesis outline

3

Sustainable design strategies

This chapter aims at answering research question *(1) What are the most effective strategies to lower the environmental impact of warehouse structures?* and the corresponding subquestions.

Sustainability in the building sector should be considered from the early structural design phase to most effectively reduce the environmental burden. Quantifying the impact of design decisions from the start is useful to generate and improve low carbon design alternatives. A selection of design strategies oriented towards environmental impact reduction are explored in this chapter, to offer structural engineers an overview of possible options applicable to warehouse structures, related to low carbon and biobased materials for instance. This study also includes global design strategies like circularity or building with nature, as well as precisely identifying which building elements have the most impact in the structural design process.

3.1 Including sustainability at the early design stage

After drawing a clear definition of sustainability, this section aims at answering the following research subquestion: *What are the benefits of including sustainability from an early design stage?*.

3.1.1 Defining sustainability

The term sustainable development was first defined by the Brundtland Commission in 1987, in the United Nations report 'Our Common Future':

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

Sustainability is often associated with low carbon, but environmental impacts cover a much broader area. Not only should be considered global warming of the atmosphere by increasing levels of greenhouse gases, expressed in CO2 equivalents, but also the depletion of natural resources or the toxicity of harmful components released in marine and terrestrial environments, often difficult or costly to remove. A single product may be responsible for a large variety of non sustainable socio-environmental impact, which should be assessed to make sensible design choices. These impacts put a burden on future generations, who may inherit a damaged environment where their needs cannot be met. These needs, mentioned by the Brundtland Commission, are representative of the level of prosperity and refer to the availability of finite resources, a clean environment, social fairness and economic growth (Jonkers, 2020).

In that regard, current projects should aim at limiting negative consequences for future generations. Building activities globally are responsible for a large share of natural resources consumption and emissions of harmful components to the environment. Scarceness of resources inevitably leads to higher prices on the market, whereas pollution of air, water and soil is directly translated into societal costs. Sustainable engineering practices keep social and environmental aspects in mind. This type of thinking often results in higher initial costs, but eventually the long-term costs to society are considerably lower. Savings may also arise in the project compared to traditional practices, if durability of the building is increased or elements are reused during construction and at the end-of-life.

3.1.2 Embodied vs operational emissions

Embodied carbon stands for the resources and energy used during the construction of a building. During the use stage, replacement and maintenance operations require additional resources to maintain the performance of the building, once again associated with additional embodied emissions. Once the building reaches its end-of-life, different embodied emissions scenarios can occur, as the resources initially used for the building elements may still have a potential for reuse (LETI, 2016).

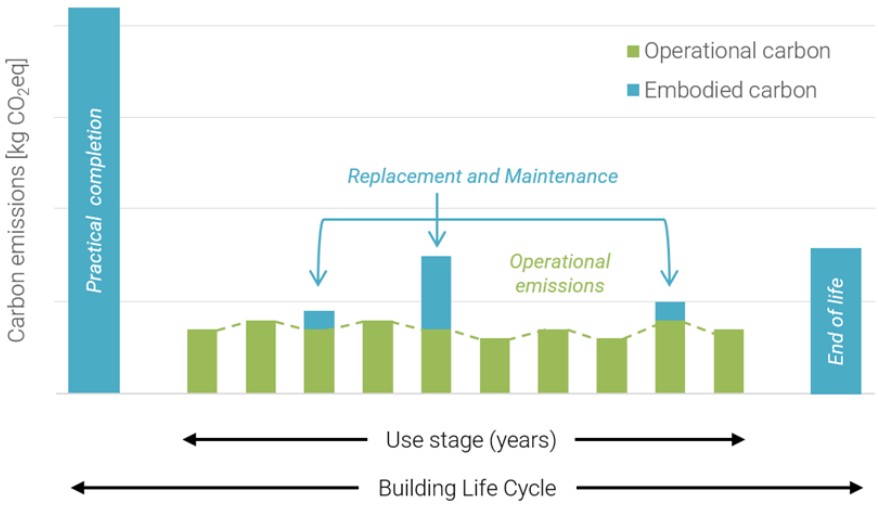


Figure 3.1: Interaction between operation and embodied carbon throughout the lifetime of a building, adapted from (LETI, 2016)

Net Zero Carbon objectives should consider building related emissions in the context of whole life carbon, not only considering the operational emissions during the use stage, but also the embodied carbon of materials and processes over the whole life cycle of the building, as illustrated on Figure 3.1. When buildings are designed for Net Zero Carbon, the first step is generally to target reductions in operational carbon emissions by switching to renewable energy sources and improving the energy performance of new and existing buildings through deep renovation, to live up to climate change targets. As buildings are becoming more energy efficient and energy sources more decarbonised, the fraction of operational carbon in whole life carbon has significantly reduced for new buildings. In this case, embodied impacts related to building materials represent the highest portion of the remaining environmental footprint, as they end up accounting for up to 40-70% of whole life carbon.

3.1.3 Carbon reduction potential in early design stages

Sustainable construction considers environmental aspects at the core of the process, taking into account impacts throughout the whole life cycle of a building. Traditionally, the design process of a building does not focus on sustainability, but rather on structural performance requirements. The environmental performance of design decisions is currently not evaluated until a later design development stage (Jonkers, 2020). Requirements for the environmental performance of buildings currently being set by official regulations will be toughened in the upcoming years, shedding light on the topic of sustainable construction and compelling designers to include embodied carbon reduction strategies in all stages of the design process.

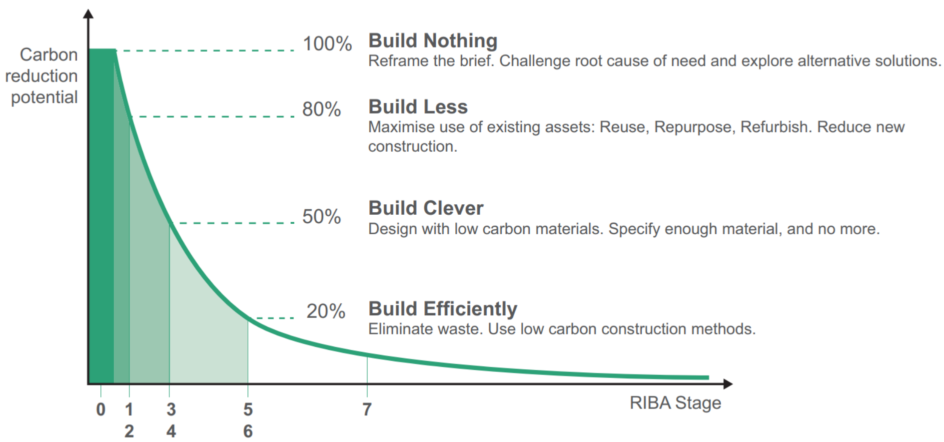


Figure 3.2: Embodied carbon reduction potential at different stages of a building project (IStructE, 2021)

As illustrated on Figure 3.2, the magnitude of embodied carbon reduction potential decreases as project stages go on. According to this graph, as much as 50% of the total embodied carbon emissions of a project could be avoided by including sustainable measures from the design stage by ”building clever”, for instance minimising material use and selecting low carbon alternatives.

The potential for embodied carbon reduction is the greatest for decisions made in early design, with only minor changes at later stages. Indeed, the design is still flexible as the main features are not well defined yet, and the costs associated with changes in design remain relatively low. The objectives towards sustainability and refurbishing opportunities should be discussed with the client at the very beginning of the project brief to define embodied carbon reduction targets early on. The building design process is usually highly fragmented with teams working on the architecture, services, building envelope or safety, among others. Material and dimensioning specifications therefore come at a later stage and reducing the building's environmental footprint becomes more complicated if designers do not understand the impact of their decisions from the start. Always should the whole design team be part of discussions on sustainability, to select low impact options for the entire building (LETI, 2016).

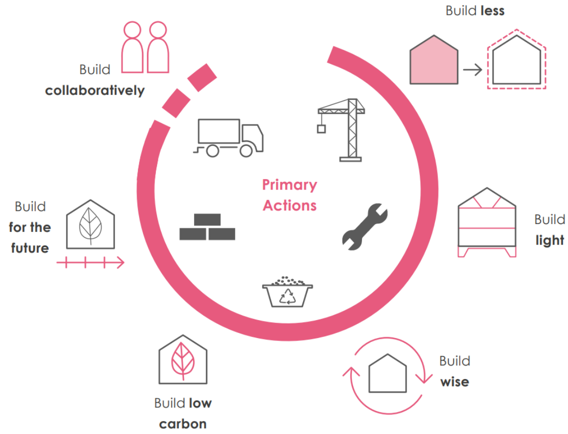


Figure 3.3: Primary actions towards sustainability to be considered in building projects (LETI, 2016)

During the the conceptual design stage, rules of thumb guidance and regular Life Cycle Assessments constitute essential tools for identifying and orienting the project towards low carbon design options. Indeed, performing environmental impact calculations in early design stages gives time and scope for effective changes towards sustainability in light of the assessment. The methodology proposed by (Basbagill et al., 2013) aims at helping designers to identify the most critical decisions for reducing a building's embodied impact, through integrated LCA calculations of design alternatives developed in a BIM environment. Other researchers have worked on integrating LCA methodology within BIM softwares and developing techniques to optimise the environmental impacts of a building during early design. In more advanced stages, carbon reduction targets can be refined and numerical analysis performed to optimise material efficiency (IStructE, 2021).

Identifying elements that contribute the most to the final embodied carbon emissions, considered as environmental hotspots, is key to design a low carbon building. This allows to look for providing the greatest opportunities for embodied carbon reduction right from early stages, the top five usually being piling, foundation, frame, upper floor and envelope. These big ticket items should be considered in priority when refining a building design towards sustainability, over less impactful parts like ceiling and floor finishes, internal walls and external works (LETI, 2016).

3.2 Low carbon building materials

A key aspect of the design process lies in the choice of building material. A sustainable building design should not only have a low environmental impact, but also fulfil defined functional performance and service life requirements. Therefore, material choices should be made based on whether they may present specific characteristics regarding their durability, compressive and tensile strength, maximum span, aesthetics, etc. (Jonkers, 2020)

For the purpose of this research, a comparison is drawn between concrete, steel and timber, considering both their structural and environmental performance. This section specifically aims at answering the following research subquestion: *What is the advantage of using biobased building materials over steel or concrete regarding sustainability?*.

3.2.1 The carbon cycle

Carbon is a chemical element, found in GHG with high global warming potential like carbon dioxide (CO_2), methane (CH_4) or chloro-fluoro-carbons (CFCs). Oceans, rocks, plants, soil, fossil fuels and the atmosphere act as reservoirs of carbon, or so-called "carbon pools". Atmospheric carbon has the strongest influence on global climate, even though it represents only a small fraction of total carbon present on Earth. It is therefore important to control and prevent atmospheric carbon levels from substantially increasing to mitigate global warming (Jonkers, 2020).

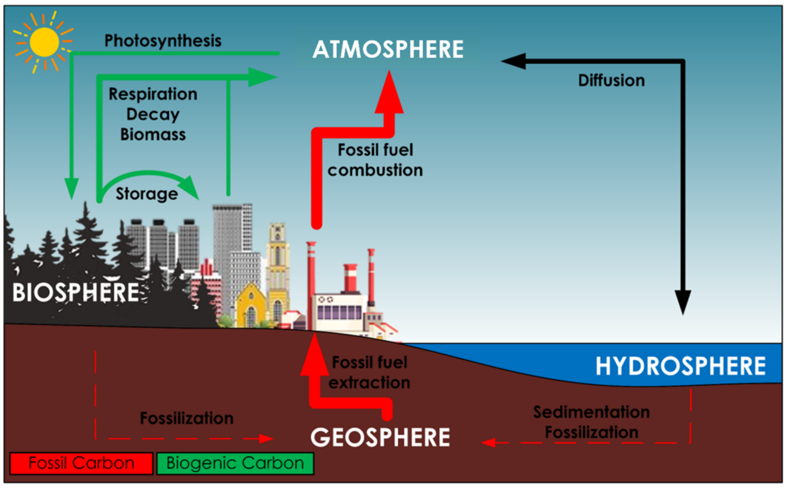


Figure 3.4: Carbon cycle (van Wijnen, 2020)

The CO_2 concentration in the atmosphere is related to the carbon cycle illustrated on Figure 3.4. This process describes carbon fluxes between the atmosphere, biosphere, geosphere and hydrosphere. It can be divided into two categories depending on the timescale across which carbon exchanges take place between reservoirs (Riebeek, 2011).

The **slow carbon cycle** takes place during millions of years between rocks, soil, oceans and the atmosphere. As rocks dissolve with acid rain from atmospheric carbon, ions are carried by rivers to the ocean to participate in the calcification of marine organisms. The calcium carbonate from shells of dead organisms sinks in deep oceans, eventually turning into sedimentary rocks. Other carbon-storing sedimentary rocks are formed from the decomposition of living organisms like plants heated and pressurised over long periods of time. Fossil fuels like oil, coal or natural gas form from layers of organic carbon when decay of organic matter is slower than their growth. Carbon from rocks returns to the atmosphere through volcanoes, naturally balancing and regulating the long-term biogeochemical cycle.

The **fast carbon cycle** is linked to carbon exchanges with the biosphere, mostly plants and phytoplankton in oceans. Biogenic carbon is fixed in biomass by photosynthesis, a chemical process fuelled by energy from the sun turning carbon and water into oxygen and sugar molecules:

$$6CO2 + 6H_2O + \text{Solar energy} \rightarrow C_6H_{12}O_6 + 6O_2 \tag{3.1}$$

At the end-of-life, the sequestered carbon is released back to the atmosphere following the reverse chemical process due to respiration, decay or burning of the material.

$$C_6H_{12}O_6 + 6O_2 \rightarrow 6CO_2 + 6H_2O + \text{Heat} \tag{3.2}$$

The carbon cycle naturally tends to naturally regulate itself to maintain steady concentrations of carbon in all reservoirs, but anthropogenic emissions are altering the balance. By extracting and burning fossil fuels, carbon locked up in the ground over millions of years is released over a very short period of time. Fossil emissions dramatically increased since the industrial revolution resulting in significant rise of carbon dioxide concentrations in the atmosphere, responsible for global warming.

3.2.2 Fossil-based materials

Steel and concrete largely govern the industry worldwide, accounting for 15% of global man-made emissions (van Ruijven et al., 2016). However, their high embodied carbon levels are a disadvantage compared to biobased building materials. Low carbon alternatives for concrete and steel often use recycled fractions to lower the share of virgin materials and reduce fossil emissions associated with their production.

3.2.2.1 Concrete

Concrete is the most used construction material around the world, as its high strength, flexibility and endurance make it suitable for a large range of applications in a building like foundations, floors, walls and frames. Its high thermal mass allows the material to store heat during daytime and release it at night, an efficient way to reduce the energy consumption of concrete buildings (LETI, 2016). Concrete is typically made from Ordinary

Portland Cement (OPC) binder, produced from cement clinker and gypsum, combined with aggregates. The production of Portland cement clinker requires heating limestone and other minerals, raw materials that are widely available, making concrete an accessible building material. However, the chemical reaction occurring this process releases roughly 1 ton of carbon dioxide per ton of cement produced, making it responsible for most of the concrete environmental footprint (Lehne & Preston, 2018).

Concrete mix designs are important to reach specific compressive and tensile strength, and durability, to comply with project-specific performance requirements. The main objective of sustainable concrete is reducing the Portland cement content, while ensuring sufficient strength of the concrete. Fine powdered fillers, called supplementary cementitious materials, are used in blended cements to replace a fraction of the ground clinker. They usually consist of by-products considered as waste materials by other industries, like fly ash from power plants, blast furnace slag from steel production, or silica fume. Limestone fillers are more and more used as substitutes for cement, as a cheaper mineral alternative often more available locally. Depending on their nature, they may alter different mechanical, physical or chemical properties of the concrete and their effects should therefore be investigated before use to determine if code requirements are met and optimise the mix design (Lehne & Preston, 2018). Demolition waste retrieved from urban mining can also be used to replace virgin materials, either cement or aggregates, to lower the impact of concrete while offering a new useful life to existing materials, encouraging circular building processes (LETI, 2016).

3.2.2.2 Steel

The construction sector uses half of all steel products in the world, in a wide range of applications. The first production method for steel is the Basic Oxgen Furnace (BOF), based on fossil fuels, which uses high proportions of virgin iron ore compared to scrap metal. The second method is the Electric Arc Furnace (EAF), usually results in lower embodied carbon of the material as it is powered by the electricity grid and can produce steel with high recycled content. This method takes advantage of the high recyclability potential of steel, which contributes to reducing raw material use by partly or completely replacing them with existing steel harvested from demolished buildings. The use of one production route or the other depending on the type of structural steel product, as steel plates and closed sections are typically produced by BOF. The amount of processing required to manufacture specific steel elements will directly influence the embodied energy of the building component (IStructE, 2021).

3.2.3 Biobased materials

Biobased materials are increasingly used because they are renewable and store biogenic carbon. Wood engineered products are also capable of delivering similar structural performance as expected for steel or concrete. As concerns rise about the topic of sustainability in the construction sector, biobased building materials are increasingly competitive on the market compared to traditional fossil-based materials (Campbell, 2019).

3.2.3.1 Advantages of wood as a structural material

Van Wijnen (2020) identified a number of advantages timber as a structural material, illustrated on Figure 3.5.

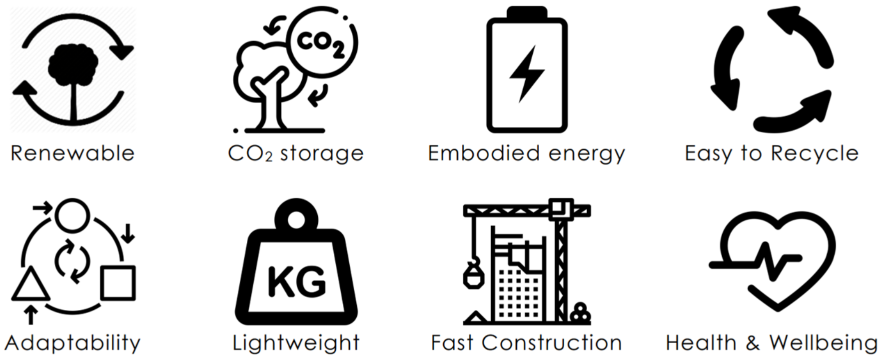


Figure 3.5: Advantages of timber structures (van Wijnen, 2020)

Renewable

Wood harvested from sustainably managed forests can be considered fully renewable, in accordance with circularity principles. Forests participate in the carbon cycle, removing carbon dioxide from the atmosphere as trees grow by photosynthesis. Around 30% of global land is forested, but these carbon pools are threatened by land use changes and deforestation, ultimately depleting the overall absorbing capacity of woodlands. Selecting timber produced from sustainably managed forest sources ensures that the harvested wood used to manufacture the building elements is effectively replaced by at least the same number of newly planted trees, to maintain carbon storage levels in the long term (LETI, 2016). Carbon sequestration rates in forests depend on the tree species, maturity, local climate, soil conditions, and forest management method. The sequestration rate is higher in the first years of a tree's growth. Therefore, the more carbon can be sequestrated from forests with shorter rotation harvests, felling trees as they reach maturity and replanting new ones immediately. Forests can also act as carbon sinks when more trees are planted than felled, enlarging the total timber stock.

Figure 3.6: Forest Stewardship Council (left) and Programme for the Endorsement of Forest Certification (right) logos – The two most well-known certificates for sustainable forestry

The increasing demand for sustainable biobased building products supports the need for appropriate forestry and harvesting practices, essential to ensure long-term health and diversity of forests. Forest certification programs guarantee not only sustainable harvesting of trees, but also respect of biodiversity, habitat protection and indigenous peoples' rights (Falk, 2010). The most well-known sustainable forestry certificates are the label of the Forest Stewardship Council (www.fsc.org) and PEFC (www.pefc.de).

Carbon storage

Wood is produced by photosynthesis, therefore biogenic carbon is naturally locked up in the material as trees grow, and then stored until wood is burnt or deteriorates. Using harvested timber for construction products extends the duration of carbon storage in the material and thus contributes to delaying re-emission to the atmosphere (LETI, 2016). This whole process highlights the benefits of planting new trees and extending the service life of timber products for as long as possible, to fully make use of the biogenic carbon storage capacity of the material.

Embodied energy

The embodied footprint of timber related to extraction and manufacturing processes remains relatively low compared to similar operations for steel and concrete which require more energy. Relative to the complexity of their processing, engineered wood products have higher levels of fossil embodied carbon than solid sawn timber, but still lower than non-biobased alternatives. Indeed, if steel or concrete are generally produced by burning fossil fuels, the primary energy source for timber is sunlight and manufacturing processes often use bioenergy from wood by-products, considered carbon neutral. In the end, there is a large gap in energy consumption between steel, concrete and timber as building materials (Falk, 2010).

Easy to recycle

Timber can be reused, broken down into smaller pieces to manufacture new products, or turned into biomass fuel.

Adaptability

Almost any desired shape can be achieved thanks to specific manufacturing techniques, making timber suitable for a large range of applications.

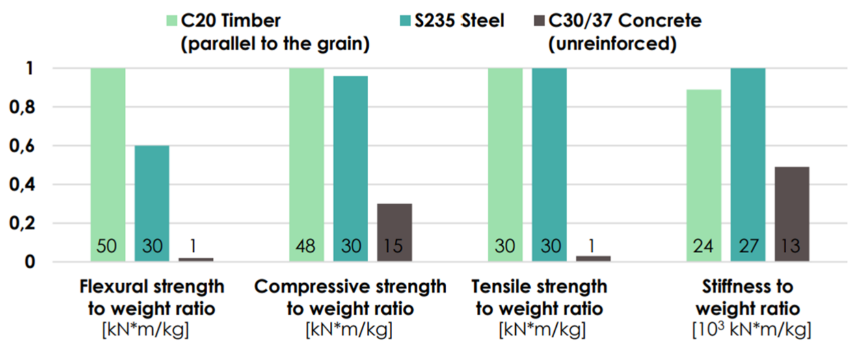


Figure 3.7: Comparison of material properties of timber, steel and concrete (van Wijnen, 2020)

Lightweight

Timber has low density and high strength-to-weight ratio (see Figure 3.7). Thanks to these properties, timber structures are generally lightweight and impose lower loads on foundations, reducing the required volume of concrete thus their environmental impact.

Fast construction

Timber structures are built from offsite prefabricated elements, resulting in quick erection on the building site. The light weight and low tolerances of the material also contribute to a fast construction process.

Health & Well-being

Other benefits of timber include the aesthetic natural aspect and indoor humidity regulation, which have been proven to enhance the health and well-being of users.

3.2.3.2 Challenges of building with timber

Structural timber elements should retain sufficient load-bearing capacity and usability for the entire service life of the building. Durability of wood is also relevant to lock up carbon in the material as long as possible and reduce embodied emissions (Campbell, 2019). Organic materials like wood are particularly vulnerable to biological organisms, the two main threats being fungi and insects. Other major challenges include protection against fire and degradation from moisture changes.

Fire safety

The relevance of fire safety for timber buildings was pointed out during catastrophic fires events in major cities like London in 1666 or Chicago in 1871, and it remains a major concern today for mass timber construction (ARUP, 2019). As a combustible material, special care should be given to the protection of timber structural members against fire, either with exposed or encapsulated timber. Exposed timber is architecturally attractive and has a positive impact on the health and wellbeing of the building users. The design process relies on the reduced cross-section method, based on the charring phenomenon: when exposed to fire, timber forms a charring layer behind which undamaged material remains. The charring rate of timber in millimetres per minute is used to predict the degradation of the material, and design for sufficiently strong residual cross-sections. Alternatively, encapsulating timber elements with gypsum boards insulates them against fire, postponing the charring phenomenon by evaporating the water content from the gypsum.

Fire resistant coatings may help reduce the spread of fire, by creating a protective insulating layer around the material or releasing fire retarding substances when heated. Certain types of engineered laminated timber products were shown to exhibit self-extinguishing properties. Ultimately, sprinkler systems can help effectively suppressing a fire when other measures are not sufficient (van Wijnen, 2020).

Biological organisms

Micro-organisms like fungi require free water and oxygen to develop. Fungal attacks are more likely to occur in wood with moisture content above 20%, optimal growth conditions being 30-60% moisture content and temperature between -2.5 and +40°C. Wood-staining fungi like moulds generally remain on the surface and cause discolouration, leaving physical features of the wood intact. Wood-destroying fungi like soft rot or brown rot can cause significant strength loss of timber structures depending on the type of fungus, by breaking down wood cellulose molecules.

Insects tend to proliferate with heat, favouring their reproduction and development. The most dangerous wood-boring insects are termites: mainly located in warmer areas, they tend to settle in external cracked layers of treated wood to lay their eggs. For beetles as well, the most damaging phenomenon is the burrowing of larvae causing cross-section reduction and strength loss.

Regulating the moisture content is important to prevent fungal attack, by avoiding warm environments, integrating waterproof layers and allowing wood to dry out with effective ventilation and water drainage systems. Heartwood is generally more resistant to pests than sapwood. Against both fungi and insects, it is recommended to select wood species with sufficient natural durability for a given application, according to durability classes defined in standard EN 350-2:1994 (Natural durability of solid wood). Other preservation measures include adequate detailing, such as structural barriers shielding the wood from the ground level or covering wood sections (Blaß & Sandhaas, 2017).

Moisture content

The hygroscopic behaviour of wood is directly related chemical composition, and influences its physical properties: increasing moisture content decreases the stiffness and strength, while increasing creep deformations, thermal conductivity and vulnerability to pests. Moisture-related issues in timber structures may be caused by liquid water infiltration or high moisture content in the air, aggravated by poor ventilation, warm environments or contact of wood with the soil. Cracks are more likely to occur in wood exposed to significant variations in weathering, and may open the way for further damage by ingress of water, fungal spores, or insects laying eggs within internal layers.

As a general rule, timber elements should be installed at the building's equilibrium moisture content, ideally comprised between 8-20% and limiting changes to seasonal variations only. Appropriate detailing should be done for parts susceptible to condensation and water ingress, with proper monitoring after construction. Watertight vapour barriers and heat insulation are essential to limit condensation, and particular attention should be given to walls and connections where water can accumulate. At the construction site and during assembly, exposed end grain surfaces should be covered to prevent water ingress by capillary effect (Blaß & Sandhaas, 2017).

Corrosion of metal fasteners

In timber structures, the durability of connections is also important for maintaining structural performance for the entire service life. Corrosion of metallic parts can affect the long-term behaviour of fasteners and cause discoloration of wood members, if the metal is not protected with paint or coatings. Examples of corrosion protection measures are given in Eurocode 5 for various service classes (Blaß & Sandhaas, 2017).

Wood preservatives

Paint and coatings protect wood against rain, sunlight or mechanical stresses by creating a physical barrier. Chemical wood preservatives are often criticised, but can be avoided if detailing of timber members and the choice of wood species are well suited for a given application. Modification processes like thermal treatments can be used to achieve higher durability, to make wood less prone to shrinking and swelling, and enhance dimensional stability (Blaß & Sandhaas, 2017).

3.2.3.3 Wood growth and material properties

Understanding the anatomy of wood is key to understand the mechanical and physical properties of timber as a structural material and use it to its full potential. Material properties depend on the scale at which it is looked at. The meso/macrostructural scale can be used to look at the properties of structural timber by considering the tree itself. It comprises knots and growth-related properties of the tree, all visible features in the cross-section of a trunk. The microstructural scale is composed of wood fibres, cells, and the chemical structure of the material.

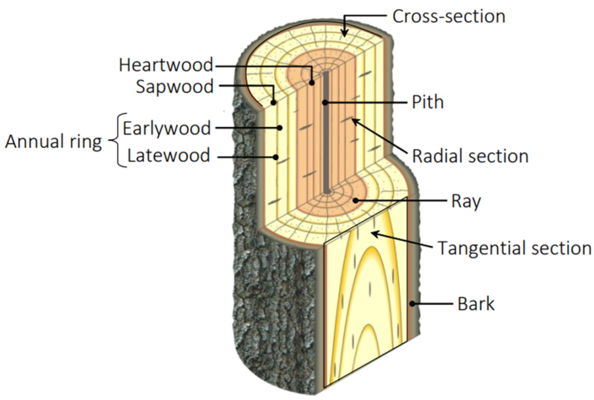


Figure 3.8: Macroscopic structure and section planes of wood (Blaß & Sandhaas, 2017)

Macrostructure

The core of a trunk, or pith, is made of juvenile wood formed early in the life of the tree. It generally exhibits lower quality than wood that was formed later. Heartwood is the centre part of the stem, composed of dead wood cells stabilising and strengthening the tree. Often more durable than sapwood, it is characterised by high levels of extractives. Sapwood is located in the lighter outer part of the stem surrounding the heartwood and contains all living wood cells, storing and transporting water and minerals from the roots to the crown. It also provides mechanical strength as it contains lignified cells.

The bark (or phloem) of the tree ensures the transportation of nutrients from the leaves to the storage organs and growing parts of the plant such as the cambium, a thin layer of dividable living cells between the inner bark and the wood (or xylem). Rays, like the pith, contain parenchyma cells for the transport of water and nutrients from the centre of the stem to the outside. Resin canals run parallel and perpendicular to the stem, protecting some tree species against wound or parasites.

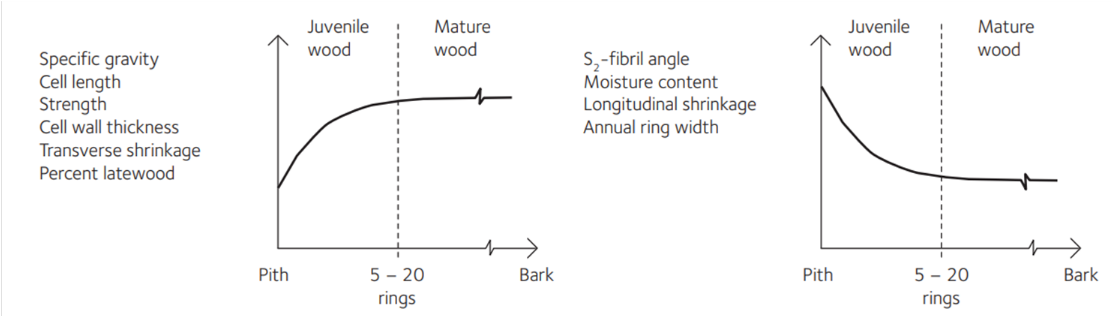


Figure 3.9: Juvenile to mature wood transition of properties (Johansson, 2016)

While a tree grows longitudinally, annual rings of varying density, colour and hardness observed in the cross-section of the stem indicate secondary growth in girth within the cambium. Early wood is formed from spring, when the water supply and temperature conditions require large pore volume and thin-walled cells for effective water transport in softwoods, and large vessels in hardwoods. Latewood appears darker, characterised by thick-walled cells in softwood and smaller vessels in hardwoods, to ensure strength of the tree as the importance of water supply decreases in autumn.

Knots are produced the location of branches causing local deviations in the grain. The size and position of knots in a timber cross-section can have detrimental effects on the strength of a structural element, hence why visual grading is important to sort specimens according to defined strength class. Reaction wood also causes unwanted deviations when a tree was forced out of its original position and attempts to go back, creating zones of compression or tension wood (Blaß & Sandhaas, 2017).

Microstructure

The microstructure of wood is dependent on the tree species, divided into two main groups differing in their growth patterns, leaf shapes and microstructures: hardwoods and softwoods. Hardwoods are deciduous trees, the most commonly encountered being oak. Hardwood comprises large water-conducting vessels or pores, and smaller tracheids providing strength. Rays are often clearly visible and may comprise multiple rows of cells transporting nutrients and water in the radial direction. Softwoods are conifers, the number one species used for construction in Europe being spruce, followed by fir. In softwood, tracheids are long and thin cells responsible for water transport in the longitudinal and perpendicular directions. Longitudinal tracheids make up more than 90% of stems, while the remaining fraction is composed of ray and longitudinal parenchyma cells, and resin canals. Rays, like the pith, contain parenchyma cells for the transport of water and nutrients from the centre of the stem to the outside. Resin canals run parallel and perpendicular to the stem, protecting some tree species against wound or parasites.

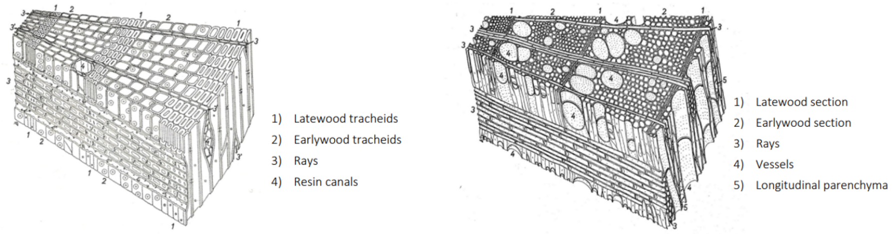


Figure 3.10: Schematic structure and cell types of softwood (left) and hardwood (right), Nardi-Berti, 1993 cited by (Blaß & Sandhaas, 2017)

The anisotropic structure of wood is due to the elongated structure of its cells, organised in bundles with cell walls oriented in the direction of the stem. Anisotropy of the material is also influenced by the orientation and distribution of ray cells, and the size of cells. The properties of wood as a structural material are mainly influenced by the fine cell wall structure, the collection of cells in clear wood (flawless wood without fibre deviation) and growth irregularities in timber.

Chemical composition

Wood is a natural organic material, formed through the process photosynthesis transforming atmospheric carbon and water into biomass. The main organic compounds contained in the material are carbon and oxygen, followed by hydrogen and smaller fractions or nitrogen and minerals. Together, they form chains and macro molecules like cellulose, hemicellulose, lignin and other chemical constituents of wood cells, each contributing to certain properties of the material.

Cellulose, contained in the fibrils of cell walls, is a chain molecule with very high tensile strength which gives its axial strength to the cell. Hemicelluloses, other components of wood cells, control the permeability of the cell membrane to absord and discharge water. Lignin is responsible for the compressive strength of the cell, as a fixed and rigid compound of micro-fibrils. While some molecules like cellulose and hemicellulose are highly hydrophilic, lignin on the other hand is hydrophobic and protects the cell wall against any ingress of water (Blaß & Sandhaas, 2017).

3.2.3.4 Wood products

After logging, trees may be sawn and directly used as structural elements. However, because of the growing process of trees, wood as a material exhibits non-homogeneous strength properties which may reveal problematic for structural applications. To achieve higher load-bearing capacity, smaller wood pieces can be processed and rearranged using adhesives to manufacture large elements exhibiting a lower variance in material properties. This results in engineered wood products or wood-based panels. The most common wood products used for structural applications are presented in this section.

Figure 3.11: Types of timber engineered products according to the wood component they are produced from, based on (Blaß & Sandhaas, 2017) and (Sandberg, 2016)

An overview of common wood products is shown on Figure 3.11. Raw materials initially come from logs, divided into a variety of shapes and sizes, in the form of sawn timber, veneers, strands, particles or fibres. Veneers are produced by rotary peeling of wet log portions, resulting in 0.5-6mm thin sheets of wood. Smaller chips and sawdust are recovered from sawmill waste or recycled wood. These wood parts are reassembled to produce wood-based panels or bigger structural elements, going beyond the limited dimensions allowed by natural timber, and exhibiting different levels of homogenisation depending on the size and rearranging of the raw components. Wood-based panels in particular benefit from approximated isotropy in plane compared to solid timber elements (Blaß & Sandhaas, 2017).

Sawn timber

Sawn timber elements may be divided into groups based on standardised cross-section dimensions, namely squared timber, planks, boards and battens. Normal sawn timber can be found only up to certain dimensions, due to the size of the trees and the industrial process. For larger dimensions it is necessary to use Engineered Wood Products.

Figure 3.12: Sawn timber

After cutting, sawn timber undergoes visual pre-grading, and is then sent to dry in kiln or outdoors to remove excess moisture. A final visual or machine grading process is carried out to assign the element a strength class as described in EN 338. Recommendations from EN 1912 indicate the visual grades and wood species to be assigned with particular strength classes. These are different for softwood and hardwood, but they allow to group a large variety of species according to load-bearing capacities, associated with material properties like characteristic strength, stiffness and density.

Glued laminated timber (glulam)

Glued Laminated Timber (glulam) is made by gluing together layers of timber boards with fibres oriented in the longitudinal direction. By homogenising the properties of wood as a material, glulam exhibits higher strength and stiffness properties than solid timber of similar dimensions, allowing for larger cross-sections and spans.

Figure 3.13: Glued Laminated Timber (glulam)

This wood engineering process, commercialised from 1906 and one of the oldest existing to this day, is used to create straight or curved structural members by finger-jointing individual lamellae. Any wood species may be used, given that appropriate adhesive is chosen (Malo & Angst, 2008). Production requirements and glulam strength classes are specified in EN 14080, but manufacturers also give their own recommendations for use. The principal stages of the manufacturing process of glulam are shown in Figure 3.14.

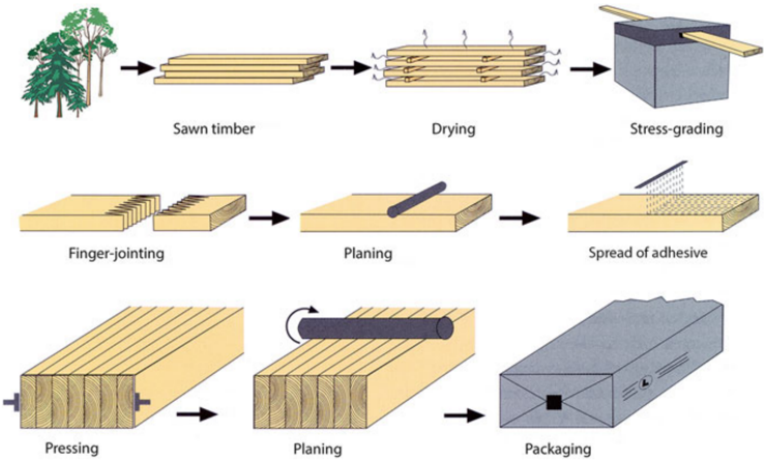


Figure 3.14: Manufacturing process of Glued Laminated Timber (Sandberg, 2016)

Where knots would result in weak points within solid sawn timber elements due to a lower modulus of elasticity, the lamination effect in glulam allows the surrounding lamellae to redistribute some of the load to adjacent stronger parts. Flawed areas can be removed before finger-jointing to increase the homogeneity of the material. The strength of a glulam member is determined by the weakest components between the individual lamellae and the finger joints. Sections of a lamella can be weakened in areas surrounding knots, while failure in a joint often results from poor initial quality. Homogeneous glulam cross-sections are made from lamellae of approximately the same strength. Considering that glulam beams are mainly used as load-bearing elements subject to bending stresses, higher strength lamellae can be put on the outer parts where the highest stresses occur. Such combined glulam cross-sections exist in different configurations, either symmetrical or asymmetrical, with variable number of high strength lamellae, or combining different wood species within a single member (Blaß & Sandhaas, 2017).

In structural design, glulam can be used for a wide range of load-bearing elements, including beams, girders, columns or truss members (Sandberg, 2016). Its high strength-to-weight ratio makes it an interesting alternative to steel when building large spans with a minimum number of intermediate supports (Malo & Angst, 2008). It is also particularly suited for architectural applications based on aesthetic curved structures, or custom cross-section shapes varying along the length (DERIX, 2019).

The most common types of adhesives used in glulam manufacturing and finger jointing of lamellae are synthetic two-component PRF (phenol-resorcinol-formaldehyde), but MUF (melamineurea-formaldehyde) types have also been gaining popularity recently. New adhesives are continually being developed as they represent a major challenge for the durability and sustainability of glued engineered wood products (Sandberg, 2016).

Cross laminated timber (CLT)

Cross Laminated Timber (CLT) is very similar to glulam, except that timber lamellae are glued crosswise to achieve higher homogeneity in material properties and strength in both directions, hence the name of the material.

Figure 3.15: Cross Laminated Timber (CLT)

As lamellae will be loaded both perpendicular and parallel to the grain, this layout provides higher in-plane isotropy, therefore good mechanical properties in bending and shear as well as better dimensional stability by reducing swelling and shrinkage effects. Due the different modulus of elasticity of wood parallel and perpendicular to the grain, the stiffness of the material is mainly determined by layers loaded parallel to their grain. Because of its ability to transfer loads in two directions, CLT is always used for load-bearing components, essentially panels for walls, floors and roofs. Typical elements are made from an uneven number of layers, generally assembling 15 to 40 mm thick boards to produce symmetric cross-sections of variable thicknesses. CLT is most often made from spruce, but other wood species like fir, pine, larch or Douglas fir can be used as well. Production requirements are specified in EN 16351 (Blaß & Sandhaas, 2017).

Plywood

Plywood is also made from glued wood layers, generally crosswise. Top layers are made of veneers, while central plies may consist of veneers, wooden strips or thinner lamellae.

Figure 3.16: Plywood

Plywood panels are typically used as sheathing in vertical and horizontal diaphragms, namely for floors, roofs and shear walls. Their mechanical properties depend on the number, thickness, arrangement or wood species of plies composing the board, all being symmetrical to the central plane. Plywood products are regulated by EN 13986 and EN 636, grouped in technical classes and in terms of bending strength and modulus (Blaß & Sandhaas, 2017).

Laminated veneer lumber (LVL)

Laminated Veneer Lumber (LVL) is made from veneer sheets and adhesives assembled under higher pressure and temperature, mostly with fibres in the longitudinal direction.

Figure 3.17: Laminated Veneer Lumber (LVL)

Typically made from spruce or pine, it is used for structural elements like beams, columns, wall panels or hollow box floor system. Layers are much thinner than in glulam, resulting in increased homogeneity of the material (Blaß & Sandhaas, 2017). Kerto is a special wood product derived from the principle of LVL, produced from 3mm thick veneer sheets. All are arranged with the grain in the same direction for Kerto-S, while 20% are laid cross-ways in Kerto-Q, to achieve different material properties (MetsäWood, 2016). Requirements concerning LVL are laid out in EN 14279 and EN 14374.

Strand based panels

Figure 3.18: Oriented Strand Board (OSB), Parallel Strand Lumber (PSL) and Laminated Strand Lumber (LSL)

Oriented Strand Board (OSB) is made from longitudinal wood strands from thin diameter logs of approximate dimensions 0.6x75x35mm, arranged in single or multiple layers and bonded together with adhesives under heat and pressure. Outer layers are oriented parallel to the production direction, while inner layers strands are perpendicular or randomly oriented. The orientation of strands influences in-plane properties of the material, as the bending strength is significantly higher in the longitudinal direction of the panel than in the orthogonal direction (Blaß & Sandhaas, 2017). OSB is the most common wood-based panels in structural applications. It is often used as sheathing material in walls or floors. Larger panels up to 3m width, 25m length and 75mm thickness are also used as structural elements, similar to CLT panels (Crocetti & Mårtensson, 2016). This type of products is regulated by European standards EN 13986 and EN 300.

Parallel Strand Lumber (PSL) is made from approximately 3mm thick waste veneer sheets, cut into strips up to 2.5m long and 23mm wide. These recycled strands are organised in the longitudinal direction, bonded and pressed to create beams of unlimited lengths. This process achieves high strength, stiffness and dimensional tolerance. They are not regulated by any products standards, but generally undergo visual and weight checks before commercialisation. PSL may be used to produce a variety of structural members such as beams, columns, purlins and trusses.

Laminated Strand Lumber (LSL) is made from wood strands of approximately 0.8x25x300mm bonded together to produce dense elements. Depending on the manufacturing process, varying strength class can be achieved by modulating the amount of trim wood strips in the perpendicular or parallel board direction (Blaß & Sandhaas, 2017).

Particle or fibre boards

To manufacture particle boards, small wood chips of different sizes are spread out and sprayed with adhesives, before hot pressing them into panels. The ratio of adhesive over wood is higher as the particle size decreases (Crocetti & Mårtensson, 2016).

Figure 3.19: Particle boards

Particle boards are mainly used as floor and roof elements, as well as structural wall sheathing. The orientation of the spread chips at the beginning has an influence on the properties of the end material: in-plane tensile and compressive strength are achieved by orienting them parallel to the panel plane, while bending strength is improved when density of the material is higher in the external layers. These products are regulated by

EN 13986 and EN 312 (Blaß & Sandhaas, 2017).

Fibre boards are composed of individual or bundled fibres, and present quasi-isotropic in-plane properties. The manufacturing process is similar as particle boards, but wet or dry production methods influence the end ratio of adhesive over wood in the panel. The Eurocodes regulating fibre boards are EN 13986, EN 316 and EN 622. Some fibre boards called masonite make use of lignin present within the wood as a binder and achieve sufficient strength to be used in structural applications (Crocetti & Mårtensson, 2016).

Built-up structural elements

Structural timber elements may be made from solid sections of wood engineered products or composite built-up sections made of one or more types of material, among those previously described.

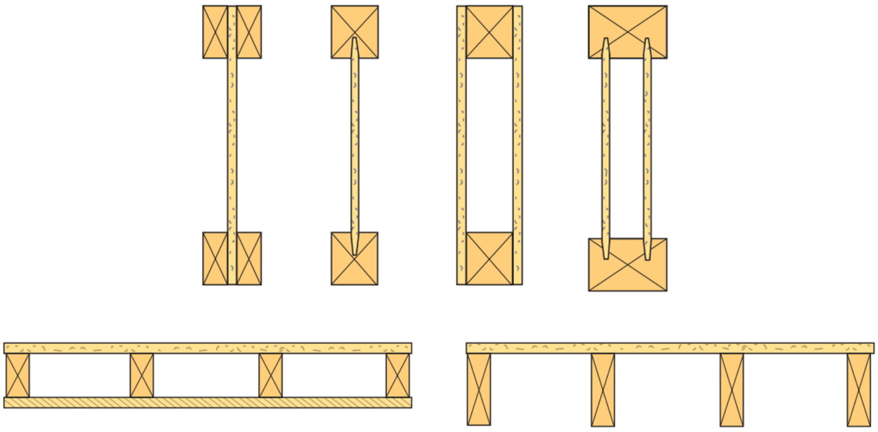


Figure 3.20: Typical cross-sections of thin webbed beams and stressed skin panels: I-sections and box-sections (top), H and T sections (bottom) (Norlin, 2016)

Composite timber elements can be glued or mechanically jointed to create various cross-section shapes, according to project requirements. Stressed skin panels belong to the category of thin-flanged elements. Box beams like Lignatur elements, and I-joists are considered thin-webbed beams. Webs absorb shear stresses, while flanges take over stresses from axial forces and bending moments, a distinction which can hardly be made for T-shaped beams. The webs are typically made with board material like plywood, OSB or masonite, and glued to flanges of finger jointed sawn timber, glulam or LVL. During the manufacturing of I-beams, flanges can be notched or cut into two separate pieces to assemble with the web (Norlin, 2016). Examples of I-sections are finnjoists, manufactured in the UK with Kerto flanges and OSB web. This type of product exhibits high strength-to-weight ratio and enables spans up to 14m. They are typically used in floor constructions to build light frames, quick and easy to install (MetsäWood, 2016).

Overview of options for timber structural members

Wood engineered products achieve higher structural performance than solid sawn timber as they exhibit more homogeneous properties across the materials. Glued laminated products like glulam, CLT and LVL are particularly well suited for large structural applications, to create strong columns, beams spanning long distances or plane structural elements like walls, floors and roof plates. These materials are compared in Table 3.1. Overall, glulam appears as the most versatile material regarding the shape of members that can be manufactured and the structural dimensions allowed even for sustaining high loads.

Table 3.1: Comparison of solid sawn timber and common laminate wood products for structural applications, adapted from (Porteous & Kermani, 2013)

Material	Solid sawn timber	Glulam	CLT	LVL
Picture				
Production	Cutting logs from a tree stem	Gluing planks in longitudinal direction	Gluing planks in crosswise direction	Gluing veneers in longitudinal or crosswise direction
Application	Low-rise timber frame structures, short span beams	Beams, columns, trusses, portal frames Suitable for long spans and high loading	Structural floors, walls and roof plates	Beams, columns, I-joists flanges – *Less often structural floors, walls and roof plates*
Common sizes	Width: 60-300mm Thickness: 12-145mm Length: 5m max, up to 16m with finger jointing	No theoretical limits to size, length or shape	Width: up to 4.8m Thickness: 50-500mm Length: 24m maximum	Width: 19-90mm Thickness: 200-600mm, up to 2.5m Length: 24m maximum

3.2.3.5 Other biobased materials

Fibre-reinforced wood products use fabric, glass or carbon fibres to strengthen structural members of glulam or plywood. Mineral bonded wood-based boards may be used as sheating material in timber frame constructions. These panels are made from wood particles or fibres mixed with mineral materials like cement or gypsum, offering additional protection against fire hazards. Wood wool boards may be used as sound and thermal insulation (Blaß & Sandhaas, 2017).

Apart from wood-based structural elements, insulation technologies increasingly make use of biobased insulating materials. Among others, cork, hemp, straw and wood fiber are commonly used in the building envelope of new or renovated constructions, inside roof or exterior wall elements. These four types of bio-based materials are widely available on the European market (Göswein et al., 2021). Other examples of common biobased regrowable materials include flax, cellulose, coconut fiber or wool used for insulation, and reed used as roofing material (Jonkers, 2020).

3.2.4 Biogenic carbon in LCA

Substituting fossil-based building materials by biobased products is an important strategy to mitigate the impact of the construction sector on climate change, replacing CO_2-intensive materials by alternatives which actually store carbon away from the atmosphere. However, the environmental impact score of a building does not always reflect the benefits of using biobased materials regarding sustainability, depending on how biogenic carbon is considered in calculations. On the global scale, the benefits of biobased materials for carbon storage may only be mentioned under the assumption of sustainable forest management, and actions encouraging active reforestation.

3.2.4.1 Static calculation methods

The general methodology for Life Cycle Assessment is detailed in appendix F. In current LCA biogenic carbon shall be reported separately, resulting in two embodied carbon figures, including and excluding carbon sequestration (LETI, 2016). Two main assessment methods are used to account for biogenic carbon storage in biobased material, within the traditional LCA framework for buildings, namely the 0/0 and -1/+1 approaches (Hoxha et al., 2020b). The main challenge for assessing carbon sequestration is the transparency of assumptions, to obtain fair and comparable results. Both of these static approaches rely on the assumption of timber being carbon neutral over the service life of the building: wood captures carbon as trees grow, carbon is stored in timber products for the duration of their service life, and it is emitted back once burnt or degraded.

The 0/0 approach, illustrated on Figure 3.21, considers complete carbon neutrality of biobased products over the lifecycle of the building system: carbon release at the end-of-life of a biobased product is assumed balanced by carbon uptake during the biomass growth, and biogenic CO2 is therefore not considered in any module of the LCA.

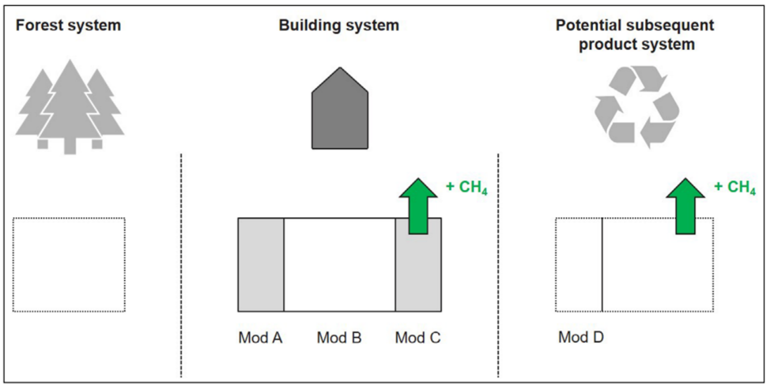


Figure 3.21: 0/0 approach for biogenic carbon uptake and release in LCA (Hoxha et al., 2020b)

The -1/+1 approach, illustrated on Figure 3.22, is different from the 0/0 method as it tracks all flows of biogenic carbon across the lifecycle of the building. Biogenic carbon uptake during the biomass growth is this time reported as a negative emission in module A, to translate the biogenic carbon transfer from the forest system to the building. Similarly, when the building reaches its end-of-life, the sequestered carbon is modelled as a transfer from module C to module D to account for different end-of-life scenarios, which results in a positive emission to be reported in module C. At the end, the biogenic carbon balance is still zero for the building system over its entire service life.

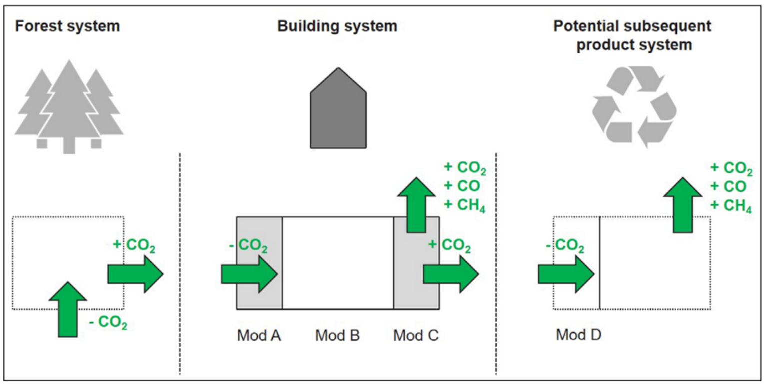


Figure 3.22: -1/+1 approach for biogenic carbon uptake and release in LCA (Hoxha et al., 2020b)

3.2.4.2 Dynamic calculation methods

Dynamic LCA calculations, illustrated on Figure 3.23, account for time delays due to forest growth, and usually give the most robust and transparent results. They either consider trees to grow and sequester before the use of harvested wood products in a building system, or that trees regrow after the production process of biobased products.

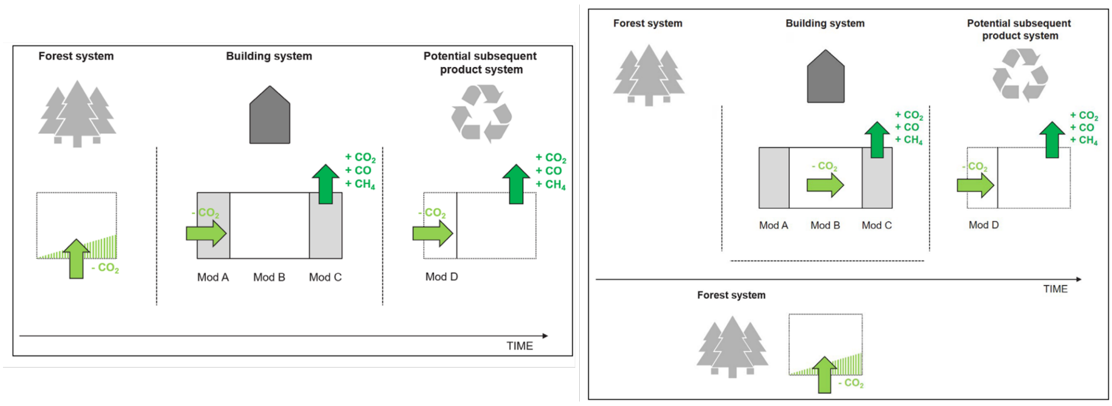


Figure 3.23: Dynamic approaches for biogenic carbon uptake and release in LCA (Hoxha et al., 2020b)

3.2.5 Circularity strategies for timber structures

This section answers the following research subquestion: *What circular building strategies can be implemented to reduce the impact of a warehouse structure?*

3.2.5.1 Discussion on temporary carbon storage

The -1/+1 approach described in the previous section may lead to negative results for the global warming when limiting the boundaries of an LCA to module A only. This could be misleading if the carbon release at the end-of-life of biobased products is completely overlooked (Hoxha et al., 2020b). However, this view is closely related to the idea of "temporary" carbon storage in biobased products. Indeed, the basic assumption for these calculation methodologies are that biobased products will store biogenic carbon for the duration of their service life, and then release it to the atmosphere when burnt or degraded. The goal is therefore to keep these products in use as long as possible to keep carbon out of the atmosphere, reusing timber elements or recycling them into strand boards to make new products, instead of immediately burning them at the end of a building's life. Circularity goals for 2050 support the reuse of biobased products and therefore the extension of carbon storage over considerably long periods of time. In this case, is it still appropriate to talk about only "temporary" carbon storage in timber, or could it be considered permanent? Then, negative results in the production stage from the -1/+1 approach would rightly translate long-term carbon storage in biobased materials with a high certainty of being reused at their end-of-life. Moreover, in the context of climate emergency even temporary storage contributed to reducing carbon levels in the atmosphere immediately, which is adequately shown in a partial -1/+1 approach (Hoxha et al., 2020a).

