ENVIRONMENTAL IMPACT OF THE STRUCTURAL SYSTEM OF HIGH-RISE BUILDINGS IN THE NETHERLANDS

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MSc Thesis report

ENVIRONMENTAL IMPACT OF THE STRUCTURAL SYSTEM OF HIGH-RISE BUILDINGS IN THE NETHERLANDS

A research of the influence of the structural systems of high-rise buildings on their environmental impact, by means of Life-Cycle Assessment.

by

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ABSTRACT

Among the current targets of the building sector, one major goal is to maximize the resource efficiency (in terms of material use and in terms of energy used to produce the buildings); furthermore, the circular economy model aims for maximizing the reuse of the structural elements; thus, minimizing the material waste and avoiding a second production of elements. The high-rise building sector faces the challenge of considering a design that is flexible and adaptable to meet the functional requirements along its life span; and additionally, to consider a design for deconstruction, where the benefits and challenges that arise form reusing and recycling the material can be considered and addressed from early design stages.

This research addresses these trends and challenges by evaluating and comparing the environmental impact of different structural systems for a high-rise building in the Netherlands (151m). The comparison of four different structural systems with two core variations provides eight different stability systems. The scope of the stability systems considers foundations, core, columns, beams, bracings and floor slabs. The design of the structural systems was performed by the elaboration of 3D FEM models with parametrical tools to ensure the structural safety and serviceability of the building. Furthermore, the assessment of the environmental impact (global warming potential) was performed by means of a Life-Cycle Aseessment comparing three scenarios of the structure: cradle to gate, cradle to cradle, and cradle to cradle with 100% reuse of the structural elements; with data from the Nationale Milieu Database and with information from a technical report from the Joint Research Center.

The analysis of the results demonstrated that the environmental impact of the structural systems with steel core is 21% higher than the one that corresponds to the structures with concrete core, However, this variation is only 4% for the two variants of the diagrid structure. Furthermore, by including the average recycle and reuse rates form the market and current construction practices, the benefits at the end-of-life stage of the building can represent up to 17% of the impacts from the production phase. Moreover, when the reuse rate is considered as 100% the benefits increase up to 42% of the impacts from the production phase.

The results indicate that the improvement of the environmental impact of high-rise buildings can be achieved by means of sustainable structural design from the early phases of the design; where the choice of materials and of the structural system play and important role on the outcome of the total environmental impact of the building, which is becoming an important driver for the decision-making of new projects.

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I INTRODUCTION

1. INTRODUCTION

1.1. MOTIVE

The building sector is changing its mindset to address the current challenges and opportunities created by the present trends (like environmental impacts) and economic models (like the circular model). However, even if this mindset is already occurring in some areas; in a few areas, like high-rise buildings, and more specifically their embodied energy, this is happening at a slower pace. The chosen stance should understand the environmental, social and economic impacts that are inherent to the building industry. Additionally, there should be awareness of how, by considering the inter-relation of these problems, they can be addressed in the first steps of the project by applying the principles of sustainable structural design.

The basis of circular economy should be considered to address every economic activity. However, the strict application of all the circular principles can sometimes result in projects with higher environmental impact. For this reason, there is not a single methodology that can assess and tackle all the problems that arise from improving the sustainability of the projects. The applied methodology should be the one that results in the better alternative to guarantee sustainability of each particular project.

In the case of high-rise structures, the purpose of the approach for sustainable design should be to maximize the efficiency of the building during its life cycle and to provide flexibility of the building to ensure a proper functionality during a long life span. Thus, an optimization of the structural elements and a minimization of the material use of the building are required. This optimization of material use does not mean only material weight, but also the embodied energy of the building and the global warming potential that results from the buildings. Said optimization can be achieved by using a proper structural design, built with materials which environmental impacts are lower than other options available.

Furthermore, the current construction trends also focus on the reuse of materials and structural elements. By applying the sustainable design approach of designing for deconstruction, there are measurable environmental benefits that arise from the reuse and recycle of structural elements and materials.

1.2. PROBLEM

The challenge that arises from applying this mindset into the high-rise building industry is to consider the design for flexible and durable buildings, while simultaneously considering the easiness for deconstruction for future reuse or recycle. In other words, there must be a balance between a very specialized design, that minimizes the environmental impacts from the production of the structural elements; and a standardized design, that maximizes the possibility of reusing the structural elements and thus, minimizes the material waste.

2. METHODOLOGICAL APPROACH

2.1. OBJECTIVE

The purpose of thesis is to compare the environmental impact of the structural systems of high-rise buildings in the Netherlands, by including different stability systems to satisfy the structural requirements and by including different materials to account for the reusability of the structural elements.

2.2. RESEARCH QUESTION

To address a small part of the many challenges that the Dutch construction industry faces while it aims for a sustainable and circular economy, the intention of this research is to study the influence of the sustainable structural design to provide more sustainable projects. More specifically, the main research question of this thesis is:

HOW TO DESIGN SUSTAINABLE HIGH-RISE BUILDINGS (150M) IN THE NETHERLAND, AND HOW DOES THE INCLUSION OF RE-USABLE STRUCTURAL ELEMENTS INFLUENCE THEIR ENVIRONMENTAL IMPACT?

The following sub-questions are addressed to provide information, data and conclusions; which, conjointly, answer the main research question.

SQ1. Which is the most suitable strategy to address and evaluate sustainable structural design of high-rise buildings in the Netherlands?

This first question is answered by means of presenting literature review and a conclusion from the information reviewed. The goal of this question can be divided into three parts:

- Goal of the Dutch government to achieve sustainability in the construction sector.
- Trends of the construction of tall buildings in the Netherlands.
- Methods to evaluate and strategies to address the sustainable structural design of a building.

SQ2. What is the best way to reduce the environmental impact of the structural system of a high-rise building?

This question is also addressed by presenting a literature review. Additionally, a set of structural models (spatial truss, frame, and shell analysis) were elaborated to assess the mechanical and structural requirements of the building. Afterwards, this set of models were assessed by means of a Life Cycle Analysis to evaluate the environmental impact of each of the models. This question can be subdivided into four parts:

- Factors that directly affect the environmental impact of a high-rise building.
- Influence of the choice of a structural stability system on the quantity of material use; and consequently, the environmental impact of a high-rise building.
- Influence on the environmental impact of high-rise buildings, by the choice of material of the structural elements (namely, concrete or steel core).
- Contribution of the substructure to the environmental footprint of a building. And, its relation to the superstructure of the building.

SQ3. What are the benefits of considering reusable structural elements in this method of sustainable design evaluation?

This question is addressed by means of literature review and, similarly to the previous sub-question, a Life Cycle Analysis was performed on the set of structural models studied. This question focuses on the results of including reusable structural elements. This question can be divided into the following parts:

- Designing for adaptability and designing for deconstruction.
- Measuring of the benefits of reuse and recycling (Module D – EN15804).
- Environmental impact differences of the different structural systems analysed by including the benefits of reuse

2.3. SCOPE

This thesis evaluates a comparison of the environmental impact of different stability systems of high-rise buildings in the Netherlands. The structural systems addressed are Moment Resistant Frame, Outrigger, Megaframe and Diagrid, made of steel elements; with two core variables per stability system: concrete core and steel braced core. The systems are composed of foundations, core, floor slabs and main load bearing structural system. To ensure structural safety, the design and analysis of the structural systems were performed by means of a 3D FEM structural model.

Moreover, the environmental impact of the stability systems analysed is addressed by means of a Life-Cycle Analysis performed at three levels: Cradle to Gate, Cradle to Cradle, and Cradle to Cradle with 100% of reuse of the structural elements.

2.4. METHODOLOGY

To address the research questions, this thesis consists of: a literature review, which studies the current practices and state-of-art technologies used for the structural and sustainable design; a structural and sustainable assessment, which is composed by the models used to derive the results; and an analysis of the assessments and the literature, to draw the conclusion from the thesis.

Since the scope of the research considers a wide number of solutions to address the objective, the assessment consists of a multidisciplinary evaluation between the fields of structural engineering and sustainable design.

First, the Structural assessment consists on the evaluation of multiple alternatives that satisfy the structural requirements of a high-rise building. This analysis was elaborated by means of parametric tools and structural models to explore different designs during early design stages. Moreover, the models were built based on considerations, simplifications and state-of-the-art technologies to obtain a precise and reliable model. Furthermore, during the modelling stage, the models used were verified by means of additional FEM models and rules of thumb, to ensure the reliability and precision of the models. Finally, the structural models were executed to define an optimized structure (superstructure and substructure) by means of parametrical design and interconnecting the optimum design of the two components.

Secondly, the Sustainability assessment consists of the environmental impact analysis of the multiple alternatives designed and studied on the Structural assessment. This analysis was performed by means of a Life-Cycle Assessment of every stage on the life cycle of the building. The LCA was elaborated with database from state-of-art methodologies and current practices of the construction industry. Moreover, due to the small quantity and the large variability of the available information, this data was analysed and adapted to satisfy the requirements of the research. Lastly, the assessment was performed by addressing the production phase and the cradle to cradle cycle of the structure; and the LCAs were verified by means of comparison with literature and by using different sets of databases.

Finally, the research questions are answered by means of the background from the literature review and from the analysis of the assessments.

2.5. THESIS STRUCTURE

This research is divided in three main sections.

First, the Literature Review section, in which the current trends, state-of-art and current practices for sustainable design of high-rise buildings are addressed and defined; and followingly, the current technologies, requirements, components and state-of-art tools for structural design of high-rise buildings are also addressed and described.

Secondly, the Assessment section, in which the two evaluations performed in this research (Structural analysis and Environmental impact analysis) are outlined by: first, defining the models and the assessment; then, by presenting the results obtained from the analysis; and lastly, by discussing the results and the assessment.

And finally, the Conclusion section, in which the conclusion from the research are drawn; followed by an acknowledgement of the limitations of this research and by a series of recommendations for further research.

II LITERATURE REVIEW

3. SUSTAINABLE DESIGN OF HIGH-RISE BUILDING

This chapter provides a summary of the literature review and of the state-of-art in sustainability material in general, and for the high-rise building sector in particular. First, general concepts and background information is described. Then, the concept of sustainable structural design is introduced and described, followed by the definition of the life-cycle analysis and how the concepts of recyclability and reusability fit within this assessment. And finally, the environmental impact of high-rise buildings is presented by comparing three different studies.

3.1. SUSTAINABILITY AND CIRCULARITY

Nowadays, the world is quickly changing and evolving. Human activities reshape the planet by using its resources and transforming them into every day's products all around the globe. This current, rapidly changing and globalized world comes with inherent global trends, such as: Population Growth, Urbanization and Environmental Impacts [1,2]. To address the conflicts generated by the mentioned global trends, the European Commission has set the goal to follow a transition to a low-carbon, climate-neutral, resource-efficient and biodiverse economy. Therefore, it is of paramount importance to reduce the dependency on non-renewable resources, and to maximize the use of sustainably managed renewable resources [3].

3.1.1. SUSTAINABILITY

The Brundlandt Commission has defined sustainable development as: "development that meets the needs of current generations without compromising the ability of future generations to meet theirs" [3]. Such development aims to improve the quality of life, allowing people to live in a healthy environment, and to improve the social, economic and environmental conditions for present and future generations.

3.1.2. CIRCULARITY

The governments around the world are adopting a model of circular economy to meet the goals defined within a sustainable framework. Before this model was proposed, the economy was based in a linear model. Which means that the materials used to make products are thrown to waste after their use. When the materials are wasted, unnecessary problems arise, like loss of the products' value, sources scarcity, pollution of the environment and climate impact, to name a few. On the other hand, in a circular economy, the products are designed to be more efficient and reusable, which prevents waste and reduces the need for new raw material [3].

3.1.3. CONSTRUCTION IN A CIRCULAR ECONOMY

To address the vision and goals of the European Economy transition to a more sustainable economy, the Construction Sector in the Netherlands has the objective of developing high-quality use and reuse of materials in a circular economy [4].

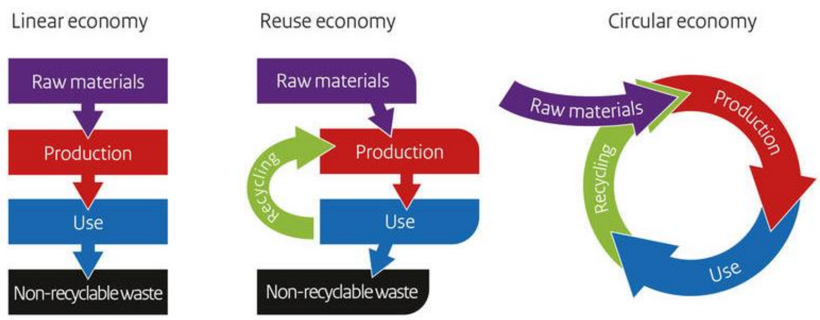


Figure 1: From linear to Circular Economy.
Source: https://www.government.nl [5]

Due to the long life of structures, circularity may seem less relevant for the construction sector. However, the construction industry must work within the boundaries of the circular economic model as well [4]. For this reason, current buildings and projects must be designed in a way that allow future generations to meet their needs. In other words, think ahead of the possible future problems.

In the same way that other industries are implementing a circular model, the construction industry must fulfil certain requirements to fit in this model. The constructions must then be designed for reusability, or at least be able to recycle the materials in an easy and efficient way to reduce waste. The constructions must be designed in a way to optimize the system performance. This is an optimization of resources, environmental impact and compatibility within the circular framework.

In the past years there have been important developments of innovative sustainable tall buildings theories, additionally, the research to improve the sustainability of buildings is increasing [6]. However, within the construction sector, the high-rise industry has responded slower to this mindset [7].

3.2. TALL BUILDINGS STATISTICS AND FORECAST

There is a common belief that high-rise buildings will be standing forever and that it is highly unlikely that the structures will ever be demolished due to their size and due to their iconic and economic value in a city. If the number of demolished buildings is compared with the number of buildings constructed, the former value is very small and does not represent a substantial percentage of the total number of buildings constructed [8]. However, the trend of the number of demolished buildings is increasing and, with an also growing demand of buildings construction, this trend will most likely continue to grow. The main reason behind most of the demolitions, is to make way for new and more modern buildings [8], where out of date or inefficient structures were located.

In the Netherlands these trends and profiles change. According to the statistics from the Global Tall Building Database of the Council on Tall Buildings and Urban Habitat [8], there has never been a demolition of a building higher than 100m. As a matter of fact, the number of buildings higher than 100m is relatively small (48 in total) with the highest building being the Maastoren, in Rotterdam, with a height of 165m. However, the tall building industry is growing in the

Netherlands. The amount of buildings under construction is significant if it is considered that the total number of buildings higher than 100 will increase 17% in the three upcoming years. Moreover, there are already 48 new buildings proposed or envisioned with a height of more than 100m

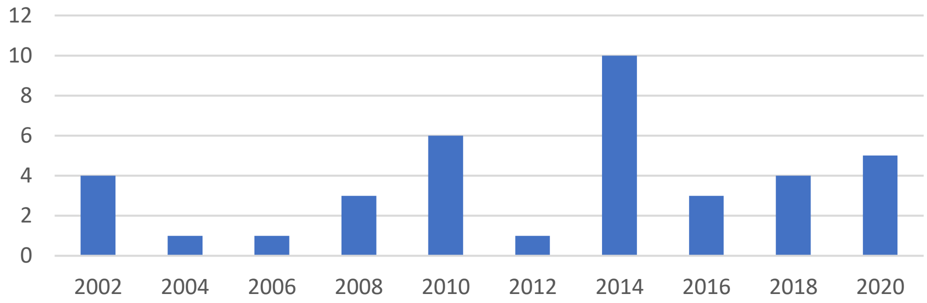


Figure 2: Number of high-rise buildings Voluntarily Demolished worldwide in the last 20 years. Source: from statistics taken from CTBUH [8]

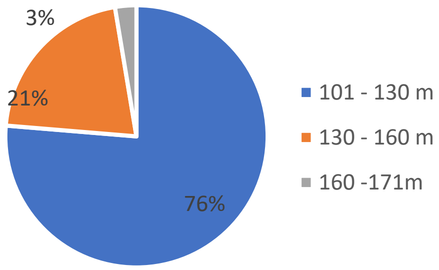


Figure 3:Percentage of Voluntarily Demolished Buildings worldwide, grouped by height. Source: frm statistics taken from CTBUH [8]

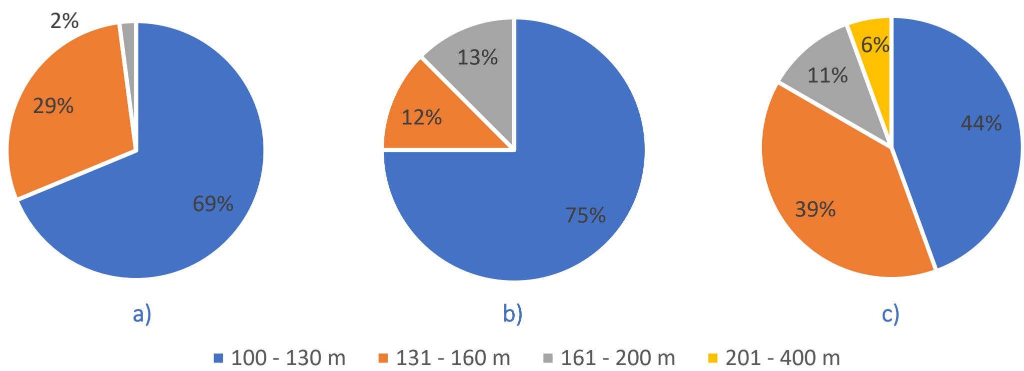


Figure 4: Building Statistics in the Netherlands.
a) Buildings Constructed by height. b) Buildings under Construction by height.
c) Buildings Proposed and Envisioned by height. Source: from statistics taken from CTBUH [8]

3.3. SUSTAINABLE STRUCTURAL DESIGN

A large amount of literature has investigated the energy consumption and CO2 emissions related to the building sector in general and most particularly to the building use phase. However, there is much less research addressing the impacts related mainly to the structural systems including waste production and raw-material use [4, 7]. As there are more technological developments over time, the relative environmental impact due to the material use becomes more relevant for new buildings, since these have a finite source, whereas energy can be renewable.

3.3.1. SUSTAINABLE DESIGN GOALS

In addition to the environmental impact, structural design has significant social and economic impact. However, to address these impacts, there is no single methodology that is able to asses all the impacts together; there are different approaches with specifics goals. The objective of sustainable structural design research is to include all the possible impacts that the structures may induce in the decision-making process, to end up with a more complete and comprehensive analysis. This goal of increasing the scope of the analysis is positive, but it makes the design process more complex, especially when the consequences of a design choice are hardly predictable [9].

Furthermore, some of these impacts and their assessment goals clash when they are analysed. Alimoradi [10] highlighted this conflict by enumerating nine questions that expose the clash and sometimes contradictory nature of the different sustainability goals. For instance, the strategies that address a structure's initial impact might have negative effects on its end-of-life impact (i.e. if a structure material use is very optimized, it might not be easily reused in a following project). In other words, the more customized a structure is, the less adaptable it becomes. Another example is the use of low carbon instead of high carbon materials. Low carbon materials, like wood, may not be as durable as carbon intense materials, like steel. Therefore, a comprehensive analysis should be performed with respect to a structure's intended use and design life.

Nevertheless, these conflicts present an opportunity to improve the assessment rather than just expose the complex relationship between the strategies. The questions derived by Alimoradi widen the scope of the design, resulting in prioritized impacts to be addressed, a use of better strategies to be adopted and a use of relevant methods to assess whether these strategies are successful or not [9].

Due to the nature and requirements of each project around the world, the models to address the sustainability in the building sector are variable. Within these models, and regarding the ecological aspects of sustainability, three main goals can be named [11]:

1. Energy efficiency: This goal aims to use efficient energy during the use phase, and during the production and construction of the buildings.
2. Resource efficiency: The objectives of this goal are to reduce the use of primary raw materials and to reduce the waste generated after the use phase of the building. These two objectives can be achieved by considering and using recycled materials in manufacturing of the construction products, and by recovering and handling the materials in the end-of-life phase of the building, minimizing the waste.

3. Reduction of emissions: By considering the whole life cycle of a construction, the decision taken in the design can play an important role on the output of the noxious emissions over the life of the building (in all its life cycle stages).

3.3.2. DESIGN STRATEGIES

It is complex and sometimes contradictory to satisfy all these requirements at the same time. For this reason, different authors define strategic approaches to tackle these problems by specifying a main goal and implementing focused actions to arrive at a sustainable solution. Two sets of strategies are described in the following sections.

3.3.2.1. DESIGN METHODOLOGIES BY DANATZKO AND SEZEN

Danatzko and Sezen [12] presented five methodologies which focus on different objectives to achieve a sustainable structural design. The main goal of these methods is focused on the minimization of the environmental impact and on the conservation of resources. The five design strategies are listed below:

1. Minimizing material use: Minimize the required raw materials for a project can be achieved by efficiently combining materials with different structural properties or through shape optimization techniques.
2. Minimizing material production energy: The sustainability of this methodology relies on the selection and use of materials with the same structural properties but with relatively smaller amount of energy cost required for their production.
3. Minimizing Embodied energies: Determine and use design strategies that lower Embodied Energy and Embodied Carbon, associated with the operation and maintenance phase over the structure's life.
4. Life Cycle Analysis: The Life Cycle Analysis (LCA) is a tool employed to evaluate and quantify the sustainability of a project. This tool can be used to assess specific stages of the life cycle of the building in order to increase its sustainability. Nevertheless, this practice is a tool that can be used with other methodologies, and not a methodology as such.
5. Maximizing structural systems reuse: The method goal is to design structural members that can be recovered and reused by generating layouts that produce the least amount of waste once the life cycle of the project over. An important remark of this approach is that the lack of definition of intended future-use of the structures, may result in overdesigned members which results in a reduced sustainability of the structural system.

According to the authors, there is not a single design methodology that can address the complex issues of structural design. Additionally, they acknowledge that the proposed methods are subjective and depend on the project's limitations. Finally, they recommend to apply each of the design methodologies individually and to assess them for their sustainable qualities.

3.3.2.2. DESIGN STRATEGIES FOR ENHACED LIFE CYCLE PERFORMANCE

The European Commission of science and knowledge service, Joint Research Centre (JRC), addresses the sustainability of buildings by proposing two design strategies[13]. The main focus of these strategies is to improve the resource efficiency. However, due to the close relation and dependence of the different goals, the energy efficiency and the reduction of emissions goals are also tackled by implementing these strategies.

3.3.2.2.1. DESIGN FOR ADAPTABILITY AND FLEXIBILITY

Buildings are commonly designed for a life period of 50 years, according to the Eurocode 1990. However, with a proper maintenance and with the capacity to accommodate changes in technical and functional requirements, buildings can last longer than the design working life. For this reason, buildings should be able to adapt to the changes (functional or technical) that may arise due to the change of function within the longer life span of the building.

According to the 2018 report Model for Life Cycle Assessment (LCA) of buildings [13], some general recommendations to consider for the design of flexible and adaptable building are:

1. Maximize the internal space of the building to enable flexibility on the usable space.
2. Consider slender internal columns to maximize the internal net space.
3. Ensure that the system is designed for loads that account for future functions of the building.
4. Allow slight overdesign of columns and foundations to enable future extensions.
5. Avoid irreversible connections between structural elements to enable easy replacement of elements.
6. Connections should be easily accessible,
7. Give preference for the columns to support the loads, instead of the partition walls.
8. Allow easy access to building services and capability to accommodate changes such as duct sizes.

3.3.2.2.2. DESIGN FOR DECONSTRUCTION

Design for deconstruction is a strategy in design that considers the way the building will be disassembled since the design phase of the building. The goal of this practice is to maximize the potential for reuse and/or recycle of the elements that make up the building [11]. Some considerations that may provide advantages towards deconstruction are:

1. Prefabrication of structural components.
2. Use modular construction systems that enable easy assemble and disassembly of the building.
3. Consider reversible connections.

1. The properties of the materials used in deconstructable systems play an important role; like the durability of the materials, the avoided use of hazardous materials, and the use of a small number of different materials to maintain simplicity on the structure

3.3.2.3. PARAMETERS

Although the connection between the sustainable structural design strategies is clear, the strategies effectiveness is very complex. To end up with an accurate and relevant study, the parameters used for the evaluation should be quantitative and accountable[9]. Therefore, it should be well defined which parameters are addressed and how they are measured.

The most common parameters for optimization of structures are the total cost or the weight of a structure. These parameters have been efficiently used when the structural systems differ only in their structural layouts, [14]. Another parameter to compare the sustainability of a building is the Embodied Energy. A comparison of the embodied energy of different structures is only useful when the energy source is the same or an appropriate conversion parameter is used [15], [16]. Lastly, CO2 emissions are an alternative parameter to evaluate sustainable structural solutions. CO_2 emissions are directly related to Embodied Energy. However, they are also affected by other factors which depend on the structural material use and on its manufacturing process [9].

3.4. ENVIRONMENTAL IMPACT ASSESSMENT

One method to assess the sustainable structural design is by using a Life Cycle Analysis (LCA). The LCA is a standardized methodology to calculate the environmental burden of a product or service. The calculation is based on a system approach of the chain of production and consumption, analysing the input and the output of the total system. All the phases that conform the methodology are described in detail by the ISO 1404 and standardized in the EN15804. This methodology standard provides a structure to ensure that all Environmental Product Declarations are derived, verified and presented in a harmonised way[17].

In addition to the Classical LCA, Vogtlander [18] differentiates an LCA which is faster and less complex than the classical LCA; the Fast Track LCA. The steps followed to perform the analysis are the same in the both LCA's; however, the difference between the approaches lies on the depth of the analysis. First, a classical LCA elaborates deeply on the impacts of each of the elements and the process that conform a product, to evaluate the steps that can be improved on the process. In the other hand, the fast track LCA performs a not so thorough analysis but yields accurate values which are used to compare the differences between several project options.

The steps to perform an evaluation using the fast track approach, proposed by Vogtlander, are the following:

1- Define the scope and goal of the analysis
2- Define the system, functional unit and system boundaries.
3- Quantify the materials and use of energy in the system.
4- Perform the computation and analyse the data.
5- Interpretation of results and conclusions.

An important remark of the LCA is that the scope of this tool is to assess only the environmental impacts, not societal and economic impacts; additionally, the functionality or technical performances are not included in the assessment [9]. This stresses the fact that the models tend to get more complex when the scope is widened, and more goals are included in the analysis.

The LCA evaluates the environmental impact during the different stages of the life cycle of the building. According to the European Norm EN15804, the life cycle of a building can be divided in the following phases:

- Production Phase (A1-A3): It is composed of the resource extraction, transport of resources, the manufacturing process, and completion of the products at the factory gate. It is also known as Cradle to Gate (C2Gt).
- Construction phase (A4-A5): It consists of all processes necessary for the erection and completion of a building. It includes transportation from the factory gate to the construction site, on-site production, and assembly processes.

- Use Phase (B1-B7): It is also known as operational stage. It englobes the time when the building is being used. The possible refurbishment and conversion of the buildings are included in this stage.
- End-of-life Phase (C1-C4): It addresses the deconstruction of a building and how the disposal of the remaining materials are executed. At this stage, the remaining elements of the building can be classified as: Structural elements and buildings materials that:
    - can be reused. Cradle to Cradle principle (C2Cr)
    - can be recycled. Cradle to Cradle principle (C2Cr)
    - must be disposed of. Cradle to Grave principle (C2Gv)
- Reuse/Recycle phase (D): it considers the impacts generated after the end-of-life stage of a building (either C2Cr or C2Gv).

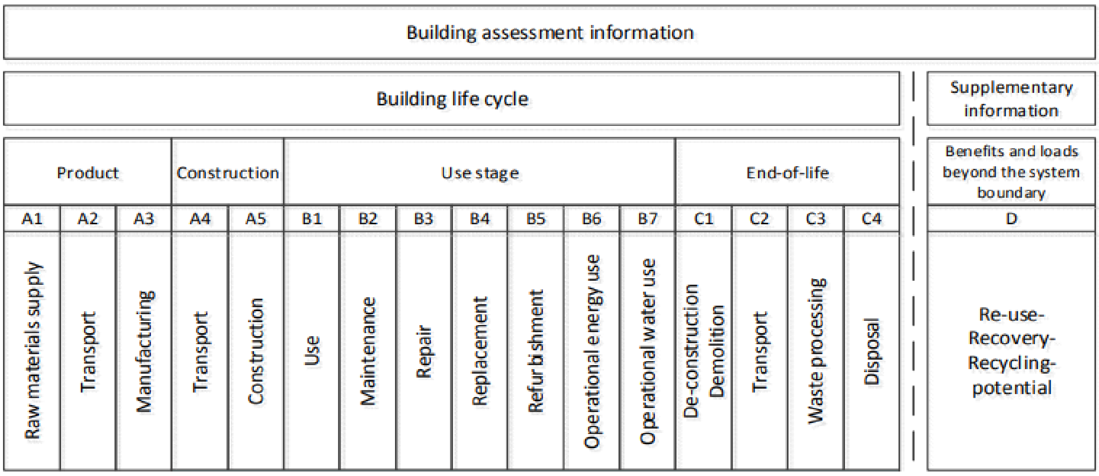


Figure 5: Building Life Cycle
SOURCE: Assessing the environmental impacts of construction [17]

As mentioned before, a large amount of literature focus on the energy consumption and CO2 emissions related to specifically the use phase (B1-B7) [7, 4]. There is much less research that addresses the impacts related to the Construction phase (A4-A5); and, not surprisingly, there is even less research which scope includes the end-of-life phase (C1-C4 and D). The lack of consistent data complicates the comparison of different projects in a reliable way.

Most of the literature that addresses the different stages of the life cycle of the building, study specific cases with very defined and locally influenced parameters. Moreover, most of these studies follow a different methodology. This lack of uniformity on the studies, results in very specific results which are not suitable to extrapolate to a more generic study. Thus, even if the data is valid and properly obtained, it does not guarantee that it is representative for more cases. It is therefore necessary to elaborate benchmarks to standardize the practice of LCA across different studies to allow data comparison and better correlation among the different studies [17, 18].

3.5. REUSABILITY AND RECYCLABILITY

One of the ecological aims of sustainable design, as already described before, is resource efficiency [11]. The two main practices to achieve efficiency of the materials used in construction is by optimizing the materials use and by reducing the amount of waste. The last one can be achieved by reusing and recycling the structural elements at the end of life stage of a building.

For an element to be reusable, its life cycle must outlive the life cycle of the whole building, and it should be able to fulfil the life cycle of a second building. In the other hand, once the element can no longer satisfy its intended first use, the element can be recycled or used with a different purpose, if the nature of the element allows it. Steel is a material that can meet these two activities. Steel recycling is a manufacturing process that has existed for years, the current average recycling rate is around 90%; moreover, the reuse of steel is estimated at around 10%. However, with the current technological developments and the growing awareness of the environmental needs, there is potential to develop its reusability and to increase the market of reused materials[11].

For reuse, further use, recycling and recovery of building components, it is essential to design for dismantling and recycling. Design for deconstruction goal is to increase resource and economic efficiency and to reduce environmental impacts in the adaptation and removal of buildings, and to recover components and materials for reuse and recycling[21].

3.5.1. RECYCLABILITY AND REUSABILITY ASSESSMENT

Within the Life Cycle framework, the module D considers the impacts generated after the end-of-life stage of a building, either by reuse or recycle of the building components. This module recognises the design for reuse, recycling and recovery concept for buildings, by indicating the potential benefits of avoiding future use of virgin materials and fuels in the production of primary products[11]. This module also considers the impacts that arise from the collection and process of the scrap after the deconstruction of the building.

To allocate the credits and burdens that arise from the recycling process between the primary system and the secondary system an allocation procedure is needed. To address the allocation credits, Gervasio and Dimova [13] summarize five different approaches that can be used depending on the scope and goals of the life cycle. These approaches address the different scenarios at the end-of-life of the material and how to include the credits and burdens from this stage. These methods are described in the following paragraphs.

- The 100:0 method allocates 100% of the benefits of using recycled materials in the production stage but neglects all the benefits of creating recycled materials at the end of life stage. This method is useful when data about the recycling of materials at the end-of-life stage is not available.
- The 0:100 method allocates 100% of the benefits of creating recycled materials at the end of life stage to the system under consideration, but it neglects all the benefits of using recycled materials in the production stage. This approach takes advantage of the use of materials with potential for reuse, recycling and recovering.
- The 50:50 method, also known as partition rule, is comprised between the 0:100 and the 100:0 approaches. This method allocates 50% of the benefits of using recycled materials in the production stage, and 50% of the benefits of creating recycled materials at the end-of-life stage, to the system under consideration.

- The PEF (Product Environmental Footprint) method was developed by the European Commission to measure environmental performance of a good or a service throughout its life cycle. It is based on the 50:50 approach. However, this method includes the differences in quality between the secondary and primary material (K) and the environmental burdens from disposal of waste where the recycled content is taken from.
- Module D method, according to EN15804, allocates to module D the net environmental benefits or loads due to recycling, reuse or energy recovery. In this approach, net impact refers has two meanings: 1) in relation to environmental impacts, and 2) in relation to mass. This method introduces a correlation factor to reflect the differences in the functional equivalence of the secondary and the primary materials (C_f). This value may be considered as the ratio between the price of the secondary material and the price of the primary material.

The adoption of an allocation approach must be consistent with the goals and scope of the life cycle study. These methods and their corresponding environmental profiles can be represented by means of the following expressions:

APPROACH	MODULES A1-A3	MODULES C1-C4	MODULE D
100:0	$[(1 - R_C)E_V + R_C * E_R]$	$[(1 - RR)E_D]$	-
50:50	$\left[\left(1 - \frac{R_C}{2}\right)E_V + \frac{R_C}{2} * E_R\right]$	$\left[\left(1 - \frac{RR}{2}\right)E_D\right]$	$\frac{RR}{2}(E_R^* - E_V^*)$
0:100	E_V	$[(1 - RR)E_D]$	$RR(E_R^* - E_V^*)$
PEF	$\left[\left(1 - \frac{R_C}{2}\right)E_V + \frac{R_C}{2} * E_R\right]$	$\left[\left(1 - \frac{RR}{2}\right)E_D - \frac{R_C}{2} * E_D^*\right]$	$\frac{RR}{2}(E_R^* - E_V^* * K)$
EN15804 Module D	$[(1 - R_C)E_V + R_C * E_R]$	$[(1 - RR)E_D]$	$(RR - R_C)(E_R^* - E_V^* * C_f)$

Table 1: Allocation approaches.
Adapted from JRC Report (2018) [13].

Where:

E_V	Environmental burdens from acquisition of virgin material.
E_R	Environmental burdens from the recycling process of the recycled material.
E_D	Environmental burdens from disposal of waste material.
E_R^*	Environmental burdens from recycling process.
E_V^*	Environmental burdens from the acquisition and pre-processing of virgin material assumed to be substituted by recycled materials.
E_D^*	Environmental burdens from disposal of waste material where the recycled content is taken from.
R_C	Recycled content of material.
RR	Recycling rate (or reuse) fraction of material at the end-of-life stage.
K	Ratio for differences in quality between secondary and primary material.
C_f	Differences in the functional equivalence of the secondary and primary materials.

3.6. ENVIRONMENTAL IMPACT OF HIGH-RISE BUILDINGS

As stated before, there are many studies that address the use phase of the buildings. While the use phase is being improved by using more efficient energies and more efficient materials, the environmental impacts attributed to this phase are reduced. This results in a more relevant environmental impact contribution to the whole life-cycle of the building due to the Material Production and Construction phases [7]. Figure 6 shows how, as there are more technological developments over time, the relative environmental impact due to the materials becomes more relevant for new buildings.

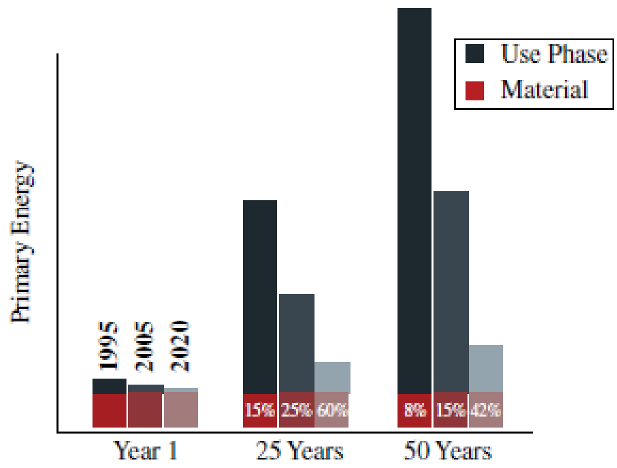


Figure 6: Primary Energy Used over the life cycle of a building.
Source: Sustainable Steel Buildings [11]

For this reason, studies that include the embodied energy content are gaining more attention. Many studies [4,12,14,20,21,22,23] show how an LCA can be used as a tool to assess the environmental impact of alternative building designs. Nevertheless, for the tall building community, this trend has a significant delay. Additionally, not all the research focus on the same environmental impacts, and they do not have the same considerations; this lack of consistent studies results in sporadic and inconsistent available data to elaborate accurate and reliable comparisons [7]. Finally, the lifecycle assessment as a discipline is very sensitive, and small decisions can lead to a significantly different impact on the final results[16].

According to Trabucco [7], the structural system plays a major role in the design of tall buildings. This reflects in the complexity of the building and its costs. By optimizing the structural system, the number of elements and material use can be reduced. Commonly, the cost market and the ease of construction are the decisive factors behind the optimization decisions; however, with and increased awareness of the environmental aspects around the building environment, the embodied energy and the embodied carbon of structures are becoming important drivers to consider during the design process.

During the past years, the number of papers related to sustainable Structural Design of High Rise buildings has been increasing; however, due to the inconsistency of the scope of the studies and, thus, the non-comparable data obtained from the studies, only a few of these studies will be discussed.

3.6.1. A Whole LCA of the Sustainable Aspects of Structural Systems in Tall Buildings [16]

Trabucco et al., conjointly with several industry leaders, conducted a research project on the life cycle assessment of different tall building structures. The research analysis included all life phases of a tall building's structural system (extraction and production of the materials, transportation, construction, demolition and end-of-life of the materials. The use phase was also considered, but the authors did not identify significant impacts during this phase). This analysis was performed by analysing two impact categories: Global Warming Potential and embodied Energy, of the most common structural system for buildings for a height of 250 and 490 meters. The study uses a building hypothetically located in Chicago, United States. For this study, the horizontal structures (beams, floor slabs, etc, represent up to 80% of the building weight for the structures of 60 stories.

Among the general conclusions of the study, the authors conclude that the analysis are very sensitive to small decisions and to the influence of local variables; for this reason they remark that their results are applicable only to the scope of their study and that the data should not be used to give conclusive results on different structural systems or materials in general.

Specific conclusions from this study are: In terms of Global Warming Potential, the buildings with concrete structural elements perform worse (in average) than other scenarios. On the other hand the buildings built with steel elements have a larger value of EE, for buildings with 60 stories. They remark that all the scenarios might additionally benefit from the recyclability of the steel at the end of the building's life if the system boundaries consider the Module D of the EN15978. Moreover, the results for the demolition and material waste are not very significant, with values ranging from 1 to 2.5% in terms of GWP, and 0.9 to 3.2% in terms of EE.

The environmental profile is directly influenced by the choice of the material suppliers. By including materials with lower GWP or EE (i.e. steel profiles with high content of recycled steel) the resultant buildings structures have a significantly lower environmental impact. Similarly, by using special designs and mixes of concrete, the embodied energy of the structure can be lowered.

3.6.2. Sustainable structural design of high rise [25]

This master thesis report written by Lankhorst (2018), addressed the influence of using different structural systems, with different materials (cast in situ concrete, precast concrete and steel) on the environmental impact of tall buildings in the Netherlands. For the environmental impact assessment, the study considered only the production (A1 to A3), by means of an LCA through the Fast Track method. Some of the results and conclusions, related to the present thesis, are listed in the following paragraphs:

First, from the models studied, the steel models score higher in terms of environmental impact (from 6% to 35% higher) than the concrete models with the same stability system.

The study also addresses the influence of different types of floor slabs over the total environmental impact of the building, he concluded that the floors are responsible for the largest contribution to the total impact of the structure. The contribution of the floors accounts from 40 to 73% for the concrete models, and from 32% to 69% for the steel models. For this study, it was concluded that by using hollow core slabs, instead of composite floors or flat slabs, the environmental impact was lower.

Furthermore, it was concluded that the stability systems that are loaded axially perform better than the systems loaded in bending and shear; i.e. systems like the Diagrid and the Outrigger perform better than the frame structures, which resulted in less material use, and thus, a lower environmental impact.

The diagrid shows the lowest scores in the environmental impact in both, concrete and steel, and for all the heights evaluated in the study.

3.6.3. Sustainable Structural design of tall buildings based on embodied energy [15]

Foraboschi et al. studied the energy required to construct tall buildings, expressed in terms of cradle-to-gate embodied energy. For the study, the scope included the use of only one structural system (reinforced concrete core) with rigid frames (made of either reinforced concrete or steel) of different heights. The different materials used resulted in different embodied energy profiles for the buildings considered. Some conclusions relevant to this study are the following:

Even if the Steel structures result in lighter buildings, the steel structures consume more embodied energy than reinforced concrete structures, due to the significatively higher embodied energy of steel.

Additionally, the results pointed out that most of the embodied energy of the building corresponds to the energy from the flooring systems. One interesting finding of this study is that by “improving” the floor structures with lightweight systems, the final embodied energy profile is higher than that of the concrete slab profile because the type of materials used to reduce the weight of the floors have a higher embodied energy than that of the material being removed.

From the referenced literature, and from many more available, it is recognized that, if the attention is directed to the carbon emissions and their impact on climate change, the results from manufacture and production of the materials are dependent on the efficiency of the production processes, moreover the type of energy used for the production can vary from country to country. Furthermore, the energy requirements during the use-phase are dependent on the climate conditions, which results in different percent of contribution due to this phase. Thus, the data obtained from the different literature reviews is subject to its own local conditions.

However, since this study aims to present a comparison between the different structural systems. The values taken from the literature are sufficient to provide a starting point for comparison. And, by disregarding the use phase from this research, the variations that may arise from different local conditions are avoided.

4. STRUCTURAL DESIGN OF HIGH-RISE BUILDINGS

This chapter describes the structural requirements and the different types of stability system used to address the structural design of high-rise buildings. Additionally, it presents a description of the foundations systems used in high-rise buildings and their influence on the structural design of the superstructure. Lastly, a description of the structural models used for the analysis of high-rise buildings is presented.

4.1. STRUCTURAL DESIGN OF HIGH-RISE BUILDINGS

At the end of the 19th century, tall buildings were constructed to increase the rentable area by stacking spaces vertically. To fulfil this economic direction, it was necessary to improve the load bearing systems used at the time. The result was the iron/steel frame, which was able to resist the loads while, in addition, maximized the building openings. As a result of the improved capability of constructing higher buildings, a skyscraper race began, in which the skyscrapers were a symbol of power and wealth. These high buildings, such as the Woolworth building (1913), the Chrysler Building (1930), and the Empire State Building (1931), were accomplished by using excessive structural materials, rather than by notable technological advances. The reason behind this is the lack of advanced structural analysis techniques at the time[26]. Later, the high-rise building sector to become more advanced and efficient by using innovative systems such as tubes, megaframes, coupled shear walls, braced frames, cores and outriggers; as well as by implementing more accurate structural engineering analysis, by means of technological developments in materials and computational tools[26]. At this time, the high-rise building industry was mainly driven by the technological capabilities, structural requirements and costs. Currently, in addition to these drivers, the environmental impacts play a substantial role on the decision-making process of the building industry in general [25,24].

4.1.1. STRUCTURAL REQUIREMENTS

The construction of high-rise buildings comes in hand with the inherent structural response to large vertical and horizontal forces. Due to the height of the buildings, gravity loads increase when the number of floors increases. These forces are the result of the weight of the structure, the applied load due to services and finishes, and the intended function of the building. On the other hand, the horizontal loads can be a result of the wind, earthquakes or the geometry of the buildings. In the Netherlands, horizontal forces are mainly originated from the wind. These wind forces cause large shear forces and base moments. Subsequently, they cause large deflections at the top of the buildings that govern the design most of times [27]. To resist the requirements that arise from these loads, the buildings require a certain stiffness, which can be achieved by placing the structural members in strategic positions to benefit from their mechanical properties and thus, restrain the building from excessive movements or to prevent its collapse.

4.1.2. STABILITY SYSTEMS FOR HIGH RISE BUILDINGS

There are many possible solutions to achieve this required stiffness. However, some of these structural solutions perform better than the others depending on the geometry, loads, materials used and the architectural requirements of the project.

Ali and Moon [26] propose a classification which encompasses the most representative structural systems for tall buildings based on the lateral load-resisting capabilities. The systems can be divided into two broad categories: Interior structures (Figure 7) and Exterior structures (Figure 8). This classification is based on the location of the components of the primary lateral load-resisting system of the building. The height of application of each system depends upon the design and serviceability criteria related to the building shape, aspect ratio, architectural functions, load conditions, building stability and site constraints. In the following sections, a general overview of the structural systems addressed in this thesis is presented.