3.2.5.2 Extending carbon storage with circular strategies

Overall, wood products are used in a sustainable way when their design lifespan is at least equal to forest rotation periods, ranging from 35 to 75 years depending on wood species and location. This corresponds to the sustainable yield logging concept, which refers to fellings not exceeding timber growth during a certain period, resulting in positive net growth of the forest timber stock and associated sequestered carbon.

Implementing circular strategies in mass timber construction appears as a necessity to increase the service life of timber products (Campbell, 2019). The cascade use principle established by the European Parliament proposes an order of priority for timber end-of-life scenarios. The preferred scenario is to reuse wood elements in new projects, as such or re-sized, to increase the duration of biogenic carbon storage. Otherwise, the material may be recycled into new wood-based products to extend its life (Ramage et al., 2017). In timber structures, reversible mechanical fixings like screws, or interlocking solutions are preferable to permanent fixings like nails and adhesives when it comes to disassembling a structure at its end-of-life. Designing long span beams creates larger clear spaces, encouraging flexibility of the structure for future uses and allowing for further cutting if needed when the structural elements are retrieved for reuse (LETI, 2016).

If reuse or recycling is not possible, wood products can be burnt and energy recovery techniques used to retrieve residual value from the material as bioenergy. In this case, special measures should be taken to handle harmful emissions from incineration of treated wood or adhesives. Wood fuel burnt directly for energy usually comes from short-rotation forests, and trees that are too small for structural use. Landfill should only be considered as a last resort option, as biodegradation of wood waste is associated GHG emissions of methane or carbon dioxide, among others. European countries are increasingly banning or penalising wood landfilling with governments taxes, to encourage material recovery and recycling. Apart from extending biogenic carbon storage, these scenarios also have the advantage of reducing the demand for newly-sourced wood and emissions from the production process (Ramage et al., 2017).

3.3 Building with nature

This section answers the following research subquestion: *How does integrating nature in the design of buildings contribute to lowering their environmental impact?*

3.3.1 Ecosystem services

Introducing nature in the built environment can provide a number of benefits, named ecosystem services. Ecosystem services are defined as "ecological characteristics, functions, or processes that directly or indirectly contribute to human wellbeing: that is, the benefits that people derive from functioning ecosystems" (Costanza et al., 1997). The benefits of ecosystem services first gained importance in the 70s when their cost-effectiveness was demonstrated under an economical prism, highlighting the positive role of vegetation as a natural capital for human well-being. Their loss was also becoming more apparent as the understanding of ecology grew.

This concept was developed further in two international studies, in 2005 for the Millennium Ecosystem Assessment (Millennium Ecosystem Assessment, 2005), and in 2010 for the TEEB studies (TEEB, 2010), both confirming the potential of using ecosystem services to mitigate the environmental impact of human activities in urban areas. From these major studies resulted different classification systems for ecosystem services now used worldwide, such as the Common International Classification of Ecosystem Services (CICES) revised in 2018.

Air quality regulation– Vegetation has a positive effect on local, regional and global air pollution as CO2 is taken up by plants during photosynthesis, and particulate matter PM2,5+PM10+NOx is adsorbed by stomata or PM10 on leaves. It mitigates global warming by reducing levels of greenhouse gases and improves air quality in areas where local road traffic emits particulate matter and NOx harmful to human health.

Climate regulation– Temperatures are generally higher in urban areas due to the urban heat island effect, affecting health of inhabitants and daily functioning. Introducing greenery in the built environment regulates local and regional climate by preventing stone-like materials and the air from heating up, by creating a natural barrier and taking advantage of mechanisms like evapotranspiration to retain low temperatures.

Disturbance regulation– In urban areas, ecosystem services mitigate the consequences of environmental fluctuations by regulating wind effects or damping the impacts of extreme events like heat, storm, flooding or drought.

Water supply and regulation– The storage and retention of water by vegetation may help maintaining drinking water sources and natural hydrological flows on a global scale.

Erosion control, soil formation– Desertification of dryland areas by human activities like deforestation or variations in climate compromises food and water availability in these ecosystems and affects climate regulation processes. Against this phenomenon, the water–soil–plant continuum helps retain soil and control erosion within an ecosystem.

Pest (biological) control, nutrient cycling, pollination, creating refugia– The rapid loss of biodiversity in human populated areas and on a global scale is threatening the balance of ecosystems and their surviving capacity. To preserve them and maintain the strong supporting service of biodiversity in built environments, the growing lack of refugia and habitat should be tackled by creating protected areas of nature and vegetation to support pollination, and avoiding excessive use of pesticides or nutrients to relieve pressure on ecosystems.

Waste treatment– Ecosystems participate in filtering and purifying waste, degrading them into harmless material by recycling nutrients or detoxifying wastewater thanks to bacteria and microorganisms in symbiosis.

Aesthetics, recreation, cultural services– Green areas have been proven to beneficially impact the well-being of people, by providing recreational opportunities, and through the pleasant and inspirational aesthetics of natural environments over built ones.

Food production and raw materials– Most food, biomass and raw materials are harvested outside cities, but their production can be brought to urban areas and represent an added value to greenery.

Genetic resources– The extinction of species is threatening genetic diversity, particularly among cultivated species, affecting the resilience of ecosystems on a global scale. Genetic resources from unique biological products provide opportunities to develop medicine or enhance the resistance of crops to pest and pathogens.

Noise reduction– In urban areas when noise is a major disturbance to inhabitants, greenery has the ability to damp sound waves and reduce noise levels on a local scale.

Health improvement– The mental and physical health of human beings has proven to be positively impacted by green. Ecosystem services contribute to disease regulation and nutrition, as well as providing resources for medicine and lowering pollution levels. As a result, the economic costs of healthcare on a regional scale can be reduced.

3.3.2 Green in the built environment

The benefits of ecosystem services can be optimally used in the built environment by making design choices which include vegetation, implementing green envelopes on existing or new buildings or specific elements aiming at protecting biodiversity in the area of the project. However, decision-making processes in design of buildings often overlook the natural capital to focus on cost-benefit analyses quantifying the performance of design options based on economic value. Another reason for the lack of green systems in the building sector is the hesitation concerning their possible disadvantages (extra maintenance, falling leaves, wall damage, insects and spiders, extra costs).

Establishing a direct link between socio-environmental issues and ecosystem services eases integration of these solutions in the built environment development process (Stache et al., 2019). The architect Eva Stache researches how implementing green in cities can help mitigating a number of socio-environmental problems, by providing specific ecosystem services. One question is to determine how much of a certain type of vegetation allows to reduce a specific issue, and how to effectively implement this amount of vegetation in the built environment to maximise durability while minimising the need for maintenance (Jonkers, 2020). If several design options developed by a structural engineer achieve the same level of embodied carbon, they should choose the solution minimising biodiversity loss, protect and enhance the existing biodiversity (Watson & Sefton, 2021). More and more, municipalities and regions are encouraging the installation of green systems by funding projects or imposing rules on new buildings to include a green roof.

The environmental impact of greening systems come from the materials required in their production, maintenance processes during its service life, and the additional material required in the load-bearing structure of the building, necessary to support the extra weight. These costs can easily be quantified using the traditional LCA methodology. The benefits of vegetation providing ecosystem services are not credited in LCA. Therefore, comparing different types of greening systems in a fair way using this method is difficult, since additional material use for supports will account for a negative impact, while positive effects will not be considered as bonuses. However, assigning a value to ecosystem services would allow them to be effectively taken into account when performing cost-benefit calculations during building design, and eventually encourage the incorporation of green in the built environment (Jonkers, 2020).

4

Design step 1 - Substituting steel/concrete by timber in the frame

This chapter aims at answering research question *(2) How much environmental impact reduction can be achieved by substituting steel or concrete by timber in the load-bearing frame of a warehouse?* and the corresponding subquestions.

As they set the basis for the analysis carried out in this first research step, the selected steel and concrete warehouse reference designs are presented in section 4.1. Two timber baseline warehouses are then designed in section 4.2, to quantify the impact of substituting steel or concrete by timber in the load-bearing frame of a warehouse. Following the LCA methodology, the environmental impact score of all four designs is calculated in section 4.3, after which the results are presented and discussed in section 4.4.

4.1 Reference designs

4.1.1 Selecting reference designs

Royal HaskoningDHV was charged for delivering a large range of distribution centres designs to their client, of different functions within the logistic chain (see Figure 4.1). The goal is to implement these logistic centres across Europe, hence the need for standard designs meant to be adapted to the requirements of each location chosen by the client.

4.1.1.1 General requirements for distribution centres

Structural design for such projects is meant to answer basic principles including rapid deployment, resilience, and structural fire safety. Short erection times ensure that the structure is quick and easy to build on site and can be used quickly after start of construction, so that the client can get maximal economic benefits as soon as possible. Flexibility of the structural layout, provided by large free spaces inside, ensures reuse potential during the life of the building and maximum functionality. These characteristics are especially important for logistic centres in a rapidly evolving commercial environment as their exact service life remains unknown.

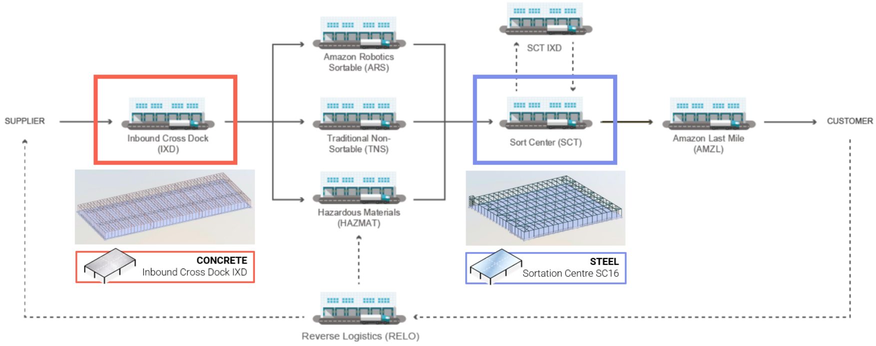


Figure 4.1: Types of fulfilment centres along the client's logistic chain and illustration of the steel and concrete reference designs selected in this project (RHDHV Internal Document, 2020)

4.1.1.2 Decision-making criteria

Materials

Logistic halls are mostly built from concrete, but steel structures may also be used. The first step of this research consists in developing a baseline distribution centre with a timber load-bearing structure, to compare the environmental impact of using biobased materials with traditional steel or concrete. Therefore, the reference designs should consist in one concrete structure, and one steel structure.

Structural layout

Using different materials influences not only the environmental performance of the building, but its structural design. In order not to make the loading situation unnecessarily complex, and focus on the impact of materials, the structural system of reference designs should remain simple, to serve as a basis for developing a comparable timber structure. Some logistic centres comprise an additional mezzanine floor to accommodate machines, complicating the load distribution. Only single storey buildings without mezzanine floors are investigated in this study. When it comes to the shape of the building, rectangular geometries are the norm. However, the grid size and dimensions in length, width and height may vary depending on the intended use of the hall.

4.1.1.3 Selected reference designs

Based on the aforementioned criteria, the references selected for this study are an IXD Inbound Cross Dock building designed with a concrete load-bearing structure, and an SC16 Sortation Centre with a steel structure. They serve different purposes in the logistic chain, illustrated on Figure 4.1. The concrete and steel reference designs are both single storeys with the same grid size, but their width, length and height differ. Their geometrical and structural characteristics are presented in sections 4.1.3 and 4.1.2 respectively. Both structural designs were finalized, but their environmental impact was not calculated yet. As they are standards meant to be adapted by structural engineers when effectively built, neither of these was fully optimised regarding structural or environmental performance. Therefore the timber baseline design shall not be optimised either for fair comparison.

4.1.2 Steel (SC16) reference design

The logistic centre SC16 design is a Sortation Centre model consisting of a steel warehouse, an office and welfare area, and two remote break rooms. Only the warehouse structure is considered in this study, with the dimensions indicated in Table 4.1.

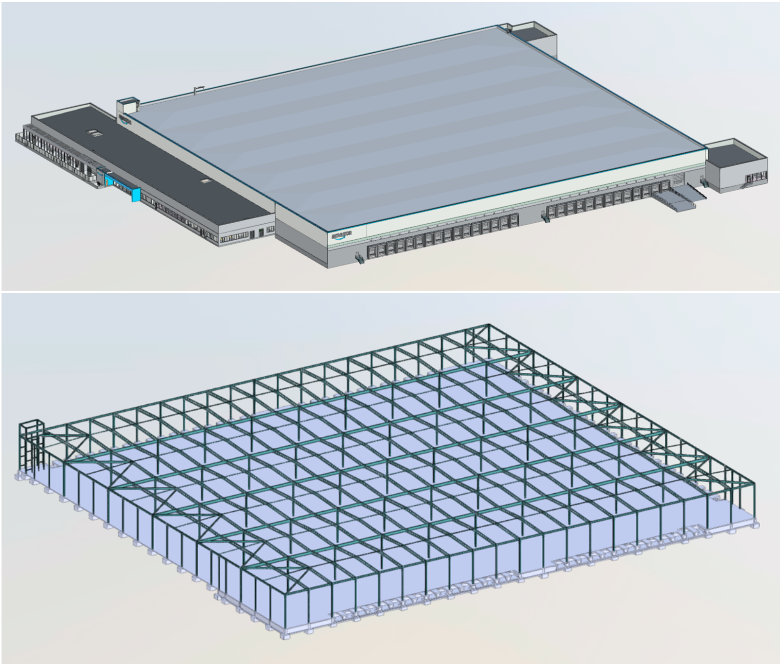


Figure 4.2: Steel (SC16) warehouse design overview (top), load-bearing frame, floor slab and foundation pads (bottom)

Table 4.1: Steel (SC16) warehouse geometry

Warehouse dimensions LxW	ca. 160x144m
Ground floor area	23040m^2
Internal clear height	10.3m
Building grid	16x24m

4.1.2.1 Stability system

Stability of the steel warehouse is ensured in both directions by sway portal frames: 18 rows in the longitudinal direction and 15 rows in the transverse direction, as illustrated on Figure 4.3. These portal frames are composed of columns and beams connected by moment-fixed connections at the top and hinged at the base. The perimeter frames of the warehouse do not take part in the stabilising structure.

In steel structures, it is relatively easy to manufacture moment fixed connections between structural elements in the workshop, or directly within the building frame. In theory, introducing fixed connections at the base of columns is also a feasible option, but it would require additional coordination between construction teams to ensure suitable anchorage in concrete foundations, combined with high accuracy required for steel detailing. Moreover, the advantage related to small steel columns enabling large clear areas inside the warehouse would be cancelled by the need for adding large base plates, necessary to provide sufficient moment capacity to the base connection. Designing moment fixed connections at the top of columns is therefore the most logical choice for stability of the steel warehouse due to manufacturing considerations.

The roof structure acts as a diaphragm by means of bracings between roof beams. The floor is made with concrete slabs. A steel sway structure, stabilised by fixed connections, is an interesting option for a logistic centre as it does not require internal or facade bracings. The stabilising capacity is thus spread throughout the entire structure, maximising the production capacity and flexibility of the building, and allowing for a faster erection time.

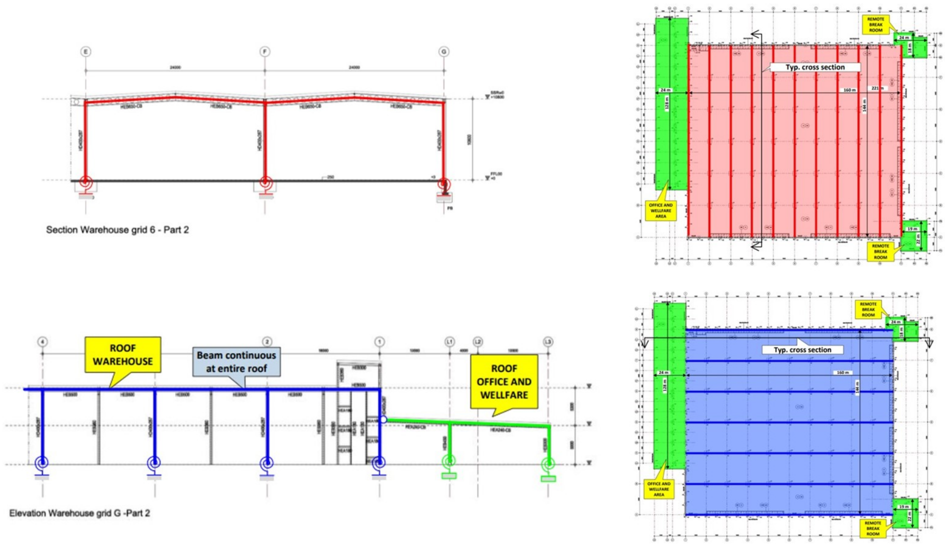


Figure 4.3: Typical sections of the steel (SC16) warehouse with stabilising portal frames in the transverse direction (red) and in the longitudinal direction (blue) (RHDHV Internal Document, 2022)

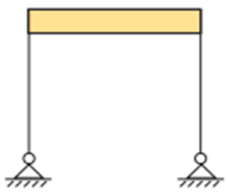


Figure 4.4: Stability system of the steel reference frame: Moment fixed connections between columns and roof beams, hinged at the base

4.1.2.2 Structural elements

Building envelope

Kingspan panels are used for the facade and the roof. Roof panels are supported by steel purlins and square hollow section beams, while facade panels are laid on cold-formed vertical steel joists.

Frame

Primary beams handle vertical loads on the structure in the transverse and longitudinal directions, and secondary floor beams are added in the transverse direction. Castellated beams are used for supporting the roof elements, to reduce the volume of material used and the weight of the structure, while allowing services to run through to save internal free height. Ducts in the other direction are placed right below the secondary beams.

Floor slab

The ground floor slab is 200mm thick in situ concrete C30/37. It is not physically connected to foundations to allow for independent shrinkage.

Foundation pads

Foundation elements, like in the concrete reference design, are made using in situ concrete C50/60, with rectangular footings under load-bearing columns and underground concrete slabs around the facade.

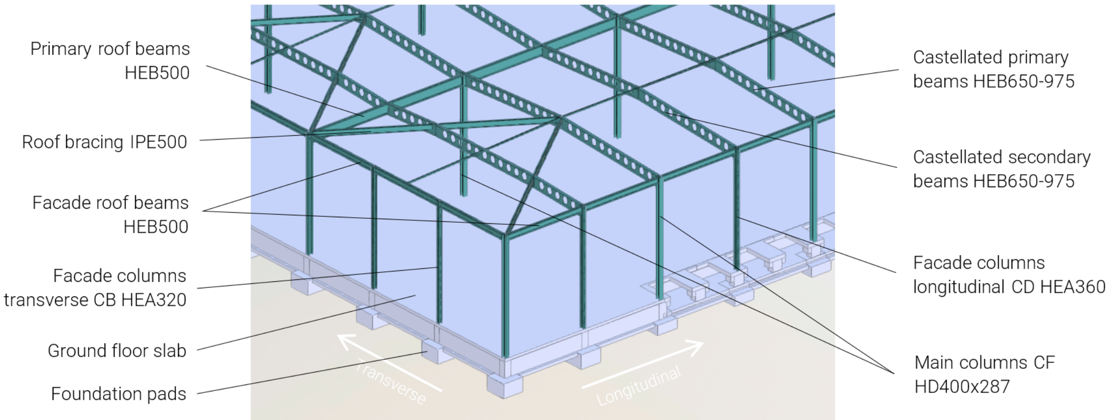


Figure 4.5: 3D view of structural elements within the steel (SC16) warehouse frame (RHDHV Internal Document, 2022)

4.1.3 Concrete (IXD) reference design

The logistic centre IXD design is an Inbound Cross Dock model consisting of a concrete warehouse, surrounded by an office and welfare area, two truckers lounges and smaller toilet clusters on the sides. Only the warehouse is considered in this study, of dimensions indicated in Table 4.2.

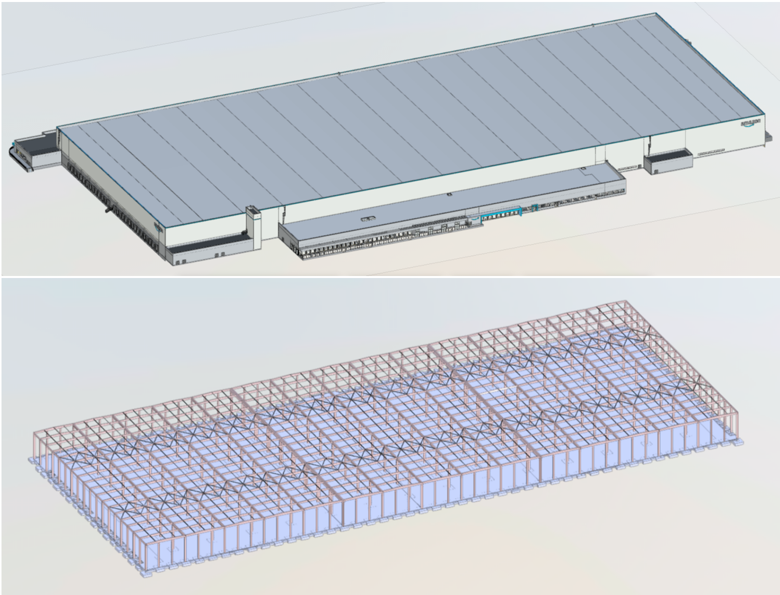


Figure 4.6: Concrete (IXD) warehouse design overview (top), load-bearing frame, floor slab and foundation pads (bottom)

Table 4.2: Concrete (IXD) warehouse geometry

Warehouse dimensions LxW	ca. 144x360m
Ground floor area	51840m^2
Internal clear height	14.3m
Building grid	16x24m

4.1.3.1 Stability system

Stability of the concrete warehouse is ensured in the longitudinal direction by 10 rows of fixed base columns, including facades. Secondary roof beams are simply supported on the columns, by a hinged connection at the top. In the transverse direction, 16 rows of portal frames including facades stabilise the structure, once again with fixed base columns. Figure 4.7 illustrates typical rows of portal frames in both directions.

The reason for fixed based columns and hinged connections with roof beams at the top (see Figure 4.8) is directly related to the use of concrete in the load-bearing structure. Indeed, at the foundation level, concrete elements are large and full rectangular sections are able to provide large moment capacity. Moment fixing in foundation is also beneficial in concrete structures because they are usually heavier, and it is relatively easy to create a clamped connection at the base of the structural columns, taking advantage of the rotational stiffness of the soil. On the contrary, it is much harder to make moment fixed connections with prefab concrete roof and floor elements, hence why cheaper and faster pin connections are preferred in this case.

The roof is braced with steel beams. The sway configuration of the load-bearing structure does not require vertical bracing. The absence of bracing members on the facade maximises the number of dock doors for trucks, while the inside space is also free of braces and optimized for maximum flexibility. Clear spans of 24m in the longitudinal direction and 16m in the transverse direction, corresponding to the grid size, are suitable to accommodate different stacking layouts inside the warehouse.

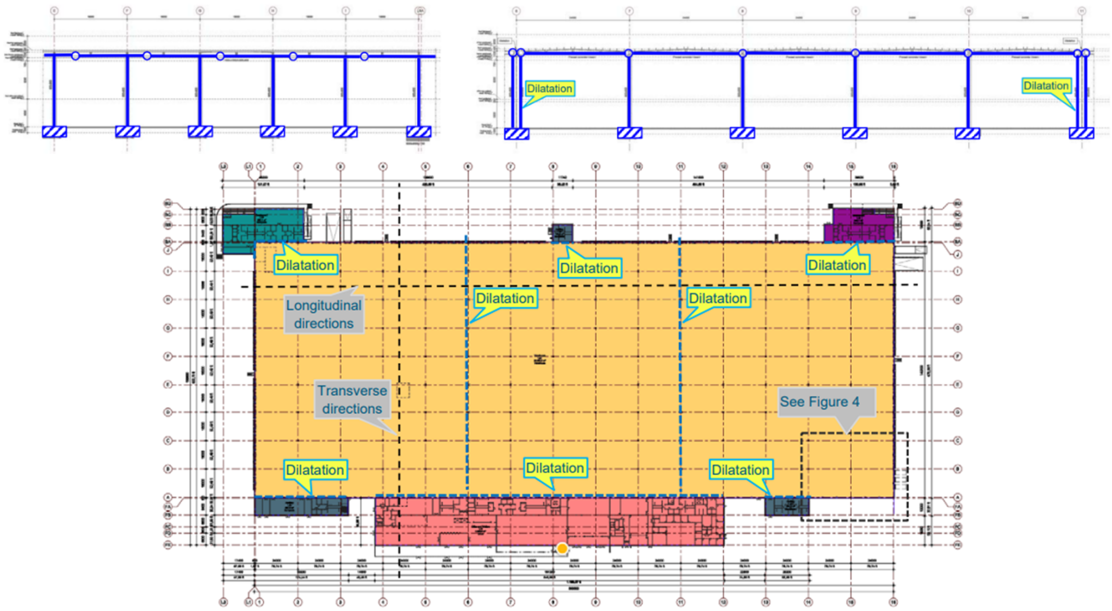


Figure 4.7: Typical sections of the concrete (IXD) warehouse with stabilising portal frames in the transverse (top left) and longitudinal (top right) directions, and general layout (bottom) (RHDHV Internal Document, 2020)

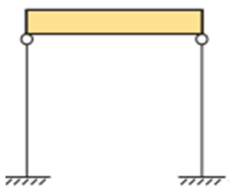


Figure 4.8: Stability system of the concrete reference frame: Fixed base columns, hinged roof beams

4.1.3.2 Structural elements

Building envelope

Façade and roofing elements consist of Kingspan sandwich panels. Roof panels are supported by secondary steel purlins and square hollow section beams, while facade panels are laid on cold-formed vertical steel joists.

Frame

The vertical load distribution is handled by trapezoid secondary beams creating the roof slope, supported by inverted T primary roof beams, to reduce the overall height of the building. These primary beams are supported by the main columns, and additional columns every 8 meter at the facade to reduce the size of perimeter roof beams. The frame structure is built on the foundations before the floor, facades and roof are installed.

Floor slab

The ground floor slab is 200mm thick. It is not physically connected to foundations to allow for independent shrinkage.

Foundation pads

The foundation is made of in situ concrete C50/60. Foundation pads are placed under the main load-bearing columns, and additional foundation beams are meant to support the loads from machines inside the building and the façade, especially at the locations of bumpouts around the central warehouse.

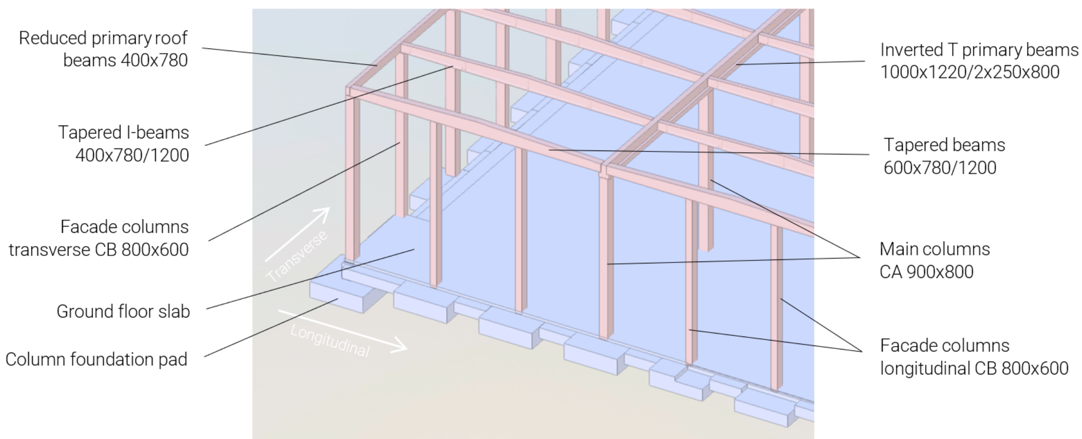


Figure 4.9: 3D view of structural elements within the concrete (IXD) warehouse frame (RHDHV Internal Document, 2020)

4.1.4 Bill of materials for the reference designs

An overview of materials used in the steel and concrete reference warehouses is presented in Tables 4.3 and 4.4 respectively. These bills are created using 2D plans and 3D Revit models. The envelope design is detailed in Appendix E. In this research, the total area of facades without openings is considered for all alternatives, as a conservative assumption.

Table 4.3: Steel reference (SC16) structural elements

Structural element	Material	Dimensions	Total amount
Roof panels	Kingspan sandwich panels		23040m^2
Roof purlins	Structural steel	Multibeam M300090270 10.64kg/m, 8.0m (x980)	8.34E+04kg
Facade panels	Kingspan sandwich panels		7964.8m^2
Facade mullions	Structural steel	IPE360 57.1kg/m, 10.3m (x76)	4.47E+04kg
Columns	Structural steel	see Figure 4.5	38.7m^3 / 3.04E+05kg
Beams	Structural steel	see Figure 4.5	122.2m^3 / 9.59E+05kg
Floor slab	Concrete C30/37	200mm thick, 23040m^2	4608.0m^3 / 1.15E+07kg
	Reinforcing steel	120kg/m^3 concrete	70.4m^3 / 5.53E+05kg
Foundation pads	Concrete C30/37	3000x3000x1000mm (x121)	1089.0m^3 / 2.72E+06kg
	Reinforcing steel	250kg/m^3 concrete	34.7m^3 / 2.72E+05kg
Others	Concrete C30/37	Foundation beams and walls	347.4m^3 / 8.68E+05kg
	Reinforcing steel		5.3m^3 / 4.17E+04kg

Table 4.4: Concrete reference (IXD) structural elements

Structural element	Material	Dimensions	Total amount
Roof panels	Kingspan sandwich panels		51840m^2
Roof purlins	Structural steel	Multibeam M300090270 10.64kg/m, 8.0m (x2178)	1.85E+05kg
Facade panels	Kingspan sandwich panels		17236.8m^2
Facade mullions	Structural steel	IPE500 90.7kg/m, 14.3m (x126)	1.63E+05kg
Columns	Concrete C50/60	see Figure 4.9	2475.6m^3 / 6.19E+06kg
	Reinforcing steel	300kg/m^3 concrete	94.6m^3 / 7.43E+05kg
Beams (concrete)	Concrete C50/60	see Figure 4.9	3940.1m^3 / 9.85E+06kg
	Reinforcing steel	200kg/m^3 concrete	100.4m^3 / 7.88E+05kg
Beams (steel)	Structural steel	see Figure 4.9	18.5m^3 / 1.45E+05kg
Floor slab	Concrete C30/37	200mm thick, 51840m^2	10368m^3 / 2.59E+07kg
	Reinforcing steel	120kg/m^3 concrete	158.5m^3 / 1.24E+06kg
Foundation pads	Concrete C30/37	5000x5000x1500mm (x238)	8925.0m^3 / 2.02E+07kg
	Reinforcing steel	250kg/m^3 concrete	284.2m^3 / 2.02E+06kg
Others	Concrete C30/37	Foundation beams and walls	1266.2m^3 / 3.17E+06kg
	Reinforcing steel		19.4m^3 / 1.52E+05kg

4.2 Design of timber baseline variants

The main goal of the timber structure developed in this first step is to compare the influence of material choice for the load-bearing frame on the environmental impact of a warehouse. This section describes design choices made for the timber baseline warehouses to compare with reference concrete and steel designs presented in section 4.1, answering the following research subquestion: *What type of structural system is appropriate for a baseline timber warehouse?*

4.2.1 Structural layout and dimensions

To allow for a fair comparison, as neither one of the reference designs were created with environmental impact optimisation in mind, the timber baseline designs should be developed only for comparison purposes, with similar geometrical and structural characteristics as the selected references. For this reason, it is decided to create two distinct timber baseline designs A and B, to compare with the steel and concrete references respectively.

Table 4.5: Geometry of timber baseline designs A and B compared to steel and concrete references

Design	Length	Width	Internal height
Steel reference (SC16) Timber baseline A	144m	160m	10.3m
Concrete reference (IXD) Timber baseline B	360m	144m	14.3m

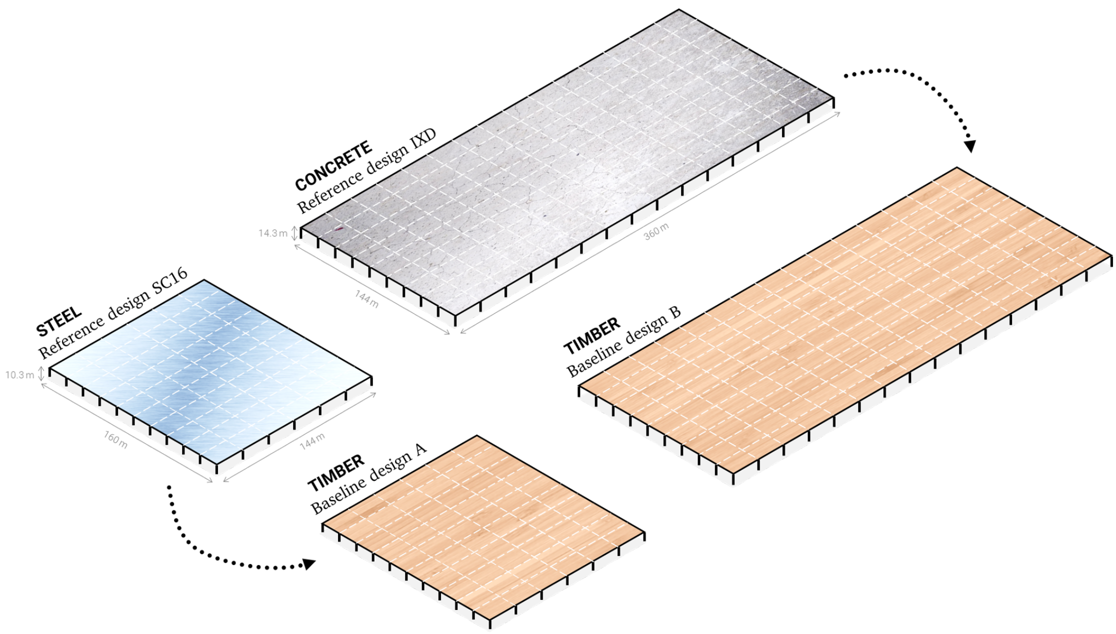


Figure 4.10: Overview of timber baseline designs A and B compared to the steel and concrete references

The dimensions are related to different intended functions of the standard designs. By creating one timber design per reference, the functional unit for comparison is based on the same clear internal volume to use for commercial activities. A grid of 16x24m is used for all designs, to ensure maximum flexibility inside the warehouse.

4.2.2 Structural elements

The scope of building components considered in the design of both timber warehouses is the same as the reference designs: envelope, frame, foundation pads and floor slab.

Building envelope

Façade and roof elements, as well as their supports are the same those of the references. They consist of Kingspan sandwich panels laid on steel purlins and mullions.

Frame

The warehouse structural grid, illustrated on Figure 4.11, defines the spacing between columns and the span of primary and secondary beams in the y and x directions respectively. Then, the total length and width of the building are defined by the number of bays in each direction. In the vertical direction following the z-axis, the relevant dimensions are the height of columns H, the height of parapet h_p, the maximum height of beams in the x or y direction and their sum, which is the total height of the structure.

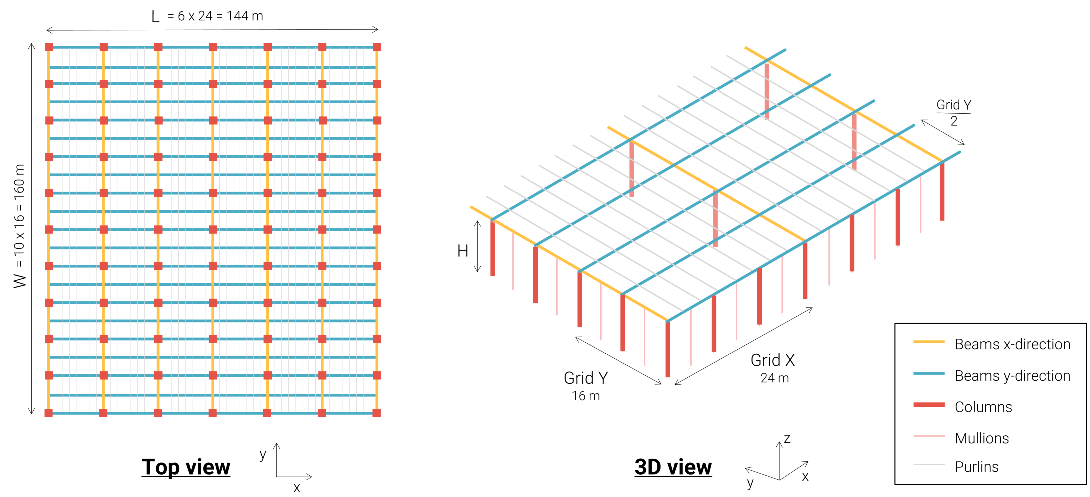


Figure 4.11: Top and 3D view of the timber baseline A load-bearing frame

Frame elements can be manufactured from engineered wood products to create high performance load-bearing systems, making advantage of their improved material properties compared to solid timber elements. The vertical load distribution is handled by rectangular glulam beams in the lateral and longitudinal directions, supported by square glulam columns. Apart from main columns aligned with the structural grid, smaller facade columns are added every 8 metres along the perimeter of the warehouse.

Floor slab

The ground floor slab is 200mm thick concrete C30/37, same as both reference designs.

Foundation pads

The foundation is made of in situ concrete C50/60. Foundation pads are placed under the main load-bearing columns and dimensioned to resist design loads at the base of columns. As lighter structures will require smaller foundations, hence a reduced volume of concrete, this aspect is relevant to the total environmental impact evaluation. The same pads 3000x3000x1000mm are used for the baseline A as in the reference steel design. However, the pads of baseline B (4000x4000x1250mm) are smaller than for the concrete reference (5000x5000x1500mm).

4.2.3 Stability system

Sway frame

The reference designs are both bi-directional sway structures, differing mainly in the location of moment fixed or hinged connections: fixed base columns and hinged beams for the concrete structure, and the opposite for the steel structure. Such choices are deeply related to the type of material. The most appropriate solution for creating a sway timber frame shall be studied independently from the steel and concrete reference designs, based on material properties and common construction methods.

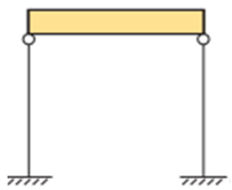


Figure 4.12: Stability system of the timber baseline frames: Fixed base columns, hinged roof beams

It is easier to create clamped connections at the base of columns in timber frames, rather than between beams and columns at the top where hinges are usually preferred. For this reason, both timber baseline warehouses are designed as bi-directional sway structures with fixed base columns and hinged beams (see Figure 4.12).

Detailing of connections

Different types of connections may be designed between timber load-bearing elements, influencing the overall stability of the building depending on the hinged or rigid behaviour of the connections. Fixation methods are numerous and can make use of glue, dowels, nails, screws or epoxy, among others.

Beams in both directions are designed with the same cross-section height to facilitate detailing. At the top of columns, a pin connects columns and beams in both directions. Four steel angles are used all around beams to keep them in place, without restraining the end rotation induced by deflection. Fork supports are also added to prevent lateral deflection of beams at their supports. Secondary beams spanning in the x-direction (blue) are also connected to primary beams (yellow) at midspan, by means of steel shoes acting as shear connections (see Figure 4.13).

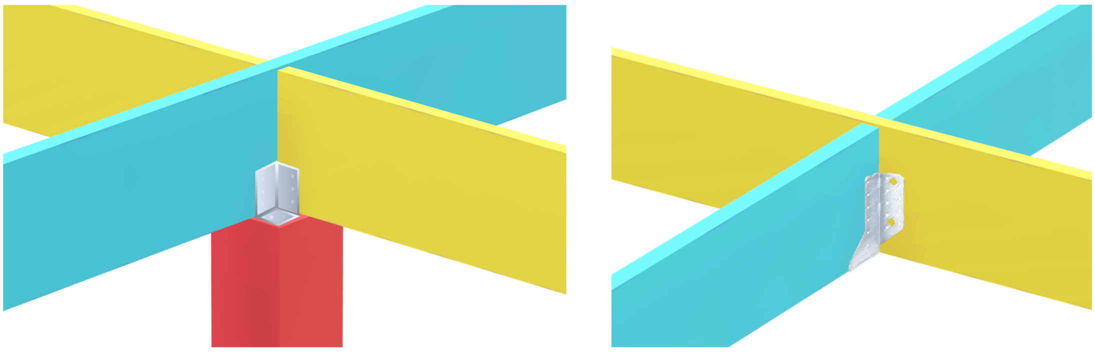


Figure 4.13: Detailing of the hinged top connections in timber baseline designs

The column base connection with foundations (see Figure 4.14) is designed with moment-resisting glued-in rods connecting the timber column to foundation pads underneath, to provide rotational stiffness and stabilise the sway frame in both directions. The complete design of this connection is carried out in Appendix C.5.

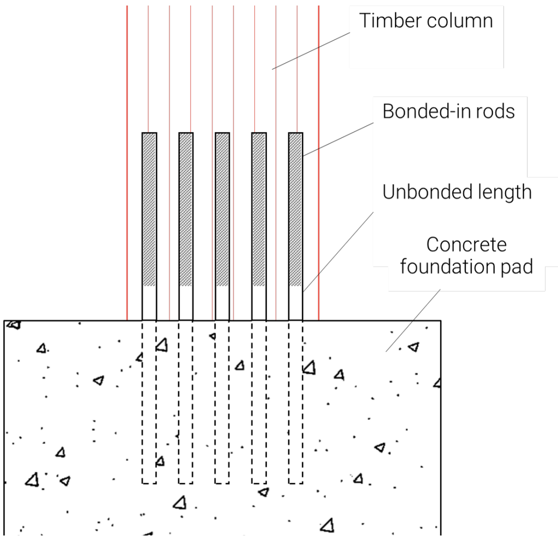


Figure 4.14: Detailing of the semi-rigid column base connection in timber baseline designs

4.2.4 Final timber designs

Loads applied on the timber frames are calculated as explained in Appendix B. Based on these, timber members in the frame and foundation pads are dimensioned following the design process detailed in Appendix C and D for baseline designs A and B respectively. The sizes of structural elements are summarised in this section.

4.2.4.1 Bill of materials for timber baseline A

The dimensions of all building elements are given in Table 4.6.

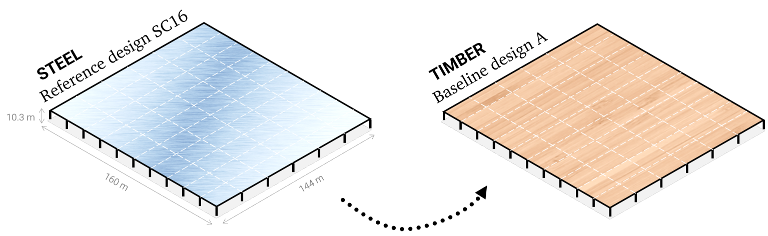


Figure 4.15: timber baseline design A to compared with steel reference design

Table 4.6: Timber baseline A structural elements

Structural element	Material	Dimensions	Total amount
Roof panels	Kingspan sandwich panels		23040m^2
Roof purlins	Structural steel	Multibeam M300090270 10.64kg/m, 8.0m (x980)	8.34E+04kg
Facade panels	Kingspan sandwich panels		7964.8m^2
Facade mullions	Structural steel	IPE360 57.1kg/m, 10.3m (x76)	4.47E+04kg
Main columns	Glulam GL28h	1000x1000x10300mm (x77)	793.1m^3 / 3.65E+05kg
Facade columns	Glulam GL28h	400x400x10300mm (x44)	72.5m^3 / 3.33E+04kg
Beams (y)	Glulam GL28h	220x1800x15720mm (x70)	435.8m^3 / 2.00E+05kg
Beams (x,large)	Glulam GL28h	280x1800x24000mm (x66)	798.3m^3 / 3.67E+05kg
Beams (x,small)	Glulam GL28h	280x1800x23780mm (x60)	798.3m^3 / 3.31E+05kg
Connections	Structural steel	0.05% timber volume	1.4m^3 / 1.11E+04kg
Floor slab	Concrete C30/37 Reinforcing steel	200mm thick, 23040m^2 120kg/m^3 concrete	4608.0m^3 / 1.15E+07kg 70.4m^3 / 5.53E+05kg
Foundation pads	Concrete C30/37 Reinforcing steel	3000x3000x1000mm (x121) 250kg/m^3 concrete	1089.0m^3 / 2.72E+06kg 34.7m^3 / 2.72E+05kg

4.2.4.2 Bill of materials for timber baseline B

The dimensions of all building elements are given in Table 4.7.

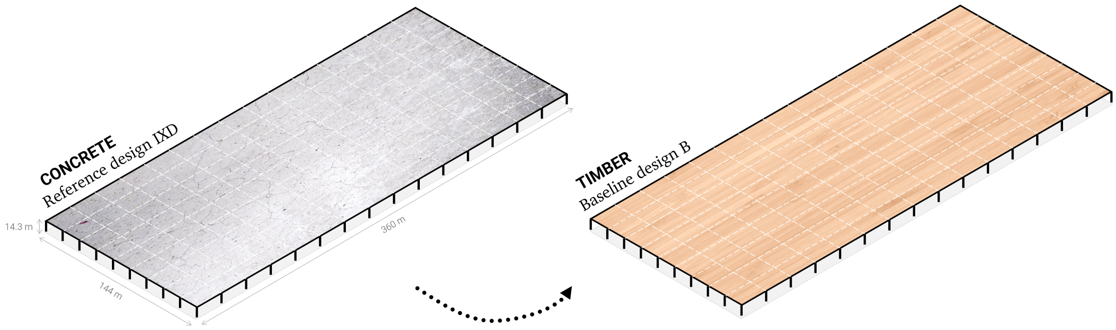


Figure 4.16: timber baseline design B to compared with concrete reference design

Table 4.7: Timber baseline B structural elements

Structural element	Material	Dimensions	Total amount
Roof panels	Kingspan sandwich panels		51840m^2
Roof purlins	Structural steel	Multibeam M300090270 10.64kg/m, 8.0m (x2178)	1.85E+05kg
Facade panels	Kingspan sandwich panels		17236.8m^2
Facade mullions	Structural steel	IPE500 90.7kg/m, 14.3m (x126)	1.63E+05kg
Main columns	Glulam GL28h	1150x1150x14300mm (x160)	3025.9m^3 / 1.39E+06kg
Facade columns	Glulam GL28h	500x500x14300mm (x78)	278.9m^3 / 1.28E+05kg
Beams (y)	Glulam GL28h	200x1800x15720mm (x144)	814.9m^3 / 3.75E+05kg
Beams (x,large)	Glulam GL28h	280x1800x24000mm (x150)	1814.4m^3 / 8.35E+05kg
Beams (x,small)	Glulam GL28h	280x1800x23800mm (x135)	1619.4m^3 / 7.45E+05kg
Connections	Structural steel	0.05% timber volume	3.8m^3 / 2.96E+04kg
Floor slab	Concrete C30/37 Reinforcing steel	200mm thick, 51840m^2 120kg/m^3 concrete	10368m^3 / 2.59E+07kg 158.5m^3 / 1.24E+06kg
Foundation pads	Concrete C30/37 Reinforcing steel	4000x4000x1250m (x238) 250kg/m^3 concrete	4760.0m^3 / 1.19E+07kg 151.6m^3 / 1.19E+06kg

4.3 Environmental impact calculations

The environmental impact of steel, concrete and timber designs studied in the first design step of this research is determined using the LCA method explained in Appendix F. This section answers the following research subquestion: *What is the environmental impact of steel and concrete warehouse structures?*

4.3.1 Goal and scope definition

The goal of this first step is to determine the environmental impact of 4 warehouse designs, to assess the effect of substituting steel or concrete by timber in the load-bearing frame.

Functional unit

The functional unit is described as warehouses of the same size, with a reference service life of 50 years. The steel reference is compared to the timber baseline warehouse A, and the concrete reference to the timber baseline B. Building elements included in the calculation of each alternative are the frame (columns, beams), foundation pads, floor slab, and building envelope (facade and roof panels, supports). The reference steel and concrete designs comprise additional elements such as foundation beams or retaining walls. However, their design does not depend on the type of material in the load-bearing frame, and they only represent a fraction of the concrete used for the ground floor and the substructure. They are classified in a separate results category titled "Others".

System boundaries

According to (**EN15804**), an EPD should always include LCA stages A1-A3. This is in line with the scope of this research project to assess upfront carbon emissions from building a distribution centre. The replacement of products having a shorter service life than required for the building is also accounted for in stage A1-A3. Indeed, they are associated with the embodied carbon of materials necessary to satisfy performance requirements throughout the whole service life of the building. The construction process stage A4-A5 is not assessed because the location of the building is unknown at this early design stage. Similarly, the use stage B1-B7 is excluded because it would require detailed information on the maintenance processes, still unknown. Stages C1-C4 and D are left out as they are associated with the end-of-life and uncertain future scenarios.

LCA methodology

The embodied carbon of the product stage A1-A3 is calculated based on environmental data retrieved from EPDs, gathered using the OneClickLCA database. This study focuses on the "Global Warming Potential (GWP)" environmental impact category to assess the embodied carbon of building components, while also considering the biogenic carbon content of biobased materials. As environmental data from EPDs may vary significantly across manufacturers, it is decided to investigate extreme datasets to get an overview of the most and least sustainable products on the market. The description of EPDs studied in this research is given in Appendix G.

4.3.2 Life cycle inventory

Quantifying the impact of materials requires multiplying the GWP factors for stages A1-A3 retrieved from EPDs, by the total amount of materials in the structure. The bill of materials for each design was presented in previous sections.

4.3.3 Environmental impact results

Figures 4.17 to 4.20 show the results from environmental impact calculations as waterfall diagrams, to have an overview of the impact of each building component relative to the total embodied carbon of the four designs. For the reference steel and concrete designs, only the embodied carbon of fossil emissions is specified, whereas for both timber designs the biogenic carbon content in biobased materials is illustrated in a separate graph.

Steel reference (SC16)

The environmental impact score of the steel reference warehouse is:

$$GWP_{fossil} = 5.87E + 06kgCO2e$$

The embodied carbon of the steel warehouse can mostly be attributed to the steel frame (33%), floor slab (28%) and envelope (28%). As the stability system of the steel warehouse relies on pinned base columns and fixed connections at the top with beams, the dimensions of foundation pads remain limited, resulting in relatively low volumes of concrete compared to fixed base connections. Additionally, the embodied carbon of steel structural elements is higher than concrete.

Concrete reference (IXD)

The environmental impact score of the concrete reference warehouse is:

$$GWP_{fossil} = 1.50E + 07kgCO2e$$

The embodied carbon is almost equally shared among all the 4 main categories of structural components in the concrete warehouse. Compared to the steel reference design, foundation pads need to be larger to provide moment resisting capacity for fixed column base connections, resulting in higher concrete and reinforcing steel consumption, hence a higher share in the total embodied carbon. Precast concrete beams and columns composing the load-bearing frame account for a quarter of the total impact score of the concrete reference design.

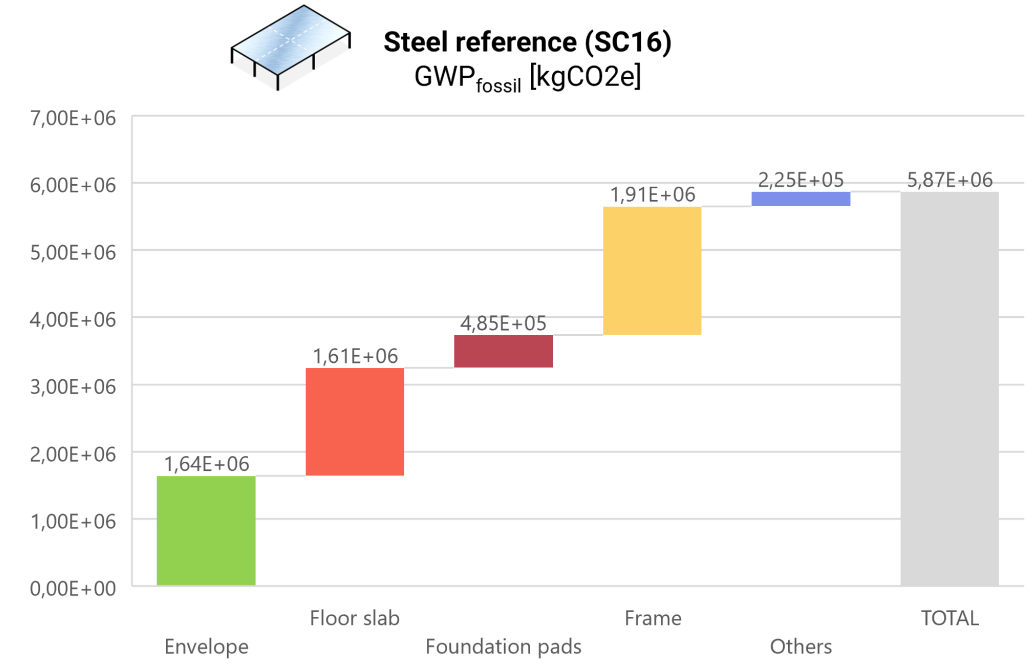


Figure 4.17: GWP_{fossil} [kgCO2e] of building components of the steel reference (SC16) warehouse

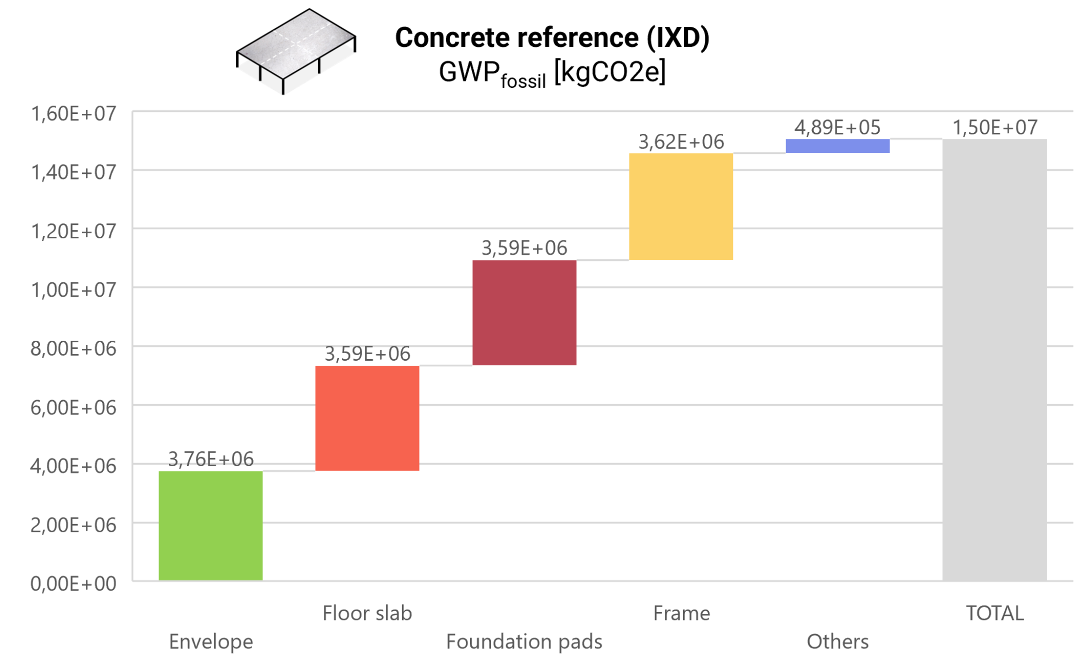


Figure 4.18: GWP_{fossil} [kgCO2e] of building components of the concrete reference (IXD) warehouse

Timber baseline A

The environmental impact of the timber baseline A warehouse is:

$$GWP_{fossil} = 4.00E + 06kgCO2e$$

$$GWP_{fossil+bio} = 1.91E + 06kgCO2e$$

The embodied carbon of the floor slab and envelope govern the impact score of the building, respectively accounting for 40% and 41% of the GWP_{fossil}. The timber frame is responsible for a considerably lower share (7%) thanks to the low embodied carbon of glulam regarding fossil related emissions. The advantage biogenic carbon storage in wood during the growth process is clearly illustrated on Figure 4.19. By accounting for biogenic carbon storage next to fossil emissions in the production phase, the impact of the timber frame becomes negative and lowers by 52% the total impact of the frame compared to fossil emissions only.

Timber baseline B

The environmental impact of the timber baseline B warehouse is:

$$GWP_{fossil} = 1.02E + 07kgCO2e$$

$$GWP_{fossil+bio} = 5.28E + 06kgCO2e$$

The environmental profile of the timber baseline warehouse B, illustrated on Figure 4.20, is similar to the previous timber design A, with the envelope and floor slab responsible for most of the embodied carbon (37% and 35% respectively). The shares of the foundation pads (21%) and frame (7%) appear to be slightly higher than for the timber baseline A, but this can be attributed to the difference in warehouse dimensions. Again, accounting for biogenic carbon stored in glulam structural elements as a negative contribution results in a reduction of 48% of the total impact compared to considering fossil emissions only.

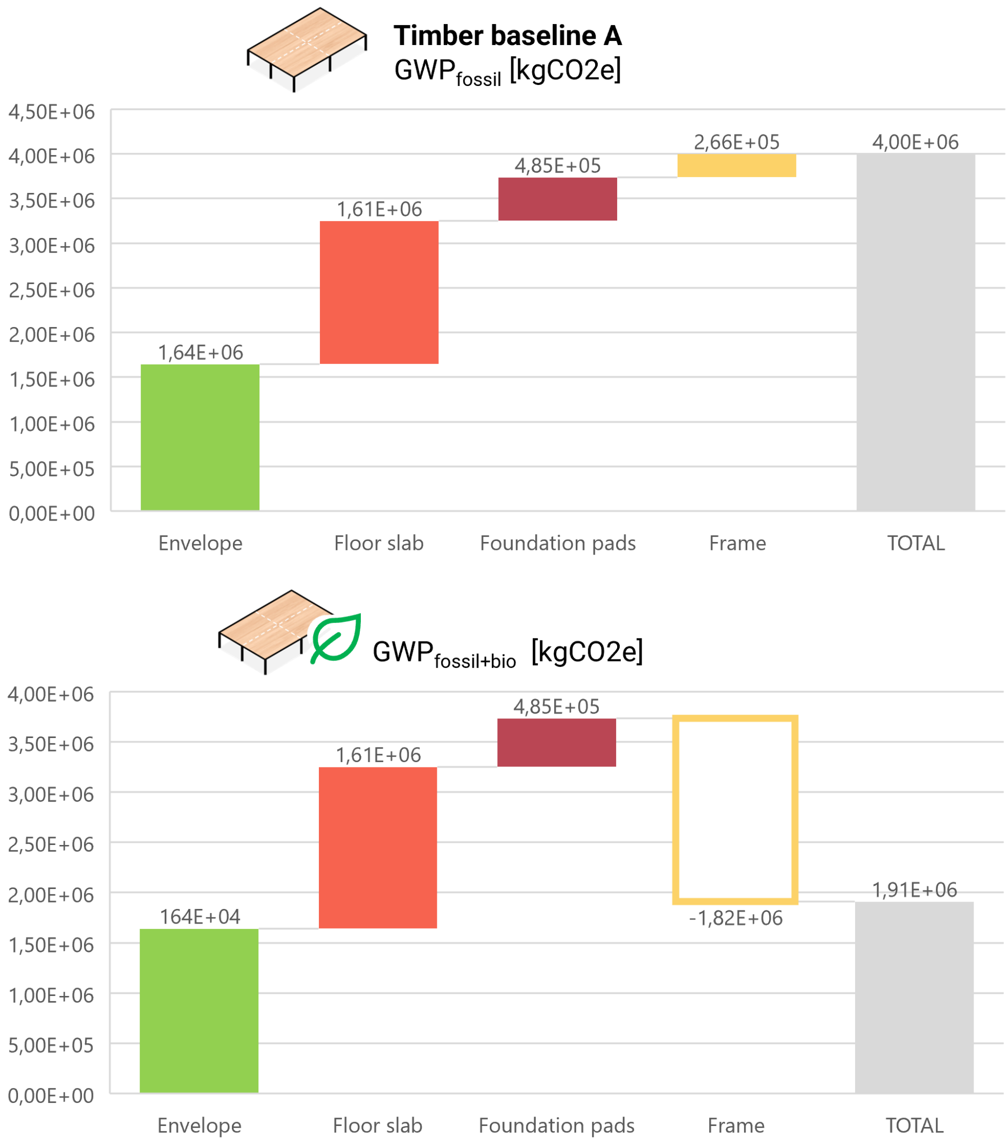


Figure 4.19: GWP_{fossil} and $GWP_{fossil+bio}$ [kgCO2e] of building components of the timber baseline A warehouse

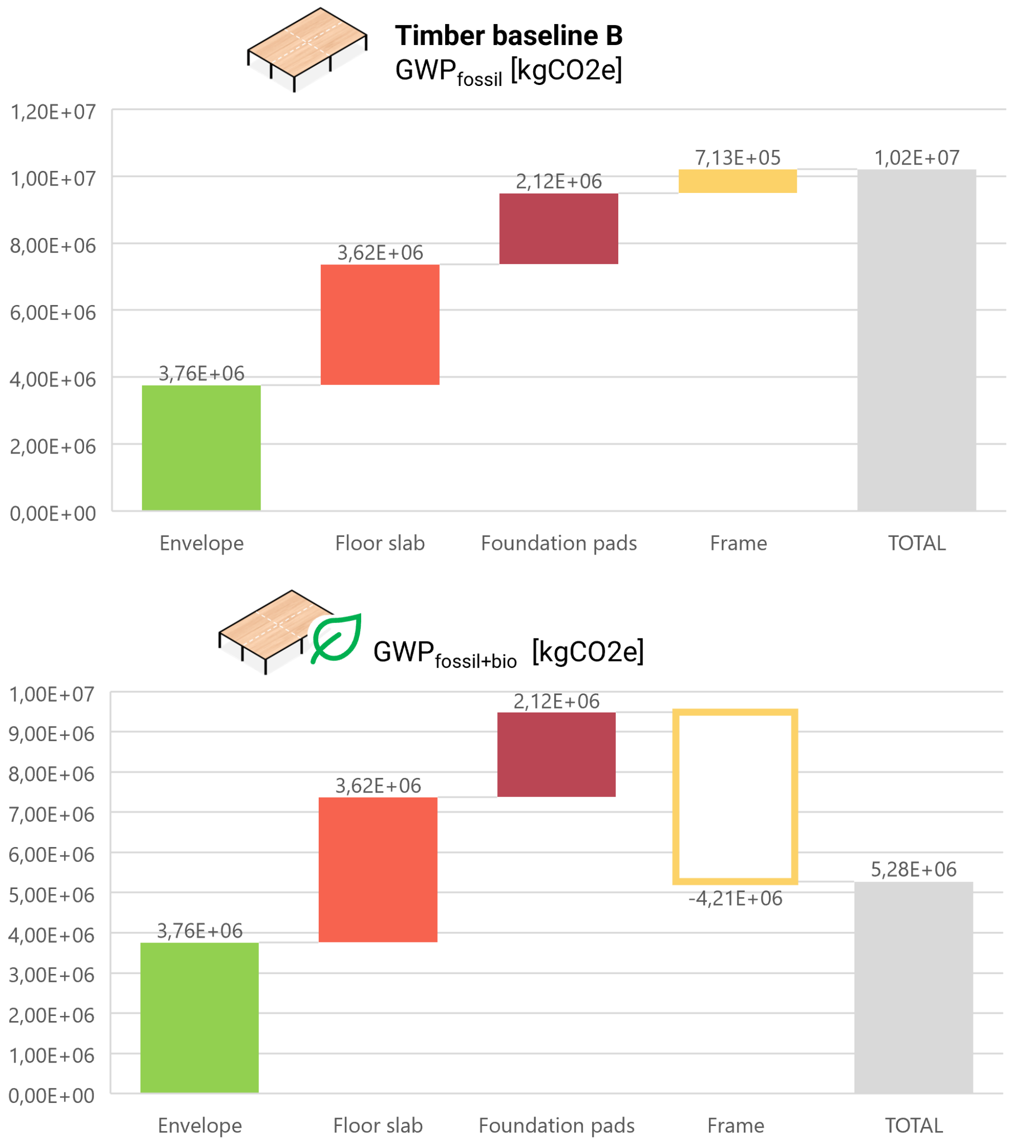


Figure 4.20: GWP_{fossil} and $GWP_{fossil+bio}$ [kgCO2e] of building components of the timber baseline B warehouse

4.4 Conclusion

The goal of first design step was to investigate the benefits of using biobased materials in the frame compared to fossil-based steel or concrete in terms of embodied carbon reduction. The corresponding research question to be answered is: **(2) How much environmental impact reduction can be achieved by substituting steel or concrete by timber in the load-bearing frame of a warehouse?**

4.4.1 Comparing the steel reference with timber baseline design A

The environmental impact of the steel reference warehouse design created by RHDHV is compared to the impact of the timber baseline design A developed in this research project, of similar dimensions. Figure 4.21 shows the comparison of their impact in kgCO2e, and the share of structural elements in the total GWP_{fossil} of each design. The impact of the timber baseline considering biogenic carbon storage is also included in the comparison.

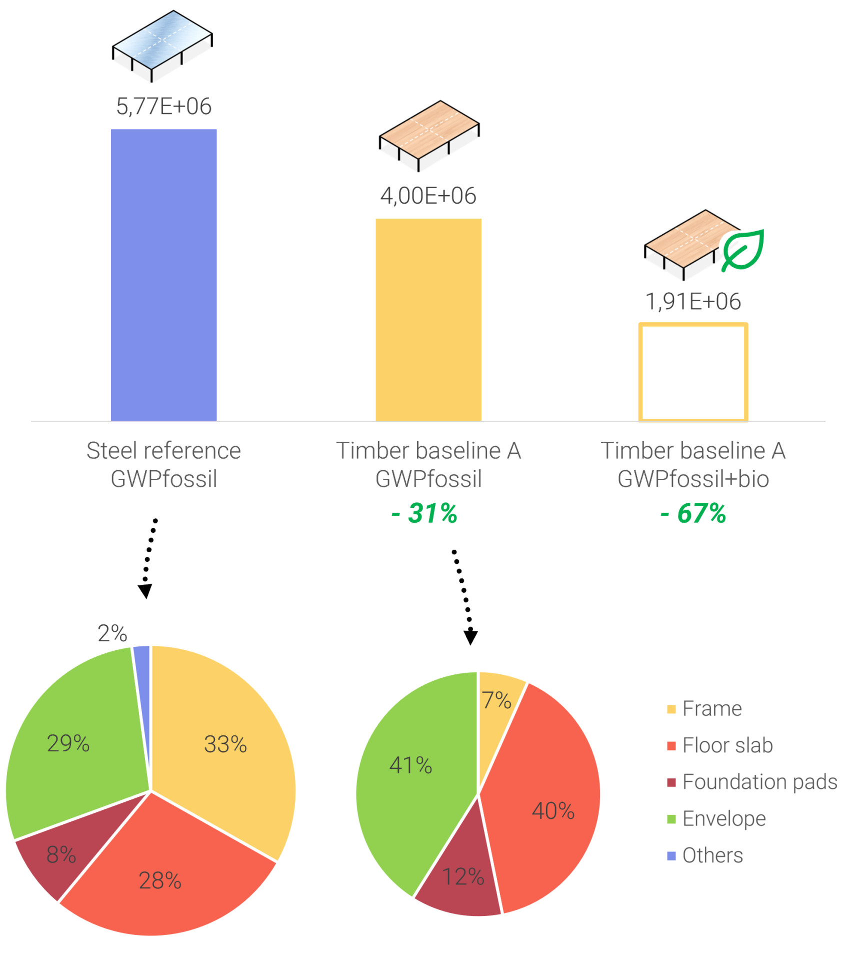


Figure 4.21: Comparison of GWP steel reference vs timber baseline A

Substituting steel by timber in the frame of a warehouse results in a reduction of approximately 31% of the total embodied carbon. By considering the benefits of biogenic carbon storage, the impact score reduction achieved by the timber warehouse compared to the steel design reaches 67%. Looking more precisely at the environmental impact of building components, it can be seen that the share of the frame effectively drops significantly. This is explained by the considerably lower embodied carbon footprint of timber compared to steel, for structural elements of similar performance. As a result of this decrease, the share of the envelope and floor slab in the total impact rises, even though their embodied carbon remains the same.

4.4.2 Comparing the concrete reference with timber baseline design B

The environmental impact comparison of the concrete reference design and the timber baseline B is presented in Figure 4.22.

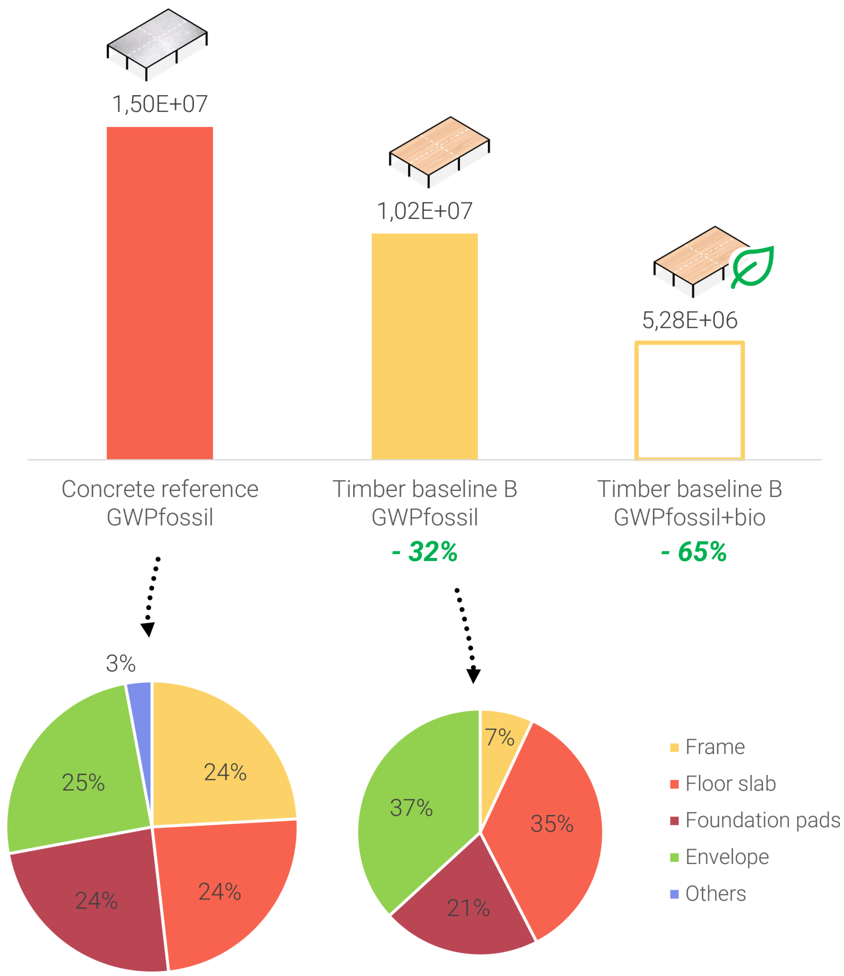


Figure 4.22: Comparison of GWP concrete reference vs timber baseline B

The embodied carbon reduction allowed by substitution of concrete by timber in the load-bearing frame is 32% when only fossil emissions are considered, and up to 65% when biogenic carbon storage in timber is accounted for. Once again, the share of the frame in the total GWP_{fossil} drops significantly when using timber instead of concrete, from representing a quarter of the total for the concrete reference, to only 7% for the timber baseline design B. The impact of the floor slab and of the envelope end up governing the embodied carbon of the timber design.

4.4.3 Discussion

Sensitivity of results to environmental data

The results of environmental impact calculations highly depend on the data used as input, either material quantities or environmental data. In this research, the lowest and highest impact EPDs are selected for each building product or material. The impact scores presented in previous sections correspond to the mean value between these two extremes, but end results may differ depending on the final choice of manufacturer. Figures 4.23 and 4.24 show the possible range in impact scores for the reference and timber designs.

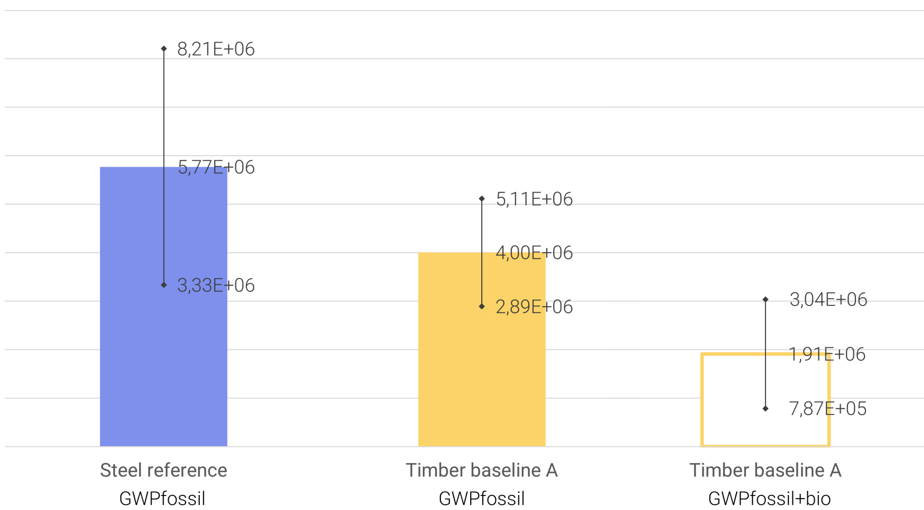


Figure 4.23: Sensitivity of impact score results in the steel vs timber frame comparison, depending on available environmental data

The impact score of the steel reference warehouse may vary by ±42% around the mean value. This range is rather large because of the significant gap in GHG emissions for steel products made of virgin material, or fully recycled. For the timber baseline A, the possible variation is limited to ±28% for fossil emissions, and ±59% with biogenic carbon storage because the mean value is lower. In the worst case scenario with only high impact materials, the influence of steel substitution by timber in the frame is 38% (instead of 31% with mean values), and 13% in the best case where lowest-impact products are used. With biogenic carbon, the impact reductions are 63% and 76% respectively.

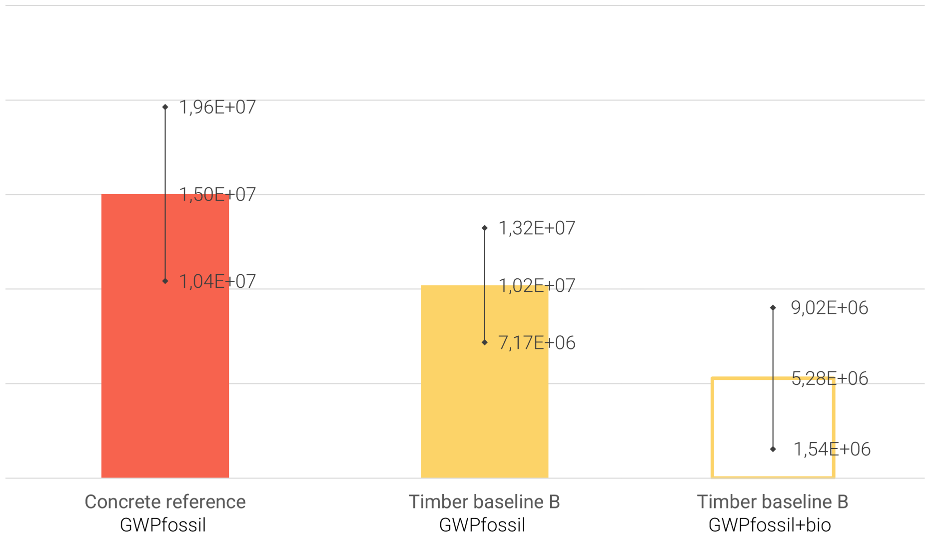


Figure 4.24: Sensitivity of impact score results in the concrete vs timber frame comparison, depending on available environmental data

The impact score of the concrete design may vary by ±31% around the mean GWP_{fossil}, a smaller range than for the steel reference. The impact variation of the timber baseline B is ±30% for fossil emissions, and ±71% considering biogenic carbon. In the worst case scenario, the influence of concrete substitution by timber in the frame is 33% (instead of 32% with mean values), and 31% in the best. With biogenic carbon, the impact reductions are 54% and 85% respectively.

Environmental impact calculation methodology

Environmental impact results only account for the Global Warming Potential indicator in kgCO2e, indicated in EPDs of building products. More complete results would be obtained by considering other impact categories, quantifying specific types of environmental burdens, but each methodology requires a different level of detail and calls for suitable data sources. European LCA methodologies prescribe a minimum of seven impact categories, while the Dutch MPG includes eleven, as detailed in Appendix F. As this project considers products from not only the Netherlands, but all of Europe, some categories prescribed by the MPG methodology are not necessarily included in the selected EPDs. Additionally, the MPG does not specify biogenic carbon and is therefore less relevant to this study than European EPDs (van Wijnen, 2020).

5

Design step 2 - Further reducing the impact of a timber warehouse

The second design step aims at investigating the most effective strategies to further reduce the environmental impact of a timber warehouse developed in Chapter 4. To do this, a number of sustainable design strategies will be investigated, specifically targeting building components responsible for the most embodied carbon, or so-called ”environmental hotspots” identified in section 5.1. Then, each design strategy will be carried out in sections 5.2 to 5.5. The impact reduction achieved with these variants will be evaluated to compare with the timber baseline warehouse.

5.1 Environmental hotspots in timber baseline design A

This section answers the following research subquestion: *Which structural elements of the timber baseline warehouse have the most influence on its total environmental impact?* The reference in this second design step is the baseline timber warehouse A. An overview of its environmental profile is presented on Figure 5.1.

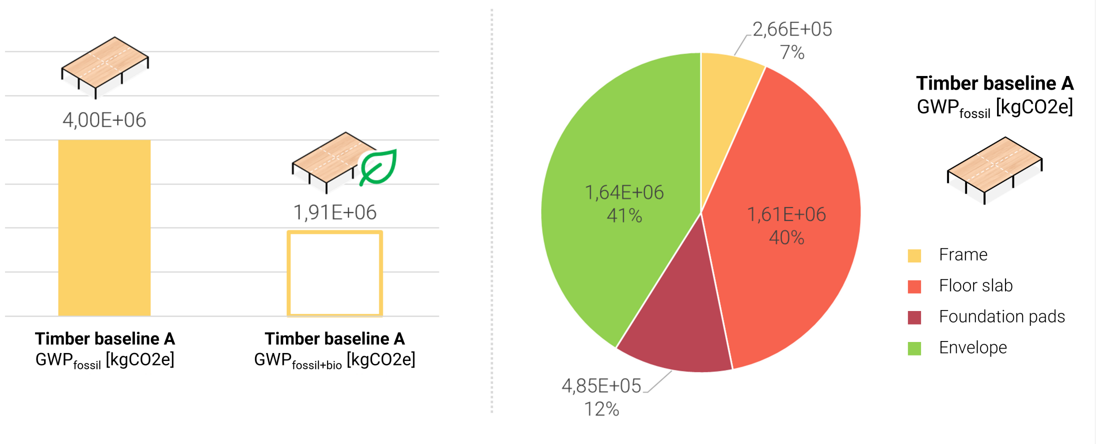


Figure 5.1: Detailed environmental profile of the timber baseline design A

The objective here is to reduce further the environmental impact of timber baseline warehouse A, created in the previous chapter to compare with the steel reference design. Sustainable strategies to be applied should be selected and ordered according to the highest potential for embodied carbon reduction. Environmental hotspots refer to components responsible for the largest share of the total environmental impact of the building. In the case of the timber baseline A and considering the scope of this research, the top elements are the floor slab (40%) and envelope (41%), followed by the foundation pads (12%) and timber frame (7%).

Strategy A - Floor slab

The 200mm thick floor slab was unchanged compared to the steel and concrete reference designs, but now comes out as an priority to effectively reduce the total embodied carbon in this second design step. This could be done by reducing the volume of concrete with adjusted slab thickness, or using low-carbon concrete alternatives.

Strategy B - Building envelope alternatives

For the first step of the research, the focus on facade and roof systems was limited: the same panels and supports were assumed for all designs, only the material of the main load-bearing frame was changed. For the second step of the analysis, however, the choice of envelope becomes a relevant variable in sustainable design strategies. In practice, improving the insulation of buildings contributes to lowering operational carbon, but can be associated with higher embodied carbon. The scope of this study excludes operational emissions during the use phase: for the sake of fair comparison between design alternatives, envelope options should exhibit consistent U-values to ensure similar thermal performance. On this basis, possible levers for impact reduction lie in the bill of materials and resulting embodied carbon. Different insulating materials can be used in sandwich panels, the most common option in warehouse envelopes. After substituting steel by timber in the main load-bearing frame, the same can be done for envelope supports (purlins, mullions) to take advantage of biogenic carbon storage. This strategy can be taken further by completely replacing sandwich panels with biobased build-ups on the roof and facades. For each envelope alternative, additional dead loads on the warehouse frame may increase the required size of beams and columns.

Strategy C - Green envelope

Roofs are considered as external works, open areas which are not used for any activity and thus can be easily accommodate plants to improve the overall environmental footprint of a building. Facades can also accommodate green systems, either plants directly climbing on the existing facade, or indirectly on supports added to the envelope. The embodied carbon of green systems and higher dead loads on the load-bearing frame may increase the negative impact of a building, but ecosystems services provided by vegetation are an added value throughout its service life.

Strategy D - Timber frame

Sustainable design strategies for the foundation pads and timber frame are discussed as general ideas for designers, as they only represent a limited share of the total impact score. The impact of foundation pads could be reduced by limiting the design loads to withstand, but they are dictated by external factors, namely the type of frame and envelope. Material substitution of steel and concrete by timber in the frame is already an effective first step towards embodied carbon reduction. Possible solutions to further reduce its impact would be to detail connections for disassembly, and minimising material use. The first measure increases the reuse potential of timber structural elements, and therefore the duration of biogenic carbon storage beyond the building's service life. The second strategy implies modifying the size of structural elements either designing varying cross-sections, limiting spans, or completely re-thinking the structural system of the sway frame. The frame variants can be investigated using parametric design tools to combine structural design and environmental impact assessment.

The following sections detail each of these sustainable strategies, to answer the following research subquestion: *How much environmental impact reduction can be achieved by choosing different design options for the structural elements of a timber warehouse, compared to the baseline design?*

5.2 Strategy A - Floor slab

The concrete ground floor slab represents 40% of total embodied carbon for the timber baseline design A. Two strategies appear relevant for reducing its impact: using less material by varying the thickness of the floor slab, or using low carbon concrete alternatives. A range of 50mm over and below the reference 200mm are investigated to determine the influence of floor slab thickness on the total embodied carbon of the warehouse, as a reference for designers. The results of this study are shown on Figure 5.2.

Varying floor slab thickness

The characteristics of a building's substructure are mainly driven by performance criteria. In practice, the thickness of a concrete floor slab on grade is determined by the stiffness properties of the soil, and structural requirements to comply with design loads from activities taking place inside the warehouse during its service life. The thickness of the slab should be sufficient to detail adequate reinforcement to fulfil these requirements. In most standard designs of distribution centres developed by RHDHV, the thickness of the slab can even be as high as 250mm. In this project, the reference slab thickness of 200mm is arbitrary and could be adapted depending on project-specific requirements, either increased or decreased. The bill of materials of each alternative is presented in Table 5.1.

Table 5.1: Bill of materials - Floor slab step 2 varying thickness

Element	Type	A [m^2]	Weight [kg/m^2]	Weight [kg/m^3]	Total [m^3]	Total [kg]
Floor slab	150mm C30/37	23040	375.0	2500	3456.0	8.64E+06
Steel rebar	Steel, 120kg/m^3 concrete			7850	52.8	4.15E+05
Floor slab	175mm C30/37	23040	437.5	2500	4032.0	1.01E+07
Steel rebar	Steel, 120kg/m^3 concrete			7850	61.6	4.84E+05
Floor slab	200mm C30/37	23040	500.0	2500	4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete			7850	70.4	5.53E+05
Floor slab	225mm C30/37	23040	562.5	2500	5184.0	1.30E+07
Steel rebar	Steel, 120kg/m^3 concrete			7850	79.3	6.22E+05
Floor slab	250mm C30/37	23040	625.0	2500	5760.0	1.44E+07
Steel rebar	Steel, 120kg/m^3 concrete			7850	88.1	6.91E+05

Overall, every 10mm of ground floor slab corresponds to approximately 2% of the timber warehouse total GWP_{fossil}. For example, reducing the floor slab thickness from 200mm to 150mm reduced the embodied carbon of the warehouse by 4.02E+05kgCO2e, that is 10% compared to the baseline timber A. The total impact reduction associated with a thinner slab will be even more significant for warehouses having larger ground areas.

Using low carbon materials

It is particularly interesting to use concrete alternatives for ground floor slabs where performance requirements are generally less restrictive than for foundations or other structural elements (LETI, 2016). Such alternatives include low carbon concrete with highly recycled reinforcing steel, replacing part of the Portland cement fraction with fillers or virgin aggregates with recycled materials. In this study, environmental data from EPDs are meant to be representative of low and high impact materials: concrete from Portland cement only or replacing 60% by GGBS, and varying recycled steel fractions in rebars. These variations in data are translated by whiskers around mean results in Figure 5.2.

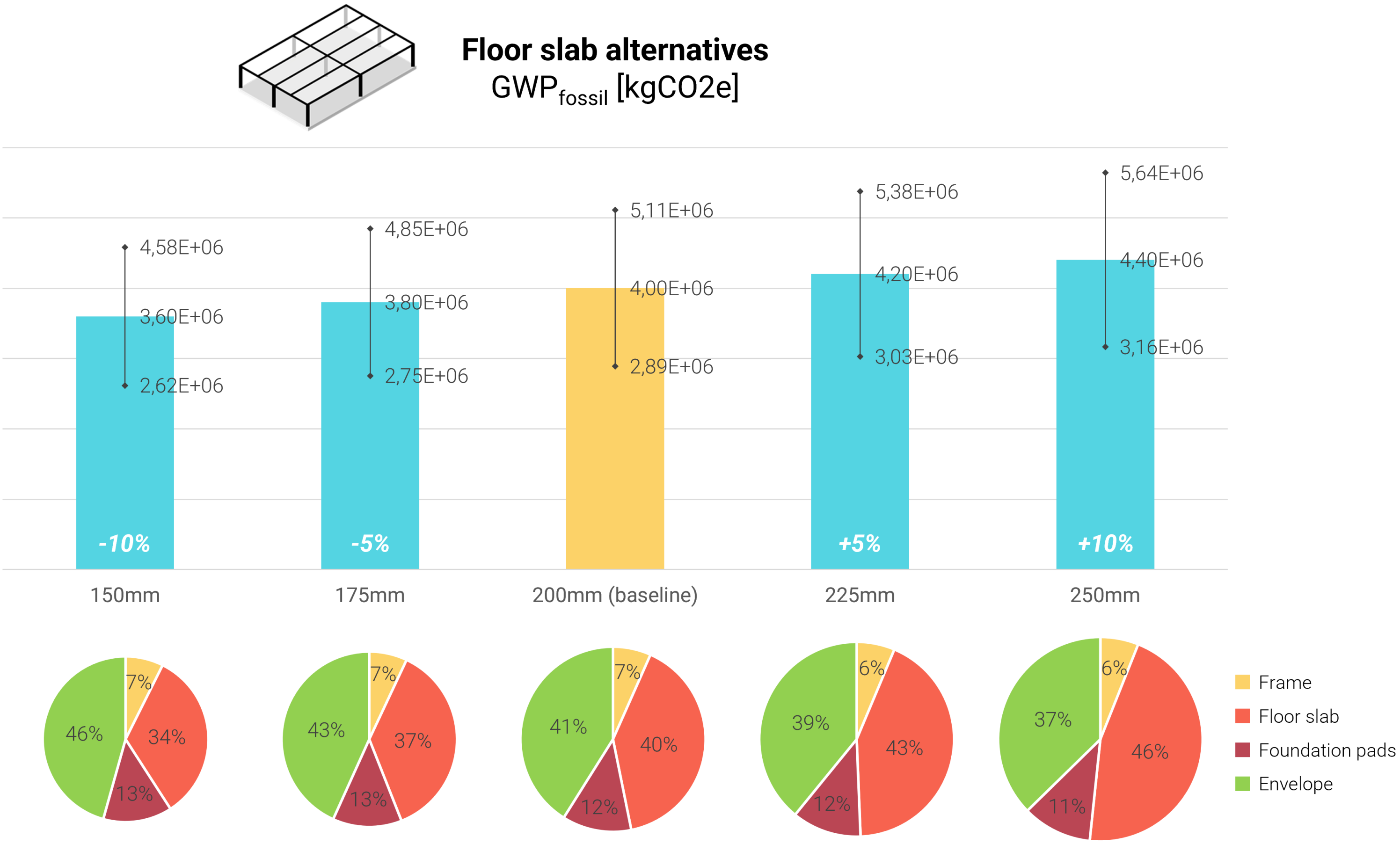


Figure 5.2: Influence of varying the thickness of the floor slab on GWP_{fossil} of a timber warehouse

5.3 Strategy B - Building envelope alternatives

The baseline envelope, composed of Kingspan sandwich panels on steel supports covering the roof and facades, represents 41% of total embodied carbon for the timber baseline design A. This section aims at investigating several envelope alternatives to lower the total impact of the warehouse, excluding green for now.

5.3.1 Basics of envelope design

5.3.1.1 Functional requirements for the building envelope

The envelope acts as a barrier and filtering layer between the outside and inside, protecting the interior building space against external climate effects to ensure favourable conditions for usage and comfort of occupants (E. Schunck et al., 2003). Multi-layer facade and roof elements combine different materials of varying thicknesses, ordered in specific sequences to meet a number of functional requirements:

- **Thermal insulation:** Stationary air inclusions or insulating materials protected against moisture should maintain comfortable temperature range and relative humidity, with minimal fluctuations. For a timber structure in particular, the moisture level of structural members shall remain under a defined critical level to ensure functionality. Air cavities help carrying away heat and moisture, acting as additional thermal insulation layer. Air barriers outside of thermal insulation layers or overlapping joints between panels increase air tightness of the envelope. Materials with good heat capacity also help regulating interior climate.
- **Aeration:** Inside the building, adequate fresh air change rate and comfortable air velocity should be ensured. The resistance to water vapour diffusion should decrease from inside to outside to prevent water condensation within the facade (vapour trap) and allow evaporation.
- **Protection against weathering:** Absorbent materials should be protected against driving rain and frost. Complete evaporation and drainage of facade runoff water should be possible. External shading devices can reduce radiation absorption of permeable layers.
- **Sound insulation:** Comfortable sound levels should be maintained inside.
- **Aesthetics:** The external aspect of the envelope is important for creating a visual relationship with external surroundings
- **Fire protection:** One of the most critical functional requirements for timber structures is fire resistance, the envelope should also contribute to this matter.
- **Services:** Facade and roof systems can also accommodate building services like air ducts to keep interior spaces clear of obstacles.

5.3.1.2 Facade and roof typologies

A roof or facade panel is formed by connecting layers together, to create a single assembly capable of fulfilling all relevant requirements at once. Constitutive materials for the envelope of a building are chosen based on the loads and stresses they should withstand, but also construction and detailing methods, and the expected lifespan of the building.

Prefabricated panels

On the building site, prefabricated facade elements reduce the erection time and are fully functional from the start, as mechanical assembly of layers takes place in the factory. They are particularly suited for large buildings with a regular structural system, like warehouses. The size of individual prefabricated panels dictates the dimensions and spacing of supporting structural elements like mullions. They are also easier to dismantle at the end-of-life. Typical examples of prefabricated elements used for commercial halls are insulated sandwich panels, like those manufactured by Kingspan and used for all design alternatives of design step 1. These all-in-one products are generally made from two steel sheets surrounding a layer of insulation materials, to form a single panel providing thermal insulation, protection against the exterior climate, and structural stiffness to span between supports. Design alternatives are restricted to products available on the market, with different insulation materials (polyurethane, mineral wool...) or steel sheet thicknesses.

Build-up panels

Alternatively, build-up facade and roof systems can be created by arranging series of independent layers together. This option offers more flexibility for designers in the choice of materials, thickness of layers and their order. Build-up systems also allows for more freedom regarding the supporting structural system, as stiffer elements can be created to span larger distances and remove the need for supports. They are particularly suited for projects with specific functional requirements, or when all-in-one solutions with uncommon materials do not exist yet on the market. This is often the case for biobased envelope alternatives, as more and more insulation materials (wood fibres, hemp, straw...) and wood-based panels become available, but all-in-one systems remain rare. Replacing fossil-based materials by renewable biobased alternatives in the envelope has the same advantages as applying this strategy in the frame, including biogenic carbon storage.

Supports and load-bearing capacity

Facades transmit vertical and horizontal loads to the main load-bearing structure (columns, roof, foundation), including dead loads, snow loads, wind loads, imposed loads or restraint forces caused by thermal or moisture-related displacements. Similar considerations apply to roof elements, which should transfer vertical loads to the main structural frame underneath. Design loads dictate the allowable span of panels, corresponding to the maximum spacing of supports. The size of metal or wood studs is directly related to the dimensions of sheathing materials and the design loads they should withstand.

5.3.2 Building envelope design alternatives

This section briefly presents the envelope alternatives considered in this step. If the envelope system is heavier than the maximum weight considered in the baseline, the structural design of the load-bearing timber frame is adjusted to support additional dead loads following the procedure explained in Appendix E.

5.3.2.1 Baseline envelope

Sandwich panels can be manufactured with different insulation materials. In the baseline envelope design, panels comprise a layer of "Quadcore" insulation, a material developed by Kingspan derived from PUR with improved thermal properties, between two layers of steel sheets. Kingspan roof elements are laid on steel purlins spanning 8 metres in the transverse direction, fixed to the top of longitudinal glulam roof beams every 3 metres. Kingspan facade elements are laid horizontally over vertical steel mullions every 4 metres. The complete design process and results are detailed in Appendix E.

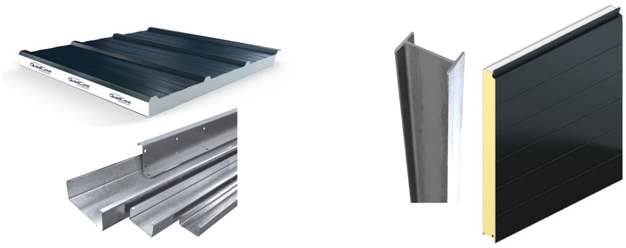


Figure 5.3: Baseline envelope - Kingspan sandwich panels on steel supports

5.3.2.2 Changing the insulation material in sandwich panels

Two sandwich panels are studied here, comparing the embodied carbon of Quadcore insulated panels with PUR and mineral wool insulation. The spanning capacity and dead weight of the sandwich panels define the size and spacing of steel supports, as detailed in Appendix E. The total weight of the system is calculated to determine if the size of primary timber structural members should be increased.

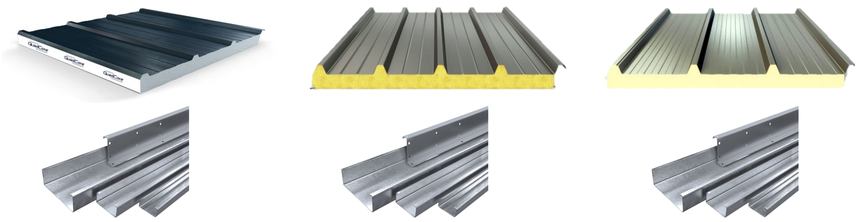


Figure 5.4: Design alternatives for sandwich panels on steel supports, insulation material from left to right: Kingspan Quadcore 100mm, polyurethane (PUR) insulation 120mm, mineral wool (MW) insulation 200mm

5.3.2.3 Replacing steel supports by timber

The same sandwich panels alternatives are considered but here, steel supports are replaced by rectangular glulam purlins and mullions, designed in Appendix E.

Figure 5.5: Design alternatives for sandwich panels on timber supports

5.3.2.4 Replacing sandwich panels and steel supports by biobased elements

Biobased roof and facade build-ups are created as alternatives to sandwich panels, limiting the use of fossil-based materials to external protection layers only. Lignatur roof panels can span 8 metres between longitudinal roof beams, without additional supports.

Figure 5.6: Biobased envelope build-up: Lignatur panels, wood fibre insulation, EPDM for the roof (left) and glulam mullions, OSB panel, wood I-joists, wood fibre insulation, steel sheathing for the facade (right)

5.3.3 Results

Figure 5.7 compares the total environmental impact of all design alternatives (envelope, frame, foundation pads, floor slab) within a bar diagram, showing GWP_{fossil} in solid color and $GWP_{fossil+bio}$ in white. Percentages represent the difference between the baseline design and each alternative. Pie charts give more insight into the best alternatives and the share of structural elements in their total embodied carbon.

Changing the insulation material in sandwich panels

Sandwich panels on steel supports have the lowest impact for PUR insulation, as they weight approximately the same as baseline Kingpsan panels. Therefore, steel purlins and the timber frame remain the same, and the lower embodied carbon of PUR sandwich panels makes this solution advantageous. On the contrary, MW sandwich panels are heavier and therefore require larger steel supports and members in the timber frame, increasing the total GWP compared to the baseline Quadcore insulated panels.

Replacing steel supports by timber

Using glulam supports decreases slightly the total embodied carbon of the building compared to an alternative with similar panels and steel supports, by approximately 4-5%. However, it is clear when accounting for biogenic carbon storage in glulam supports that these alternatives contribute to reducing the impact of the envelope, and consequently of the entire warehouse. As expected from the last round of sandwich panel alternatives, the lowest impact is achieved with the PUR sandwich panels on glulam supports.

Replacing sandwich panels and steel supports by biobased elements

Using biobased panels, supported by glulam elements in the facade and self-supported in the roof, is the most effective solution for reducing the embodied carbon of the envelope compared to the baseline (19% reduction). The EPDM layer on the roof is assumed replaced after 40 years, therefore impact calculations account for twice the amount of material in the total service life of the building. The amount of steel is greatly reduced, as only the external layer of facades in protected by a steel sheet instead of the double faced sandwich panels. In the end, the share of the envelope in the total GWP_{fossil} drops to 27% instead of 41% in the baseline design. Moreover, biobased insulation materials contribute to biogenic carbon storage, to achieve a negative footprint of the building when taking it into account (169% reduction).

5.3.4 Discussion

A number of assumptions were taken in the design of envelope alternatives, possibly influencing the impact results:

- Design alternatives are meant to give a conservative estimate of the volume of materials used in the envelope. The dimensions of panels and supports are based on preliminary guidelines provided by manufacturers, or hand structural calculations for glulam supports. The spacing of supports in particular was often underestimated to fit several alternatives at once and could be refined, for instance by precisely adjusting them to the spanning capacity of panels to save material.
- Environmental data is less easily available for specific products like sandwich panels, therefore the minimum and maximum GWP are often close, or equal when only one EPD could be found.
- For biobased materials in particular, more and more products are becoming available on the market as sustainable regulations tighten for the building sector and the demand increases. The biobased facade build-up in this study is just an example of what could be done in practice, to assess the benefits of carbon storage in the envelope materials, but could definitely be optimised for specific technical requirements or replaced by all-in-one alternatives if available.
- In build-up systems, specific attention should be given to the lifespan of individual elements, to anticipate the need for any replacements during the design service life of the building.

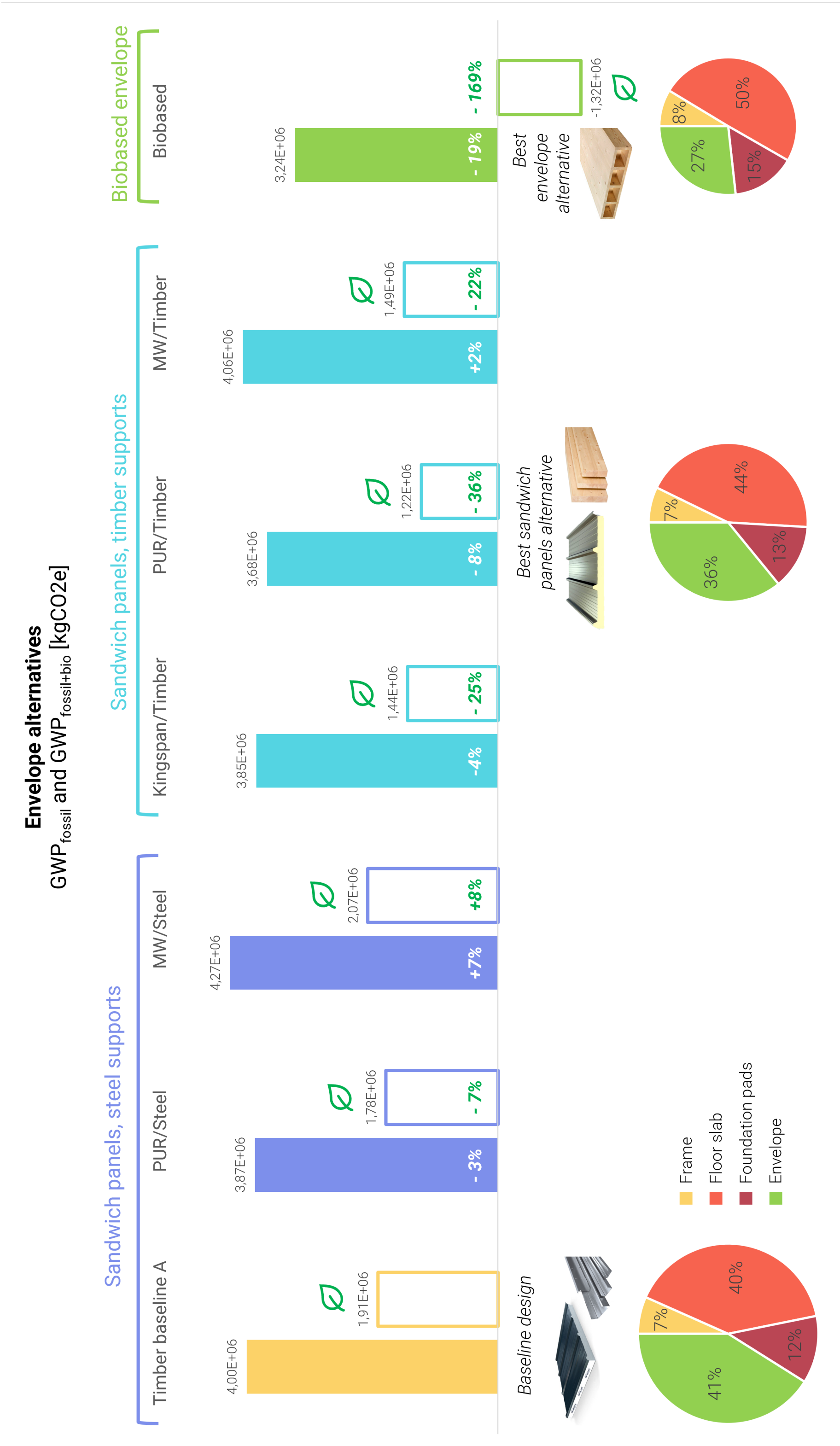


Figure 5.7: Influence of changing envelope materials on GWP$_{fossil}$ and GWP$_{fossil+bio}$ of a timber warehouse

5.4 Strategy C - Green envelope

After focusing on materials within the envelope itself, this next strategy proposes adding vegetation around it to benefit from ecosystem services, as described in Section 5.4. A large variety of greening systems exist today on the market, either vertical to be applied on facades or horizontal for roofs. By investigating the impact of a green envelope on a warehouse, this section in particular aims at answering the following research subquestion: *When does a green envelope contribute to reducing the environmental impact of a warehouse?*

5.4.1 Vertical greening options

An overview is given in Figure 5.8 of the main types of vertical greening systems.

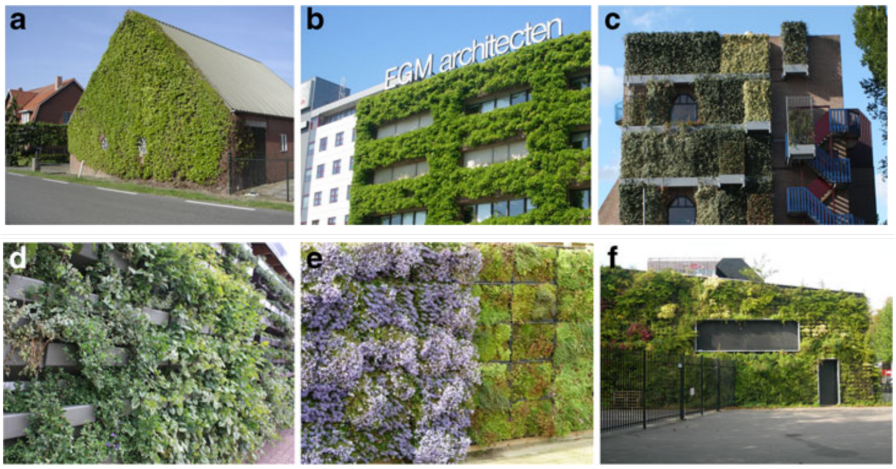


Figure 5.8: Vertical greening systems: Direct greening system (a), indirect greening system (b), indirect greening system combined with planter boxes (c), LWS based on planter boxes (d), LWS based on foam substrate (e), LWS based on felt layers (f) (Perini et al., 2013)

- **Direct facade greening [5 kg/m^2]:** Climber plant species are often used to create green facades at low price, planted at the base of the building and directly attached to the vertical surface as they grow. They require little maintenance, and a vertical growth of 1 metre per year can be expected for the fastest species, up to 10-25m total height. Evergreen or deciduous species can be applied.
- **Indirect facade greening [<10 kg/m^2]:** The same climber plants can also be supported by cables or trellis to create a vertical vegetation layer that is separate from the building's facade elements. The supporting structure can be made from different materials depending on functional and aesthetic requirements. Among the most popular, stainless steel meshes or ropes come in various sizes depending on the plant species and facade area. This choice determines the weight of the green system applied on the load-bearing structure, and its environmental impact.

- **Living Wall Systems (LWS) [>100 kg/m^2]:** When an indirect greening system is combined with planter boxes with insufficient rooting space, it requires nutrients and a watering system, creating a Living Wall System (LWS). They are modular solutions for creating green walls, from soil or artificial growing mediums. Heavier than other greening options and demanding high maintenance, they nevertheless offer more variety by allowing evergreen shrubs as well as climbing plants, resulting in a higher potential for creativity, aesthetics and natural added value.

5.4.2 Horizontal greening options

Green roofs are generally classified according to the substrate depth, related to their dead weight, and the type of plants that can grow on them.

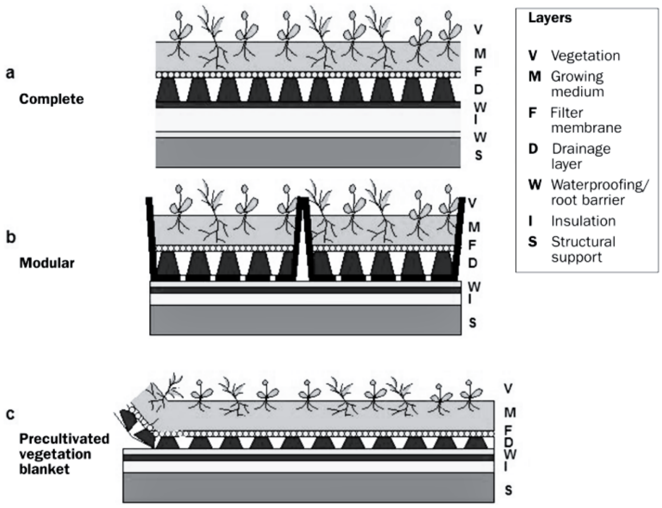


Figure 5.9: Extensive green roof systems (Oberndorfer et al., 2007)

- **Extensive green roof [50-150 kg/m^2]:** Their light weight and relatively shallow build-up height makes them the easiest horizontal greening solution to implement on existing or new structures. Covered by sedum species, herbs and grasses, they require low levels of maintenance. Different types of extensive greening systems are illustrated on Figure 5.9.
- **Intensive green roof [>150-300 kg/m^2]:** Their substrate depth is substantially higher to accommodate for a larger diversity of plants, and even trees depending on the project. They are similar gardens on top of a building, creating recreational space for users. Besides social benefits, the advantages of such systems also lay in a better retention and delay of storm water, temperature control and potential food production. However, as they require more weight and a deeper system built-up, as well as high maintenance, they may not represent feasible solutions for all projects.

5.4.3 Green envelope design alternatives

Loads requirement

Lightweight solutions can easily be implemented on existing or new building, as they require low maintenance and almost no change to the main load-bearing frame. Heavier systems may be suited for projects specifically designed for a green envelope, or on existing structures reinforced for this purpose. These demanding structural requirements are nevertheless balanced by a higher added value for biodiversity or better thermal insulation, among others. Manufacturers often offer custom services for adapting green envelopes to the requirements of each specific project. In this research, only lightweight greening systems are considered for the envelope of the timber warehouse.

Selected greening systems

For facades, the scope is limited to indirect systems where climber plants are attached to a vertical steel mesh, considering the volume of stainless steel supports in the environmental impact assessment of the envelope.

Only extensive green roofs are considered in the design, with dead weights varying from 50 to 150 kg/m^2 to limit additional loads on the timber frame. The embodied carbon of roof layers, as well as the increase dimensions of timber framing elements to support the dead loads should be accounted for in the environmental impact assessment of the envelope.

Existing warehouse envelope

Two alternatives are considered for the existing envelope of the timber warehouse: the baseline envelope, using Kingspan sandwich panels on steel supports, and the biobased envelope created in Strategy B, which resulted in the highest embodied carbon reduction of all envelope designs. The dimensions of structural elements are adapted to resists dead loads of 50, 100 and 150kg/m^2 from possible extensive green roof systems. The detailed bill of materials for each design is presented in Appendix E.

5.4.4 Results

The results of environmental impact calculations are shown in Figure 5.10. The total GWP_{fossil} of the warehouse is shown in solid color, and $GWP_{fossil+bio}$ in white. Percentages represent the difference between the baseline design and each alternative.

The impact of greening systems is the same for all dead weight options, as higher loads are attributed to substrate depth. Most of the additional impact is related to steel or plastic-based materials used as green facade supports or layers in green roofs, rather than plants or substrate. The load-bearing frame also has to be adapted to resist higher dead loads from the green envelope, increasing material use. Therefore, environmental impact variations mainly translate the consequences of higher dead loads on the design of the frame and of the envelope.