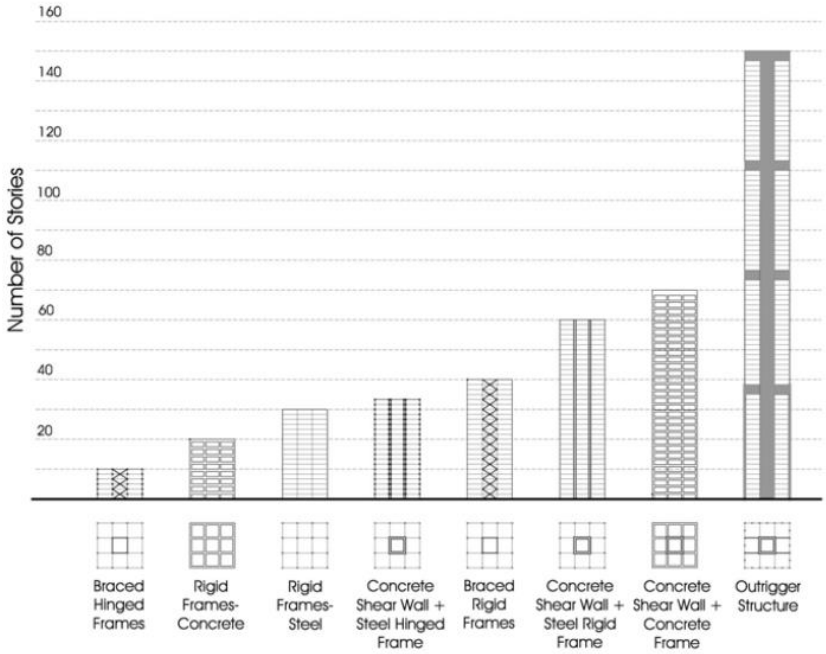


Figure 7: Interior Structures.[26]

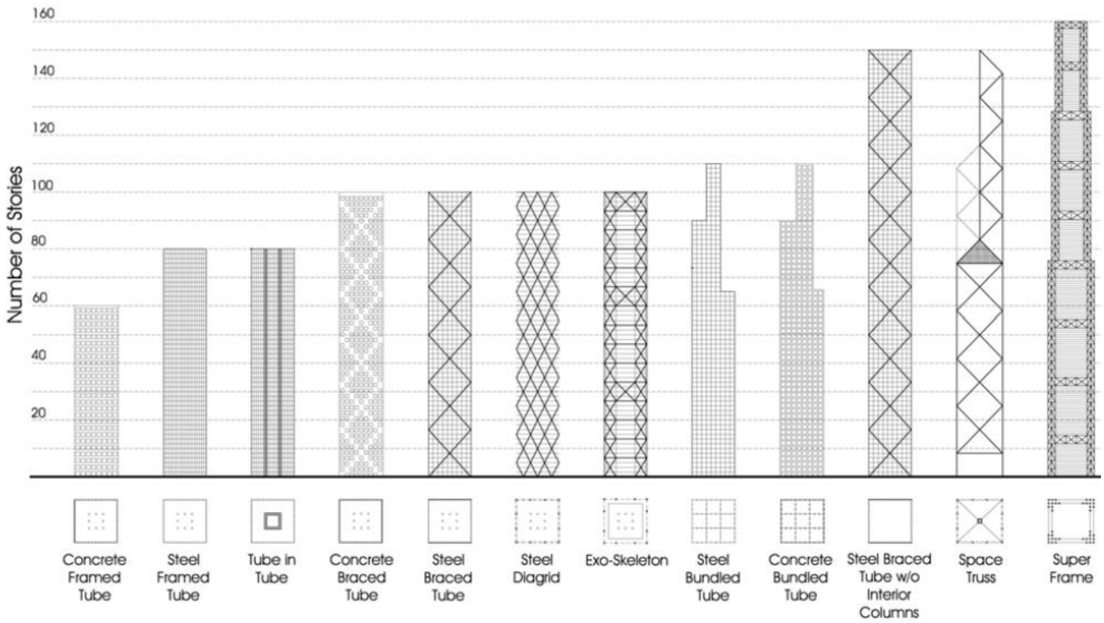


Figure 8: Exterior Structures. [26]

4.1.2.1. INTERIOR STRUCTURES

4.1.2.1.1. MOMENT RESISTANT FRAMES (MRF)

This system consists in a planar grid, made up by rigidly connected vertical and horizontal members (columns and girders). The gravity loads directly dictate the size of the vertical elements, since these loads are accumulated towards the base of the building. In the other hand, horizontal loads affect the size of both the horizontal and vertical elements, to assure a proper stiffness of the frame to withstand the lateral sway of the building [26].

The applied horizontal loads that act on the structure, result in the bending of the columns and the bending of the beams. Moreover, the internal forces in the column are a result of their individual bending, plus the additional axial forces that result from the bending moment acting on the frame.

The dependency on the rigidity of the columns and beams results in deformations conformed by two components: a shear mode deflection that results from the double curvature in the beams and columns, and a bending mode deflection that results from the axial strains in the columns[28].

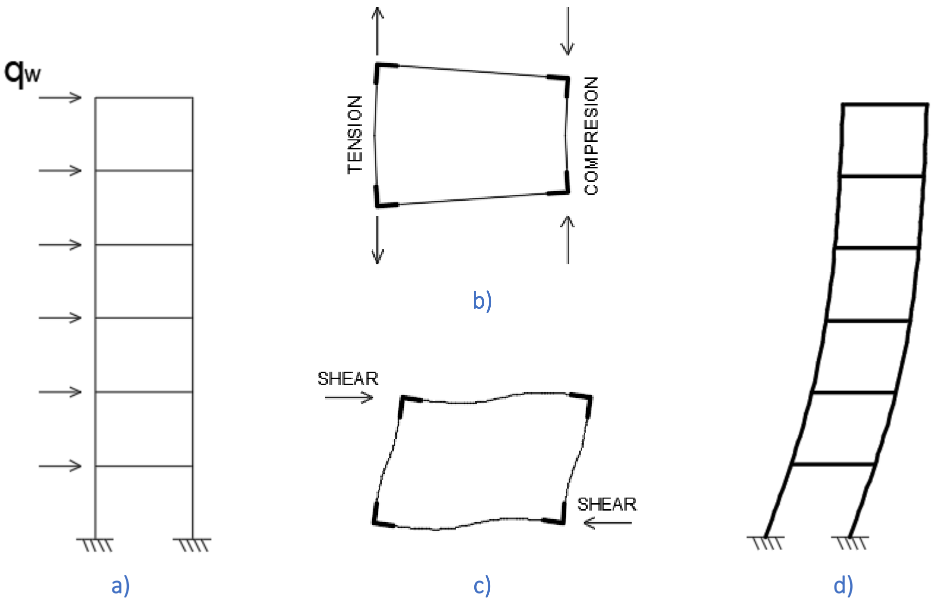


Figure 9: Moment Resistant Frame.
a) Model and Loads. b) Axial strains in the columns. c) Double Curvature of beams and columns. d) Deflection shape.

4.1.2.1.2. BRACED FRAMES

This system consists of laterally supported vertical steel trusses, which resist lateral loads primarily through the axial stiffness of the members. These frames behave as vertical cantilever trusses, where the columns act as the chord members, and the braces as web members. The brace members can be concentric (N, K, V or X braces) or eccentric. Concentric braces are connected to the column-girder joints which introduce only axial loads into the elements. Eccentric braces are connected to the floor girders with axial offset; thus, the braces introduce additional flexure and shear into the frame. This results in a higher weight to stiffness ratio, but it also increases the ductility. The location of the braced frames is generally in the core areas of tall buildings, and the braces are enclosed within the walls of the core.

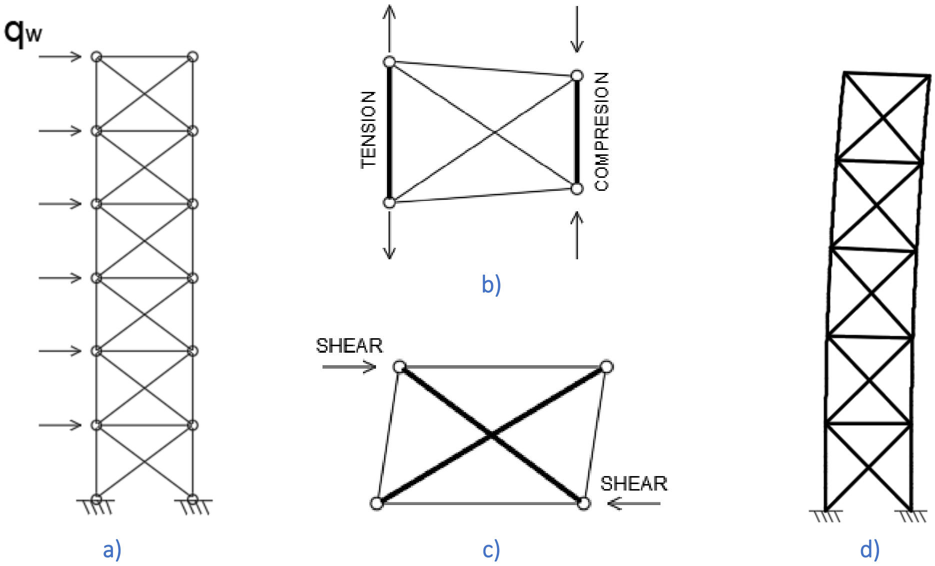


Figure 10: Trussed Frame.
a) Model and Loads. b) Axial strains in the columns. c) Axial Strain in the braces. d) Deflection shape.

The bending moments in a trussed structure are resisted by axial forces in the columns, while the applied shear loads on the structure cause axial forces in the diagonals. These two reactions create both, bending and shear like displacements, however, the bending mode is dominant in tall structures[28].

4.1.2.1.3. CONCRETE CORE

This system consists of a hollow concrete core that behaves as a vertical cantilever beam fixed to the ground. The deformation of this system is a resultant of bending mode displacement and of shear mode displacement. The walls deflect in a bending-like behaviour and, if sufficient stiffness is provided by the lintels of the core doors and/or by a stiff floor, an additional axial resistance occurs in said walls. Since the lintel of the door acts as a connection between the walls, the shear deformation is directly related to this element which experiences a high concentration of forces. In general, the concrete cores behave as a bending beam if enough stiffness is provided by the lintels [28].

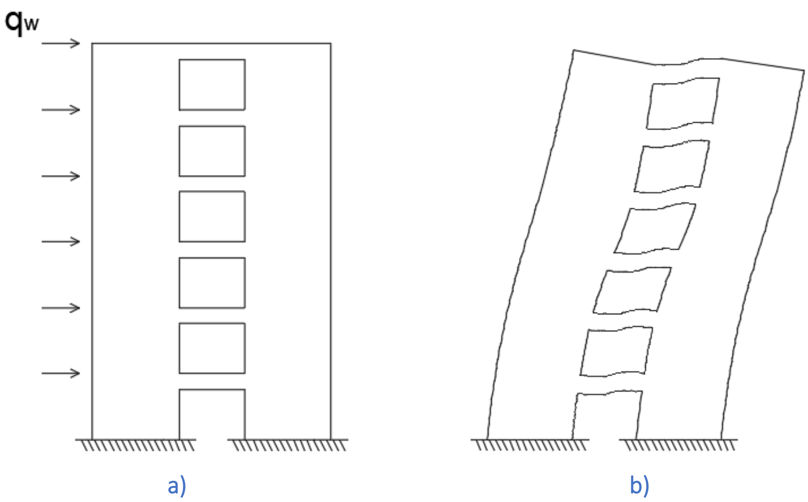


Figure 11: Concrete Core
a) Model and Loads. b) Deflection shape.

4.1.2.1.4. COMBINED SYSTEMS

Moment resistant frames can be combined with vertical steel trusses or with reinforced concrete shear walls to create an interactive system. This system behaves as a parallel system, which can be modelled as two basic systems that work together [29]. In other words, the two systems have the same deformation along their length and share the external horizontal load. In this structural model, the Moment Resistant Frame deflects like a cantilever beam with a shear deformed shape; and the Shear Walls deflect like a cantilever beam with a flexural deformed shape. Since the two components work together, by means of the slabs and girders, and their deflection is the same, this results in a combined behaviour. At the bottom of the building, the flexural cantilever (Shear Walls) restrains the excessive deformation of the frame. On the other hand, at the top of the building the interaction occurs the other way around, the moment resistant frame reduces the deflections of the flexural cantilever. This effect produces an increased lateral rigidity of the building. This type of combined systems is usually applied in buildings with 40 to 60 stories [26]

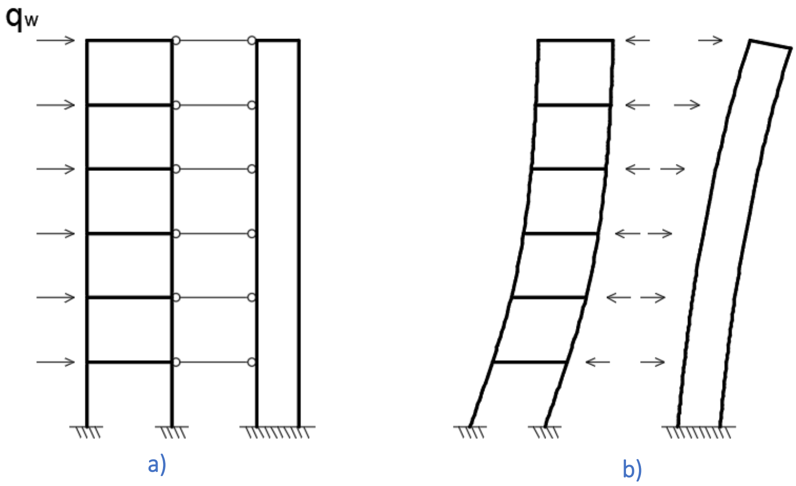
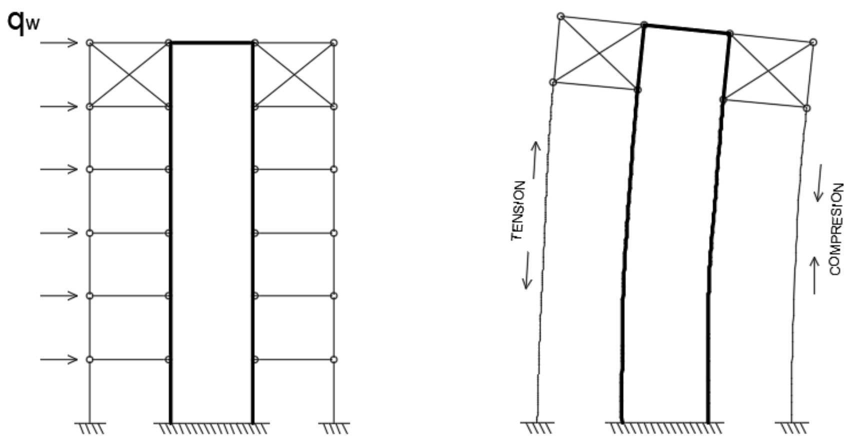


Figure 12: Combined System. Moment Resistant Frame + Core
a) Model and Loads. b) Interaction of both systems.

4.1.2.1.5. OUTRIGGER SYSTEMS

In this type of system, there are moment resistant floors (outriggers and/or belt trusses) located at a certain height. These floors are generally steel trusses or reinforced concrete walls which are connected to the core and to the outer columns. The connection between these elements allows the load transfer from the core to the external columns and thus, the bending moment acting on the core is reduced. Moreover, the resultant forces acting in the external columns are only axial forces; for this reason, the columns can be simply supported. [28].

Steel braced cores and reinforced concrete core walls can effectively resist the lateral loads for buildings from 30 to 70 stories respectively. However, for higher and more slender buildings, the core-outrigger systems can solve the problems that are generated by the overturning forces in the foundation and the excessive uplift forces in the core columns that occur in steel braced and concrete core walls systems[26].



a) b)

Figure 13: Outrigger
a) Model and Loads. b) Deflection shape, and axial forces in the external columns.

4.1.2.2. EXTERIOR STRUCTURES

Due to the correlation of an increased lateral load (wind load) as the height of the building increases; an efficient solution to attain the required stiffness of high-rise building, is to increase the structural depth of the lateral load bearing system. To achieve this, the lateral load bearing system must be located at the façade columns of the building. The incremented distance between the columns results on an increased lever arm to counteract the bending moments that occur on the building. Consequently, the concentrated axial loads that act on the perimeter columns of the building are reduced.

The most typical exterior structure is the tube, which can be modelled as a three-dimensional structural system in which the building perimeter resist the lateral loads. The use of three-dimensional tube systems offers the advantage that the relation between the parallel and perpendicular façades are included, while on the conventional planar rigid frame system, only the parallel frames are taken into consideration [28].

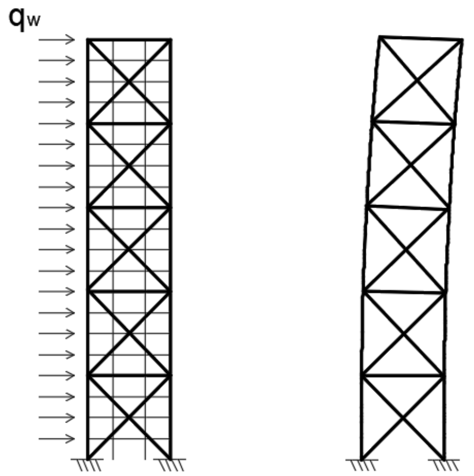
The tubular systems can be divided into different types depending on the location of the structural elements and on the structural demands. Some examples of the types of tubular systems are: Framed tube system, braced tube, tube in tube, diagrid system and megaframes.

4.1.2.2.1. MEGAFRAME

The megaframe stability system behaves in the same manner than the braced frames. However, its advantage relies on placing the braces along all the length of the façade which increments the lever arm between the structural elements and consequently increases the stiffness of the building.

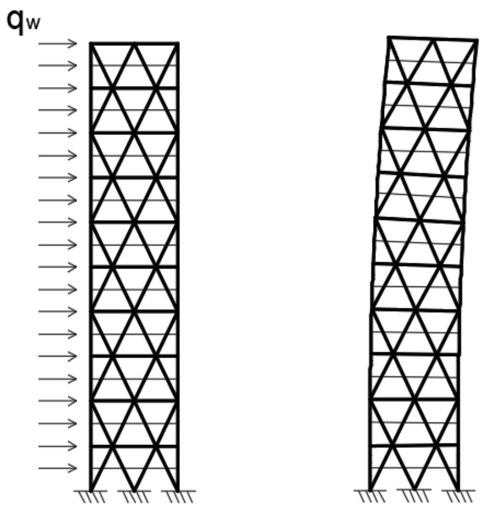
4.1.2.2.2. DIAGRID

Recently, the use of perimeter diagonals for structural effectiveness and aesthetics has become more popular. The difference between this system and a conventional braced tube system is that almost all the vertical columns are eliminated because the diagonal members are able to carry gravity and lateral forces, due to their triangular configuration. The removal of columns results in aesthetical façades and more free spans. Furthermore, the stiffness of this structural system is a result of the capability of the diagonals to resist the shear deformation along the building and at the same time it efficiently transfers the loads to the corner columns. Since all the structural elements of the stability system are only axially loaded, the efficiency of the system is higher than other systems[14], an thus, lighter structures can be achieved.



a) b)

Figure 14: Megaframe.
a) Model and Loads. b) Deflection shape.



a) b)

Figure 15: Diagrid
a) Model and Loads. b) Deflection Shape.

4.2. FOUNDATIONS FOR TALL BUILDINGS

In the previously mentioned studies (Section 3.6), the contribution of the substructure to the environmental impact of the building was left out of the scope of the research [14,23]. Therefore, it is of the author's interest to study how this system influences the environmental impact profile for different structural systems. In the following paragraphs, a description of the foundations and their interaction with the superstructure is described.

The load path of every structure ends up, naturally, in the substructure that supports it. The type of said substructures, also named foundations, depends on the superstructure that lays above them, and on the mechanical properties of the soil used on every specific project.

When designing foundations, there are properties of the superstructure that may have significant influence on the substructure [30]. These characteristics are listed below:

- Building weight and vertical loads. These loads are directly supported by the foundation. Moreover, the building weight increases non-linearly with the height, so both, the ultimate capacity and the settlement are significatively affected by this parameter.
- Differential settlements between high and low-rise structures. To avoid cracks and the breaking of secondary elements.
- Lateral forces imposed by the wind loading and the consequent moments of the foundation. These moments can impose increased vertical loads on the foundation, especially on the outer piles within the substructure.
- The wind lateral loads and moments are cyclic in nature. Therefore, it is important to understand the influence that cyclic loading can influence over foundation capacity and its influence of increased settlements.
- Seismic action and the transfer of forces mechanisms that can affect the foundation.
- Wind and Seismic loads have potential to give rise to resonance within the structure due to their dynamic nature.

4.2.1. FOUNDATIONS OPTIONS:

To satisfy the needs derived from the superstructure, there are different types of foundations that can be used in tall buildings. According to Poulos [30], the most common types are: Raft foundations, Compensated raft foundations, Piled foundations, Piled raft foundations and Compensated piled raft foundations. Every type of foundation has benefits and drawbacks when compared with the others. The choice of type of foundation is influenced by the following factors:

- Location and type of structure
- Magnitude and distribution of loading
- Ground Conditions
- Construction feasibility
- Effects on surrounding structures
- Relative costs
- Local Construction Process

For this study it is chosen to use piled raft foundations. This choice and the pile system properties are described in the following section.

4.2.2. PILED FOUNDATIONS

The soil in the Rotterdam area consists mainly of clay and peat [31]. The mechanical properties of these type of soils, require long and deep foundation which can transfer the load to a stronger layer conformed by sand or rock layer, and/or to bear the load by friction. Pile foundations are used when the ground conditions at site are not suitable for shallow raft foundations. Additionally, due to the magnitude of vertical and lateral loads for tall buildings, pile foundations can transfer the loads to stronger layers or bear the load by friction along the length of the piles when in contact with cohesive soil layers[32]. Moreover, by means of the raft above the piles, the individual settlements of each support are homogenized, therefore, differential settlements and unwanted displacements can be minimized.

This foundation system consists of single piles or pile groups located beneath columns and load bearing walls. It must be remarked that the load resistance of a pile group may be different than that of the sum of the single piles that conform it. Consequently, the settlements in a pile group may also differ from that of a single pile at the same average load level due to group effect. This phenomenon is a result of the interaction of the piles with the mechanical properties of the different type of soils [32].

The capacity of a pile depends on the soil type, skin friction of the pile, the end bearing capacity, the negative skin friction, the pile properties, the driving process, the hammer type and the type of loads. Moreover, the pile resistance of the group is also dependent on said parameters [32]. As a rule of thumb, to obtain the resistance of the piles, it is common to multiply the resistance of a single pile by the number of piles and by an efficiency factor which depends on the soil properties and the geometry of the foundation [33].

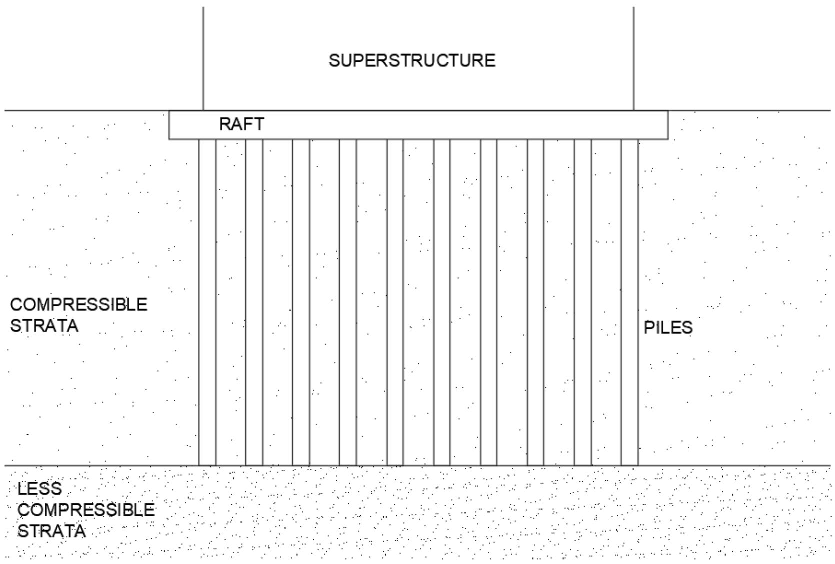


Figure 16: Piled Raft Foundations

4.2.3. SUPERSTRUCTURE-SUBSTRUCTURE INTERACTION

According to Poulos [30], the larger the relative stiffness of the superstructure, the smaller the differential deflection in piled raft foundations. One efficient method to address the superstructure-substructure interaction is to represent the piles by springs. The stiffness of these springs must include the effects of interaction of the piles with the soil layers. This stiffness is computed by a geotechnical expert, and the included as springs in the supports of the superstructure.

Furthermore, as a preliminary design, the ultimate limit state bearing capacity can be estimated by calculating the capacity of the pile group. This capacity should be larger than the load acting above it [30].

In addition to the capacity of the piles to satisfy the resistance requirements the foundations play an important role on the horizontal displacements that occur on the superstructure. The horizontal loads (e.g. wind load) generate a bending moment on the base of the building that can be represented as a couple of forces occurring at the supports. These forces are additional to the gravitational forces that act over the foundations and, since the act in opposite directions, they result in elongations (tension) of the piles located at one side of the building, and in shortening (compression) of the piles located at the other side. These differential displacements lead to the rotation of the base of the building. Consequently, the rotation at the bottom of the superstructure causes horizontal displacements at the top of the superstructure[28].

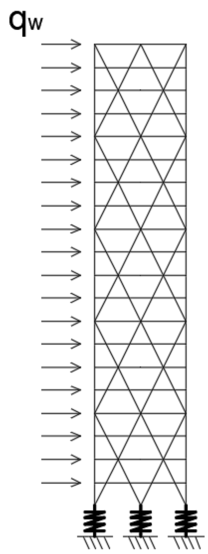


Figure 17: Substructure modelled as spring supports .

4.3. CALCULATION METHODS AND MODELS

There are different approaches to asses a building model. The choice of one approach is mainly influenced by the time for setting up the models, the information available to formulate assumptions, and by the degree of accuracy that it is intended on the study [27].

4.3.1. 1D MODELS

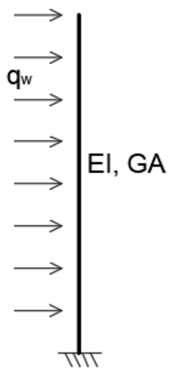
One of the simpler modes to describe the deformation of the buildings is a one-dimensional model defined by differential equations.

These models give an accurate result with small computation time in software like MathLab or Maple. Every structural system behaviour is dictated by a specific differential equation. However, these are derived from the same basis: One vertical cantilever Timoshenko-beam, with a clamped support at the base. The wind loads are represented as perpendicular loads distributed along the beam [29].

When differentiating one structural system to the others, it must be noted that each structural system has a specific bending stiffness EI_i, axial stiffness EA_i, and shear stiffness GA_i; depending on their geometry, slenderness, cross-sections area, cross-sections inertia, beam-to-column connection stiffness, and modulus of elasticity. To define these stiffnesses it is possible to use a bent model (Figure 19) [28], in which the contribution of these elements are working together and the deflection of the system depends on said stiffnesses.

From the deformed shapes of the structures, the resultant shear deformation of one system is different from one system to another. The same yields for the flexural deformation and for the axial deformation. This variation of stiffnesses results in complex and different models for each of the diverse systems.

One complexity of these models with multiparameter dependence is that different equations for simplifications may vary depending on the parameters and the ratio of the parameters. Furthermore, the stiffnesses can vary along the element. This variability results in decisions and assumptions made by the designer which yield results very sensible to said assumptions [34].



a)

$$EI\frac{d^2\varphi}{dx^2} - GA_{eff}\left(\frac{dw}{dx} + \varphi\right) = 0$$

$$GA_{eff}\left(\frac{d^2w}{dx^2} + \frac{d\varphi}{dx}\right) = -q$$

b)

Figure 18: Timoshenko beam.
a) Model with Loads. b) System of coupled Differential equations

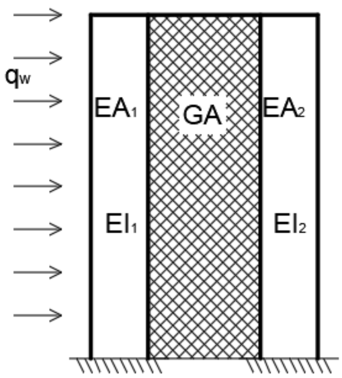


Figure 19: A bent model considers the contribution form all the elements to the total stiffness.

To define the complete structure behaviour, first, the specific structural system stiffness of each system must be derived by defining the geometry of the systems and its individual shear, flexural and axial stiffness. Secondly, to represent the coupled behaviour of the main and secondary structural systems (i.e. core and façade elements), it is possible to use a combined parallel system which represent this behaviour with an equation which considers the stiffness from the two stability systems [29].

This type of model can be very complex if each of the discrete models is defined by a bent model and the stiffness varies along the length of the model. A further complication of the 1D Model arises from the incorporation of foundations to the model. This addition can be simplified as a rotational spring which can be obtained by multiplying the resistance of each of the foundation piles with the distance to the middle of the building structure's base. However, the use of different structural systems results in diverse reactions from one system to another, thus, the contribution from the piles is also variable and the simplified model becomes more complex (like a 2D model).

4.3.2. 2D MODELS

The immediate option after discarding the 1D models are the 2D models. 2D models can be built in frame analysis software or with a finite element analysis software. The façades and the core are modelled in a planar space by including the geometry, cross sections, materials, supports, joint definition and load requirements. The façades and the core are linked by including beams which will work as the slabs. These links assure the same deformation on each of the systems [27].

These models yield accurate results with relatively short programming and computation time. Nevertheless, these models were not used in the study given the following reasons:

First, by creating a 2D model like this, the 3D model is partially done. Instead of placing the frames next to each other over the same plane, it takes the same amount of time and programming to place them in another, parallel, plane. Thus, the amount of time to define the geometry does not vary significatively. However, it is simpler to define the joint releases and degrees of freedom in a 2D model.

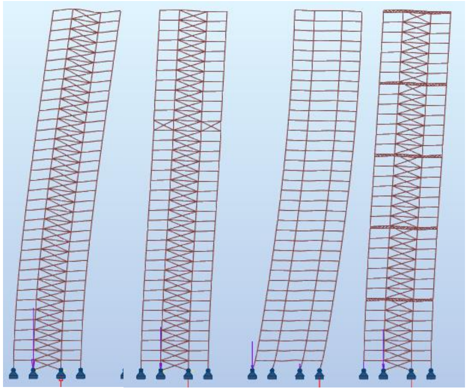


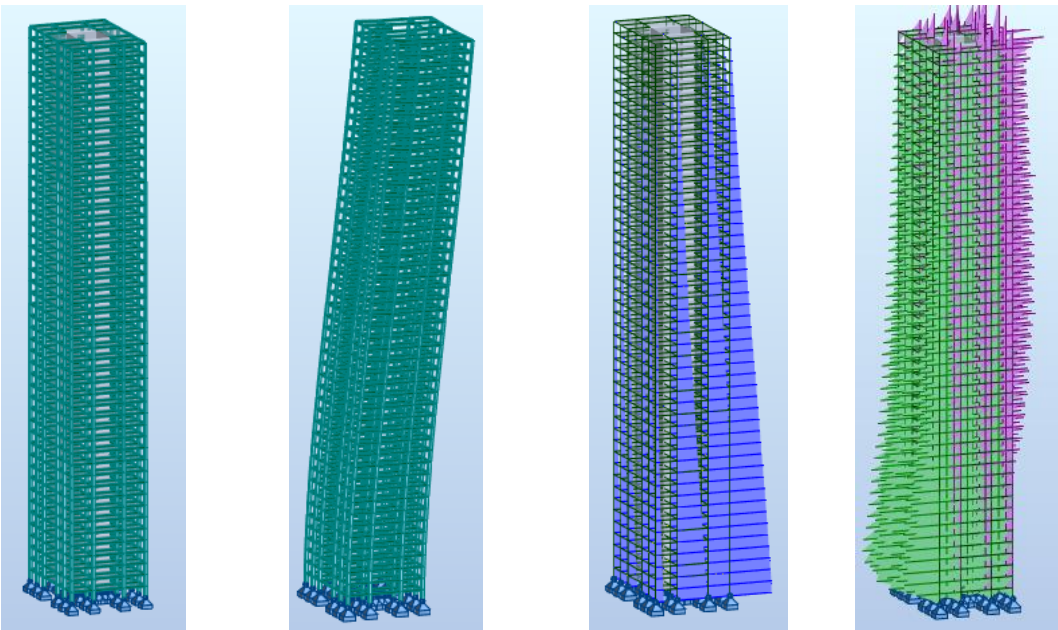
Figure 20: Example of 2D Models.

Secondly, the vertical load contribution to the stability system also depends on the stiffness of the elements. This dependency creates a loop where the stiffer the elements, the bigger vertical load they carry, thus the reactions at the supports are affected. To consider this load distribution in a 2D plan, several assumptions should be done. These assumptions can be avoided by building the models in 3D, which in return yields a more accurate model.

4.3.3. 3D MODELS

These models can be built in a 3D frame analysis or in a Finite Element Analysis software. The geometry is modelled in a 3D space and the cross sections, materials, supports, joint definition and load requirements are defined in the software.

The results from the 3D models are more accurate than the results from a 1D or a 2D model, given the fact that less assumptions and simplifications are made. The result is an accurate model with a low sensibility to assumptions, which directly influences the results accuracy.



a) b) c) d)

Figure 21: Example of 3D Model.
a) Model. b) Deflection. c) Axial Forces. d) Bending Moments

5. CONCLUSIONS FROM THE LITERATURE REVIEW

The governments propose a model of circular economy to tackle the challenges that arise from setting the goal of sustainable development. The building industry is implementing this mindset in the new projects; however, the high-rise building sector is responding slower to this mindset, and particularly the area related to the building embodied energy is slowly being addressed. Therefore, it is important to elaborate research that addresses this improvable area.

According to the statistics of the CTBUH, the number of voluntarily demolished buildings is very small when compared to the number of buildings under construction. For this reason, the new high-rise building projects must address flexibility in their architectural distribution and function, instead of their circularity. However, there is a possibility that new buildings are deconstructed. Moreover, if the deconstruction is assessed from the design phase, the future works of demolition and deconstruction can be substantially improved, and the environmental impact due to waste generation can be greatly reduced.

Furthermore, as there are new technologies developed to address the use phase of the building, the environmental impacts that result from this phase are greatly reduced. Therefore, the remaining phases (namely production, construction and deconstruction) become more relevant. To address this, the environmental impact of this stages can also be also reduced by means of sustainable structural design.

Sustainable Structural Design can be achieved in different ways, depending on the target and the goal of the project. It is fundamental to clearly define the goals, boundaries and parameters to study to generate a valuable design strategy. For the design of high-rise buildings in the Netherlands, it is concluded that the most suitable strategy for sustainable structural design, from the ones proposed by the JRC [13], is to design for adaptability and flexibility. This arises from the fact that there is a high uncertainty about the end of life of the buildings, and from the very low probability of a high-rise building being demolished. Additionally, the long life cycle of a building might result in variable functional requirements during its service life. For this reason, buildings should be adaptable to new solicitations. Additionally, besides to the design for adaptability strategy for sustainability, the possible benefits from reusing and recycling can be accounted for, by considering the use of demountable structural elements.

These two approaches yield two principal parameters that were studied in this research. Material Optimization, by means of using an efficient structural system; and the benefits at the End-of-Life stage of the building, by including reusable structural elements.

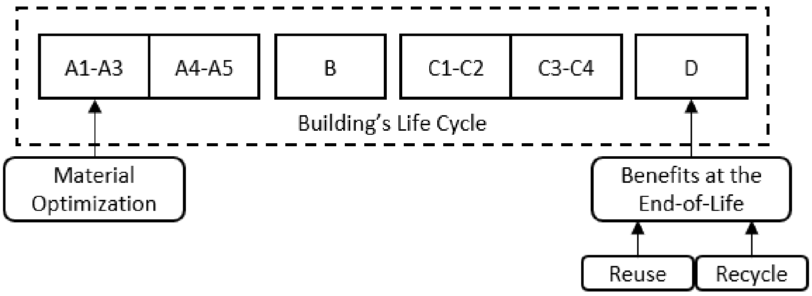


Figure 22: Parameters studied in the thesis.

The structural systems that satisfy the structural requirements in a more efficient way are the ones with an exterior stability system. These types of systems benefit from the large lever arm between the structural members and from the reduced forces that act in them. By reducing the internal forces of the structural elements, the cross section of said elements can be reduced and thus, lighter structures can satisfy the resistance criteria. However, according to the classification given by Ali and Moon[26], some internal stability systems are adequate for the geometry of the building studied. For both reasons, this research includes both types of structural systems (internal and external) on the models assessed.

Furthermore, there has been few research that addresses the environmental impact of foundations [35]. Besides, the research about these structural elements is addressed separately from the superstructure, which yields isolated results that cannot be directly extrapolated and generalized to studies which address the superstructure. For this reason, the foundations were included on the research to evaluate their influence on the structural behaviour and their contribution to the environmental impact of the structural systems.

3D parametrical models were used to build the numerical models. The use of 3D models minimizes assumptions and inaccuracies inherent to decisions that affect the accuracy of the results. Additionally, the parametric models allow a fast comparison of the different systems in the early stages of the research. Finally, by creating parametric 3D models, the element properties are gathered easier and faster. These numerical models address the structural requirements of the building and their results provided the data required for the environmental impact assessment.

To assess the environmental impact of the buildings, the Fast Track LCA method is used as a mean to obtain the environmental footprint from different structural system options and compare them. The assessment of different systems is a result of the interest on defining how the different choice of materials influences the structural design choices, and how the material use can be improved by using an adequate structural system.

From the articles and studies reviewed, it was found that the boundaries and the parameters used in the different studies are not consistent. This yields non-consistent results that vary depending on the scope used and the local requirements. For this reason, it is remarked that defining the parameters, boundaries and goals of the study is of paramount importance. Furthermore, the lack of consistence and correlation among the results makes that the data obtained from these studies is not extrapolatable to other studies. Therefore, not all the available information is useful. For this reason, the LCA of the buildings was evaluated by using the available data from the Dutch environmental database (NMD) [36] and from the European science and knowledge commission (JRC) technical report [13].

Finally, there are different approaches to allocate the benefits and burdens that result from reusing or recycling materials and elements at the end-of-life cycle of the buildings. The adoption of an allocation approach must be consistent with the goals and scope of the life cycle study.

III ASSESSMENT

As established in the objectives of the thesis, the main goal of this research is to compare the environmental impact profiles of buildings with different structural stability systems. To perform such a comparison, this research is divided into two assessments: Structural Analysis, and Environmental Impact Analysis.

The first assessment (6. Structural Analysis) addresses the structural requirements of the building and evaluates the influence of using different stability systems on the response of the structure. This assessment includes the design of the stability systems, by performing a general optimization of each of the structural systems; and the design of the foundation, by including the 3D model of the superstructure. Furthermore, the stability systems are designed to satisfy the ultimate limit state of the structural elements and the serviceability limit state of the whole building.

The second assessment (7. Environmental Impact Analysis) addresses the environmental impacts of each of the structural systems used on the first assessment (Structural Design). To perform this analysis, the data that corresponds to the building properties is derived from the results of the structural design, and the data that correspond to the factors to give an environmental impact to each of the materials is retrieved from the Nationale Milieu Database and from the Joint Research Centre. This evaluation is performed by means of a Life Cycle Analysis that considers a cradle to cradle assessment.

6. STRUCTURAL ANALYSIS

This chapter describes the structural design and analysis of the stability systems of the high-rise buildings studied. First, the structural assessment is defined by describing the requirements, the general conditions of the structural systems and the parameters used for the evaluation. Next, the geometry of the different stability systems is described. And finally, the results are presented and discussed.

6.1. STRUCTURAL DESIGN AND MODELS

This section describes the properties, geometry and characteristics that were considered for the 3D models of the stability systems studied. Likewise, the requirements and considerations taken for the evaluation of the structural models are described in the following subsections.

6.1.1. BUILDING GEOMETRY

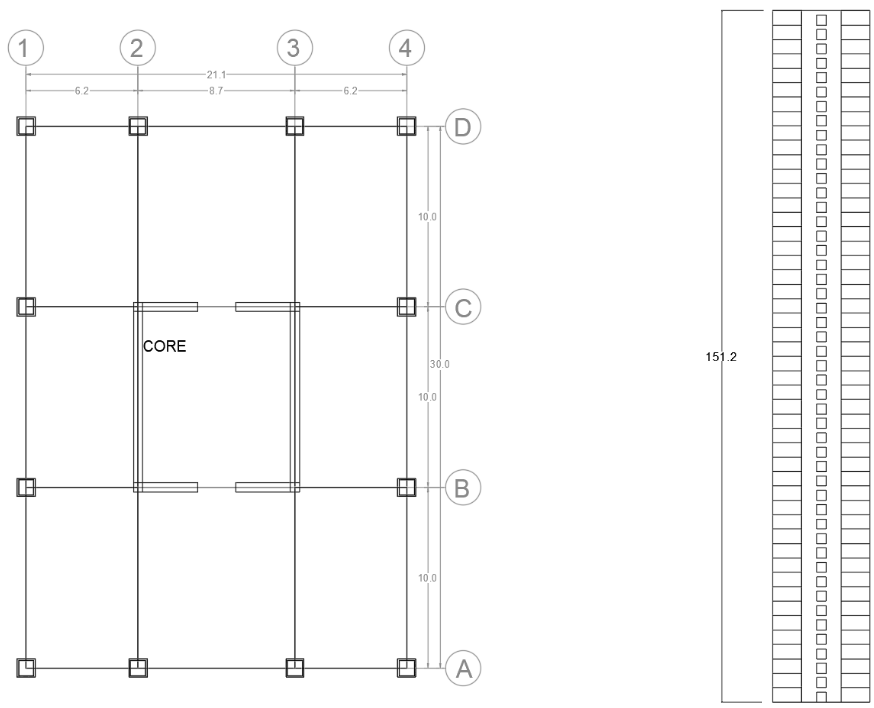
To homogenize the floor plans and to perform a comparison of a standardized building, the same location was considered for the columns and beams, for all the structural systems analysed. With this disposition, all the systems have the same architectural advantages; thus, only the structural differences are compared. In this assessment, the material optimization goal of the analysis refers to the optimum location of the structural members rather than a thorough optimization of each structural system used.

The building is considered as an office building, with the geometry based on a reference project from IMd Raadgevende Ingenieurs. The dimensions were slightly manipulated in a way that the final geometry simplifies the modelling phase for all the different structural systems.

The storey height, taken from the case study, was set to 3.15m. This height satisfies the minimum floor height (2.60m according to the Dutch code [37], while also leaving enough space for the slabs, floor finishes, ceiling finishes, service ducts, and beams. The building has 48 floors, resulting in a total height of 151.2m (Figure 23 a).

The floor plan consists of a rectangular shape with 30m width by 21.1m depth, which results in a surface of 633m^2 by floor. The dimensions of the core are 10m width by 8.7m depth; these values are similar to the ones required in the case study (Figure 23 b). The size of this core meets the requirements established by the Dutch regulations regarding natural daylight illumination in office spaces [37].

When a project is defined, the dimensions of the core are determined by both architectural and structural requirements. For this study these dimensions were set as the default dimensions for all the structural systems studied, i.e. the core has the same size for the different systems studied. However, it is acknowledged that the variation of the dimensions of the core can result in a more optimum solution when a specific system is studied. The influence of this parameter (core size) was left out of the scope of this study.



a) b)

Figure 23: a) Standard floor plan and b) elevation of the building.

6.1.2. LOADS

The loads considered for the models are vertical and horizontal loads, and their values were defined according to the Eurocode[37,38, 39] and the Dutch National Annex[41].

6.1.2.1. Vertical Loads

The vertical loads were considered as follows:

DEAD LOAD		
• **Self-weight**	**Variable**	$\boldsymbol{kN/m^2}$
• **Additional Dead Load**	**3.52**	$\boldsymbol{kN/m^2}$
• Partition Walls	1.00	kN/m^2
• Floor and Ceiling finishing	1.25	kN/m^2
• Ducts and Services	0.25	kN/m^2
• Façade load*	1.02	kN/m^2
LIVE LOAD		
• **Live Load****	**2.50**	$\boldsymbol{kN/m^2}$

Table 2: Dead and Live Loads for the structural design

* For the global analysis, the façade loads are distributed over each floor.
** According to the Office Area category, as defined in the Dutch National Annex[41].

6.1.2.2. Horizontal Loads

The location of the building is in Rotterdam, The Netherlands. The horizontal loads considered are the wind loads, determined by the Eurocode[40] and the Dutch National Annex [41].

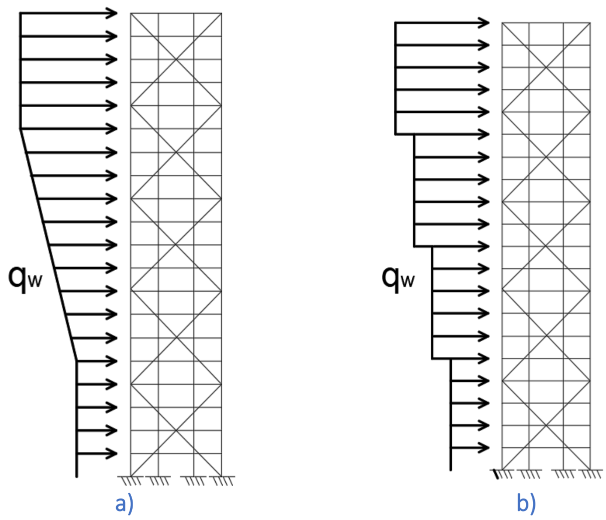


Figure 24: Wind load profile

When the loads were calculated, it was observed that the most unfavourable results were derived from the wind load in the direction perpendicular to the face with the larger width. For this reason, the calculations were reduced to solely the wind load in this direction.

The wind resultant values of the wind pressure are: $2.40\ kN/m^2$ at the top area of the building, and $1.44kN/m^2$ at the bottom of the building. See Appendix A for a detailed calculation of the wind pressure along the building.

The wind load profile has the shape as presented in Figure 24 a. However, to simplify the models the building was divided in four fields along its height, which resulted in the load profile represented in Figure 24 b.

6.1.2.3. Load Combinations

According to the Eurocode and the Dutch National Annex, the following Load combinations were used to assess the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS) [Appendix A]:

ULTIMATE LIMIT STATE	
$ULS1$:	$1.5G + 0.825q_v$
$ULS2$:	$1.3G + 1.65q_v$
$ULS3$:	$1.3G + 0.825q_v + 1.65q_w$
$ULS4$:	$0.9G + 0.825q_v + 1.65q_w$
SERVICEABILITY LIMIT STATE	
$SLS1$:	$1.0G + 1.0q_v$
$SLS2$:	$1.0G + 0.5q_v + 1.0q_w$

Table 3: Load Combinations

6.1.3. STRUCTURAL SYSTEM DEFINITIONS

The main goal of this study is to compare the environmental impact of different structural systems and, in this line, to compare how different materials impact on these environmental profiles. With this in consideration, the structural systems compared were the following: Moment Resistant Frame, Outrigger, Megaframe and Diagrid. Each one of these systems with two variables: Steel Core or Concrete core. Table 4 contains the structural systems used in the study, and the definition of their components is described in the following subsections.

STRUCTURAL SYSTEM	ABBREVIATION
Steel Core with Steel Moment Resistant Frame	STMRF
Steel Core with Steel Outrigger	STOUT
Steel Core with Steel Megaframe	STMGF
Steel Core with Steel Diagrid	STDGD
Concrete Core with Steel Moment Resistant Frame	CCMRF
Concrete Core with Steel Outrigger	CCOUT
Concrete Core with Steel Megaframe	CCMGF
Concrete Core with Steel Diagrid	CCDGD

Table 4: Structural Systems Used for the study.

6.1.3.1. FLOOR SLABS

To take into consideration the demountability of the structure, hollow core slabs were used for the floor system. These type of slabs have a lower production environmental impact than flat slabs and composite slabs[25]. Moreover, the connection to the structural system can be done without chemical connections, which is fundamental for demountability. On the other hand, due to the required easiness of deconstruction, the use of the cover layer above the hollow core slabs (screed) was neglected to avoid casted elements which would complicate the disassembly of the building. However, to assure that the floor acts as a diaphragm and to control the floor displacements, it was considered that the façade beams work as tension belts, which homogenize and restrict the individual displacements of the slabs and other structural elements.

The slabs were modelled as simply supported, one-directional slabs with the geometry specified in section 6.1.1 Building Geometry. These elements were modelled with relatively low stiffness to decrease the contribution of the floors to the global stiffness of the stability system. However, it was modelled rigid enough to ensure a homogeneous displacement throughout each floor. This was achieved by reducing the depth of the slabs on the model but including the original weight.

6.1.3.2. CORES

From the literature, it was concluded that the structural system which have exterior stability system are able to resist larger lateral forces. However, due to safety reasons, the buildings must include a core to allow the evacuation in case of a threat (mainly fire safety). Since the buildings already need a core it is common to use this area to allocate the stairs and elevators of the building. Furthermore, this core can be used as part of the stability system, which becomes beneficial to reduce the structural requirements on the façade elements. To consider different types of materials and their influence on the system, two types of core were defined: concrete core and chevron braced frame (Figure 27).

6.1.3.2.1. CONCRETE CORE

The structural systems with a concrete core (C50/60) (Figure 25) were designed with a core thickness of 500mm at the bottom, 420mm from a quarter of the height to middle height, where it changes to 340mm; and finally, it ends with a thickness of 250mm from three quarters of the height to the top of the building.

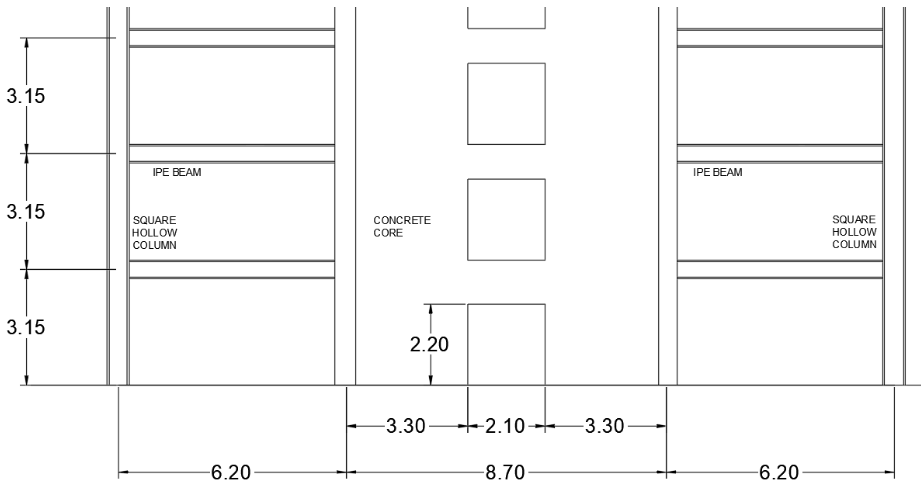


Figure 25: Example of Concrete Core used for the models.

These thicknesses were determined based on experience from other reference projects from IMd Raadgevende Ingenieurs. The use of non-variable thickness values affects the results in a way that if environmental impact optimization is sought, the use of a stiffer core reduces the need of stocky secondary stability elements; and, in an inverse manner, the use of a weaker core increases the quantity of material required for the non-core stability elements. This proportionality affects the quantity of material needed on each of the stability systems, and thus, it influences the environmental impact profile. For this thesis, this correlation was not studied, but a default core size worked as a departure point for the comparison.

6.1.3.2.2. STEEL CORE

For the structural systems made up entirely of steel (S355), a chevron braced frame was used as the "steel core" (Figure 26). This steel core was not be the only stability system but only a part of it, since it worked conjointly with the other elements outside of the core to resist the horizontal loads solicitations.

The chevron bracing system has the advantage of allowing enough clearance in the middle of the frame to allocate doors to access the core. Additionally, the use of chevron braces decreases the span of the core beams by half, which consequently reduces substantially the deflection and bending moments in the beams. According to Siddiqi [42], this type of bracing outperforms other bracing systems when the displacement are compared.

One characteristic of this bracing system is that the braces work one tension and the other one in compression at the same time. However, the tension braces remain in tension while the braces in compression lose their axial load capacity after buckling. This contributes to large bending moments on the intersection of beam and braces, as well as an unbalanced distribution of lateral forces. One solution to overcome this drawback is the use of a strong beam which is capable of providing adequate strength [43].

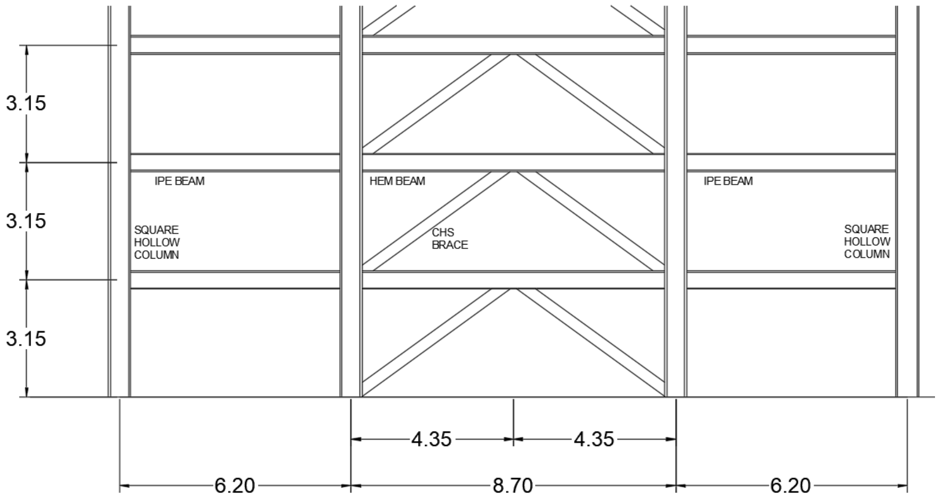
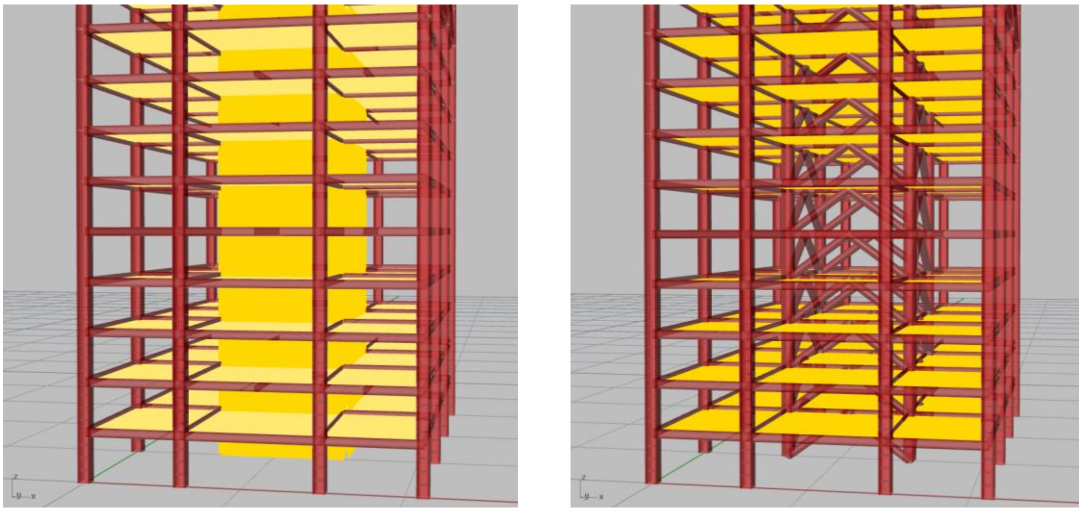


Figure 26: Example of Chevron bracing used for the models with steel core.



a) Concrete core

b) Steel core

Figure 27: Types of core used.

6.1.3.3. FOUNDATIONS

The use of lighter and stronger materials (steel instead of concrete) results in lighter structures. By decreasing the weight of the superstructure, the size of the substructure can be consequentially reduced. Furthermore, by using different structural systems with the main stability system located either internally (Moment Resistance Frames and Outrigger) or externally (Megaframe and Diagrid), the distribution of forces and reactions is variable from one system to another, and hence the corresponding piles on each system are also variable. The number and distribution of piles is dependent on the structural system used, for this reason, the piles were modelled in the structural model of the superstructure by adding springs on the supports of the building [30]. [Appendix E]

For this thesis, the foundation characteristics and properties (Table 5) were obtained from the foundation's supplier of the base case study from IMd Raadgevende Ingenieurs.

PILES PROPERTIES		
Resistance	8000	kN
Diameter	0.85	m
Length	29	m
E	600 x 10^6	N/m
Density	2500	kg/m^3
Area	0.57	m^2
Centre to centre	1.87	m

Table 5: Pile Properties

Furthermore, the inclusion of the piles on the FEM models provides a more accurate model by accounting for the contribution of the foundations to the horizontal displacement of the structure. Hence, the superstructure can be designed more efficiently.

6.1.4. PARTICULAR GEOMETRY DEFINITION OF THE SYSTEMS

The definition of the geometry of each individual system is described in this section.

6.1.4.1. MOMENT RESISTANT FRAMES (MRF) AND BRACED FRAMES

Additionally to the core (concrete or braced steel frame), six moment resistant frames (MRF), made of steel elements (S355), were positioned equidistantly in the direction parallel to the wind load. The joints were all defined as rigid. The stiffness of each floor frame depends on the geometry of the frame (i.e. height, spans length, number of bays) and of the size and inertia of the structural elements (i.e. area of the beams and columns, and the ratio of the inertia of the beams over the inertia of the columns). Additionally, the stiffness of the whole structure depends on the height and slenderness of the building [34].

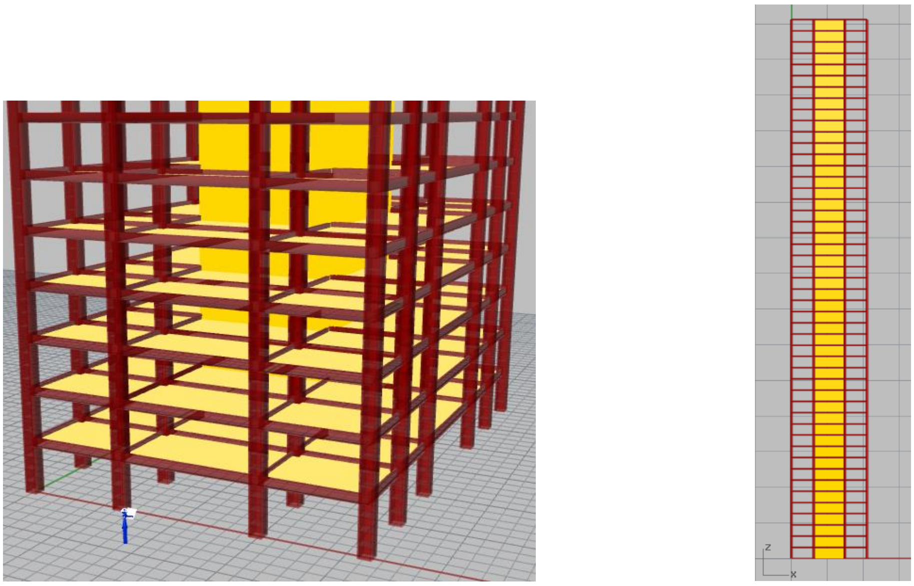
For the other structural systems, it sufficed to only add four frames (instead of the six proposed with the MRF) with their joints defined as pinned working together with the specific characteristics of each of the different structural systems.

By using only four frames for the following systems the use of intermediate columns in the building was avoided. This resulted in large and free spans on each floor which addresses the sustainable structural design strategy of design for flexibility.

6.1.4.2. OUTRIGGER

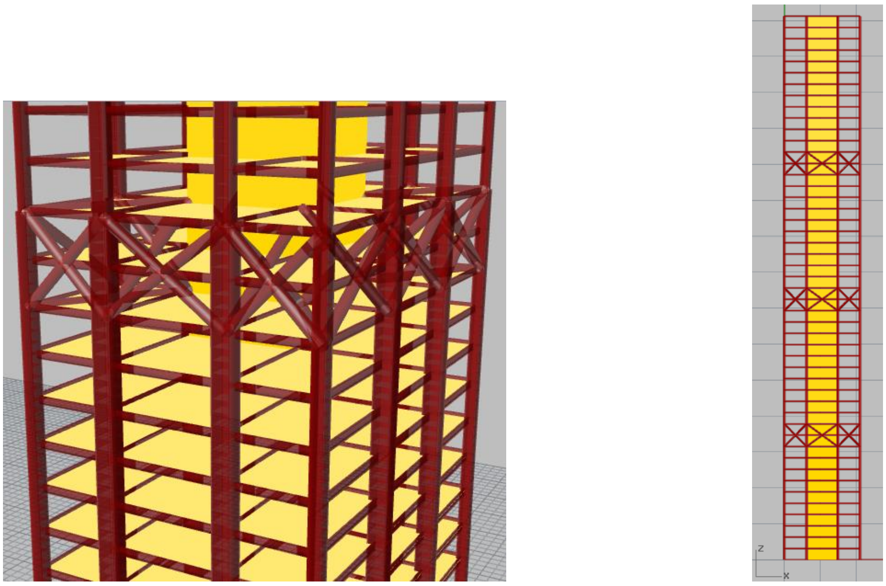
For the outrigger system, it was chosen to use three outriggers. The first located at ¼ H, the second one located at middle height and the third one located at ¾ H. These locations were chosen by using the optimized location to decrease the horizontal deflection of the building.

The outriggers span over 2 floors to reduce the axial forces in the elements that conform them. These outriggers were also modelled with an additional belt truss to spread the load to all the façade columns instead of only the columns aligned with the axis of the core; therefore, increasing the stiffness of the building. These outriggers reduce the moment in the core, by transferring the loads to the columns.



a) b)

Figure 28: Moment Resistant Frame 3D Model



a) b)

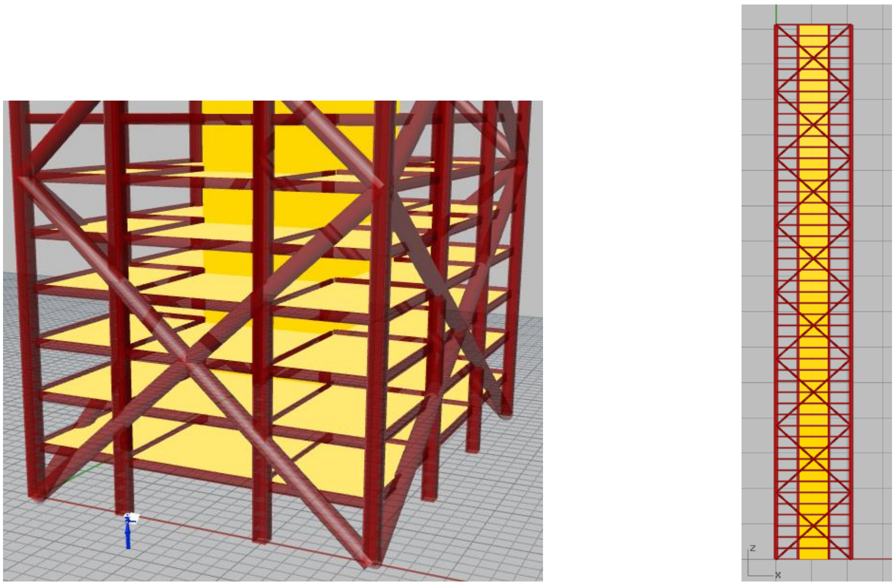
Figure 29: 3D Model - Outrigger with concrete core.

6.1.4.3. MEGAFRAME

The Megaframe used in this study consisted of eight braces located at the façades. This braces spanned from one façade to opposite one over a height of 6 floors each. This arrangement of braces resulted in an angle of 42º which is close to the optimum slope of 45º [43]. The braces were considered of circular hollow sections. With this geometry, the columns contribute only to the vertical solicitations, with the exception of the corner columns, which are axially loaded by the horizontal forces.

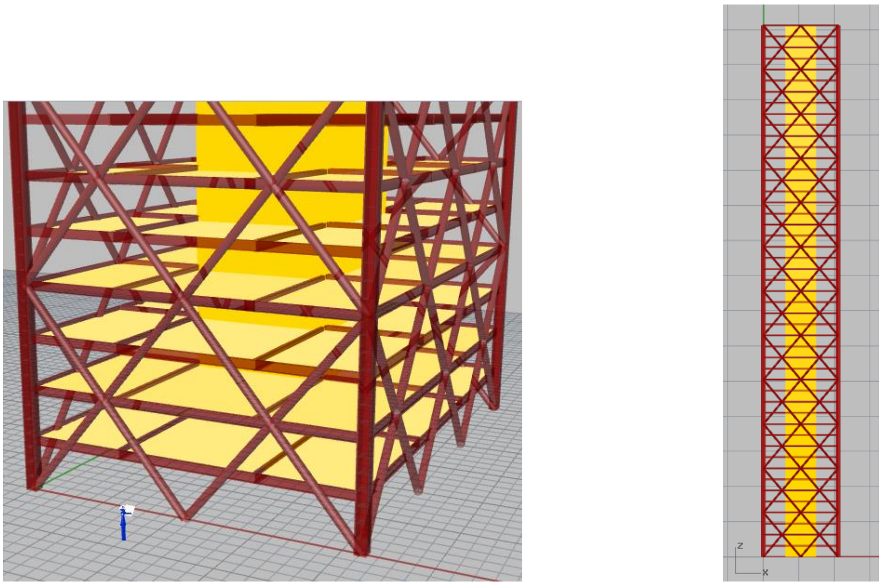
6.1.4.4. DIAGRID

The diagrid was defined by diagonal braces which spanned over two stories. The slope of these elements was 50º which is smaller than the optimal [44]. However, the reason of choosing a 50º is that, for this study, the slope of 50 resulted in smaller deflections than other possible arranges. The braces contribute to the resistance of both, vertical and horizontal load; for this reason, only the columns in the corner of the façades were additionally considered (Figure 31).



a) b)

Figure 30: 3D Model - Megaframe with concrete core.



a) b)

Figure 31: 3D Model - Diagrid with concrete core.

6.1.5. ASSESSMENT

The designed buildings must comply with the structural requirements defined by the Eurocode and National Annex; namely, the load bearing resistance and the admissible deformations.

In high buildings, the governing solicitation is usually the global displacement at the top floor of the building. To assess the buildings, this deflection was used as the limit and as the criterion to compare the response of the different structural systems. The deformation was calculated by 3D models built in grasshopper and computed in Karamba3D (SLS check). Once the deflection criterion was met, the resistance of the elements was checked with the utilisation ratio tool incorporated in Karamba3D (ULS check) [Appendix C]

6.1.5.1. SERVICEABILITY LIMIT STATE LIMITS

According to the Dutch National Annex [41] the permissible deflection at the top of the building (δ) must be equal or smaller than the height of the building divided than 500 ($\delta = H/500$). Additionally, the vertical deflection of beams and floors (w_{max}) is limited to $w_{max} = l/250$, and the drift between the storeys (δ_{drift}) is limited to $\delta_{drift} = h/300$. Where l is the length of the span of the element analysed, and h is the storey height.

The deflection at the top of the building is a result of a flexural deflection of the structure (i.e. the deflection of the load bearing elements due to the shear and bending deflections which result from the external loads) and a rotational displacement (i.e. the external loads acting on the building generate a rotation at the bottom of the building, which is resisted by the foundation, and this rotation results in horizontal displacements at the top of the building). A deeper analysis and description of this behaviour is described in the Section 4.2.3.

Given the fact that the soil in the Netherlands is composed mainly by clay and peat[31], it is considered that it has a relatively low stiffness value. For this reason, the foundations must be deep to assure a proper attachment to the ground. It is a common practice to attribute half of the total deflection at the top of the building as a result of the rotational deflection, and the other half to the deflection due to the flexural deflection[27].

6.1.5.2. 3D MODELLING FOR THE STUDY

The three-dimensional models are the models which fit better with the setting up time, accuracy of results and degree of reliability required for the present thesis. The geometry of the models was generated with Grasshopper [45] and visualized in Rhinoceros 3D [46]. The structural analysis (global analysis and member utilization check) was performed with the Finite Element plug-in, Karamba3D [47]. To verify the Karamba3D models, the Finite Element Analysis software, Robot Structural Analysis [48], was used. One fundamental decision of using Grasshopper to build the geometry was the possibility to define the input data with parametrical tools. The easy manipulation of parameters results in a flexible and fast tool for early design steps to compare different options, arrays and characteristics, to verify the fulfilment of the requirements.

The buildings were modelled in three dimensions with the general geometry and each of the structural system improvements by the addition of its own characteristic stability system, as mentioned above. The steps to run the model and gather the results are depicted in Figure 32 and are briefly described in the following paragraphs. See Appendix C for a more elaborated description.

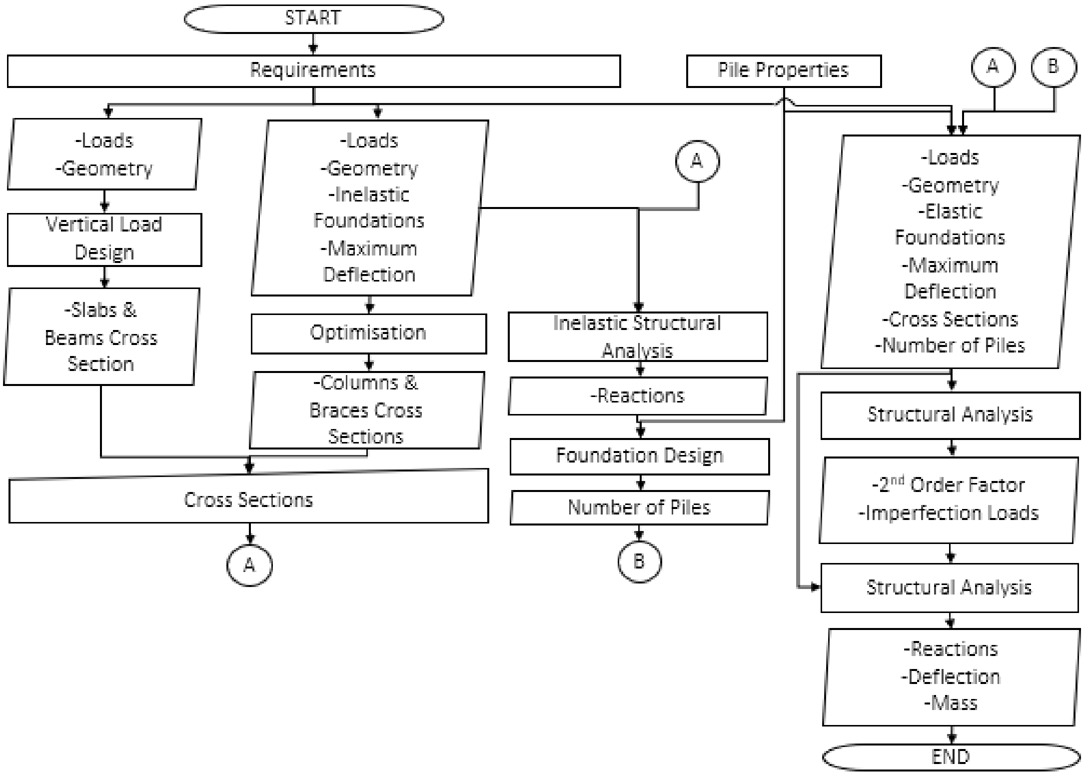


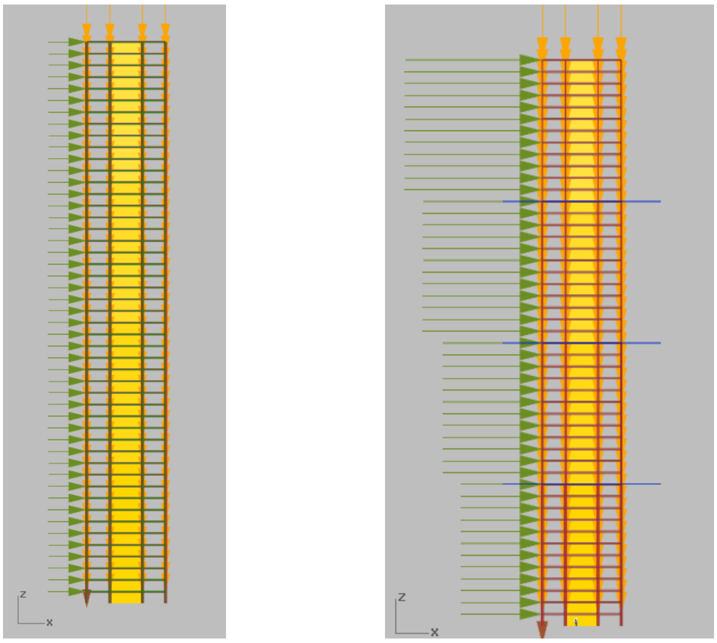
Figure 32: Flowchart of the modelling process and execution

First, the floors and beams were designed and dimensioned to resist the vertical loads of each floor [Appendix A]. Then, the Karamba3D Models were modelled with both, a uniformly distributed horizontal load (wind load) and the vertical loads originated by the dead loads and live loads (Figure 33 a). With the optimization component of Karamba3D, the structure was optimized for the load combination SLS2, in which the contribution from the wind load defined the deflection of the structure. This optimization had a target of minimizing the weight of the structure while fulfilling the deflection requirement of $\delta = 0.8 * H/500$. In this first step the foundations were disregarded. Additionally, the 0.8 factor used in the deflection was defined after analysing the different structures and determining that the displacements due to the rotation of the foundation only account for around 20% of the total deflection for this study.

Secondly, the optimized cross sections were input into a model conformed by a building divided in four fields along its height (i.e. first field from 0H to H/4, the second field from H/4 to H/2, the third field from H/2 to 3/4H, and last field from 3/4H to H). This division was done to have a more accurate distribution of element sizes and to simulate a more accurate wind load action (Figure 33 b).

These models were run in Karamba3D considering all the load combinations (Ultimate Limit Sate and Serviceability Limit State), and the reactions were computed and streamed to a .csv file.

Subsequently, with the value of the reactions from the .csv file, the properties of the foundations and with the location of the supports; the number of piles per support was calculated with an Excel Spreadsheet [Appendix E]. The number of piles was determined by the minimum number of piles to resist the applied reaction. This was done considering all the load combinations requirements and with the pile properties provided by IMd (Table 5).



a) “Optimization Model” b) “4 Fields Model”

Figure 33: 3D models used for the structural analysis.

Afterwards, the number of piles were modelled and input as elastic foundations in the grasshopper model and the 3D model was run one more time to calculate the deflections, reactions and properties of the model. This second time the model was considered with elastic supports [Appendix E].

Finally, with the displacements fulfilling the deflection requirements (SLS) and the structural members satisfying the resistance solicitations (ULS), the second order effects and initial imperfection factors were verified with a script in grasshopper [Appendix C].

The results of these models were deflections, mass of the superstructure, reactions, number and location of the foundation piles and mass of the substructure.

6.2. STRUCTURAL ANALYSIS RESULTS

As stated in the structural requirements, the maximum allowable displacement at the top of the building should be less than H/500, according to the Eurocode and the Dutch National Annex. This criterion was the main target to optimize the use of structural elements in the building. To perform the material optimization of the whole structure, the optimization component from Karamba3D solved solve the analysis by finding the lightest structural elements that satisfy the resistance requirements, while the structure is still able to satisfy the displacement requirements. In this way, the environmental impact of the systems is reduced by decreasing the mass of the structural elements. The following sections present the results obtained from the structural analysis step of the study. The information about the calculations, structural elements cross sections, and additional information can be found in the Appendices A to E.

6.2.1. DEFLECTIONS

According to the SLS requirements, the maximum allowable deflection at the top of the building should be smaller than the height of the building divided by 500 ($\delta_{max} = H/500$). For this study $\delta_{max} = 302.4mm$.

Figure 34 represents the normalized displacement of the eight different structural systems analysed. All the structural systems analysed satisfy the Serviceability Limit State. On average, all the structural systems' deflection is close to the maximum allowable; the deflections of the steel moment resistant frame (STMRF), the steel outrigger (STOUT), the steel megaframe (STMGF) and the concrete megaframe (CCMGF) are on average 3% smaller than the maximum allowable displacement; and the deflection of the steel diagrid (STDGD), the moment resistant frame with concrete core (CCMRF) and the diagrid with concrete core (CCDGD) are on average 11% smaller than the maximum allowable displacement. This is not the case for the outrigger with concrete core (CCOUT) which's deflection is significatively lower (-31%) than the maximum permissible (Figure 34).

Moreover, given that the structural systems present different degrees of deflection, the stability systems can be categorized according to their stiffness. i.e. the ratio of the maximum allowable displacement and the actual deflection of the 3D model. Table 6 lists the stiffness of the stability systems studied. This stiffness was defined as $\delta_{max}/\delta_{total}$.

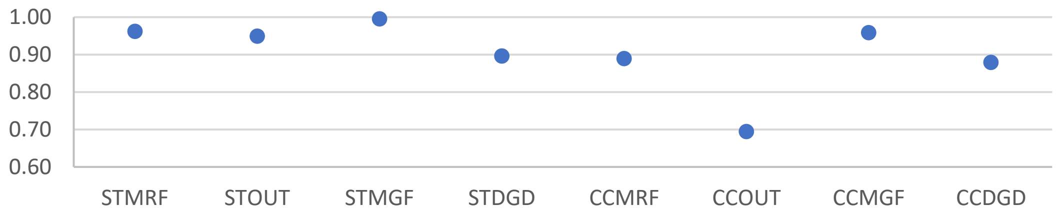


Figure 34: Normalized Displacement
Model Displacement / Maximum allowable displacement

Structural System	STMRF	STOUT	STMGF	STDGD	CCMR	CCOUT	CCMGF	CCDGD
Stiffness ratio	1.0	1.1	1.0	1.1	1.1	1.4	1.0	1.1

Table 6: Stiffness of the Structural systems.

In addition to the total displacement of the structure, this value was further dissected into its flexural and rotational components. Figure 35 shows the contribution to the displacement by the foundation's rotation and by the superstructure deflection. On average, the deflection due to the foundation rotation is 5 cm and it contributes to 20% of the total deflection. On average, the flexural deflection accounts for 80% of the total displacement. For the flexural stiffness, the structural systems with concrete core represent a lower contribution (on average 78%) than for the structural systems with steel core (on average 83%).

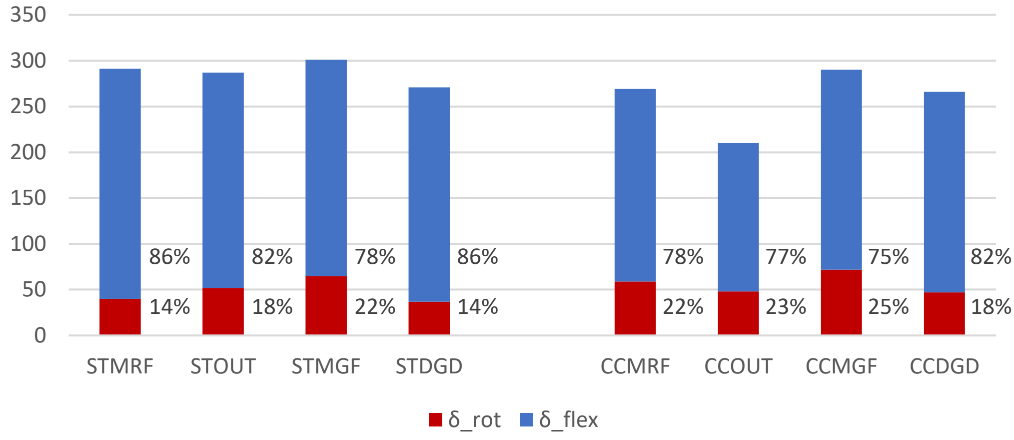


Figure 35:Building deflection in [mm].
Displacement due to the rotation of the foundation in Red.
Displacement due to the structure's deflection in Blue.

6.2.2. MASS

Table 7 presents a summary of the mass that result from the different structural systems. These values are divided on four structural elements types to analyse them.

	ton	Foundation Piles	Hollow Core Slabs	Concrete Core	Steel Structure
STMRF	Steel Core Moment Resistant Frame	4279	10634	0	7452
STOUT	Steel Core Outrigger	4114	10634	0	6973
STMGF	Steel Core Megaframe	3785	10634	0	6092
STDGD	Steel Core Diagrid	3291	10634	0	3927
CCMRF	Concrete Core Moment Resistant Frame	4114	10634	5337	4643
CCOUT	Concrete Core Steel Outrigger	4114	10634	5337	3887
CCMGF	Concrete Core Megaframe	3950	10634	5337	3677
CCDGD	Concrete Core Diagrid	3620	10634	5337	2809

Table 7: Mass Summary of all the structural systems. [ton]

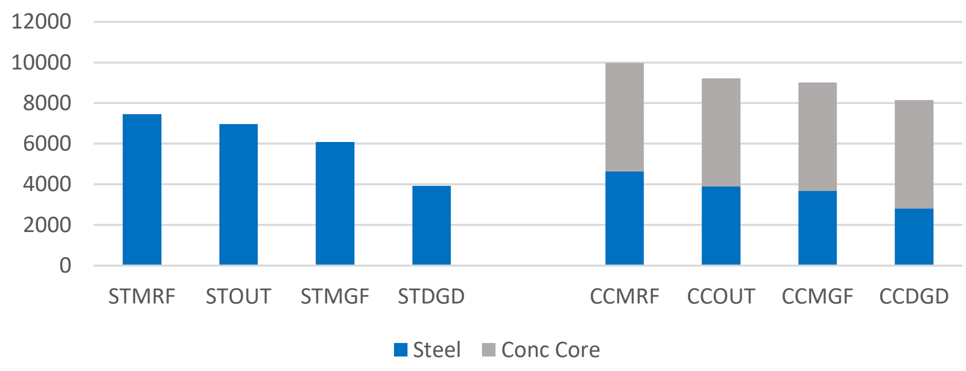


Figure 36: Mass Summation of the Superstructure of the different structural systems [ton]

Figure 36 presents the mass profile of the superstructure of the different systems. The weight of the slabs was taken out of the comparison because it was considered the same for all the systems.

The structures with a concrete core are heavier than the analogue structures with steel core. The Moment Resistant Frame is 34% heavier, the Outrigger 32%, the Megaframe 48% and the Diagrid is 107% heavier than the same structural systems but with steel core.

When only the structural systems are compared, considering the same material for the core, the diagrid is the lightest structure. On average the diagrid with concrete core is 15% lighter than the other solution with concrete core. For the steel options, the diagrid is also the lightest system, which is on average 70% lighter than the other three structural systems. Furthermore, if the steel diagrid is compared to the concrete solutions, it is on average 131% lighter.

Figure 37 presents the mass of the structural systems of the buildings (Core and steel structural elements) and the total displacement of each system. The structures that have an exterior stability system (diagrid and megaframes) are lighter than the structures which have an internal one (moment frame and outrigger).

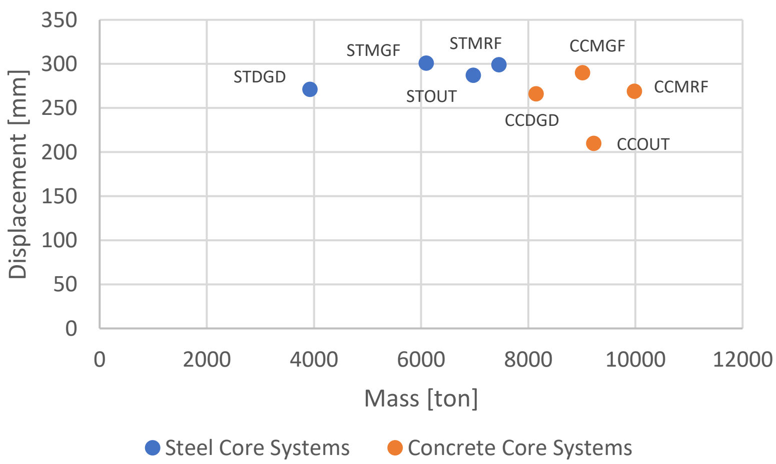


Figure 37: Stability system mass and total displacement.

6.2.3. FOUNDATIONS

On addition to the superstructure weight, Figure 38 includes the weight of the foundation for all the structural systems analysed. The weight of the slabs is not considered on the comparison since it is considered the same for all the systems.

The average value of the foundations' mass is 3,908kg (+/- 10%) except for the foundation of the steel diagrid system (3,291kg), which is 16% smaller than the average value.

On average, the mass of the foundations is equal to 44% of the weight of the structural systems with concrete core. For the systems with steel core its weight is equal to 60% of weight of the structure, except for the steel diagrid system where its weight is equal to 84% of the superstructure's weight.

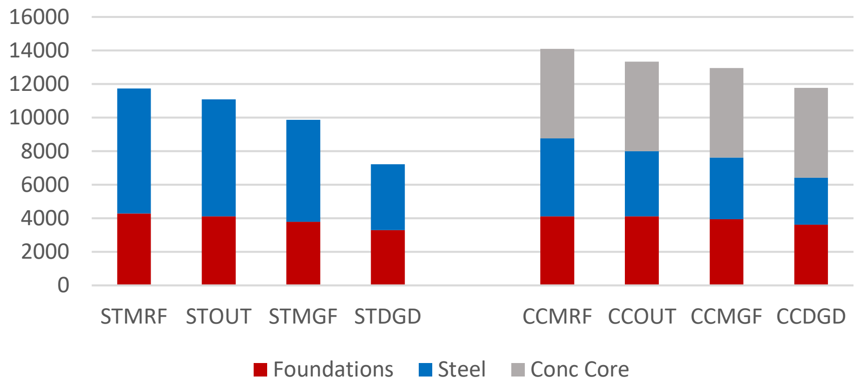


Figure 38:Mass of the Superstructure and Substructure [ton]

6.2.3.1. *MASS DIFFERENCES*

In general, the weight of the superstructure can be significantly reduced by using steel cores instead of concrete cores (Figure 36). However, even if there is a considerable reduction on the superstructure, the total mass of the foundations does not vary substantially.

The case which benefits more from the reduction of weight, by using steel instead of concrete cores, is the diagrid which's weight is reduced by 52% (Figure 36). However, the weight of its foundations was only reduced by 9%, when steel was used instead of concrete on the diagrid systems (Figure 39).

This small reduction in the weight of the foundation results from the fact that the weight of the superstructure only contributes to a small fraction (around 20%) of the total vertical loads. The main share (around 80%) corresponds to the contribution of fixed loads such as Live Load, Additional Dead Load and Floor Slabs. Therefore, by changing a structural system or by optimizing the structure, the benefit of reducing the weight will affect only the ~20% of the share (Figure 40).

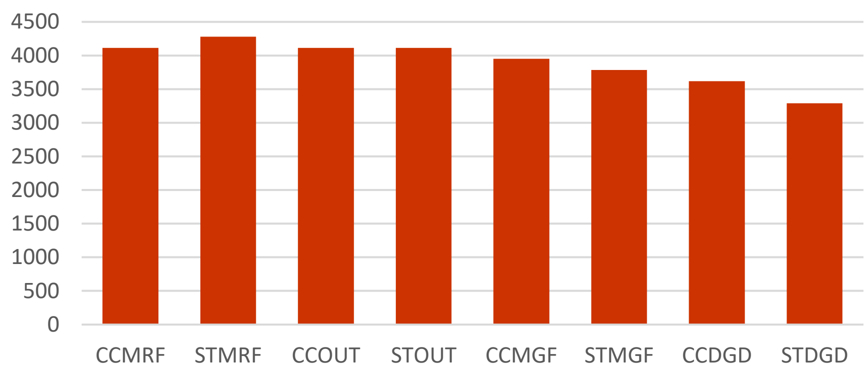
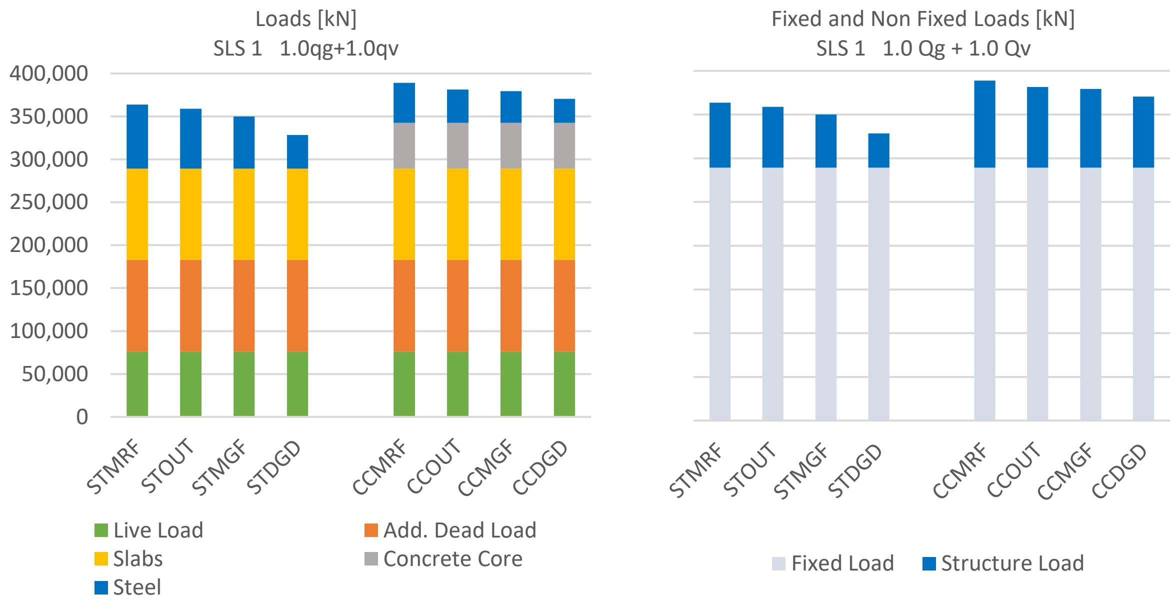


Figure 39: Foundation Mass [ton]



a) Gravitational Loads according to SLS

b) Fixed and Variable gravitational loads

Figure 40: Gravitational Loads Share

The use of Hollow Core Slabs can result in a demountable structural system. For this reason, this study considers this type of floor slabs for all the structural systems. The contribution from these elements to the total gravitational loads is around 30%. The use of different floor types or different geometries, according to the specific requirements of each system, may result in different contributions to the vertical load from these elements.

All the contributions previously addressed correspond to the Load Case SLS1. However, a similar trend and contribution ratios occur on the other load combinations analysed.

To study the relationship between the different structural systems and the number of foundations and their location, three distinctions are defined:

1. Contribution from Vertical Gravitational Forces: The foundations must resist the vertical reactions due to the gravitational forces.

2. Contribution from Bending Moment due to Horizontal Forces: In addition to the gravitational forces, the bending moment at the base of the building generates vertical reactions at the supports.

3. Contribution from Horizontal Reactions: Their contribution, even if not neglectable, is very small in comparison to the vertical components, Therefore, these forces will not be addressed in this study.

Said contributions were addressed by separating the number of piles needed for each of the first two contributions. The following table contains the summary of the piles needed for the cases when a) all Load Combinations were considered, and b) only Load Case ULS1 was considered (thus, only the most onerous gravitational requirements). The fourth column contains the difference between the first two rows, which corresponds to the number of piles required to support the lateral wind load. Finally, the fifth and sixth column contain the percentage of the piles required for the vertical and horizontal requirements.

	Total Piles	Piles for Vertical Loads	Piles for Horizontal Loads	%VERT	%HOR
STMRF	104	76	28	73%	27%
STOUT	100	64	36	64%	36%
STMGF	92	68	24	74%	26%
STDGD	80	62	18	78%	23%
CCMRF	100	78	22	78%	22%
CCOUT	100	68	32	68%	32%
CCMGF	96	76	20	79%	21%
CCDGD	88	72	16	82%	18%

Table 8: Number of piles required for the gravitational and wind solicitation.

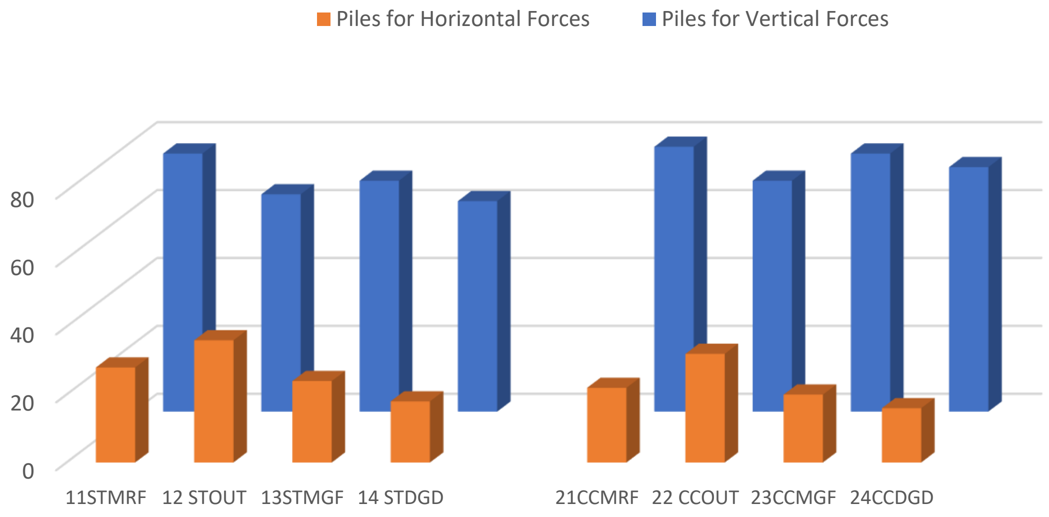


Figure 41: Number of piles for gravitational and wind solicitations.

6.2.3.2. VERTICAL CONTRIBUTION

Most of the foundation's resistance corresponds to the required resistance for the gravitational forces (from 64% to 82%); the remaining available resistance corresponds to the solicitations due to horizontal loads. In the following comparisons, only the foundations required for the gravitational loads are compared (Figure 41).

When using steel instead of concrete core, the number of piles is reduced. 3% reduction (-2 piles) for the Moment Resistant Frame, 6% reduction (-4 piles) for the Outrigger, 11% reduction (-8 piles) for the Megaframe, and 14% reduction (-10 piles) for the Diagrid.

When comparing the structural systems, with the same material conforming the core; first, for the structures with Steel Core the amount of piles required for the Outrigger, Megaframe and Diagrid does not vary significantly among the three of them (+/- 5% of variation or +/- 3 piles), but the Moment Resistant Frame requires 18% more piles (+11 piles) when compared against the diagrid. On the other hand, for the structures with a Concrete Core the amount of piles required for all the concrete systems is less variable among the 4 systems. There is a variation of +/-6% (+/-4 piles) (among the 3 systems) and +8% (+6 piles) for the Moment Resistant Frame when compared with the Diagrid.