Greening the baseline envelope

The maximum centre to centre distance between steel purlins decreases as higher loads are applied on roofing panels, resulting in more material to be used for supports. Added to the embodied carbon of greening systems, this results in GWP_{fossil} increase between 2 and 7%. On the contrary, because the frame needs larger elements to support higher loads and thus higher volumes of timber, $GWP_{fossil+bio}$ decreases considerably.

Greening the biobased envelope

Similar conclusions can be drawn when the biobased envelope alternative is installed on the frame, underneath the green layer. In this case, the dimensions of Lignatur panels and the amount of wood fibre insulation are adapted to resist higher loads, increasing the amount of biogenic carbon stored in biobased materials, thus further decreasing the $GWP_{fossil+bio}$.

Ecosystem services provided by vegetation

Adding green to the envelope of a warehouse adds environmental value to the structure, thanks to ecosystem services provided by the vegetation layer. Distribution centres are generally built in rural areas, close to highways connecting cities to easily transfer goods and quickly deliver to consumers. Plants contribute to reducing fine dust levels from traffic, particularly relevant in this context, therefore improving air quality and reducing pollution (Perini et al., 2013).

Usually, the aesthetics of external envelope layers clash with the natural surroundings. Greening the envelope first mitigates the negative visual impact of these huge buildings, to integrate them better in their environment. The aesthetics of vegetation have been proven to enhance the well-being of users as well. Additional layers around the envelope also improve the durability of layers underneath by protecting them from external climate like wind, rain and snow, potentially avoiding the need for replacements during the service life of the building. The construction and use phase of the warehouse disturb existing ecosystems. Green roofs and facade contribute to enhancing the biodiversity throughout the service life of the building, providing nesting opportunities for microorganisms and small animals (bees, bats, birds...), hence reducing habitat losses from urbanisation and human activities (Stache et al., 2019).

A single green roof generally uses several species of plants, appropriate to local climate conditions, because diversity helps maximising environmental benefits. Sedum extensive green roofs are of the most popular solutions, as they provides high shading against solar radiation while requiring only limited amounts of water to thrive. However, they have a lower thermal resistance than heavier extensive roofs planted with grasses. Extensive roofs are can mitigate water runoff by 25-60%. Stormwater retention is a non-negligible advantage for warehouses surrounded by waterproof layers and bitumen parking areas all around, to delay potential flooding episodes (Berardi et al., 2014).

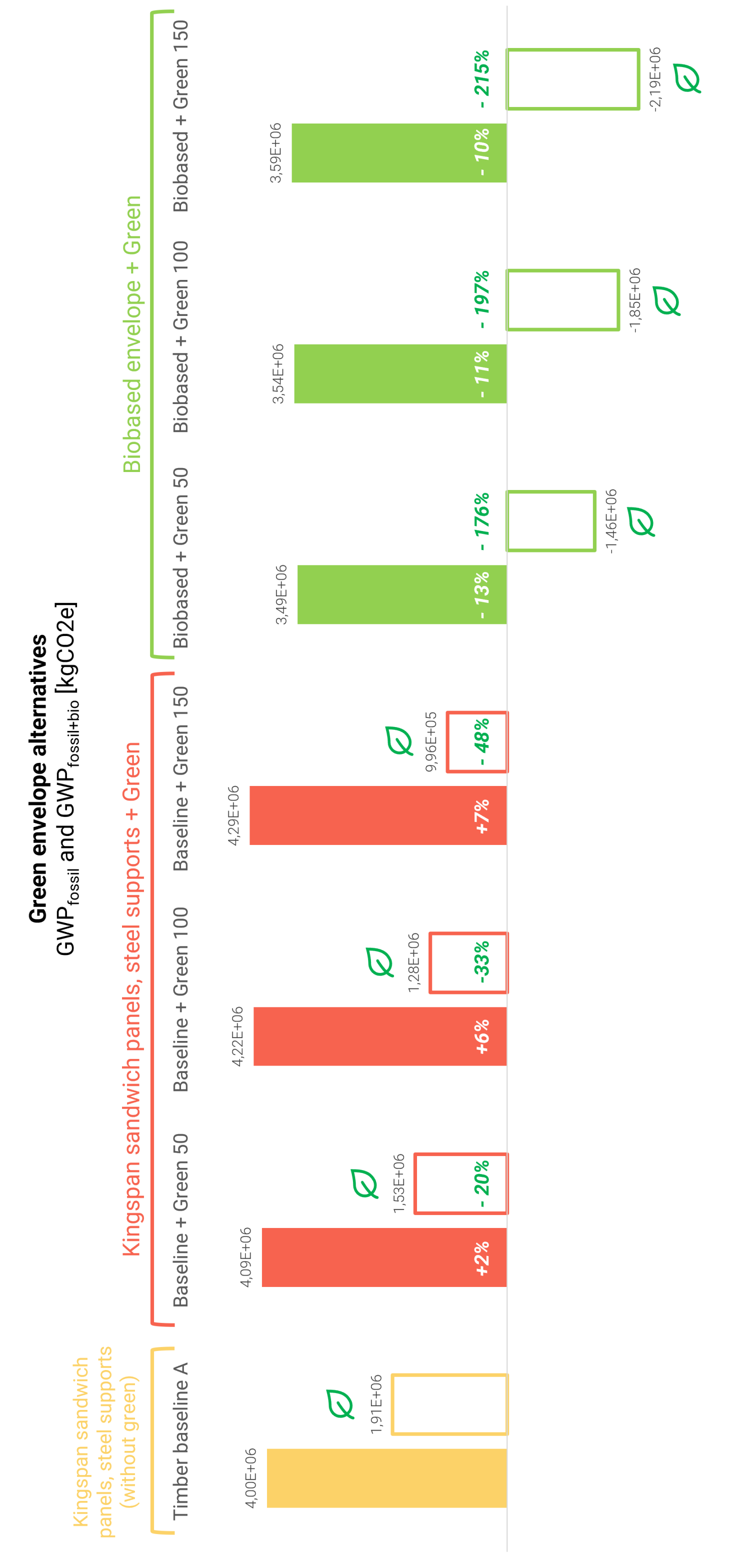


Figure 5.10: Influence of adding a green envelope on GWP$_{fossil}$ and GWP$_{fossil+bio}$ of a timber warehouse, considering the embodied carbon of greening systems and stronger frame alternatives

5.4.5 Discussion

Assumptions in the design and impact calculations of green alternatives are discussed in the following points:

- Benefits from ecosystem services are not quantified within the scope of this thesis, but for the balance to be positive, they should at least overcome environmental costs from extra materials required in the installation. In the end, the choice falls to designers to weigh costs against benefits.
- Indirect facade greening using steel mesh supports was chosen in this study, to give an idea of the maximum impact of a vertical green system on a warehouse. Opting for a direct facade greening system instead would suppress the embodied carbon from stainless steel and result in a lower overall impact.
- There is only limited data available to evaluate the environmental impact of green systems. Only a few green roof products have their own EPDs, two of which were used here, but the impact of different build-ups could also be calculated by hand, considering the impact of each layer separately for more precise results. This method was actually carried out for the facade systems, as no EPDs were found and the volume of stainless steel was roughly estimated by hand.
- The addition of vegetation around the envelope was only considered in terms of additional loads on the structure. Possibly enhanced thermal insulation from the green layer could be evaluated to reduce the thickness of insulation materials in the envelope accordingly, and save material.

5.5 Strategy D - Timber frame

The last sustainable design strategy proposes structural modifications of the load-bearing timber frame, to encourage reuse and minimise material use.

5.5.1 Demountable base connection

With appropriate detailing of connections, the timber frame may be dismantled to save individual timber elements at the end-of-life of the building, provided structural members retain sufficient residual value to be reused. Examples of demountable connections in timber structures include screwing elements to simply unscrew them later. In the timber baseline design, connections at the base of columns were designed to provide rotational stiffness, using glued-in rods within the glulam column, running down into the concrete foundation pads. Glued connections need to be broken apart, but designing with large columns elements allow to get around the problem of disassembling tricky connections by simply cutting out smaller elements to reuse the material at the end-of-life of the building, without running into connectors (Campbell, 2019).

The disassembly process can be facilitated by detailing the connection to be demountable from the design phase. The baseline detail can be modified by adding a base plate between the column base and the concrete foundation pads, as illustrated on Figure 5.11. The rods are still glued in the column base, but instead of extending into the foundation they are welded to the base plate. Then, the plate is fixed to the foundation by means of anchor rods. At the end-of-life, the connection between the anchor rods and the base plate can easily be dismantled to reuse the entire timber column in a new construction project without sawing it above the rods.

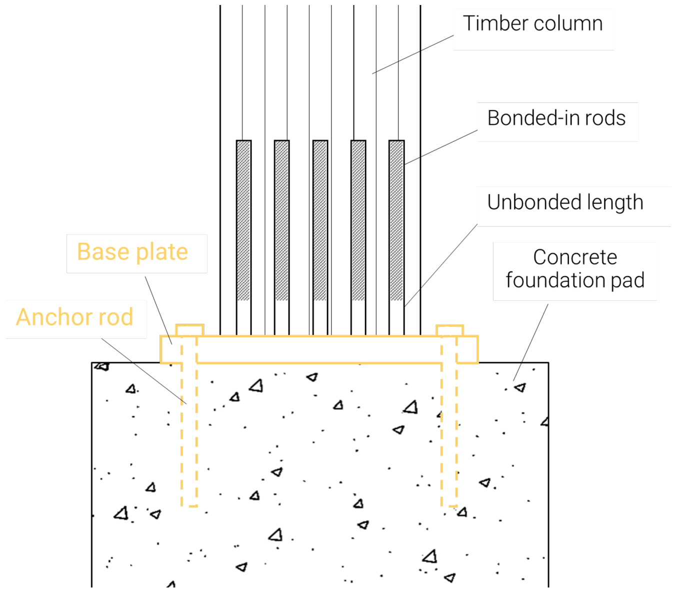


Figure 5.11: Detailing of a demountable glued-in rods connection

Due to the limited scope of this thesis to the building stage, the benefits of reusing timber elements after the end-of-life of the warehouse are not quantified. Nevertheless, is it obvious that extending the lifespan of these biobased materials will also extend the duration of biogenic carbon storage in time. Mass timber elements can last for over 100 years in the right conditions, and the advantage of carbon sequestration in biobased materials relies specifically on the assumption that they will be used for the longest time possible, to keep this carbon out of the atmosphere until the material degrades. Considering targets for the construction sector to become fully circular by 2050, the hypothesis that a large timber structure with such valuable glulam elements will effectively be disassembled for reuse is higly probable.

5.5.2 Change in structural system

The baseline timber frame is sway in both directions, structural members have straight rectangular sections, and the structural grid size is fixed to 16x24m. In case the environmental impact of the frame really needs to be reduced, a change in structural system could be worth considering to reduce to the size of glulam elements and save material.

Adapting sections of structural members

The easiest way to reduce material use without directly altering the stability system and grid size is to swap the straight rectangular members for beams of varying cross-sections along their length, to best fit structural requirements and limit overdesign. For instance, simply supported beams subject to distributed loads require more bending capacity at midspan, calling for higher sections there than at the location of supports. Almost any size and shape can be achieved with glulam, an illustration of common options is illustrated in Figure 5.12.

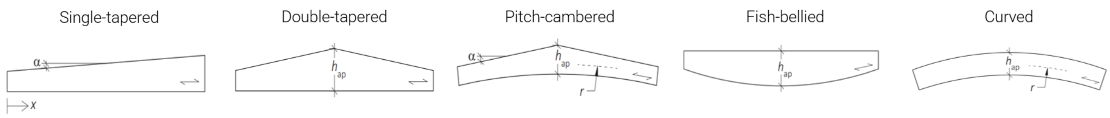


Figure 5.12: Common shapes of glulam beams

Similarly, material consumption in columns can be reduced by choosing for rectangular sections instead of square, to adjust for different design moments in the x and y-directions of the building. Adopting cruciform rather than full rectangular sections is also a solution to achieve a more economic design (see Figure 5.13).

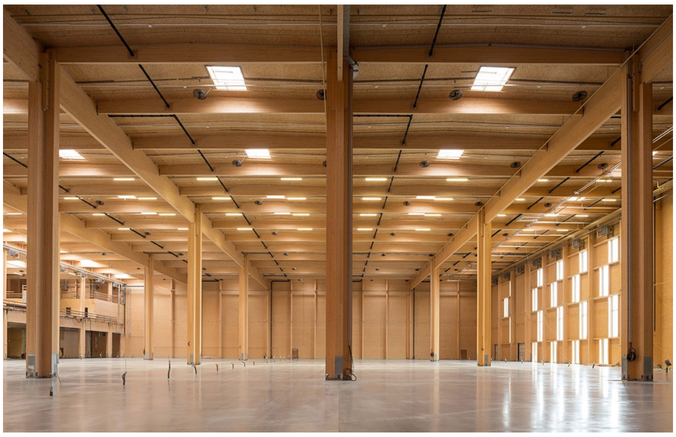


Figure 5.13: Ilogistics Center (Fischamend, Austria) with cruciform glulam columns

Changing the structural layout

Reducing the span of beams or the height of columns is an effective way to reduce bending moments and deflection, and therefore the size of elements (IStructE, 2021). This requires modifying the structural grid, by bringing main columns closer to each other, especially in the x-direction where timber beams span 24 metres. Smaller columns reduce the area of facades as well, therefore decreasing the total impact of the structure and envelope. However, modifications in layout aiming at material savings should not compromise flexibility inside the warehouse.

Changing the stability system

Sway timber frames are rarely designed in practice because of the considerable size of structural elements required to provide sufficient lateral stability. Bracing the sway frame along vertical facades for instance would remove the need for large column sections designed for moment capacity. Lateral stability would rely on the bracing system, and pinned connections easier to manufacture. If bi-directional bracing is not feasible, then bracing at least one direction of the frame shall be considered. If full-height diagonals are too restrictive for flexibility of the warehouse and usable areas for truck openings, smaller knee bracings could be added at the top of columns in one or both directions stiffen top connections and dampen moment capacity requirements at the base.

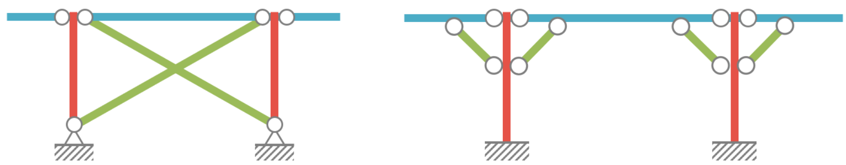


Figure 5.14: Diagonal elements (left) and knee bracings (right) to improve lateral stability of the frame

5.5.3 Parametric design to evaluate frame variants

Parametric design makes it easy to combine different assortments of options to create a range of frame variants to investigate. All design options are considered as input variables for constructing a coherent and flexible structural model. The parametric environment also allows to effectively integrate sustainability matters early in the structural design process (van der Linden, 2018).

An example of flowchart for setting up a parametric model is proposed on Figure 5.15. Environmental impact calculations are performed based on material quantities taken directly from the structural model and updated as the design changes. When each variant is defined, the model is launched to check the structural system and automatically calculates the impact, indicating environmental hotspots to modify first. When a sufficiently low environmental impact score is reached, another design variant is investigated. At the end, all variants are compared based on their environmental performance to determine the most effective design strategies for reducing the impact of the frame.

This model can be set up using tools like Grasshopper, a parametric scripting interface working with McNeel's CAD software Rhinoceros3d. The structural design process can be carried out using the Karamba3D plugin, or linking the parametric model with SCIA to check the results. Then, environmental impact calculations can be performed using the OneClickLCA plugin, used to retrieve environmental data directly from their online database and combined them with the volume of materials calculated in the model.

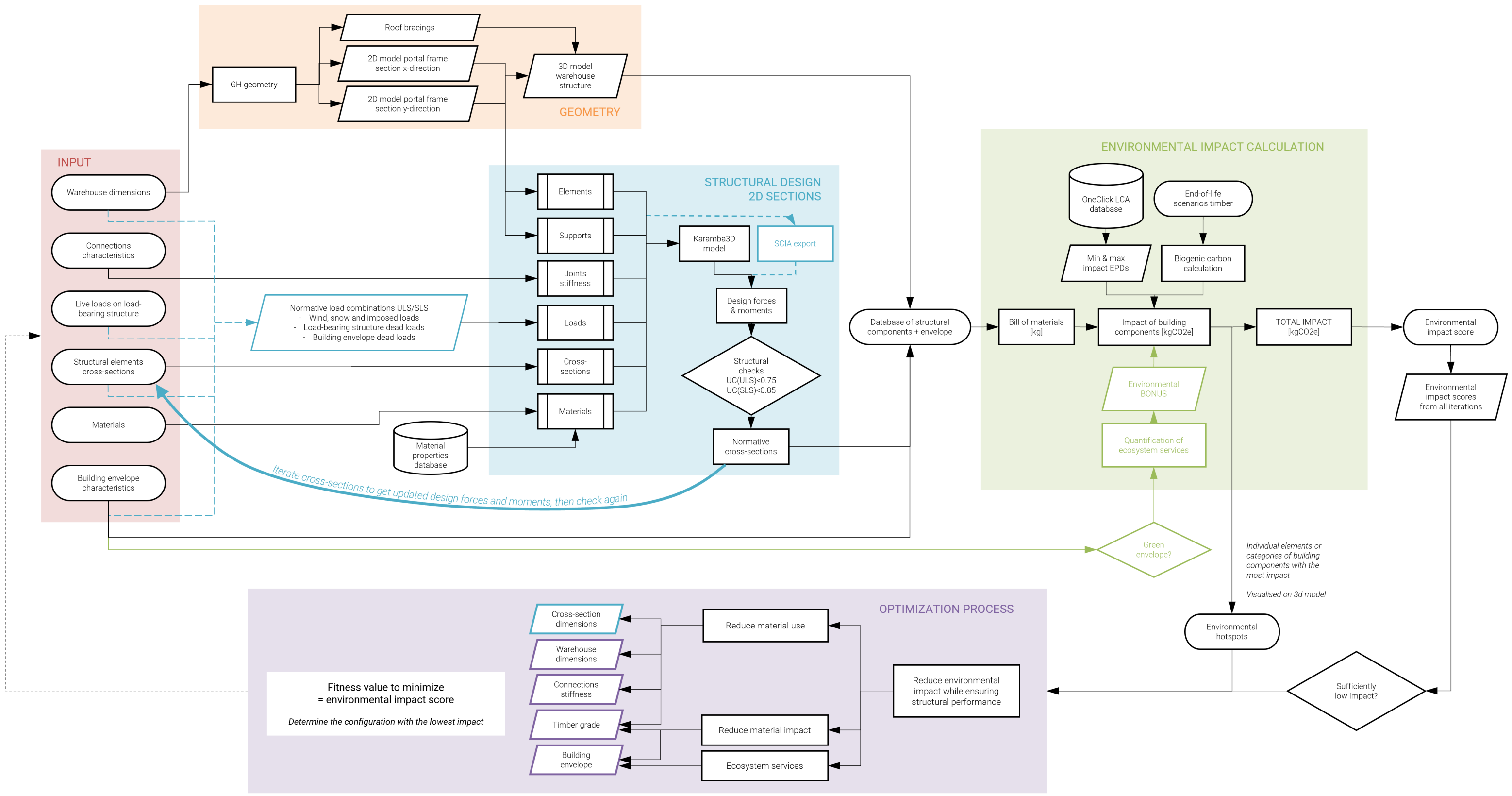


Figure 5.15: Proposition of a parametric design flowchart for the optimisation of timber frame elements, based on an integrated process combining structural and environmental impact calculations

5.6 Conclusion

The research question to be answered in this chapter is:

3. What design strategies should be considered in priority to effectively reduce the environmental impact of a green enveloped biobased warehouse?

Starting from the timber baseline warehouse A designed in the Chapter 4, four sustainable strategies were proposed in this second design step, ranked according to the share of each element in the total embodied carbon of the warehouse.

Strategy A showed that every 10mm reduction in the floor slab thickness for a warehouse of these dimensions was equivalent to a 2% reduction of the total impact score.

Then, strategy B investigated a number of envelope design alternatives to replace the baseline Kingspan sandwich panels on steel supports. The most sustainable variant turned out to be a biobased build-up system created for the purpose of this research. Hollow core timber panels were used on the roof because they can span 8m between roof beams without additional purlins as supports, and they can fit wood fibre thermal insulation inside. With this alternative, fossil carbon could be reduced by 19%, while the impact score considering biogenic carbon storage as well dropped by 169%, making the warehouse carbon positive.

Strategy C proposed adding a green envelope to benefit from ecosystem services provided by vegetation. In particular, an indirect facade greening system and an extensive green roofs of saturated weight between 50 and 150kg/m^2 were selected. Their own embodied carbon footprint, as well as the impact of additional materials required to support these loads, were calculated. Combining the biobased envelope with the heaviest green roof still results in a 10% reduction of the warehouse GWP_{fossil}, and up to 13% for the lightest option. Because the positive effects of ecosystem services (air cleaning, aesthetics, durability, biodiversity...) were not quantified, only a qualitative comparison of the costs and benefits of green can be performed.

Finally, strategy D exposed some leads for changing the design of the timber frame to reduce material use and facilitate disassembly. As other strategies prove to be sufficient in achieving the target set in this project, these solutions are rather meant as suggestions for designers interested in sustainable timber frame design.

6

Conclusion and recommendations

The results of design steps 1 and 2 were presented and discussed in Chapters 4 and 5. Based on these results, this chapter draws the main conclusions of the project, answering each research question to present the main findings in Section 6.1. Recommendations for future research are given at the end of the chapter 6.2.

6.1 Conclusion

The research is divided in three main parts, namely a theoretical presentation of sustainable design strategies, followed by two design steps evaluating their effect regarding embodied carbon reduction. In this section, the research subquestions are answered to finally answer the main research question: *How to reduce the environmental impact score of reference concrete and steel distribution centre designs by at least half?*

Research question 1: What are the most effective strategies to lower the environmental impact of warehouse structures?

Including sustainability from an early design stage enables a potential of 50% embodied carbon reduction for a new build project excluding reuse of existing elements, compared to a traditional building process. The environmental impact of design alternatives is assessed throughout the whole project duration, giving timing to make LCA-driven adjustments as it goes. Then, big ticket items responsible for the highest share of embodied carbon can be identified and tackled in priority.

The impact of building materials is mainly related to their production process. Upfront GHG emissions from fossil-based materials like steel or concrete can be reduced by introducing recycled fractions to replace virgin steel or cement. Timber and other renewable biobased materials, on the other hand, grow from photosynthesis using energy from the sun and capturing carbon from the atmosphere, to store it until burnt or degraded. Because their manufacturing process is far less carbon-intensive and they can fulfil similar functional requirements, biobased building products represent a solid sustainable alternative to traditional fossil-based materials. Biogenic carbon storage can only be considered beneficial if the material's lifespan is at least longer than sustainably managed forests rotation period, to maintain or increase the timber stock. It can be extended beyond the building's end-of-life by designing for long-term reuse or recycling, in compliance with circularity guidelines.

Integrating nature in the design of a building capitalises on ecosystem services provided by vegetation to enhance the mitigate the overall environmental impact. Greening the envelope, in particular, improves air quality locally, increases its thermal performance and supports biodiversity. Depending on project-specific requirements (plant species, layout, load-bearing capacity of the structural frame), direct or indirect greening systems can be applied on facades, while green roofs are generally classified as extensive if lightweight, or intensive for heavier solutions.

Research question 2: How much environmental impact reduction can be achieved by substituting steel or concrete by timber in the load-bearing frame of a warehouse?

The first sustainable design strategy investigated in the research consists in substituting fossil-based materials in the frame by timber. Two reference case studies are selected from existing warehouse projects by RHDHV, with steel and concrete frames. A prerequisite to fairly compare environmental impacts of warehouse variants is to design timber frames with similar dimensions and structural system, for a consistent functional unit. The scope of the analysis is limited to the frame (beams, columns), foundation pads, floor slab and envelope. Two timber baseline warehouses are created to compare with the steel and concrete references, respectively. Both consist of bi-directional sway frames with semi-rigid glued-in rods connections at the base. The floor slab and envelope are kept identical to the references.

Substituting steel by timber in the warehouse load-bearing frame results in a 31% reduction of embodied carbon, considering GWP_{fossil} indicators in kgCO2e for LCA modules A1-A3. Similarly, the concrete to timber material substitution results in a 32% reduction of upfront emissions. Accounting for biogenic carbon stored in timber as negative emissions, the impact reduction reaches up to 67% and 65% for the steel and concrete case studies respectively, exceeding the initial target of 50% set for this research.

Research question 3: What design strategies should be considered in priority to effectively reduce the environmental impact of a green enveloped biobased warehouse?

According to the analysis of the environmental profile of the timber baseline warehouse A, designed in the first design step of the project, the structural components associated with the most embodied carbon are the ground floor slab (40%) and the envelope (41%). These elements can be considered as "environmental hotspots" having the most potential for embodied carbon reduction, and shall be considered in priority when taking design decisions oriented towards sustainability in the second design step.

Strategy A studies the influence of the ground floor slab thickness on the total impact of the warehouse. Indeed, the floor slab is associated with the largest portion of the total embodied carbon of the warehouse, because of the large volume of concrete it represents. It is shown that reducing the floor slab is effective to reduce this volume and therefore the associated embodied carbon, as every 10mm reduction of the thickness results in 2% reduction of the total embodied carbon of the warehouse. For warehouses of larger floor areas, this reduction would be even greater.

Strategy B investigates a number design alternatives for the building envelope, comprising roof and facade panels and including their supports when applicable. The building envelope was initially responsible for 40% of the timber baseline A. This fraction can be reduced to 27% when replacing Kingspan panels with steel supports by biobased build-up alternatives on the roof and facades, reducing the total impact of the building by 19%. If biogenic carbon storage is taken into account, the embodied carbon of the timber warehouse drops by 169% to become negative.

Strategy C proposes adding green to the existing envelope, to benefit from the added value ecosystem services provided by the vegetation layer throughout the service life of the building. A green envelope is also responsible for environmental costs, from the materials used to create the system (cables for an indirect green facade, layers of a green roof built-up), additional loads applied on the load-bearing structure requiring larger frame elements. In this study, green is applied to the baseline and biobased envelope alternatives, considering extensive green roof systems of saturated weights ranging from 50 to 150kg/m^2. Fossil emissions increase up to 7% for the baseline envelope because of bigger frame elements and an increased number of steel purlins required to support higher loads. Overall, one should determine whether the benefits from vegetation balance the environmental costs associated with the installation of greening systems. As sustainability shall tackle not only climate change, but also biodiversity loss among others, the value of ecosystem services may be hard to quantify for now but green undeniably contributes to making a building more respectful of the environment.

Strategy D proposes leads to specifically reduce the impact of the timber frame, first focusing on demountable connections to increase the reuse potential of timber elements and prolong biogenic carbon storage. The volume of timber could be reduced by adapting the sectional area of structural members, limiting spans, or bracing the frame for instance. These strategies are not investigated further, but parametric tools could be used to design and evaluate the impact of structural variants if need be.

Main research question: How to reduce the environmental impact score of reference concrete and steel distribution centre designs by at least half?

The environmental impact of steel and concrete warehouses can be reduced by substituting fossil based materials in the frame and envelope by biobased alternatives of similar functional performance. Using timber in the load-bearing frame as done in design step 1, without modifying the floor slab or envelope, results in approximately 30% embodied carbon reduction considering only upfront fossil emissions. Adding the benefits of biogenic carbon storage in timber to the final impact score makes it drop by 65% compared to the reference designs, fulfilling the main goal of this research. However, current LCA methods require biogenic carbon storage to be credited separately from fossil emissions. Long-term carbon storage could be considered only if wood products are sustainably sourced and their lifespan is longer than forest rotation periods, possibly extended over 100 years thanks to circular design methods.

If only fossil carbon is considered, material substitution in the frame is therefore not sufficient to reach the 50% impact reduction target. From design step 2, it appears that the most effective strategies are reducing the floor slab thickness, while opting for biobased materials and stiff panels in the envelope to remove the need for supports, thus saving material. By combining a 50mm reduction of floor slab thickness, with a biobased envelope applied on a timber frame, a total embodied carbon reduction of 51% is achieved compared to the steel reference warehouse.

6.2 Recommendations for future research

Based on the research presented in this report, some recommendations can be formulated for potential topics to explore further in future projects.

Quantifying the benefits of ecosystem services

Only the embodied carbon caused by the installation of greening systems is quantified in this project. A number of studies have been performed over the past 50 years to quantify the positive effects of specific ecosystem services, like improved thermal insulation or air purification around vegetation. Summarising existing findings from literature and possibly developing new calculation tools, to measure embodied carbon and ecosystem services based on a similar unit, would be highly relevant to help designers compare the environmental costs and benefits of adding green to a building envelope.

Timber structural system variants

The last strategy suggested in the second design step, revolving around the timber frame design, was left out of the scope as it was not a priority for the overall structure, and significant environmental impact reduction was already achieved with other measures. However, for a project focusing solely on the timber frame, the leads mentioned in this thesis could be explored in more detail. Apart from detailing connections for disassembly, the complete structural system could be re-designed and variants compared by parametric modelling. The most restrictive requirement in this project was the bi-directional sway system, as lateral stability could only be provided by semi-rigid connections, requiring huge timber columns to provide sufficient moment capacity. If client's requirement for flexibility allow it, making the structural grid smaller to reduce the span of beams, or bracing the structure in one or both directions could help saving material, as it is already the standard practice for large timber buildings. Fire safety requirements, left out of the scope in this project, should also be included to evaluate the resulting increase in timber volume.

Biogenic carbon storage and circularity

Only the product stage (LCA modules A1-A3) is included the scope of environmental impact calculations in this research. The topic of biogenic carbon accounting in different LCA methodologies, related to the duration of carbon storage in biobased materials, is increasingly discussed to evaluate the sustainability shifting to biobased products in the building industry. Analysing end-of-life scenarios of timber structural elements, depending on the detailing of connections, could be interesting to extend the scope of the life cycle assessment. Dynamic LCA methods are developed modelling time-dependent timber stocks in forests, and could be used in research projects like this one as they appear to give more precise results. Carbon flows at the forest, building and biobased product scales could be compared to highlight best practices in a circular economy regarding long-term biogenic carbon storage in buildings.

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A

Guidelines for structural calculations

A.1 Limit state design and partial factor method

In Europe, the structural design of buildings is regulated by Eurocodes and the national annex of each country. According to the limit state design principle, the state of a structure may be classified as satisfactory or unsatisfactory depending on the limit state design criteria being fulfilled or not. Eurocode EN 1990 (Basis of Structural Design) defines two types of limit states related to structural safety and usability requirements:

- Ultimate Limit State (ULS) includes collapse or other failures affecting the structure and putting personal safety at risk due to instabilities, excessive deformations, or rupture of structural elements.
- Serviceability Limit State (SLS) comprises deformations affecting the appearance, level of comfort or planned functionality of a structure, therefore disrupting the normal use, causing damage, or having long-term effects on its durability

The structural design process should ensure a sufficiently low probability of failure, determined by the overlap between the frequency curves of actions E exerted on the structure and the resistance R.

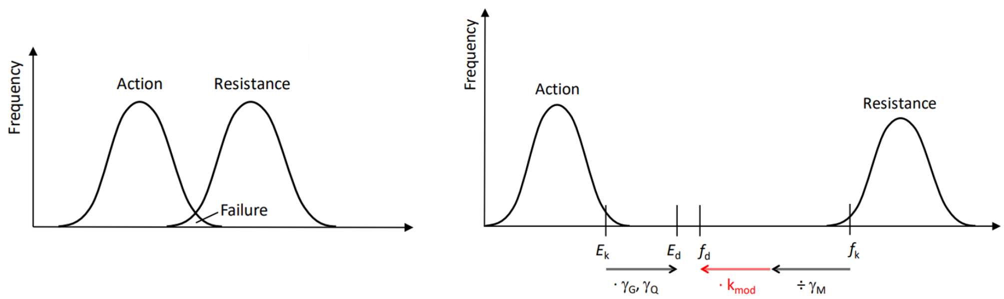


Figure A.1: Frequency distributions for action and resistance (left) and reduction of the probability of failure through partial safety factors and k_{mod} (right) (Blaß & Sandhaas, 2017)

Structural design according to the Eurocodes is based on partial factors, allowing low probability of failure to be attained. Characteristic values X_k of variables are determined from normal distribution functions and are multiplied or divided by partial factors to obtain design values X_d. The effects of actions are the structural response to imposed actions, and comprise internal stress resultants (moments, shear or axial forces, stress, strain) as well as structural deformations (deflections, rotations).

Strength checks at ULS are performed using the partial factor method, by comparing the design stress to the design strength modified by appropriate strength factors ($E_d \leq R_d$). At SLS the design values of action effects should remain below key limit values such as maximum allowed deflection ($E_d \leq C_d$).

A.2 Timber structural design according to Eurocode 5

A.2.1 Material properties

For timber, the design value of a material property or resistance is determined according to clause 2.4.1 and 2.4.3 respectively, of EN 1995-1-1:

$$X_d = k_{mod}.X_k/\gamma_M \tag{A.1}$$

The partial safety factor γ_M depends on the limit state and type of structural element.

Table A.1: Partial safety factors for material properties and resistances according to Table 2.3 of EN 1995-1-1 (NEN-EN 1995-1-1:2005, 2005)

Material	γ_M
Solid timber	1.3
Glulam	1.25
Bondline failure	1.3
Connections with dowel-type fasteners	1.3

The strength properties of timber decrease as the duration of loading on the element increases. To account for this effect, load duration classes are associated with periods of time likely to apply to loads encountered in engineering practice.

The strength properties and creep behaviour of timber are also influenced by the moisture content in the material, temperature and relative humidity conditions. The strength tends to decrease with increasing moisture content and reaches its minimum value at the fibre saturation point. To account for this effect, service classes have been defined to cover typical environmental conditions timber structures may be exposed to. For a load-bearing structure located inside, service class 1 applies: it is characterised by a dry climate, with the moisture content in materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65% for a few weeks per year. Under such conditions, the average moisture content of most softwoods does not exceed 12%.

The modification factor k_{mod} translates the effects of load duration and moisture content on the strength properties of the material. It depends on the service class of the structure and the load-duration class. In design checks, the modification factor should be selected based on the action with the shortest duration in the governing load combination.

Table A.2: Modification factors for solid timber and glulam (service class 1 and 2) according to Tables 2.1, 2.2 and 3.1 of EN 1995-1-1 (NEN-EN 1995-1-1:2005, 2005)

Load duration class	**Duration**	**Examples of loads**	**k_{mod}**
Permanent	> 10 years	Self-weight	0.6
Long-term	6 months - 10 years	Storage	0.7
Medium-term	1 week - 6 months	Imposed floor load	0.8
Short-term	< 1 week	Snow, wind, maintenance	0.9
Instantaneous	>10 years	Accidental load	1.1

Stiffness properties at ULS

At the ULS, design checks are performed to validate the strength and stability behaviour, based on the loading with the greatest effect from the defined load combinations. The values of stiffness properties to use depend on the type of analysis:

- For a first order linear elastic analysis, where the stiffness distribution within the structure does not affect the internal stress resultants, mean values shall be used:

$$E_{d,ULS} = E_{mean} \; ; G_{d,ULS} = G_{mean} \; ; K_{d,ULS} = K_{ser} \tag{A.2}$$

- For a first order linear elastic analysis, where the stiffness distribution within the structure affects the internal stress resultants for the final condition, final mean values adjusted to the load component causing the largest stress in relation to strength shall be used:

$$E_{d,ULS} = \frac{E_{mean}}{1+\psi_2 k_{def}} \; ; G_{d,ULS} = \frac{G_{mean}}{1+\psi_2 k_{def}} \; ; K_{d,ULS} = \frac{K_{ser}}{1+\psi_2 k_{def}} \tag{A.3}$$

- For a second order linear elastic analysis, design values not adjusted for the duration of loads shall be used:

$$E_d = \frac{E_{mean}}{\gamma_M} \; ; G_d = \frac{G_{mean}}{\gamma_M} \; ; K_u = \frac{2}{3} K_{ser} \tag{A.4}$$

Stiffness properties at SLS

The relevant SLS for timber structures are vibrations and deflections. In this research, only the horizontal and vertical deflections of timber structural members are considered relative to limiting design criteria. To demonstrate compliance with SLS criteria at the instantaneous and final displacement conditions, displacement analyses must be undertaken at each condition. The instantaneous deformation should be calculated for the characteristic combination of actions, using mean values of stiffness properties. The final deformation should be calculated for the quasi-permanent combination of actions. Where a structure comprises components with different time-dependent properties, the effect of creep on stiffness properties must be considered. Therefore, for SLS verifications, the final mean values of stiffness properties are calculated as:

$$E_{mean,fin} = \frac{E_{mean}}{1 + k_{def}} \; ; G_{mean,fin} = \frac{G_{mean}}{1 + k_{def}} \; ; K_{ser,fin} = \frac{K_{ser}}{1 + k_{def}} \tag{A.5}$$

A.2.2 Flexural and axially loaded members

Flexural members are structural elements subjected to bending. In timber structures, typical examples include beams, joists, rafters and purlins. Because of the material properties of timber, additional structural checks should be performed compared to concrete or steel members in bending. Indeed, wood exhibits anisotropic behaviour, and its characteristics are influenced by the moisture content and load duration. Special design requirements apply to glulam beams and composite cross-sections (thin webbed and thin flanged beams).

Structural members subjected to axial loading, or a combination of axial and flexural actions, are commonly used in timber structures. Typical examples include columns, vertical wall studs and bracing elements. Combined axial and bending effects may come from eccentric connections, wind loading or rigid frame action for instance. Axial loads applying at an angle to the grain and should be considered in relation with specific design checks to account for the appropriate material strength.

Table A.3: Design requirements for flexural or axially loaded timber members

	ULS	**SLS**
Flexural members	Retention of static equilibrium Bending Lateral torsional stability Shear Bearing	Deflection
Axially loaded members	Retention of static equilibrium Axial load Lateral stability	Deflection

The design requirements presented in the following sections are only applicable to straight timber members, with uniform cross-sections and fibres running parallel to the member length.

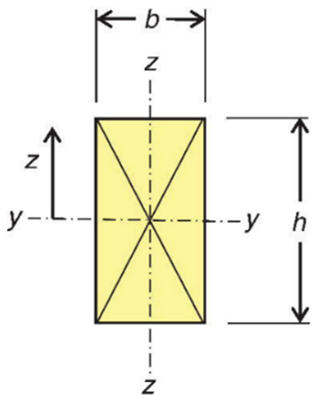


Figure A.2: Rectangular section

A.2.3 Design for ULS

A.2.3.1 Tension parallel and perpendicular to the grain (Clauses 6.1.2 and 6.1.3)

Tensile stresses parallel to the grain

Tension members act as ties in pin-jointed trusses or contribute to the tensile resistance to stud walls against overturning forces, and are not at risk of buckling. The resistance of tension members should be checked at the weakest point, that is often at the location of the connection. The net cross-sectional area accounts for the loss of sectional area due to connections. The design requirement for a member in tension parallel to the grain states that the design tensile stress $\sigma_{t,0,d}$ should be less that the design tensile strength $f_{t,0,d}$:

$$\sigma_{t,0,d} \leq f_{t,0,d} \tag{A.6}$$

$$f_{t,0,d} = k_{mod} k_{sys} k_h f_{t,0,k} \tag{A.7}$$

- $\sigma_{t,0,d} = N_d / A_{net}$ is the design tensile stress parallel to the grain
  - N_d is the design axial load on the member, parallel to the grain
  - A_{net} is the net cross-sectional area allowing for the effect of the connections (if different connections are used at each end of the member, the minimum value should be used)
- k_h is the modification factor for member size effects based on the largest cross-sectional dimension
  - For solid timber:

$$k_h = \min\left(\left(\frac{150}{h}\right)^{0.2}; 1.3\right) \tag{A.8}$$

  - For laminated wood products:

$$k_h = \min\left(\left(\frac{600}{h}\right)^{0.1}; 1.1\right) \tag{A.9}$$

Tensile stresses perpendicular to the grain

In straight timber members under axial tension, tension stresses only occur parallel to the grain of the material. Tension stresses perpendicular to the grain would occur in tapered or curved beams and connections.

A.2.3.2 Compression parallel to the grain (Clause 6.1.4)

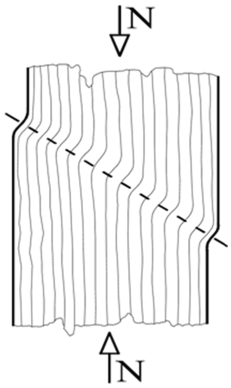


Figure A.3: Failure mechanisms in compression (NEN-EN 1995-1-1:2005, 2005)

The design check for compression parallel to the grain is based on the design compressive strength $f_{c,0,d}$:

$$\sigma_{c,0,d} \leq f_{c,0,d} \tag{A.10}$$

$$f_{c,0,d} = k_{mod} k_{sys} \frac{f_{c,0,k}}{\gamma_M} \tag{A.11}$$

- $\sigma_{c,0,d} = F_{c,0,d}/A$ is the design compressive stress parallel to the grain
  - $F_{c,0,d}$ is the design compressive load on the member, parallel to the grain
  - A is the cross-sectional area
  - $f_{c,0,k}$ is the characteristic compressive strength perpendicular to the grain
  - k_{mod} is the modification factor for load duration and service classes
  - k_{sys} is a strength factor for load-sharing systems, taken as 1.0 for columns

A.2.3.3 Compression perpendicular to the grain (Clause 6.1.5)

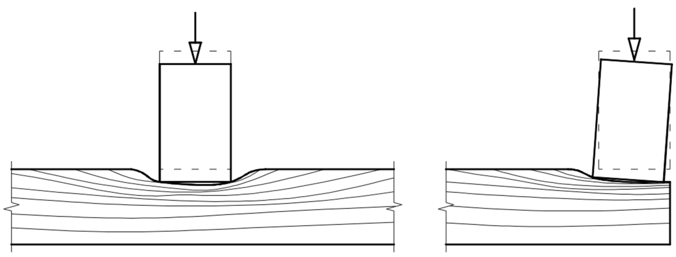


Figure A.4: Bearing effects at supports and points of concentrated load application (NEN-EN 1995-1-1:2005, 2005)

Where a timber member is subject to compression perpendicular to the grain, for instance at the location of supports, the design requirement for bearing is:

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} \tag{A.12}$$

$$f_{c,90,d} = k_{mod} k_{sys} \frac{f_{c,90,k}}{\gamma_M} \tag{A.13}$$

- $\sigma_{c,90,d}=F_{c,90,d}/A_{ef}$ is the design compressive stress perpendicular to the grain
  - $F_{c,90,d}$ is the design compressive load on the member, perpendicular to the grain
  - A_{ef}=bl is the effective contact area in compression perpendicular to the grain
- $f_{c,90,d}$ is the design compressive strength perpendicular to the grain
- $f_{c,90,k}$ is the characteristic compressive strength perpendicular to the grain
- $k_{c,90}$ is a factor accounting for the load configuration, the risk of splitting and the degree of strain deformation of the member under compression. For members on discrete supports, provided that $l_1 \leq 2h$:
  - $k_{c,90}$=1.5 for solid softwood timber
  - $k_{c,90}$=1.75 for glulam members provided that $l \leq 400mm$

A.2.3.4 Bending (Clause 6.1.6)

Design bending stress

In a solid rectangular timber beam subjected to a bending moment M about the strong y-axis, the design bending stress in the cross-section at a distance z from this axis is:

$$\sigma(z) = \frac{M}{W_y} = \frac{Mz}{I_y} \tag{A.14}$$

- $W_y=I_y/z$ is the section modulus about the axis of bending
- $I_y=bh^3/12$ is the second moment of area of a rectangular section about the axis of bending

At the extreme fibre location in a rectangular section, the design bending stress resulting from a design bending moment M_d is:

$$\sigma_{m,y,d} = \frac{M_d}{W_y} \text{ with } W_y(h/2) = \frac{bh^2}{6} \tag{A.15}$$

Similar formulas can be derived in case of a bending moment applied about the weak z-axis. The maximum stress induced in a section subjected to bi-axial bending is:

$$\sigma_{max} = \frac{M_y}{W_y} + \frac{M_z}{W_z} \tag{A.16}$$

Design bending strength

When subjected to uniaxial bending, the design requirement for bending strength of a timber member is that the maximum value of the design bending stress $\sigma_{m,d}$ in the section should not exceed the design bending strength $f_{m,d}$:

$$\sigma_{m,d} \leq f_{m,d} \tag{A.17}$$

The design bending strength is expressed as from the characteristic bending strength of the material $f_{m,k}$ modified with partial factors:

$$f_{m,d} = k_{mod} k_{sys} k_h \frac{f_{m,k}}{\gamma_M} \tag{A.18}$$

- $f_{m,k}$ is the characteristic bending strength (dependent of the strength class of the material)
- γ_M is the partial factor for material properties
- k_{mod} is the modification factor for load duration and service classes
- k_{sys} is the system strength factor
- k_h is the modification factor for member size effects (Equation (3.1) of EC5), accounting for the reduced effects of defects when the member size is less that the reference size
  - For solid timber:

$$k_h = \min\left(\left(\frac{150}{h}\right)^{0.2}; 1.3\right) \tag{A.19}$$

  - For glulam:

$$k_h = \min\left(\left(\frac{600}{h}\right)^{0.1}; 1.1\right) \tag{A.20}$$

When subjected to bi-axial bending, about both the y and z-axes, the design conditions to be met are:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{A.21}$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{A.22}$$

- $\sigma_{m,y,d}$ is the design bending stress about the y-axis
- $\sigma_{m,z,d}$ is the design bending stress about the z-axis
- $f_{m,y,d}$ is the design bending strength about the y-axis
- $f_{m,z,d}$ is the design bending strength about the z-axis

- k_m is a modification factor used to account for the redistribution of stresses when a section is subjected to bending about both axes, and the effect of inhomogeneities of the material in the cross-section (Clause 6.1.6(2) of EC5):
  - For solid timber, glulam and LVL with rectangular sections: k_m=0.7
  - For other cross-sections or other timber products: k_m=1.0

Lateral torsional instability cannot occur in sections where the second moment of area is equal about the y and z axes. In square sections for instance, the design conditions for bending about both axes become:

$$(1 + k_m)\frac{\sigma_{m,d}}{f_{m,d}} \leq 1 \tag{A.23}$$

A.2.3.5 Shear (Clause 6.1.7 – Equation (6.13))

Design shear stress

In a beam subjected to bending and lateral loading, shear stresses of equal magnitude are generated parallel and perpendicular to the longitudinal axis of the beam:

$$\tau = \frac{VS}{Ib} \tag{A.24}$$

- V is the shear force at the considered location in the cross-section
- S=Az is the first moment of area above the shear stress level about the neutral axis
- I=bd^3/12 is the second moment of area of the cross-section about the neutral axis
- b is the width of the cross-section at the shear stress level
- b_{ef}=k_{cr}b is the effective width
- k_{cr} is a modification factor accounting for possible cracks in the section reducing the shear strength of the member
  - k_{cr}=0.67 for structural timber and glulam
  - k_{cr}=1.0 for other wood-based products

The maximum shear stress in a rectangular section of a beam occurs at the neutral axis, at mid-depth:

$$\tau_d = \frac{3}{2}\frac{V_d}{bh} \tag{A.25}$$

Design shear strength

The design requirement for shear states that design shear stresses τ_d with a component parallel to the grain, or both perpendicular as for rolling shear, should remain below the design shear strength $f_{v,d}$:

$$\tau_d \leq f_{v,d} \tag{A.26}$$

$$f_{v,d} = k_{mod}k_{sys}\frac{f_{v,k}}{\gamma_M} \tag{A.27}$$

- $f_{v,k}$ is the characteristic shear strength
- k_{mod} is the modification factor for load duration and service classes
- k_{sys} is the system strength factor

(left) Member with a shear stress component parallel to the grain (right) Member with both stress components perpendicular to the grain (rolling shear)

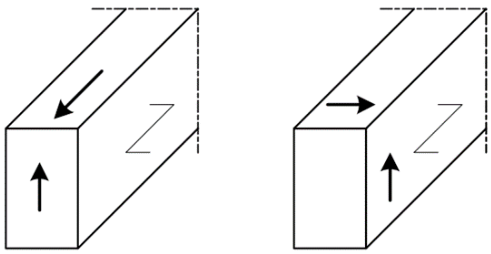


Figure A.5: Left: Member with a shear stress component parallel to the grain. Right: Member with both stress components perpendicular to the grain (rolling shear) (NEN-EN 1995-1-1:2005, 2005)

A.2.3.6 Compression at an angle to the grain (Clause 6.2.2)

The failure strength of a member subjected to compression at an angle α to the grain is derived from the compression strengths parallel and perpendicular to the grain, by the empirical Hankinson equation:

$$f_{c,\alpha} = \frac{f_{c,0} f_{c,90}}{f_{c,0} \sin^2 \alpha + f_{c,90} \cos^2 \alpha} \tag{A.28}$$

Based on this expression and the design requirement for compression perpendicular to the grain, the design compressive stress at an angle to the grain $\sigma_{c,\alpha,d}$ should satisfy:

$$\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{k_{c,90} f_{c,90,d}} \sin^2 \alpha + \cos^2 \alpha} \tag{A.29}$$

- $\sigma_{c,\alpha,d}$ is the design compressive stress at an angle α to the grain
- $k_{c,90}$ is a factor accounting for the effects of any stresses perpendicular to the grain

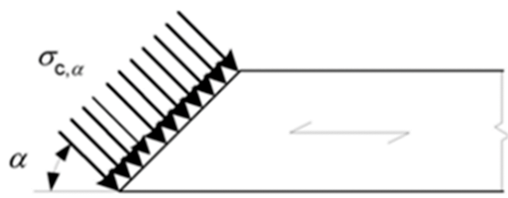


Figure A.6: Compressive stresses at an angle to the grain (NEN-EN 1995-1-1:2005, 2005)

A.2.3.7 Combined bending and axial tension (Clause 6.2.3)

For bi-axial bending combined with axial tension, the following interaction equations are used:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{A.30}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{A.31}$$

For square cross-sections these expressions become:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + (1 + k_m) \frac{\sigma_{m,d}}{f_{m,d}} \leq 1.0 \tag{A.32}$$

A.2.3.8 Combined bending and axial compression (Clause 6.3.2)

Under compression and bi-axial bending, when no strength reduction due to lateral torsional buckling or flexural buckling is necessary ($\lambda_{rel,y} \leq 0.3$ and $\lambda_{rel,z} \leq 0.3$), the design equations are:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{A.33}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{A.34}$$

- k_m is a factor used to account for the re-distribution of stresses and the effect of inhomogeneities in the cross-section.
  - For solid timber, glulam and LVL: k_m=0.7
  - For other timber products or when h/b>4: k_m=1.0

For square cross-sections these expressions become:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + (1 + k_m) \frac{\sigma_{m,d}}{f_{m,d}} \leq 1.0 \tag{A.35}$$

A.2.3.9 Flexural buckling of columns subjected to either compression or combined bending and compression (Clause 6.3.2)

Compression members include columns or struts, and their design is often governed by the flexural buckling. This failure mode is caused by secondary bending effects due to eccentric loading situations, geometrical imperfections in the member or variations in material properties, eventually leading to premature failure of the element by buckling. Buckling in a compression member occurs about the axis with the highest slenderness ratio λ, defined as:

$$\lambda = \frac{L_e}{i} \tag{A.36}$$

- L_e is the effective length of the member, or the distance between adjacent points of contra-flexure where the bending moment is zero
- $i = \sqrt{I/A}$ is the radius of gyration about the buckling axis
  - $i_y = h/\sqrt{12}$ and $i_z = b/\sqrt{12}$
  - I is the moment of inertia about this axis
  - A is the cross-sectional area

The theoretical stress at which buckling will occur is called Euler buckling stress σ_{crit} and is used to define the relative slenderness λ_{rel} of the member about each axis, depending on the slenderness ratio λ:

$$\sigma_{crit} = \frac{\pi^2 E_{0.05}}{\lambda^2} \tag{A.37}$$

$$\lambda_{rel} = \sqrt{\frac{f_{c,0,k}}{\sigma_{crit}}} = \frac{\lambda}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \tag{A.38}$$

- $E_{0.05}$ is the 5% value of the modulus of elasticity
- $f_{c,0,k}$ is the characteristic compressive strength parallel to the grain

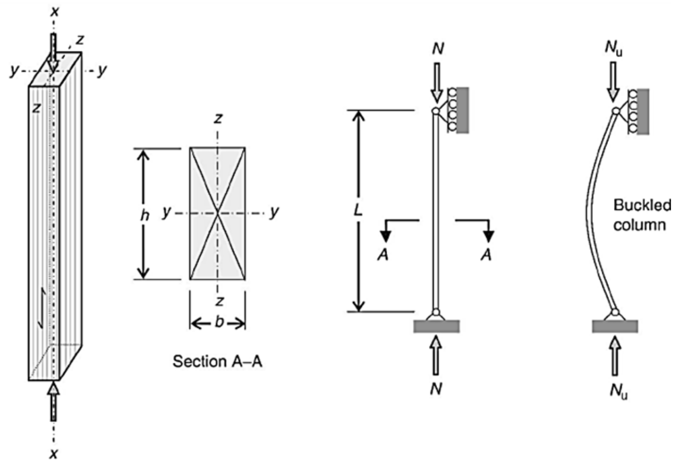


Figure A.7: Buckling of a column (Porteous & Kermani, 2013)

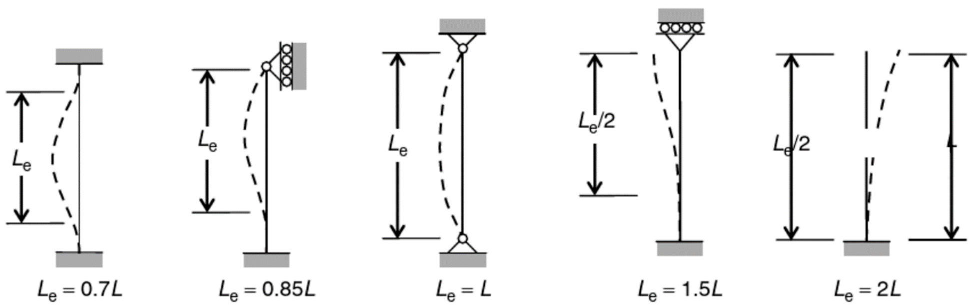


Figure A.8: Effective lengths related to end conditions of compression members

Members subjected to compression only

When compression members are susceptible to flexural buckling ($\lambda_{rel,y} > 0.3$ and/or $\lambda_{rel,z} > 0.3$), the design compressive strength $f_{c,0,d}$ of the member is reduced by an instability factor k_c about the axis where the relative slenderness λ_{rel} is over 0.3: When $\lambda_{rel,y} > 0.3$:

$$\sigma_{c,0,d} \leq k_{c,y} f_{c,0,d} \tag{A.39}$$

When $\lambda_{rel,z} > 0.3$:

$$\sigma_{c,0,d} \leq k_{c,z} f_{c,0,d} \tag{A.40}$$

- $k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}}$
- $k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}$
  - $k_y = 0.5\left(1 + \beta_c\left(\lambda_{rel,y} - 0.3\right) + \lambda_{rel,y}^2\right)$
  - $k_z = 0.5\left(1 + \beta_c\left(\lambda_{rel,z} - 0.3\right) + \lambda_{rel,z}^2\right)$
    - $\beta_c = 0.2$ for solid timber
    - $\beta_c = 0.1$ for glulam and LVL

When compression members are not susceptible to flexural buckling ($\lambda_{rel,y} \leq 0.3$ and $\lambda_{rel,z} \leq 0.3$), the design check for compression parallel to the grain is based on the full design compressive strength $f_{c,0,d}$ as previously described.

Members subjected to compression and bi-axial bending

Under compression and bi-axial bending, when flexural buckling occurs ($\lambda_{rel,y} > 0.3$ and/or $\lambda_{rel,z} > 0.3$) and provided that there is no lateral torsional buckling, the stresses increased due to deflection should satisfy the following checks: When $\lambda_{rel,y} > 0.3$:

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{A.41}$$

When $\lambda_{rel,z} > 0.3$:

$$\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{A.42}$$

For square cross-sections, if $\lambda_{rel} > 0.3$ these expressions become:

$$\frac{\sigma_{c,0,d}}{k_c f_{c,0,d}} + (1 + k_m)\frac{\sigma_{m,d}}{f_{m,d}} \leq 1.0 \tag{A.43}$$

A.2.3.10 Lateral torsional buckling of beams subjected to either bending or combined bending and compression (Clause 6.3.3)

Lateral torsional buckling is a type of instability of a slender beam with large depth and length compared to the width, characterised by rotation and deflection in the vertical and horizontal directions. It can occur for beams where rotation along the beam axis is restrained at the supports, for example by fork supports.

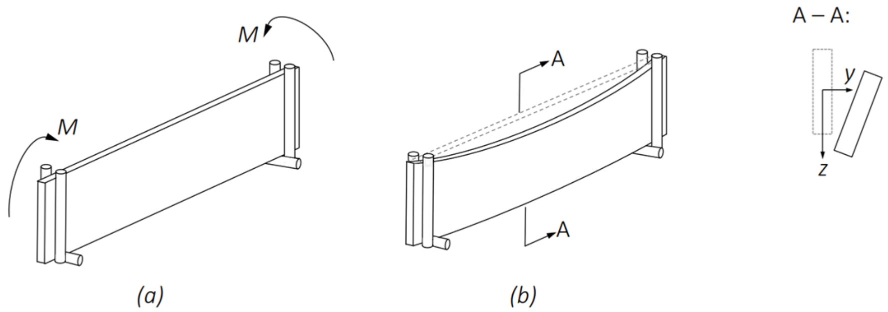


Figure A.9: Lateral torsional buckling of a simply supported beam subject to constant moment. (a) simply supported beam with restrained supports, (b) buckled beam (Blaß & Sandhaas, 2017)

The design bending strength $f_{m,d}$ of a timber member should be reduced by a factor k_{crit} whenever lateral torsional buckling is susceptible to happen, that is when the member is subject to bending about the major axis and the relative slenderness ratio for bending $\lambda_{rel,m,y}$ is over 0.75:

$$\lambda_{rel,m,y} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} > 0.75 \tag{A.44}$$

Where the critical bending strength $\sigma_{m,crit}$ is expressed from the elastic critical moment $M_{y,crit}$ at which elastic buckling occurs, divided by the section modulus W_y:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi}{W_y l_{ef}} \sqrt{E_{0.05} I_z G_{0.05} I_t} \tag{A.45}$$

- l_{ef} is the effective length of the beam (Table 6.1 of EC5)
- $E_{0.05}$ is the 5% value of the modulus of elasticity
- I_z is the second moment of area of the section
- $E_{0.05}$ is the 5% value of the shear modulus
- $I_t = \frac{hb^3}{3}\left(1 - 0.63\frac{b}{h}\right)$ is the torsional moment of inertia

In case the compression flange of the beam is laterally supported along its length and torsional rotation is prevented at the supports, the beam can be considered to be fully restrained and the factor k_{crit} may be taken as 1.0.

Table A.4: Values of k_{crit} (NEN-EN 1995-1-1:2005, 2005)

k_{crit}	**Relative slenderness** $\lambda_{rel,m}$
1	$\lambda_{rel} \leq 0.75$
$1.56 - 0.75\lambda_{rel}$	$0.75 < \lambda_{rel} \leq 1.4$
$1/\lambda_{rel}^2$	$\lambda_{rel} > 1.4$

Members subjected to bending about the major axis only

The design requirement for bending is modified when checking for lateral torsional buckling:

$$\sigma_{m,d} \leq k_{crit} f_{m,d} \tag{A.46}$$

Members subjected to compression and bending about the major axis

When a member is subjected to combined compression and bending about the major axis, if lateral torsional buckling can occur, the design requirement resulting from the interaction of these actions is:

$$\left(\frac{\sigma_{m,d}}{k_{crit} f_{m,d}}\right)^2 + \frac{\sigma_{c,d}}{k_{c,z} f_{c,0,d}} \leq 1.0 \tag{A.47}$$

A.2.4 Design for SLS

A.2.4.1 Deflection (Clause 7.2)

The Eurocode defined limiting values for the deformation components of a timber beam. The net final deflection $w_{net,fin}$ of a beam below a straight line joining supports should be taken as a combination of the following components:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \tag{A.48}$$

- w_c is the precamber, where used
- w_{inst} is the instantaneous deflection, permitted immediately under the action of the design load
- w_{creep} is the creep deflection, arising with time under the combination of loading causing creep
- w_{fin} is the final deflection, combining the instantaneous and creep deformations

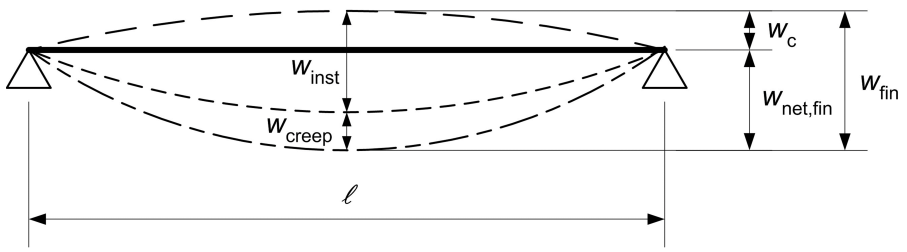


Figure A.10: Components of deflection (NEN-EN 1995-1-1:2005, 2005)

The actual final deflection of a structural member under loading is the sum of the final deformations associated with the applied loads, each comprising an instantaneous and a creep component:

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + \sum_{i>1} u_{fin,Qi} \tag{A.49}$$

- $u_{fin,G} = u_{inst,G} + u_{creep,G} = u_{inst,G}\left(1 + k_{def}\right)$ is the final deformation for permanent actions G
- $u_{fin,Q1} = u_{inst,Q1}\left(1 + \psi_{2,1} k_{def}\right)$ for the leading variable action Q_1
- $u_{fin,Qi} = u_{inst,Qi}\left(\psi_{0,i} + \psi_{2,i} k_{def}\right)$ for accompanying variable actions $Q_i(i > 1)$
- $u_{inst,G}$, $u_{inst,Q1}$ and $u_{inst,Qi}$ are the instantaneous deformations for actions G, Q_1 and Q_i respectively
- $\psi_{2,1}$ and $\psi_{2,i}$ are the factors for the quasi-permanent value of variable actions
- $\psi_{0,i}$ are the factors for the combination value of variable actions
- k_{def} is the creep factor (Table 3.2 of EC5)

Table A.5: Creep factors for solid timber, glulam and LVL according to Table 3.2 of EN 1995-1-1 (NEN-EN 1995-1-1:2005, 2005)

Material	**Standard**	**Service class 1**	**Service class 2**	**Service class 3**
Solid timber	EN 14081-1	0.60	0.80	2.00
Glulam	EN 14080	0.60	0.80	2.00

B

Load calculations

This appendix presents an example of load calculations performed during the structural design process of the timber baseline warehouse A. Numerical values may be different for other designs in this project, but the reasoning remains identical.

The loads acting on the structure can be divided into permanent and variable actions, determined from EN 1991 Actions on structures. Permanent actions (G) include the self-weight of structural elements and of other building components to be supported. Variable actions (Q) consist of imposed live loads, wind and snow loads. Accidental and seismic actions are left out of the scope of this study. The Eurocodes used to calculate loads on the structure are listed in Table B.1.

Table B.1: Eurocodes

Eurocode reference	Title
NEN-EN 1990+A1+A1/C2	Basis of structural design
NEN-EN 1991-1-1+C1+C11	Actions on structures - Densities, self-weight, imposed loads for buildings
NEN-EN 1991-1-3+C1	Actions on structures - General actions - Snow loads
NEN-EN 1991-1-4+A1+C2	Actions on structures - General actions - Wind actions

B.1 Permanent loads

Two types of permanent actions are included in the model, namely the weight of building components and dead loads from services:

Self-weight of the main load-bearing frame

It is determined from the dimensions of beams and columns, and material density

Self-weight of roofing system = 0.30 kN/m^2

It corresponds to the maximum average weight of roof sandwich panels on steel purlins.

Self-weight of facade system = 0.50 kN/m^2

It corresponds to the maximum average weight of horizontal Kingspan panels (0.30 kN/m^2) on steel rails (0.20 kN/m^2).

Services = 0.10 kN/m^2

It corresponds to ducts and piping elements within the roof such as heating units, electrical wires or air conditioning.

B.2 Imposed loads

Imposed actions depend on the building usage, and are related to the movement of people, furniture, and storage activities.

Imposed loads on the ground floor

As the warehouse structure shall be used for storage and industrial activities, it belongs to classes E1/E2 defined in EN 1991-1-1. However, imposed loads on the ground floor are not relevant for the design of the timber load-bearing structure.

Imposed loads on the roof q_k = 0.40 kN/m^2

The roof of the warehouse belongs to category H (see Table B.2) as is it not accessible, except for normal maintenance and repair operations. Therefore, imposed loads on the roof are determined from the recommended values given in Table 6.10 of EN 1991-1-1.

Table B.2: Categorization of roofs (NEN-EN 1991-1-1:2002, 2002)

Categories of loaded area	Specific use
H	Roofs not accessible except for normal maintenance and repair
I	Roofs accessible with occupancy according to categories A to D
K	Roofs accessible for special services, such as helicopter landing areas

The uniformly distributed load $q_k = 0.40$ kN/m^2 is modelled as a short-term distributed load on the roof surface. It is assumed that the structure is able to withstand a concentrated load $Q_k = 1.0$ kN exerted on a square area of 50mm side length at the least favourable spot of the roof surface, related to maintenance and repair operations. This assumption is supported by the use of conservative unity checks in structural calculations.

Air units Q_k = 30 kN

Air units are located on top of every column and modelled as concentrated loads of long-term duration.

B.3 Snow loads

Snow loads are determined from EN 1991-1-3 based on snow depth and density. As the exact location of the building in Europe remains unknown, the characteristic snow load on the ground is taken as $s_k = 1.2$ kN/m^2, the most prevalent value in the eleven potential countries where the distribution centre could be implemented, for areas located at an altitude lower than 500m above sea level. The warehouse is the tallest building of the total distribution centre, therefore snow accumulation is disregarded and the characteristic value $s_k = 1.2$ kN/m^2 is governing the design.

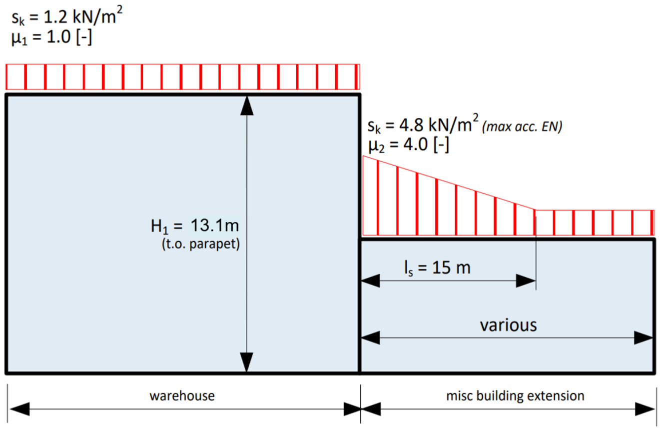


Figure B.1: Snow load on the roof of the warehouse (RHDHV Internal Document, 2022)

Drifting at roof parapet is determined in accordance with EN 1991-1-3. Considering the large area of the roof, the overloading caused by any drifting is not considered in structural checks of the 2D warehouse sections. However, snow drift at the parapet should be considered when designing roof and facade systems in details.

B.4 Wind loads

Wind loading on the structure is determined following the guidelines of EN 1991-1-4. Wind loads are positive if they exert a pressure towards the considered surface, and negative if moving away from the surface (see Figure B.2).

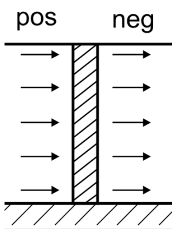


Figure B.2: Definition of positive and negative pressure (NEN-EN 1991-1-4:2005, 2005)

Project data

Due to the nature of the project, the exact location of the distribution centre is unknown. The basic wind velocity and terrain category are the main factors of influence on the wind loading in Europe:

- Usually varying between 21 to 36 m/s, the reference wind velocity is here taken as v_b=30 m/s to cover a large part of the project area
- The terrain category is related to the density or openness of the area. Among those described in Table 4.1 of EN 1991-1-4, the most suitable categories for a distribution centre are II, II and IV. Here, the reference for design is set to terrain category II (see Figure B.3) corresponding to areas with low vegetation, where the highest wind loads are found.

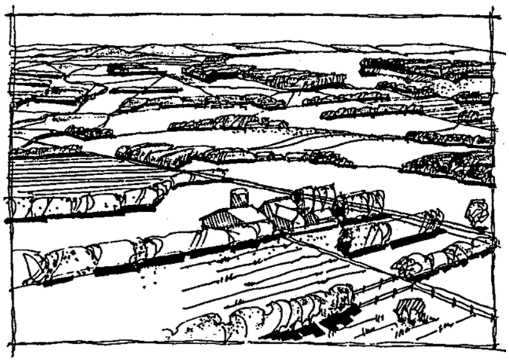


Figure B.3: Terrain category II (NEN-EN 1991-1-4:2005, 2005)

Table B.3: Project data for the calculation of wind loads

Description	Symbol	Value	Unit
Length	L	144	m
Width	W	160	m
Column height	h	10.3	m
Beams height	h_b	1.8	m
Parapet height	h_p	1.0	m
Total height incl. parapet	H	13.1	m

Basic wind velocity	v_b	30	m/s
Terrain category		II	

Peak velocity pressure

According to EN 1991-1-4, when the height of the building is smaller than the width, the wind can be equally distributed over the height of the building, taken as reference height. This is applicable to both directions of the warehouse.

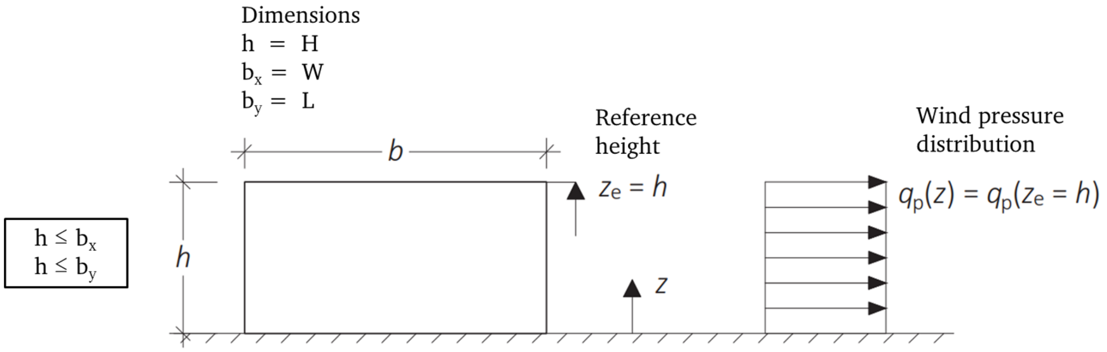


Figure B.4: Reference height z_e depending on h and b and wind pressure distribution (NEN-EN 1991-1-4:2005, 2005)

The reference mean velocity pressure q_b is calculated from the assumed basic wind velocity v_b and the air density ρ. Based on this value, the peak velocity pressure can be determined depending on the construction height and the resulting exposure factor $c_e(z)$ taken from Figure B.5. The peak velocity pressure can be calculated for a wind area of $1m^2$, and for the entire load-bearing structure it is distributed over a $10m^2$ area. The detailed calculation procedure is presented in Table B.4.

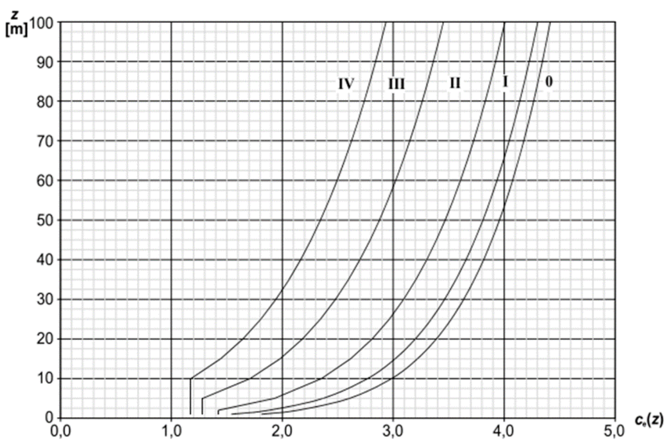


Figure B.5: Illustrations of the exposure factor $c_e(z)$ (NEN-EN 1991-1-4:2005, 2005)

Table B.4: Calculation procedure for peak velocity pressure

Description	Symbol	Value	Unit
Ratio height over crosswind dimension (x dir.)	h/b_x	0.08	
Ratio height over crosswind dimension (y dir.)	h/b_y	0.09	
Reference height	$z_e=H$	13.1	m
Air density	ρ_0	1.25	kg/m^3
Air density	$\rho_a=\rho_0/101.97$	0.012	kN/m^3
Reference mean (basic) velocity pressure	$q_b=1/2.\rho_a.v_b^2$	5.52	kN/m^2
Exposure factor	$c_e(z_e)$	2.55	
Peak velocity pressure	$q_p(z_e)=c_e(z_e).q_b$	14.07	kN/m^2
Peak velocity pressure for $1m^2$	$q_{p1}=q_p(z)$	14.07	kN/m^2
Wind area for load-bearing structure	A	10	m^2
Peak velocity pressure for load-bearing structure	$q_{p10}=q_{p1}/A$	1.41	$kN/10m^2$

External wind loads on vertical walls (horizontal)

Pressure coefficients define the wind pressure exerted on the wall surfaces of a building when wind is directed perpendicular to them. For the calculation of wind forces acting on the main load-bearing system, the coefficient $c_{pe,10}$ is used. The dimensional area facing the wind is defined as e=min(b,2h) and determines the pressure distribution over the height of the building. The dimensions of wind zones for external walls are indicated on Figure B.6. External pressure coefficients for zones A to E are then taken from Table 7.1 of EN 1991-1-4 for $h/d \leq 0.25$ in both directions of the building.

Table B.5: External pressure coefficients $c_{pe,1}$ and $c_{pe,10}$

Zone	A	B	C	D	E	F	G	H	I
$c_{pe,1}$	-1.4	-1.1	-0.5	1.0	-0.3	-1.9	-1.5	-1.2	-0.2/0.2
$c_{pe,10}$	-1.2	-0.8	-0.5	0.7	-0.3	-1.3	-0.8	-0.7	-0.2/0.2

The building factor $c_s c_d$ is taken as 1.0 when the building height is smaller than four times its depth parallel to the wind direction. This is the case for both directions of the warehouse. The external wind loads are calculated for each zone with the following formula:

$$F_{w,e,i} = q_{p10}.c_s c_d.c_{pe,10,i} \tag{B.1}$$

Because of the large dimensions of the building, there is no correlation between the wind effects on the windward side (zone D) and on the leeward side (zone E). With $h/d \leq 1$

the resulting wind forces should be multiplied by a correction factor of 0.85 to account for this effect.

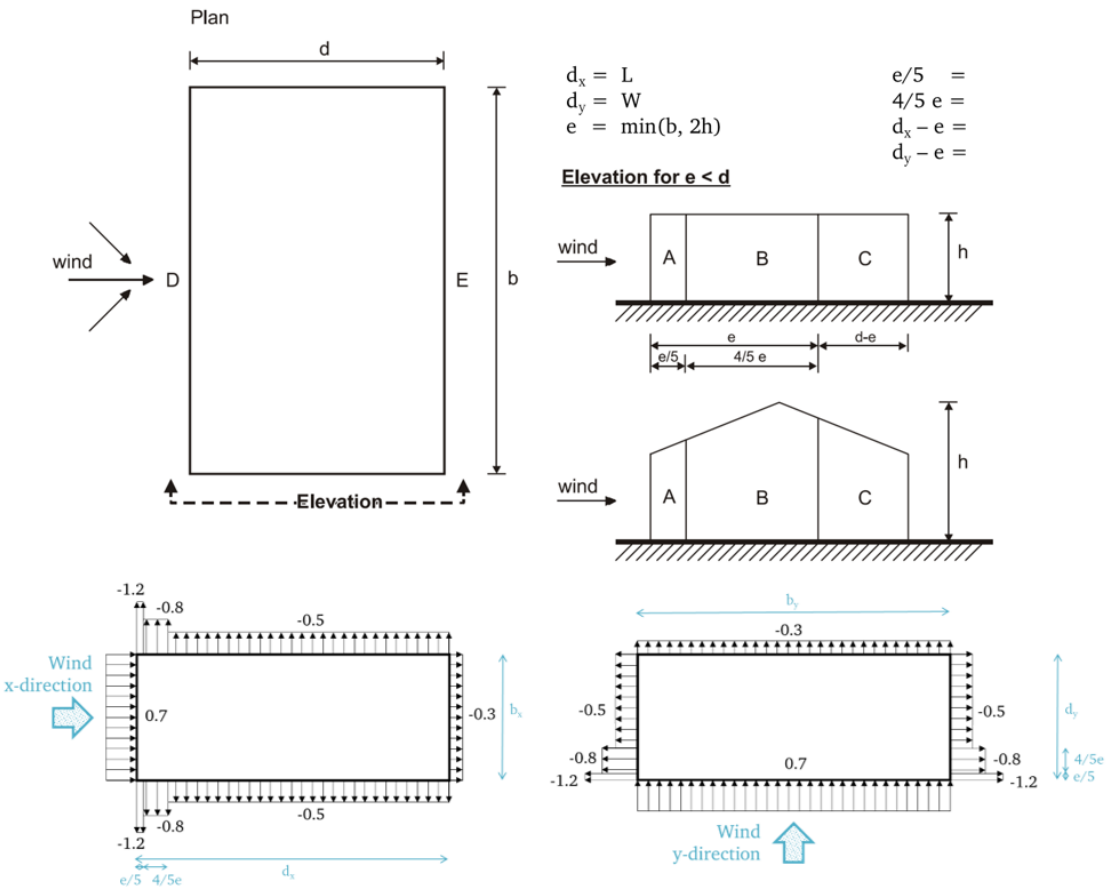


Figure B.6: Pressure coefficients $c_{pe,10}$ for vertical walls, zones A to E ($h/d \leq 0.25$, $A \geq 10m^2$) in accordance with EN 1991-1-4

Table B.6: Calculation procedure for horizontal external wind loads on vertical walls in both directions

Description	Symbol	Value	Unit
Ratio height over along-wind dimension (x dir.)	h/d_x	0.09	
Ratio height over along-wind dimension (y dir.)	h/d_y	0.08	
Structural factor	$c_s c_d$	1.0	
Dimensional area facing the wind (x and y dir.)	e=min(b,2H)	26.2	m
Ratio e over along-wind dimension (x dir.)	e/d_x	0.18	

Ratio e over along-wind dimension (y dir.)	e/d_y	0.16	
External load A (x and y dir.)	$F_{w,e,A}=q_{p10}.c_sc_d.c_{pe,10,A}$	-1.69	kN/m^2
External load B (x and y dir.)	$F_{w,e,B}=q_{p10}.c_sc_d.c_{pe,10,B}$	-1.13	kN/m^2
External load C (x and y dir.)	$F_{w,e,C}=q_{p10}.c_sc_d.c_{pe,10,C}$	-0.70	kN/m^2
External load D (x and y dir.)	$F_{w,e,D}=q_{p10}.c_sc_d.c_{pe,10,D}$	0.98	kN/m^2
External load E (x and y dir.)	$F_{w,e,E}=q_{p10}.c_sc_d.c_{pe,10,E}$	-0.42	kN/m^2
Corrected external load D (x and y dir.)	$F_{w,e,D,corr}=0.85F_{w,e,D}$	0.84	kN/m^2
Corrected external load E (x and y dir.)	$F_{w,e,E,corr}=0.85F_{w,e,E}$	-0.36	kN/m^2

The wind action on facades parallel to the wind direction results in opposite forces on both sides of the building. They compensate, and therefore do not influence the overall stability of the structure. For this reason, they are not considered in the verification of structural members as only 2D sections in the x and y directions of the structure are checked. However, they should be taken into account in the selection of suitable facade systems, especially near the corners of the building in areas A, where loads can reach their maximum value.

External wind loads on roof (vertical)

Wind forces on roofs should also be considered, as uplift forces are particularly important for the design of joints and structural members. The roof is considered flat as the roof pitch is lower than 15°. The dimensions of wind zones for the roof follow again from e=min(b,2h) and are indicated on Figure B.7.

Table B.7: Calculation procedure for vertical external wind loads on the roof in both directions

Description	**Symbol**	**Value**	**Unit**
Ratio parapet over column and beams height	$h_p/(h+h_b)$	0.08	
External load F (x and y dir.)	$F_{w,e,F}=q_{p10}.c_sc_d.c_{pe,10,F}$	-1.79	kN/m^2
External load G (x and y dir.)	$F_{w,e,G}=q_{p10}.c_sc_d.c_{pe,10,G}$	-1.17	kN/m^2
External load H (x and y dir.)	$F_{w,e,H}=q_{p10}.c_sc_d.c_{pe,10,H}$	-0.98	kN/m^2
External load I- (x and y dir.)	$F_{w,e,I-}=q_{p10}.c_sc_d.c_{pe,10,I-}$	-0.28	kN/m^2
External load I+ (x and y dir.)	$F_{w,e,I+}=q_{p10}.c_sc_d.c_{pe,10,I+}$	0.28	kN/m^2

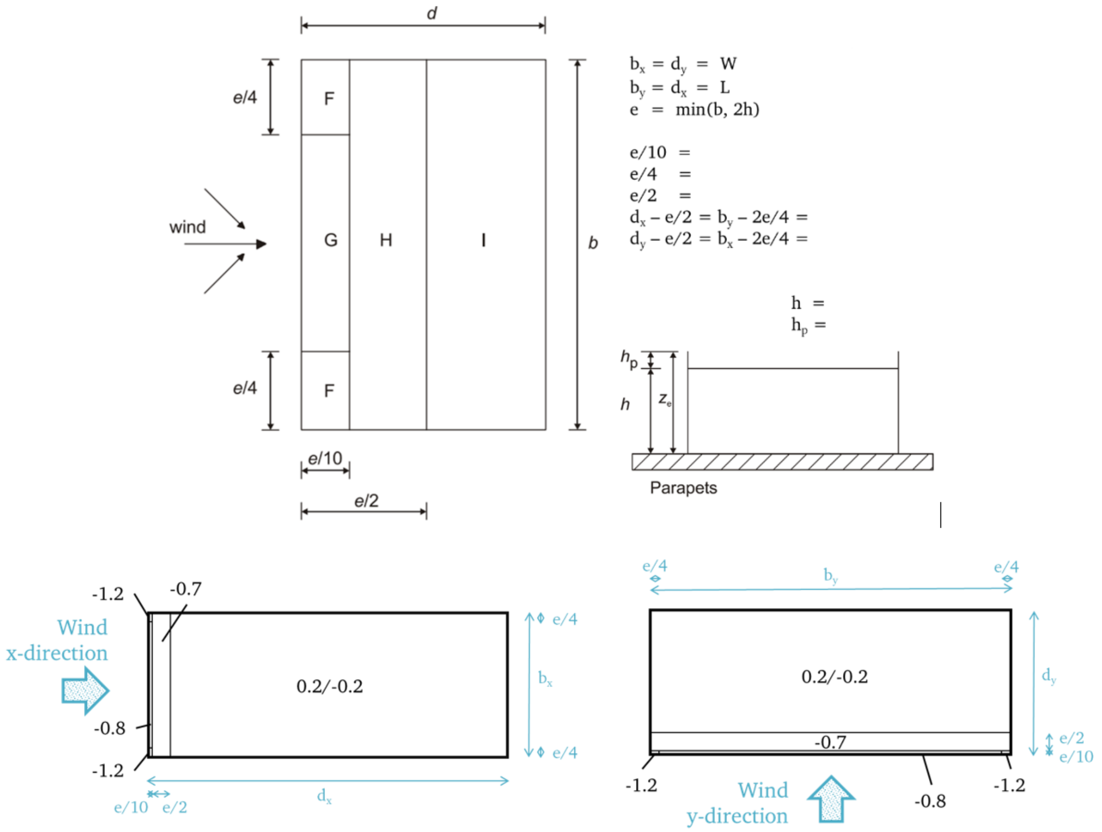


Figure B.7: Pressure coefficients $c_{pe,10}$ for flat roofs with parapets, zones F to I (h/h_p=0.1, A≥10m^2) in accordance with EN 1991-1-4

As a simplification, only area I is considered as external wind load on the roof in structural checks of the 2D sections, as it covers the largest part of the roof. The highest vertical loads are found near the facade hit by wind, especially at the corners of the building in areas F. These extreme values should be considered when precisely detailing roof systems.

Frictional forces (horizontal)

Frictional forces are exerted on surfaces parallel to the wind direction. Their value depends on the area A_{fr} of this surface, a friction coefficient c_{fr} taken from Table 7.10 of EN 1991-1-4 translating the roughness, and the peak velocity pressure $q_p(z_e)$ at the reference height. In the design, the friction force on the roof is divided over two points, first at the connection with the column and second halfway along the beam length.

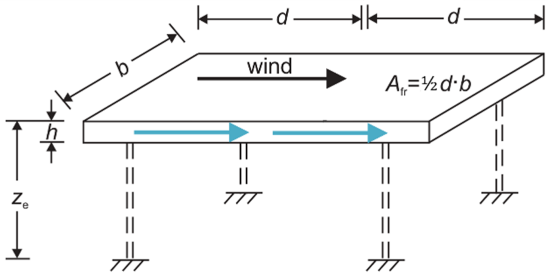


Figure B.8: Reference area from friction divided over two points along the length of roof beams, in accordance with EN 1991-1-4

Table B.8: Calculation procedure for friction forces on the roof in both directions

Description	Symbol	Value	Unit
Friction coefficient	c_{fr}	0.04	
Ctc distance columns (x dir.)	b	24	m
Ctc distance columns (y dir.)	d	16	m
Reference area for friction (x dir.)	$A_{fr,x}$=d/2.b	192	m^2
Reference area for friction (y dir.)	$A_{fr,y}$=d.b/2	192	m^2
Friction force	F_{fr}=c_{fr}.q_{p10}.A_{fr}	10.80	kN

Wind loads on the parapet (horizontal)

The largest part of the parapet is located in zone D, therefore it is the only area considered for simplification. The net pressure coefficient $c_{net,D}$=1.2 used to calculate the horizontal wind pressure is taken from Table 7.9 of EN 1991-1-4:

$$F_{w,e,D,parapet} = q_{p10}.c_s c_d.c_{net,D} = 1.69kN/m^2 \tag{B.2}$$

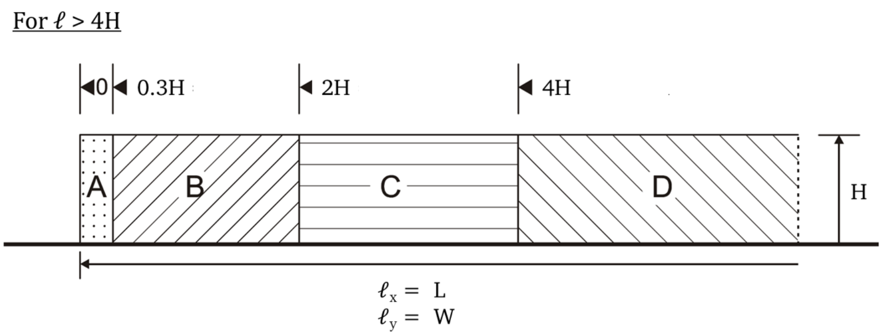


Figure B.9: Loading zones of parapets in accordance with EN 1991-1-4

Internal wind pressure (horizontal and vertical)

The internal wind pressure is calculated from internal pressure coefficients taken as -0.2 and +0.3. Where an external opening (door or a window) would be dominant when open but is considered to be closed in the ULS, during severe windstorms, this situation should be considered as an accidental design situation in accordance with EN 1990.

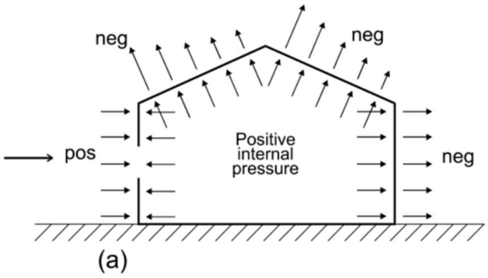


Figure B.10: Definition of positive and negative internal wind pressures

Table B.9: Calculation procedure for external wind loads on the roof in both directions

Description	Symbol	Value	Unit
General internal overpressure coefficient	$c_{p,i-}$	0.2	
General internal underpressure coefficient	$c_{p,i+}$	-0.3	
Internal load overpressure (x and y dir.)	$F_{w,i-} = q_{p10}.c_s c_d.c_{p,i-}$	0.28	kN/m^2
Internal load underpressure (x and y dir.)	$F_{w,i+} = q_{p10}.c_s c_d.c_{p,i+}$	-0.42	kN/m^2

B.5 Overview of loads

All loads applied on the structure are classified as load cases and associated into the following load groups:

- LG1 = Permanent loads
- LG2 = Live loads on the roof
- LG3 = Snow loads
- LG4 = External vertical wind, either upwards or downwards
- LG5 = External horizontal wind, either right to left or left to right
- LG6 = Internal wind, either overpressure or underpressure

B.5.1 Direction of wind actions

The division of wind load cases within groups LG4 to LG6 reflects possible orientations of wind actions on the structure, which cannot occur at the same time. External wind actions on a frame section have a vertical component oriented either upwards or downwards, and a horizontal component oriented left or right. Similarly, internal wind pressure can act towards the exterior of the building in case of overpressure, or towards the interior if there is underpressure. All possible combinations of wind load cases based on the direction of action are listed in Table B.10.

Table B.10: Configurations A to H of wind load actions depending on their direction

Wind combination	Ext. vertical wind	Ext. horizontal wind	Internal wind
A	Upwards	Right to left	Overpressure
B	Upwards	Right to left	Underpressure
C	Upwards	Left to right	Overpressure
D	Upwards	Left to right	Underpressure
E	Downwards	Right to left	Overpressure
F	Downwards	Right to left	Underpressure
G	Downwards	Left to right	Overpressure
H	Downwards	Left to right	Underpressure

B.5.2 Load combinations

Load combinations are established following EN 1990 Basis of structural design, using appropriate load factors. They describe compatible load configurations or deformations and imperfections. For the ULS, the decisive load combination is taken as:

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\psi_{0,i} Q_{k,i} \tag{B.3}$$

The first term represents the design value for the permanent load, the second term concerns the leading variable load, and the third term is the design combination value of all other variable loads.

For the SLS, the characteristic load combination is:

$$\sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \tag{B.4}$$

Partial safety factors γ are associated with individual actions on the building, to account for the influence of uncertainties and possible unfavourable deviations in their effect. Taken from EN 1990, they are indicated in Table B.11 at ULS and SLS.

Table B.11: Partial safety factors

	Permanent loading		Variable action	
Load combinations	**Unfavourable** $\gamma_{G,sup}$	**Favourable** $\gamma_{G,inf}$	**Leading** $\gamma_{Q,1}$	**Accompanying** $\gamma_{Q,i}$
Ultimate limit states (ULS)	1.35	1.00	1.50	$1.50\psi_{0,i}(i > 1)$
Serviceability limit states (SLS)	1.00	1.00	1.00	$1.00\psi_{0,i}(i > 1)$

Variable actions are associated with combination factors ψ, indicated in Table B.12 for specific types of variable loads. The characteristic combination value $\psi_0 Q_k$ is exceeded 2% of the time, the frequent value $\psi_1 Q_k$ is exceeded 5% of the time, and the quasi-permanent value $\psi_2 Q_k$ is equal to the average value over a period of time. These values are used for verifications at ULS, and to calculate short term deformations at SLS. For long-term effects like creep, G_k and $\psi_2 Q_k$ should be used in combination with the material deformation factor k_{def}.

Table B.12: Combination factors

Variable loading		ψ_0	ψ_1	ψ_2
Imposed loads	Category E: storage areas	1.0	0.9	0.8
	Category H: roofs	0.0	0.0	0.0
Snow loads		0.5	0.2	0.0
Wind loads		0.6	0.2	0.0

In our study, the possible factors for each defined load case are presented in Table B.11. Depending on the direction of wind actions, they can be either favourable or unfavourable in combination with the other permanent and variable loads. When vertical components associated with external and internal wind pressures are oriented in the opposite direction, they tend to compensate. However, the vertical wind resultant is maximum when external and internal wind both act in the same direction. Dead loads and other variable loads act downwards, either with or against the vertical wind resultant. Based on these considerations, not all wind combinations are relevant, and the governing load combinations are presented in Table B.12 with the multiplication factors for each load case.

Load group	LG1 Permanent loads				LG2.1 Cat H	LG2.2 Others	LG3 Snow	LG4 Ext. vertical wind		LG5 Ext. horizontal wind		LG6 Internal wind	
Load case	LC1 Selfweight	LC2 Roof	LC3 Façade	LC4 Services	LC5 Maintenance	LC6 Air units	LC7 Snow	LC8 Up	LC9 Down	LC10 R2L	LC11 L2R	LC12 Over	LC13 Under
Load combination	$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}$				$\gamma_{Q,1}Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$								
Gamma factors γ *Permanent and leading variable coefficients*	ULS unfavourable $\gamma_{G,sup}=1.35$ ULS favourable $\gamma_{G,inf}=1.00$ SLS $\gamma_G=1.00$				ULS $\gamma_Q=1.50$ SLS $\gamma_Q=1.00$								
Psi factors $\psi_{0,i}$	-				$\psi_0=0.0$	$\psi_0=1.0$	$\psi_0=0.5$	$\psi_0=0.6$					
$\gamma_{Q,i}\psi_{0,i}$ *Accompanying variable coefficient*					ULS = 0.00 SLS = 0.00	ULS = 1.50 SLS = 1.00	ULS = 0.75 SLS = 0.50	ULS = 0.90 SLS = 0.60					

Figure B.11: Possible factors for each defined load case

A-H = wind direction 0XX = ULS unfav G 0XX = ULS fav G 1XX = SLS	XX1 Maintenance XX2 Air units XX3 Snow XX4 Wind	LG1 Permanent loads				LG2.1 Cat H	LG2.2 Others	LG3 Snow	LG4 Ext. vertical wind		LG5 Ext. horizontal wind		LG6 Internal wind	
		LC1 Selfweight	LC2 Roof	LC3 Facade	LC4 Services	LC5 Maintenance	LC6 Air units	LC7 Snow	LC8 Up	LC9 Down	LC10 Right to Left	LC11 Left to Right	LC12 Overpressure	LC13 Underpressure
A	WIND 8.10.12 Maximum wind UP													
A004	Wind - UP (no snow)	1.00	1.00	1.00	1.00	0.00	1.50	0.00	**1.50**		**1.50**		**1.50**	
A104	Wind - UP (no snow)	1.00	1.00	1.00	1.00	0.00	1.00	0.00	**1.00**		**1.00**		**1.00**	
B	WIND 8.10.13 Vertical external and internal compensate													
C	WIND 8.11.12 Maximum wind UP													
C004	Wind - UP (no snow)	1.00	1.00	1.00	1.00	0.00	1.50	0.00	**1.50**			**1.50**	**1.50**	
C104	Wind - UP (no snow)	1.00	1.00	1.00	1.00	0.00	1.00	0.00	**1.00**			**1.00**	**1.00**	
D	WIND 8.11.13 Vertical external and internal compensate													
E	WIND 9.10.12 Vertical external and internal compensate													
F	WIND 9.10.13 Maximum wind DOWN													
F001	Maintenance - DOWN	1.35	1.35	1.35	1.35	**1.50**	1.50	0.75		0.90	0.90			0.90
F002	Air units - DOWN	1.35	1.35	1.35	1.35	0.00	**1.50**	0.75		0.90	0.90			0.90
F003	Snow - DOWN	1.35	1.35	1.35	1.35	0.00	1.50	**1.50**		0.90	0.90			0.90
F004	Wind - DOWN	1.35	1.35	1.35	1.35	0.00	1.50	0.75		**1.50**	**1.50**			**1.50**
F101	Maintenance - DOWN	1.00	1.00	1.00	1.00	**1.00**	1.00	0.50		0.60	0.60			0.60
F102	Air units - DOWN	1.00	1.00	1.00	1.00	0.00	**1.00**	0.50		0.60	0.60			0.60
F103	Snow - DOWN	1.00	1.00	1.00	1.00	0.00	1.00	**1.00**		0.60	0.60			0.60
F104	Wind - DOWN	1.00	1.00	1.00	1.00	0.00	1.00	0.50		**1.00**	**1.00**			**1.00**
G	WIND 9.11.12 Vertical external and internal compensate													
H	WIND 9.11.13 Maximum wind DOWN													
H001	Maintenance - DOWN	1.35	1.35	1.35	1.35	**1.50**	1.50	0.75		0.90		0.90		0.90
H002	Air units - DOWN	1.35	1.35	1.35	1.35	0.00	**1.50**	0.75		0.90		0.90		0.90
H003	Snow - DOWN	1.35	1.35	1.35	1.35	0.00	1.50	**1.50**		0.90		0.90		0.90
H004	Wind - DOWN	1.35	1.35	1.35	1.35	0.00	1.50	0.75		**1.50**		**1.50**		**1.50**
H101	Maintenance - DOWN	1.00	1.00	1.00	1.00	**1.00**	1.00	0.50		0.60		0.60		0.60
H102	Air units - DOWN	1.00	1.00	1.00	1.00	0.00	**1.00**	0.50		0.60		0.60		0.60
H103	Snow - DOWN	1.00	1.00	1.00	1.00	0.00	1.00	**1.00**		0.60		0.60		0.60
H104	Wind - DOWN	1.00	1.00	1.00	1.00	0.00	1.00	0.50		**1.00**		**1.00**		**1.00**

Figure B.12: Governing load combinations with multiplication factors

C

Structural design of timber baseline A

The goal of this section is to determine specific profiles for the structural members of the timber baseline design A and to detail connections such that they withstand the loads applied on the building. Ultimately, the environmental impact of timber baseline A will be compared with the impact of the steel reference warehouse. The design process is based upon the results obtained from the SCIA model and structural checks at ULS and SLS as explained in Appendix A.

C.1 General design process

The steps of the structural design process are illustrated on Figure C.1. After setting up and running the structural model, three design steps are carried out, respectively checking the magnitude of second order effects, detailing the column base connection, and dimensioning cross-sections of timber beams and columns according to Eurocode requirements. The main variables of the iterative design process are the size of structural members, adjusted to verify the design checks at each step. All relevant design requirements should be satisfied before moving on to the next step. The inputs of the SCIA model and Excel calculations should be carefully defined in every iteration, as cross-section dimensions also influence the detailing of column base connections or the self-weight of structural members.

The goal of this process is to calculate the dimensions of structural members and the connection details such that all relevant Eurocode requirements are fulfilled, while maximising utility checks to achieve an economic timber structure and minimise material use. At an initial design level, the number of structural checks to be performed is limited to the essential and elements should be optimised for conservative unity checks at ULS and SLS, to account for the limited level of detail at that stage and possible changes in the future design. The deflection requirements at SLS are based upon the national annex of EN 1990 A1.4 and rules provided by the client. Design checks and conservative values of unity checks are specified in Table C.1.

Table C.1: ULS and SLS design criteria

ULS		**SLS**
Section checks (columns & beams) - Compression - Bending - Shear - Combination of bending and axial force	Stability checks - Flexural buckling - Lateral torsional buckling - Beam-column buckling	- Maximum horizontal deflection u_{max}=h/175 - Maximum deflection of roof elements $w_{net,fin}$=L/250
Unity check $\leq$ 0.85	**Unity check $\leq$ 0.80**	**Unity check $\leq$ 0.80**
Design of connections		
Type of connection (shear or moment connection) Rotational stiffness of moment connections		

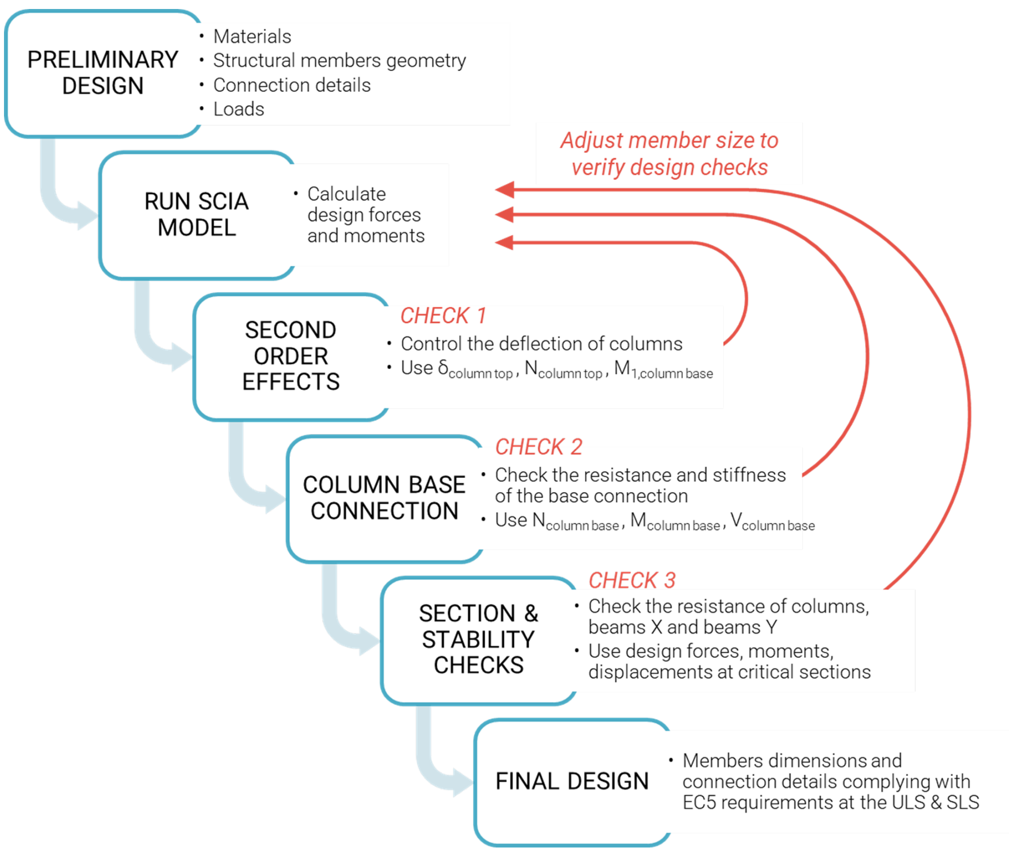


Figure C.1: Overview of the structural design process

C.2 SCIA model

The three-dimensional stability system of the load-bearing frame is modelled as 2D sections in the longitudinal and transverse directions of the warehouse, as illustrated on Figure C.2. The roof beams and main columns are connected by hinges at the top, and lateral stiffness is provided by semi-rigid column base connections. Additional facade columns on each side are modelled and dimensioned separately from the two sections.

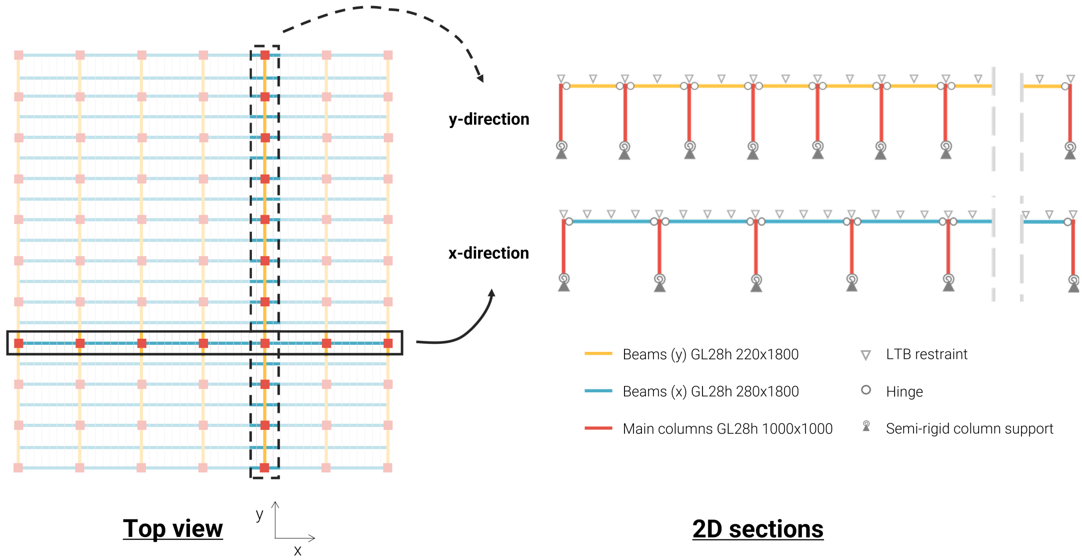


Figure C.2: Mechanics scheme of the stabilising frames in the x and y-directions for timber baseline A. Columns and beams are connected by pinned connections in both directions, the stiffness of the column base connection is 110 MNm/rad as calculated in Section C.5.

C.3 Loads

Based on load paths, loads applied on the 3D structure should be converted to their equivalent in the plane of the modelled 2D sections in both directions, as illustrated on Figure C.3. Therefore, surface loads should be converted to line and point loads, and out-of-plane line loads should be turned into point loads. Uniformly distributed loads extend over the area or length of structural elements and include the dead weight of building components, live snow loads on a roof or wind loads on walls.

Surface roof loads, for instance, will first be transferred from roof panels and purlins to roof beams (x) as line loads, then to roof beams (y) as points loads at the location of roof beams (x) supports, and finally to the main columns, all the way down into the foundations and ground. Similarly, on the facade, asymmetric lateral wind loads are taken up by facade columns and mullions, and then transferred to the roof and foundations only by facade columns participating in the main stability system. Therefore, wind actions on surrounding mullions that are to be transferred to the roof diaphragm are modelled by a point load at the top of the main facade columns.

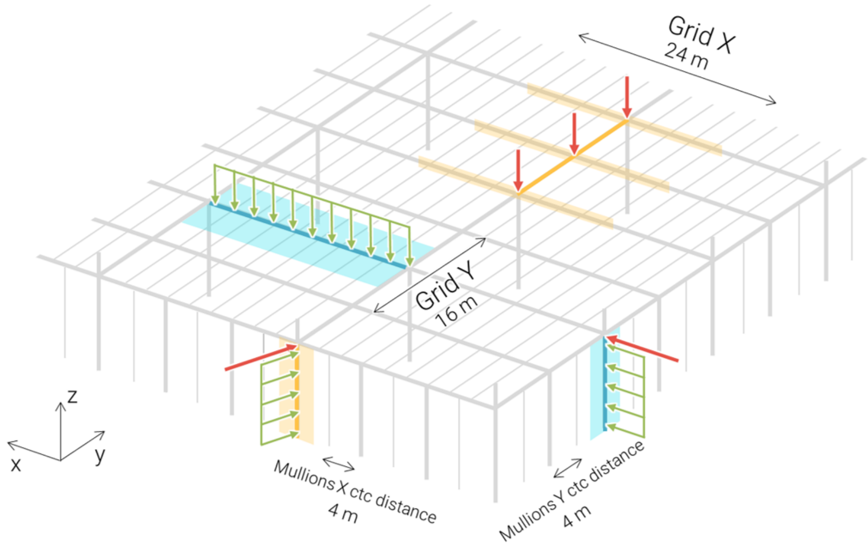


Figure C.3: Surface loads on the roof and facades are converted to line loads and point loads in the 2D models. Loads on section (x) are represented in blue and loads on section Y are represented in yellow

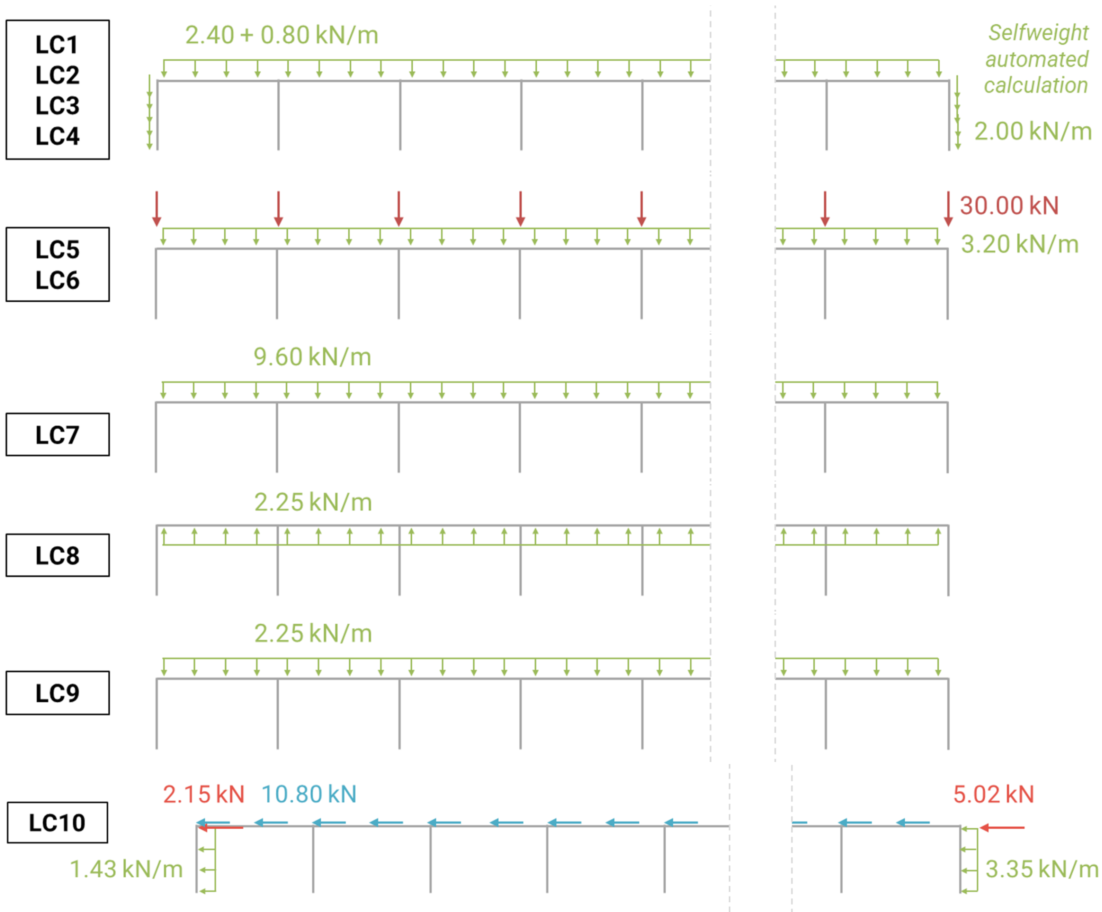
C.3.1 Loads on the 2D model (x-direction)

The structural design of timber baseline A warehouse is done based on 2D models, therefore surface loads are translated into line loads and point loads depending on the centre-to-centre distance between structural members. Horizontal wind loads on the facade are taken up by vertical elements, facade columns and mullions, placed every 4 metres. Vertical loads on the roof are first carried by roof beams in the x-direction, with a centre-to-centre distance set to 8 metres. An overview of the loads and distances considered for the 2D section (x) is given in Table C.2, and all load cases are visualised on Figure C.4.