6.2.3.3. HORIZONTAL CONTRIBUTION

The external wind load and the surface subject to the load is the same in all the structural systems used. Therefore, there is no difference on the bending moment at the base of the building due to the horizontal loads. The differences among the foundations are the result of the supports disposition. Naturally, the foundations which main supports are located at the façades will require less piles than the ones on which the main supports are located internally (i.e. below the core).

The following comparisons are held only on the 18% to 36% of the contribution that corresponds to the horizontal requirements (Figure 41).

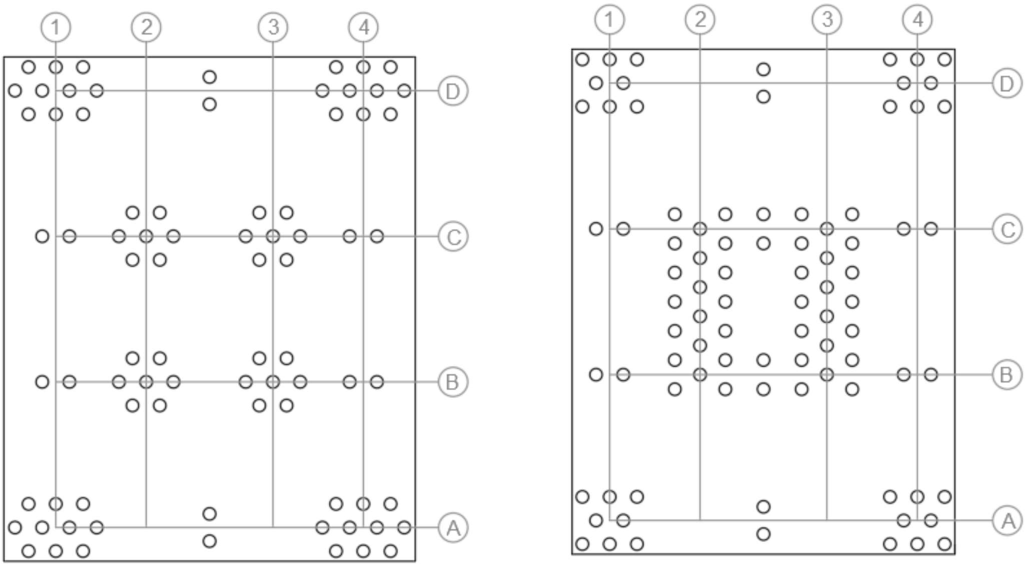
When using steel instead of concrete core, the number of piles is increased: there is a 27% increment (+6 piles) for the Moment Resistant Frame, 13% increment (+4 piles) for the Outrigger, 20% increment (+4 piles) for the Megaframe and 13% increment (+2 piles) for the Diagrid. On the other hand, when the structural systems with the same material on the core are compared, the variability is higher than on the previous case. When comparing the Diagrid system with steel core, against the remaining systems built with Steel Core: the Moment Resistant Frame requires 56% more (+10 piles), the Outrigger requires 100% more (+18 piles), and the Megaframe requires 33% more (+6 piles) than the (18) piles required for the Diagrid. Finally, the trend is similar when the systems built with Concrete Core are compared: the Moment Resistant Frame requires 38% more (+6 piles), the Outrigger requires 100% more (+16 piles), and the Megaframe requires 25% more (+4 piles) than the (16) piles required for the Diagrid with concrete core.

6.2.3.4. PILE DISTRIBUTION

Figure 42 presents the location and distribution of two different structural systems. The location and disposition for the other structural systems can be found on the Appendix E.

For the all the structural systems with concrete core the distribution of piles is similar; on average 46% of the piles are located below the core and 54% are located below the façades and columns supports.

These ratios change significatively for the structural systems with steel concrete core; on average, 38% of the piles are located below the core and 62% are located on the façades. This is not the case for the Moment Resistant Frame with steel Core, where 27% of the piles are located at the core and 73% are distributed over the remaining columns. This last result arises from a more even distribution both of weight and of the forces due to bending moments over all the columns.



a) b)

Figure 42: Foundations and pile Location.
a) Diagrid with Steel Core. b) Diagrid with Concrete Core.

6.3. DISCUSSION

It is complex to perform a standard comparison that englobes all the advantages of each of the possible structural systems since each of the system benefits from different geometrical characteristics. However, this study was performed under simplified conditions which would decrease the number of variable parameter and the results could be evaluated under the same architectural requirements.

If every structural system is addressed individually and thoroughly, it can be concluded that there is room for improvement for each of them. The design and optimization considered in this study is not final, but it corresponds to an adequate point to evaluate the compared options. The improvement of each of the systems was not part of the scope of this research.

6.3.1. DEFLECTION

All the structural systems studied fulfil the SLS and the ULS criteria. Buildings made with these type of stability systems are feasible.

There is not a clear trend that demonstrates a relationship between type of material / deformation and type of structural system / deformation. This is because all the systems were designed with a target function to satisfy the displacement. The higher or lower displacement is a result of the level of optimization achieved on each of the structural systems. For this reason, most of the systems present a similar stiffness ratio (1.0 and 1.1) except for the Outrigger structure with Concrete Core (1.4).

The odd behaviour of the Outrigger with Concrete Core resulted from the designing phase, where the governing requirements were the ULS criteria. As the structural elements' cross sections were increased to fulfil the ULS requirements, the flexural stiffness of the building was also increased, and consequently, the displacement at the top of the building was reduced. Therefore, the deflection at the top of the building was not a governing factor. For the other structural systems this behaviour was not as pronounced as it was for the Outrigger with concrete core.

For the flexural stiffness, the structural systems with concrete core represent a contribution of 78% on average, and the structural systems with steel core contribute to 83%. This behaviour results because the concrete core acts as the main bearing system of the building, thus its smaller lever arm allows it to rotate more. On the other hand, the steel structures' main bearing system is located at the façade, thus, the large lever arm restricts more the rotation of the building. Nevertheless, whichever the structural system is used, the interaction of the substructure and the superstructure makes that these two contributions maintain a certain equilibrium. I.e. a system with interior stability will require more supports at the interior, thus reducing the rotation. Therefore, for this type of soil and for the structural systems studied, the rotational behaviour can be attributed mainly to the soil characteristics, more than to the disposition of the superstructure.

6.3.2. MASS

All the structural systems were optimized to fulfil the displacement criterion (Figure 37). Thus, all the displacements are around the same order of magnitude (210mm to 301mm) while the variation of the weight is higher (3,927 ton to 9,980 ton). In other words, the global stiffness of the building is similar on all the structural systems used, but the amount of material is more variable.

From Figure 37, the structures that have an exterior stability system (diagrid and megaframes) are lighter than the structures which have an internal one (moment frame and outrigger) (-31% for the steel core structures and -11% for the concrete core structures, on average). This is a resultant of the resultant internal forces on these elements. For systems with an exterior stability system the lever arm between the elements results in lower internal forces. On the other hand, for systems with an internal stability system, the smaller lever arm between the elements result in larger forces acting on the elements.

The steel structures are lighter than their analogue structures with concrete core (on average 33% lighter). The highest difference on weight is between the diagrid structures, on which the steel structure's weight is less than half of the weight of the structure with concrete core. Furthermore, the Diagrid structures are lighter than the remaining structural systems (MRF, OUT and MGF) in both cases (on average, -42% for the steel core, and -13% for the concrete core).

The lighter weight of the structures is a resultant of the high resistance of steel which allows elements with smaller cross section, and from the fact that the stability systems of the steel structures is more efficient. (e.g. the stiffness of the diagrid system entirely built with steel is a result of the stiffness from the diagrid (mainly) and from the steel core (in less contribution). This allows small cross sections on the outer façades (as a result of the large lever arm) and the remaining contribution form the core is smaller. On the other hand, for the diagrid system with a concrete core, the stiffness of the system is a result mainly form the concrete core, and in less contributions from the diagrid structure. This results in the heavy weight of the core and light weight of the façades.

6.3.3. FOUNDATIONS

For the specific soil type and the properties of the substructure, the foundation stiffness account for 20% of the deflection at the top of the building (as opposed to 50%, as recommended in the literature). This ratio is variable from project to project. It depends on the soil and foundation properties and, in a lower extent, to the geometry and characteristics of the superstructure. However, the smaller contribution of the displacement due to rotation is beneficial when the superstructure is being optimized.

In general, there is not a substantial difference on the weight of the foundation of the different structural systems (3,908kg +/- 10%). The highest difference is for the foundations of the steel diagrid system (3,291 kg) which is 16% lower than the average foundation weight.

To address the differences in weight it is necessary to differentiate two contributors to the design of the foundation: the gravitational forces and the Horizontal forces. For this study, most of the foundation resistance corresponds to the resistance required to satisfy the gravitational loads, and the remaining resistance is a consequence of the requirements derived by the overturning forces acting of the foundations due to the horizontal loads.

For the gravitational forces, a low weight of the substructure corresponds to a low weight of the superstructure. However, most of the requirements (~80% of all the gravitational load) are a resultant of fixed loads that are considered for the design of the building; e.g. Live load, according to the function of the building; Dead Load, of elements of the buildings such as floor cover, wall partitions, services, etc; and the weight of the slabs, which for this study is considered constant for all the systems. On the other hand, only a small fraction (~20%) of all the gravitational loads corresponds to the weight of the structure. For this reason, even If the weight of the superstructure varies drastically (up to +150% difference of weight between the lightest and the heaviest superstructure), the variation of the weight of the substructure is not as high (up to +30% difference of weight between the lightest and the heaviest substructure). Therefore, the reduction of the weight of the superstructure does not contribute substantially to a reduction of the foundations required to counteract the gravitational loads (+/- 10% weight of the foundation).

Given that the external wind load and the surface subject to the load is the same in all the structural systems used, there is no difference on the bending moment at the base of the building due to the second contributor (horizontal loads). The differences among the foundations arise from the supports disposition. These differences and the material efficiency of the substructure follow the same principle than the material efficiency for the superstructure. I.e. the larger the distance between the foundation supports, the larger the lever arm available to resist the resultant bending moments at the bottom of the building. Consequently, the large lever arm between the supports results in lower reactions acting on the piles. This is the case for structures with the main supports at the façades which require

On the other hand, the structures with the main support located internally (i.e. below the core) have a shorter lever arm between the supports, which results on large forces acting on said supports. The use of an external stability system instead of an internal one can result in a substantial reduction of piles; for instance, the use of a Diagrid structure with steel core requires on average 29% less piles than the other systems. Moreover, the concrete outrigger requires 100% more piles than the concrete diagrid to satisfy the requirements from the horizontal loads.

To conclude with the foundations, there is not a significant reduction or increment of piles when using one system or another, however, the diagrid systems propose an advantage due to their lightweight and large lever arm. Even if the reduction of weight of the foundations does not represent a relevant change on the mass of the whole building, it can represent a considerable benefit when only the foundations are addressed. I.e. the contribution of the variation of the weight of the foundations (+/-288 ton) to the total weight of the structure is on average 3%, but this variation can be up to 19% when only the weight of the foundation is regarded (i.e. the weight of the steel diagrid's foundations is 3,290 ton, and it is 617 ton lighter than the average weight of all the foundations (3908 ton)).

7. ENVIRONMENTAL IMPACT ANALYSIS

The main goal of this thesis is to compare the environmental impact profile of buildings with different structural stability systems. To elaborate this evaluation, a Fast Track Life Cycle Analysis was performed with the data obtained from the structural systems analysed [6. Structural Analysis] and with environmental impact data obtained from Literature [Appendix H].

This analysis was elaborated at two levels: Cradle to gate and Cradle to cradle. The definition and evaluation of these assessments is described in the following sections.

7.1. DEFINITION OF THE LIFE-CYCLE ANALYSIS

To evaluate the Life-Cycle Assessments (LCA), a fast track LCA method was employed. According to Vogtlander [18], the steps to perform this evaluation are:

1. Scope and Goal of the Analysis
2. Functional Unit, System Definition and System Boundaries.
3. Quantify the materials and use of energy
4. Perform the computation and analyse the Data
5. Interpretation of the Results.

The steps, definition and approach used for this research are described in the following paragraphs:

1. The scope of the research is to compare the environmental impact of buildings with the same functional unit and structural requirements, but with different stability systems. The goal is to define how, by using an optimum structural system and a proper choice of materials, the environmental impact can be substantially different (i.e. achieve a lower environmental impact by means of sustainable structural design).
   There are many indicators to address the environmental impact of a building, however, the target value to elaborate this research is the Global Warming Potential, in terms of equivalent kg of CO2 emissions, because it is the indicator which has been studied the most, and thus, there is more available data.
   Appendix H. 6 presents a comparison of the shadow costs against the GWP for an LCA Cradle to Gate. However, this was done only to understand and illustrate the differences that imply using other impact categories. This evaluation was not considered as the main data for the study due to the few information available for the other assessments (Cradle to Cradle Assessments).

2. Functional Unit: All the buildings were designed with the same structural requirements and with the same architectural spaces. The only differences rely on the structural system used and thus, the quantity of material used on each of the different systems.
   The functional equivalent for this study is defined as: “1m^2 of a high-rise building (151 meters high) in Rotterdam, The Netherlands”. The area considered for this functional unit is the total Gross Floor Area of the building.
   The variables of the projects to compare are, as specified in the previous section: the use of different structural system and the use of different materials. Moreover, due to the omission of the occupancy phase of the building and since the impact of the structural components

during a building's use phase is not measurable [16], this study did not used a specific rate of use to consider the life span of the building

System Definition: The structural systems to compare are the same that were used on the structural analysis:

STRUCTURAL SYSTEM	ABBREVIATION
Steel Core + Steel Moment Resistant Frame	STMRF
Steel Core + Steel Outrigger	STOUT
Steel Core + Steel Megaframe	STMGF
Steel Core + Steel Diagrid	STDGD
Concrete Core + Steel Moment Resistant Frame	CCMRF
Concrete Core + Steel Outrigger	CCOUT
Concrete Core + Steel Megaframe	CCMGF
Concrete Core + Steel Diagrid	CCDGD

The structural elements considered in each of these systems are: Foundations, Concrete Core (if applicable), Steel Beams, Steel Columns, Steel Braces (If applicable), Hollow Core Slabs and Fire-resistant coat.

Boundaries: The Analysis covers a cradle to cradle assessment (phases A1-A5, C1-C4, and module D). Use phase (Modules B1-B7), correspond to the phase with the highest impact within the life cycle of the building [7]; nevertheless, the use phase was left out of the study since it is assumed that the influence of this phase does not vary from one system to another. Thus, no relevant data would be obtained from the inclusion of this phase in the analysis (Figure 43).

Additionally, a Cradle to Gate assessment (C2Gt) was performed to evaluate the contribution from the production phase (modules A1 to A3) (Figure 44).

This division was made due to the large difference of the values found in literature and in the Databases. After an exhaustive research It was found that there was not a database which included all the information required. Some values were averaged or extrapolated to complete the missing information from the databases [Appendix H]. For this reason, a comparison of the results from different datasets was done on the Cradle to Gate Assessment.

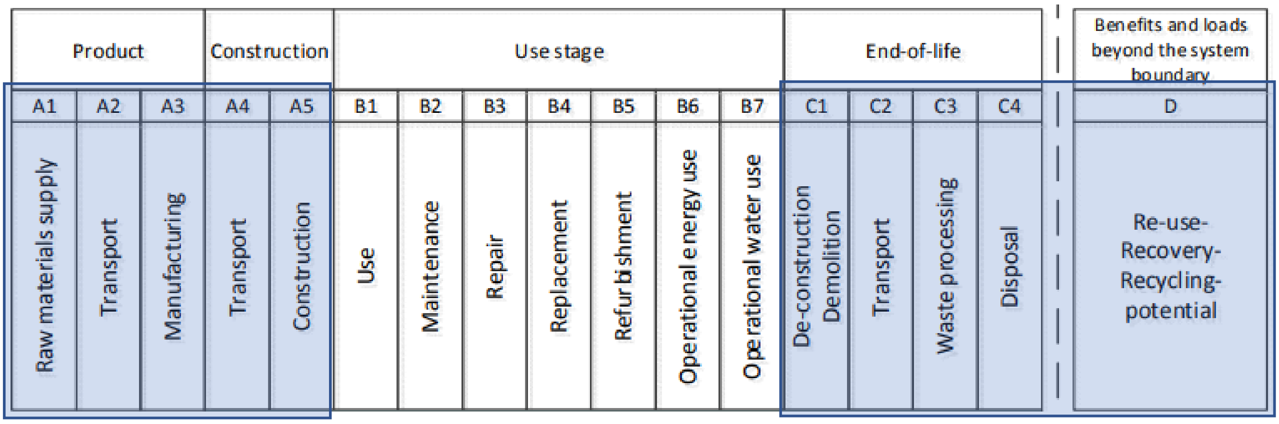


Figure 43: Modules included on the Cradle to Cradle assessment (C2Cr)

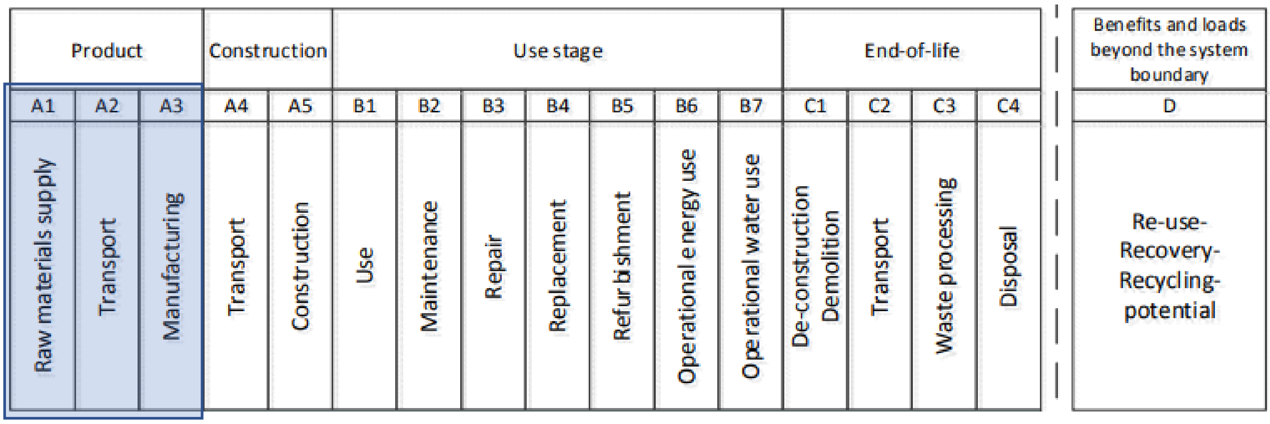


Figure 44: Modules included on a Cradle the Gate Assessment (C2Gt)

3. Materials and use energy:
   The bill of materials used for each of the structural elements is listed on Table 8. The amount of material used (kg/m2 of the building) of each of the structural elements is listed on the Table 10. These values were taken from the results of the structural analysis [6 Structural Analysis] and from LCA data from different sources [Appendix H].

STRUCTURAL ELEMENT	MAIN MATERIAL	SECONDARY MATERIAL
Foundations	Concrete 55/67	Rebar (FeB500)
Concrete Core	Concrete 50/60	Rebar (FeB500)
Steel Columns	S355	Intumescent Paint
Steel Beams	S355	Intumescent Paint
Steel Braces	S355	Intumescent Paint
Hollow Core Slabs	Concrete 50/60	Prestressing Steel (PT)

Table 9: List of Materials per structural element

Material mass / Building area [kg / m2]	FOUNDATIONS		SLABS		CONCRETE CORE		STEEL ELEMENTS	
	Concrete	Steel Reinf.	H.C.Slabs	Prestr. Steel	Concrete	Steel Reinf	Steel	Fire proofing
	C55/67	FeB500	C50/60	PT	C50/60	FeB500	S355	Paint
Steel Core Moment Resistant Frame	140.83	1.68	350.0	4.27	0.00	0.00	245.26	1.79
Steel Core Outrigger	135.40	1.61	350.0	4.27	0.00	0.00	229.50	1.68
Steel Core Megaframe	124.57	1.48	350.0	4.27	0.00	0.00	200.50	1.46
Steel Core Diagrid	108.31	1.29	350.0	4.27	0.00	0.00	129.25	0.94
Concrete Core Moment Resistant Frame	135.40	1.61	350.0	4.27	175.65	3.92	152.81	1.12
Concrete Core Steel Outrigger	135.40	1.61	350.0	4.27	175.65	3.92	127.93	0.93
Concrete Core Megaframe	130.00	1.55	350.0	4.27	175.65	3.92	121.02	0.88
Concrete Core Diagrid	119.14	1.42	350.0	4.27	175.65	3.92	92.45	0.67

Table 10: Materials' mass per building's square meter

Three assessments were performed: Cradle to Gate (C2Gt), Cradle to Cradle (C2Cr) and Cradle to Cradle with 100% Reuse (C2Cr100). The following Sections describe the data used for each of them. Subsequently, the results and analysis of both assessments are presented in the following section.

7.1.1. LIFE-CYCLE ANALYSIS CRADLE TO GATE (C2Gt)

For this assessment it was found that the available data from different sources can be largely variable. For this reason, a comparison of the structural systems, by means of different databases and indicators was performed. These differences arise from the different values used for the local production processes (for instance, the energy efficiency of a blast furnace factory in one country can be significatively different than in another country. Additionally, the resources used to power said facility may come from different sources and in different ratios, i.e. use of electricity generated by wind power or by fossil fuel burning).

The characterization factors used for the materials' Global Warming Potential (GWP) of phases A1 to A3 are listed on Table 11. The values from different sources were used to highlight the differences that may arise from using different databases. The databases used and the sources of said information are briefly described in the following paragraphs.

GWP [kgCO2eq / kg]	Source				
Material	NMD [36]	NMD+MRPI [36],[49]	CTBUH [16]	BCSA [50]	JRC [13]
Structural Steel	1.82E+00	9.08E-01	1.17E+00	2.01E+00	1.69E+00
Reinforcement Steel (FeB500)	1.49E+00	1.49E+00	1.24E+00	1.32E+00	2.13E+00
Prestressing Steel (PT)	1.79E+00	1.79E+00	1.50E+00	1.76E+00	*2.13E+00*
Concrete C50/60	1.12E-01	1.12E-01	1.70E-01	1.70E-01	*1.19E-01*
Concrete C55/67	1.18E-01	1.18E-01	1.70E-01	1.70E-01	*1.18E-01*
Fire Proofing	1.89E+00	2.40E+00	2.60E-01	2.91E+00	2.40E+00

Table 11: Global Warming Potential (GWP) of the materials used.
[Appendix H]

The National Milieu Database (NMD) [36] (Version January 2014), is a Database obtained by the Stichting Bouwkwaliteit from the construction sector in the Netherlands. Also in the Netherlands, the Milieu Relevante Product Informatie (MRPI) [49], provides reliable and quantitative environmental information for building products independently verified in an Environmental Product Declaration (EPD) according to EN15804. Their data is obtained by Bouwen met Staal with national information about steel production and use (the data obtained from this source (Version January 2013) is now part of the database available at the NMD website). The third database reviewed corresponds to The British Constructional Steelwork Association Ltd (BCSA) [50], which is a national organisation for the steel construction industry that promotes the use of structural steelwork. Their database (version 2014) is an assemble of multiple studies and other databases. Another database used was the one obtained from the Council on Tall Buildings and Urban Habitat (CTBUH) [51] which is an international body in the field of tall buildings and sustainable urban design. The values of this database were derived from the research by Trabucco et al. (2016) [16], where the factors were retrieved from the Ecoinvent database, and some were calculated based on the information from the same research. Finally, values from the European Joint Research Centre (JRC) Technical Report[13] were used. This report is part of the EFI Resources research (EFI Resources: Resource Efficient Construction towards Sustainable Design) which goal is to the develop a performance-based approach for sustainable design. One of its goals is to enable benchmarking to enable resource efficiency of building in early stages of building design.

7.1.2. LIFE-CYCLE ANALYSIS CRADLE TO CRADLE (C2Cr)

Similarly than in the LCA C2Gt assessment, the values and contribution of the different processes included on a Cradle to Cradle assessment (C2Cr) are highly variable. These differences arise from the differences on the assumptions and on the modelling of each study. For instance, the building techniques and technologies may be different from one country to another. Additionally, the use of new or old machinery, and the execution times considered for the assembly of the building affect significatively the outcome of the results.

Due to the high variability of the available information found, it was not possible to find a complete database or reference values which included complete information from all the stages of the building's life cycle. Furthermore, it is important to remark that it is not recommended to use values from different databases for the same material during different life phases. The use of different allocation methods on each of the databases makes these factors non-exchangeable among the different databases. By doing so, the end-of-lifecycle values may result in an unbalanced total mass and the possible double count of environmental benefits/loads.

For this reason, the information from the JRC report [13] was used to elaborate a database for this assessment (Table 12), which is the database with more homogenous information, from the sources mentioned. Moreover, the transparency of the report allows to choose the data with more certainty. This information contains the characterization factors for concrete and steel in all the life cycles of the materials. For some materials it was necessary to extrapolate the information and calculate the characterization factor due to the incompleteness of the data. These assumptions and calculations can be found in Appendix H.

Structural Element	Material	A1-A3	A4	A5	C1	C2	C3	C4	D
Foundations	C55/67	0.19000	0.00140	0.00070	0.01500	0.02840	0.00189	0.00481	-0.00192
Foundations	FeB500	2.13000	0.02840	0.00070	0.05250	0.02840	0.00189	0.00481	-0.58600
Floor Slabs	C50/60	0.18028	0.02840	0.01989	0.00702	0.02840	0.00189	0.00481	-0.00192
Floor Slabs	PT	2.13000	0.02840	0.01989	0.02457	0.02840	0.00189	0.00481	-0.58600
Concrete Core	C50/60	0.18000	0.00140	0.00033	0.00702	0.02840	0.00189	0.00481	-0.00192
Concrete Core	FeB500	2.13000	0.02840	0.00033	0.02457	0.02840	0.00189	0.00481	-0.58600
Structural Steel	S355	1.94350	0.02840	0.01989	0.02457	0.02840	0.00242	0.00160	-0.40200
Structural Steel	Paint	2.40000	0.02840	0.01989	0.02547	0.02840	0.00242	0.00160	0.00000

Table 12: GWP values [kgCO2eq/kg] for the Cradle to Cradle scenario (LCA C2Cr)

The values for module D of Table 12 are negative. This negative value is the benefit of recycling and/or reusing the materials. These values can be derived by different allocation methods. In addition, the use of a specific allocation method also influences the other modules (production and end-of-life). The factors used on this research, were taken from the report from the JRC [13] which considers the allocation of the credits according to the Module D approach of EN 15804. Therefore, this factors already include the recycling and reuse rates according to the average values from the JRC research.

	Recycled Content (Rc)	Reuse and Recycle Ratio (RR)
Concrete	0%	70%
Steel Reinforcement	38%	70%
Structural Steel	64%	98%

Table 13: Rc and RR from the JRC report.

7.1.3. LIFE-CYCLE ANALYSIS WITH 100% REUSE (LCA C2Cr100)

The factors used with the data obtained from the JRC report already include the reuse and recycle rates according to the research, thus the result obtained by using this data corresponds to the average future scenario, were, according to the studies, a defined proportion of the material will be reused, other will be recycled and the rest will be wasted.

In addition to the assessment of the Life Cycle by means of the JRC database, this study implemented the consideration of 100% reuse of the structural steel elements and 100% reuse of the hollow core slabs. The description of this assessment (LCA C2Cr100) is addressed in the following paragraphs and the results are presented in the following section.

The consideration of 100% material reuse scenario is utterly optimistic. This scenario requires that all the elements are in conditions to reuse in a second life of the structure, which is highly improbable due to damages that may occur to the elements during deconstruction and transportation. However, the 100% reuse is considered to obtain the highest possible benefit from reusing the elements. I.e. any other scenario will be between this value and the values obtained from the previous Life-Cycle Assessment (LCA C2Cr).

The characterization factors used for this assessment (LCA C2Cr100) are listed in Table 14. Most of the values are the same as the ones previously used. However, the derivation of the factors which are different is briefly described below. A more thorough derivation of these values is described on the Appendix H.

Structural Element	Material	A1-A3	A4	A5	C1	C2	C3	C4	D
Foundations	C55/67	0.19000	0.00140	0.00070	0.01500	0.02840	0.00189	0.00481	-0.00192
Foundations	FeB500	2.13000	0.02840	0.00070	0.05250	0.02840	0.00189	0.00481	-0.58600
Floor Slabs	C50/60	0.18028	0.02840	0.01989	0.00702	0.03800	0.00000	0.00000	-0.17839
Floor Slabs	PT	2.13000	0.02840	0.01989	0.02457	0.03800	0.00000	0.00000	-1.31943
Concrete Core	C50/60	0.18000	0.00140	0.00033	0.00702	0.02840	0.00189	0.00481	-0.00192
Concrete Core	FeB500	2.13000	0.02840	0.00033	0.02457	0.02840	0.00189	0.00481	-0.58600
Structural Steel	S355	1.94350	0.02840	0.01989	0.02457	0.03800	0.00000	0.00000	-0.69966
Structural Steel	Paint	2.40000	0.02840	0.01989	0.02547	0.03800	0.00000	0.00000	2.40000

Table 14: GWP values [kgCO2eq/kg] for the 100% Reuse scenario (LCA C2Cr100)

According to the Module D method of the EN15804, the total environmental impact of a material, during all its life cycle, can be calculated using the following expression:

$$[(1-R_C)E_V + R_C * E_R] + [(1-RR)E_D] + \left[(RR-R_C)\left(E_R^* - E_V^* * C_f\right)\right]$$

Therefore, to calculate this hypothetical scenario where 100% of the structural elements are reused, the characterization factors for this assessment were calculated as follows:

MODULE	EXPRESSION	COMMENTS
MODULE A1-A3	$[(1-R_C)E_V + R_C * E_R]$	The same precedence of the material will be considered Therefore, E_V, E_R *and* R_C remain the same. Thus, the value remains the same
MODULES C1-C4	$[(1-RR)E_D]$	The reuse rate (RR) will be considered as 100% Additionally, the impacts E_D for the phases C3 and C4 are avoided thus $E_D = 0$ (on those modules) and $E_D = E_D$ on modules C1 and C2
MODULE D	$\left[(RR-R_C)\left(E_R^* - E_V^* * C_f\right)\right]$	The reuse rate (RR) will be considered as 100% Additionally, the impact of recycling is avoided, but an additional impact due to storage and sorting of the element must be applied, thus $E_R^* \neq 0$. The impacts from E_V^* are considered as the impacts to fabricate a structural element, thus, they are equal to the values of module A1-A3

Table 15: Considerations for the characterization factor for the assessment with 100% reuse scenario

7.2. LIFE-CYCLE ANALYSIS RESULTS

This section presents the results of the environmental impacts assessments defined in the previous section. This Section is divided into four parts. First, the results from the LCA Cradle to Gate (LCA C2Gt) are presented; then the Results from the LCA Cradle to Cradle (LCA C2Cr); and the results from the Cradle to Cradle with 100% (LCA C2Cr100) Reuse are addressed. Finally, a comparison between the two LCA Cradle to cradle scenarios is presented. The analysis and discussion of these results is addressed in the following chapter

7.2.1. RESULTS from LCA Cradle to Gate (C2Gt)

Figure 45 presents the total GWP of the production phase when the values from different databases (Table 11) are used to elaborate the LCA.

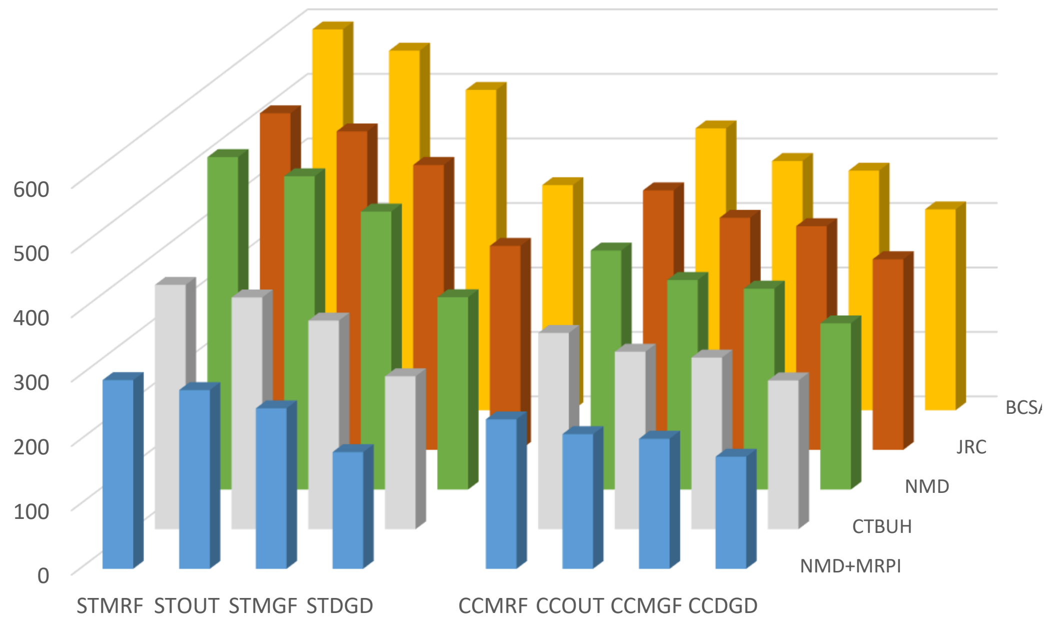


Figure 45: Total GWP [kgCO2eq/m2] by structural system, using different values.

When the results obtained from the evaluation that uses the NMDI+MRPI values are used as a base for comparison, the environmental impacts obtained from the other databases are on average 1.63 (for the NMD), 1.31 (for the CTBUH), 1.91 (for the BCSA) and 1.74 for the (JRC) times higher. These differences arise from the dominant difference of the steel's characterization factors (Table 16).

GWP [kgCO2eq / kg]	Source				
	NMD	NMD+MRPI	CTBUH	BCSA	JRC
Material	[36]	[36],[49]	[16]	[50]	[13]
Structural Steel	1.82E+00	9.08E-01	1.17E+00	2.01E+00	1.69E+00

Table 16: Steel characterization factors

Given that the structure is located in Rotterdam, The Netherlands, the database from the NMD and the MRPI will be used for the following results and analysis. Figure 46 and Table 17 summarize the results obtained with said data. The results from the other databases can be found on the Appendix H.

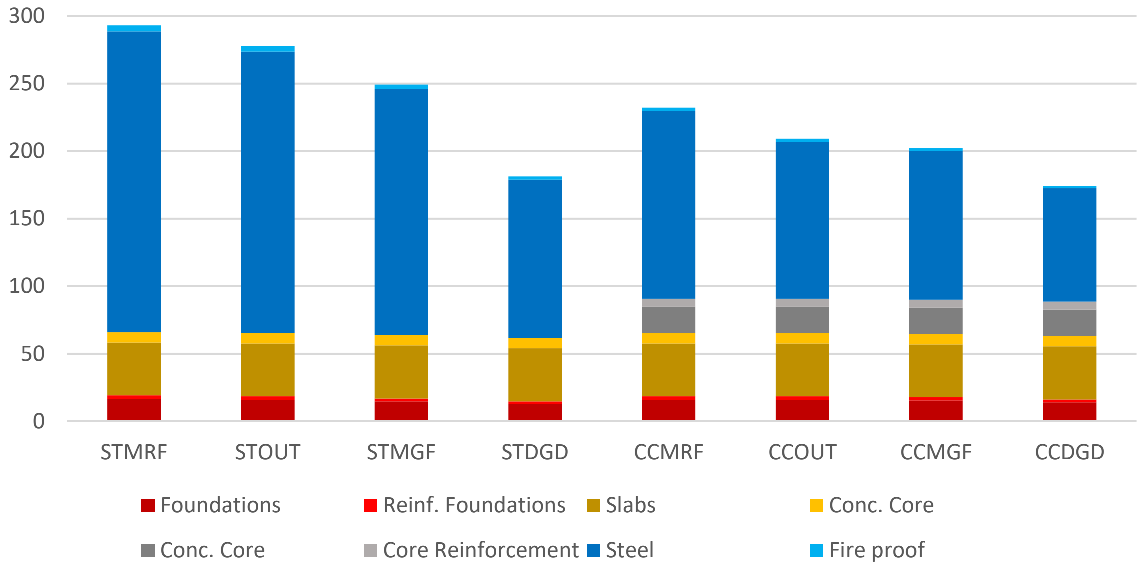


Figure 46: Global Warming Potential [kgCO2eq/m2] of the production phase (modules A1-A3) with data from the NMD and MRPI.

GWP (A1-A3) with NMD+MRPI [kgCO2 eq/m2]	FOUNDATIONS		SLABS		CONCRETE CORE		STEEL ELEMENTS		TOTAL
	Concrete Found. C55/67	Reinf. Found. FeB500	Slabs C50/60	Prestres Steel PT	Concrete Core C50/60	Reinf. Core FeB500	Steel S355	Fire-proof Paint	
STMRF	17	2	39	8	0	0	223	4	293
STOUT	16	2	39	8	0	0	208	4	278
STMGF	15	2	39	8	0	0	182	4	249
STDGD	13	2	39	8	0	0	117	2	181
CCMRF	16	2	39	8	20	6	139	3	232
CCOUT	16	2	39	8	20	6	116	2	209
CCMGF	15	2	39	8	20	6	110	2	202
CCDGD	14	2	39	8	20	6	84	2	174

Table 17: Summary of the GWP values per square meter (modules A1-A3).

In all the systems studied, steel elements (structural steel elements and fireproofing coat) are the highest contributor to the GWP (from 49% to 77% of the total GWP), with an average value of 150.2 kgCO2eq/m2.

The contribution due to the hollow core slabs and its correspondent prestressing steel remains constant for all the structural systems (46.8 kgCO2eq/m2), since these elements were considered constant on all the structural designs.

The foundations have the smallest contribution to the whole impact (on average 17.5 kgCO2eq/m2). Additionally, there is not a substantial difference on the impact of the foundations of the different structural systems studied. Except on both diagrid systems, where the GWP of the foundation is significantly lower (in average -18%) than the GWP of the other systems' foundations.

The environmental impact of the structural systems built with steel core is on average 27% higher than the environmental impact of the structural systems that are designed with concrete cores. Except for the diagrid structure with steel core, which has a comparable GWP value (181.2 kgCO2/m2) to the one of the its concrete core analogue (174.1 kgCO2eq).

When comparing the structural systems with the same core, the exterior stability systems present a lower GWP than the internal stability systems. For instance, the GWP of the Diagrid with Steel Core is 38% lower than the GWP of a Moment Resistant Frame with steel core; and similarly, the concrete analogues of these systems have a difference of 25%.

7.2.2. RESULTS from LCA Cradle to Cradle (C2Cr)

This section addresses the evaluation and comparison of the environmental impacts generated by the eight different structural systems used, when the Cradle to Cradle assessment (LCA C2Cr) was considered.

For the previous analysis, the conclusions were drawn based on the results from the LCA using the NMD + MRPI characterization factors since these values are the correspondent to the Dutch construction sector. However, the information for an LCA with more life stages (i.e. C2Cr) is not available on the free versions of these databases. For this reason, the information from the JRC report, which contains the complete characterization factors for concrete and steel in all the life cycles of the materials, was used.

7.2.2.1. GENERAL RESULTS.

Figure 47 presents the global warming potential of the structural systems studied on the different life cycle stages.

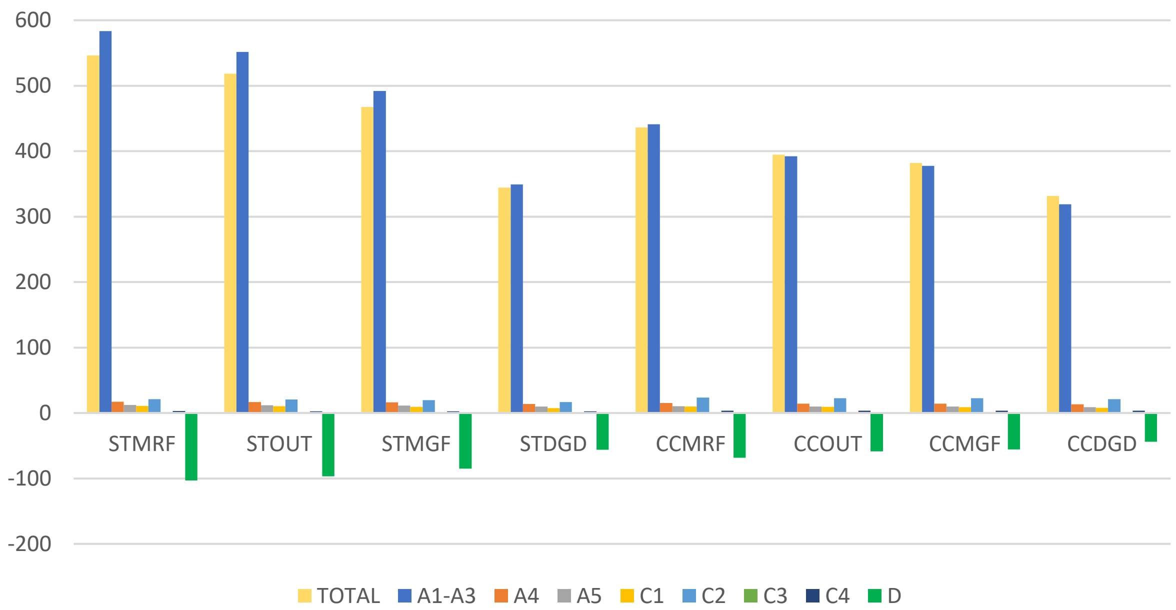


Figure 47: GWP [kgCO2eq/m2] of the structural systems studied, by module (LCA C2Cr)

From Figure 47, the production phase (modules A1-A3) represents on average 105% of the total GWP for the systems built with steel. This ratio is lower for the systems with a concrete core, on average 99%. Additionally, the construction phase (modules A4-A5) represents an average of 6% of the total GWP for all the structural systems. For the impact of the end-of-life phase (modules C1-C4), the average contribution to the total GWP corresponds to 7% and 9% for the systems that consider a core built of steel and concrete, respectively. Finally, the contribution from module D to the total GWP accounts for an average of -18% for the steel core structures, and -15% for the concrete ones.

7.2.2.2. COMPARISON OF THE STRUCTURAL SYSTEMS

Figure 48 presents the impact contributions from each of the modules for all the structural systems studied. When a same system with concrete core is compared with a system with a steel core, the total GWP of the former is 2%(MRF), 31%(OUT), 22%(MGF) and 4%(DGD) smaller than its steel core analogue.

Furthermore when the structural systems compared have the same core, but different structure, the Diagrid systems are the structures with less impact, 344.5 kgCO2eq/m2 for the steel core and 331.4 kgCO2eq for the concrete core, which are in average 33% and 18% lower, respectively, than the other systems.

Table 18 lists the ratio between the benefits derived from module D and the impacts from the production phase for the whole building.

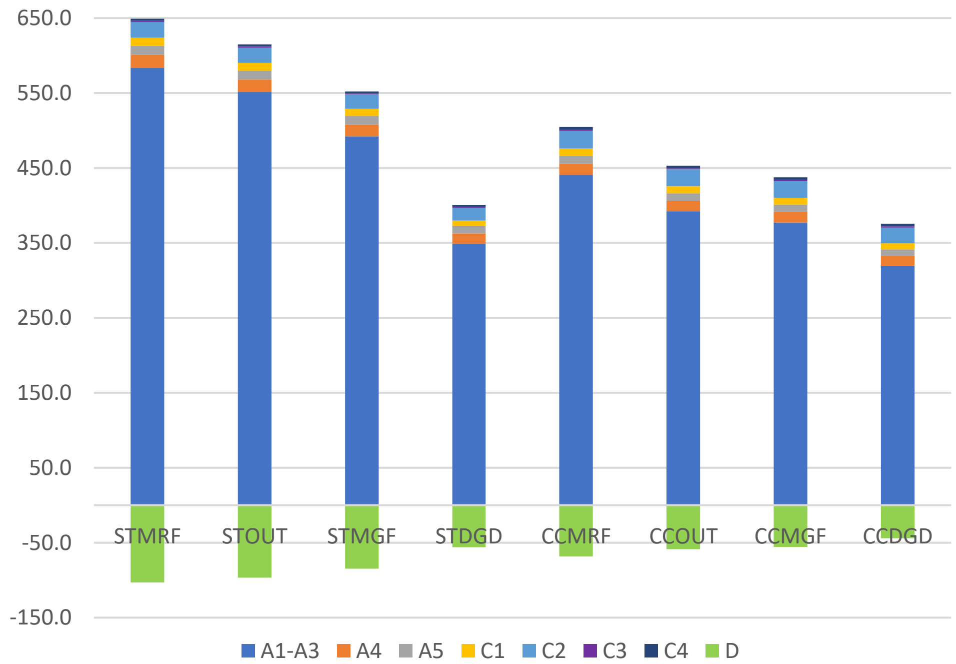


Figure 48: GWP [kgCO2eq/m2] of the 8 structural systems (LCA C2Cr)

Structural System	STMRF	STOUT	STMGF	STDGD	CCMRF	CCOUT	CCMGF	CCDGD
$\frac{Module\ D}{Modules(A1-A3)}$	-17.66%	-17.53%	-17.25%	-16.07%	-15.52%	-14.90%	-14.73%	-13.80%

Table 18: Percentage of Module D compared to the Production Stage (LCA C2Cr).

7.2.2.3. SINGLE SYSTEM RESULTS

To discuss the results of a single system during all its life cycle, the results of the environmental impacts of the Megaframe with Concrete Core are plotted on the following graphs. These impacts account for each of the modules and each of the materials considered by structural element. It was chosen to address this system since it contains a concrete core (thus, the burdens of using concrete in the superstructure are included), and because it represents a system with the average value of the systems which are considered with concrete core. Moreover, the trends of the different systems are similar to the one correspondent to the Megaframe Figure 51. Therefore, even if the results do not correspond to the same magnitude, the results and conclusions drawn from this system are representative for the other systems.