Table C.2: Overview of loads applied on the 2D warehouse section in the x-direction

Description	3D load		CTC		2D load	
Permanent actions						
LC1 - Self-weight frame	*Automatically calculated by SCIA*					
LC2 - Roof purlins and panels	0.30	kN/m^2	8	m	2.40	kN/m
LC3 - Facade rails and cladding	0.50	kN/m^2	4	m	2.00	kN/m
LC4 - Services	0.10	kN/m^2	8	m	0.80	kN/m
Live loads (vertical)						
LC5 - Maintenance load	0.40	kN/m^2	8	m	3.20	kN/m
LC6 - Air unit (on top of every column)	30.0	kN			30.0	kN
Snow loads (vertical)						
LC7 - Snow load	1.20	kN/m^2	8	m	9.60	kN/m
External wind (vertical)						
LC8 - Wind I (upwards)	0.28	kN/m^2	8	m	2.25	kN/m
LC9 - Wind I (downwards)	0.28	kN/m^2	8	m	2.25	kN/m

External wind (horizontal, left to right (LC10) or right to left (LC11))						
LC10/LC11 - Friction due to wind	10.8	kN			10.8	kN
LC10/LC11 - Wind D on facade column	0.84	kN/m^2	4	m	3.35	kN/m
LC10/LC11 - Wind D from mullions	0.84	kN/m^2	4	m	5.02	kN
LC10/LC11 - Wind E on facade column	0.36	kN/m^2	4	m	1.43	kN/m
LC10/LC11 - Wind E from mullions	0.36	kN/m^2	4	m	2.15	kN
Internal wind (vertical + horizontal)						
LC12 - Wind F (up - overpressure)	0.28	kN/m^2	8	m	2.25	kN/m
LC12 - Wind F (up - overpressure)	0.28	kN/m^2	4	m	1.13	kN/m
LC13 - Wind G (down - underpressure)	0.42	kN/m^2	8	m	3.38	kN/m
LC13 - Wind G (down - underpressure)	0.42	kN/m^2	4	m	1.69	kN/m



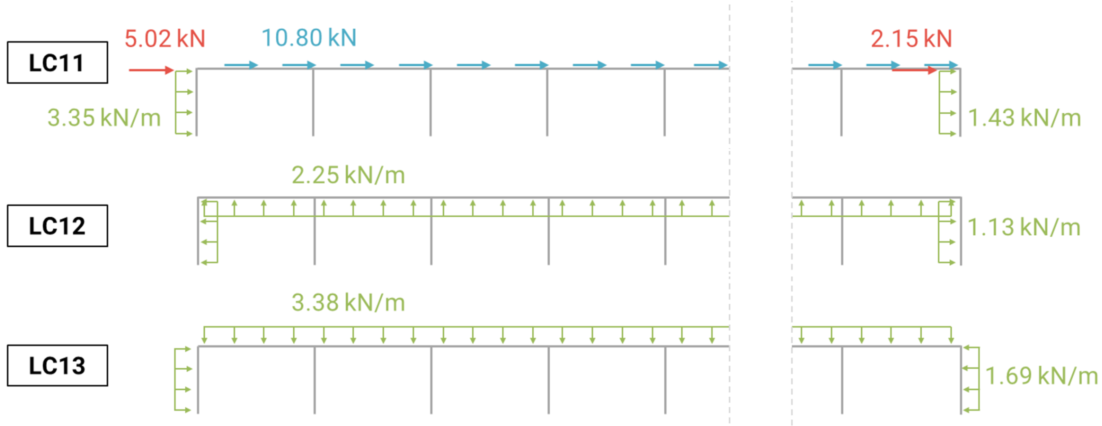


Figure C.4: Overview of loads applied on the 2D warehouse section in the x-direction

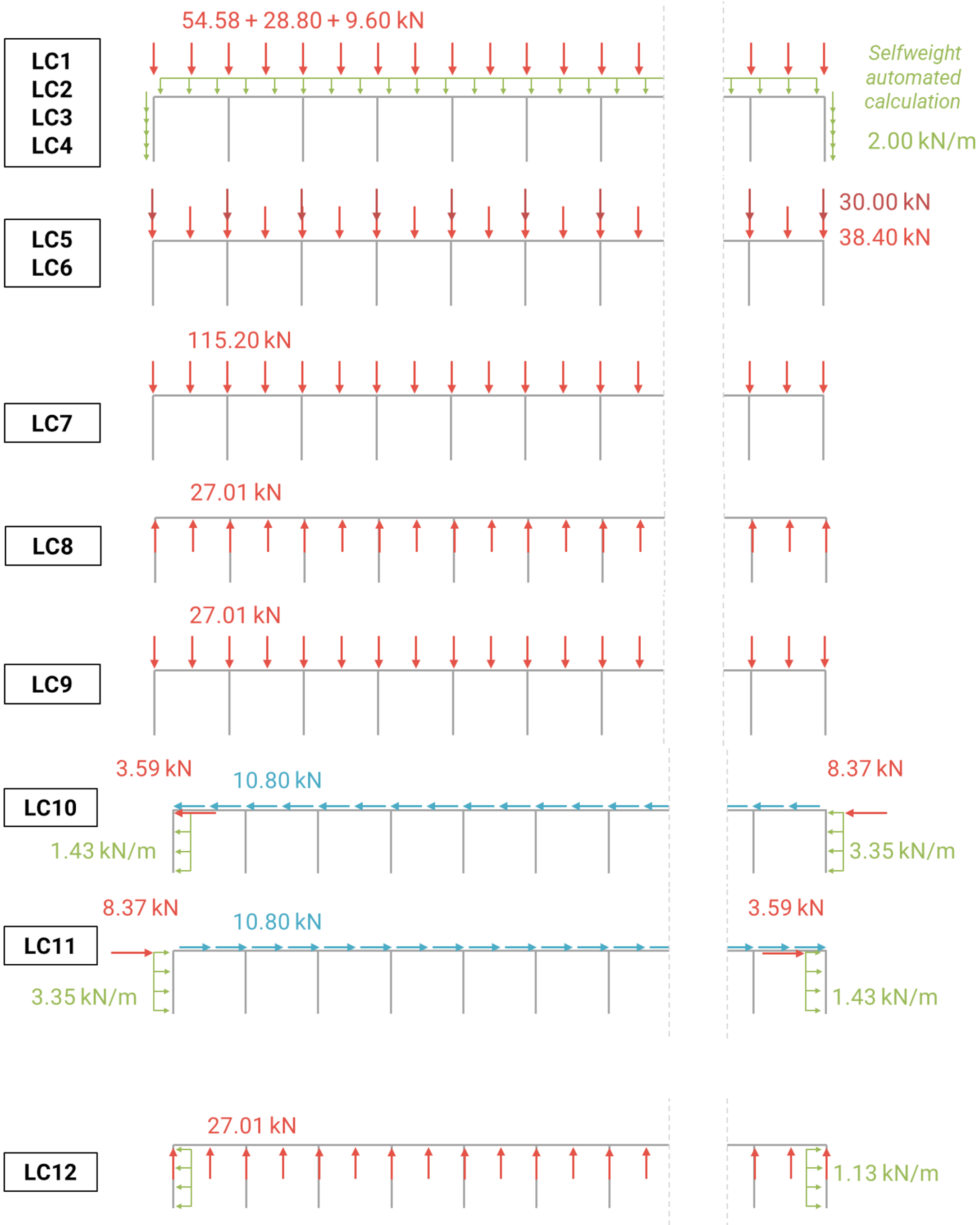
C.3.2 Loads on the 2D model (y-direction)

Vertical elements on the facade are also placed every 4 metres in the other direction. Vertical loads on the roof as well as the selfweight of 24 metres span roof beams (x), are transferred to roof beams (y) at midspan and to the columns at the ends as point loads at the location of connections. An overview of the loads considered for the 2D section (y) is given in Table C.3, and all load cases are visualised on Figure C.5.

Table C.3: Overview of loads applied on the 2D warehouse section in the y-direction

Description	3D load		CTC		2D load	
Permanent actions						
LC1 - Self-weight frame	*Automatically calculated by SCIA*					
LC2.1 - Weight roof beams (x)	2.27	kN/m	24	m	54.58	kN
LC2.2 - Roof purlins and panels	2.40	kN/m	24	m	28.80	kN
LC3 - Facade rails and cladding	0.50	kN/m^2	4	m	2.00	kN/m
LC4 - Services	0.80	kN/m	24	m	9.60	kN
Live loads (vertical)						
LC5 - Maintenance load	3.20	kN/m	24	m	38.40	kN
LC6 - Air unit (on top of every column)	30.0	kN			30.0	kN
Snow loads (vertical)						
LC7 - Snow load	9.60	kN/m	24	m	115.20	kN
External wind (vertical)						
LC8 - Wind I (upwards)	2.25	kN/m	24	m	27.01	kN
LC9 - Wind I (downwards)	2.25	kN/m	24	m	27.01	kN
External wind (horizontal, left to right (LC10) or right to left (LC11))						
LC10/LC11 - Friction due to wind	10.8	kN			10.8	kN
LC10/LC11 - Wind D on facade column	0.84	kN/m^2	4	m	3.35	kN/m
LC10/LC11 - Wind D from mullions	0.84	kN/m^2	4	m	8.37	kN
LC10/LC11 - Wind E on facade column	0.36	kN/m^2	4	m	1.43	kN/m
LC10/LC11 - Wind E from mullions	0.36	kN/m^2	4	m	3.59	kN
Internal wind (vertical + horizontal)						

LC12 - Wind F (up - overpressure)	2.25	kN/m	24	m	27.01	kN
LC12 - Wind F (up - overpressure)	0.28	kN/m^2	4	m	1.13	kN/m
LC13 - Wind G (down - underpressure)	3.38	kN/m	24	m	40.51	kN
LC13 - Wind G (down - underpressure)	0.42	kN/m^2	4	m	1.69	kN/m



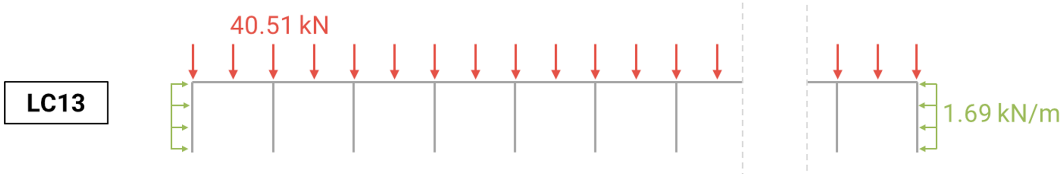


Figure C.5: Overview of loads applied on the 2D warehouse section in the y-direction

C.3.3 Loads on facade columns

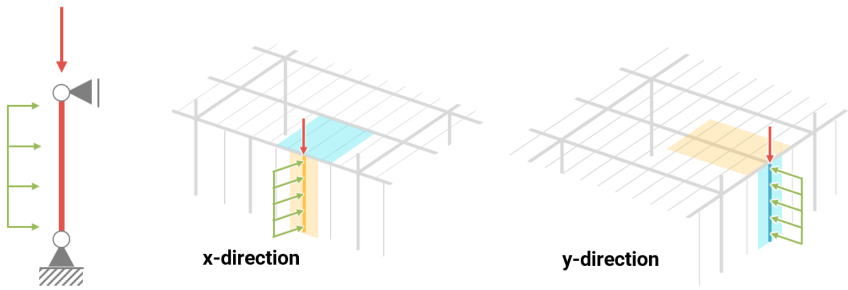


Figure C.6: Facade column pinned at both ends, subjected to vertical loads at the top and horizontal distributed loads along the height

Facade columns are added in addition to the main columns, every 8 metres around the perimeter of the warehouse. Pinned at both ends, they transfer horizontal wind loads to the roof and foundation pads, and carry dead loads from roof beams. The relevant loading areas are indicated on Figure C.6, and the loads on facade columns in both directions are detailed in Tables C.4 and C.5.

Table C.4: Overview of loads applied on facade columns along the warehouse length (x-direction)

Description	3D load		CTC/Area		2D load	
Permanent actions						
LC1 - Self-weight frame	*Automatically calculated by SCIA*					
LC2.1 - Weight roof beams (y)	1.79	kN/m	8	m	14.30	kN
LC2.1 - Weight roof beams (x)	2.27	kN/m	12	m	27.29	kN
LC2.2 - Roof purlins and panels	0.30	kN/m^2	64	m^2	19.20	kN
LC3 - Facade rails and cladding	0.50	kN/m^2	4	m	2.00	kN/m
LC4 - Services	0.10	kN/m^2	64	m^2	6.40	kN
Live loads (vertical)						
LC5 - Maintenance load	0.40	kN/m^2	64	m^2	25.60	kN
Snow loads (vertical)						
LC7 - Snow load	1.20	kN/m^2	64	m^2	76.80	kN
External wind (vertical)						
LC8 - Wind I (upwards)	0.28	kN/m^2	64	m^2	18.01	kN
LC9 - Wind I (downwards)	0.28	kN/m^2	64	m^2	18.01	kN
External wind (horizontal, left to right (LC10) or right to left (LC11))						
LC10/LC11 - Wind D on facade column	0.84	kN/m^2	4	m	3.35	kN/m

LC10/LC11 - Wind E on facade column	0.36	kN/m^2	4	m	1.43	kN/m
Internal wind (vertical + horizontal)						
LC12 - Wind F (up - overpressure)	0.28	kN/m^2	64	m^2	18.01	kN
LC12 - Wind F (up - overpressure)	0.28	kN/m^2	4	m	1.13	kN/m
LC13 - Wind G (down - underpressure)	0.42	kN/m^2	64	m^2	27.01	kN
LC13 - Wind G (down - underpressure)	0.42	kN/m^2	4	m	1.69	kN/m

Table C.5: Overview of loads applied on facade columns along the warehouse width (y-direction)

Description	**3D load**		**CTC/Area**		**2D load**	
Permanent actions						
LC1 - Self-weight frame	*Automatically calculated by SCIA*					
LC2.1 - Weight roof beams (x)	2.27	kN/m	12	m	27.29	kN
LC2.2 - Roof purlins and panels	0.30	kN/m^2	96	m^2	28.80	kN
LC3 - Facade rails and cladding	0.50	kN/m^2	4	m	2.00	kN/m
LC4 - Services	0.10	kN/m^2	96	m^2	9.60	kN
Live loads (vertical)						
LC5 - Maintenance load	0.40	kN/m^2	96	m^2	38.40	kN
Snow loads (vertical)						
LC7 - Snow load	1.20	kN/m^2	96	m^2	115.20	kN
External wind (vertical)						
LC8 - Wind I (upwards)	0.28	kN/m^2	96	m^2	27.01	kN
LC9 - Wind I (downwards)	0.28	kN/m^2	96	m^2	27.01	kN
External wind (horizontal, left to right (LC10) or right to left (LC11))						
LC10/LC11 - Wind D on facade column	0.84	kN/m^2	4	m	3.35	kN/m
LC10/LC11 - Wind E on facade column	0.36	kN/m^2	4	m	1.43	kN/m
Internal wind (vertical + horizontal)						
LC12 - Wind F (up - overpressure)	0.28	kN/m^2	96	m^2	27.01	kN
LC12 - Wind F (up - overpressure)	0.28	kN/m^2	4	m	1.13	kN/m
LC13 - Wind G (down - underpressure)	0.42	kN/m^2	96	m^2	40.51	kN
LC13 - Wind G (down - underpressure)	0.42	kN/m^2	4	m	1.69	kN/m

C.3.4 Load combinations

The load combinations described in section B.5.2 are applied in both directions. The governing load combinations are summarised in Table C.6.

Table C.6: Governing load combinations for the design of timber structural members

Load combination	Critical check
F003/H003 - Snow down	Beams: compression, bending, shear Columns: compression
F004/H004 - Wind down	Columns: bending, shear
A004/C004 - Wind up, no snow	Beams: tension
F103/H103 - Snow down	Beams: deflection
F104/H104 - Wind down	Columns: deflection

C.4 Second order effects

Internal forces and moments may be determined using either:

- First order analysis, based on the initial geometry of the structure
- Second order analysis, based on the deflected state of the structure

Second order effects are caused by the combination of gravitational and horizontal loads. Horizontal actions like wind and equivalent forces from geometric imperfections induce lateral deflection of the structure, therefore shifting its centre of gravity relative to the base. As a result, gravitational loads are displaced and create an additional moment, further increasing the horizontal deflection. For a sway structure particularly, this second order (or $P-\Delta$)effect should be considered as it is more sensitive to deflections. In this project, it is also relevant when considering increased vertical loads caused by heavy green roof systems.

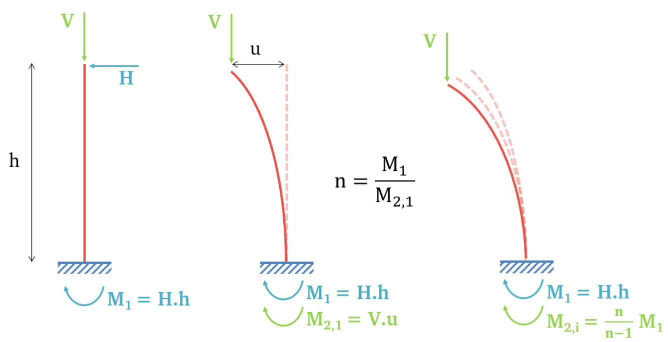


Figure C.7: Second order effect increasing bending moments for fixed-based columns subjected to combined gravitational and horizontal loads

The evaluation of second order effects is carried out based on results from the SCIA model:

- δ_{top} = 46.80mm the maximal calculated deflection at the main columns top (the maximum allowable horizontal deflection is $\delta_{allowable}$ = h/175 = 58.86mm, therefore $UC_{SLS} = \delta_{top}/\delta_{allowable}$ = 0.80).

- N_{top} = 8.41E+05 N the compressive force acting at the column top
- $M_{1,base}$ = 3.85E+08 Nmm the first order moment calculated at the column base

The calculation procedure, illustrated on Figure C.7, aims at evaluating the magnitude of a factor n to determine whether horizontal deflections induced by second order effects can be controlled:

- If n < 6 horizontal deflections will increase beyond reasonable limits, and the stability of the structure will be compromised. The dimensions of the column cross-section should be increased.
- If 6 ≤ n < 10 horizontal deflections remain within reasonable limits and can be controlled The second order moment $M_{2,i}$ generated at the column base should be taken as the design value for the detailing of the column base connection and column section checks.
- If 10 ≤ n second order effects can be neglected. The first order moment M_1 calculated at the column base is kept as the design value for the detailing of the column base connection and column section checks.

Table C.7: Calculation procedure for second order effects of the main columns in timber baseline A

Description	**Symbol**	**Formula**	**Value**	**Unit**
Second order moment at the base	$M_{2,1}$	$N_{top}\delta_{top}$	3.93E+07	Nmm
n factor	n	M_1/M_2	9.77	
Second order ratio	$n/(n-1)$		1.11	
Design moment from second order effects	$M_{2,i}$	$\frac{n}{n-1}M_1$	4.28E+08	Nmm

Because n is comprised between 6 and 10, the second order moment $M_{2,i}$ should be used for design checks at the column base.

C.5 Column base connection

The next step of the design process is to check the capacity of the column base connection, using bonded-in rods parallel to the grain of the timber column.

C.5.1 Glued-in rods connection

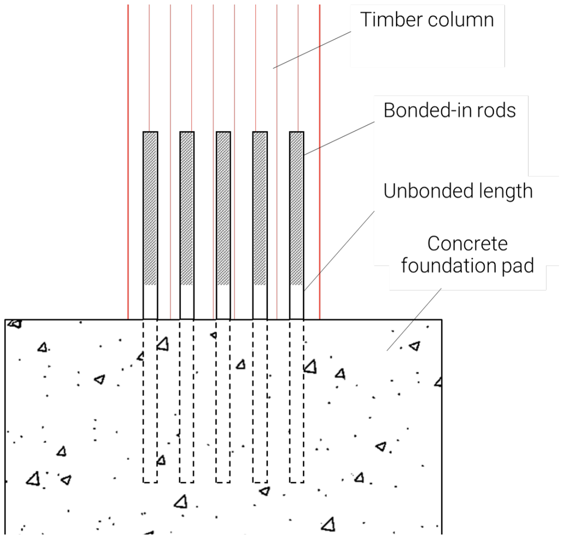


Figure C.8: Main components of the glued-in rods connection

The base connection detail is illustrated in Figure C.8, the dimensions and number of rods are not representative of the real design. Steel rods are glued into the timber section, first pouring the glue in holes drilled at the column based and then inserting the rods. At the other end, they are bonded directly in the in situ concrete foundation pads, with sufficient length to be considered fixed.

Geometrical and material characteristics

The characteristics of the glued-in steel rods are illustrated on Figure C.9 and design requirements from the new timber Eurocode draft are detailed in Table C.8.

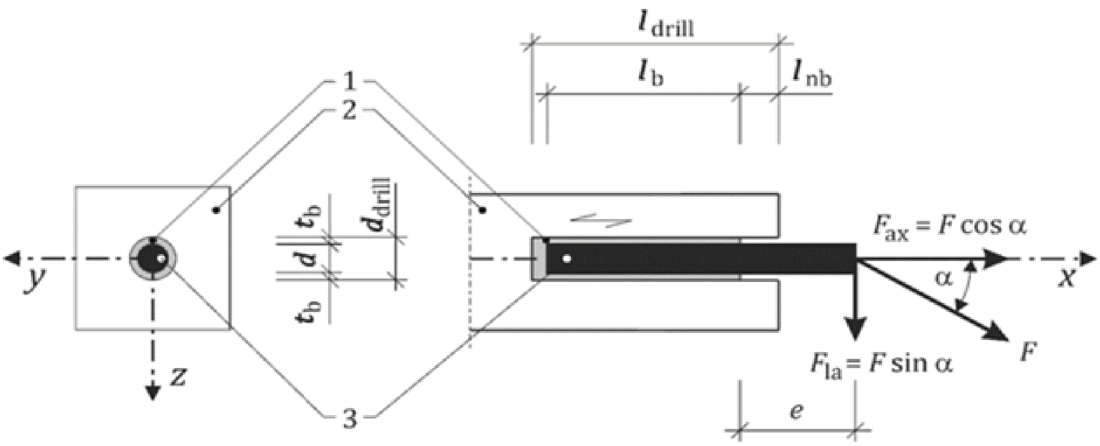


Figure C.9: Geometrical characteristics of glued-in rods

Table C.8: Glued in rods connection - input data

Description	Symbol	Formula	Value	Unit
Steel rod characteristics – Material				
Steel rod strength class			5.6	
Characteristic yield strength	$f_{y,k}$		300	N/mm^2
Characteristic ultimate strength	$f_{u,k}$		500	N/mm^2
Required minimum characteristic bond-line strength (EN 17334)	$f_{vr,k}$	4.0 if l_{bef} $\leq$250mm 5.25-0.005l_{bef} if 250<l_{bef} $\leq$500mm 3.5-0.0015l_{bef} if 500<l_{bef} $\leq$1000mm	2.53	N/mm^2
Steel modulus of elasticity	E_s		210000	N/mm^2
Steel rod characteristics – Geometry				
Metric threaded rod		6 < d < 30mm	M18	
Bond line thickness	t_b		2	mm
Rod diameter	d		18	mm
Rod nominal stress area	$A_{s,nom}$		192	mm
Drill hole diameter	d_{drill}	$2t_b + d$	22	mm
Angle of rods to the grain	ϵ		0	
Rod length	l_{rod}		730	mm
Anchorage length	l_b	max(0.5d^2;10d) $\leq l_b \leq$ 3000mm	650	mm
Ratio anchorage length over rod diameter	l_b/d	$l_b/d < 110$	36	
Distance between lateral load and bondline	e	$l_{rod} - l_b \geq 0$	80	mm
Unbonded length	l_{nb}	$l_{nb} \geq 5d$ reduces risk of splitting	80	mm
Drill hole end length	l_{end}		0	mm
Drill hole length	l_{drill}	$l_{end} + l_b + l_{nb}$	730	mm
Timber column characteristics – Glulam GL28h				
Failure strain of softwood timber parallel to the grain	$\epsilon_{u,timber}$		0.0024	
Tensile strength parallel to the grain	f_{t0k}		22.3	N/mm^2
Mean modulus of elasticity	E_{0mean}		12600	N/mm^2
Design modulus of elasticity	E_d	E_{0mean}/γ_M	10080	N/mm^2
Characteristic density	ρ_k		425	kg/m^3
Cross-sectional area	A	bh	1.00E+06	mm^2
Modification factor	k_{mod}		0.90	

The number of rods determines the moment capacity of the connection, as well as its axial and lateral resistance. Therefore, the size of the column may need to be adapted to fit a certain amount of rods and achieve sufficient strength of the base. The layout of rods within the column section is illustrated on Figure C.10.

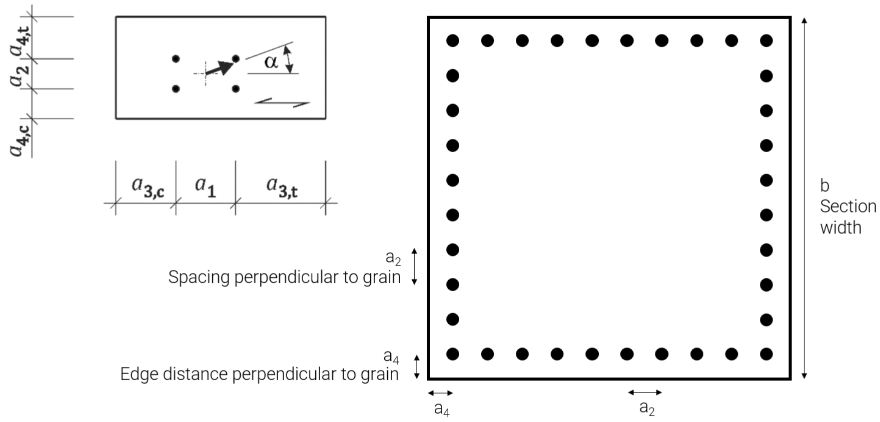


Figure C.10: Glued-in rods layout within the timber section

The maximum number of rods to fit within the timber section is derived from the minimum distance between rods and with the edge, for rods bonded-in parallel to the grain, axially or laterally loaded. These distances are expressed as functions of the rod diameter, as indicated in Table C.9.

Table C.9: Glued in rods connection - Rods spacing and total number within the timber section

Description	Symbol	Formula	Value	Unit
Steel rod characteristics – Spacing, edge and end distances within the timber section				
Min. spacing between rods	$a_{2,min}$	5d	90	mm
Min. compressive edge distance	$a_{4c,min}$	2.5d	45	mm
Min. tensile edge distance	$a_{4t,min}$	4d	72	mm
Min. edge distance	$a_{4,min}$	$max(a_{4c,min}; a_{4t,min})$	72	mm
Column width	b		1000	mm
Max. nb of rods along the section width	N	$b \geq 2a_{4,min} + (N-1)\,a_{2,min}$	10	
Distance rods to edge	a_4		72	mm
Spacing between rods	a_2	(b-2a_4)/(N-1)	95	mm
Total nb of rods in the section	N_{tot}	4(N-1)	36	

Moment capacity

The moment capacity of the base connection is derived from on the interaction between the timber column and bonded-in rods in rotation. Tensile forces are taken up by steel rods, while compression is assumed taken up only by part of the timber section. It is assumed that plane sections remain plane. Although the actual stress-strain relationship in timber products is generally non-linear when loaded to failure, an elastic behaviour is assumed in the timber zone under compression. The explanation of the calculation procedure is followed by numerical results, indicated in Table C.10.

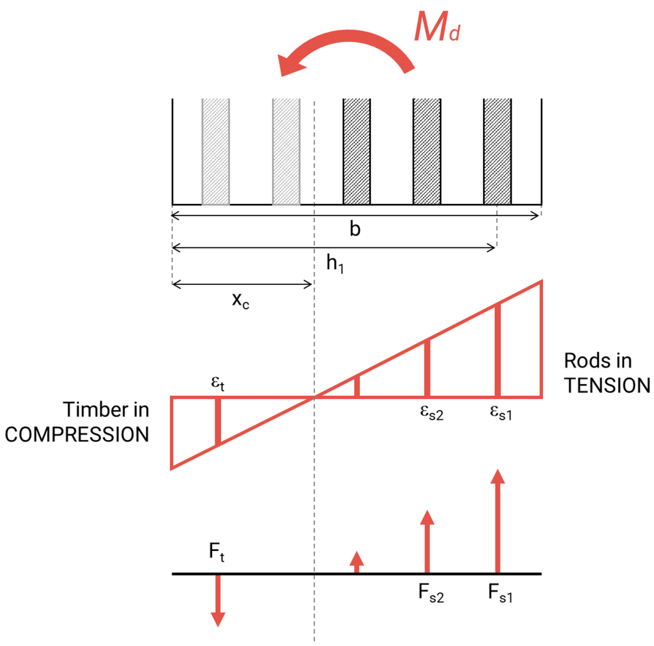


Figure C.11: Principle of moment capacity calculation for a glued-in rods connection, strain/forces diagram

Rods in tension are numbered as indicated on Figure C.11, the first row being closest to the tensile edge, and increasing indices until the last row in tension, closest to the neutral axis. Because of the square layout of rods within the section, rows closest to the edge include N=10 rods, while inside rows include 2 only. The distance of row i to the compressive edge:

$$h_i = a_4 + (N - i)a_2 \tag{C.1}$$

The size of the compressive zone x_c indicates the location of the neutral axis in the timber section, and is determined from an iterative calculation from the moment capacity. The moment capacity is reached when the first row of rods in tension yields ($\epsilon_{s1} = f_{yd}/E_s$). From the linear strain diagram:

$$\frac{\epsilon_{si}}{h_i - x_c} = \frac{\epsilon_{s1}}{h_1 - x_c} \Rightarrow \epsilon_{si} = \frac{f_{yd}}{E_s}\frac{h_i - x_c}{h_1 - x_c} \tag{C.2}$$

The area of steel rods in tension A_{si} in row i is calculated by multiplying the nominal stress area $A_{s,nom}$ of a single rod by the number of rods $N_{rods,i}$ in that row. It is used to calculate the tensile force F_{si} in rods of row i:

$$F_{si} = A_{si} f_{yd} \frac{h_i - x_c}{h_1 - x_c} \tag{C.3}$$

The index N_t of the last row of rods in tension is determined from the size of the compressive zone and geometrical characteristics:

$$x_c < h_{Nt} = a_4 + (N_{rows} - N_t)a_2 \Rightarrow N_t < (a_4 - x_c)/a_2 + N_{rows} \tag{C.4}$$

Assuming elastic behaviour of timber ($\sigma_t = \epsilon_t E_t$), the resulting compressive force is calculated as follows:

$$\epsilon_t = \frac{x_c \epsilon_{s1}}{h_1 - x_c} \tag{C.5}$$

$$F_t = \frac{1}{2} b x_c \sigma_t = \frac{b x_c^2 f_y}{2(h_1 - x_c)} \frac{E_t}{E_s} \tag{C.6}$$

The moment capacity is determined from the equilibrium between tensile forces in rods and the compressive force in timber:

$$M_{Rd} = F_t \frac{2x_c}{3} + \sum_{i=1}^{N_t} F_{si}(h_i - x_c) \tag{C.7}$$

Table C.10: Glued in rods connection - Moment capacity calculation

Description	Symbol	Formula	Value	Unit
Steel rods in tension				
Position of first yielding rods relative to compressive edge	h_1	$h_i = a_4 + (N-1)a_2$	928	mm
Index of last row in tension	N_t	$N_t < (a_4 - x_c)/a_2 + N_{rows}$	7	
Sum of tensile forces	$\sum_{i=1}^{N_t} F_{si}$		7.27E+05	N
	$\sum_{i=1}^{N_t} F_{si} h_i$		6.09E+08	Nmm
Timber in compression				
Design yield strength rods	f_{yd}	f_{yk}/γ_{M2}	240	N/mm^2
Compressive force in timber	F_t	$\frac{1}{2} b x_c^2 f_{yd}/(h_1 - x_c) E_t/E_s$	7.28E+05	N
Neutral axis				
Position of the neutral axis	x_c	From iterative calculation	285	mm
Moment capacity				
Moment capacity	M_{Rd}	$\frac{2}{3} F_t x_c + \Sigma F_{si} h_i - x_c \Sigma F_{si}$	5.39E+08	Nmm
Design bending moment	M_d	From SCIA	4.28E+08	Nmm
Unity check moment capacity	UC_m	M_d/M_{Rd}	0.79	

Axial resistance

The axial load-carrying capacity of a connection with rods parallel to the grain is checked according to the draft version of the upcoming timber Eurocode. The calculation procedure is detailed in Table C.11. Assuming uniform load-distribution in a group of simultaneously acting glued-in rods, the selected rods should satisfy the requirement ensuring that ductile failure in tension (failure mode 1) occurs prior to the brittle failure of the bond-line, or any other brittle failure modes in the timber:

$$\frac{F_{b,d}}{A_s f_{y,k}/\gamma_{M0}} = 1.51 \geq 1.5 \tag{C.8}$$

Table C.11: Glued in rods connection - Axial resistance

Description	Symbol	Formula	Value	Unit
FAILURE MODE 1 – Tension failure of the rod (11.12.5.1)				
Effective anchorage length	$l_{b,ef}$	$min(l_b; 40d; 1000mm)$	650	mm
Nominal stress area for threaded rods (EN ISO 898-1)	A_s		192	mm^2
Design value of threaded rods tensile resistance	$F_{t,d}$	$min(A_s f_{yk}/\gamma_{M0}; 0.9 A_s f_{uk}/\gamma_{M2})$	4.43E+04	N
FAILURE MODE 3 – Failure of the adhesive in the bondline and its bond to rod and timber (11.12.5.1) **FAILURE MODE 4 – Shear failure of the timber adjacent to the bondline (11.12.5.1)**				
Characteristic shear resistance of adhesive in bondline (FM3)	$F_{b,k3}$	$\pi d l_{b,ef} f_{vr,k}$	9.28E+04	N
Design shear resistance of adhesive in bondline (FM3)	$F_{b,d3}$	$k_{mod} F_{b,k3}/\gamma_M$	6.68E+04	N
Characteristic shear resistance of timber adjacent to bondline (FM4)	$F_{b,k4}$	$E_s A_s \epsilon_{u,timber}$	9.68E+04	N
Design shear resistance of timber adjacent to bondline (FM4)	$F_{b,d4}$	$k_{mod} F_{b,k4}/\gamma_M$	6.97E+04	N
Characteristic bondline resistance	$F_{b,k}$	$min(F_{b,k3}; F_{b,k4})$	9.28E+04	N
Design bondline resistance	$F_{b,d}$	$k_{mod} F_{b,k}/\gamma_M$	6.68E+04	N
Axial resistance				
Design resistance of bonded-in rod	$F_{ax,Rd}$	$\min(F_{t,d}; F_{b,d})$	4.43+04	N
Design tensile force	$N_t d$	Yielding of rods at moment capacity	4.43E+04	N
Unity check axial loading	$UC_{fm1,3,4}$	$N_{td}/F_{ax,Rd}$	1.00	

Lateral resistance

The calculation procedure for checking the lateral load-carrying capacity of the connection is detailed in Table C.12.

Table C.12: Glued in rods connection - Lateral resistance

Description	Symbol	Formula	Value	Unit
FAILURE MODE 5 – Splitting of the timber departing from the bonded-in rods (11.12.7)				
Characteristic embedment strength glulam (screws, rods with wood screw thread)	$f_{h,k}$	$(0.019\rho_k^{1.24}d^{-0.3})/(2.5cos(\epsilon)^2+sin(\epsilon)^2)$	5.80	N/mm^2
Modified embedment strength	$f_{h,k}$	12.5% x 10% x $f_{h,k}$	0.73	N/mm^2
Characteristic yield moment of the rod	$M_{y,k}$	$0.3f_{u,k}d^{2.6}$	2.75E+05	Nmm
Dowel-effect contribution - failure mode (a1)	a_1	$df_{h,k}[\sqrt{(l_b+2e)^2+l_b^2}-l_b-2e]$	2.98E+03	N
Dowel-effect contribution - failure mode (b1)	b_1	$df_{h,k}[\sqrt{e^2+\frac{2M_{y,k}}{df_{h,k}}}-e]$	1.83E+03	N
Characteristic value of the dowel-effect contribution	$F_{D,k}$	$min(a_1;b_1)$	1.83E+03	N
Characteristic head pull-through resistance	F_{pk}	not relevant	/	N
	k_ρ	$1.25-0.05d$	0.35	
	f_{wk}	$8.2k_w(=1)k_{mat}(=1)d^{-0.33}(\frac{\rho_k}{350})^{k_\rho}$	3.38	N/mm^2
Characteristic withdrawal resistance of a fastener	F_{wk}	$\pi dl_w(=l_b)f_{wk}$	1.24E+05	N
Characteristic tensile resistance	F_{tk}	$\frac{0.9\pi d^2}{4}f_{uk}$	1.15E+05	N
	$F_{ax,tk}$	$min(F_{pk};F_{wk};F_{tk})$	1.15E+05	N
Factor for rope effect	k_{rp1}	General	0.25	
Limitation factor for rope effect	k_{rp2}	Laterally bonded-in rods	1.00	
Characteristic rope effect contribution per shear plane per fastener	$F_{rp,k}$	$min(k_{rp1}F_{ax,tk};k_{rp2}F_{Dk})$	1.83E+03	N
	k_{WD}	Non wooden dowels	1.00	
Characteristic lateral resistance per shear plane for a single fastener	$F_{v,k}$	$k_{WD}F_{D,k}+F_{rp,k}$	3.67E+03	N
Design lateral resistance per shear plane for a single fastener	$F_{v,d}$	$F_{v,k}/\gamma_M(=1.3)$	2.82E+03	N
Design lateral resistance for the connection	$F_{v,Rd}$	$n_{ef}(=N_{tot}\ for\ a\ column)F_{vd}$	1.02E+05	N
Design shear force	V_d		7.63E+04	N
Unity check lateral loading FM5	UC_{fm5}	$V_d/F_{v,Rd}$	0.75	

C.5.2 Foundation pads

In both steel and concrete reference designs, foundations pads are made from in situ concrete of strength class C30/37 with a reinforcing steel percentage of 30% (volume). Pad footings are located under the main columns, as well as facade columns. They should be able to withstand the normal forces and moments applied at the column base and transfer these loads from the superstructure to the soil.

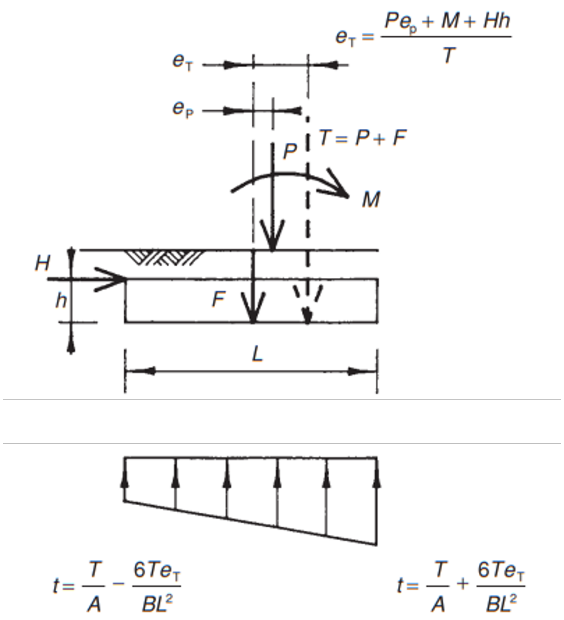


Figure C.12: Capacity check for foundation pads (Curtin et al., 2006)

All foundation pads are designed with square sections based on the maximum design moment in both directions of the warehouse. The required area of the pads depends on the design loads and the soil bearing capacity. The allowable soil pressure increases linearly below ground surface, from 150kPa at surface level to 600kPa at 1.5m depth. The timber warehouse is designed with fixed base columns, like the concrete reference; in this case soil stiffness is a governing criterion to ensure sufficient rotational stiffness to the column base. The bedding stiffness of the soil is assumed equal to 20 MN/m^3.

The procedure for checking the capacity of pad footings under all columns of the timber warehouse is described in Table C.13, based on the loading situation illustrated in Figure C.12.

Table C.13: Foundation pads timber baseline A

Description	Symbol	Formula	Value	Unit
Design loads (ULS, from SCIA)				
Design bending moment	M		3.85E+08	Nmm
Design axial force	P		9.03E+05	N
Design shear force	H		7.63E+04	N
Foundation pad characteristics				
Assumed length of square foundation pad	L		3000	mm
Assumed depth of square foundation pad	h		1000	mm
Area of foundation pad	A	L^2	9.0	m^2
Concrete density	$\rho_{concrete}$		2500	kg/m^3
Steel density	ρ_{steel}		7850	kg/m^3
Reinforcement in foundation pads	α_{rebar}		120	kg/m^3
Volume of concrete foundation pad	V_{pad}	L^2h	9.0	m^3
Mass of concrete in foundation pad	$M_{concrete1pad}$	$V_{pad}\rho_{concrete}$	22500	kg
Mass of reinforcement in foundation pad	$M_{steel1pad}$	$V_{pad}\alpha_{rebar}$	1080	kg
Total mass of foundation pad	$M_{tot1pad}$	$M_{concrete1pad} + M_{steel1pad}$	23580	kg
Bearing pressure checks				
Gravity constant	g		9.81	m/s^2
Vertical load from pad weight	F	$gM_{tot1pad}$	2.31E+05	N
Total vertical load below the pad	T	$F + P$	1.13E+06	N
Eccentricity of total load	e_T	$(Pe_p + M + Hh)/T$	406	mm
	$e_T/(L/6)$	Full pad in compression if < 1	0.81	2
Maximum bearing pressure below the pad	t_{max}	$T/A + Te_t/(BL^2/6)$ if $e_T/(L/6) < 1$	2.28E-01	N/mm^2
Minimum bearing pressure below the pad	t_{min}	$T/A - Te_t/(BL^2/6)$ if $e_T/(L/6) < 1$	2.37E-02	N/mm^2
Maximum allowable bearing pressure at depth d	p_{max}	Linear interpolation based on assumed d and soil bearing capacities	4.50E-01	N/mm^2
Unity check	UC_pad	t_{max}/p_{max}	0.51	

C.5.3 Rotational stiffness calculation

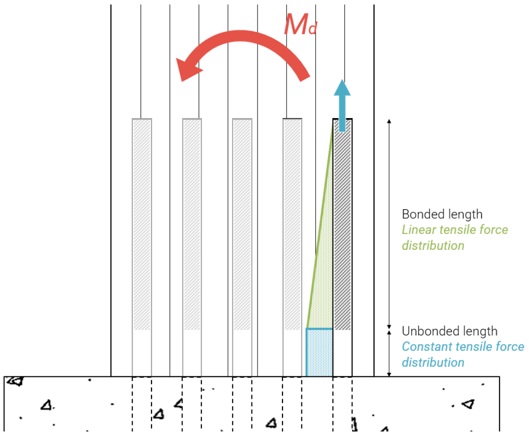


Figure C.13: Glued-in rods connection rotational stiffness calculation model

The rotational stiffness of the base connection can be calculated based on the elongation of the first row of glued-in rods in tension when maximum moment capacity is reached. At the point of yielding, elongation of the rods is determined from the stress-strain diagram of steel, assuming a linear elastic behaviour. The corresponding rotation θ of the base connection is:

$$\Delta l = \epsilon l = \frac{f_{yd}}{E_s}(l_{nb} + \frac{1}{2}l_b) \tag{C.9}$$

$$tan\theta = \theta = \frac{\Delta l}{X} = \frac{f_{yd}(l_{nb} + \frac{1}{2}l_b)}{E_s(h_1 - x_c)} \tag{C.10}$$

$$k_{r,connection} = \frac{M_{foundation}}{\theta} = \frac{M(h_1 - x_c)E_s}{f_{yd}(l_{nb} + \frac{1}{2}l_b)} = 5.95E + 11Nmm/rad \tag{C.11}$$

Table C.14: Rotational stiffness base connection

Description	Symbol	Formula	Value	Unit
Rotational stiffness connection				
Design bending moment	M_d		4.28E+08	N/mm^2
First yielding rods to compressive edge	h_1		928	mm
Position of the neutral axis	x_c		285	mm
Steel modulus of elasticity	E_s		210000	N/mm^2
Design yield strength rods	f_{yd}		240	N/mm^2
Unbonded length	l_{nb}		80	mm
Anchorage length	l_b		650	mm
Rotational stiffness of the connection	k_{r1}	$\frac{M_d(h_1-x_c)E_s}{f_{yd}(l_{nb}+\frac{l_b}{2})}$	5.95E+11	Nmm/rad

The foundation pad is subjected to N, V and M. The total load under the pad is T, associated with an eccentricity of e_T. The condition for full compression under the foundation pad is $e_T < L/6$. The Winkler elastic foundation model is used to study the interaction between the soil and the square footing of 3 by 3 metres, represented as a bar of length L=3m on a continuous spring support modelling the soil stiffness of 20MN/m^3. The rotation θ of the pad under differential settlement can be used to determine the rotational stiffness of the pad footing. The pressure p under the pad is related to soil settlements w using the modulus of subgrade reaction k: $p = wk$.

$$w_{max} = t_{max}/k \tag{C.12}$$

$$w_{min} = t_{min}/k \tag{C.13}$$

$$tan\theta = \theta = \frac{w_{max} - w_{min}}{L} = \frac{t_{max} - t_{min}}{kL} \tag{C.14}$$

$$k_{r,foundation} = \frac{M_{foundation}}{\theta} = \frac{Te_T kL}{t_{max} - t_{min}} = 1.35E + 11Nmm/rad \tag{C.15}$$

Table C.15: Rotational stiffness foundation pads

Description	Symbol	Formula	Value	Unit
Rotational stiffness foundation				
Assumed length of square foundation pad	L		3000	mm
Assumed depth of square foundation pad	h		1000	mm
Total vertical load below the pad	T		1.13E+06	N
Eccentricity of total load	e_T		406	mm
Maximum bearing pressure below the pad	p_{min}		2.28E-01	N/mm^2
Minimum bearing pressure below the pad	p_{max}		2.37E-02	N/mm^2
Modulus of subgrade reaction	k		2.00E+04	N/mm^3
Rotational stiffness of foundation pad on soil	k_{r2}	$\frac{Te_T Lk}{t_{max}-t_{min}}$	1.35E+11	Nmm/rad

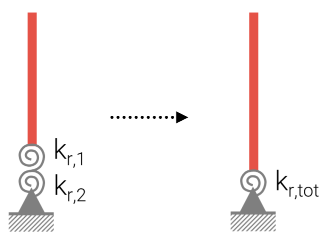


Figure C.14: Total rotational stiffness of the base connection

The total stiffness of the column base is calculated by combining the stiffness of the glued-in rods connection and the stiffness of the footing, as illustrated on Figure C.14:

$$\frac{1}{k_{r,tot}} = \frac{1}{k_{r,1}} + \frac{1}{k_{r,2}} = 1.10E + 11Nmm/rad \tag{C.16}$$

C.6 Timber sections and stability checks

The results of structural calculations for timber members are given in this section. The dimensions of beams and columns are verified against governing load combinations at ULS and SLS. For timber baseline warehouse A, detailed calculations are presented in Section C.6.3 for each type of timber element (beams (x) and (y), main columns, facade columns). For all other timber frame designs, only governing checks at ULS and SLS will be presented, after carrying out the same procedure as in this example.

C.6.1 ULS results

The results of section and stability checks at ULS are given in Table C.16. The governing load combinations in the design of the different members are H003, H004 and C004.

Table C.16: Governing unity checks at ULS for structural members of timber baseline A

Element	Description	$UC_{section}$	$UC_{stability}$
Roof beams (x)	280x1800mm	0.81	0.64
Roof beams (y)	220x1800mm	0.84	0.63
Main columns	1000x1000mm	0.13	0.35
Facade columns	400x400mm	0.48	0.77

The buckling length of main columns in the main direction is calculated in SCIA, considering the rotational stiffness of the base connection previously determined and taken as an input in the model. As expected in a sway frame, for columns pinned at the top and with a semi-rigid connection at the base, the buckling factor is larger than 1. In the out-of-plane direction, the buckling length is manually entered in the model based on the 2D model in the other direction, as it cannot be done automatically by SCIA. Facade columns are pinned at both ends, their buckling is taken equal to the length of the member. Roof beams act as non-sway, therefore their length is also equal to their total length.

C.6.2 SLS results

The results of SLS checks are given in Table C.17. Load combination H103 is governing the vertical deflection of roof beams, H104 governs the horizontal deflection of columns.

Table C.17: Governing unity checks at SLS for structural members of timber baseline A

Element	Description	Max calc.	Max allowable	UC
Vertical deformation - h/175				
Roof beams (x)	280x1800mm	75.3mm	96.0mm	0.78
Roof beams (y)	220x1800mm	31.6mm	62.9mm	0.50
Horizontal deformation - L/250				
Main columns	1000x1000mm	46.8mm	58.9mm	0.80
Facade columns	400x400mm	42.4mm	58.9mm	0.72

C.6.3 Detailed calculations

C.6.3.1 ULS calculation of a governing roof beam in the x-direction

Table C.18: Roof beams (x)

Description	Symbol	Formula	Value	Unit
Beam geometric properties				
Length	L		24000	mm
Buckling factor about y-axis	β_y		1	
Buckling factor about z-axis	β_z		0.13	
Effective length about the y-axis	L_{ey}	$\beta_y L$	24000	mm
Effective length about the z-axis	L_{ez}	$\beta_z L$	3000	mm
Cross-section width	b		280	mm
Cross-section depth	h		1800	mm
Cross-section area	A	bh	5.04E+05	mm^2
Second moment of area about the y-axis	I_y	$bh^3/12$	1.36E+11	mm^4
Section modulus about the y-axis	W_y	$bh^2/6$	1.51E+08	mm^3
Radius of gyration about the y-axis	i_y	$\sqrt{I_y/A}$	519.62	mm
Slenderness ratio about the y-axis	λ_y	L_{ey}/i_y	46.2	
Second moment of area about the z-axis	I_z	$hb^3/12$	3.29E+09	mm^4
Section modulus about the z-axis	W_z	$hb^2/6$	2.35E+07	mm^3
Radius of gyration about the z-axis	i_z	$\sqrt{I_z/A}$	80.83	mm
Slenderness ratio about the z-axis	λ_z	L_{ez}/i_z	37.1	
Bearing length at each end	l_c	$b_{column}/2$	500	mm
System strength, load sharing factor	k_{sys}		1.00	
Size factor for glulam (reference dimension <600mm)	k_h	$min\left(600/h\right)^{0.1}; 1.1)$	1.00	
Modification factor for the influence of cracks	k_{cr}		0.67	
Design loads (ULS, from SCIA)				
Design moment about the y-axis	M_{yd}		1.93E+09	Nmm
Design moment about the z-axis	M_{zd}		0	Nmm
Design compressive force	F_{c0d}		-4.18E+04	N
Design tensile force	F_{t0d}		1.98E+04	N
Design shear force	V_d		3.22E+05	N
Design bearing force	F_{c90d}		3.22E+05	N

Axial tension				
Design tensile stress parallel to the grain	σ_{t0d}	F_{t0d}/A	0.04	N/mm^2
Design tensile strength parallel to the grain	f_{t0d}	$k_{mod}k_{sys}f_{t0k}/\gamma_M$	16.06	N/mm^2
Unity check axial tension	$UC_{tension}$	σ_{t0d}/f_{t0d}	**0.00**	
Axial compression				
Design compressive stress parallel to the grain	σ_{c0d}	F_{c0d}/A	0.08	N/mm^2
Design compressive strength parallel to the grain	f_{c0d}	$k_{mod}k_{sys}f_{c0k}/\gamma_M$	20.16	N/mm^2
Unity check axial compression	UC_{comp}	σ_{c0d}/f_{c0d}	**0.00**	
Compression perpendicular to the grain				
Effective depth	h_{ef}	$min(140; 0.4h)$	140	mm
Effective spreading length of compressive stress	l_{ef}	$l_c + h_{ef}$	640	mm
Compression factor for strain effects	k_{c90}	$\sqrt{l_{ef}/l_c}$	1.13	
Effective contact area	A_{ef}	bl_c	140000	mm^2
Design bearing stress	σ_{c90d}	F_{c90d}/A_{ef}	2.30	N/mm^2
Design bearing strength	f_{c90d}	$k_{mod}k_{sys}f_{c90k}/\gamma_M$	1.80	N/mm^2
Factor for deformations	k_p		1.40	
Factored design bearing strength	$k_pk_{c90}f_{c90d}$		2.85	N/mm^2
Unity check compression perpendicular to the grain	UC_{comp90}	$\sigma_{c90d}/k_pk_{c90}f_{c90d}$	**0.81**	
Bending				
Design bending stress y-axis	σ_{myd}	M_{yd}/W_y	12.79	N/mm^2
Design bending strength y-axis	f_{myd}	$k_{mod}k_{sys}k_{hy}f_{mk}/\gamma_M$	20.16	N/mm^2
Unity check bending (M_{yd} only)	$UC_{bending}$	σ_{myd}/f_{myd}	**0.63**	
Shear				
Effective shear width	b_{ef}	$k_{cr}b$	188	mm
Effective shear height	h		1800	mm
Design shear stress	τ_{vd}	$1.5V_d/(b_{ef}h)$	1.43	N/mm^2
Design shear strength	f_{vd}	$k_{mod}k_{sys}f_{vk}/\gamma_M$	2.52	N/mm^2
Unity check shear	UC_{shear}	$\tau_{vd}/(k_vf_{vd})$	**0.57**	
Combined stresses				
Combined bending and tension	$UC_{bend+tens}$	$\sigma_{t0d}/f_{t0d} + \sigma_{myd}/f_{myd}$	**0.64**	
Combined bending and compression	$UC_{bend+comp}$	$(\sigma_{c0d}/f_{c0d})^2 + \sigma_{myd}/f_{myd}$	**0.63**	

Flexural buckling under axial compression and major axis bending				
Relative slenderness about the y-axis	$\lambda_{rel,y}$	$\lambda_y/\pi\sqrt{f_{c0k}/E_{0.05}}$	0.76	>0.3
Relative slenderness about the z-axis	$\lambda_{rel,z}$	$\lambda_z/\pi\sqrt{f_{c0k}/E_{0.05}}$	0.61	>0.3
Factor for glulam	β_c		0.1	
Factor y-axis	k_y	$0.5(1+\beta c(\lambda_{rel,y}-0.3)+\lambda_{rel,y}^2)$	0.81	
Instability factor y-axis	k_{cy}	$1/(k_y+\sqrt{k_y^2-\lambda_{rel,y}^2})$	0.91	
Factor z-axis	k_z	$0.5(1+\beta c(\lambda_{rel,z}-0.3)+\lambda_{rel,z}^2)$	0.70	
Instability factor z-axis	k_{cz}	$1/(k_z+\sqrt{k_z^2-\lambda_{rel,z}^2})$	0.95	
Redistribution factor for a rectangular section	k_m		0.7	
Unity check flexural buckling condition about y-axis	UC_{flex1}	$\sigma_{c0d}/(k_{cy}f_{c0d})+\sigma_{myd}/f_{myd}+k_m\sigma_{mzd}/f_{mzd}$	0.64	
Unity check flexural buckling condition about z-axis	UC_{flex2}	$\sigma_{c0d}/(k_{cz}f_{c0d})+k_m\sigma_{myd}/f_{myd}+\sigma_{mzd}/f_{mzd}$	0.64	
Lateral torsional buckling under major axis bending				
Nb of lateral supports along the length (incl supports)	N_{LTB}		9	
Member length between lateral supports	l_{LTB}	$L/(N_{LTB}-1)$	3000	mm
Effective length	l_{ef}	$0.9l_{LTB}$	2700	mm
Critical bending stress	$\sigma_{m,crit}$	$\frac{\pi b^2}{hl_{ef}}\sqrt{E_{0.05}G_{0.05}\,(1-0.63b/h)}$	114.55	N/mm^2
Relative slenderness for bending	$\lambda_{rel,m}$	$\sqrt{f_{mk}/\sigma_{m,crit}}$	0.49	
Lateral stability of the beam	k_{crit}	1 if lateral displacement along compressive edge and torsional rotation at supports are prevented	1.00	
Unity check lateral torsional buckling	UC_{LTB}	$\sigma_{myd}/(k_{crit}f_{myd})$	0.63	

C.6.3.2 ULS calculation of a governing roof beam in the y-direction

Table C.19: Roof beams (y)

Description	Symbol	Formula	Value	Unit
Beam geometric properties				
Length	L		15720	mm
Buckling factor about y-axis	β_y		1	
Buckling factor about z-axis	β_z		0.25	
Effective length about the y-axis	L_{ey}	$\beta_y L$	15720	mm
Effective length about the z-axis	L_{ez}	$\beta_z L$	3930	mm

Cross-section width	b		220	mm
Cross-section depth	h		1800	mm
Cross-section area	A	bh	3.96+05	mm^2
Second moment of area about the y-axis	I_y	$bh^3/12$	1.07E+11	mm^4
Section modulus about the y-axis	W_y	$bh^2/6$	1.19E+08	mm^3
Radius of gyration about the y-axis	i_y	$\sqrt{I_y/A}$	519.62	mm
Slenderness ratio about the y-axis	λ_y	L_{ey}/i_y	30.3	
Second moment of area about the z-axis	I_z	$hb^3/12$	1.60E+09	mm^4
Section modulus about the z-axis	W_z	$hb^2/6$	1.45E+07	mm^3
Radius of gyration about the z-axis	i_z	$\sqrt{I_z/A}$	63.5	mm
Slenderness ratio about the z-axis	λ_z	L_{ez}/i_z	61.9	
Bearing length at each end	l_c	$b_{column}/2$	360	mm
Size factor for glulam (reference dimension <600mm)	k_h	$min\left(600/h)^{0.1}; 1.1\right)$	1.00	
Design loads (ULS, from SCIA)				
Design moment about the y-axis	M_{yd}		1.50E+09	Nmm
Design moment about the z-axis	M_{zd}		0	Nmm
Design compressive force	F_{c0d}		-4.66E+04	N
Design tensile force	F_{t0d}		2.20E+04	N
Design shear force	V_d		1.99E+05	N
Design bearing force	F_{c90d}		1.99E+05	N
Axial tension				
Design tensile stress parallel to the grain	σ_{t0d}	F_{t0d}/A	0.06	N/mm^2
Unity check axial tension	$UC_{tension}$	σ_{t0d}/f_{t0d}	0.00	
Axial compression				
Design compressive stress parallel to the grain	σ_{c0d}	F_{c0d}/A	0.12	N/mm^2
Unity check axial compression	UC_{comp}	σ_{c0d}/f_{c0d}	0.01	
Compression perpendicular to the grain				
Effective depth	h_{ef}	$min(140; 0.4h)$	140	mm
Effective spreading length of compressive stress	l_{ef}	$l_c + h_{ef}$	500	mm
Compression factor for strain effects	k_{c90}	$\sqrt{l_{ef}/l_c}$	1.18	
Effective contact area	A_{ef}	bl_c	79200	mm^2
Design bearing stress	σ_{c90d}	F_{c90d}/A_{ef}	2.51	N/mm^2
Unity check compression perpendicular to the grain	UC_{comp90}	$\sigma_{c90d}/k_p k_{c90} f_{c90d}$	0.84	

Bending				
Design bending stress y-axis	σ_{myd}	M_{yd}/W_y	12.61	N/mm^2
Unity check bending (M_{yd} only)	$UC_{bending}$	σ_{myd}/f_{myd}	0.63	
Shear				
Effective shear width	b_{ef}	$k_{cr}b$	147	mm
Effective shear height	h		1800	mm
Design shear stress	τ_{vd}	$1.5V_d/(b_{ef}h)$	1.12	N/mm^2
Unity check shear	UC_{shear}	$\tau_{vd}/(k_v f_{vd})$	0.45	
Combined stresses				
Combined bending and tension	$UC_{bend+tens}$	$\sigma_{t0d}/f_{t0d} + \sigma_{myd}/f_{myd}$	0.63	
Combined bending and compression	$UC_{bend+comp}$	$(\sigma_{c0d}/f_{c0d})^2 + \sigma_{myd}/f_{myd}$	0.63	
Flexural buckling under axial compression and major axis bending				
Relative slenderness about the y-axis	$\lambda_{rel,y}$	$\lambda_y/\pi\sqrt{f_{c0k}/E_{0.05}}$	0.50	>0.3
Relative slenderness about the z-axis	$\lambda_{rel,z}$	$\lambda_z/\pi\sqrt{f_{c0k}/E_{0.05}}$	1.02	>0.3
Factor y-axis	k_y	$0.5(1+\beta c(\lambda_{rel,y}-0.3)+\lambda_{rel,y}^2)$	0.63	
Instability factor y-axis	k_{cy}	$1/(k_y+\sqrt{k_y^2-\lambda_{rel,y}^2})$	0.97	
Factor z-axis	k_z	$0.5(1+\beta c(\lambda_{rel,z}-0.3)+\lambda_{rel,z}^2)$	1.05	
Instability factor z-axis	k_{cz}	$1/(k_z+\sqrt{k_z^2-\lambda_{rel,z}^2})$	0.75	
Unity check flexural buckling condition about y-axis	UC_{flex1}	$\sigma_{c0d}/(k_{cy}f_{c0d})+\sigma_{myd}/f_{myd}+k_m\sigma_{mzd}/f_{mzd}$	0.63	
Unity check flexural buckling condition about z-axis	UC_{flex2}	$\sigma_{c0d}/(k_{cz}f_{c0d})+k_m\sigma_{myd}/f_{myd}+\sigma_{mzd}/f_{mzd}$	0.45	
Lateral torsional buckling under major axis bending				
Nb of lateral supports along the length (incl supports)	N_{LTB}		5	
Member length between lateral supports	l_{LTB}	$L/(N_{LTB}-1)$	3930	mm
Effective length	l_{ef}	$0.9l_{LTB}$	3537	mm
Critical bending stress	$\sigma_{m,crit}$	$\frac{\pi b^2}{hl_{ef}}\sqrt{E_{0.05}G_{0.05}\,(1-0.63b/h)}$	54.61	N/mm^2
Relative slenderness for bending	$\lambda_{rel,m}$	$\sqrt{f_{mk}/\sigma_{m,crit}}$	0.72	
Lateral stability of the beam	k_{crit}		1.00	
Unity check lateral torsional buckling	UC_{LTB}	$\sigma_{myd}/(k_{crit}f_{myd})$	0.63	

C.6.3.3 ULS calculation of a governing column

Table C.20: Main columns

Description	Symbol	Formula	Value	Unit
Beam geometric properties				
Length	L		10300	mm
Buckling factor about y-axis	β_y	Calculated in SCIA	3.6	
Effective length about the y-axis	L_{ey}	$\beta_y L$	37080	mm
Cross-section width and depth	$b = h$		1000	mm
Cross-section area	A	bh	1.00+06	mm^2
Second moment of area about the y-axis	I_y	$bh^3/12$	8.33E+10	mm^4
Section modulus about the y-axis	W_y	$bh^2/6$	1.67E+08	mm^3
Radius of gyration about the y-axis	i_y	$\sqrt{I_y/A}$	288.68	mm
Slenderness ratio about the y-axis	λ_y	L_{ey}/i_y	128.45	
Size factor for glulam (reference dimension <600mm)	k_{hy}	$min((\frac{600}{h})^{0.1}; 1.1)$	1.00	
Design loads (ULS, from SCIA)				
Design moment about the y-axis	M_{yd}		4.28E+08	Nmm
Design moment about the z-axis	M_{zd}		0	Nmm
Design compressive force	F_{c0d}		9.03E+05	N
Design tensile force	F_{t0d}		0	N
Design shear force	V_d		7.63E+04	N
Design bearing force	F_{c90d}		0	N
Axial tension				
Design tensile stress parallel to the grain	σ_{t0d}	F_{t0d}/A	0.00	N/mm^2
Unity check axial tension	$UC_{tension}$	σ_{t0d}/f_{t0d}	0.00	
Axial compression				
Design compressive stress parallel to the grain	σ_{c0d}	F_{c0d}/A	0.90	N/mm^2
Unity check axial compression	UC_{comp}	σ_{c0d}/f_{c0d}	0.04	
Bending				
Design bending stress y-axis	σ_{myd}	M_{yd}/W_y	2.57	N/mm^2
Unity check bending condition about y-axis	$UC_{bending1}$	$\sigma_{myd}/f_{myd} + k_m \sigma_{mzd}/f_{mzd}$	0.13	

Shear				
Effective shear width	b_{ef}	$k_{cr}b$	670	mm
Effective shear height	h		1000	mm
Design shear stress	τ_{vd}	$1.5V_d/(b_{ef}h)$	0.17	N/mm^2
Unity check shear	UC_{shear}	$\tau_{vd}/(k_v f_{vd})$	0.07	
Combined stresses				
Combined bending and tension about y-axis	$UC_{bend+tens}$	$\sigma_{t0d}/f_{t0d} + \sigma_{myd}/f_{myd} + k_m\sigma_{mzd}/f_{mzd}$	0.13	
Combined bending and compression about y-axis	$UC_{bend+comp}$	$(\sigma_{c0d}/f_{c0d})^2 + \sigma_{myd}/f_{myd} + k_m\sigma_{mzd}/f_{mzd}$	0.13	
Flexural buckling under axial compression and major axis bending				
Relative slenderness about the y-axis	$\lambda_{rel,y}$	$\lambda_y/\pi\sqrt{f_{c0k}/E_{0.05}}$	2.11	>0.3
Factor y-axis	k_y	$0.5(1+\beta c(\lambda_{rel,y}-0.3)+\lambda_{rel,y}^2)$	2.82	
Instability factor y-axis	k_{cy}	$1/(k_y+\sqrt{k_y^2-\lambda_{rel,y}^2})$	0.21	
Unity check flexural buckling condition about y-axis	UC_{flex1}	$\sigma_{c0d}/(k_{cy}f_{c0d}) + \sigma_{myd}/f_{myd} + k_m\sigma_{mzd}/f_{mzd}$	0.34	

C.6.3.4 ULS calculation of a facade column

Table C.21: Facade columns

Description	Symbol	Formula	Value	Unit
Beam geometric properties				
Length	L		10300	mm
Buckling factor about y-axis	β_y		1	
Effective length about the y-axis	L_{ey}	$\beta_y L$	10300	mm
Cross-section width and depth	$b = h$		400	mm
Cross-section area	A	bh	1.60+05	mm^2
Second moment of area about the y-axis	I_y	$bh^3/12$	2.13E+09	mm^4
Section modulus about the y-axis	W_y	$bh^2/6$	1.07E+07	mm^3
Radius of gyration about the y-axis	i_y	$\sqrt{I_y/A}$	115.47	mm
Slenderness ratio about the y-axis	λ_y	L_{ey}/i_y	89.2	
Size factor for glulam (reference dimension <600mm)	k_{hy}	$min((\frac{600}{h})^{0.1};1.1)$	1.00	

Design loads (ULS, from SCIA)				
Design moment about the y-axis	M_{yd}		1.00E+08	Nmm
Design moment about the z-axis	M_{zd}		0	Nmm
Design compressive force	F_{c0d}		3.85E+05	N
Design tensile force	F_{t0d}		0	N
Design shear force	V_d		3.89E+04	N
Design bearing force	F_{c90d}		0	N
Axial tension				
Design tensile stress parallel to the grain	σ_{t0d}	F_{t0d}/A	0.00	N/mm^2
Unity check axial tension	$UC_{tension}$	σ_{t0d}/f_{t0d}	0.00	
Axial compression				
Design compressive stress parallel to the grain	σ_{c0d}	F_{c0d}/A	2.41	N/mm^2
Unity check axial compression	UC_{comp}	σ_{c0d}/f_{c0d}	0.12	
Bending				
Design bending stress y-axis	σ_{myd}	M_{yd}/W_y	9.40	N/mm^2
Unity check bending condition about y-axis	$UC_{bending1}$	$\sigma_{myd}/f_{myd} + k_m\sigma_{mzd}/f_{mzd}$	0.47	
Shear				
Effective shear width	b_{ef}	$k_{cr}b$	268	mm
Effective shear height	h		400	mm
Design shear stress	τ_{vd}	$1.5V_d/(b_{ef}h)$	0.54	N/mm^2
Unity check shear	UC_{shear}	$\tau_{vd}/(k_v f_{vd})$	0.22	
Combined stresses - bending and tension				
Combined bending and tension about y-axis	$UC_{bend+tens}$	$\sigma_{t0d}/f_{t0d} + \sigma_{myd}/f_{myd} + k_m\sigma_{mzd}/f_{mzd}$	0.47	
Combined bending and compression about y-axis	$UC_{bend+comp}$	$(\sigma_{c0d}/f_{c0d})^2 + \sigma_{myd}/f_{myd} + k_m\sigma_{mzd}/f_{mzd}$	0.48	
Flexural buckling under axial compression and major axis bending				
Relative slenderness about the y-axis	$\lambda_{rel,y}$	$\lambda_y/\pi\sqrt{f_{c0k}/E_{0.05}}$	1.47	>0.3
Factor y-axis	k_y	$0.5(1+\beta c(\lambda_{rel,y}-0.3)+\lambda_{rel,y}^2)$	1.69	
Instability factor y-axis	k_{cy}	$1/(k_y+\sqrt{k_y^2-\lambda_{rel,y}^2})$	0.39	
Unity check flexural buckling condition about y-axis	UC_{flex1}	$\sigma_{c0d}/(k_{cy}f_{c0d}) + \sigma_{myd}/f_{myd} + k_m\sigma_{mzd}/f_{mzd}$	0.77	

C.7 Bill of materials - Timber baseline A

The bill of materials of the timber baseline warehouse A is summarised in Table C.22, excluding envelope elements. The baseline envelope design is detailed in Appendix E, along with the bill of materials for these elements in Table E.11.

Table C.22: Bill of materials for the timber baseline warehouse A (frame, floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	400x400mm GL28h			10.30	73.6	1.65	460	44	72.5	3.33E+04
Columns total									*865.6*	*3.98E+05*
Beams (y)	220x1800mm GL28h			15.72	182.2	6.23	460	70	435.8	2.00E+05
Beams (x,large)	280x1800mm GL28h			24.00	231.8	12.10	460	66	798.3	3.67E+05
Beams (x,small)	280x1800mm GL28h			23.78	231.8	11.99	460	60	719.1	3.31E+05
Beams total									*1953.2*	*8.98E+05*
Timber total									**2818.8**	**1.30E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.4	1.11E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

D

Structural design of timber baseline B

The approach followed to design the structure of timber baseline warehouse B is the same as the step-by-step process detailed extensively in Appendix C. This appendix offers a simplified overview of the design process of timber baseline B, and draws the final bill of materials.

D.1 SCIA model and loads

Once again, the 3D warehouse frame is modelled in SCIA as 2D sections in the x and y directions. The height of columns is increased to 14.3m compared to baseline A, while the structural grid remains unchanged (16x24m). The updated dimensions of beams and columns are indicated on Figure D.1, with the stiffness of connections.

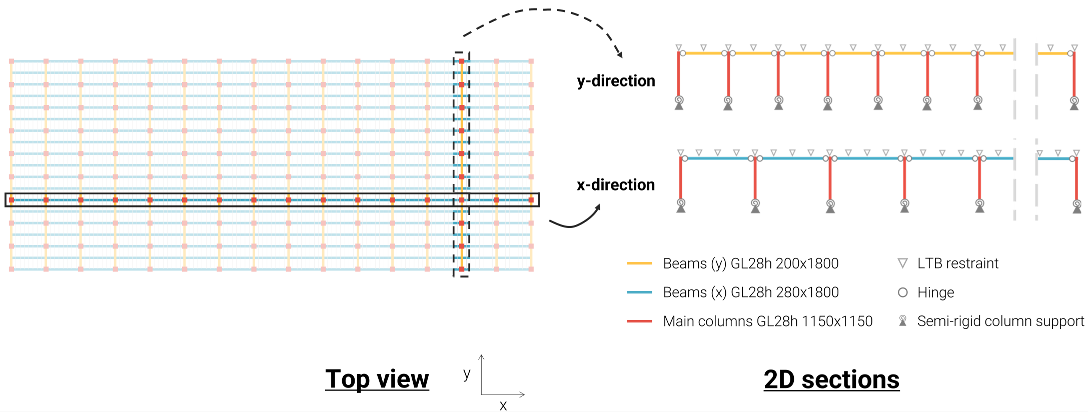


Figure D.1: Mechanics scheme of the stabilising frames in the x and y-directions for timber baseline B. Columns and beams are pinned in both directions, the base connection stiffness is 201 MNm/rad (see Section D.3).

The magnitude of wind loads increases with the building height. Load calculations are performed following the guidelines of Appendix B. The main difference in this design revolves around the increased height of columns which requires a larger section, to improve the base connection stiffness and resist higher lateral wind loads.

D.2 Second order effects

The evaluation of second order effects is carried out based on results from the SCIA model:

- δ_{top} = 60.10mm the maximal calculated deflection at the main columns top (the maximum allowable horizontal deflection is $\delta_{allowable}$ = h/175 = 81.71mm, therefore $UC_{SLS} = \delta_{top}/\delta_{allowable}$ = 0.74).
- N_{top} = 8.44E+05 N the compressive force acting at the column top
- $M_{1,base}$ = 6.17E+08 Nmm the first order moment calculated at the column base

Results of second order calculations are indicated in Table D.1: because n is higher than 10, the first order moment M_1 can be kept for design checks at the column base.

Table D.1: Calculation procedure for second order effects of the main columns in timber baseline A

Description	Symbol	Formula	Value	Unit
Second order moment at the base	$M_{2,1}$	$N_{top}\delta_{top}$	5.08E+07	Nmm
n factor	n	M_1/M_2	12.17	
Second order ratio	$n/(n-1)$		1.09	
Design moment from second order effects	$M_{2,i}$	$\frac{n}{n-1}M_1$	6.73E+08	Nmm

D.3 Column base connection

D.3.1 Glued-in rods connection

Steel rods in the column base detail of timber baseline warehouse B have the same characteristics as in timber baseline A, listed in Table C.8. Only the number of rods within the section is increased to resist larger design moments, resulting in larger columns. A summary of results from the connection design process is presented in Table D.2.

Table D.2: Glued in rods connections - Summary

Description	Symbol	Formula	Value	Unit
Steel rod characteristics – Spacing, edge and end distances within the timber section				
Column width	b		1150	mm
Max. rods along width	N		12	
Distance rods to edge	a_4		72	mm
Spacing between rods	a_2		91	mm
Total nb of rods in the section	N_{tot}		44	
Moment capacity				
Moment capacity	M_{Rd}		5.39E+08	Nmm
Design bending moment	M_d		4.28E+08	Nmm
Unity check moment capacity	UC_m	M_d/M_{Rd}	0.79	

Ductile failure requirement				
Ductile failure of steel rods		$F_{b,d}/A_s f_{y,k}/\gamma_{M0} \geq 1.5$	1.51	
Axial resistance				
Design axial resistance	$F_{ax,Rd}$		4.43+04	N
Design tensile force	N_td	Yielding at moment capacity	4.43E+04	N
Unity check axial loading	$UC_{fm1,3,4}$	$N_{td}/F_{ax,Rd}$	1.00	
Lateral resistance				
Design lateral resistance	$F_{v,d}$	Per shear plane, single fastener	2.82E+03	N
Design lateral resistance connection	$F_{v,Rd}$		1.02E+05	N
Design shear force	V_d		7.63E+04	N
Unity check lateral loading	UC_{fm5}	$V_d/F_{v,Rd}$	0.75	

D.3.1.1 Foundation pads

The dimensions of foundation pads of timber baseline B are taken as 4000x4000x1250mm. The capacity of the pads is checked with the same procedure as in Section C.5.2, using the relevant design loads and ensuring full compression under the pads. The final verification of maximum allowable bearing pressure at depth 1.25m, against the maximum bearing pressure below pads, results in a unity check of 0.45.

D.3.1.2 Rotational stiffness

The rotational stiffness $k_{r,1}$ of the base glued-in rods connection, and $k_{r,2}$ for foundation pads on soil are combined to get the total stiffness $k_{r,tot}$ of the column base:

$$k_{r,1} = \frac{M_d(h_1 - x_c)E_s}{f_{yd}(l_{nb} + \frac{l_b}{2})} = 1.01E + 12Nmm/rad \tag{D.1}$$

$$k_{r,2} = \frac{Te_T Lk}{t_{max} - t_{min}} = 2.50E + 11Nmm/rad \tag{D.2}$$

$$\frac{1}{k_{r,tot}} = \frac{1}{k_{r,1}} + \frac{1}{k_{r,2}} = 2.01E + 11Nmm/rad \tag{D.3}$$

D.4 Timber sections and stability checks

Table D.3: Governing unity checks at ULS and SLS for structural members of timber baseline B

Element	Description	ULS checks		SLS checks		
		UC$_{\text{section}}$	UC$_{\text{stability}}$	δ_{max}	$\delta_{\text{allowable}}$	UC$_{\text{deflection}}$
Roof beams (x)	280x1800mm	0.72	0.65	76.2mm	96.0mm	0.79
Roof beams (y)	200x1800mm	0.79	0.71	34.8mm	62.9mm	0.55
Main columns	1150x1150mm	0.12	0.21	60.1mm	81.7mm	0.74
Facade columns	500x500mm	0.49	0.75	65.3mm	81.7mm	0.80

D.5 Bill of materials - Timber baseline B

Ultimately, the environmental impact of timber baseline B will be compared with the impact of the concrete reference warehouse. The bill of materials of the timber baseline warehouse B is summarised in Table D.4, excluding envelope elements. The baseline envelope design is detailed in Appendix E, along with the bill of materials for these elements in Table E.11.

Table D.4: Bill of materials for the timber baseline warehouse B (frame, floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Frame										
Main columns	1150x1150mm GL28h			14.30	608.5	18.91	460	160	3025.9	1.39E+06
Facade columns	500x500mm GL28h			14.30	115.0	3.58	460	78	278.9	1.28E+05
Columns total									*3304.7*	*1.52E+06*
Beams (y)	200x1800mm GL28h			15.72	165.6	5.66	460	144	814.9	3.75E+05
Beams (x,large)	280x1800mm GL28h			24.00	231.8	12.10	460	150	1814.4	8.35E+05
Beams (x,small)	280x1800mm GL28h			23.80	231.8	11.99	460	135	1619.4	7.45E+05
Beams total									*4248.7*	*1.95E+06*
Timber total									**7553.4**	**3.47E+06**
Steel connections	Steel, 0.05% timber volume						7850		3.8	2.96E+04
Floor slab										
Floor slab	200mm C30/37	51840	500.0				2500		10368.0	2.59E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		158.5	1.24E+06
Foundation pads										
Foundation pads	4000x4000x1250m C30/37					20.0	2500	238	4760.0	1.19E+07
Steel rebar	Steel, 250kg/m^3 concrete						7850		151.6	1.19E+06

E

Building envelope alternatives

This appendix details the design process of all envelope alternatives created in this research. In each case, the total weight on the warehouse load-bearing frame is calculated to adjust the size of structural beams and columns accordingly, as explained in Section E.1. The scope of envelope design is limited to the selection of design alternatives relevant for a warehouse, of known weight, dimensions and environmental impact. The baseline option detailed in Section E.3, is used for all warehouses of design step 1 (reference steel and concrete, timber A and B) and consists of Kingspan panels on steel supports applied to the roof and facades. Alternatives from design step 2 are developed in Sections E.4 to E.9, for a warehouse design based on the timber baseline A only.

E.1 Reinforced frames for higher roof loads

The first step when creating envelope design alternatives is to ensure that the load-bearing timber frame will be able to resist potential higher dead loads than the baseline timber frame, by adjusting the size of structural elements accordingly. In this research, four situations are considered based on the timber baseline design A, assuming dead weights of the roof ranging from 50 to 200kg/m^2. The maximum weight of the facade is set to 100kg/m^2 to accommodate for various vertical green systems. An overview of frame options and maximum allowable weights is given in Table E.1.

Table E.1: Maximum dead loads from roof and facade systems considered in the structural design of the baseline load-bearing frame A, and adjusted frames a to d for heavier envelope alternatives

Frame	**Roof maximum weight** [kN/m^2]	[kg/m^2]	**Facade maximum weight** [kN/m^2]	[kg/m^2]
Baseline	*0.30*	30.6	*0.50*	51.0
Frame (a)	*0.49*	50	*0.98*	100
Frame (b)	*0.98*	100	*0.98*	100
Frame (c)	*1.47*	150	*0.98*	100
Frame (d)	*1.96*	200	*0.98*	100

The design process is the same as described extensively in Appendix C, and all structural members are made from glulam GL28h. The size of column footings remains the same as in the baseline for all alternatives, that is 3000x3000x1000mm pads of reinforced in situ concrete C30/37.

E.1.1 Frame (a)

The first frame alternative (a) is designed for a maximum of 50kg/m^2 dead loads from the roofing system, and 100kg/m^2 from facades. The dimensions of main columns are governed by the required number of rods in the section to achieve sufficient moment capacity of the base connection ($UC_{base} = 0.79$), resulting in a total rotational stiffness of 110 MNm/rad.

Table E.2: Governing unity checks at ULS and SLS for structural members of Frame (a)

Element	Description	ULS checks		SLS checks		
		$UC_{section}$	$UC_{stability}$	δ_{max}	$\delta_{allowable}$	$UC_{deflection}$
Roof beams (x)	300x1800mm	0.82	0.65	76.3mm	96.0mm	0.79
Roof beams (y)	240x1800mm	0.85	0.63	31.5mm	62.9mm	0.50
Main columns	1000x1000mm	0.13	0.21	45.9mm	58.9mm	0.78
Facade columns	400x400mm	0.48	0.75	42.4mm	58.9mm	0.72

E.1.2 Frame (b)

The second frame alternative (b) is designed for a maximum of 100kg/m^2 dead loads from the roofing system, and 100kg/m^2 from facades. The dimensions of main columns are governed by the moment capacity of the base connection ($UC_{base} = 0.83$), resulting in a total rotational stiffness of 111 MNm/rad.

Table E.3: Governing unity checks at ULS and SLS for structural members of Frame (b)

Element	Description	ULS checks		SLS checks		
		$UC_{section}$	$UC_{stability}$	δ_{max}	$\delta_{allowable}$	$UC_{deflection}$
Roof beams (x)	360x1800mm	0.82	0.65	76.7mm	96.0mm	0.80
Roof beams (y)	320x1800mm	0.85	0.57	28.8mm	62.9mm	0.46
Main columns	1000x1000mm	0.13	0.23	47.1mm	58.9mm	0.80
Facade columns	410x410mm	0.45	0.74	38.4mm	58.9mm	0.65

E.1.3 Frame (c)

The third frame alternative (c) is designed for a maximum of 150kg/m^2 dead loads from the roofing system, and 100kg/m^2 from facades. The dimensions of main columns are governed by deflections. The moment capacity requirement of the base connection is satisfied ($UC_{base} = 0.73$), and the total rotational stiffness is 112 MNm/rad.

Table E.4: Governing unity checks at ULS and SLS for structural members of Frame (c)

Element	Description	ULS checks		SLS checks		
		$UC_{section}$	$UC_{stability}$	δ_{max}	$\delta_{allowable}$	$UC_{deflection}$
Roof beams (x)	420x1800mm	0.79	0.65	76.9mm	96.0mm	0.80
Roof beams (y)	380x1800mm	0.85	0.56	28.5mm	62.3mm	0.46
Main columns	1050x1050mm	0.12	0.21	45.9mm	58.9mm	0.78
Facade columns	410x410mm	0.45	0.79	38.4mm	58.9mm	0.65

E.1.4 Frame (d)

The fourth and last frame alternative (d) is designed for a maximum of 200kg/m^2 dead loads from the roofing system, and 100kg/m^2 from facades. The dimensions of main columns are governed by deflections. The moment capacity requirement of the base connection is satisfied ($UC_{base} = 0.76$), and the total rotational stiffness is 113 MNm/rad.

Table E.5: Governing unity checks at ULS and SLS for structural members of Frame (d)

Element	Description	ULS checks		SLS checks		
		$UC_{section}$	$UC_{stability}$	δ_{max}	$\delta_{allowable}$	$UC_{deflection}$
Roof beams (x)	480x1800mm	0.79	0.64	77.2mm	96.0mm	0.80
Roof beams (y)	460x1800mm	0.85	0.53	27.3mm	62.1mm	0.44
Main columns	1050x1050mm	0.13	0.23	46.9mm	58.9mm	0.80
Facade columns	420x420mm	0.43	0.78	35.0mm	58.9mm	0.59

E.2 Loads and envelope design principles

A panel structural capacity depends on its thickness and the loading it should withstand. The design of facades is governed by horizontal wind loads, while roofs also have to resist vertical gravity loads. The combination of these two factors gives the maximum allowable span of a panel, defining the required number of supports and their spacing. In general, the best compromise should be found between lighter panels requiring closer supports, or heavier panels stiff enough to span larger distances, reducing the need for additional supports. Multispan elements are particularly suited to cover large areas on the roof and facade, to speed up construction time and better control deflections.

An overview of design loads is given in Table E.6 for all roofing alternatives, and in Table E.7 for facade designs. Surface loads are translated into line loads using centre-to-centre distance between supporting elements, and point loads considering their length. These values are compared to those from design tables by manufacturers, or included in structural calculations to dimension panels and supports.

Table E.6: Loads for the selection and design of roof panels and purlins in envelope alternatives

Description	ULS [kN/m^2]	SLS [kN/m^2]	CTC [m]	ULS [kN/m]	SLS [kN/m]	Length [m]	ULS [kN]	SLS [kN]
F003 (ULS) / F103 (SLS) - Snow down								
LC2 - Weight Kingspan sandwich panel	1.35*0.11 =0.15	1.0*0.11 =0.11						
LC2 - Weight PUR sandwich panel	1.35*0.13 =0.18	1.0*0.13 =0.13						
LC2 - Weight MW sandwich panel	1.35*0.29 =0.39	1.0*0.29 =0.29						
LC7 - Snow	1.5*1.20 =1.80	1.0*1.20 =1.20						
LC9/LC13 - Wind pressure A	0.9*0.70 =0.63	0.6*0.70 =0.42						
LC9/LC13 - Wind pressure B	0.9*0.75 =0.68	0.6*0.75 =0.45						
Total A - without panels	2.43	1.62	3	7.29	4.86	8	58.32	38.88
Total A - Kingspan panels	2.58	1.73	3	7.74	5.19	8	61.92	41.52
Total A - PUR panels	2.61	1.75	3	7.83	5.25	8	62.64	42.00
Total A - MW panels	2.82	1.91	3	8.46	5.73	8	67.68	45.84
Total A - 50kg/m^2 roof	3.09	2.11	2	6.18	4.22	8	49.47	33.76
Total A - 100kg/m^2 roof	3.75	2.60	1	3.75	2.60	8	30.03	20.80
Total A - 150kg/m^2 roof	4.42	3.09	1	4.42	3.09	8	35.32	24.72
Total A - 200kg/m^2 roof	5.08	3.58	1	5.08	3.58	8	40.62	28.65
Total B - without panels	2.48	1.65	3	7.44	4.95	8	59.52	39.60
Total B - Kingspan panels	2.63	1.76	3	7.89	5.28	8	63.12	42.24

Table E.7: Loads for the selection and design of facade panels and mullions in envelope alternatives

Description	ULS [kN/m^2]	SLS [kN/m^2]	CTC [m]	ULS [kN/m]	SLS [kN/m]	Length [m]	ULS [kN]	SLS [kN]
F004 (ULS) / F104 (SLS) - Wind down								
LC9/LC13 - Wind pressure A	1.5*1.26 =1.89	1.0*1.26 =1.26						
LC9/LC13 - Wind pressure B	1.5*1.34 =2.01	1.0*1.34 =1.34						
Total A - 4m spacing	1.89	1.26	4	7.56	5.04	10.3	77.87	51.91
Total A - 2.67m spacing	1.89	1.26	2.67	5.04	3.36	10.3	51.91	34.61
Total B - 4m spacing	2.01	1.34	4	8.04	5.36	14.3	114.97	76.55

Table E.8: Description of Kingspan sandwich panels with steel sheets and Quadcore rigid foam insulation core

Description	Dimensions	Span [m]	Thickness [mm]	U-value [m2K/W]	Weight [kg/m^2]
Kingspan Quadcore KS1000RW Roof sandwich panel (0.465/0.32 steel) (Kingspan, 2019b)	Length 1.8-14.5m Width 1000m	Single = 4.6 Double = 3.0 Triple = 3.5	100	0.18	11.3
Kingspan Quadcore Evolution Facade sandwich panel (0.63/0.4 steel) (Kingspan, 2019a)	Length 1.8-12m Width 1000m	Single = 8.5 Double = 7.7 Triple = 8.0	100	0.19	13.1

Table E.9: Description of sandwich panels with steel sheets and polyurethane (PUR) insulation core

Description	Dimensions	Span [m]	Thickness [mm]	U-value [m2K/W]	Weight [kg/m^2]
PUR insulation roof sandwich panel (0.5/0.4 steel) (ISOPAN, 2015b)	Unlimited length Width 1000m	Single = 3.8	120	0.19	13.7
PUR insulation facade sandwich panel (0.5/0.4 steel) (ISOPAN, 2015a)	Unlimited length Width 1000m	Single = 3.7 Multi = 3.3	120	0.19	12.8

Table E.10: Description of sandwich panels with steel sheets and mineral wool (MW) insulation core

Description	Dimensions	Span [m]	Thickness [mm]	U-value [m2K/W]	Weight [kg/m^2]
MW insulation roof sandwich panel (0.5/0.5 steel) (ISOPAN, 2015a)	Unlimited length Width 1000m	Single = 3.0	200	0.20	29.4
MW insulation facade sandwich panel (0.5/0.5 steel) (ISOPAN, 2015c)	Unlimited length Width 1000m	Single = 3.6 Multi = 2.0	200	0.20	28.2

E.3 Baseline - Kingspan sandwich panels, steel supports

The envelope of both timber baseline warehouses developed in this research is the same as for the reference steel and concrete designs: Kingspan sandwich panels with Quadcore insulation are installed on steel supports, on the roof and facades. The Quadcore insulation material is a type of rigid foam insulation created by Kingspan, with improved thermal performance and fire protection compared to standard PUR. The characteristics of these Kingspan roof and facade panels are detailed in Table E.8. The bill of materials used in the baseline envelope is presented in Table E.11.

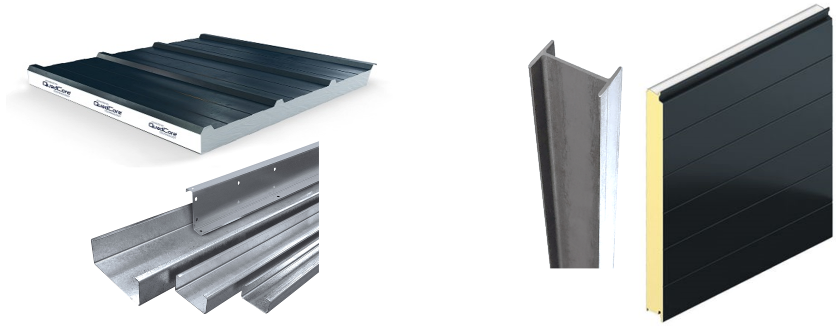


Figure E.1: Baseline envelope - Kingspan roof sandwich panels & steel Multibeam purlins (left), Kingspan facade sandwich panels & steel IPE mullions (right)

E.3.1 Roof design - steel purlins and Kingspan panels

Kingspan panels on the roof are placed on top of steel purlins, spaced every 3 metres. Steel purlins are pinned at both ends, spanning in the y-direction and connected at the top of roof beams spanning perpendicularly in the x-direction. The dimensions of steel Multibeam purlins are selected from Kingspan design tables (Kingspan, 2010), to resist live loads on the roof, the dead weight of panels and their selfweight. The same panels and purlins are selected for all warehouse designs of step 1. The weight of the roof system is calculated considering panels (11.3) and purlins (3.6), resulting in a total weight of 14.9kg/m^2 applied on the timber load-bearing frame. This value complies with the requirements of the baseline frame.

E.3.2 Facade design - steel mullions and Kingspan panels

Panels should be oriented perpendicular to the supporting members: here, horizontal sandwich panels are laid on vertical mullions along the facade. Mullions are added at regular intervals between facade columns so that Kingspan facade panels are supported every 4 metres. These mullions consist in steel IPE members pinned at both ends on foundation pads and roof beams. They are dimensioned to resist horizontal wind loads, the weight of facade panels and their selfweight. The magnitude of wind loads is larger for taller warehouses (concrete reference and timber baseline B), calling for stronger mullions cross-sections than for smaller structures (steel reference and timber baseline A).

Table E.11: Bill of materials of envelope elements for the reference and timber baseline designs

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Envelope - Steel reference SC16 and timber baseline A										
Roof panels	Kingspan sandwich panels	23040	11.3							
Roof purlins (ctc=3m)	Steel Multibeam M300090270			8.0	10.64		7850	980		8.34E+04
Facade panels	Kingspan sandwich panels	7964.8	13.1							
Facade mullions (ctc=4m)	IPE360 S355			10.3	57.1		7850	76		4.47E+04
Envelope - Concrete reference IXD and timber baseline B										
Roof panels	Kingspan sandwich panels	51840	11.3							
Roof purlins (ctc=3m)	Steel Multibeam M300090270			8.0	10.64		7850	2178		1.85E+05
Facade panels	Kingspan sandwich panels	17236.8	13.1							
Facade mullions (ctc=4m)	IPE500 S355			14.3	90.7		7850	126		1.63E+05

E.4 PUR sandwich panels, steel supports

In the first design alternative, sandwich panels with a polyurethane (PUR) insulating core (see Figure E.2) replace Kingspan panels on the roof and facades. Because their weight is of the same magnitude, steel supports remain the same as in the baseline envelope. The characteristics of these panels are detailed in Table E.9.

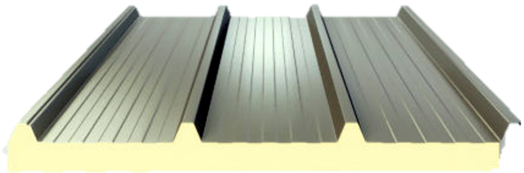


Figure E.2: PUR insulation core sandwich panel

The weight of the roof system is calculated considering panels (13.7) and purlins (3.6), resulting in a total weight of 17.3kg/m^2 on the timber frame. This value complies with the requirements of the baseline frame. The bill of all materials used in this design alternative (frame, foundation pads, floor slab, envelope) is presented in Table E.12.

E.5 MW sandwich panels, steel supports

In this design alternative, sandwich panels with a mineral wool (MW) insulating core (see Figure E.3) replace Kingspan panels on the roof and facades. Because they are heavier, the size of steel supports is larger than in the baseline envelope. The characteristics of these panels are detailed in Table E.10.

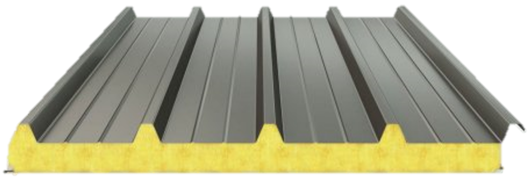


Figure E.3: MW insulation core sandwich panel

The weight of the roof system is calculated considering panels (29.4) and purlins (4.0), resulting in a total weight of 33.4kg/m^2 on the timber frame. This value does not comply with the requirements of the baseline frame, which shall be replaced by frame (a). The bill of all materials used in this design alternative (frame, foundation pads, floor slab, envelope) is presented in Table E.13.

Table E.12: Bill of materials for the "PUR sandwich panels on steel supports" alternative (envelope, baseline frame, floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Envelope										
Roof panels	PUR sandwich panels	23040	13.7							
Roof purlins (ctc=3m)	Steel Multibeam M300090270			8.0	10.64		7850	980		8.34E+04
Facade panels	PUR sandwich panels	7964.8	12.8							
Facade mullions (ctc=2.67m)	IPE330 S355			10.3	49.1		7850	152		7.69E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	400x400mm GL28h			10.30	73.6	1.65	460	44	72.5	3.33E+04
Columns total									*865.6*	*3.98E+05*
Beams (y)	220x1800mm GL28h			15.72	182.2	6.23	460	70	435.8	2.00E+05
Beams (x,large)	280x1800mm GL28h			24.00	231.8	12.10	460	66	798.3	3.67E+05
Beams (x,small)	280x1800mm GL28h			23.78	231.8	11.99	460	60	719.1	3.31E+05
Beams total									*1953.2*	*8.98E+05*
Timber total									**2818.8**	**1.30E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.4	1.11E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.13: Bill of materials for the "MW sandwich panels on steel supports" alternative (envelope, frame (a), floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Envelope										
Roof panels	MW sandwich panels	23040	29.4							
Roof purlins (ctc=3m)	Steel Multibeam M350090270			8.0	11.66		7850	980		9.14E+04
Facade panels	MW sandwich panels	7964.8	28.2							
Facade mullions (ctc=2.67m)	IPE330 S355			10.3	49.1		7850	152		7.69E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	400x400mm GL28h			10.30	73.6	1.65	460	44	72.5	3.33E+04
Columns total									*865.6*	*3.98E+05*
Beams (y)	240x1800mm GL28h			15.70	198.7	6.78	460	70	474.8	2.18E+05
Beams (x,large)	300x1800mm GL28h			24.00	248.4	12.96	460	66	855.4	3.93E+05
Beams (x,small)	300x1800mm GL28h			23.76	248.4	12.83	460	60	769.8	3.54E+05
Beams total									*2100.0*	*9.66E+05*
Timber total									**2965.6**	**1.36E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.5	1.16E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

E.6 Kingspan sandwich panels, timber supports

In this design alternative, Kingspan sandwich panels remain the same, but steel supports are replaced by glulam members (see Figure E.4). Glulam purlins are connected to the sides of roof beams spanning in the x-direction with shear connectors placed in the top part, and are considered pinned on both sides.

Figure E.4: Kingspan roof sandwich panels & glulam purlins (left), Kingspan facade sandwich panels & glulam mullions (right)

The weight of the roof system is calculated considering panels (11.3) and purlins (6.2), resulting in a total weight of 17.5kg/m^2 on the timber frame. This value complies with the requirements of the baseline frame. The bill of all materials used in this design alternative (frame, foundation pads, floor slab, envelope) is presented in Table E.14.

E.7 PUR sandwich panels, timber supports

This design alternative combines PUR sandwich panels, described in Table E.9, with the same glulam supports as the previous alternative. The weight of the roof system is calculated considering panels (13.7) and purlins (6.2), resulting in a total weight of 19.9kg/m^2 on the timber frame. This value complies with the requirements of the baseline frame. The bill of all materials used in this design alternative (frame, foundation pads, floor slab, envelope) is presented in Table E.15.

E.8 MW sandwich panels, timber supports

This design alternative combines MW sandwich panels, described in Table E.10, with glulam supports. The weight of the roof system is calculated considering panels (29.4) and purlins (6.3), resulting in a total weight of 35.7kg/m^2 on the timber frame. This value does not comply with the requirements of the baseline frame, which shall be replaced by frame (a). The bill of all materials used in this design alternative (frame, foundation pads, floor slab, envelope) is presented in Table E.16.

Table E.14: Bill of materials for the "Kingspan sandwich panels on timber supports" alternative (envelope, baseline frame, floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Envelope										
Roof panels	Kingspan sandwich panels	23040	13.7							
Roof purlins (ctc=3m)	100x480mm GL28h			7.72	22.1		460	840		1.43E+05
Facade panels	Kingspan sandwich panels	7964.8	12.8							
Facade mullions (ctc=4m)	390x390mm GL28h			10.3	70.0		460	76		5.48E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	400x400mm GL28h			10.30	73.6	1.65	460	44	72.5	3.33E+04
Columns total									*865.6*	*3.98E+05*
Beams (y)	220x1800mm GL28h			15.72	182.2	6.23	460	70	435.8	2.00E+05
Beams (x,large)	280x1800mm GL28h			24.00	231.8	12.10	460	66	798.3	3.67E+05
Beams (x,small)	280x1800mm GL28h			23.78	231.8	11.99	460	60	719.1	3.31E+05
Beams total									*1953.2*	*8.98E+05*
Timber total									**2818.8**	**1.30E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.4	1.11E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.15: Bill of materials for the "PUR sandwich panels on timber supports" alternative (envelope, baseline frame, floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Envelope										
Roof panels	PUR sandwich panels	23040	13.7							
Roof purlins (ctc=3m)	100x480mm GL28h			7.72	22.1		460	840		1.43E+05
Facade panels	PUR sandwich panels	7964.8	12.8							
Facade mullions (ctc=2.67m)	350x350mm GL28h			10.3	56.4		460	152		8.82E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	400x400mm GL28h			10.30	73.6	1.65	460	44	72.5	3.33E+04
Columns total									*865.6*	*3.98E+05*
Beams (y)	220x1800mm GL28h			15.72	182.2	6.23	460	70	435.8	2.00E+05
Beams (x,large)	280x1800mm GL28h			24.00	231.8	12.10	460	66	798.3	3.67E+05
Beams (x,small)	280x1800mm GL28h			23.78	231.8	11.99	460	60	719.1	3.31E+05
Beams total									*1953.2*	*8.98E+05*
Timber total									**2818.8**	**1.30E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.4	1.11E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.16: Bill of materials for the "MW sandwich panels on timber supports" alternative (envelope, frame (a), floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Envelope										
Roof panels	MW sandwich panels	23040	29.4							
Roof purlins (ctc=3m)	100x490mm GL28h			7.70	22.5		460	840		1.46E+05
Facade panels	MW sandwich panels	7964.8	28.2							
Facade mullions (ctc=2.67m)	350x350mm GL28h			10.3	56.4		460	152		8.82E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	400x400mm GL28h			10.30	73.6	1.65	460	44	72.5	3.33E+04
Columns total									*865.6*	*3.98E+05*
Beams (y)	240x1800mm GL28h			15.70	198.7	6.78	460	70	474.8	2.18E+05
Beams (x,large)	300x1800mm GL28h			24.00	248.4	12.96	460	66	855.4	3.93E+05
Beams (x,small)	300x1800mm GL28h			23.76	248.4	12.83	460	60	769.8	3.54E+05
Beams total									*2100.0*	*9.66E+05*
Timber total									**2965.6**	**1.36E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.5	1.16E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

E.9 Biobased envelope

This section describes each layer of the biobased roofing and facade alternatives (see Figure E.5). The total bill of materials of this design is presented in Table E.17.

Figure E.5: Biobased build-up roofing (left) and facade system (right)

E.9.1 Biobased build-up roofing system

The weight of the system is calculated considering panels (38.0) and insulation (9.0), resulting in a total weight of 47.0kg/m^2 on the timber frame. This value does not comply with the requirements of the baseline frame, which shall be replaced by frame (a).

Lignatur panels

Lignatur surface elements are prefabricated wood-based load-bearing stressed skin panels, made from finger jointed softwood boards separated by stabilising stiffeners at regular distances. These box elements can fit thermal insulation or ballast weight depending on the intended use of the panel, in roofs or floors. The intended working life of such elements is 50 years when subject to proper installation, use and maintenance.

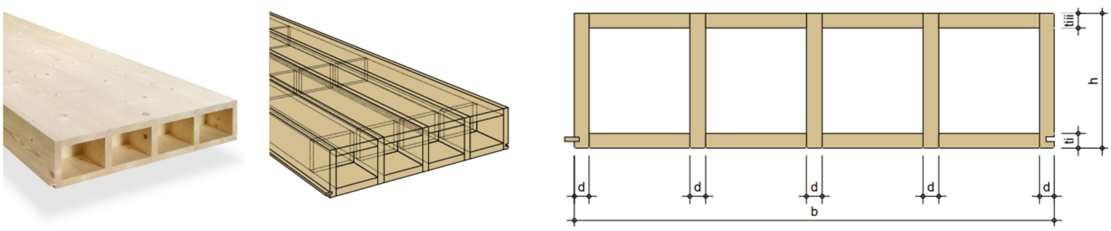


Figure E.6: Dimensions of Lignatur surface element (LFE): height h=180mm, width b=1000mm, thickness of ribs d=31mm, thickness skin t_i=31mm, thickness skin t_{iii}=31mm, number of boxes n=4, Length L=8m, weight=38kg/m^2 (ETA-11/013, 2021)

The panels dimensions, illustrated on Figure E.6, are chosen based on design indications from the product European Technical Assessment (ETA-11/013, 2021) to satisfy deflection requirements under design wind and snow loads at SLS. A spanning capacity of 8 metres between roof beams discards the need for additional purlins as supports. The environmental impact of Lignatur panels is calculated from their weight and environmental data for glulam, to account for adhesives used to assemble wooden elements.

Wood fibre insulation

Many different biobased materials can be used as insulation. In this project, wood fibres are chosen to fill the inside of roof box elements, and a 80mm layer is added on top to achieve a thermal insulation value U=0.20W/m^{2}K, as illustrated on Figure E.7. The density of wood fibre insulation is assumed to be 50 kg/m^2, and the volume of material used in a 1m^2 area is: $V = (b - 5d)(h - (t_i + t_{iii})) + 0.080 = 0.18m^3$

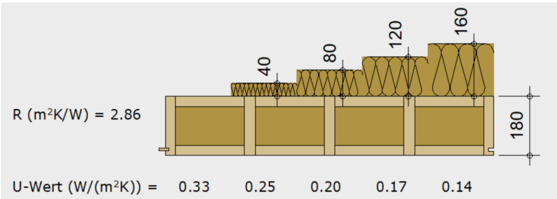


Figure E.7: Wood fibre thermal insulation in Lignatur surface element (LFE) - The heat conductivity of coniferous wood is taken as λ=0.130 W/mK, and for wood fibre λ=0.040 W/mK.

EPDM layer

To prevent Lignatur panels and the wood fibre insulation layer from being directly exposed to the weather, a protective EPDM layer is added on top over the entire roof surface. The service life of this material is 40 years, less that the 50 years reference service life of the warehouse. Environmental impact calculations should consider the total amount of materials used in the warehouse over its whole lifecycle, therefore replacing of the EPDM layer once translates as considering twice the amount of material at the production stage.

E.9.2 Biobased build-up facade system

Glulam mullions

Facade mullions are 350x350mm elements made of glulam GL28h.

OSB panel

Wood-based panels are used on the inside of the facade build-up to provide stiffness and encase the insulation layer inside. Here, 15mm thick OSB panels are used.

Wood fibre insulation and wooden I-joists

The thickness of the insulation layer is calculated to satisfy thermal insulation requirements, also considering the OSB layer. This results in a 200mm thickness when using the same wood fibre insulation as in the roofing system. Vertical wood-based I-joists are also added every metre inside this layer to stiffen the build-up panel.

Steel sheet

To prevent biobased layers from being directly exposed to the weather, a protective steel sheet is added on the external side of the facade system.

Table E.17: Bill of materials for the ”Biobased envelope” alternative (envelope, frame (a), floor slab, foundation pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Envelope										
Roof panels	Lignatur panels 180mm	23040	38.0							8.76E+05
	Wood fibre insulation 80mm	23040	9.0				50		4.14E+03	2.07E+05
	EPDM protective layer	46080								
Facade panels	OSB panels 15mm	7964.8							1.19E+02	
	Wood fibre insulation 200mm	7964.8	10.0						1.59E+03	7.96E+04
	Wood I-joists ctc=1m			13.1	4.4			604	L=7.91E+04m	3.48E+04
	Steel sheet	7964.8								
Facade mullions (ctc=2.67m)	350x350mm GL28h			10.3	56.4		460	152		8.82E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	400x400mm GL28h			10.30	73.6	1.65	460	44	72.5	3.33E+04
Columns total									*865.6*	*3.98E+05*
Beams (y)	240x1800mm GL28h			15.70	198.7	6.78	460	70	474.8	2.18E+05
Beams (x,large)	300x1800mm GL28h			24.00	248.4	12.96	460	66	855.4	3.93E+05
Beams (x,small)	300x1800mm GL28h			23.76	248.4	12.83	460	60	769.8	3.54E+05
Beams total									*2100.0*	*9.66E+05*
Timber total									**2965.6**	**1.36E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.5	1.16E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

E.10 Green envelope design

The design of the green envelope is limited to the description of additional dead loads on the frame, and establishing the bill of materials for environmental impact calculations.

E.10.1 Green roof

Horizontal greening is executed with an extensive green roof system covering the entire roof area. Typical layers of an extensive green roof are illustrated on Figure E.8.

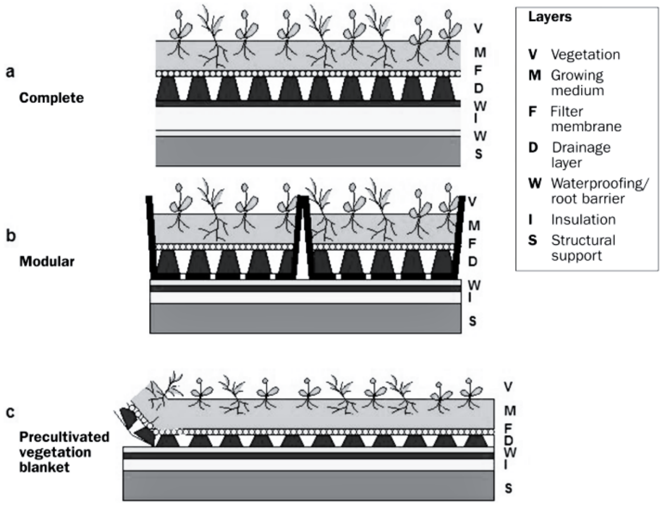


Figure E.8: Types of extensive green-roofing technology. (a) Complete systems form integral part of the roof, (b) Modular systems are installed above the existing roof system, (c) Precultivated vegetation blankets are rolled onto the existing roof (Oberndorfer et al., 2007)

Dead loads are determined based on the saturated weight of the system. In this study, three substrate depths are considered to be representative of extensive roof systems of weights ranging between 50 and 150kg/m^2. Environmental impact calculations consider the same score for extensive roof systems weighting 50, 100 or 150kg/m^2 as layers responsible for the most impact are not the organic substrate or vegetation, but generally fossil-based materials composing other layers (drainage, waterproofing...).

The two selected EPDs (see Appendix G) consider extensive roof systems from Urbanscape and Nature Impact AS. The first consists of pre-cultivated rolls of sedum around 40mm thick, on top of a drainage layer made of 100% recyclable polypropylene. The second is made of sedum cuttings on subtrate, with plastic trays underneath for a total thickness of 60mm. A root barrier membrane could be added, but is assumed unnecessary.

E.10.2 Green facade

The green facade considered in this study is realised with climber plants, growing on steel mesh supports attached to the warehouse facade. The design of this indirect greening system is based on recommendations from the British manufacturer Jakob Rope Systems

(Jakob Rope Systems, 2020). The environmental impact of the green facade system comes from materials used in the support system. Here, the volume of stainless steel is estimated based on the dimensions illustrated on Figure E.9.