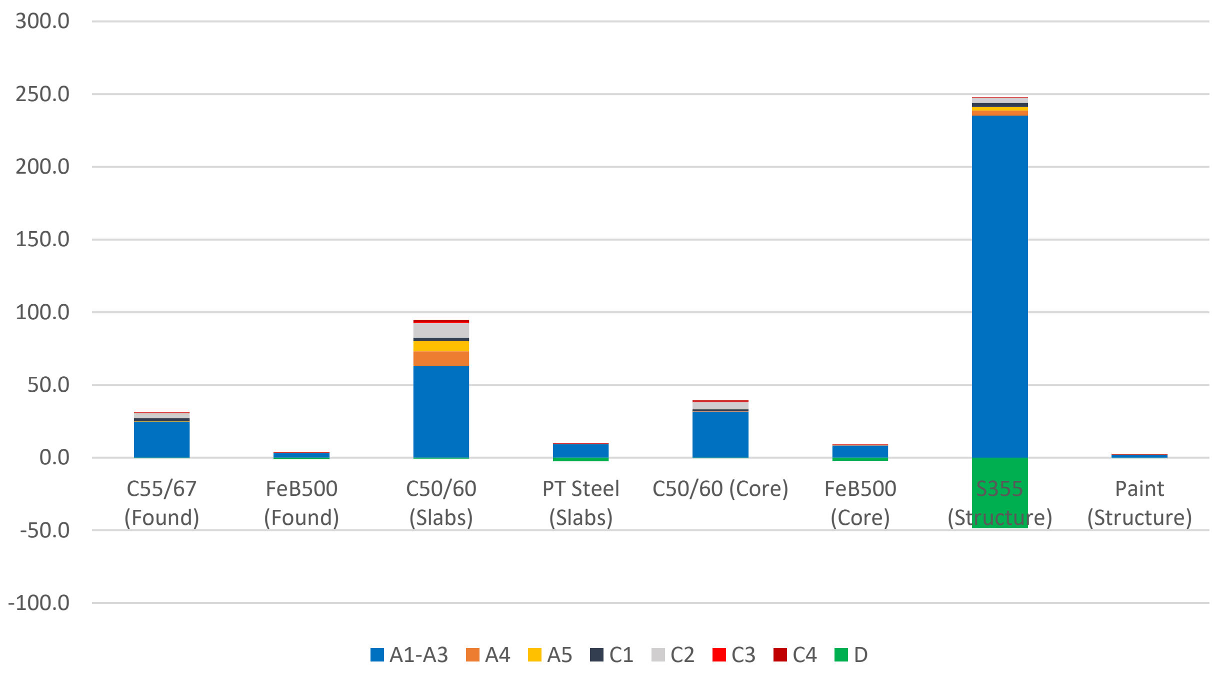


Figure 49: GWP [kgCO2eq/m2] of each material, divided by module. (Megaframe with Concrete Core LCA C2Cr)

From Figure 49, similar to the assessment of C2Gt, structural steel is the main contributor to the total greenhouse gases emissions of the C2Cr assessment (52% of the total). The benefits of including the end-of-life assessment of the whole structure are attributable to the steel elements (from which 87% correspond to the structural steel, 4% to the prestressing steel, and 6% to all the steel reinforcements). The ratio of the benefits of module D and the production phase of structural steel is 21%.

Following steel, the second material with the most impact is the concrete from the hollow core slabs, with a GWP of 94.07 kgCO2eq/m2 (25% of the total), afterwards the GWP of the foundations (33.81 kgCO2eq/m2 including concrete and reinforcement) represents 8.9% of the total global warming potential of the building; and lastly, the GWP of the concrete core (45.38kgCO2eq/m2), including concrete and reinforcement, represents 11.9% of the total impact. Additionally, there are not substantial benefits from the end-of-life phase of these structural elements.

Finally, the benefits of recycling this concrete (considering all the structural elements) account for a total of -1.3kgCO2eq/m2 which represents 2.3% of the total benefits on module D.

Figure 50 represents the results of each of the materials, analysed at each of the modules of the Megaframe with concrete core. From Figure 50 it is remarked once more, yet in a different way, that the contribution from the production phase is the highest of the phases. It accounts for 98.9% of the total GWP. The construction phase (Modules A4-A5 with a GWP of 23.7 kgCO2eq/m2) and the end-of-life phase (modules C1-C4 with a value of 36.2 kgCO2eq/m2) represent a contribution of 6.2% and 9.5% of the total GWP, respectively. And finally, the benefits from including module D (-55.6 kgCO2eq/m2) account for -14.6% of the total GWP.

The previous results correspond to the Megaframe structure with Concrete core, however, these ratios and trends are similar for the other structural systems studied (Figure 51). Therefore, the previous conclusions are illustrative for the rest of the systems used on this research.

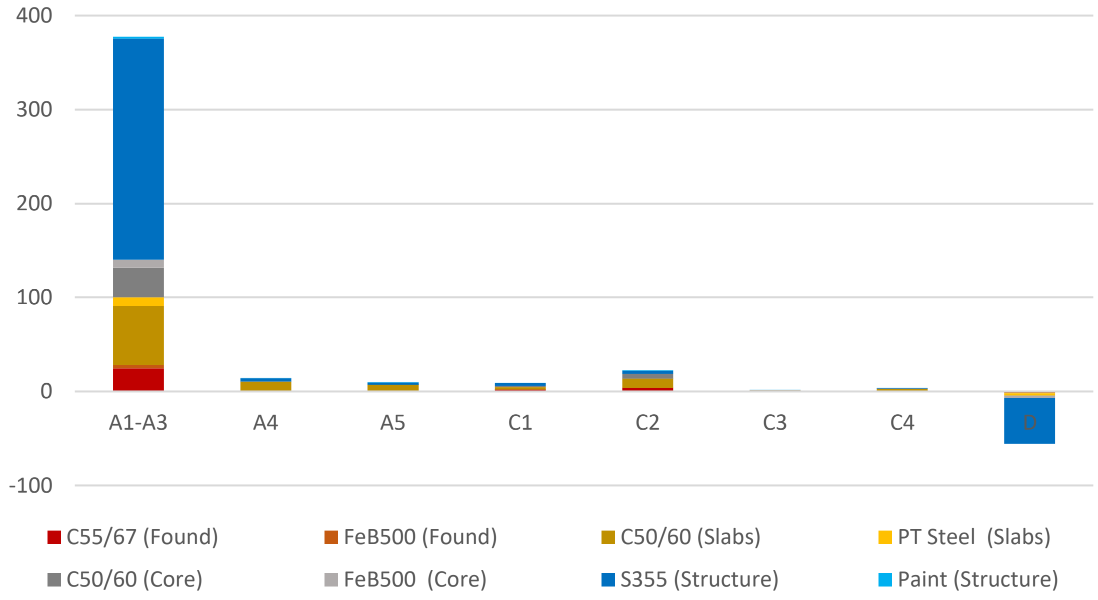


Figure 50:GWP [kgCO2eq/m2] of module, divided by material. (Megaframe with Concrete Core LCA C2Cr)

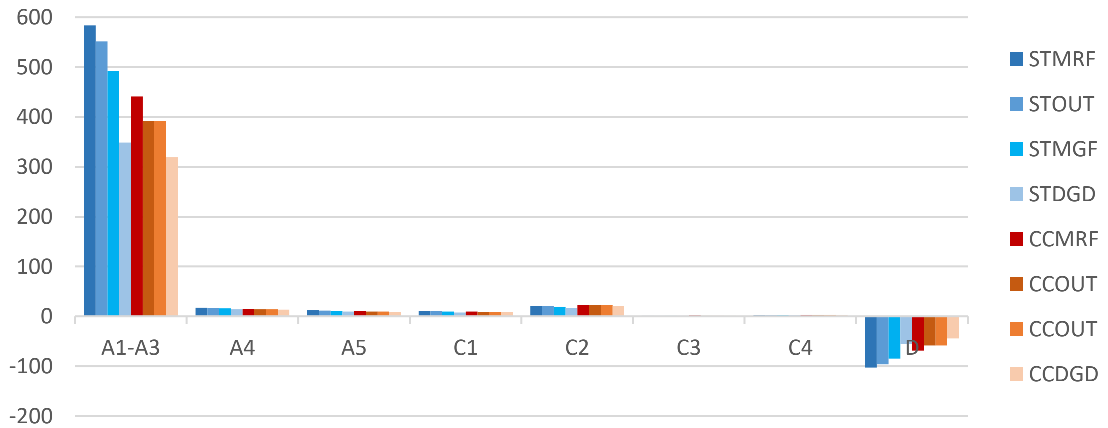


Figure 51: GWP [kgCO2eq/m2] of each of the modules, for the 8 structural systems studied (LCAC2Cr)

7.2.3. RESULTS from LCA Cradle to Cradle with 100% of reuse (C2Cr100)

The data from the JRC research [13] was used to perform a Cradle to Cradle assessment of different structural systems. For this assessment a 100% reuse of the steel structural elements and the 100% reuse of the floor slabs was considered. The scarce practice of deconstruction (deconstruction of high-rise buildings more specifically), the little literature about this phase and the high uncertainty of the materials end and its end-of-life cycle, make the choice of a scenario highly uncertain. However, by considering a 100% reuse scenario, the most optimistic outcome is expected (since higher production costs are avoided): thus, it can provide the upper limit of the environmental impact.

7.2.3.1. GENERAL RESULTS

From Figure 52, the production phase (modules A1-A3) represents on average 141% of the total GWP for the systems built with steel. This ratio decreases to 133% for the systems with a concrete core. Additionally, the construction phase (modules A4-A5) represents an average of 8% of the total GWP for all the structural systems. For the impact of the end-of-life phase (modules C1-C4) the average contribution to the total GWP corresponds to 10% and 13% for the systems that consider a core built of steel and concrete, respectively. Finally, the contribution from module D accounts for an average of -59% for the steel core structures, and -54% for the concrete ones.

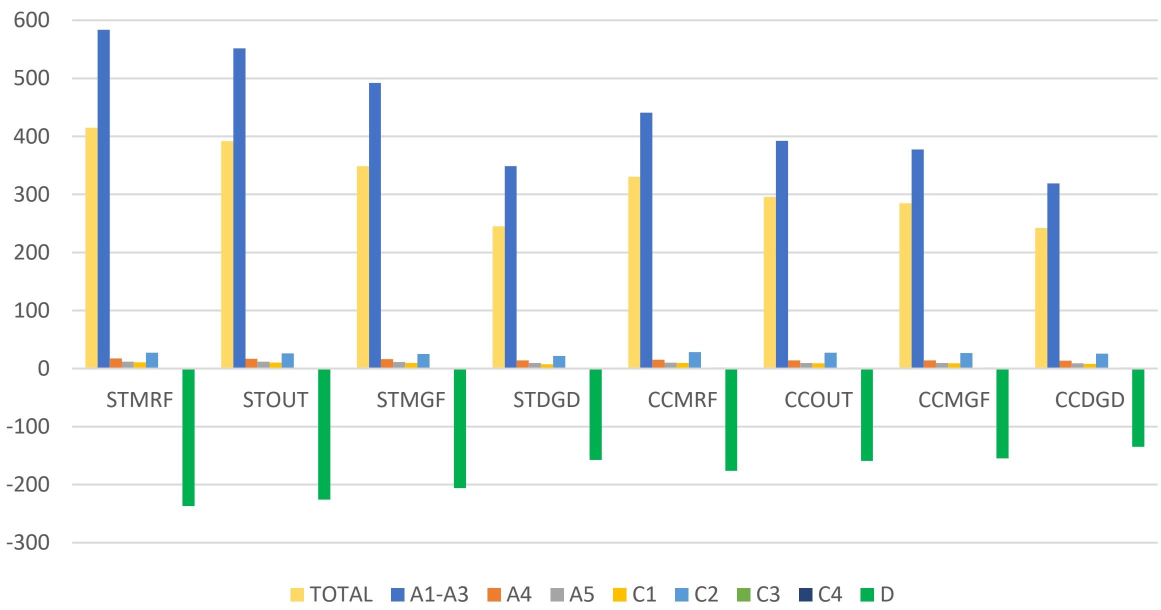


Figure 52: GWP [kgCO2eq/m2] of the structural systems studies, by module (LCA C2Cr100)

7.2.3.2. COMPARISON OF THE STRUCTURAL SYSTEMS

Figure 53 presents the impact contributions from each of the modules for all the structural systems studied. When a structural system is selected, and its two core variants (concrete core or steel bracing) are compared, the total GWP of the systems with concrete core is 26%(MRF), 32%(OUT), 22%(MGF) and 1%(DGD) smaller than the total GWP of the steel core systems.

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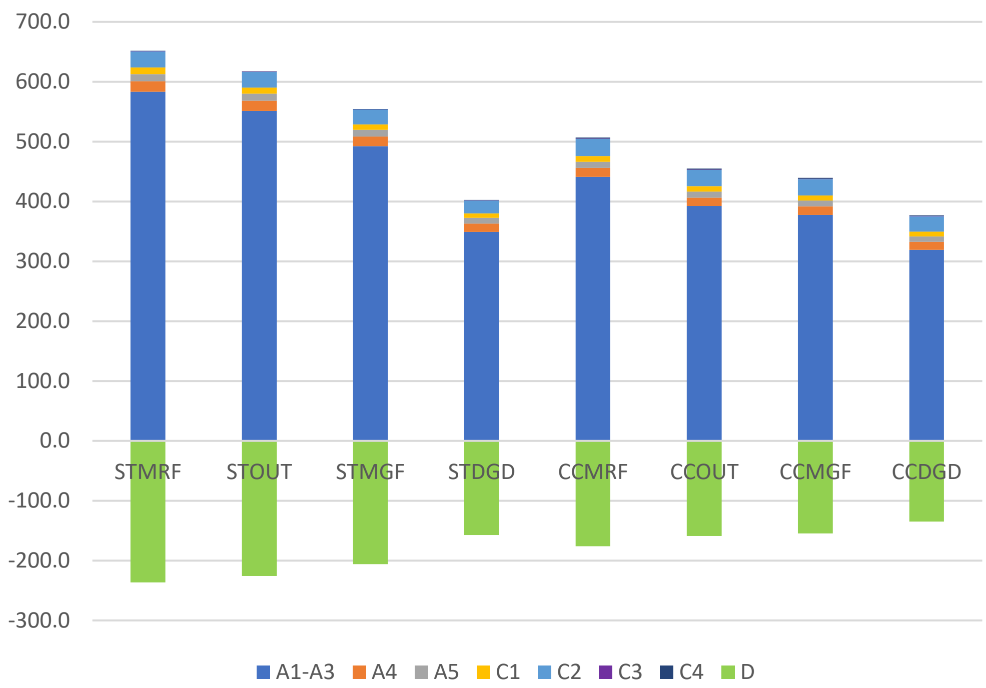


Figure 53: GWP [kgCO2eq/m2] of the 8 structural systems (LCA C2Cr100).

Structural System	STMRF	STOUT	STMGF	STDGD	CCMRF	CCOUT	CCMGF	CCDGD
$\frac{Module\ D}{Modules(A1-A3)}$	-40.55%	-40.95%	-41.84%	-45.05%	-39.94%	-40.59%	-40.91%	-42.25%

Table 19: Percentage of Module D compared to the Production Stage (LCA C2Cr100)

Furthermore when the structural systems compared have the same core but a different stability system, the Diagrid systems are the structures with less impact, 245.2 kgCO2eq/m2 for the steel core and 242.2 kgCO2eq for the concrete core, which are in average 36% and 20% lower, respectively, than the other systems.

Table 19 lists the ratio between the benefits derived from module D and the impacts from the production phase for the whole building

7.2.3.3. SINGLE SYSTEM RESULTS

In the same way that on the previous analysis, the results of the environmental impacts of the Megaframe with Concrete Core will be used to discuss the results of a single system during all its life cycle. These impacts account for each of the modules and each of the materials considered by structural element. It was chosen to address this system because it includes a concrete core (thus, the burdens of using concrete in the superstructure are included), and because it represents a system with the average value of the systems which are considered with concrete core. Furthermore, the trends of the different systems are similar to the one of the Megaframe Figure 56. Therefore, even if the results do not correspond to the same magnitude, the conclusions drawn from this system are representative for the others.

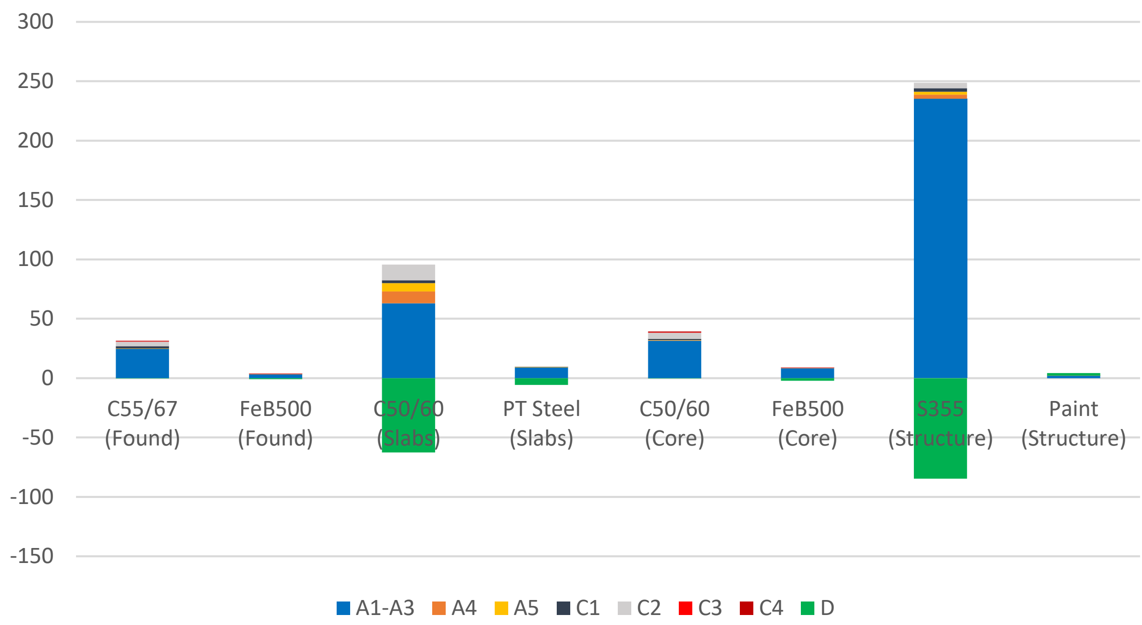


Figure 54: GWP [kgCO2eq/m2] of each material, divided by module. (Megaframe with Concrete Core - LCA C2Cr100)

From Figure 54, structural steel is the main contributor to the total greenhouse gases emissions of the C2Cr assessment (58% of the total). In the other hand, most of the benefits of including the end-of-life assessment of the whole structure (module D) are attributable to the steel elements (from which 54.8% correspond to the structural steel, 3.6% to the prestressing steel, and 2.1% to all the steel reinforcements).

When only the production, construction and deconstruction phases are regarded (modules A1 to C4), the concrete of the Hollow Core Slabs is the second material that impacts the most to the GWP (95.75 kgCO2eq/m2, or 22% of the total). However, by including the benefits from module D the total GWP of this material is significatively reduced (to 33.32 kgCO2eq/m2) and its contribution (11.7%) to the total GWP of the building becomes comparable to that of the other concrete elements. I.e. the GWP of the foundations' concrete is 31.24 kgCO2eq (11% of the total GWP) and the one of the Core's concrete is 38.98 kgCO2eq/m2(13.7% of the total GWP).

Additionally, the inclusion of the post-production processes affects more this material than the others. I.e. the construction stage(A4-A5) is equal to 27% of the value of the production phase, the end-of-life stage (C1-C4) proportion is 25% and the benefits module (D) sum up to -99%.

The foundations represent 11.9% of the total GWP of the building. This percentage is slightly smaller than the one corresponding to the structure with concrete core (15.9%). Additionally, there are not substantial benefits from the end-of-life phase of these structural elements.

For the other elements there are not substantial changes when other modules, in addition to the production phase, are considered; and thus, they were not be addressed.

Figure 55 represents the results of each of the materials, analysed at each of the modules. This graph also corresponds to the result from the Megaframe with concrete with a C2Cr assessment with 100% of reuse. From Figure 55, the contribution from the production phase (377 kgCO2eq/m2) is the highest of the phases. It accounts for 132.5% of the total GWP. Moreover, the construction phase (Modules A4-A5 with a GWP of 23.7 kgCO2eq/m2) and the end-of-life (modules C1-C4 with a GWP of 38 (377 kgCO2eq/m2) phases represent a contribution of 8.3% and 13.3% of the total GWP, respectively.

Finally, the benefits from module D (-154 kgCO2eq/m2) account for -54.2% of the total GWP. On this case, the reuse of steel structural elements accounts for -82.55 kgCO2eq/m2 (53.5% of the total module D) and the Hollow Core Slabs account for -68.07 kgCO2eq/m2 (44.1% of the total module D).

The previous results correspond to the Megaframe structure with Concrete core; thus, these values are applicable only to this system. However, these ratios trends are similar for the other structural systems studied (Figure 56). Therefore, the previous results are representative for all the systems of this research.

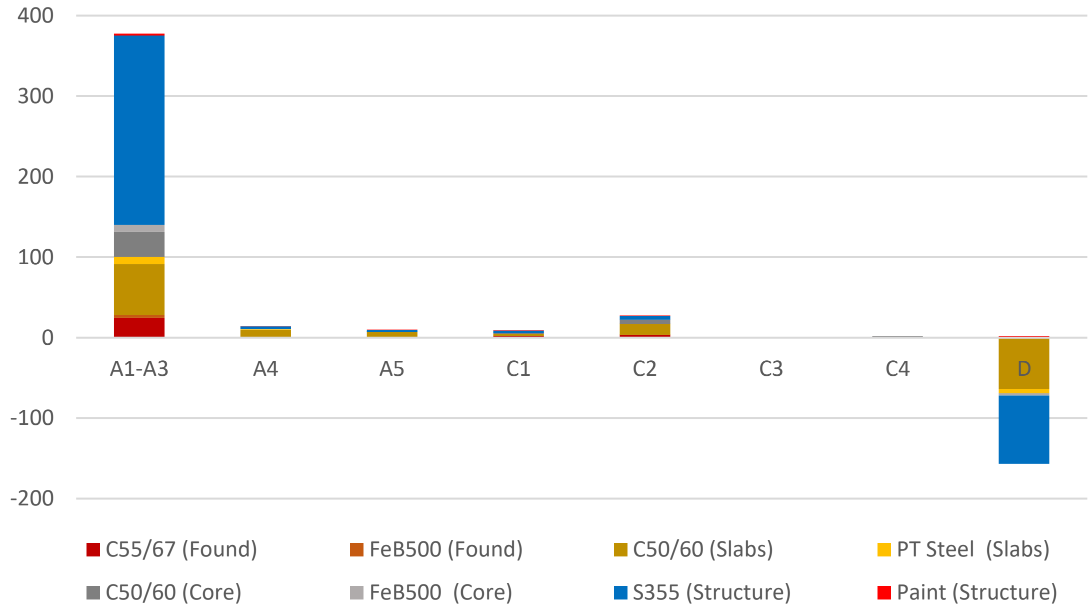


Figure 55:GWP of module, divided by material. (Megaframe with Concrete Core - LCA C2Cr100)

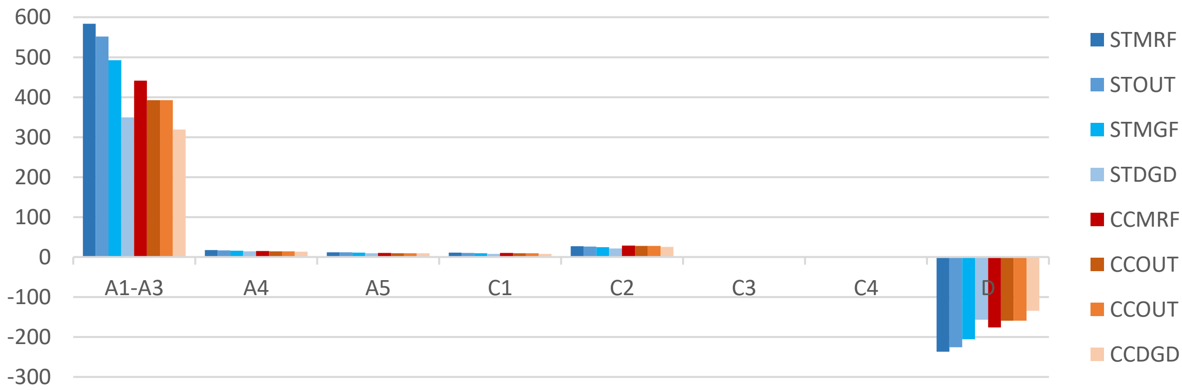


Figure 56: GWP [kgCO2eq] of each of the modules, for the 8 structural systems studied (LCA C2Cr100).

7.2.4. COMPARISON OF LCA C2Cr AND LCA C2Cr100

This section addresses the comparison of the two cradle to cradle assessments.

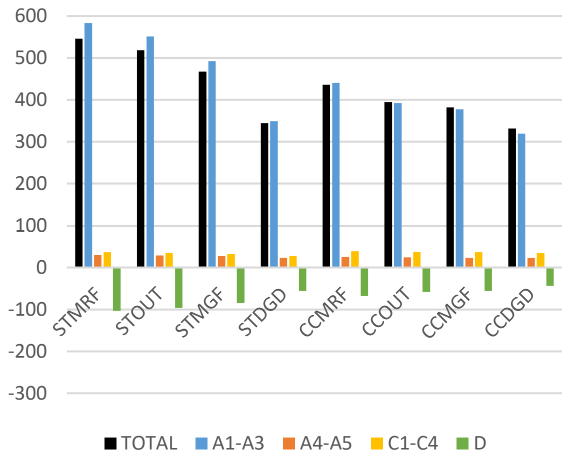
7.2.4.1. GENERAL COMPARISON

Figure 57 depicts the Global Warming Potential from the eight structural systems studied. Figure 57a presents the results from the first cradle to cradle assessment (C2Cr) using the values from the JRC report (Table 12), which represent the average values according to the countries involved on that study as well as the recycling and industry influence on the production of the materials. Figure 57b presents the results from the second cradle to cradle assessment (C2Cr100) which includes a 100% reuse rate for the structural elements (Beams, Columns and Braces) and 100% reuse rate of the floor system (Hollow Core Slabs) (Table 14).

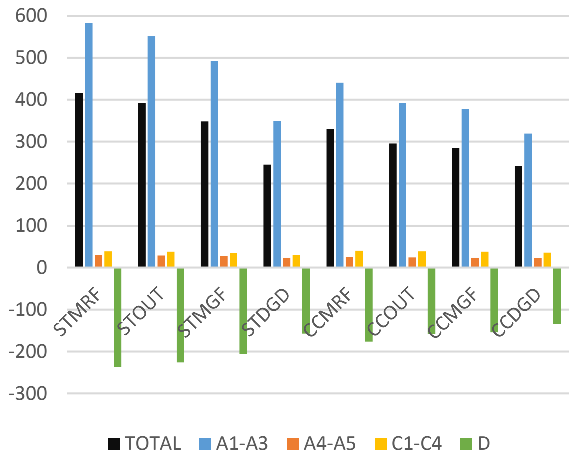
From the comparison of the two graphs, the production and erection phases (modules A1-A3 and A4-A5) have the same values in both assessments. Moreover, for the end-of-life stage (module C1-C4) the results from the second scenario (100% reuse of the elements) are on average 6% higher than the results from the first scenario (7% for steel cores systems and 5% for the concrete core systems). Furthermore, the values of the module D are higher on the second scenario. On average the module D value of the second scenario is -191.27 kgCO2eq/m. On the other hand, the average value for the first scenario assessed is equal to -70.89 kgCO2eq/m2.

Lastly, all the structural systems show a beneficial result when the second scenario is compared to the base scenario. On average, the total GWP of the 100% reuse scenario (319.2 kgCO2eq/m2) is 25% smaller than the GWP that corresponds to the base scenario (427.6 kgCO2eq/m2).

Table 20 presents a comparison of the ratio of the benefits from module D over the modules (A1-A3)



a) GWP from LCA C2Cr



b) GWP from LCA C2Cr100

Figure 57: GWP [kgCO2eq/m2] by structural system

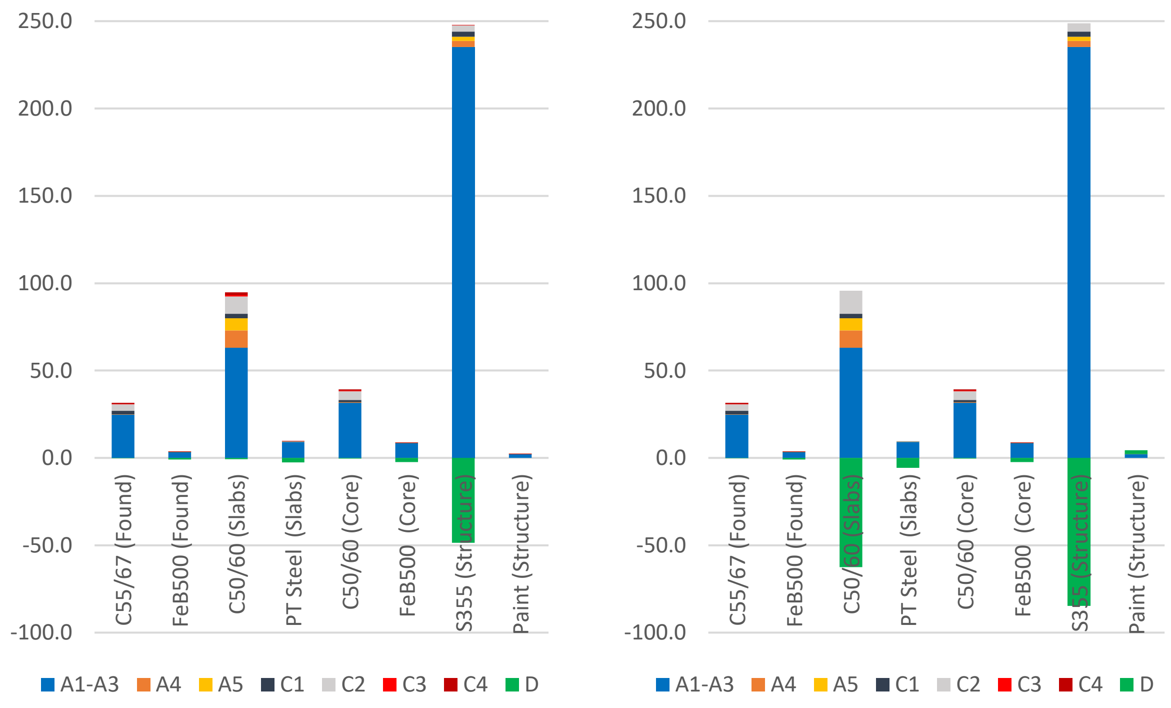
		STMRF	STOUT	STMGF	STDGD	CCMRF	CCOUT	CCMGF	CCDGD
$Ratio\ (LCAC2Cr)$	$\frac{Module\ D}{Modules(A1-A3)}$	-17.7%	-17.5%	-17.3%	-16.1%	-15.5%	-14.9%	-14.7%	-13.8%
$Ratio\ (LCA\ C2Cr100)$	$\frac{Module\ D}{Modules(A1-A3)}$	-40.6%	-41.0%	-41.8%	-45.1%	-39.9%	-40.6%	-40.9%	-42.3%
$Increment$	$\frac{LCA\ C2Cr100 - LCA\ C2Cr}{LCA\ C2Cr}$	+130%	+134%	+143%	+180%	+257%	+172%	+178%	+206%

Table 20: Comparison of the ratios from Module D / Modules A1-A3.

7.2.4.2. COMPARISON OF CONCRETE CORE MEGAFRAME CRADLE TO CRADLE WITH CONCRETE CORE MEGAFRAME CRADLE TO CRADLE WHEN 100% REUSE IS CONSIDERED.

When only a single structural system is analysed, Figure 58 displays that almost all the modules, for all the materials, have the same value. The main difference arises on module D.

The benefits form Module D for the structural steel present an increment of 75% (from -48.6 to -84.7 kgCO2eq/m2); and the benefits for the hollow core slabs present an increment of 2030% (from -3.2 to 68.1 kgCO2eq/m2).



a) GWP [kgCO2eq/m2] of Megaframe with Concrete Core (LCA C2Cr)

b) GWP [kgCO2eq/m2] of Megaframe with Concrete Core (LCA C2Cr100)

Figure 58: GWP of Megaframe with concrete core.

7.3. DISCUSSION

This section addresses an analysis and discussion of the results obtained from the environmental impact assessments. Each of the subsections addresses one of the LCA's proposed.

7.3.1. LCA CRADLE TO GATE (LCA C2Gt)

The LCA Cradle to Gate represents most of the environmental impacts of the lifecycle of the building (when the use phase is disregarded). The large amount of data available regarding this stage makes it easier to evaluate the environmental impact corresponding to the production. However, the characterization factors for this life stage must be evaluated to define if the data is relevant and useful for the scope and goal of the study.

The large differences that arise on the results of the cradle to gate assessment, when different databases are used, are mainly a result of the large variability of the steel's GWP factor among the databases. The value of the steel's GWP presents a variation of +/- 40% from the average value. This large variability of the characterization factor, in addition to the large contribution in weight from this material to the total weight of the building (on average 22.3%), results in large differences of the results, which can go up to a +91% difference in the total GWP value.

The characterization factor of the production phase of steel is mainly influenced by the following factors:

1. The energy to produce virgin material (i.e. blast furnaces), which demands large amounts of energy.
2. The energy to recycle scrap material to produce steel products (i.e. electric arc furnace).
3. The efficiency of the previously mentioned processes.
4. The nature of the energy used for these processes (i.e. fossil fuels, renewable energy, etc.)
5. The quantities of material recycled that is considered in each of the processes.

The different regulation and practices from one country to another may result in different input for these factors, thus, the characterization factor of the steel production is highly variable among different countries. Therefore, it is of paramount importance to define a correct set of values and apply the factors according to the local regulations and practices.

Since the structure in study is located in Rotterdam, The Netherlands, the values used for the analysis of the results of this LCA are the ones that correspond to the NMD+MRPI database. Thus, the following discussion only considers this characterization factors.

Structural steel is the main contributor to the GWP of the building (49% to 77%). This is expected since the GWP/kg of steel elements (beams, columns and braces) is several times higher (+690% on average) than that of the concrete used in the core, foundations and slabs. Furthermore, almost all the structural elements are built with steel; thus, it is expected to have most of the impact as a consequence of this.

The substructure is the structural element that contributes the less to the total GWP of the structural system (with an average value of 17.5 kgCO2eq/m2, which represents 7% of the total GWP). This is due to the small mass of the substructure in comparison to the mass of the superstructure (17.7%) and the small characterization factor of concrete (0.118 kgCO2eq/kg).

The only substantial difference on the environmental impact of the foundations is present on the diagrid systems, where the average GWP of the foundations (13.4 kgCO2eq/m2) is 18% lower than the GWP of the other systems' foundations. This difference is due to the large lever arm between the piles and thus, there is a reduced need for material to resist the reactions. However, this improvement just represents a slight change when the impact of the whole structure is considered.

The average value of the GWP of the structures considered with steel core is 250 kgCO2eq/m2, and the average value of the structures with concrete core is 204.3 kgCO2eq/m2. The benefit of decreasing the weight of the structure by using steel cores instead of concrete core is counteracted by the fact that the value of steel's environmental impact (0.908 kgCO2eq/kg) is around 7.9 times the value of the impact of concrete (0.115 kgCO2eq/kg on average). Despite the weight decrease of around 10% on the MRF, the OUT and the MGF systems, when using steel cores instead of concrete cores; their corresponding environmental impact is 26%, 33% and 23% higher than their concrete analogues. In the other hand, only the Steel Diagrid structure has a comparable GWP profile (181 kgCO2eq/m2) to the one of the its concrete core analogue (174 kgCO2eq/m2).

Finally, the use of buildings with external stability systems presents a lower GWP than the buildings with internal stability systems. This result is correlated to the amount of steel used on each system. For instance, the GWP of the diagrid structure with steel core (181.2 kgCO2eq/m2) is 38% lower than the GWP of a moment resistant frame with steel core (293 kgCO2eq/m2); when the weights of these buildings are compared, the diagrid structure (594 kg/m2) is 25% lower than the weight of the moment resistant frame (743 kg/m2). Where the main difference in weight form these two systems results from the structural steel.

7.3.2. LCA CRADLE TO CRADLE

The previously addressed assessment (LCA Cradle to Gate) represents most of the environmental impacts of the lifecycle of the building (when the use phase is disregarded). However, this phase does not completely consider the benefits of reusing and recycling materials. In addition to the currently mandatory norms to include all the life cycle stages on a product declaration of the building, by including all the life cycle stages, there is an increase of the about the environmental impacts that are present during the whole life cycle of the building. This results in an enhanced culture of recycling and reuse in the building industry. For these reasons it is important to consider all the stages of the life cycle of the building.

The production phase represents most of the contribution of the life cycle of a building, when the use phase is disregarded. On average it corresponds to 99% of the total environmental impact for the systems with a concrete core and 105% for the systems with steel core. Moreover, the construction phase and the end of life phase represent a small portion of the total GWP (on average 6% and 8% respectively). And lastly, the benefits that arise form recycling and reusing correspond to on average 18% for the structures with steel cores and 15% for the structures with concrete cores.

The low contribution from the Construction and the End-of-Life phases highlights two characteristics from these processes:

First, these values are a result of the input data used, which depends on the database used. By considering average values from different researches, there may be differences to the real values at a local level. However, the small contribution from these processes to the total GWP makes that the differences that may exist on the values of these processes do not alter substantially the total GWP of the building. In other words, changing these values to more accurate values due to local processes, will not affect substantially the outcome, unless this change is atypically large.

Secondly, most of the environmental impacts correspond to the production phase and the module D. Therefore, it is more relevant to address the differences that arise from these two processes within the different structural systems. Therefore, special attention must be taken to analyse the values considered for these two modules. Furthermore, the allocation methodology must be consistent during the or all the characterization factors to avoid possible double counting of the loads/benefits of the impacts.

The ratio between these two values (module D and production) yields the ratio of recycling benefits of the whole structure. From the values displayed on Table 18**,** the benefits ratio for the structural systems with steel core is on average 17%, and 15% for the structures with concrete core. The larger value of the steel structures arises from the fact that, there is more steel available to recycle on those systems.

The average value of the total GWP of the systems with concrete core (386 kgCO2eq/m2) is 18% smaller than average value of the GWP of the systems with steel core (469 kgCO2eq/m2). However, the diagrid system has only a 4% difference between the two core variations (344.5 and 331.4 kgCO2eq/m2 for the steel and concrete core, respectively). This is a result of the higher environmental impact per kg of material of steel when compared to concrete.

Moreover, when the structural systems are compared, the diagrid structures are the structures with the lower GWP, which have an average value of 338 kgCO2eq/m2. This lower environmental impact arises from the fact that these structures require less material for the stability system to satisfy the structural requirements, thus, the production impacts are substantially lower.

When only the Megaframe structure with concrete core is analysed, steel is the material that contributes the most to the GWP of the building (199.29 kgCO2eq/m2 which represent 52% of the total). This is due to the production process which demands plenty of energy. Moreover, most of the benefits of including the end-of-life module of the whole structure (-55.6 kgCO2eq/m2) are attributable to the steel elements; from which 48.6kgCO2eq/m2 (87%) correspond to the structural steel, 2.3 kgCO2eq/m2 (4%) to the prestressing steel, and 3.2 kgCO2eq/m2 (6%) to all the steel reinforcement. The ratio of the benefits of module D and the production phase of structural steel is 21%.

The second material that impacts the most to the GWP is the concrete from the Hollow Core Slabs, with a GWP of 94.07 kgCO2eq/m2 (25% of the total). This is mainly due to the volume of the material and not by the production impacts per unit, per se. The production of this material contributes to 67% of its individual share, and the remaining contribution (33%) arises from the handling of the material.

It can be remarked that the inclusion of the post-production processes affects more this material than the others. This results from the consideration of this material as a structural element, rather than just raw material. I.e. the vast weight of the total material multiplied by a high characterization factor due to transportation and assembly of the structural elements is much higher than if the mass was multiplied by the transportation and casting factors of concrete raw concrete.

For this assessment, the foundations represent 8.9% and the concrete core 11.9% of the total GWP of the building. Furthermore, there are not substantial benefits from the end-of-life phase of these structural elements.

Finally, for concrete in general, even if the total mass of concrete represents most of the structure mass (83%), the benefits of recycling this material (-1.3kgCO2eq/m2) do not represent a substantial contribution to the total credits from module D (total of 2%). This is a result of the low production costs and due to the downcycling of the material.

For the Megaframe structure with concrete core, most of the GWP corresponds to the impacts from the production phase 377.5 kgCO2eq/m2, which represent 98.9% of the total impacts. A more thorough analysis of this phase is presented on the previous assessment (LCA C2Gt). The construction phase (Modules A4-A5) and the end-of-life phase (modules C1-C4) represent a contribution of 6.2% and 9.5% of the total GWP, respectively. And finally, the module D accounts for -14.6% of the total GWP. As it was already addressed, steel material accounts for most of these benefits (97.7%, from which 87.5% correspond to the structural steel).

The previous analysis was done considering the Megaframe structure with Concrete core, thus, these values are applicable only to this system. However, given that the ratios and trends of the other structural systems studied, the previous discussion may be illustrative for the rest of the systems used on this research.

7.3.3. LCA CRADLE TO CRADLE WITH 100% REUSE

This assessment considers a cradle to cradle scenario with 100% reuse of the structural elements (structural steel and hollow core slabs). This case scenario is utterly optimistic and not completely realistic. The 100% reuse requires that all the structural elements are in perfect conditions for a second life, which can be difficult to achieve when the building is being deconstructed. However, by considering this scenario, the results include the extreme values that can be achieved by implementing reuse.

The production phase (modules A1-A3) represents most of the environmental impacts of the whole LCA. For the systems built with steel core, it corresponds to an average of 141% of the total GWP. This ratio decreases to 133% for the systems with a concrete core. Additionally, the construction phase (modules A4-A5) represents an average of 8% of the total GWP, and the end-of-life phase (modules C1-C4) corresponds to an average of 11.5%, for all the structural system.

Finally, the contribution from module D accounts for an average of -59% for the steel core structures, and -54% for the concrete ones.

In this assessment, the contribution from the Construction and the End-of Life is relatively small when compared to the impacts form the production and with the benefits of module D. The small contribution from these phases to the whole life cycle makes that the differences that may arise by using different database can be significative at a local level, but they are not relevant when the whole life cycle of the building is regarded (unless this changes are atypically large).

Moreover, most of the environmental impacts correspond to the production phase and the module D. Therefore, it is more relevant to address the differences that arise from these two processes within the different structural systems. The ratio between these two values yields the ratio of recycling benefits of the whole structure. From the values displayed on Table 19, the benefits ratio for the structural systems with steel core is on average 42%, and 41% for the structures with concrete core.

The average value of the total GWP of the systems with concrete core (288 kgCO2eq/m2) is 18% smaller than average value of the GWP of the systems with steel core (350 kgCO2eq/m2). However, the diagrid system has only a 1% difference between the two core variations (245.2 and 242.2 kgCO2eq/m2 for the steel and concrete core, respectively).

Moreover, when the structural systems are compared, the diagrid structures are the structures with the lower GWP, which have an average value of 243.7 kgCO2eq/m2. This lower environmental impact arises from the fact that these structures require less material for the stability system to satisfy the structural requirements, thus, the production impacts are significatively lower.

When only the Megaframe structure with concrete core is analysed, steel is the material that contributes the most to the GWP of the building (164.0 kgCO2eq/m2 which represent 58% of the total). This is due to the production process which a highly energy-intensive process. Moreover, most of the benefits of including the end-of-life module of the whole structure (-54.4 kgCO2eq/m2) are attributable to the steel elements; from which 93.5 kgCO2eq/m2 (55%) correspond to the structural steel, 5.6 kgCO2eq/m2 (4%) to the prestressing steel, and 3.2 kgCO2eq/m2 (2%) to all the steel reinforcement. The ratio of the benefits of module D and the production phase of structural steel is 36%.

For this assessment, the concrete core (45.38 kgCO2eq/m2) is the second most influential element of the structure (16%), followed by the hollow core slabs (37.25 kgCO2eq/m2 which contribute to 13%), and finally, by the foundations (33.81 kgCO2eq/m2 which contributes to 12%).

It can be remarked that the inclusion of the post-production processes affects more the hollow core slabs than the other elements. The ratio of the benefits of module D and the production phase sum up to -99%, Thus almost all the environmental impact can be attributed to the handling of the structural elements like construction, transportation, deconstruction and storage.

For the construction and deconstruction phases: the higher environmental impact is a result of considering this material as a structural element, rather than just raw material. I.e. the vast weight of the total material multiplied by a high characterization factor due to transportation and assembly of the structural elements is much higher than if the mass was multiplied by the transportation and casting factors of concrete raw concrete.

For the benefits of module D: this vast reduction (almost equal to the productions impact) results from considering the highest recycle rate of concrete and the lowest recycled content in the material. This is explained on the following paragraph:

> By considering the Hollow Core Slabs as completely reusable elements the GWP of module D is calculated as follows:
>
> $$(RR - R_C)\left(E_R^* - E_V^* * C_f\right) = E_{C3} - E_{A1.3}$$
>
> With the following considerations for this study:

$RR = 1$	100% reusability
$R_C = 0$	The study does not consider recycled content for the production of concrete.
$E_R^* = E_{C3}$	The impacts of storing and sorting are considered as the impacts from C3 from the JRC research
$E_V^* = E_{A1.3}$	The avoided impact includes the production of concrete and the production of the hollow cores
$C_f = 1$	No downcycle is considered on this study. The recovered elements will maintain their first intended function during their second life cycle.

> Since $E_{C3} \ll E_{A1.3}$, the benefit of considering a 100% reusable floor system results in a value almost the equal to the value of producing it (i.e. Module(D) ≈ Module(A1-A3). Hence, the total environmental impact of this material corresponds only to its handling, assembly and disassembly. This value is certainly changed when the reuse ratio is changed (RR < 1) and when recycled materials are considered on the production process (Rc > 0).

As mentioned before, considering the recycle rate (RR) equal to 100% and the functional equivalence (Cf) equal to 1 is the most optimistic scenario and thus, the extreme value that could be obtained in a the Life-Cycle Assessment.

For the other elements there are not substantial changes when other modules, in addition to the production phase, are considered; and thus, they were not addressed.

For the Megaframe structure with concrete core, most of the GWP corresponds to the impacts from the production phase 377.5 kgCO2eq/m2, which represent 132.5% of the total impacts. This value is higher than 100% because the total GWP considers the benefits from the module D. A more thorough analysis of this phase is presented on the previous assessment (LCA C2Gt). The construction phase (Modules A4-A5) and the end-of-life phase (modules C1-C4) represent a contribution of 8.3% and 13.3% of the total GWP, respectively. And finally, the module D accounts for -54.2% of the total GWP. In this case, the steel contribution to the benefits of module D account for 53.5% of the total and the hollow core slabs account for 44.1%. The description of the large contribution from the hollow core slabs was addressed on the previous paragraphs.

The previous analysis was done considering the Megaframe structure with Concrete core, thus, these values are applicable only to this system. However, given that the ratios and trends of the other structural systems studied, the previous discussion may be illustrative for the rest of the systems used on this research.

7.3.4. COMPARISON OF LCA C2Cr AND LCA C2Cr100

This subsection discusses the results from the comparison of the two life-cycle assessments performed (LCA C2Cr and LCA C2Cr100).

From the comparison of the two assessments, the production and erection phases (modules A1-A3 and A4-A5) have the same values. This is expected since the same factors were used on both scenarios.

Moreover, for the end-of-life stage (module C1-C4) the results from the second scenario (100% reuse of the elements) are on average 6% higher than the results from the first scenario. This results from the environmental impacts that correspond to material waste and disposal, which are higher in the scenario that considers less reusing.

Furthermore, the values of the module D are largely higher on the second scenario. On average the module D value of the second scenario is -191.27 kgCO2eq/m2. Whereas the average value for the first scenario assessed is equal to -70.89 kgCO2eq/m2. This represents an increment of the benefits of module D of +169%. These larger values arise from the avoidance of producing structural elements for the reuse life cycle of the building.

Lastly, all the structural systems show a beneficial result, on the total GWP, when the second scenario is compared to the base scenario. On average, the total GWP of the 100% reuse scenario (319.2 kgCO2eq/m2) is 25% smaller than the GWP that corresponds to the base scenario (427.6 kgCO2eq/m2). Furthermore, when the ratios of module D over modules A1-A3 are compared for the two scenarios the results show an increment of +160% on average.

When only the Megaframe system with concrete core is analysed, almost all the modules have the same value for both scenarios, the main differences are present on module D. First, for the first scenario, the recycle of the concrete elements results in downcycled materials, and thus, the benefits that may arise from this practice are almost neglectable. In the case of steel, its high ratio of recyclability results in a substantial benefit to the environmental impacts. However, due to the high energy demanded on its production phase, the benefits of recycling it do not counteract the burdens of production.

Secondly, for the scenario with 100% reuse, the most substantial benefits arise from reusing steel. However, even if all the steel is being reused, the benefits are only partially increased. This results from the fact that steel production already considers recycled material, thus the benefit of reusing steel must subtract the percentage of steel that was already accounted for recycled content of the material. In this case, it accounted for 64% of the material, thus, the highest value due to reusability it can reach is about 36% of its production impact. Furthermore, the second largest benefit from reusing material arises from the floor slabs. Since this material does not considered recycled content for its production, it is not necessary to reduce its reuse ratio. Additionally, the slabs represent 44% of the mass of the system considered. Therefore, even if the avoided production impact (-62.4 kgCO2eq/m2) is low (16.5% of the total production), this benefits account for 40.4% of all the benefits of module D, for this system.

IV CONCLUSIONS

8. CONCLUSIONS

The presented research addressed the comparison of the environmental impact of the structural systems of high-rise buildings in the Netherlands, by including different stability systems to satisfy the structural requirements and by including different materials to account for the reusability of the structural elements. This objective was approached by means of a multidisciplinary assessment which interlinked the fields of structural engineering and sustainable design. The results and analysis of the assessment provide relevant information to answer the proposed research questions.

SQ1. Which is the most suitable strategy to address and evaluate sustainable structural design of high-rise buildings in the Netherlands?

The Dutch construction sector is adapting a model of circular economy to tackle the challenges that arise from setting the goal of sustainable development. This model implies the development of projects that are resource efficient and that, additionally, maximize the reuse and recyclability of materials at the end-of-life stage of the building.

First, the objective of resource efficiency can be achieved by means of designing adaptable and flexible buildings which can meet the requirements of the buildings along its lifecycle and the possible future requirements that arise from the lifespan of the buildings that overpasses the design life of the building. Moreover, these structures must be optimized to minimize the material use, while also considering structural safety and functionality for the possible future uses of the building. Secondly, the objective of efficient reuse and recycle of materials can be achieved by considering this possibility at the design stage of the project. This avoids the waste generated at the end-of-life stages of the buildings and reduces the need for new materials.

For the building sector in the Netherlands, the high-rise building sector corresponds to structures with a height between 100 and 170m. This sector is growing (there will be 17% more new tall buildings until 2023) and will continue to grow in the following years (with up to 48 buildings proposed for 2023). Furthermore, the voluntary demolition of buildings worldwide represents a small number when it is compared to the total number of buildings being constructed, However, around the globe, there is an increasing trend to deconstruct outdated tall buildings to make space for newer and more modern buildings. This trend of building obsolescence and replacement is inherent to the functional use of the buildings. For this reason, these practices may become relevant for the Netherlands in the coming years.

To conclude, the sustainable design of High-Rise buildings in the Netherlands can be achieved by a design that aims for flexible and adaptable buildings, which can fulfil multiple functionalities over the usable life of the structure. Furthermore, the low probability of deconstruction of the buildings, yields a high uncertainty about the reuse life of a high-rise building; however, by considering deconstructable structures, the benefits of reuse and recycle can be included to decrease the environmental impact of the building, while minimizing future problems that may arise at the deconstruction stage of the building.

SQ2. What is the best way to reduce the environmental impact of the structural system of a high-rise building?

(This question is answered by addressing the production phase of the structural elements, and It is addresses by using the data form the NMD. Refer to the following question for the conclusions that arise from the cradle to cradle analysis, which was addressed with information form the JRC technical report).

There are multiple factors that influence the environmental footprint during the life cycle of a building. The largest impact during the life cycle of a building arises from the use phase (90% to 60% depending on the energy efficiency of the building), where the energy requirements, mainly for electricity and heating, result in a high emission of greenhouse gases. However, this stage is not relevant to the structural stability system of the building. The following highest contribution to the environmental impact of a building arises from the embodied energy of the materials used for the construction (around 85% of the remaining impact). Therefore, it is of paramount importance to maximize the resource efficiency and to consider materials with low environmental impact.

First, in addition to ensuring the structural safety of a building, the structural system of a building plays an important role in defining the quantity of required material. The location and size of the structural elements is directly correlated to the stiffness of the building. Therefore, the sustainable structural design goal is to achieve buildings with the required stiffness to meet the resistance criterion, while minimizing the environmental impact from the production of the structure.

The structural systems conformed by an exterior stability system result in lighter structures than the structural systems with internal stability systems (737 kg/m2 of the diagrid with concrete core and 814 kg/m2 of the moment resistant frame with concrete core). Therefore, the environmental impact due to material production (modules A1-A3) is lower for structural elements with external stability systems (174 kgCO2eq/m2 of the diagrid with concrete core and 232 kgCO2eq/m2 of the moment resistant frame with concrete core).

Moreover, the use of concrete or steel results in a variation on the environmental impact profiles. On average, the structural systems with a steel core have a larger impact (250 kgCO2eq/m2) due to GWP from the production phase, than the structures with a concrete core (204 kgCO2eq/m2). However, this difference is lower for the diagrid structures (181 kgCO2eq/m2 for the steel core variation and 174 kgCO2eq/m2 for the concrete core variation).

Furthermore, the influence of the substructure on the superstructure and its influence on the environmental impact of the whole structure was also addressed in this research. First, by using lighter structures, like the diagrid with steel core, the gravitational reactions to be resisted by the foundations were lower than the gravitational reactions of a heavier structure, like the moment resistant frame with a concrete core. Moreover, the use of external stability systems is benefitted from the large lever arm between the structural elements, which results in a reduction of the forces that act on them. This reduction of forces results in lower reactions and thus, a lower number of required piles. Thus, the structural systems with an external stability system benefit from both, a lighter structure, and lower reactions due to the bending moment at the base of the building. The number of piles (80 piles, and 108 kg/m2) required for a light diagrid structure with steel core (588 kg/m2) which has an external stability system, is (20%) lower than the number of piles (100 piles, and 135 kg/m2) required for a heavier moment resistant frame structure with concrete core (814 kg/m2) which has an internal stability system. However, even if this reduction of piles (20 piles and 27kg/m2) represents a large reduction on the environmental impact at local level (the GWP of the foundations is 14.7 kgCO2eq/m2 for the diagrid structure with steel core, and it is 18.4 kgCO2eq/m2 for the moment resistant frame with concrete core). It does not represent a large influence on the total environmental impact of the whole structure (181.2 kgCO2eq/m2 for the diagrid with steel core, and 232 kgCO2eq/m2 for the moment resistant frame with concrete core).

Summarizing the previous statements, it is concluded that: The inclusion of the substructure into the structural assessment results in a more optimized and accurate design of the structures than if it is left out of the assessment. Furthermore, by including the substructure, the difference in weight between the substructure of the lightest structure (diagrid with steel core) to the heaviest structure (moment resistant frame with concrete core) can account for up to 27 kg/m2 and 3.7 kgCO2eq/m2 of the environmental impacts of the foundations. However, this benefit is not substantial for the whole weight 588 kg/m2 and the total GWP 181 kgCO2eq/m2 of the structure (diagrid structure with steel core).

And finally, that the use of diagrid structures results in light (588 kg/m2 for the steel core and 737 kg/m2 for the concrete core) and efficient structural systems that have a lower environmental impact (181 kgCO2eq/m2 of the diagrid with steel core and 174 kgCO2eq/m2 of the diagrid with concrete core) than the other systems. (In average 250 kgCO2eq/m2 for the rest of the systems with steel core in average; and 204 kgCO2eq/m2 for the rest of the systems with concrete core). Furthermore, the drawbacks that arise from using a material with a higher environmental impact (like steel in comparison to concrete (GWP/kg of material)) are reduced, and the total GWP of both diagrid systems is similar is not as variable (4% variation)

SQ3. What are the benefits of considering reusable structural elements in this method of sustainable design evaluation?

(This question is answered by addressing cradle to cradle analysis which was performed with information from the JRC technical report. Refer to the previous question for the conclusions that arise from the cradle to gate analysis performed with data from the NMD)

When the whole life cycle of a building is being addressed, the module D contains the benefits and burdens that occur from avoiding the production of materials or products after the end-of-life of the building. To define its value, multiple allocation methods can be used. The use of an allocation method should be consistent with the goals and scope of the study. Furthermore, since there are different allocation methods and the derivation of this value is variable from one country to another, it is necessary to evaluate the applicability of the characterization factors to the research.

On this research, the benefits of including the module D on the life cycle assessment result in up to, on average, 85 kgCO2q/m2 for the structures with steel core, and to 57 kgCO2eq/m2 for the structures with concrete core; which account for 17.1% and 14.7% of the environmental impacts from the production phase (in average 494 kgCO2eq/m2 for the structures with steel core and 382 kgCO2eq/m2 for the structures with concrete core). However, since the whole life cycle is considered, the load from construction and deconstruction are also accounted for (on average 6% and 8%, respectively, of the production phase). On average, the total GWP of the buildings for the structures with concrete core is 386 kgCO2eqm2 and 469 kgCO2eq/m2 for the structures with steel core; which respectively represent 96% and 101%, when compared to the production phase of the structures.

The previous results arise from the consideration of average reuse and recycling values from the European market and practices (64% of recycled content and 90% of recycle rate; and 70% concrete recycle rate).

However, when 100% reuse of the structural elements (steel column, steel beams, steel braces and floor slabs) is considered, the benefit of module D is increased to 206 kgCO2eq/m2 for the structural systems with steel core and to 156 kgCO2eq/m2 for the structures with concrete core. These increments of benefit correspond to +142% and +174% increase, respectively, when compared to the base cradle to cradle scenario. The benefit from the 100% reuse of the hollow core slabs is 68 kgCO2eq/m2, which correspond to an increment of +2,028%, when compared to the cradle to cradle scenario. These large values arise from the low recycled content included for the primary production of the material, and from the high volume of the material (354kg/m2, 48% of the total weight in average). On the other hand, the benefit of the 100% reuse of the steel elements increases to 110 kgCO2eq/m2, on average, which corresponds to an increment of 68% when compared to the cradle to cradle scenario. This lower increment results from the high content of recycled material (64%) considered for the production of steel and from its energy intensive production.

Summarizing, it is concluded that: In this assessment, the diagrid structures are again the structural systems with the lower environmental impact, with a GWP of 344kgCO2eq/m2 and 331kgCO2eq/m2, for the steel and concrete core variations, respectively. The rest of the structural systems studied have an average GWP of 457kgCO2eq/m2. And finally, by including structural elements that are 100% reusable, the benefits from module D are increased on average 142% for the structures with steel core and 174% for the structures with concrete core. Finally, the consideration of 100% reuse of the structural elements is utterly optimistic, However, it provides the most extreme value that can be obtained from considering material reuse.

9. LIMITATIONS AND RECOMMENDATIONS

This chapter summarizes the limitations of this research and lists a series of recommendations for further study, derived from the work of this thesis.

9.1. LIMITATIONS

STRUCTURAL ANALYSIS:

- On this research two possible concrete cores were studied: Concrete core and steel braced core. The properties of concrete make it a multipurpose material, were its robustness and its physical existence in the core serves as a barrier to let the space inside the core intact. By using steel, a secondary material must be added to provide said barrier. This material would influence the total environmental impact of the building. However, this consideration was left out of the scope.
- The size of the concrete core was defined by default and non-variable thickness among the different structural systems. The detailed design and optimization of the concrete core may result in a thinner and lighter core.
- This study includes only the use of hollow core slabs for the design of the floors. This was done to consider the demountability of the floor systems.
- The design of the buildings was done with the optimization algorithm from Karamba3D, addressing the serviceability limit state of the global structure and the ultimate limit state of the structural elements. A more thorough and detailed optimization of the structural systems may yield structures with different weight.

ENVIRONMENTAL IMPACT ANALYSIS:

- This research addresses three scenarios for the environmental impact assessment. First, an LCA cradle to gate with data from the Dutch construction sector; Secondly, an LCA cradle to cradle with information taken from a technical report from the JRC which includes data from the average values of the construction sector in Europe; and lastly an LCA cradle to cradle of a hypothetical scenario with 100% reuse, also with information from the JRC report.
- This research addresses only the structural system of the building. Additional requirements and impacts that may arise from considering additional elements are disregarded.
- This research is based only on the global warming potential of the structures. This was done due to the small and inconsistent available data for other impact categories for the environmental impact.
- The use phase of the building was disregarded. For this reason , the functional unit of the assessment does not include a time range. However, for the structural analysis, a period of 50 years was used to define the design.
- The quality of the results is directly dependent on the quality of the databases. As stated before, the databases are sometimes inconsistent which complicates the cross comparison of results. It is therefore important to work with benchmarked data to obtain reliable results.
- Some of the characterization factors were extrapolated form the available databases, this was done due to the lack of all the information required for this research.

- Additionally, the values of the secondary materials were calculated form reference projects to reduce the computation time and precise detailing of multiple structures.
- The values obtained from this research serve as a point of comparison between the structural systems studied. Due to the many assumptions made for this research, and from the particularities of each project, these values cannot be directly extrapolated to other studies. However, the values and results presented, do represent a fair comparison within the boundaries of this research.

OUT OF SCOPE

Considerations due to cost and time of execution are considered on the scope of this study. However, it is important to remark that high costs may derive from the use of hollow core slabs in a high-rise building (i.e. vertical transport becomes more relevant) both for construction and deconstruction of the building. The use of hollow core slabs on the system is with the intention of 100% material recovery with relatively easy construction and deconstruction. Furthermore, the reuse of structural elements also requires inspection, storage and possible remanufacture of some elements, this was left out of the scope, as well as the costs that may arise from this practice.

9.2. RECOMMENDATIONS

- A more thorough design of all the structural system may result in differences. However, with the literature reviewed, and with the results from this study, it is concluded that it is better to focus on structural systems with external stability systems, i.e. address the diagrid or Megaframe structures.
- A more thorough design and optimization of the structural elements should be performed to explore the detailed design and feasibility of the structural systems. I.e. demountable bolted connections and sizes and requirements of the structural elements.
- Additionally, key parts of the building should be investigated, where there is a high concentration of forces (i.e. shear forces on lower floors or core lintels) or where the differential displacements at the same floor may provide structural challenges (i.e. the deflection of the core and the façade at the top floor).
- In this research, the connection between the floor slabs and the façade beams was considered as demountable and rigid enough to transfer the horizontal loads without the need of the screed above the floors. Further research on the detailed design of this demountable system that provides sufficient stiffness should be considered.
- This study considers structural steel S355, the use of stronger materials like S460 may reduce the weight of the steel sections and thus reduce environmental impacts and costs.
- The Structural and environmental response of both diagrid structures was similar. It is of interest to perform a more thorough comparison of both systems, and possibly include other variant like concrete braces and columns.
- Define a modular arrangement of the building, with an external stability system, either in concrete or steel, to be mountable and demountable for easy and fast reuse and relocation. However, consider the level of reusability of steel elements and of concrete elements.
- Consider the foundations in all the structural systems to study. Even if the substructure represents a low contribution to the whole system, it can be easily modelled with spring elements and it can improve the accuracy and detail level of the results. Furthermore, at local level, the use of external stability systems does improve substantially the efficiency of the foundations.

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APPENDIX

A. LOADS

A. 1. VERTICAL LOADS

DEAD LOADS

Self-weight:

The **self-weight** accounts for the weight of the structural members.

Additional Dead Load:

Additionally, according to the architectural floor plan, the next dead loads are also assumed and considered:

1.00 kN/m2 for partition walls.
1.25 kN/m2 for ceiling and floor finishing.
0.25 kN/m2 for mechanical Installations.
2.00 kN/m2 for the façade weight. (For the global model the façade load will be evenly distributed in the total area of the story)

Total Additional Dead Load = **3.52 kN/m2**

LIVE LOAD

According to table 6.1, the category to use for the live load is in the category B. Office Areas.

According to Table 6.2 the uniformly distributed load qk is set to **2.5 kN/m2.**

A. 2. HORIZONTAL LOADS

WIND LOAD

Wind Load according to Eurocode EN.1991.1.4
Eurocode defines the wind load by
$F_w(z) = C_s C_d C_f q_p(z) A_{ref}$

3 Modelling of wind actions.
3.2 Wind action is represented by a simplified set of pressures.
3.3 Wind actions are classified as variable fixed actions.
3.4 The wind actions calculated are characteristic values. Which have a mean return period of 50 years.

4 Wind velocity and velocity pressures.

4.2 Basic wind velocity

Basic Wind Velocity: $V_b = 27\frac{m}{s}$

From:
$V_b = C_{dir} * C_{season} * V_{b0}$ (eq. 4.1)
$C_{dir} = 1, C_{season} = 1, V_{b0} = 27\ m/s$ (NEN Section 4.2, Wind Area II NB.1)

4.3 Mean Wind

Mean Wind at Z=151.2m :

From:
Mean wind Velocity: $V_m = 31.8\frac{m}{s}$
$V_m = C_r(z) * C_o(z) * V_b$ (eq. 4.3)
$z = 151.2, z_0 = 1, z_{0II} = 0.05,\ z_{min} = 10.$ (Table 4.1)
$C_0 = 1$ for H/L < 0.05, $C_r = k_r * \ln\left(\frac{z}{z_0}\right), k_r = 0.19 * \left(\frac{z_0}{z_{0II}}\right)^{0.07}$ (eq. A.1, eq. 4.4, and eq. 4.5)

Mean wind at Z=30m :

Mean wind Velocity: $V_m = 21.5\frac{m}{s}$
$V_m = C_r(z) * C_o(z) * V_b$ (eq. 4.3)
$z = 30, z_0 = 1, z_{0II} = 0.05,\ z_{min} = 10.$ (Table 4.1)
$C_0 = 1$ for H/L < 0.05, $C_r = k_r * \ln\left(\frac{z}{z_0}\right), k_r = 0.19 * \left(\frac{z_0}{z_{0II}}\right)^{0.07}$ (eq. A.1, eq. 4.4, and eq. 4.5)

4.5 Peak Velocity Pressure

Peak Velocity Pressure at Z=151m

Peak Pressure: $q_p = 1{,}509 \frac{N}{m^2}$

From:

$k_l = 1, \rho = 1.25\ kg/m^3$ (NEN Section 4.4, NEN 4.5)

$I_v = \frac{k_l}{C_0 * \ln\left(\frac{z}{z_0}\right)}$ (eq. 4.7)

$q_p(z) = [1 + 7I_v(z)] * \frac{1}{2}\rho V_m^2(z) = 1{,}509 \frac{N}{m^2}$ (eq. 4.8)

Peak velocity Pressure at Z=30m

Peak Pressure: $q_p = 885 \frac{N}{m^2}$

From:

$k_l = 1, \rho = 1.25\ kg/m^3$ (NEN Section 4.4, NEN 4.5)

$I_v = \frac{k_l}{C_0 * \ln\left(\frac{z}{z_0}\right)}$ (eq. 4.7)

$q_p(z) = [1 + 7I_v(z)] * \frac{1}{2}\rho V_m^2(z) = 885 \frac{N}{m^2}$ (eq. 4.8)

5 Wind Actions

5.2 Wind pressure on surfaces

Wind pressure acting on external surfaces w_e and wind pressure acting on internal surfaces w_i should be obtained from equations 5.1 and 5.2 respectively.

The variables for these equations are computed according to Section 7.

7.2.2 External air pressures in the building

External Pressure:

$w_e = q_p(z) * c_{pe}$ (eq. 5.1)

$z_e = 151,$ (Section 7.2.2)

$z_b = 30,$ (Section 7.2.2)

$c_{pe} = c_{pe10}$ (Section 7.2.2)

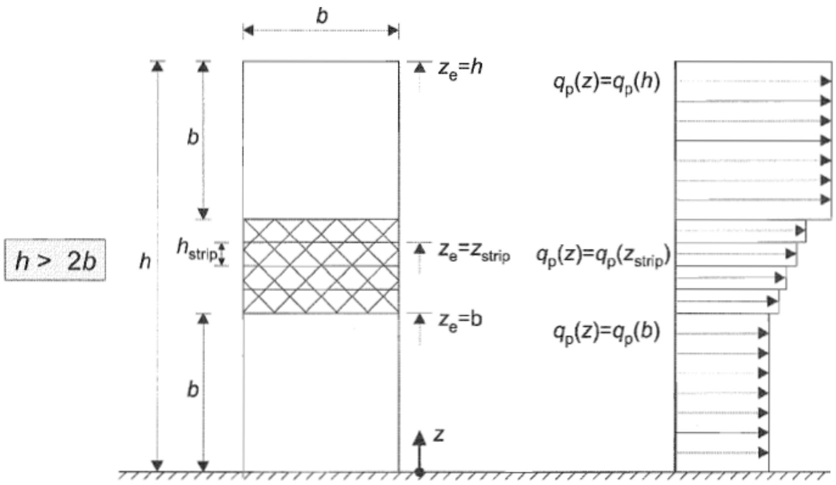
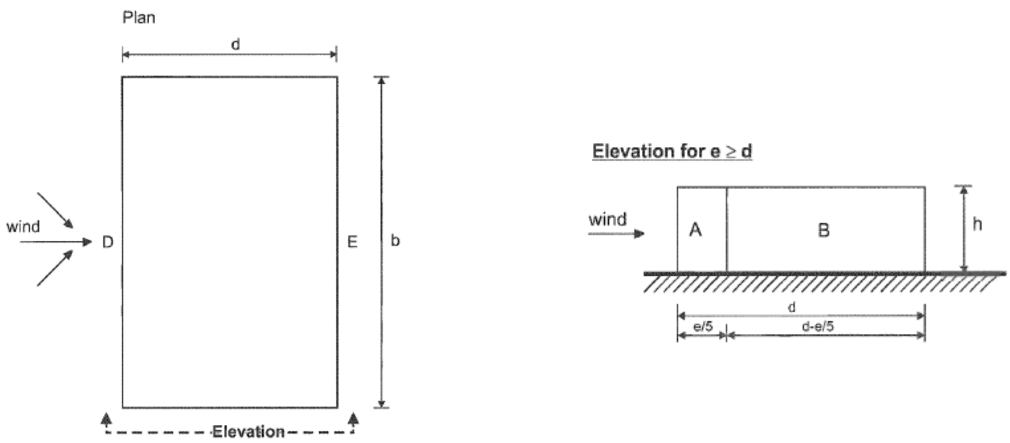


Figure 7.4 - Reference height, Ze, depending on h and b, and corresponding velocity pressure profile

Using h=151m, b=30m. and for the length included in the z_{strip}, a linear trapezoidal load will be used.



Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Table 7.1 - Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

CASE 1 wind in Z direction, D=21.1; h/d=6.7 > 5

CASE 2 wind in Y direction, D=30.0; h/d=4.7 ≈ 5

According to the Table 7.1, The values for h/d = 5 will be used in both cases.

Z=151m:
$w_{eD} = q_p(z_e) * C_{pe10} = 1.5 * +0.8 = +1.20$ kN/m^2
$w_{eE} = q_p(z_e) * C_{pe10} = 1.5 * -0.7 = -1.05$ kN/m^2

Z=30m:
$w_{eD} = q_p(z_b) * C_{pe10} = 0.9 * +0.8 = +0.72$ kN/m^2
$w_{eE} = q_p(z_b) * C_{pe10} = 0.9 * -0.7 = -0.63$ kN/m^2

7.2.9 Maximum and minimum internal air pressures in the building

Internal and external pressures are considered to act at the same time. The worst combination of these pressures is considered.

The internal pressure coefficient, c_{pi} depends on the size and distribution of the openings in the building envelope. Since no faces are considered as dominant, the c_{pi} coefficient is taken as +0.2, and as -0.3; whichever case yields the most onerous scenario.

Internal Pressure:
$w_i = q_p(z_e) * C_{pi}$ (eq. 5.2)

$z_e = 151$ (Section 7.2)
$z_b = 30,$ (Section 7.2.2)
$c_{pi} = +0.2\ or\ -0.3$ since $\mu < 0.3$ (eq. 7.3)

$w_{i2} = q_p(z_e) * C_{pi,2} = 1.5 * +0.2 = +0.30$ kN/m^2
$w_{i3} = q_p(z_e) * C_{pi,3} = 1.5 * -0.3 = -0.45$ kN/m^2

$w_{i2} = q_p(z_b) * C_{pi,2} = 0.9 * +0.2 = +0.18$ kN/m^2
$w_{i3} = q_p(z_b) * C_{pi,3} = 0.9 * -0.3 = -0.27$ kN/m^2

5.2 Total Pressure

According to Section 5.2, and figure 5.1. The external pressure and the internal pressure are added to determine the net pressure acting on the surfaces.

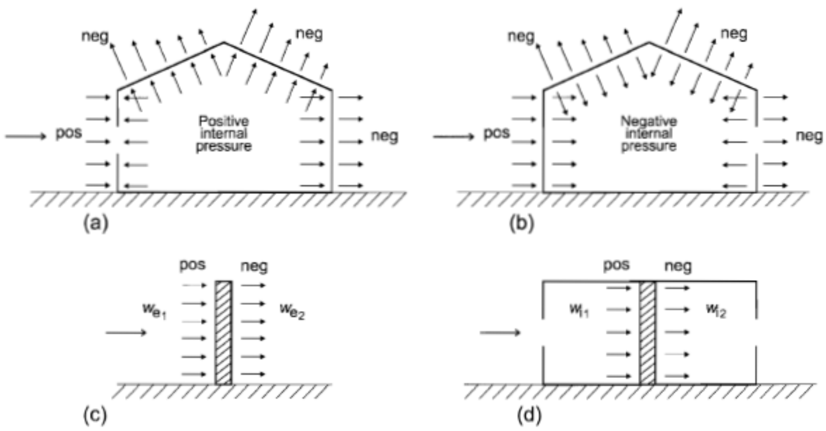


Figure 5.1 — Pressure on surfaces

5.3 WIND FORCES

$F_w(z) = C_s C_d C_f q_p(z) A_{ref}$

The wind forces for the whole structure can be determined by calculating forces from surface pressures:

The wind force F_w may be determined by vectoral summation of the forces $F_{ew}, F_{wi}, and\ F_{fr}$

$(3) F_{we} = c_s c_d * \Sigma w_e * A_{ref}$ eq. 5.5

$F_{wi} = \Sigma w_i * A_{ref}$ eq. 5.6

$F_{fr} = c_{fr} * q_p(z_e) * A_{fr}$ eq. 5.7

according to the point (4) of this Section, the effects of wind friction can be disregarded for this geometry.

Then the wind Force can be written as:

$$F = (c_s c_d * \Sigma w_e - \Sigma w_i) A_{ref}$$

CsCd = 1.07 (extrapolating the value from figure D.1 Section 6)

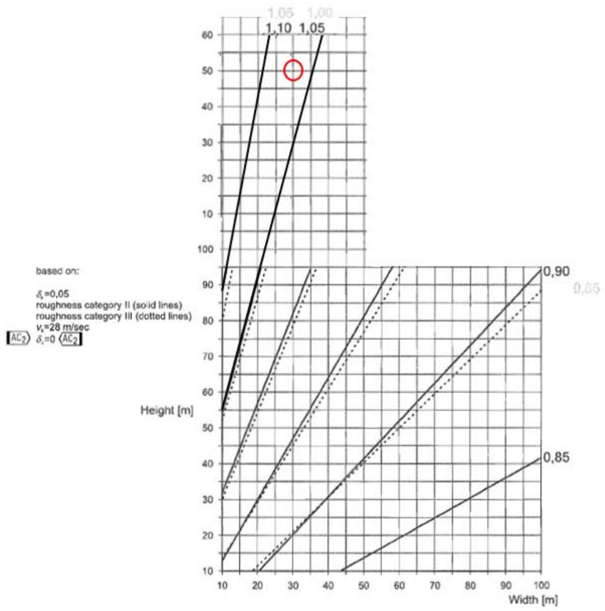
Since the forces will be used as either surface or linear loads, these summations are presented as pressures and will be later applied to the related geometry

6 STRUCTURAL FACTOR CsCd

EN 1991.1.4, Annex D:

From a linear extrapolation of figure D.1:

CsCd = 1.07



NOTE For values exceeding 1,1 the detailed procedure given in 6.3 may be applied (approved minimum value of $c_s c_d$ = 0,85)

Adapted from Eurocode EN.1991.1.4 Figure D.1 - CsCd for multistorey steel buildings with rectangular ground plan and vertical external walls, with regular distribution of stiffness and mass (frequency according to Expression (F.2)).

*Total pressure at Ze = 121m to 151m

$$w_{1D} = +1.20 * 1.07 - 0.30 = +0.98 \qquad w_{1D} = +1.20 * 1.07 + 0.45 = +1.73$$
$$w_{1E} = -1.05 * 1.07 - 0.30 = -1.42 \quad \text{or} \quad w_{1E} = -1.05 * 1.07 + 0.45 = -0.37$$

For a simplified load, that can be represented as a single load that acts on one side of the structure, Pressure D and Pressure E are added.

Total wind pressure = Pressure in D + Pressure in E.
For both cases: Total pressure =2.40 kN/m2 in direction of the wind.

*Total pressure at Zb = 0m to 30m

$$w_{1D} = +0.72 * 1.07 - 0.18 = +0.59 \qquad w_{1D} = +0.72 * 1.07 + 0.27 = +1.04$$
$$w_{1E} = -0.63 * 1.07 - 0.18 = -0.85 \quad \text{or} \quad w_{1E} = -0.63 * 1.07 + 0.27 = -0.40$$

For a simplified load, that can be represented as a single load that acts on one side of the structure, Pressure D and Pressure E are added.

Total wind pressure = Pressure in D + Pressure in E.
For both cases: Total pressure =1.44 kN/m2 in direction of the wind.

*Total Pressure at Z = 30m to 121m

The wind pressure in the area between h=30m and h=121m is represented as a trapezoidal load with a maximum of 2.40 kN/m2 at 121m and a minimum of 1.44 kN/m2at 30m

Loads

Case 1 (Z direction):
Load at 110m: 2.40*21.1 = 50.6 kN/m   Load at 30m: 1.44*21.1 = 30.4 kN/m

Case 2 (Y direction):
Load at 110 m: 2.40*30.0 = 72.0 kN/m   Load at 30m: 1.44*30.0 = 43.2 kN/m

In preliminary design, to consider the second order effects, a common practice is to multiply the loads with a factor 1.2.

Case 1 (Z direction):
Load at 121m: 60.72 kN/m   Load at 30m: 36.48 kN/m

Case 2 (Y direction):
Load at 121 m: 86.40 kN/m   Load at 30m: 51.84 kN/m

A. 3. LOAD COMBINATIONS

To assess the ultimate limit state the Eurocode and the Dutch National Annex, list the following combinations for buildings with class consequence CC3:

$CULS1$: $1.5G + \sum 1.65\psi_0 Q_i$
$CULS2$: $0.9G + \sum 1.65\psi_0 Q_i$
$CULS3$: $1.3G + 1.65Q_1 + \sum 1.65\psi_0 Q_j$
$CULS4$: $0.9G + 1.65Q_1 + \sum 1.65\psi_0 Q_j$

From these combinations, the live and wind loads are interchanged as leading load. This results in eight equations, (two equations for each of the previous four):

Using the values of ψ specified by the Dutch National Annex:

Category B: Office Areas: $\psi_0 = 0.5,\ \ \psi_1 = 0.5,\ \ \psi_2 = 0.3$

Wind Loads: $\psi_0 = 0.0,\ \ \psi_1 = 0.2,\ \ \psi_3 = 0.0$

The load combinations to perform the analysis can be reduced to the following set of equations

$ULS1$: $1.5G + 0.825q_v$
$ULS2$: $1.3G + 1.65q_v$
$ULS3$: $1.3G + 0.825q_v + 1.65q_w$
$ULS4$: $0.9G + 0.825q_v + 1.65q_w$

Additionally, according to the EN and the National Annex, the load combinations used to check the serviceability limit state are the following:

$CSLS1$: $G + Q_1 + \psi_0 Q_2$

This results in the two expressions:

$SLS1$: $1.0G + 1.0q_v$
$SLS2$: $1.0G + 0.5q_v + 1.0q_w$

B. GRAVITATIONAL LOADS DESIGN

B. 1. LOADS

DEAD LOAD

Self-weight

The **self-weight** accounts for the weight of the structural members.

Additional Dead Load

Additionally, according to the architectural floor plan, the next dead loads are also assumed and considered:

1.00 kN/m2 for partition walls.
1.25 kN/m2 for ceiling and floor finishing.
0.25 kN/m2 for mechanical Installations.

Additional Dead Load = **2.50 kN/m2**

The façade load was only considered in the façade beams. This load was not accounted for to design the floors and beams, but it was considered on the global model.

LIVE LOAD

According to table 6.1, the category to use for the live load is in the category B. Office Areas.

According to Dutch National Annex the uniformly distributed load qk is set to **2.5 kN/m2.**

B. 2. STRUCTURAL ELEMENTS

B. 2. 1. SLABS

The slabs will be located as depicted in the following figure:

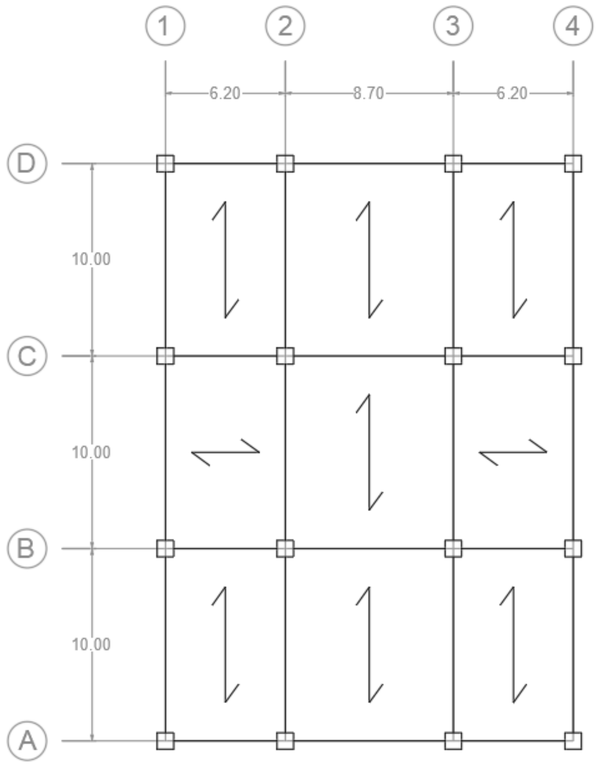


FIGURE: Slabs Location

This arrange makes the longer span of 10m which, according to supplier tables, a 260mm Hollow Core Slab – Non-Composite, Bison floor deck, can satisfy the structural requirements with a span of 10m.

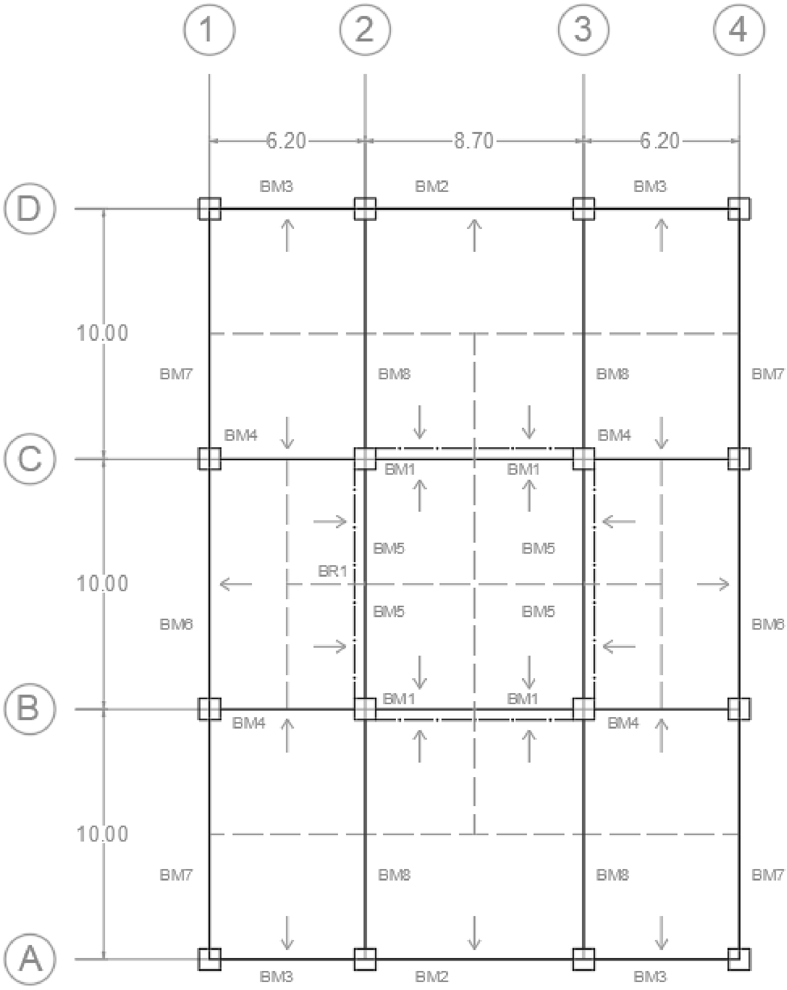
			Spans indicated below allow for characteristic imposed load plus self weight plus 2.1kN/m²								
Bison Ref	Unit Depth (mm)	Slab Self Weight kN/m²	Characteristic imposed load kN/m²								
			0.75	1.5	2.0	2.5	3	4	5	5	1
			Clear span (m)								
100 (solid plank)	100	2.40	5.15	5.00	4.80	4.60	4.45	4.15	3.95	3.80	3.
150	150	2.40	7.45	7.45	7.40	7.40	6.90	6.55	6.25	5.75	4.
150 (sound slab)	150	3.10	7.35	7.30	7.30	7.05	6.80	6.40	6.05	5.85	4.
200	200	3.10	8.90	8.85	8.85	8.85	8.75	8.25	7.80	7.55	6.
250	250	3.30	10.10	10.05	10.05	10.00	9.80	9.20	8.70	8.55	6.
260	260	3.55	10.35	10.35	10.30	10.30	10.30	10.25	9.95	9.50	7
			$\psi_0 = 0.7$	$\psi_1 = 0.5$	$\psi_2 = 0.3$		$\psi_0 = 0.7$	$\psi_1 = 0.7$	$\psi_2 = 0.6$	$\psi_0 = 1.0$	$\psi_1 =$
			Category A/B - Domestic, residential / office Areas				Category C/D - Congregation areas /shopping			Category E -	
			Floor Category of Use (From BS EN 1991-1-1:2002), used for determining the combination of a								

Source: https://www.bison.co.uk/products/hollowcore-floors/

B. 2. 2. BEAMS

The same Loads are considered, with the difference that an additional 3.55 kN/m2 load from the slabs is considered.

According to the geometry and how the slabs are disposed, there are 8 different beams:



Beam notation and tributary areas.

With these arrange of beams, and the disposition of the hollow core slabs; the beams with lower smaller spans (6.2, 8.7 and 6.2) carry most of the vertical load, and the ones with longer spans (10m) carry a relatively smaller load.

The beams are designed as simply supported beams, for all the cases. From the support conditions and with general mechanics relations, the following requirements can be derived:

Serviceability Limit State

For a simply supported beam, the maximum deflection is $\delta_{pinned} = \frac{5}{384}\frac{ql^4}{EI}$, at midspan. However, since the maximum allowable deflection for the serviceability limit state is $\delta = L/250$. The following relation can be derived to find a profile with the 2nd Moment of Inertia that fulfills the requirement: $I = \frac{5}{384}\frac{qL^4}{E*L/250}$

Serviceability Limit State (EN1990 6.5.3):

$E_d = \{G + P + Q_1 + \psi_o Q_2\}$

CS1: 1.00 qg + 1.00 qv

ULS

For a simply supported beam, the maximum bending moment has a value of $M_{pinned} = \frac{1}{8}ql^2$ at midspan. With the elastic section equation $M = W\,\sigma$, the required value of the Elastic section can be derived: $W = \frac{1}{8}\frac{ql^2}{fy}$

*The use of Elastic behaviour instead of plastic, yields conservative results. However, this will be done to simplify and homogenize the comparisons.

Ultimate Limit State: (EN1990 6.4.3.2)

$E = \{\gamma_G G + \gamma_p P + \gamma_{q1} Q_1 + \gamma_{q2}\psi_o q_{v2}\}$

CU1: 1.50 qg + 0.83 qv

CU2: 1.30 qg + 1.65qv

These two requirements were evaluated using the above-mentioned Load Combinations.

The following set of beams were obtained:

Beam	Length [m]	Tributary Area [m^2]	Façade load [kN/m]	Load q [kN/m]	Required I (SLS) [x10^6 mm^4]	Required W (ULS) [x10^3 mm^3]
1	4.4	43.5	0.0	85.0	108.45	794.5
2	8.7	43.5	6.3	48.8	498.12	1,807.4
3	6.2	31.0	6.3	48.8	180.28	917.9
4	6.2	31.0	0.0	42.5	157.00	807.0
5	5	15.5	0.0	26.4	51.06	301.9
6	10	31.0	6.3	32.7	506.11	1,590.1
7	10	0.0	6.3	6.3	97.66	288.4

Beam 8 does not contribute to the load bearing of the vertical loads; thus, it is left out of the vertical load design. However, this beam provides support to the columns and transfers the forces when the building is loaded by horizontal forces.

It is chosen to use Beams IPE550.

IPE550 Iyel= 671.2x10^6 mm4 and Wel=2441x10^3 mm3

Beam	Profile	Iy [x10^6 mm^4]	Required I (SLS) [x10^6 mm^4]	Ratio	W [x10^3 mm^3]	Required W (ULS) [x10^3 mm^3]	Ratio
1	IPE550	671.2	108.4	0.16	2441	794	0.33
2	IPE550	671.2	498.1	0.74	2441	1810	0.74
3	IPE550	671.2	180.3	0.27	2441	918	0.38
4	IPE550	671.2	157.0	0.23	2441	807	0.33
5	IPE550	671.2	51.6	0.08	2441	325	0.13
6	IPE550	671.2	506.1	0.75	2441	1590	0.65
7	IPE550	671.2	97.7	0.15	2441	288	0.12

Beams 1, 3, 4, 5 and 7 are overdesigned. There is still room for optimization. However, the optimization of these elements is left out of the scope of this thesis. This optimization would imply a much more detailed design of each of the elements, which is variable for all the systems. Thus, it would further complicate the 3D models. This simplification will affect all the structural systems in a similar way; thus, they can still be compared without compromising the results.

It is not on the scope of this thesis to optimize every element of the structure, but to have a broad comparison of the whole structure. By, not optimizing every detail, all of the systems have a similar disadvantage. It is recommended to realize this study by optimizing every system.

The maximum deflection and the maximum bending moment of the core beams differs from the one expressed in the previous equation, due to the fact that the chevron bracing acts as an intermediate support; thus, the beam behaves as a continuous beam, for which the deflection and bending moments are smaller than the ones obtained with a simply supported beam. However, the values of the deflection and the structural response of a simply supported beam with a length equal to half of the span was used as a conservative simplification.

B. 2. 3. COLUMNS

The cross sections of the columns were obtained with the Karamba3D optimization algorithm, since the stress in these elements is heavily influenced by the horizontal loads.

C. KARAMBA 3D MODEL

This appendix describes the process to model and analyse the structural 3D models with Karamba3D software. This Sections is divided three subsections: C. 1 Model Description, C. 2 Model Script and 0 Model Visualization.

C. 1. MODEL DESCRIPTION

To perform the structural analysis, two different scripts where used to program the Finite Element Models.

1. OPTIMIZATION MODEL
2. 4 FIELDS MODEL

Both models part from the same geometrical model with the same geometry, element properties and requirements. However, the model was divided into two parts to obtain specific results and to reduce the file size, which reduced the computational time.

The general model was created using the Grasshopper parametrical properties. This model included the information of the 4 structural systems studied (Fig. App. 2) (Moment Resistant Frame, Outrigger, Megabrace and Diagrid), and of the two types of core used (Fig. App. 3) (Concrete Core and Steel Chevron Braced Frame). All the properties were scripted in a parametrical way to evaluate the behaviour of the different systems in early steps of the design. The number of bays, floor height, number of floors, and size of the core were the main parameters of the general geometry.

Subsequently, the geometry was translated into structural members by using bar and shell elements components of Karamba3D, and their characteristics and properties (Materials, Cross Sections, Loads, Load Combinations, Support conditions and Joint Releases) were also included.

Defining the proper joint releases and the interaction between the elements is the most arduous task during the modelling process. However, the success or failure of the model relies greatly on this phase. To ensure that the model built in Karamba3D is a reliable model, multiple models were built with Robot Structural Analysis to compare the results at different steps during the elaboration of the model. The results compared were mainly displacements (horizontal and vertical), axial forces, bending moments, shear forces, support reactions, plate stresses (for the concrete core and slabs).

Fig. App. 1 presents the flowchart and the relationship between the two models ("Optimization Model" and "4 Fields Model"). This process is described in the following paragraphs.