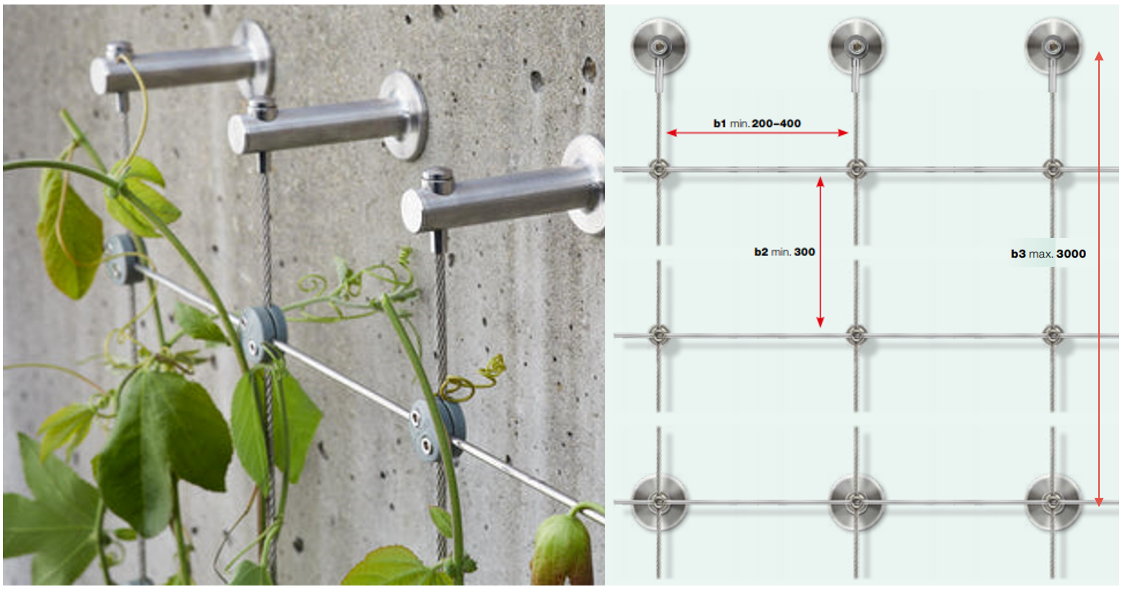


Figure E.9: Indirect greening system and supports dimensions (Jakob Rope Systems, 2020)

The green system is assumed to span 12.5m within the total height of the facade (13.1m), while covering its entire perimeter. Like in previous calculations, any openings such as truck doors are disregarded. The vertical distance between spacers is taken as b_3=3000mm, the horizontal distance b_1=500mm and the vertical distance between smaller cross clamps is b_2=1000mm. From these dimensions, the number of vertical ropes and horizontal rods covering the facade can be determined, as well as the number of spacers and cross clamps. By estimating the volume of each element, the bill of materials can be drawn:

- **Vertical ropes:** 4mm diameter, 12.5m length (green system height), 1216 in total along the perimeter = 1.91E-01m^3
- **Horizontal rods:** 3.7mm diameter, 608m length (warehouse perimeter), 12 in total along the height = 7.84E-02m^3
- **Spacers:** approx. 4.32E-05m^3 each, 6080 in total = 2.63E-01m^3
- **Cross clamps:** approx. 6.60E-06m^3 each, 14592 in total = 9.63E-02m^3
- **Total volume of stainless steel = 6.28E-01m^3**
- **Total mass of stainless steel = 4.93E+03kg**

The total environmental impact of facade greening supports is calculated using GWP values from the stainless steel EPD described in Appendix G. The impact of plants is considered negligible. The same vertical greening system is kept in all green alternatives.

E.10.3 Green on baseline envelope

The green envelope imposes higher dead loads on the structure and building envelope underneath. For the roof, in particular, the additional weight shall be accounted for to adapt the design of the envelope, reducing the centre-to-centre distance between purlins following the lower spanning capacity of roof panels under increased loading. Based on the loads calculated in Table E.6 and the maximum allowable span of Kingpsan sandwich panels from design tables (Kingspan, 2019b), the configuration of steel purlins is determined and indicated in Table E.18. Bills of materials for each design alternative are given in Tables E.20 to E.22.

Table E.18: Kingspan panels on steel supports roof design based on green roof possible weight

Green roof weight [kg/m^2]	Steel purlins	CTC [m]	Baseline roof weight [kg/m^2]	Total weight [kg/m^2]	Frame
0	M300090270 10.64 kg/m	3.0	11.3+3.6=14.9	14.9	Baseline
50	M300090200 7.86 kg/m	2.0	11.3+4.0=15.3	65.3	Frame (b)
100	M265065200 6.43 kg/m	1.0	11.3+6.5=17.8	117.8	Frame (c)
150	M265065220 7.03 kg/m	1.0	11.3+7.1=18.4	168.4	Frame (d)

E.10.4 Green on biobased envelope

The dimensions of wood-based Lignatur roof elements in the biobased envelope design should be adapted to resist higher loads when supporting a green roof. Table E.19 offers a recap of how the biobased roof can be adapted considering green roofs between 50 and 150kg/m^2, compared to no green roof. The main variables are the height H of the Lignatur panel, and the thickness of the wood fibre insulation layer on top required to achieve a thermal insulation value U=0.20W/m^{2}K. The weight of the biobased roof is calculated by summing the weight of Lignatur panels filled with wood fibres, and the weight of the insulation layer on top. When considering the green roof system as well, we can determine which timber frame is the most adapted to resist the total dead load. Bills of materials for each design alternative are given in Tables E.23 to E.25.

Table E.19: Biobased roof design based on green roof possible weight

Green roof weight [kg/m^2]	Lignatur H [mm]	Insulation layer [mm]	Biobased roof weight [kg/m^2]	Total weight [kg/m^2]	Frame
0	180	80	38.0+9.0=47.0	47.0	Frame (a)
50	200	70	39.0+9.3=48.3	98.3	Frame (b)
100	220	50	41.0+9.2=50.2	150.2	Frame (c)
150	240	30	42.0+9.0=51.0	201.0	Frame (d)

Table E.20: Bill of materials for the ”Baseline envelope & Green roof 50kg/m^2” alternative (green, envelope, frame (b), floor slab, pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Green										
Green roof	Plants, substrate, drainage	23040	50.0							
Green facade	Plants, stainless steel mesh	7964.6	0.62				7850		6.28E-01	4.93E+03
Envelope										
Roof panels	Kingspan sandwich panels	23040	11.3							
Roof purlins (ctc=2m)	Steel Multibeam M300090200			8.0	7.86		7850	1460		9.18E+04
Facade panels	Kingspan sandwich panels	7964.8	13.1							
Facade mullions (ctc=4m)	IPE360 S355			10.3	57.1		7850	76		4.47E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	410x410mm GL28h			10.30	77.3	1.73	460	44	76.2	3.50E+04
Columns total									*869.3*	*4.00E+05*
Beams (y)	320x1800mm GL28h			15.64	198.7	9.01	460	70	630.6	2.90E+05
Beams (x,large)	360x1800mm GL28h			24.00	298.1	15.55	460	66	1026.4	4.72E+05
Beams (x,small)	360x1800mm GL28h			23.68	298.1	15.34	460	60	920.7	4.24E+05
Beams total									*2577.7*	*1.19E+06*
Timber total									**3447.0**	**1.59E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.7	1.35E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.21: Bill of materials for the "Baseline envelope & Green roof 100kg/m^2" alternative (green, envelope, frame (c), floor slab, pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Green										
Green roof	Plants, substrate, drainage	23040	100.0							
Green facade	Plants, stainless steel mesh	7964.6	0.62				7850		6.28E-01	4.93E+03
Envelope										
Roof panels	Kingspan sandwich panels	23040	11.3							
Roof purlins (ctc=1m)	Steel Multibeam M265065200			8.0	6.43		7850	2900		1.49E+05
Facade panels	Kingspan sandwich panels	7964.8	13.1							
Facade mullions (ctc=4m)	IPE360 S355			10.3	57.1		7850	76		4.47E+04
Frame										
Main columns	1050x1050mm GL28h			10.30	507.2	11.36	460	77	874.4	4.02E+5
Facade columns	410x410mm GL28h			10.30	77.3	1.73	460	44	76.2	3.50E+04
Columns total									*950.6*	*4.37E+05*
Beams (y)	380x1800mm GL28h			15.58	314.6	10.66	460	70	746.0	3.43E+05
Beams (x,large)	420x1800mm GL28h			24.00	347.8	18.14	460	66	1197.5	5.51E+05
Beams (x,small)	420x1800mm GL28h			23.62	347.8	17.86	460	60	1071.4	4.93E+05
Beams total									*3014.9*	*1.39E+06*
Timber total									**3965.5**	**1.82E+06**
Steel connections	Steel, 0.05% timber volume						7850		2.0	1.35E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.22: Bill of materials for the ”Baseline envelope & Green roof 150kg/m^2” alternative (green, envelope, frame (d), floor slab, pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Green										
Green roof	Plants, substrate, drainage	23040	150.0							
Green facade	Plants, stainless steel mesh	7964.6	0.62				7850		6.28E-01	4.93E+03
Envelope										
Roof panels	Kingspan sandwich panels	23040	11.3							
Roof purlins (ctc=1m)	Steel Multibeam M265065220			8.0	7.03		7850	2900		1.63E+05
Facade panels	Kingspan sandwich panels	7964.8	13.1							
Facade mullions (ctc=4m)	IPE360 S355			10.3	57.1		7850	76		4.47E+04
Frame										
Main columns	1050x1050mm GL28h			10.30	507.2	11.36	460	77	874.4	4.02E+05
Facade columns	420x420mm GL28h			10.30	81.1	1.82	460	44	79.9	3.68E+04
Columns total									*954.3*	*4.39E+05*
Beams (y)	460x1800mm GL28h			15.52	380.9	12.85	460	70	899.5	4.14E+05
Beams (x,large)	480x1800mm GL28h			24.00	397.4	20.74	460	66	1368.6	6.30E+05
Beams (x,small)	480x1800mm GL28h			23.54	397.4	20.34	460	60	1220.3	5.61E+05
Beams total									*3488.4*	*1.60E+06*
Timber total									**4442.8**	**1.82E+06**
Steel connections	Steel, 0.05% timber volume						7850		2.22	1.35E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.24: Bill of materials for the "Biobased envelope & Green roof 100kg/m^2" alternative (green, envelope, frame (c), floor slab, pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Green										
Green roof	Plants, substrate, drainage	23040	100.0							
Green facade	Plants, stainless steel mesh	7964.6	0.62				7850		6.28E-01	4.93E+03
Envelope										
Roof panels	Lignatur panels 220mm	23040	41.0							9.45E+05
	Wood fibre insulation 50mm	23040	9.2				50		4.23E+03	2.11E+05
	EPDM protective layer	46080								
Facade panels	OSB panels 15mm	7964.8							1.19E+02	
	Wood fibre insulation 200mm	7964.8	10.0						1.59E+03	7.96E+04
	Wood I-joists ctc=1m			13.1	4.4			604	L=7.91E+04m	3.48E+04
	Steel sheet	7964.8								
Facade mullions (ctc=2.67m)	350x350mm GL28h			10.3	56.4		460	152		8.82E+04
Frame										
Main columns	1050x1050mm GL28h			10.30	507.2	11.36	460	77	874.4	4.02E+5
Facade columns	410x410mm GL28h			10.30	77.3	1.73	460	44	76.2	3.50E+04
Columns total									*950.6*	*4.37E+05*
Beams (y)	380x1800mm GL28h			15.58	314.6	10.66	460	70	746.0	3.43E+05
Beams (x,large)	420x1800mm GL28h			24.00	347.8	18.14	460	66	1197.5	5.51E+05
Beams (x,small)	420x1800mm GL28h			23.62	347.8	17.86	460	60	1071.4	4.93E+05
Beams total									*3014.9*	*1.39E+06*
Timber total									**3965.5**	**1.82E+06**
Steel connections	Steel, 0.05% timber volume						7850		2.0	1.35E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.25: Bill of materials for the "Biobased envelope & Green roof 150kg/m^2" alternative (green, envelope, frame (d), floor slab, pads)

Element	Type	A [m^2]	Weight [kg/m^2]	L [m]	Weight [kg/m]	V [m^3]	Weight [kg/m^3]	Nb	Total [m^3]	Total [kg]
Green										
Green roof	Plants, substrate, drainage	23040	150.0							
Green facade	Plants, stainless steel mesh	7964.6	0.62				7850		6.28E-01	4.93E+03
Envelope										
Roof panels	Lignatur panels 240mm	23040	42.0							9.68E+05
	Wood fibre insulation 30mm	23040	9.0				50		4.16+03	2.09E+05
	EPDM protective layer	46080								
Facade panels	OSB panels 15mm	7964.8							1.19E+02	
	Wood fibre insulation 200mm	7964.8	10.0						1.59E+03	7.96E+04
	Wood I-joists ctc=1m			13.1	4.4			604	L=7.91E+04m	3.48E+04
	Steel sheet	7964.8								
Facade mullions (ctc=2.67m)	350x350mm GL28h			10.3	56.4		460	152		8.82E+04
Frame										
Main columns	1050x1050mm GL28h			10.30	507.2	11.36	460	77	874.4	4.02E+05
Facade columns	420x420mm GL28h			10.30	81.1	1.82	460	44	79.9	3.68E+04
Columns total									*954.3*	*4.39E+05*
Beams (y)	460x1800mm GL28h			15.52	380.9	12.85	460	70	899.5	4.14E+05
Beams (x,large)	480x1800mm GL28h			24.00	397.4	20.74	460	66	1368.6	6.30E+05
Beams (x,small)	480x1800mm GL28h			23.54	397.4	20.34	460	60	1220.3	5.61E+05
Beams total									*3488.4*	*1.60E+06*
Timber total									**4442.8**	**1.82E+06**
Steel connections	Steel, 0.05% timber volume						7850		2.22	1.35E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m^3 concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m^3 concrete						7850		34.7	2.72E+05

Table E.23: Bill of materials for the "Biobased envelope & Green roof 50kg/m²" alternative (green, envelope, frame (b), floor slab, pads)

Element	Type	A [m²]	Weight [kg/m²]	L [m]	Weight [kg/m]	V [m³]	Weight [kg/m³]	Nb	Total [m³]	Total [kg]
Green										
Green roof	Plants, substrate, drainage	23040	50.0							
Green facade	Plants, stainless steel mesh	7964.6	0.62				7850		6.28E-01	4.93E+03
Envelope										
Roof panels	Lignatur panels 200mm	23040	39.0							8.99E+05
	Wood fibre insulation 70mm	23040	9.3				50		4.30E+03	2.15E+05
	EPDM protective layer	46080								
Facade panels	OSB panels 15mm	7964.8							1.19E+02	
	Wood fibre insulation 200mm	7964.8	10.0						1.59E+03	7.96E+04
	Wood I-joists ctc=1m			13.1	4.4			604	L=7.91E+04m	3.48E+04
	Steel sheet	7964.8								
Facade mullions (ctc=2.67m)	350x350mm GL28h			10.3	56.4		460	152		8.82E+04
Frame										
Main columns	1000x1000mm GL28h			10.30	460.0	10.30	460	77	793.1	3.65E+05
Facade columns	410x410mm GL28h			10.30	77.3	1.73	460	44	76.2	3.50E+04
Columns total									*869.3*	*4.00E+05*
Beams (y)	320x1800mm GL28h			15.64	198.7	9.01	460	70	630.6	2.90E+05
Beams (x,large)	360x1800mm GL28h			24.00	298.1	15.55	460	66	1026.4	4.72E+05
Beams (x,small)	360x1800mm GL28h			23.68	298.1	15.34	460	60	920.7	4.24E+05
Beams total									*2577.7*	*1.19E+06*
Timber total									**3447.0**	**1.59E+06**
Steel connections	Steel, 0.05% timber volume						7850		1.7	1.35E+04
Floor slab										
Floor slab	200mm C30/37	23040	500.0				2500		4608.0	1.15E+07
Steel rebar	Steel, 120kg/m³ concrete						7850		70.4	5.53E+05
Foundation pads										
Foundation pads	3000x3000x1000m C30/37					9.0	2500	121	1089.0	2.72E+06
Steel rebar	Steel, 250kg/m³ concrete						7850		34.7	2.72E+05

F

Guidelines for environmental impact calculations

This appendix elaborates the methodology followed to calculate the environmental impact of all design variants in this research.

F.1 Assessing the environmental performance of a building based on LCA

The LCA methodology is an efficient tool developed to evaluate the environmental performance of a product or process, assessing environmental and health impacts throughout its life cycle stages. Applied to buildings, an LCA accounts for emissions over the entire life cycle of a building's materials and products. It is especially helpful for identifying the life cycle stage with the most impact or hotspots over the entire life cycle of the building, because it reveals potential for environmental performance optimisation. Then, effective measures can be taken, by prioritising certain impact reduction strategies depending on resource constraints. Another goal of LCA is to compare the environmental performance of different design options of similar functionality (NEN-EN 15978:2011, 2011).

F.1.1 LCA framework

Initially developed at Leiden University in the Netherlands in 1992, the LCA method was later formally defined in European standards ISO 14040 and ISO 14044. More specific rules are given for construction works in standard EN 15978, EN 15804 and ISO 14025. The procedure for calculating the environmental impact of buildings is described by the European standard EN 15978 (Sustainability of construction works - Assessment of environmental performance of buildings – Calculation).

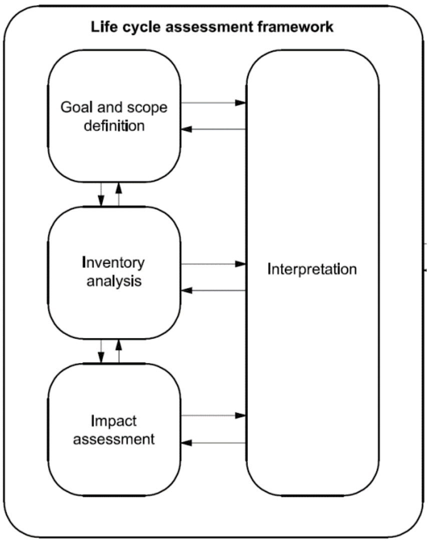


Figure F.1: LCA framework (ISO 14040:2006, 2006)

According to the international standard ISO 14040:2006, four steps should be taken to perform an LCA, illustrated in Figure F.1. First, the goal and scope of the study should be defined along the functional unit of the product or process, so that design variants can be compared on a fair basis. Then, the Life Cycle Inventory (LCI) is made to list all relevant inputs and outputs of the study over all life cycle stages considered. The quantities calculated for materials, products and processes can be represented within a process tree to give a clear overview of the data at hand. The third step consists in assessing the environmental impact of the material and energy flows identified in the previous step, considering a number of environmental impact categories. Finally, the results of the environmental impact assessment are interpreted, refined and discussed.

F.1.2 Life Cycle Stages

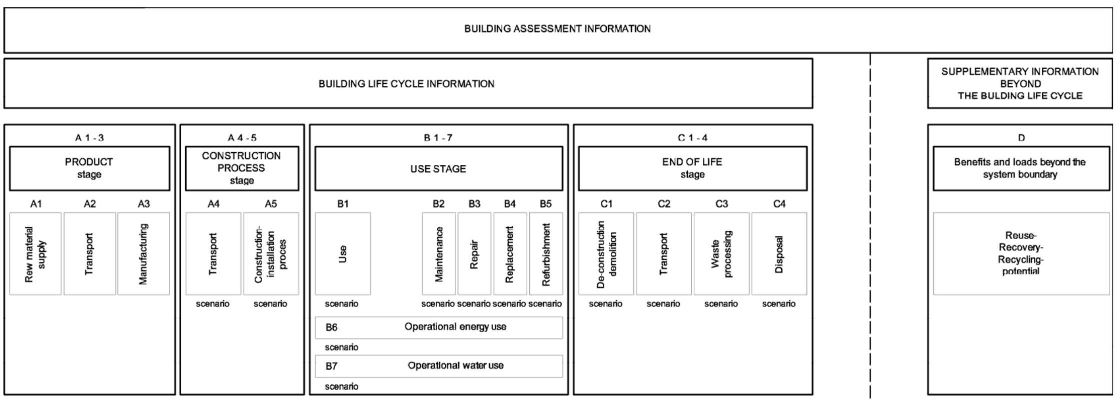


Figure F.2: Life cycle stages (NEN-EN 15978:2011, 2011)

The environmental impact of a building can be described following the LCA methodology. The complete life of the building is divided into five main stages, as described in EN 15978. Each of these stages are themselves divided into modules, detailed on Figure F.2. The product stage (A1-A3) represents the minimum stages to consider according to the EN 15804 standard for construction products. They are the modules considered in so-called "Cradle-to-gate" assessments. Modules A4-A5 represent the construction process stage. Then, the use stage (B1-B7) comprises all processes occurring during the service life of the building. The end-of-life stage (C1-C4) describes the impacts which take place after the service life terminates. "Cradle-tograve" assessments consider life cycles stages from production to end-of-life (A-B-C). The last stage D gives additional information beyond the life cycle of the building. It covers potential benefits from resources to be used as input for other life cycles and is typically included in "Cradle-to-cradle" assessments to encourage circular flows of materials.

F.1.3 Environmental impact categories

Environmental impact categories each refer to a type of damage that can occur to the environment, as a result of human activities depleting finite resources and releasing harmful components at all life cycle stages. These emissions may affect the biotic (living) and abiotic (non-living) environment. Environmental impact categories mainly address damage that is, to a certain extent, quantifiable. Depending on the methodology that is selected to perform the LCA, different bundles of environmental impact categories may be considered. The Dutch 'Bouwbesluit 2012' regulation, for example, requires including 11 basic environmental impact categories when assessing the environmental impact of building products in the Netherlands (Jonkers, 2020). A short description of the most common environmental impact categories is given below.

GWP Global Warming Potential– The Global Warming Potential (GWP) is an anthropogenic phenomenon caused by GHG emissions from human activities, increasing the heat radiation absorbing capacity of the lower atmosphere and resulting in an increase of temperature at the Earths' surface. The most important GHG include water vapor (H2O), carbon dioxide (CO2), methane (CH4), Chlorofluorocarbons (CFCs), Ozone (O3) and Nitrous oxide (N2O). The total effect of these gases is converted to a single reference unit in kg CO2 equivalent, based on their climate forcing effect. The climate forcing effect of a GHG depends on its concentration, atmospheric lifetime, and specific heat absorptive strength.

ODP Ozone Depletion Potential– Ozone (O3) is a powerful GHG in the troposphere, but it also acts as a filter against harmful UV radiation in the stratosphere, a higher atmospheric layer. The Ozone Layer Depletion (ODP) describes the damage caused to the ozone layer by halogenated compounds like CFCs (chloro-fluoro-carbons), HCFCs (hydro-chloro-fluoro-carbons) or halons (bromo-chlorofluoro-carbons), when decomposed by ultraviolets. The total effect of all gases is converted to the reference unit kg CFC-11 equivalent.

AP Acidification Potential– The Acidification Potential (AP) originates from acidic compounds emitted by fossil fuel combustion reacting with water in the atmosphere to form protons, and create acid rain. Such phenomenons have detrimental effects on the natural and built environment like acid attacks damaging construction materials. Typical examples of acidifying compounds include gasses and ions like SO2 (sulfur dioxide), NOx (nitrogen oxides, NO and NO2), and NH4+ (ammonium). Their effect is expressed in kg SO2 equivalent.

EP Eutrophication Potential– The Eutrophication Potential (EP) describes the process of excess nutrient loading of the environment leading to disproportional organic growth in an ecosystem. Nutrients like nitrogen (N) and phosphorous (P) compounds may come from agricultural fertilizers or fuel combustion gases. The reference unit is kg PO4 3-equivalent.

POCP Photochemical Ozone Creation Potential– The Photochemical Ozone Creation Potential (POCP) comes from the reaction of emitted airborne pollutants with sunlight to form compounds that may be harmful to human health and the environment. Pollutants like carbon monoxide (CO), nitrogen oxides (NOx), sulphur dioxide (SO2) and volatile organic compounds (VOCs) mostly originate from the combustion of fossil fuels. Photochemical oxidants include ozone (O3), filtering UV radiation hence lowering global warming in the stratosphere, but harmful in the lower atmosphere at high concentrations. The reference unit is kg ethylene (C2H4) equivalent.

ADP Abiotic Depletion Potential– The Abiotic Depletion Fuel (ADP) relates to the depletion of abiotic (non-living) finite resources, and is divided into two subcategories, namely Abiotic Depletion Fuel (ADPf) and Abiotic Depletion Non-fuel (ADPe). ADPe is about non-fossil resources such as certain types of minerals and metals, and is expressed with a reference unit of kg antimony (Sb) equivalent. ADPf describes fossil resources, using either the same reference unit as ADPe, or MJ net calorific value using the conversion factor 4.81E-4 kg antimony per MJ.

HTP Human Toxicity Potential– The Human Toxicity Potential (HTP) relates to toxic compounds affecting human health. They usually end up in air, water and soil at different concentrations after being emitted from industry or traffic. The reference unit is kg 1,4 dichlorobenzene (DB) equivalent.

FAETP Freshwater Aquatic Ecotoxicity Potential– The Freshwater Ecotoxicity (FAETP) quantifies toxic compounds affecting organisms living in aquatic freshwater environments. Such compounds are released with wastewater, fossil fuel extraction or heavy metals, for instance during the production of steel and cement. The reference unit is kg 1,4 dichlorobenzene (DB) equivalent.

MAETP Marine Aquatic Ecotoxicity Potential– The Marine Ecotoxicity Potential (MAETP) is similar to FAETP, for organisms living in aquatic marine ecosystems. Examples of harmful substances include persistent organic pollutants (POPs), accumulating in the food chain and reaching toxic levels, as they are highly resistant to degradation in the marine environment. The reference unit is kg 1,4 dichlorobenzene (DB) equivalent.

TETP Terrestrial Ecotoxicity Potential– The Terrestrial Ecotoxicity Potential (TETP) revolves around toxic compounds for organisms in terrestrial (land) environments. Harmful substances include pesticides from agricultural activities, toxic for terrestrial plants and animals when they accumulate in the food chain. The reference unit is kg 1,4 dichlorobenzene (DB) equivalent.

F.1.4 Defining the goal and scope

The goal of the study should describe the intended application, the reason behind performing the LCA and the target audience. The scope of the study comprises definitions of the functional unit and system boundaries, as well as a description of the chosen LCA methodology (environmental impact categories, scoring method), the origin, nature and quality of data used for building products, and finally requirements for the final review (Jonkers, 2020).

Defining a functional unit is necessary to enable fair/transparent/unbiased comparison of design variants based on functional equivalence. It represents the major functional and technical characteristics of the object of assessment as a basis for comparison. It shall at least include the type of building, relevant technical and functional requirements, its pattern of use, and its required service life (RSL) (NEN-EN 15978:2011, 2011). Most EPDs only cover the production stage (A1-A3). EPDs are provided by manufacturers to inform clients of the environmental performance of a product over selected modules of its entire life cycle. LCA are used to collect useful information to establish the Environmental Product Declaration (EPD) of a product (NEN-EN 15978:2011, 2011).

F.1.5 Life cycle inventory (LCI)

The LCI is meant to list all relevant inputs and outputs of the considered LCA stages. Inputs may include raw materials and processes feeding the life cycle modules, and outputs as the waste materials and emissions produced and leaving the system (Jonkers, 2020).

Depending on the complexity of the study, large amounts of data may be collected, hence the interesting of using specialized EPD or LCA softwares instead of manually processing it. Environmental databases gather LCI data of commonly used raw materials, elements and processes from the construction sector. Individual Environmental Product Declaration (EPD) may be used for specific products, assuming the right life cycle stages are included and match the required service life of the end building.

F.1.6 Life cycle impact assessmment (LCIA)

The output data identified and collected in the LCI are assigned to a selection of environmental impact categories, each associated with a unit equivalent, for compounds from a single category to be weighed and compared. Then, the unit equivalents are converted to a specific category indicator result expressed in points or another unit depending on the characterisation method, for all impact categories to be represented within an environmental profile *diagram (Jonkers, 2020).*

Optionally, ISO 14040 indicates possible normalization and weighing of the environmental impact category scores, or aggregation of several categories within thematic groups. Additionally, unit equivalents may be converted into monetary value, or shadow costs, to obtain the Environmental Cost Indicator (ECI) as described in the Dutch 'Bouwbesluit 2012 - Bepalingsmethode'. Specific scores are calculated for each category, and they are aggregated within a single value to get the final environmental impact score of each product. The overall footprint of the building is obtained by summing the scores of all individual building products. It is also possible to sum the scores of individual environmental impact categories to draw the environmental profile of the building.

F.1.7 Life cycle interpretation

The last step of the LCA procedure involves a discussion of the results from the LCI and LCIA steps, as well as a possible sensitivity analysis and identifying the limitations of the chosen methods. Conclusions and recommendations should be drawn for further studies (Jonkers, 2020).

F.1.8 End-of-life considerations

Stage D of a building's life cycle is treated differently depending on regulations, due to the uncertainty on the potential reuse or recycling of elements in the future. According to European standards, residual value of elements after the end of service life of a building cannot be considered as bonus points in LCA environmental impact calculations. However, national standards like the Dutch 'Bouwbesluit 2012 - Bepalingsmethode' may allocate bonuses for such reusable or recyclable materials (Jonkers, 2020).

G

Environmental data

Environmental impact data is retrieved from EPDs, using the OneClick LCA online tool. Environmental profiles are classified on a scale from “very low” to “very high”, as indicated in Table G.1.

Table G.1: Environmental profiles from OneClick LCA tool

OneClick LCA icon	CO_2	CO_2	CO_2	CO_2	CO_2
Environmental profile	Very low	Low	Average	High	Very high

The relevant EPDs selected for the environmental impact calculations of step 1 (concrete, steel and timber warehouse structures) and the additional data required for design variants in step 2 (building envelope materials) are described in the following tables. Given the geographical scope of the project, only European data is considered.

- **Reinforcing steel:** It is possible to find generic data in the OneClickLCA database for rebars made only from virgin materials, with 0% recycled content (GWP_{fossil} = 2.865 kgCO2e/kg). However, the choice is made for reinforcing steel EPDs to focus on materials currently available on the European market, mostly produced from recycled scrap, therefore having a much lower impact but enabling more representative results.
- **Concrete C50/60:** The impact of high strength concrete mixes is taken from generic data found in the database, representative of materials containing only Portland cement, compared to sustainable alternative using GGBS.
- **Wood I-joists, stainless steel:** For these products, only one suitable EPD was found in the database, therefore only one value is used instead of lower and higher boundaries of the GWP.

Table G.2: Characteristics of EPDs selected from OneClick LCA online tool, representative of the lowest and highest impact building materials and products available on the European market (incl. fossil and biogenic GWP, for A1-A3 LCA modules)

IHULT steel profiles		
EPD owner	**ArcelorMittal Europe**	**SULB Company B.S.C.**
Type of material	Structural steel sections and merchant bars, 7850kg/m^3, Xcarb	Hot rolled structural steel, 7850kg/m^3, I and H beams, parallel flange channels, universal beams and columns
Declared unit	1 ton	1 ton
Environmental profile	GWP_{fossil} = 333 kgCO2e/ton	GWP_{fossil} = 2670 kgCO2e/ton
Material composition	100% recycled material, Electric Arc Furnace	0% recycled material, Direct reduction of iron (DRI), electric arc furnace (EAF), ladle furnace (LF), continuous casting (CCM), and hot rolling
End of validity	18/07/2026	24/04/2023
Production site	Differdange, Belval and Rodange (Luxembourg); Hunedoara (Romania); Bergara & Olaberria (Spain)	Al Hidd (Bahrein)

Hollow steel profiles		
EPD owner	**ArcelorMittal**	**Norsk Stal**
Type of material	Structural hollow steel sections, 7850kg/m^3	Hot-formed hollow steel profiles, 7850kg/m^3, 3-250mm
Declared unit	1 ton	1kg
Environmental profile	GWP_{fossil} = 2270 kgCO2e/ton	GWP_{fossil} = 2.82 kgCO2e/kg
Material composition	36.2% secondary material	100% steel Varmformet hulprofil
End of validity	28/04/2025	09/11/2025
Production site	Lexy (FR), Iasi (RO), Krakow (PL), Karvina (CZ), Rettel (FR), Roman (RO)	Norwegian plants in Søgne, Larvik, Horten, Stavanger, Klepp, Brumunddal, Bergen, Trondheim and Harstad

Reinforcing steel		
EPD owner	**ArcelorMittal Europe**	**Rotherham Steel & Bar**
Type of material	Reinforcing steel (rebar), 7850kg/m^3, XCarb	Carbon steel reinforcing bar (rebar) (secondary production route – scrap), 7850kg/m^3
Declared unit	1 ton	1 ton
Environmental profile	GWP_{fossil} = 300 kgCO2e/ton	GWP_{fossil} = 1180 kgCO2e/kg
Material composition	105% secondary material	97% Iron, 3% C, Mn, Si, V, Ni, Cu, Cr, Mo and others
End of validity	28/10/2026	09/12/2023
Production site	Warsaw (Poland)	Rotherham (United Kingdom)

Concrete C30/37		
EPD owner	**FED Beton**	**Thomas Betong AB**
Type of material	Typical Belgian Ready-Mixed Concrete	Ready-mix concrete
Declared unit	1 m^3 of ready-mixed concrete of strength class C30/37	1 m^3 of ready-mixed concrete of strength class C30/37 (B30 M60) WCR 0.55, exposure class XC4, CEM I 42,5 N - SR 3 MH/LA
Environmental profile	GWP_{fossil} = 201.95 kgCO2e/m^3	GWP_{fossil} = 318.0 kgCO2e/m^3
Material composition	Only new materials are uses as input: 13% cement, 35% sand, 44% gravel, 7% water, <1% admixtures	Cement 14,5%, aggregates 77,5%, water 7,9%, superplasticizer 0,1%
End of validity	20/12/2024	22/02/2024
Production site	123 concrete plants in Belgium, 55 manufacturers of FED-BETON	Sweden, 35 factories

Concrete C50/60		
EPD owner	**OneClickLCA**	**OneClickLCA**
Type of material	Ready-mix concrete, high strength, generic, C50/60 with CEM III/A, 60% GGBS content in cement (430 kg/m^3 total cement)	Ready-mix concrete, high strength, generic, C50/60 with CEM I (430 kg/m^3 total cement)
Declared unit	1m^3	1m^3
Environmental profile	GWP_{fossil} = 233.0 kgCO2e/m^3	GWP_{fossil} = 429.0 kgCO2e/m^3
Material composition	C50/60 with CEM III/A, 60% GGBS content in cement	C50/60 with CEM I
End of validity	-	-
Production site	-	-

Glulam		
EPD owner	**Holmen Wood Products AB**	**Studiengemeinschaft Holzleimbau e.V.**
Type of material	Glued laminated timber (Glulam), 468 kg/m^3, moisture content 12%	Glued laminated timber (GLT), 483.21 kg/m^3, moisture content 12%
Declared unit	1m^3	1m^3
Environmental profile	GWP_{fossil} = 33 kgCO2e/m^3 GWP_{bio} = -758 kgCO2e/m^3 GWP_{total} = -726 kgCO2e/m^3	GWP_{fossil} = 158 kgCO2e/m^3 GWP_{bio} = -773 kgCO2e/m^3 GWP_{total} = -615 kgCO2e/m^3
Material composition	Spruce	Spruce, Pine, Larch, Fir 0.03% PUR, 2.04% MUF, 0.1% PRF
End of validity	26/04/2026	12/08/2023
Production site	Nordic countries	Germany

Kingspan Quadcore sandwich panels		
EPD owner	**Kingspan**	**Kingspan**
Type of material	Kingspan KS1000 Quadcore Topdek insulated panel	Kingspan KS1000RW
Declared unit	$1m^2$	$1m^2$
Environmental profile	CO_2 GWP_{fossil} = 43.70 $kgCO2e/m^2$	CO_2 GWP_{fossil} = 49.90 $kgCO2e/m^2$
Material composition	53% Internal Steel Liner, 30% Insulation, 17% External Membrane	QuadCore insulation panel
End of validity	05/03/2024	28/05/2025
Production site	Sherburn (United Kingdom)	France

PUR insulation sandwich panels		
EPD owner	**ISOPAN**	**Marcegaglia**
Type of material	ISOPAN wall isobox 0.5/0.4mm 120mm	Marcegaglia wall/roof panel 0.5/0.4mm 120mm
Declared unit	$1m^2$	$1m^2$
Environmental profile	CO_2 GWP_{fossil} = 36.43 $kgCO2e/m^2$	CO_2 GWP_{fossil} = 45.80 $kgCO2e/m^2$
Material composition	61.0% Steel, 39.0% Insulating core	60% Steel, 40% Insulating material
End of validity	26/03/2024	13/12/2026
Production site	Frosinone and Verona (Italy), Tarragona (Spain), Bucharest (Romania), Halle (Germany), Volgograd (Russia), Guanajuato (Mexico)	Pozzolo Formigaro (Italy)

Mineral wool insulation sandwich panels		
EPD owner	**ACH**	**ISOPAN**
Type of material	ACH Panel sandwich with mineral wool core	ISOPAN roof isofire 0.5/0.5mm 200mm
Declared unit	$1m^2$	$1m^2$
Environmental profile	GWP_{fossil} = 48.93 $kgCO2e/m^2$	GWP_{fossil} = 57.30 $kgCO2e/m^2$
Material composition	Not precised	59.4% Steel, 40.6% Insulating core
End of validity	18/08/2024	26/03/2024
Production site	Alovera (Spain)	Frosinone and Verona (Italy), Tarragona (Spain), Bucharest (Romania), Halle (Germany), Volgograd (Russia), Guanajuato (Mexico)

EPDM layer		
EPD owner	**Firestone Building Products**	**OKOBAUDAT**
Type of material	EPDM membrane	EPDM roof sheets, EPDM, 2.0 kg/m2
Declared unit	$1m^2$	$1m^2$
Environmental profile	GWP_{fossil} = 4.14 $kgCO2e/m^2$	GWP_{fossil} = 8.60 $kgCO2e/m^2$
Material composition	34.0% Base resin, 30.0% Carbon black, 20.5% Plasticizer (paraffinic oil), 11.5% White filler (aluminium silicate), 1.5% Activators (zinc stearate), 1.5% Vulcanization system sulphur based crosslinking), 1.0% Processing aids (stearic acid)	Not precised
End of validity	12/12/2023	2022
Production site	Terrassa (Spain)	Germany

OSB panels		
EPD owner	**MEDITE SMARTPLY**	**SWISS KRONO TEX GmbH & Co**
Type of material	MEDITE Smartply OSB2 (600kg/m^2)	SWISS KRONO OSB-panel
Declared unit	1 ton	1m^3
Environmental profile	CO2 GWP_{fossil} = 282 kgCO2e/ton GWP_{bio} = -1700 kgCO2e/ton GWP_{total} = -1420 kgCO2e/ton	CO2 GWP_{fossil} = 855.47 kgCO2e/m^3 GWP_{bio} = -1745.47 kgCO2e/m^3 GWP_{total} = -890.0 kgCO2e/m^3
Material composition	91.97% Softwood pulp wood logs (at 0% moisture), 2.65% MDI, 0.59% MPU, 0.1% Wax (paraffin), 0.57% Release agent, 4.11% Moisture (ambient conditions)	90% Round fresh wood, 2 - 4% Glue with basis polymers diphenylmethanediisocyanate (PMDI-Leim), 4 - 8% Moisture content, < 1% Wax emulsion
End of validity	11/11/2024	14/06/2026
Production site	Belview, Waterford, Slieverue (United Kingdom)	Heiligengrabe (Germany), Vasarosnameny (Hungary), Sully Sur Loire (France), Zary (Poland)

Steel sheets		
EPD owner	**OneClickLCA**	**SAB-profiel, Tata Steel Europe**
Type of material	Hot-dip galvanized steel sheets, Steel thickness range: 0.4-3.0 mm, zinc coating: 20µm (0.28kg/m^2 sheet steel)	Weathering prefinished steel profile, 0.75 mm sheet thickness, 1035 mm width, 7.11 kg/m^2, SAB 35/10135 wall profile
Declared unit	1m^2	1m^2
Environmental profile	CO2 GWP_{fossil} = 4.14 kgCO2e/m^2	CO2 GWP_{fossil} = 23.30 kgCO2e/m^2
Material composition	100% recycled content	100% pre-finished steel
End of validity	-	22/03/2025
Production site		IJsselstein and IJmuiden (Netherlands), Port Talbot, Llanwern and Shotton (United Kingdom)

Wood fibre insulation		
EPD owner	**Hunton**	**STEICO**
Type of material	Wood fiber insulation plate, Lambda = 0.038 W/mK, 50 kg/m^3, Lambda=0.038 W/(m.K)	Wood fibre insulation board, from dry process, L = 0.040 W/mK, 40-200 mm, 140 kg/m^3, Lambda=0.04 W/(m.K), STEICOspecial dry
Declared unit	1m^3	1m^3
Environmental profile	CO_2 GWP_{fossil} = 14.89 kgCO2e/m^3 GWP_{bio} = -77.53 kgCO2e/m^3 GWP_{total} = -62.63 kgCO2e/m^3	CO_2 GWP_{fossil} = 154.83 kgCO2e/m^3 GWP_{bio} = -236.40 kgCO2e/m^3 GWP_{total} = -81.57 kgCO2e/m^3
Material composition	81.2% wood fibre, 8.0% water, 8.1% ammonium phosphate, 2.7% polyolefin fiber	86% wood, 5% water, 5% glue, 4% hydrophobic agents
End of validity	06/07/2025	19/11/2025
Production site	Norway	Germany

Wood I-joists	
EPD owner	STEICO
Type of material	STEICOjoist
Declared unit	1m
Environmental profile	CO_2 GWP_{fossil} = 4.25 kgCO2e/mGWP_{bio} = -7.07 kgCO2e/mGWP_{total} = -2.82 kgCO2e/m
Material composition	87.44% coniferous wood (spruce and/or pine) 4.5% PF adhesive, 0.03% MUF adhesive, 0.03% hot-melt adhesive, 8% water
End of validity	12/11/2024
Production site	Czamkow (Poland)

Green facade	
EPD owner	Outokumpu Oyj
Type of material	Stainless Steel Long Product
Declared unit	1 ton
Environmental profile	CO_2 GWP_{fossil} = 2890 kgCO2e/ton
Material composition	10.5% to 30% Chromium, 38% max Nickel, 11% max Molybdenum, 1.2% max Carbon, >50% Iron
End of validity	18/09/2024
Production site	Sheffield (United Kingdom), Richburg (USA), Fagersta (Sweden)

Green roof		
EPD owner	**VegTech**	**Knauf Insulation**
Type of material	Urbanscape Green Roof System	
Declared unit	$1/m^2$	$1/m^2$
Environmental profile	CO_2 GWP_{fossil} = 7.73 $kgCO2e/m^2$	CO_2 GWP_{fossil} = 13.7 $kgCO2e/m^2$
Material composition	Drainage layer (100% recyclable polypropylene), Green roll (growing media of binder-free rock mineral wool: diabase, dolomite and briquettes of recycled mineral wool), Sedum blanket (12 different species)	Plastic trays, substrate and plants – at least 8-12 species of sedum
End of validity	20/04/2027	05/01/2026
Production site	Škofja Loka (Slovenia), Italy	Poland, Denmark, Europe

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Environmental impact results in design steps 1 and 2

This appendix details the numerical environmental impact results calculated in each step of this research. For each design alternative, the total GWP_{fossil}, $GWP_{fossil+bio}$ of the warehouse are given, specifying the mean value, variations due to EPDs lowest and highest data, and the % reduction of the design variant compared to the steel/concrete reference, and compared to the baseline timber design. Additionally, the GWP_{fossil}, $GWP_{fossil+bio}$ of each category of building elements are given, along with their share (%) in the total warehouse GWP_{fossil}.

H.1 Design step 1

Table H.1: Reference steel and concrete designs vs timber baseline A and B - Mean GWP of the warehouse and deviation from EPDs lowest and highest datasets, % reduction of the GWP compared to the reference design

Design options	**Mean**		**Deviation**		**% reduction ref**	
	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$
Concrete reference	1.50E+07	1.50E+07	4.61E+06	4.61E+06		
Timber baseline B	1.02E+07	5.28E+06	3.03E+06	3.74E+06	32%	65%
Steel reference	5.77E+06	5.77E+06	2.44E+06	2.44E+06		
Timber baseline A	4.00E+06	1.91E+06	1.11E+06	1.12E+06	31%	67%

Table H.2: Reference steel and concrete designs vs timber baseline A and B - GWP_{fossil} of each element category

Design options	**Frame**	**Floor slab**	**Foundation pads**	**Envelope**	**Others**	**TOTAL**
Concrete reference	3.62E+06	3.62E+06	3.59E+06	3.76E+06	4.42E+05	1.50E+07
Timber baseline B	7.13E+05	3.62E+06	2.12E+06	3.76E+06		1.02E+07
Steel reference	1.91E+06	1.61E+06	4.85E+05	1.64E+06	1.21E+05	5.77E+06
Timber baseline A	2.66E+05	1.61E+06	4.85E+05	1.64E+06		4.00E+06

Table H.3: Reference steel and concrete designs vs timber baseline A and B - $GWP_{fossil+bio}$ of each element category

Design options	Frame	Floor slab	Foundation pads	Envelope	Others	TOTAL
Concrete reference	3.62E+06	3.62E+06	3.59E+06	3.76E+06	4.42E+05	1.50E+07
Timber baseline B	-4.21E+06	3.62E+06	2.12E+06	3.76E+06		5.28E+06
Steel reference	1.91E+06	1.61E+06	4.85E+05	1.64E+06	1.21E+05	5.77E+06
Timber baseline A	-1.82E+06	1.61E+06	4.85E+05	1.64E+06		1.91E+06

Table H.4: Reference steel and concrete designs vs timber baseline A and B - % share of each element category in the total GWP_{fossil} of the warehouse

Design options	Frame	Floor slab	Foundation pads	Envelope	Others
Concrete reference	24.1%	24.1%	23.9%	25.0%	2.9%
Timber baseline B	7%	35%	21%	37%	
Steel reference	33%	28%	8%	28%	2%
Timber baseline A	7%	40%	12%	41%	

H.2 Design step 2

H.2.1 Strategy A - Floor slab

Table H.5: Floor slab thickness variants - Mean GWP of the warehouse and deviation from EPDs lowest and highest datasets, % reduction of the GWP compared to the steel reference design, and compared to timber baseline A

Design options	Mean		Deviation		% reduction ref		% reduction baseline	
	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$
150mm	3.60E+06	1.51E+06	9.83E+05	9.97E+05	38%	74%	10%	21%
175mm	3.80E+06	1.71E+06	1.05E+06	1.06E+06	34%	70%	5%	11%
200mm (baseline)	4.00E+06	1.91E+06	1.11E+06	1.12E+06				
225mm	4.20E+06	2.11E+06	1.17E+06	1.19E+06	27%	63%	-5%	-11%
250mm	4.40E+06	2.31E+06	1.24E+06	1.25E+06	24%	60%	-10%	-21%
Combined biobased + 150mm	2.84E+06	-1.73E+06	1.40E+06	1.02E+06	51%	130%	29%	190%

Table H.6: Floor slab thickness variants - GWP_{fossil} of each element category

Design options	Frame	Floor slab	Foundation pads	Envelope	Others	TOTAL
150mm	2.66E+05	1.21E+06	4.85E+05	1.64E+06		3.60E+06
175mm	2.66E+05	1.41E+06	4.85E+05	1.64E+06		3.80E+06
200mm (baseline)	2.66E+05	1.61E+06	4.85E+05	1.64E+06		4.00E+06
225mm	2.66E+05	1.81E+06	4.85E+05	1.64E+06		4.20E+06
250mm	2.66E+05	2.01E+06	4.85E+05	1.64E+06		4.40E+06

Table H.7: Floor slab thickness variants - $GWP_{fossil+bio}$ of each element category

Design options	Frame	Floor slab	Foundation pads	Envelope	Others	TOTAL
150mm	-1.82E+06	1.21E+06	4.85E+05	1.64E+06		1.51E+06
175mm	-1.82E+06	1.41E+06	4.85E+05	1.64E+06		1.71E+06
200mm (baseline)	-1.82E+06	1.61E+06	4.85E+05	1.64E+06		1.91E+06
225mm	-1.82E+06	1.81E+06	4.85E+05	1.64E+06		2.11E+06
250mm	-1.82E+06	2.01E+06	4.85E+05	1.64E+06		2.31E+06

Table H.8: Floor slab thickness variants - % share of each element category in the total GWP_{fossil} of the warehouse

Design options	Frame	Floor slab	Foundation pads	Envelope	Others
150mm	7%	33%	13%	46%	
175mm	7%	37%	13%	43%	
200mm (baseline)	7%	40%	12%	41%	
225mm	6%	43%	12%	39%	
250mm	6%	46%	11%	37%	

H.2.2 Strategy B - Building envelope

Table H.9: Envelope variants - Mean GWP and deviation from EPDs lowest and highest datasets, % reduction of the GWP compared to the steel reference design, and compared to timber baseline A

Design options	Mean		Deviation		% reduction ref		% reduction baseline	
	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$
Timber baseline A	4.00E+06	1.91E+06	1.11E+06	1.12E+06	31%	67%		
PUR/Steel	3.87E+06	1.78E+06	1.20E+06	1.21E+06	33%	69%	3%	7%
MW/Steel	4.27E+06	2.07E+06	1.20E+06	1.22E+06	26%	64%	-7%	-8%
Kingspan/Timber	3.85E+06	1.44E+06	9.86E+05	1.00E+06	33%	75%	4%	25%
PUR/Timber	3.68E+06	1.22E+06	1.04E+06	1.06E+06	36%	79%	8%	36%
MW/Timber	4.06E+06	1.49E+06	1.03E+06	1.05E+06	30%	74%	-2%	22%
Biobased	3.24E+06	-1.32E+06	1.53E+06	1.14E+06	44%	123%	19%	169%

Table H.10: Envelope variants - GWP_{fossil} of each element category

Design options	Frame	Floor slab	Foundation pads	Envelope	Others	TOTAL
Timber baseline A	2.66E+05	1.61E+06	4.85E+05	1.64E+06		4.00E+06
PUR/Steel	2.66E+05	1.61E+06	4.85E+05	1.52E+06		3.87E+06
MW/Steel	2.80E+05	1.61E+06	4.85E+05	1.90E+06		4.27E+06
Kingspan/Timber	2.66E+05	1.61E+06	4.85E+05	1.49E+06		3.85E+06
PUR/Timber	2.66E+05	1.61E+06	4.85E+05	1.32E+06		3.68E+06
MW/Timber	2.80E+05	1.61E+06	4.85E+05	1.69E+06		4.06E+06
Biobased	2.80E+05	1.61E+06	4.85E+05	8.66E+05		3.24E+06

Table H.11: Envelope variants - $GWP_{fossil+bio}$ of each element category

Design options	Frame	Floor slab	Foundation pads	Envelope	Others	TOTAL
Timber baseline A	-1.82E+06	1.61E+06	4.85E+05	1.64E+06		1.91E+06
PUR/Steel	-1.82E+06	1.61E+06	4.85E+05	1.52E+06		1.78E+06
MW/Steel	-1.92E+06	1.61E+06	4.85E+05	1.90E+06		2.07E+06
Kingspan/Timber	-1.82E+06	1.61E+06	4.85E+05	1.17E+06		1.44E+06
PUR/Timber	-1.82E+06	1.61E+06	4.85E+05	9.48E+05		1.22E+06
MW/Timber	-1.92E+06	1.61E+06	4.85E+05	1.32E+06		1.49E+06
Biobased	-1.92E+06	1.61E+06	4.85E+05	-1.50E+06		-1.32E+06

Table H.12: Envelope variants - % share of each element category in the total GWP_{fossil} of the warehouse

Design options	Frame	Floor slab	Foundation pads	Envelope	Others
Timber baseline A	7%	40%	12%	41%	
PUR/Steel	7%	41%	13%	39%	
MW/Steel	7%	38%	11%	44%	
Kingspan/Timber	7%	42%	13%	39%	
PUR/Timber	7%	44%	13%	36%	
MW/Timber	7%	40%	12%	42%	
Biobased	9%	50%	15%	27%	

H.2.3 Strategy C - Green envelope

Table H.13: Green envelope variants - Mean GWP and deviation from EPDs lowest and highest datasets, % reduction of the GWP compared to the steel reference design, and compared to timber baseline A

Design options	Mean		Deviation		% reduction ref		% reduction baseline	
	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$	GWP_{fossil}	$GWP_{fossil+bio}$
Timber baseline A	4.00E+06	1.91E+06	1.11E+06	1.12E+06				
Baseline + Green 50	4.09E+06	1.53E+06	1.16E+06	1.18E+06	29%	73%	-2%	20%
Baseline + Green 100	4.22E+06	1.28E+06	1.26E+06	1.28E+06	27%	78%	-6%	33%
Baseline + Green 150	4.29E+06	9.96E+05	1.30E+06	1.32E+06	26%	83%	-7%	48%
Biobased + Green 50	3.49E+06	-1.46E+06	1.75E+06	1.32E+06	39%	125%	13%	176%
Biobased + Green 100	3.54E+06	-1.85E+06	1.78E+06	1.36E+06	39%	132%	11%	197%
Biobased + Green 150	3.59E+06	-2.19E+06	1.81E+06	1.40E+06	38%	138%	10%	215%

Table H.14: Green envelope variants - GWP_{fossil} of each element category

Design options	Frame	Floor slab	Foundation pads	Envelope	Others	TOTAL
Timber baseline A	2.66E+05	1.61E+06	4.85E+05	1.64E+06		4.00E+06
Baseline + Green 50	3.25E+05	1.61E+06	4.85E+05	1.67E+06		4.09E+06
Baseline + Green 100	3.74E+05	1.61E+06	4.85E+05	1.76E+06		4.22E+06
Baseline + Green 150	4.19E+05	1.61E+06	4.85E+05	1.78E+06		4.29E+06
Biobased + Green 50	3.25E+05	1.61E+06	4.85E+05	1.07E+06		3.49E+06
Biobased + Green 100	3.74E+05	1.61E+06	4.85E+05	1.08E+06		3.54E+06
Biobased + Green 150	4.19E+05	1.61E+06	4.85E+05	1.08E+06		3.59E+06

Table H.15: Green envelope variants - $GWP_{fossil+bio}$ of each element category

Design options	Frame	Floor slab	Foundation pads	Envelope	Others	TOTAL
Timber baseline A	-1.82E+06	1.61E+06	4.85E+05	1.64E+06		1.91E+06
Baseline + Green 50	-2.23E+06	1.61E+06	4.85E+05	1.67E+06		1.53E+06
Baseline + Green 100	-2.56E+06	1.61E+06	4.85E+05	1.76E+06		1.28E+06
Baseline + Green 150	-2.87E+06	1.61E+06	4.85E+05	1.78E+06		9.96E+05
Biobased + Green 50	-2.23E+06	1.61E+06	4.85E+05	-1.32E+06		-1.46E+06
Biobased + Green 100	-2.56E+06	1.61E+06	4.85E+05	-1.38E+06		-1.85E+06
Biobased + Green 150	-2.87E+06	1.61E+06	4.85E+05	-1.41E+06		-2.19E+06

Table H.16: Green envelope variants - % share of each element category in the total GWP_{fossil} of the warehouse

Design options	Frame	Floor slab	Foundation pads	Envelope	Others
Timber baseline A	7%	40%	12%	41%	
Baseline + Green 50	8%	39%	12%	41%	
Baseline + Green 100	9%	38%	11%	42%	
Baseline + Green 150	10%	37%	11%	41%	
Biobased + Green 50	9%	46%	14%	31%	
Biobased + Green 100	11%	45%	14%	30%	
Biobased + Green 150	12%	45%	14%	30%	