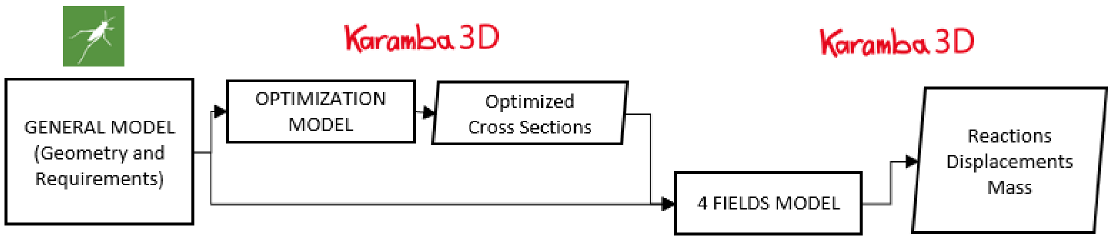
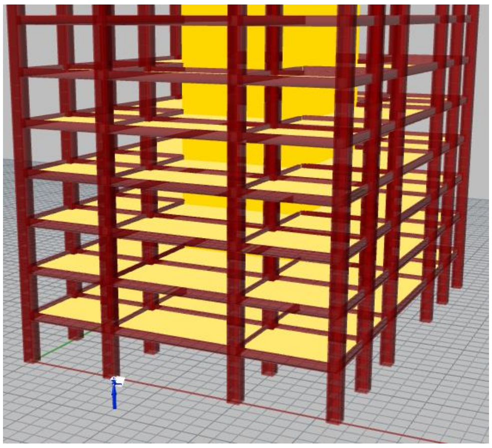
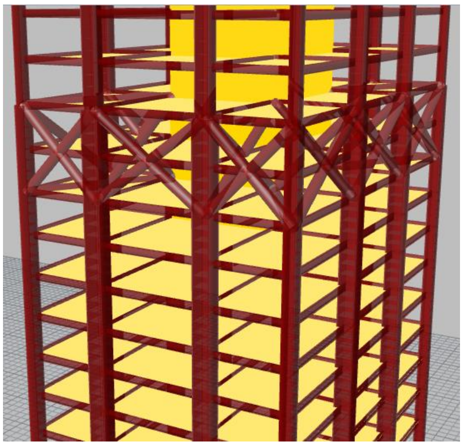


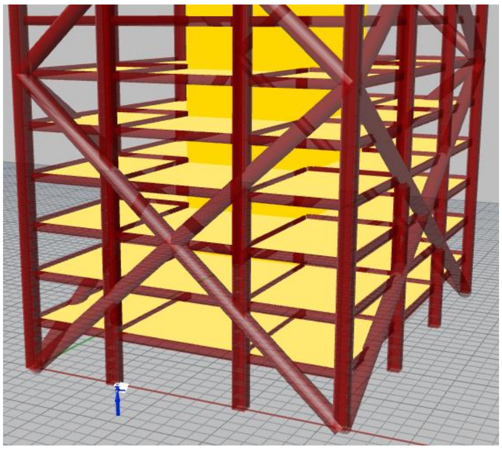
Fig. App. 1: "Optimization Model" and "4 Fields Models" flowchart.



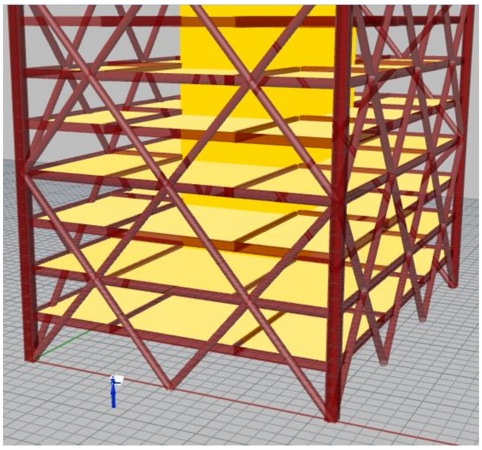
a) Moment Resistant Frame



b) Outrigger

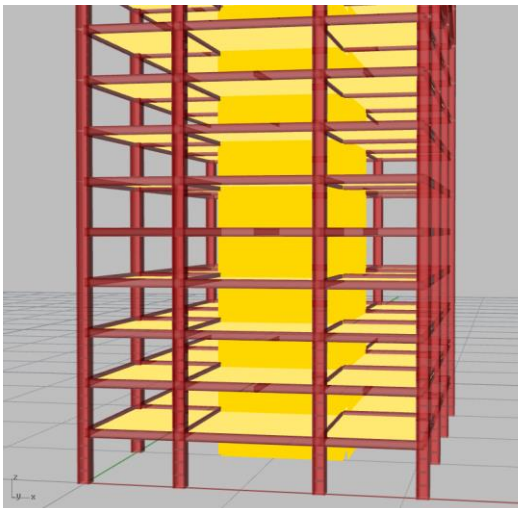


c) Megaframe

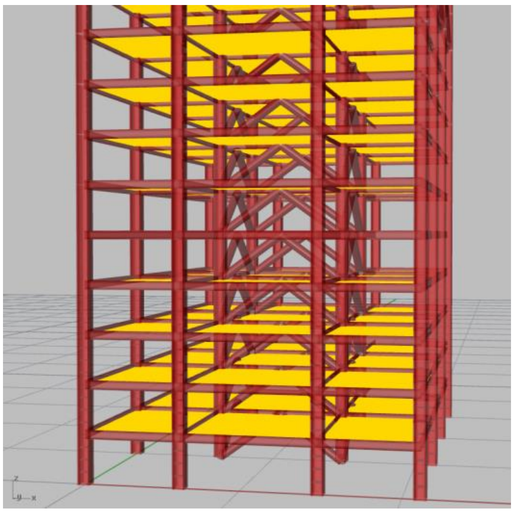


d) Diagrid

Fig. App. 2: Geometry of the different structural systems used in the Karamba3D model.



c) Concrete core



d) Steel core

Fig. App. 3: Types of core used.

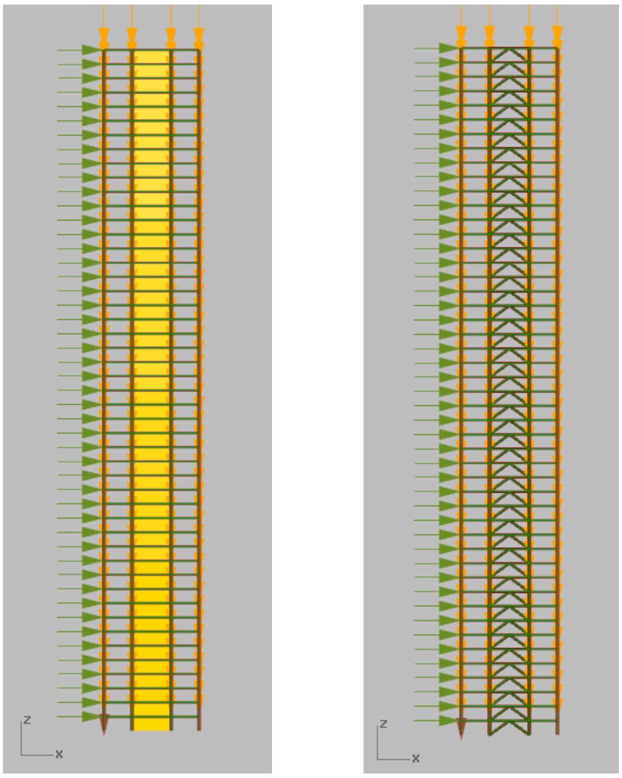
OPTIMIZATION MODEL

The first model, "Optimization Model", is intended to optimize the structure. For this model the loads were defined as a surface loads acting over the slabs (vertical loads) and a horizontal uniformly distributed load acting on the beams from one façade (wind load) (Fig. App. 4).

The model was run with the regular analysis component from Karamba3D and the maximum deflection at the top of the building was obtained. Then, the model was optimized by means of the "optimize cross section" component of Karamba3D. This component optimizes the weight of the structure for a given target displacement.

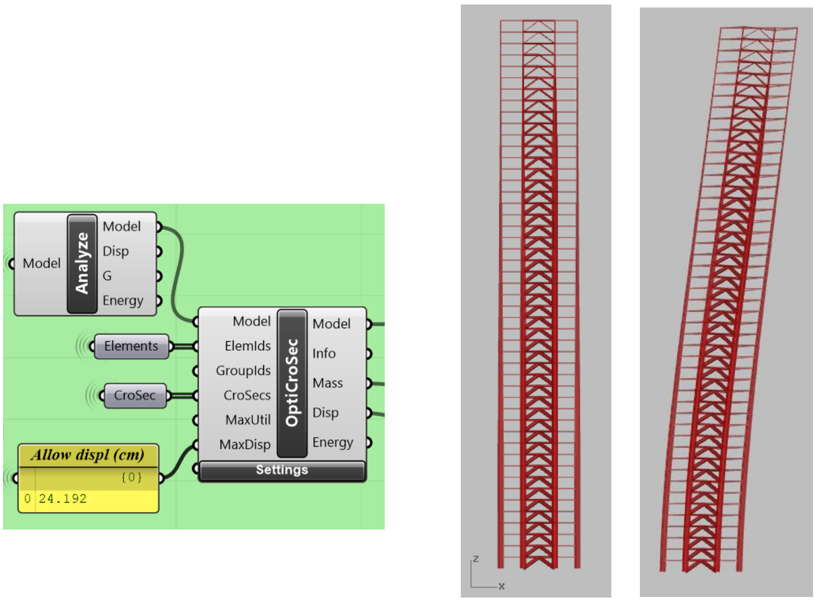
The input data of this component are the elements to be optimized, the cross-section database to choose an adequate profile, and the displacement requirements. The elements were defined from the model geometry, the cross-section database was defined by means of parametric cross sections (Appendix D), and the target displacement was defined as $0.8\delta_{admisible}$. The "optimize cross section" component runs the analysis with all the available cross sections from the database and chooses the ones that yield the lowest mass, while the target function is still satisfied (or nearly satisfied) (Fig. App. 5).

This optimization yields a structure where all the components are optimized, resulting in a building with a no realistic distribution of cross sections. For this reason, a manual selection of significative profiles to use in a more realistic model was performed after running the optimization component.



a) b)

Fig. App. 4: "Optimization Model" with Horizontal Loads

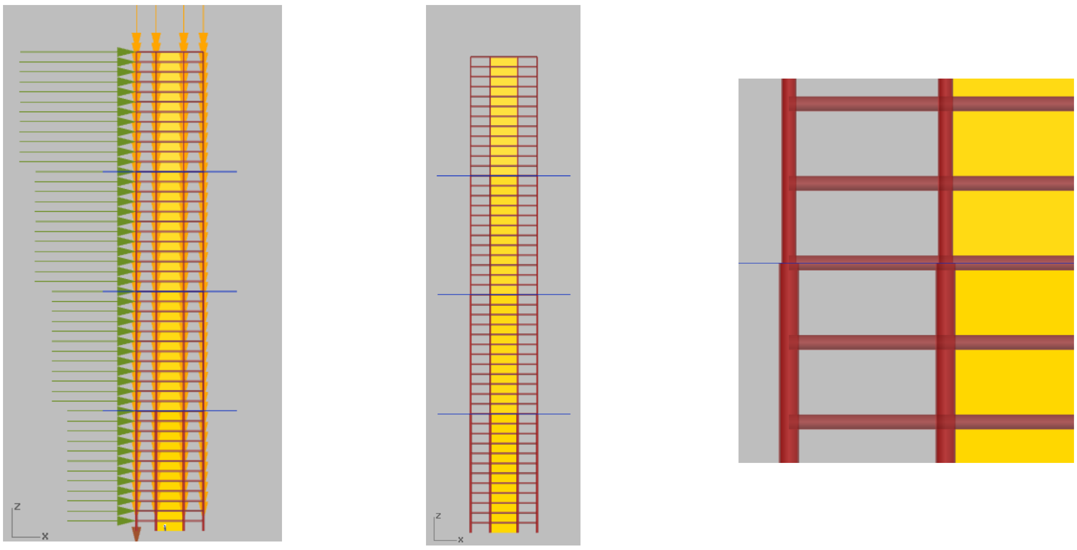


a) b) c)

Fig. App. 5: Optimize function and optimized model.

4 FIELDS MODEL

The second model ("4 Fields Model") is intended to represent a more accurate and realistic building. This model is divided in four fields, along the height of the building. This division was done to be able to use a wind load with a profile closer to the real load profile, and to be able to use different cross sections along the height of the building.



a) Wind load profile

b) Optimized cross sections by field

c) Detail of different Cross Sections between two fields.

Fig. App. 6: 4 Fields Model with Loads

The cross sections used for this model are the ones manually selected from the previous optimization step. Once the "4 Fields Model" was computed the results included displacements, bending moments, axial forces, shear forces, utilization of the cross sections (Utility Check) and support reactions.

To analyse the model and evaluate it, first, the displacements were evaluated to check that the displacement was within the maximum allowable value (SLS). Secondly, the utilization check was reviewed to check that the structural elements satisfied the resistance criteria (ULS).

If these requirements were not satisfied, the cross sections were changed until both criteria were met. After these requirements were met, the foundations were defined and modelled. Fig. App. 7 depicts the flowchart of the foundations design, and Appendix E addresses this subprocess more thoroughly.

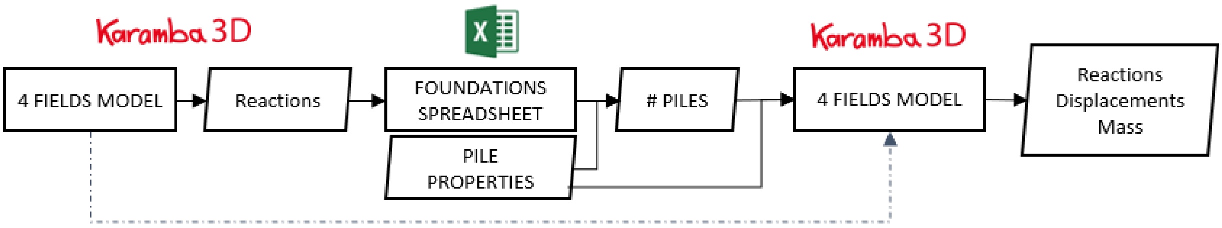


Fig. App. 7: Foundations definition flowchart.

For this process, the reactions at each of the supports, obtained from the "4 Fields Model" were exported to an excel spreadsheet. This spreadsheet calculated the minimum number of piles needed on each support to satisfy the requirements from all the load combinations, according to the resistance of the piles (Appendix E).

Next, the number of piles required to meet the resistance criteria was exported back to the "4 Fields Model" and the foundations were modelled as spring elements on each support (Fig. App. 8 and Fig. App. 9). The properties of the stiffness of the foundations were obtained from a case study project from IMd.

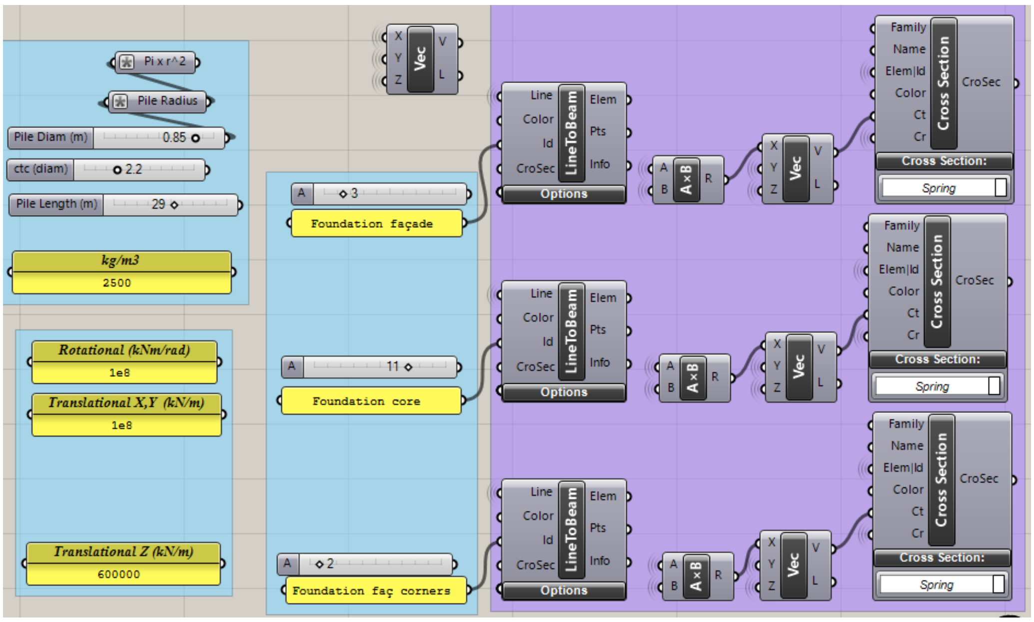


Fig. App. 8: Foundations as spring elements (Script)

Fig. App. 9 depicts the location of the supports and the length of the piles. The number of piles per support is not displayed on this figure. However, the contribution from multiple piles per support was considered in the spring models (Fig. App. 8).

The "4 Fields Model" was computed once more. This time, the results included the displacement at the top due to the flexural behaviour of the structure and due to the rotation from the foundations (Fig. App. 10). This deflection must be smaller than the maximum allowed by the Serviceability Limit State.

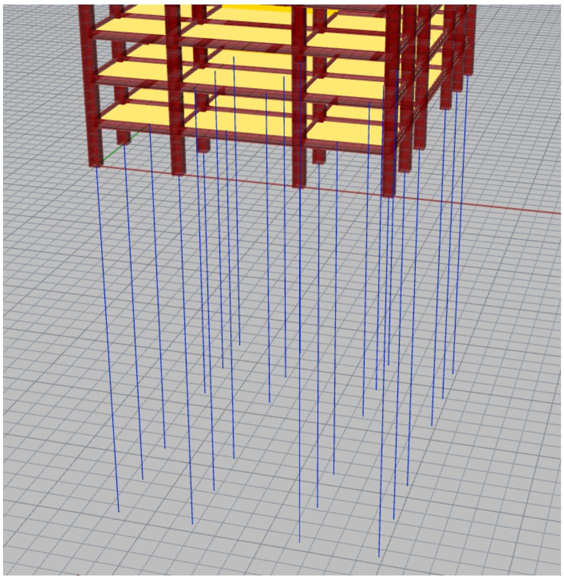


Fig. App. 9: Foundations visualization.

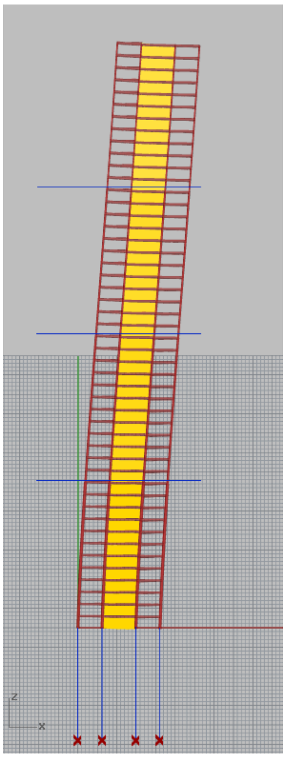
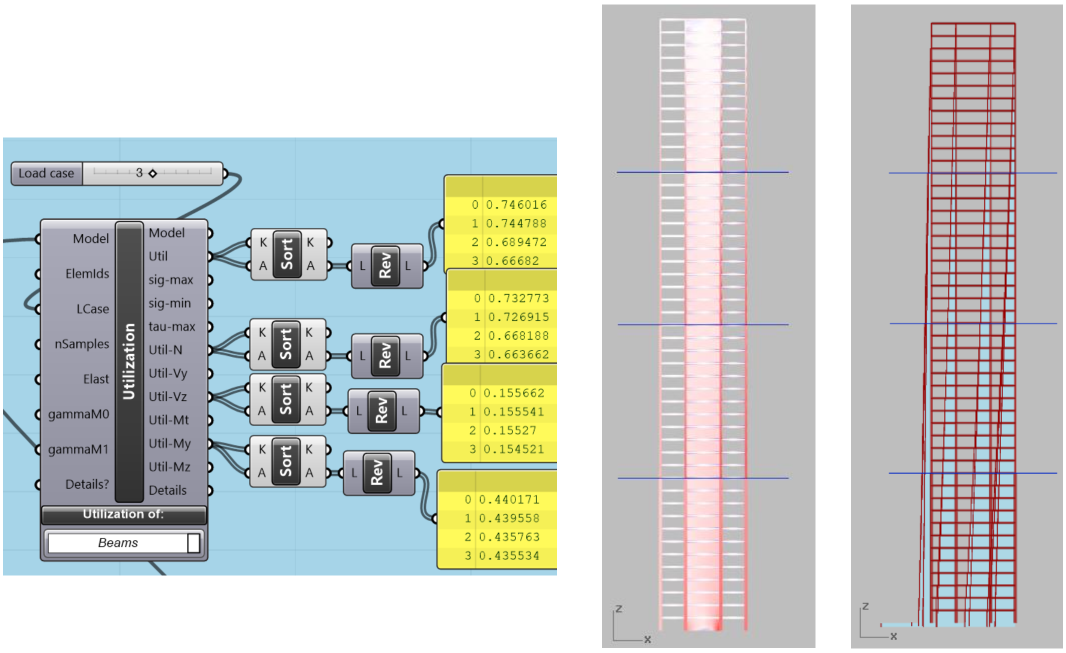


Fig. App. 10: Total deflection of the 3D model.



a) Utilization component

b) Visual representation of the element's utilization.

c) Axial forces acting on the structural elements

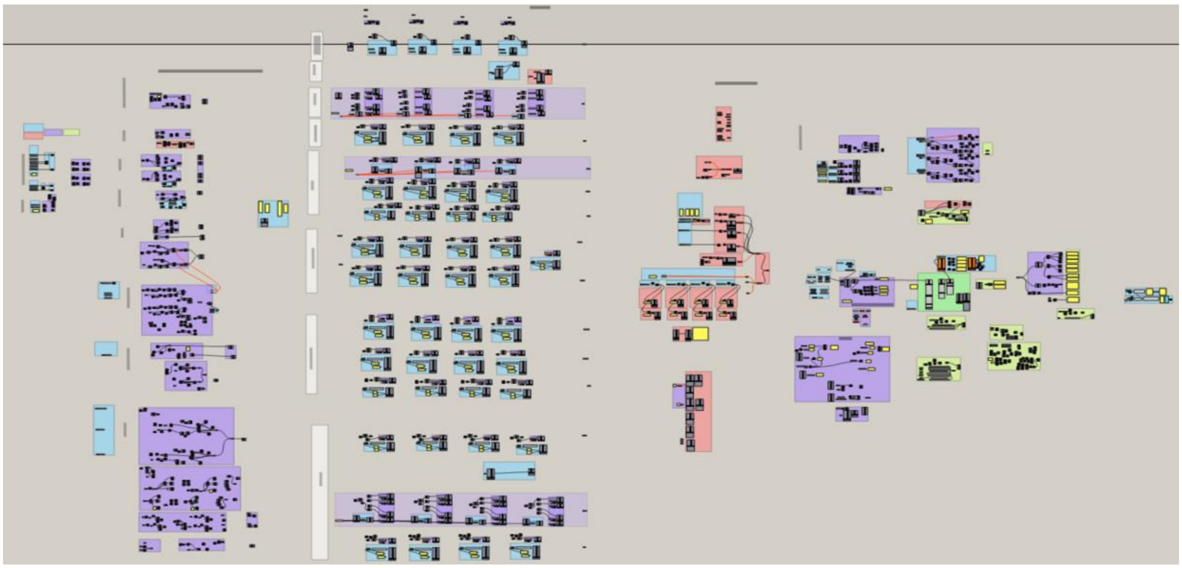
Fig. App. 11: Utilization of Structural elements.

Finally, the elements were checked with the "utilization ratio" component of Karamba3D (Fig. App. 11). This component shows the ratio of utilization (the external requirement divided by the resistance of the structural member) of the elements included in the analysis by including the checks listed in the EN:1993. The utilization ratio was checked for bending moments, axial forces, and shear forces, acting in the structural elements; and considering all the loads cases.

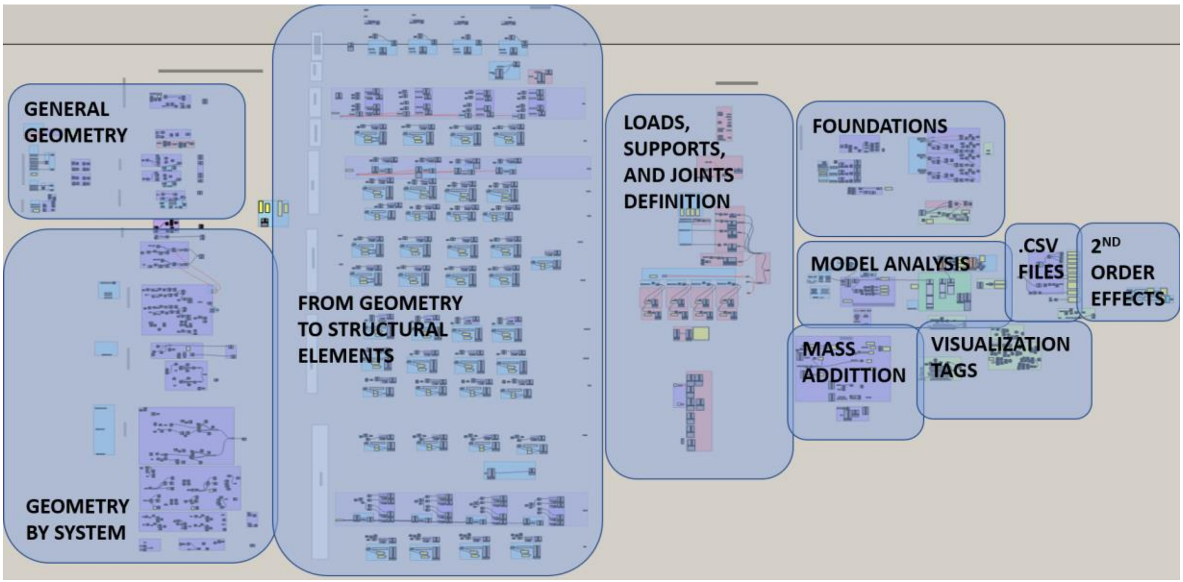
This study only considered the global analysis of the whole structure. For this reason, and due to time constraints, this unity check was considered as sufficient. However, to perform a more accurate design, the local design of each of the elements should be performed.

C. 2. MODEL SCRIPT

This section provides multiple figures of the scripts used for the modelling in Grasshopper and the structural analysis in Karamba3D. Figures (Fig. App. 12) to (Fig. App. 27) contain information relevant to the "4 Fields Model", and figures (Fig. App. 28) to (Fig. App. 30) contain information relevant to the "Optimization Model". These scripts are described in the previous section (C. 1 Model Description).



a) General Layout of the Karamba3D Script



b) Differentiation of each of the sections of the Karamba3D script

Fig. App. 12: General Overview of the Karamba3D "4 Fields Model" (Script).

Fig. App. 12 presents the overview of the script, the following Figures provide a detailed and view of the different sections.

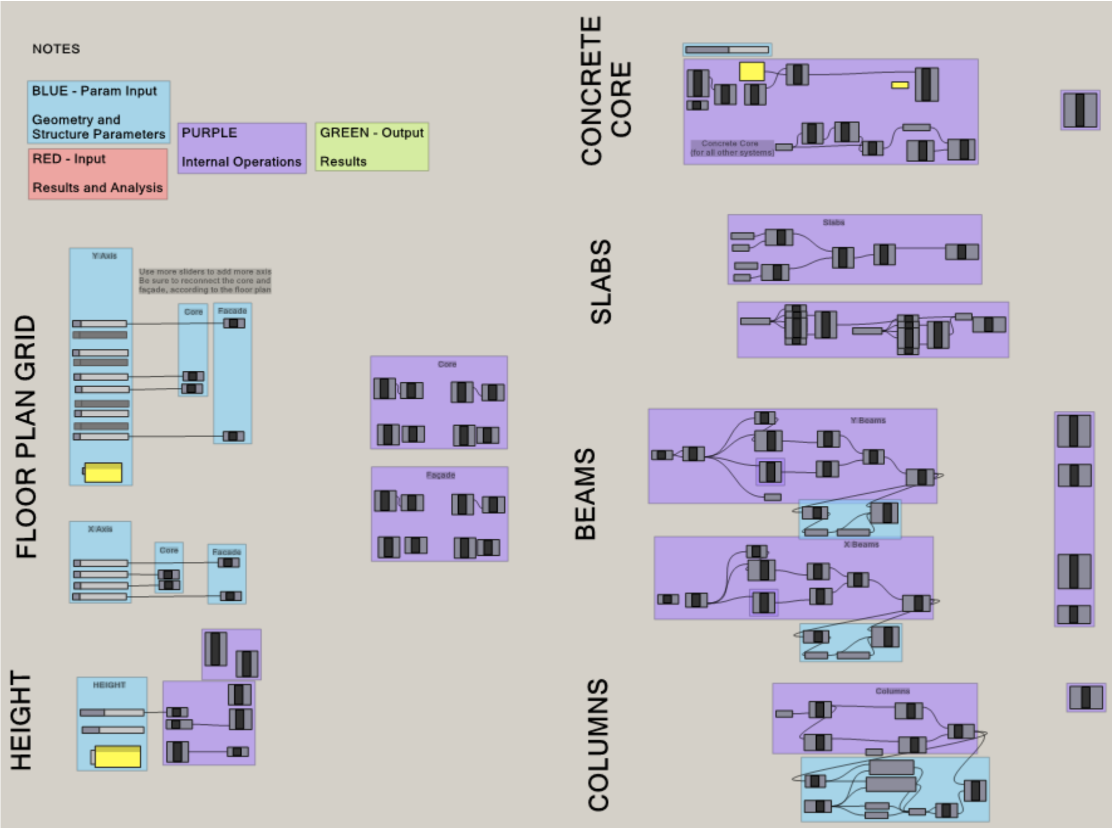


Fig. App. 13: General Geometry of the Building (Script).

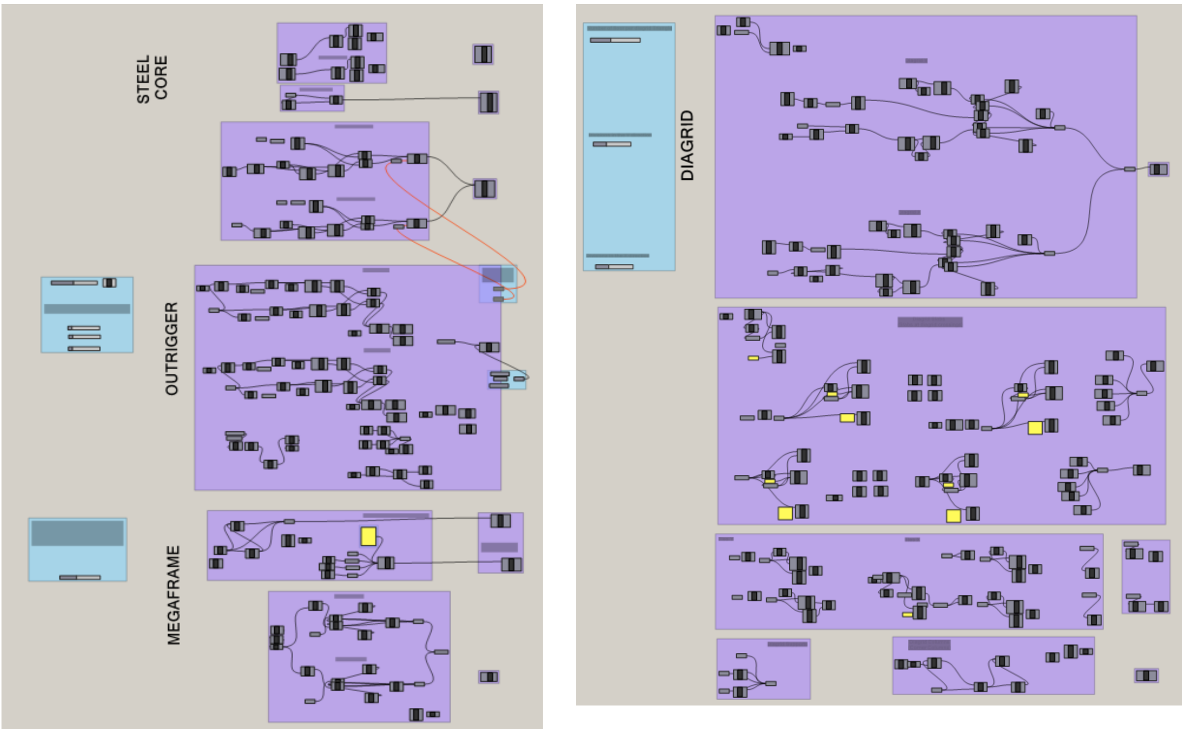
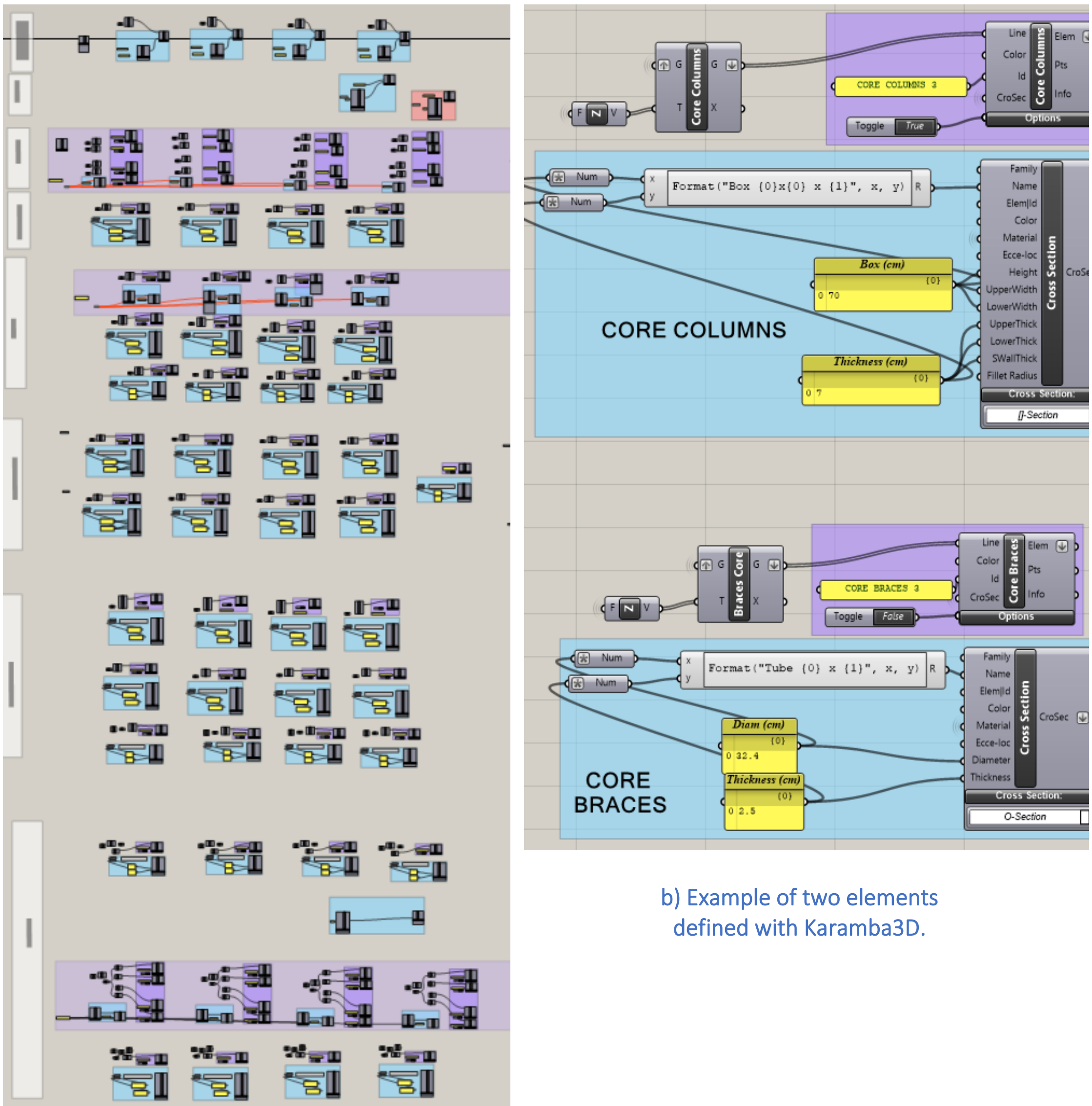


Fig. App. 14: Geometry of each structural system (Script).



b) Example of two elements defined with Karamba3D.

a) Layout of all the elements and systems.

Fig. App. 15: From Grasshopper geometry to Karamba3D elements (Script).

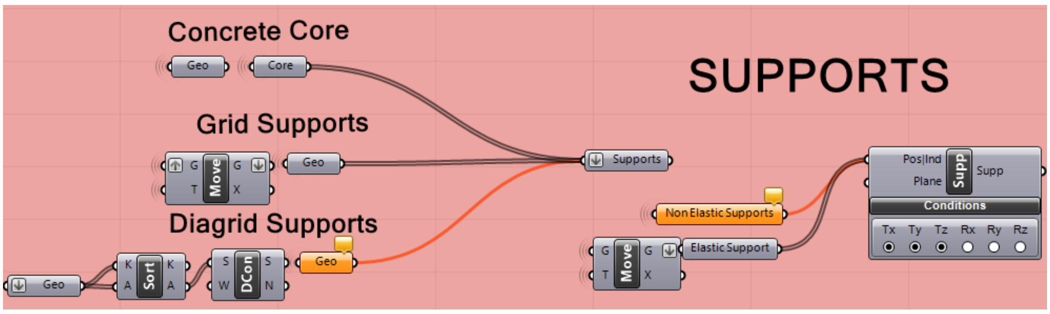


Fig. App. 16: Support definition (Script).

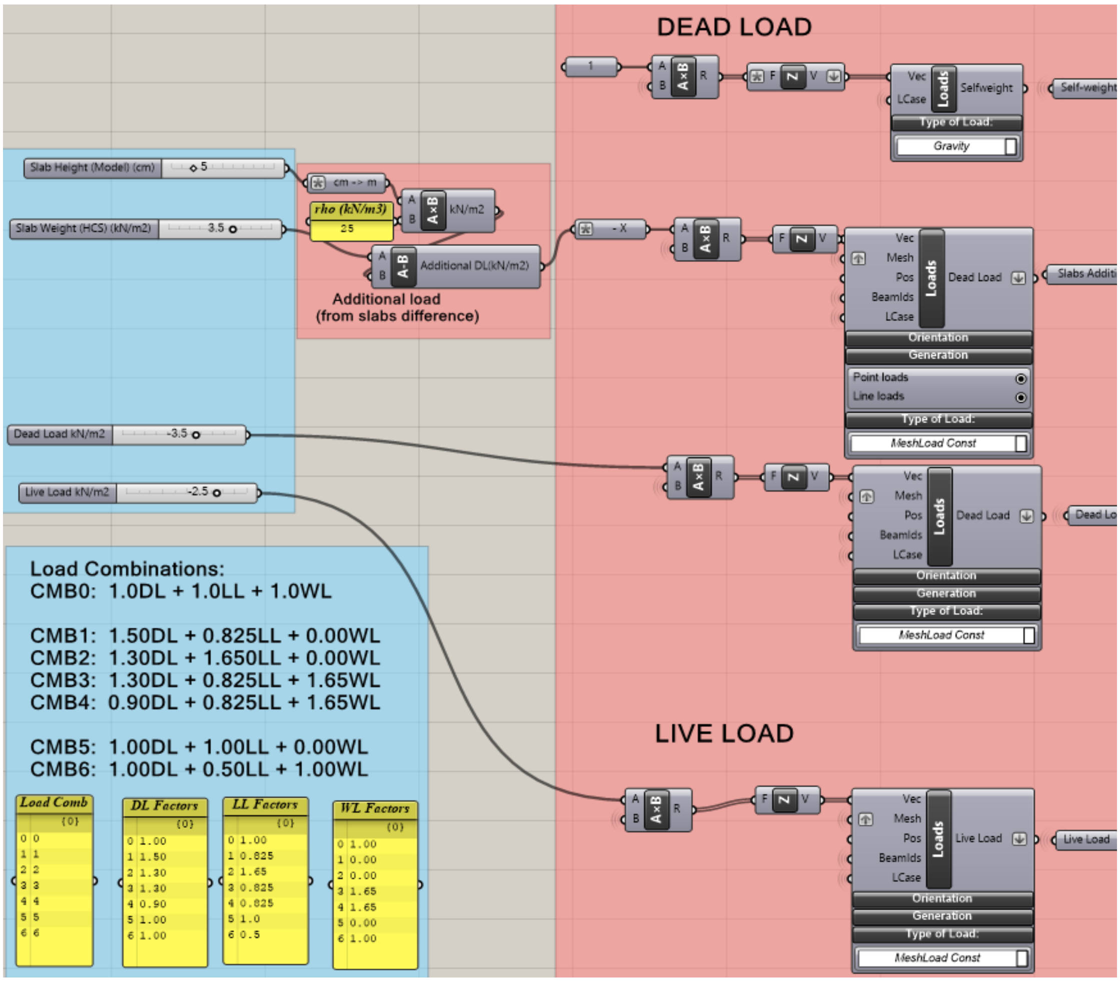


Fig. App. 17: Vertical Loads (Script).

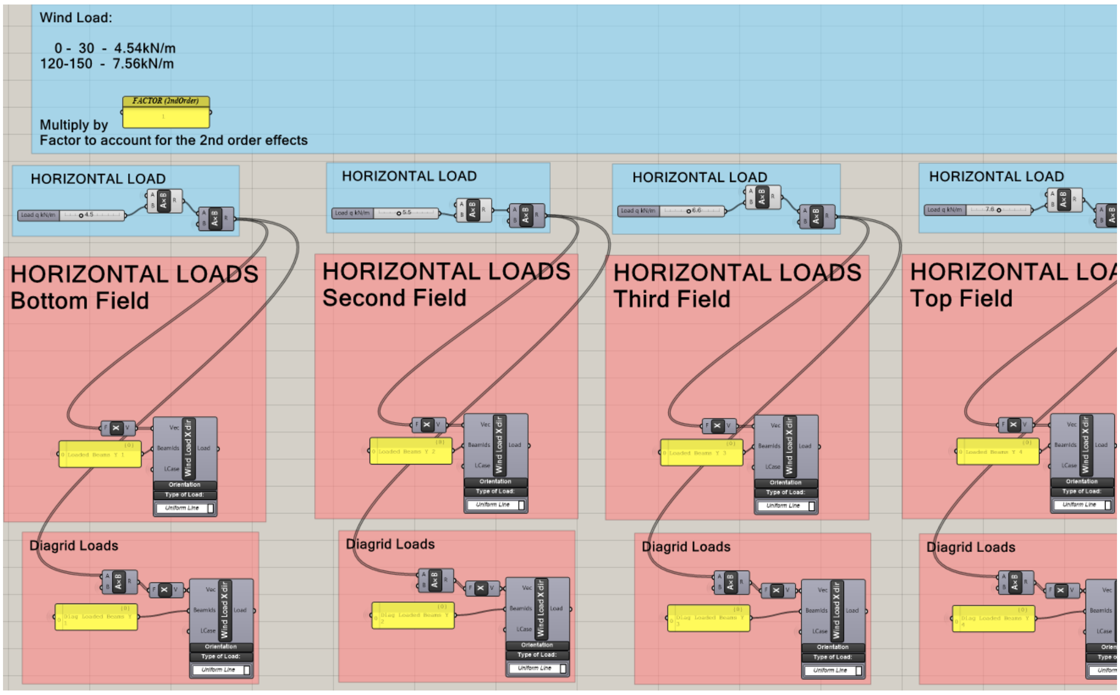


Fig. App. 18: Horizontal Loads (Script).

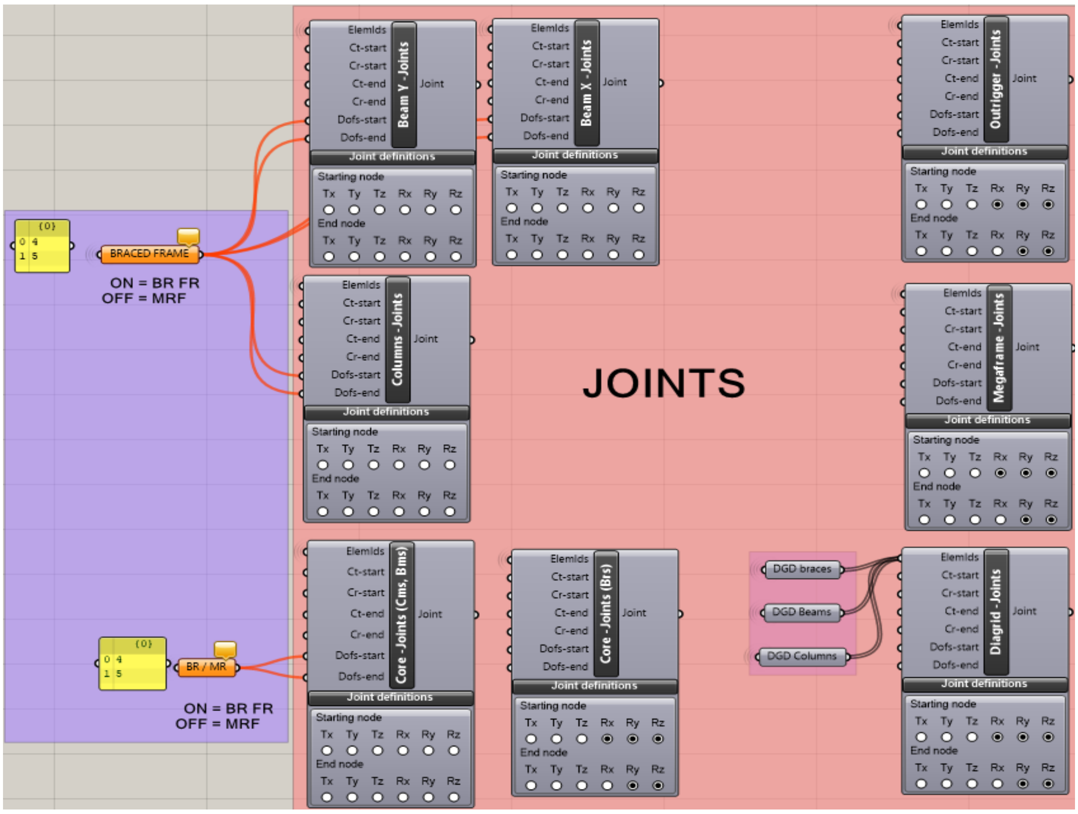


Fig. App. 19: Joint Releases (Script).

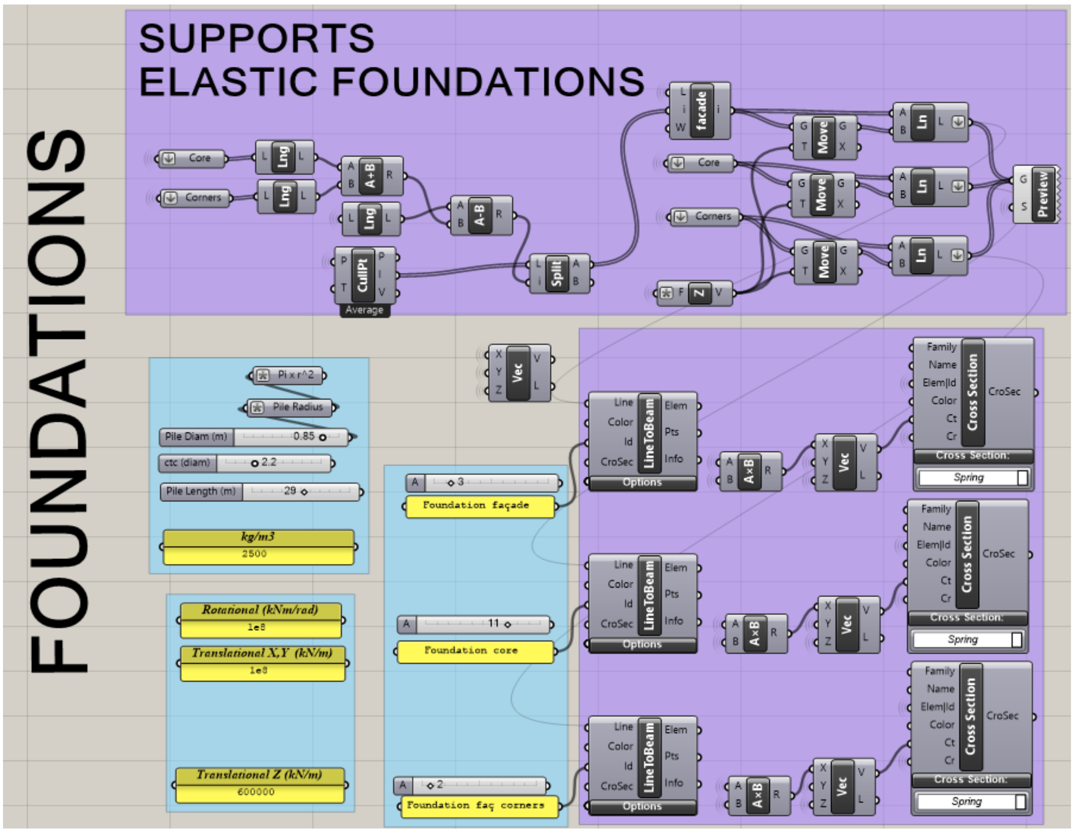


Fig. App. 20: Foundations Modelling (Script).

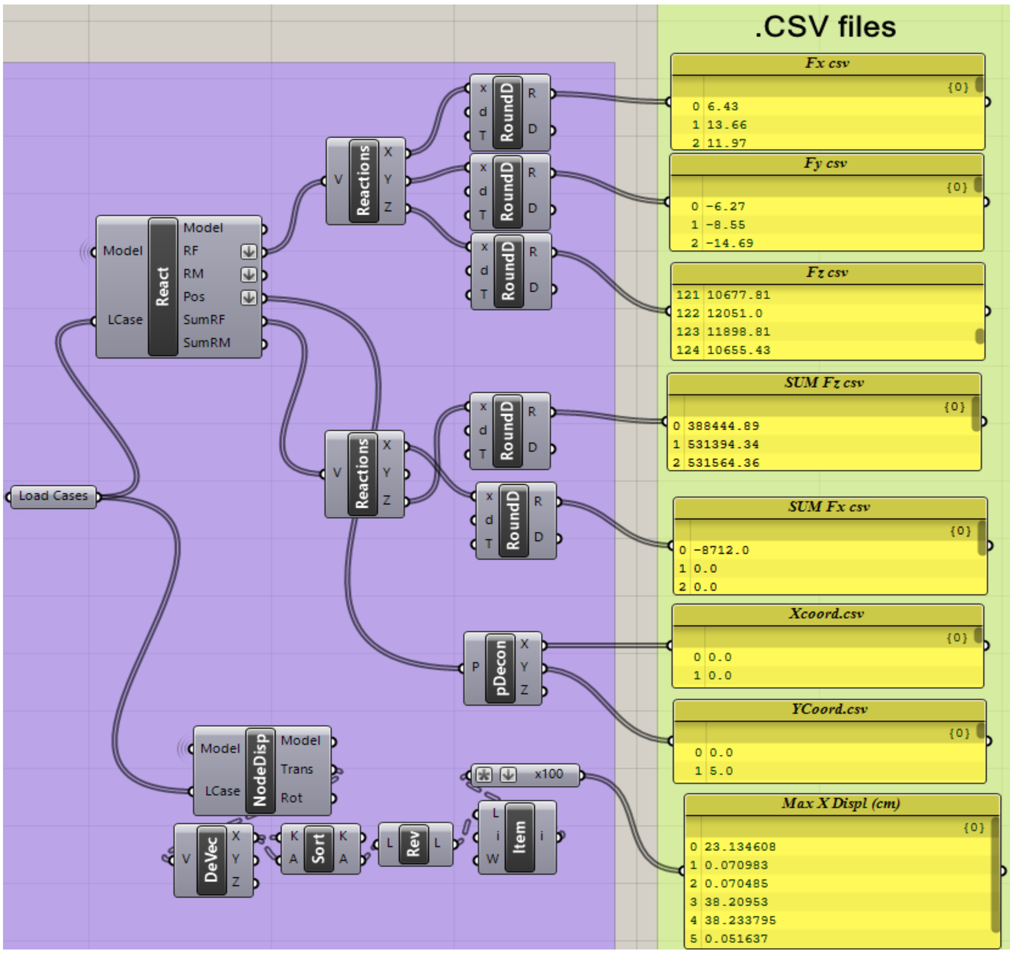


Fig. App. 21: .csv files for foundation design (Script)

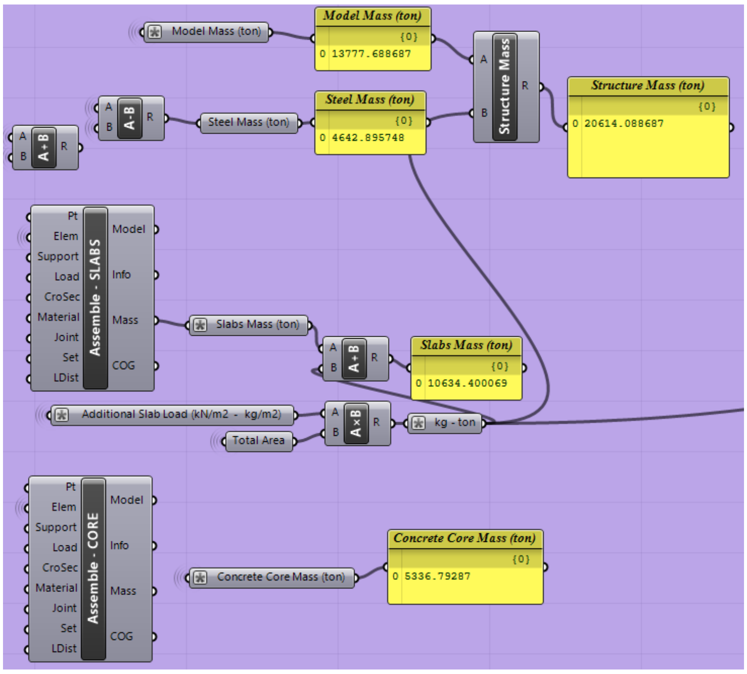


Fig. App. 22: Mass addition (Script).

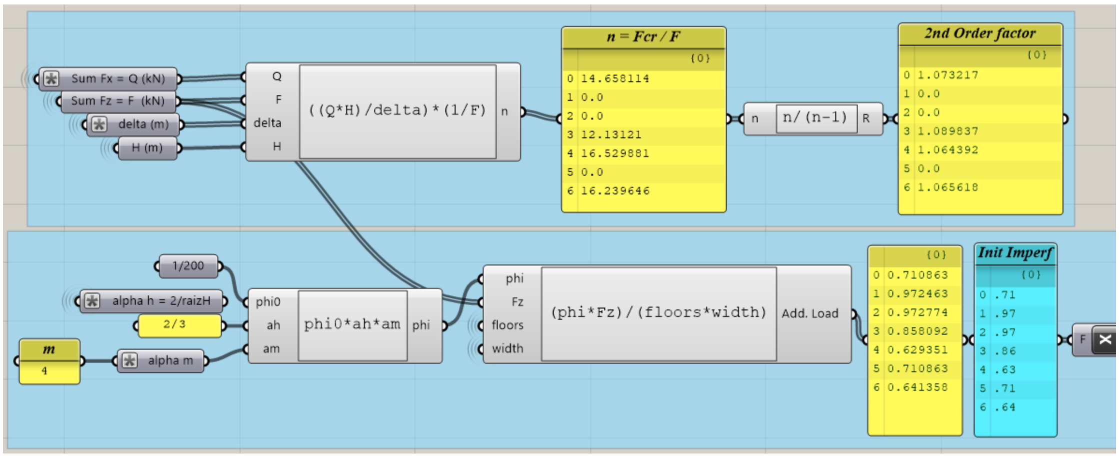


Fig. App. 23: 2nd Order Effects and Initial Imperfections (Script).

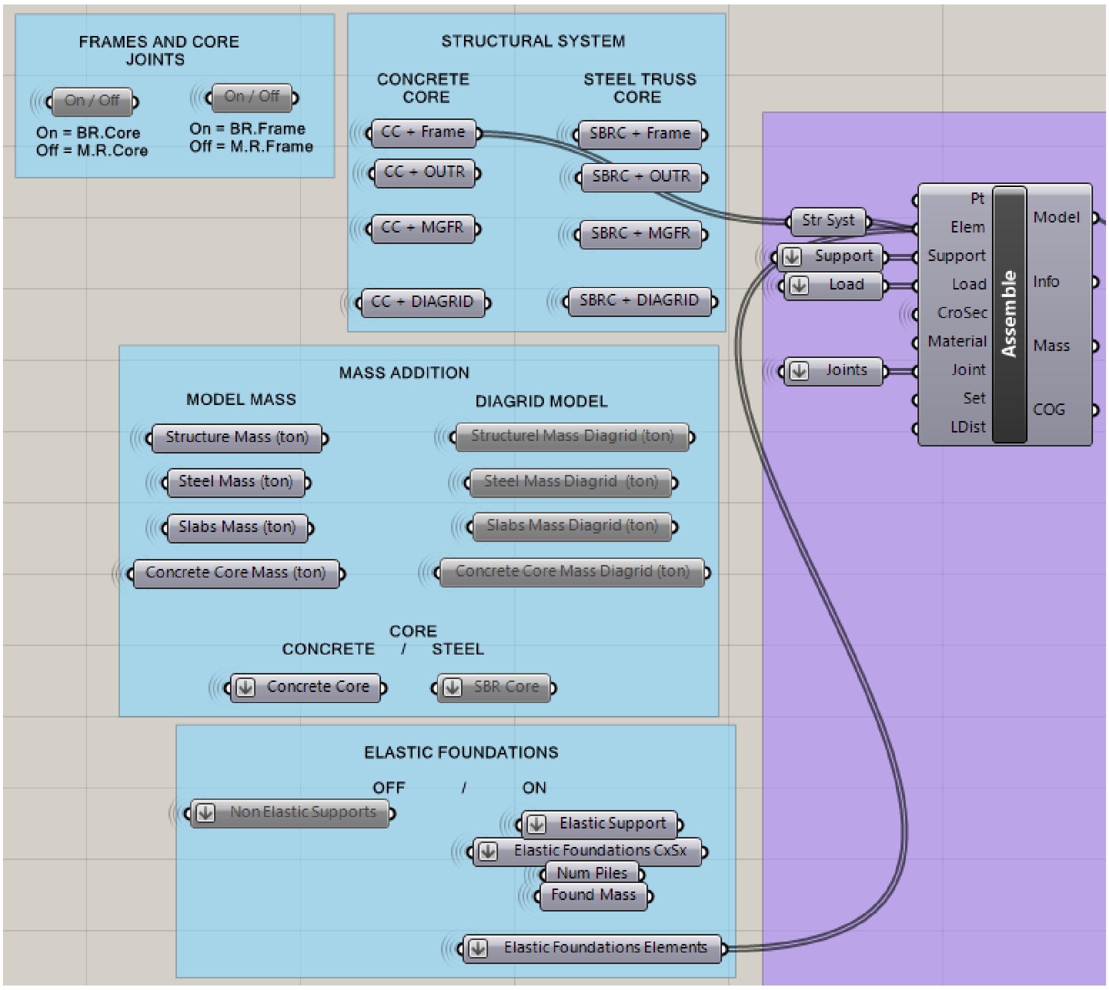


Fig. App. 24: Structural analysis dashboard (Script).

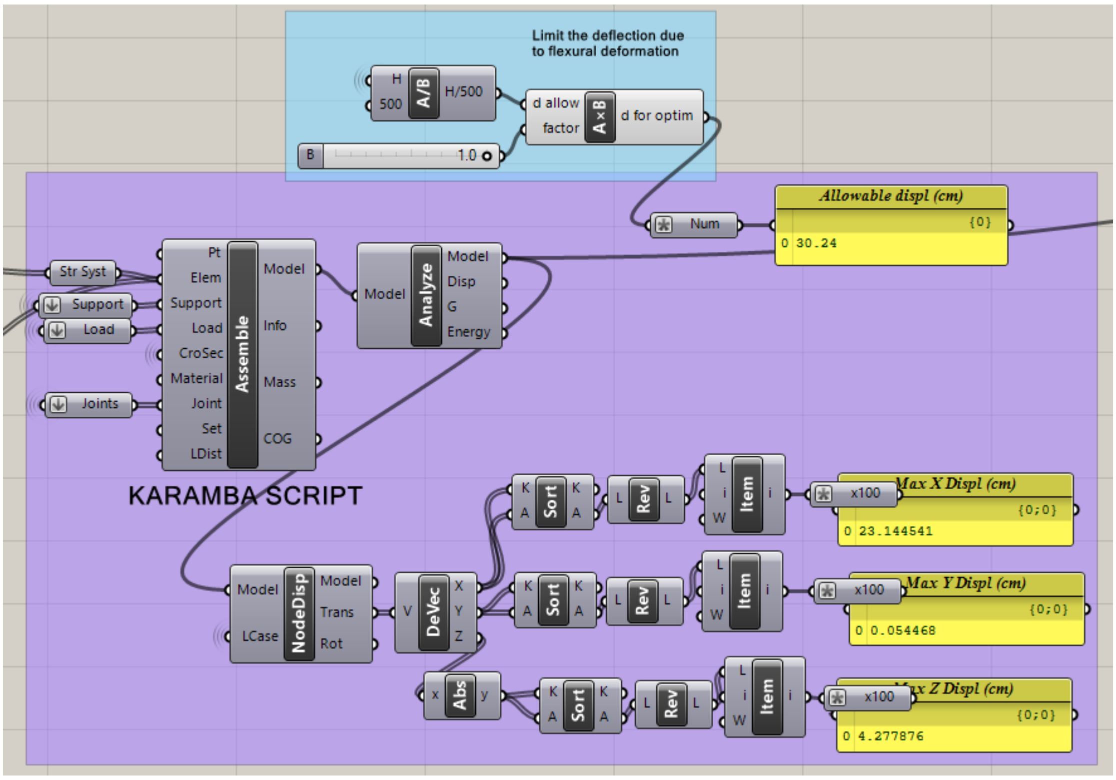


Fig. App. 25: Analysis Karamba3D (Script).

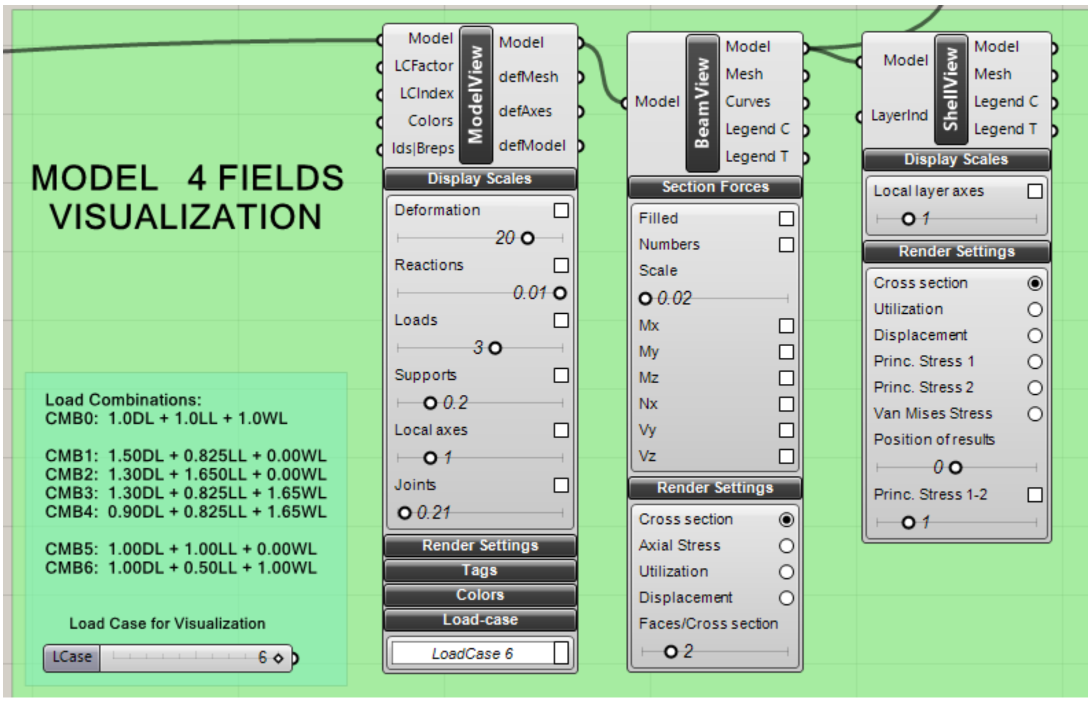


Fig. App. 26: Karamba3D Visualization (Script)

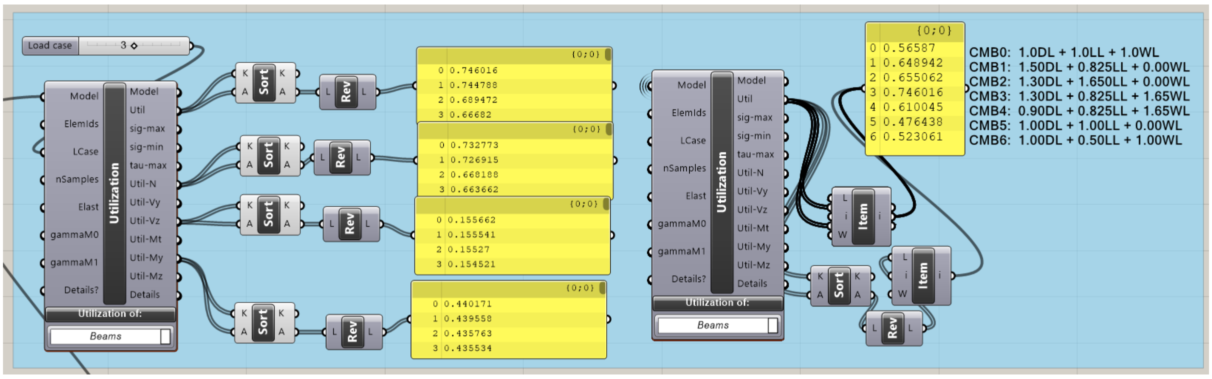


Fig. App. 27: Utilization Check (Script).

Figures (Fig. App. 28) to (Fig. App. 30) present the script for the “Optimization” model used.

The Geometry definition, the Loads, Supports, Joints and Analysis are built with the same components and geometry from the “4 Fields Model” and they work in the same manner. The differences arise on the optimization part of the script (Fig. App. 28).

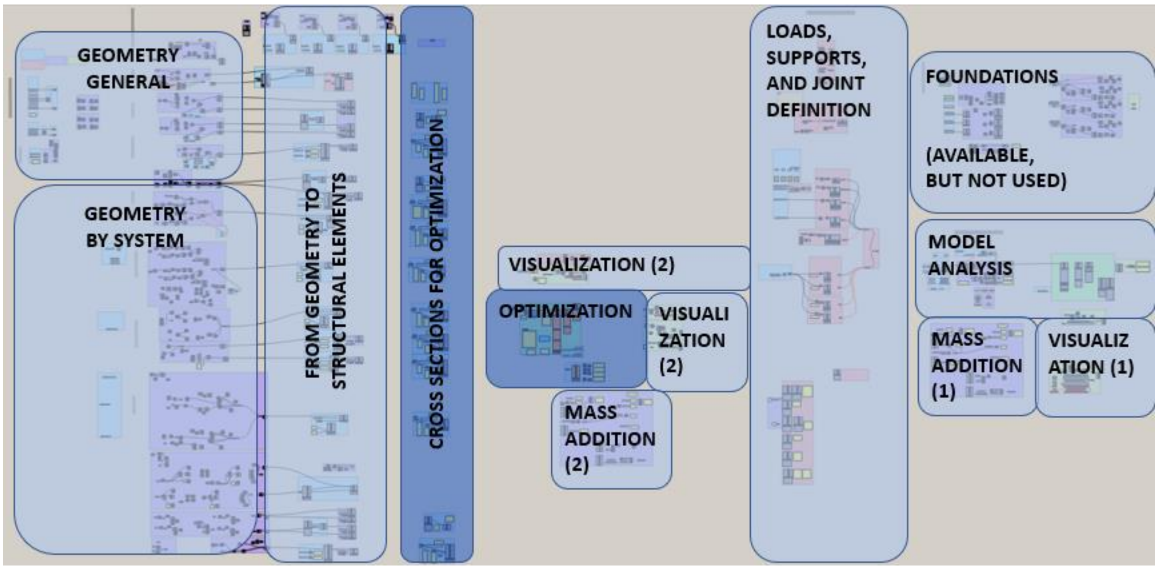


Fig. App. 28: General Overview of the “Optimization” Karamba3D Model (Script).

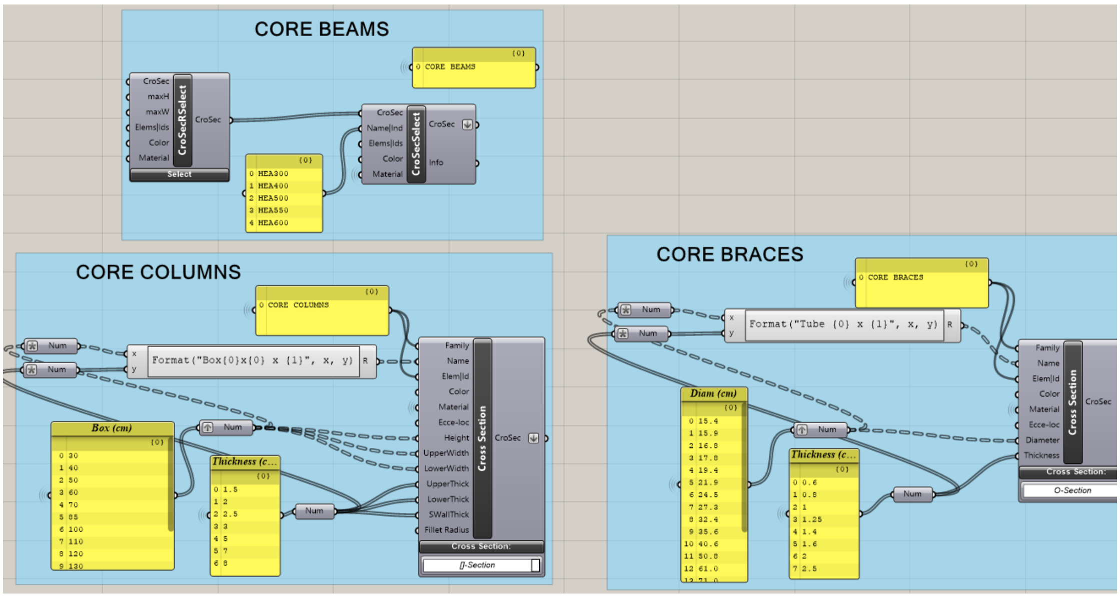


Fig. App. 29: Example of cross sections available for optimization (Script)

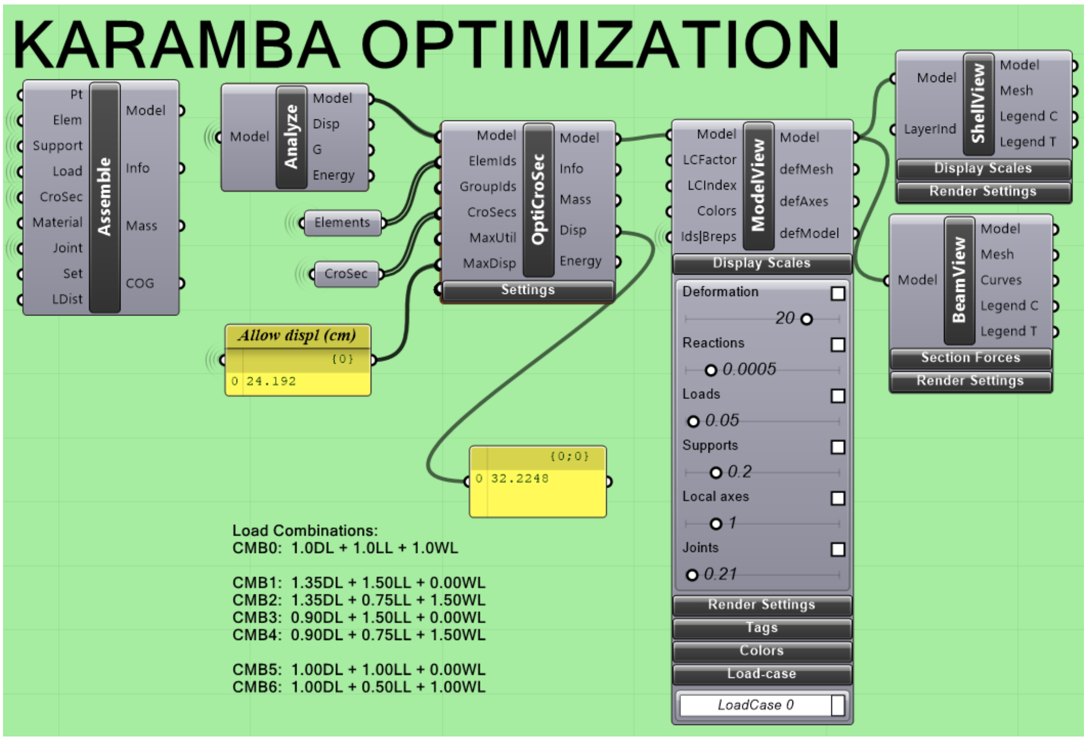


Fig. App. 30: Optimization component (Script)

C. 3. MODEL VISUALIZATION

This section presents the visualization of some models built with Karamba3D of the different structural systems studied.

Figures Fig. App. 31, Fig. App. 32 and Fig. App. 33 present the model, deformation and utility ratio of the "Megaframe with concrete core" structural system during the three steps of the design: before optimization, after optimization, and after manual choice of the cross sections.

Figures Fig. App. 34 and Fig. App. 35 present the models of the eight structural systems studied and their deflection.

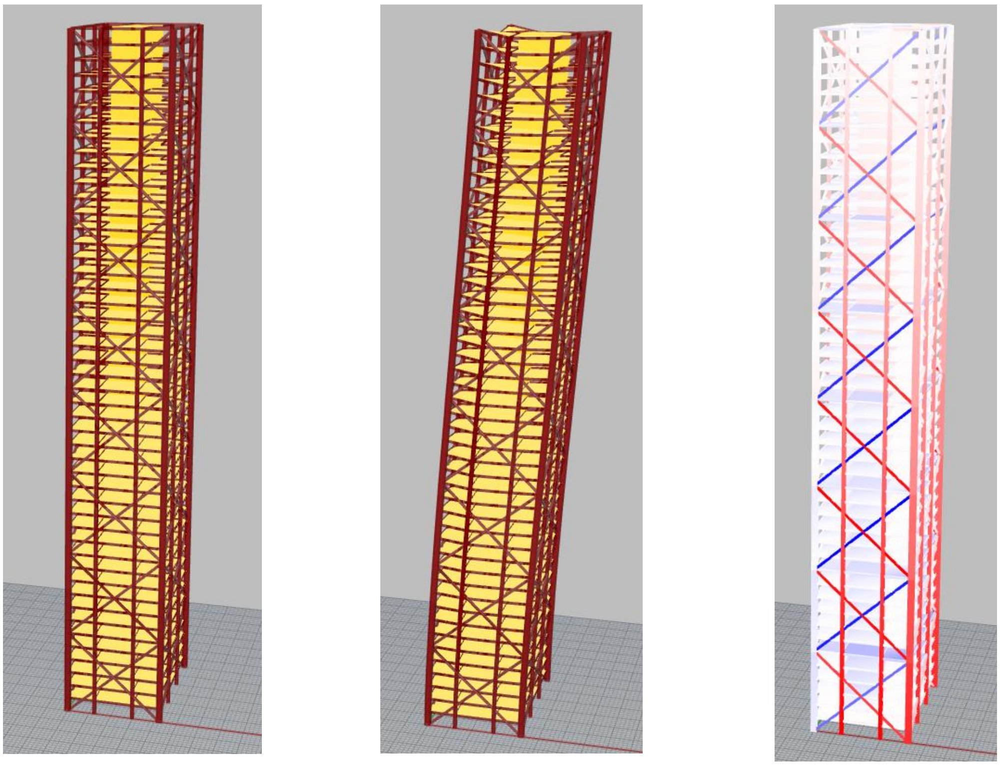


Fig. App. 31: Megaframe with concrete core. Before Optimization.

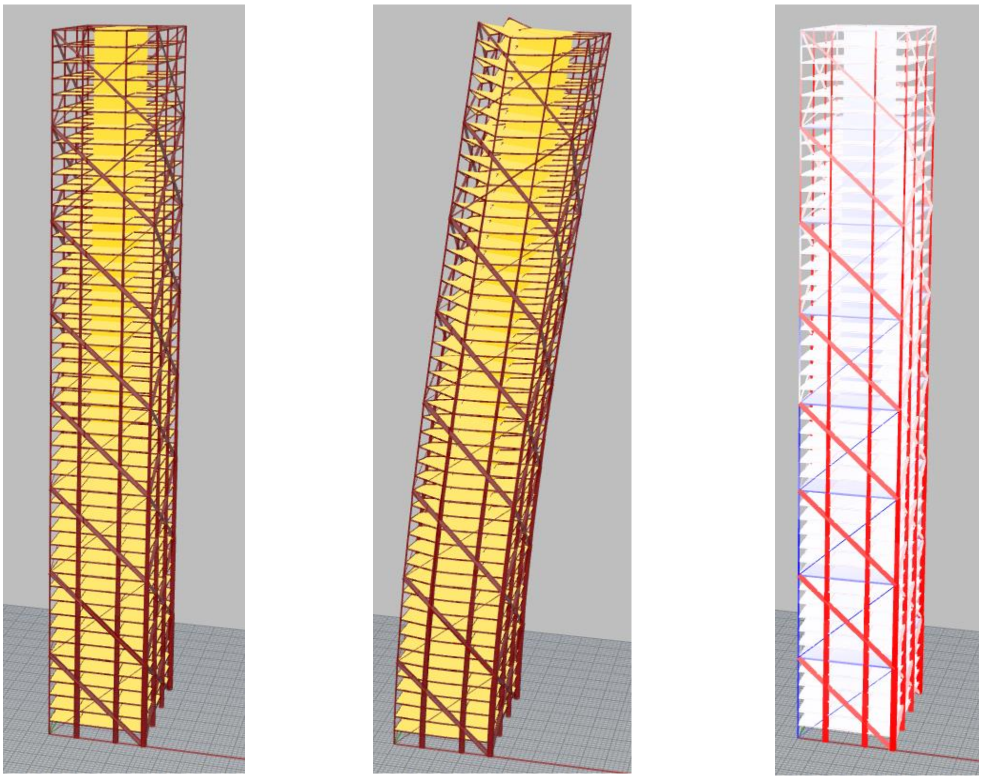


Fig. App. 32Megaframe with Concrete core. After optimization

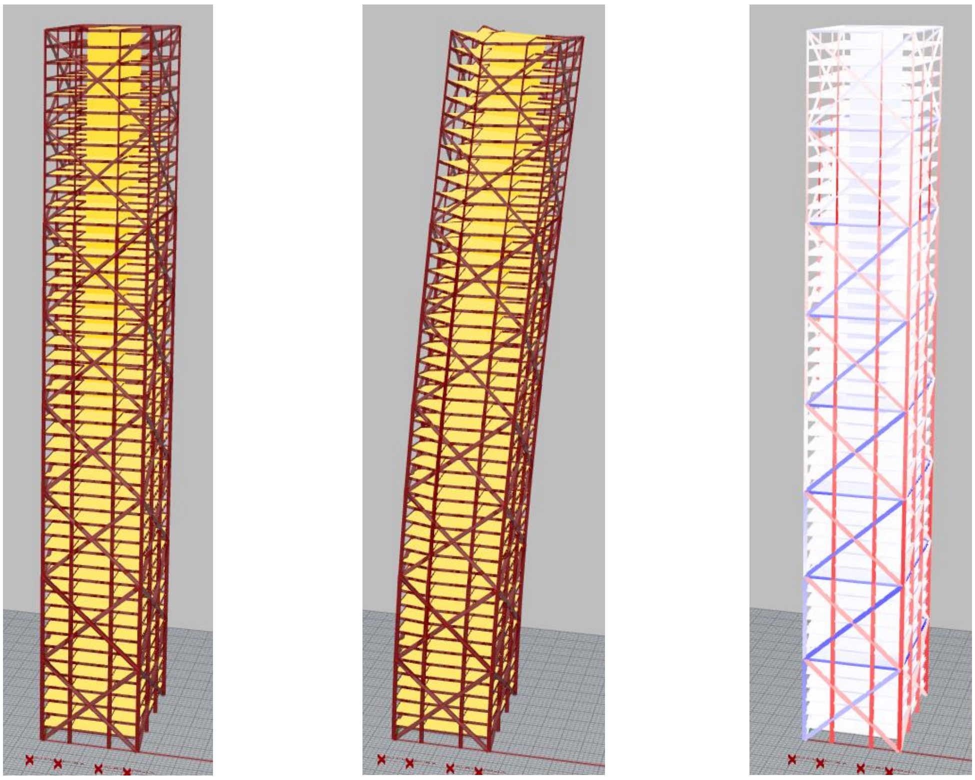
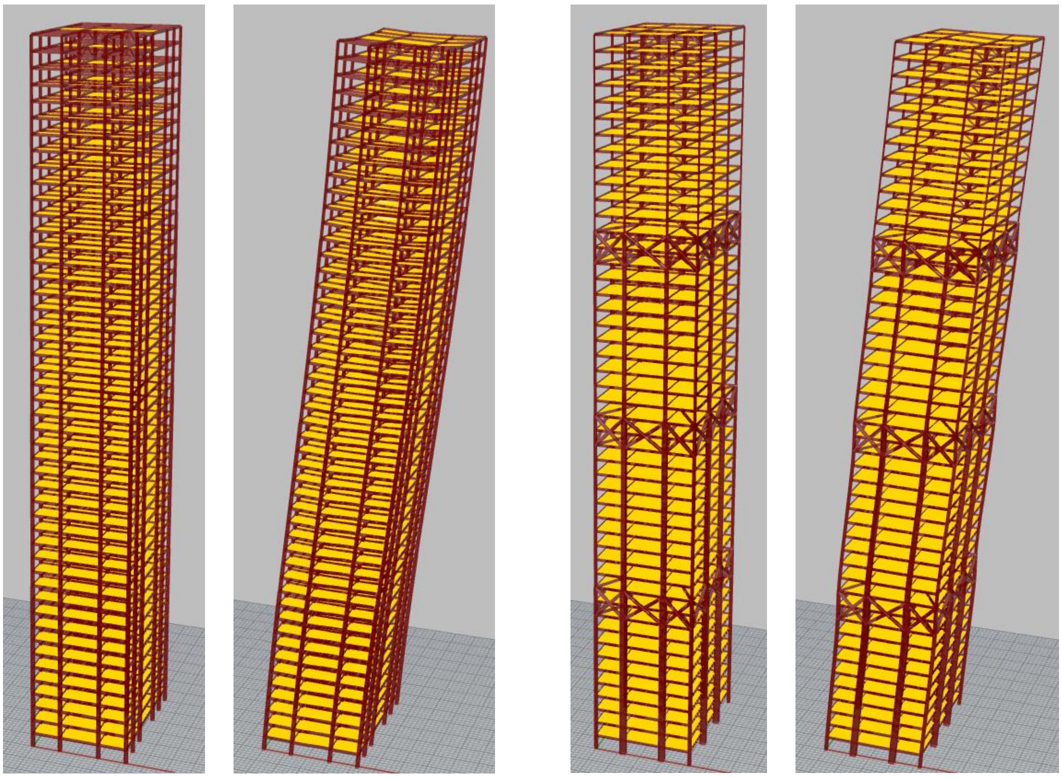
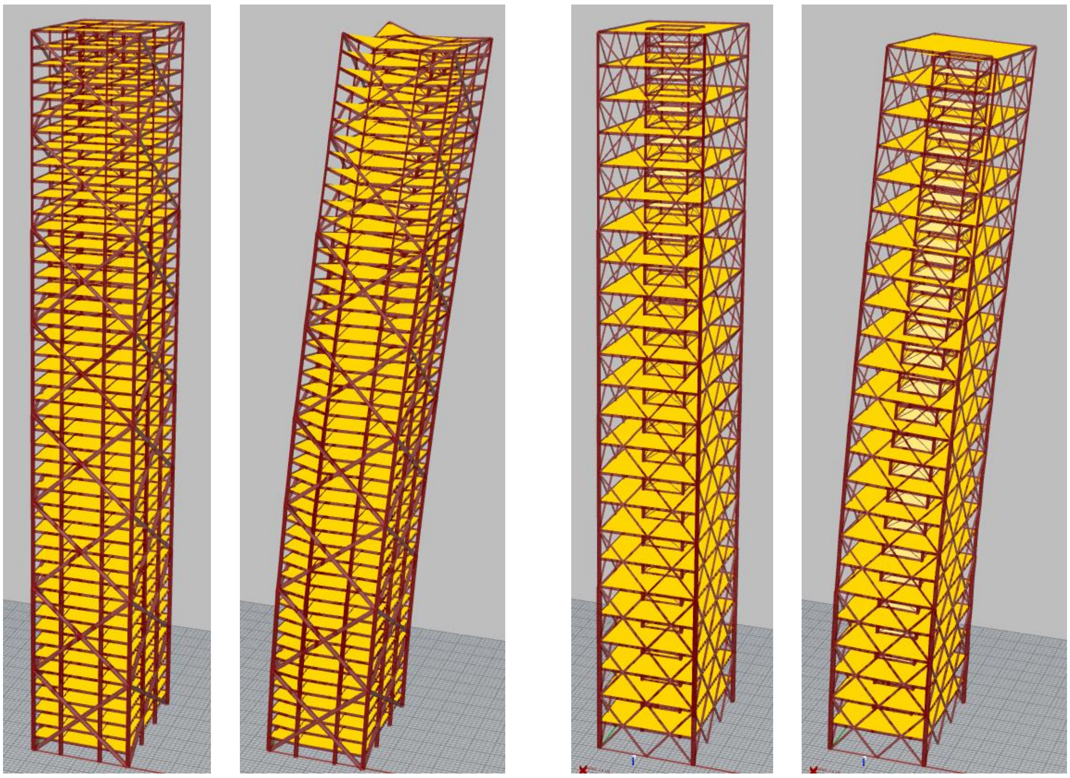


Fig. App. 33: Megaframe with concrete core. Modelled with "4 Fields Model".

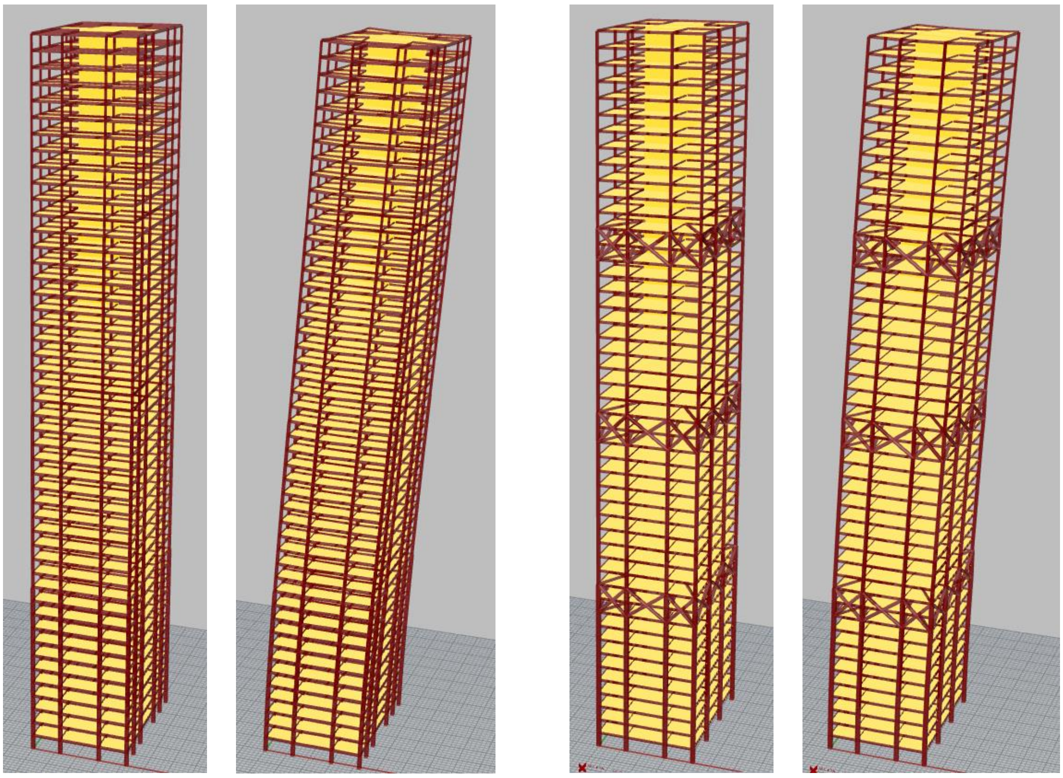


a) Moment Resistant Frame b) Outrigger

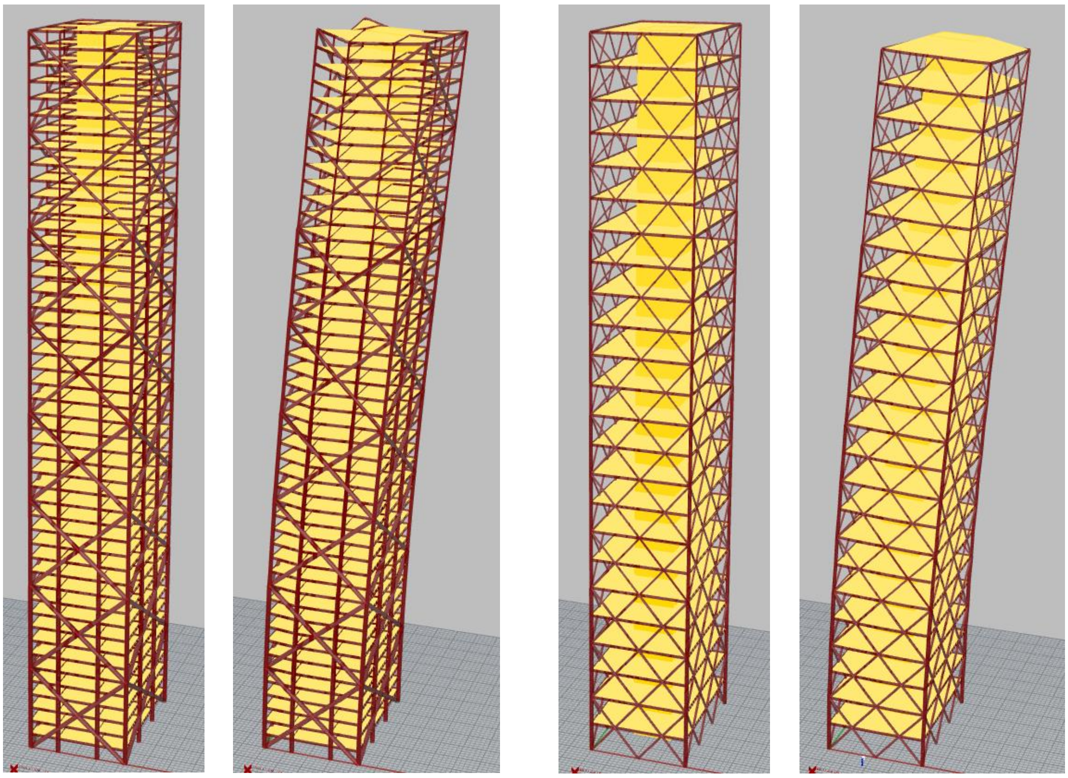


c) Megaframe d) Diagrid

Fig. App. 34: Model and deflection of the structural systems with steel core (“4 Fields Model").



a) Moment Resistant Frame b) Outrigger



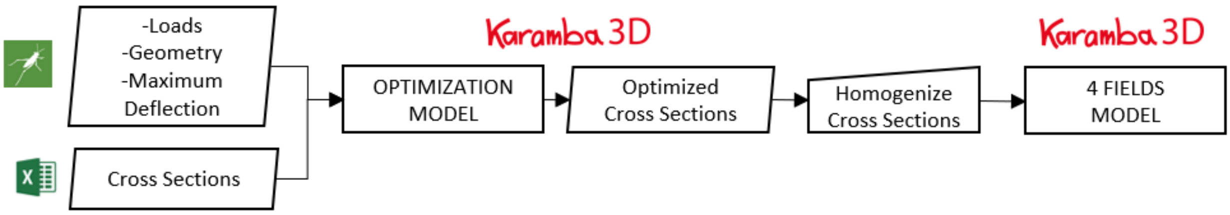
c) Megaframe d) Diagrid

Fig. App. 35: Model and deflection of the structural systems with concrete core (“4 Fields Model").

D. CROSS SECTION SUMMARY

In the following tables a summary of the cross sections used on the different structural systems is presented.

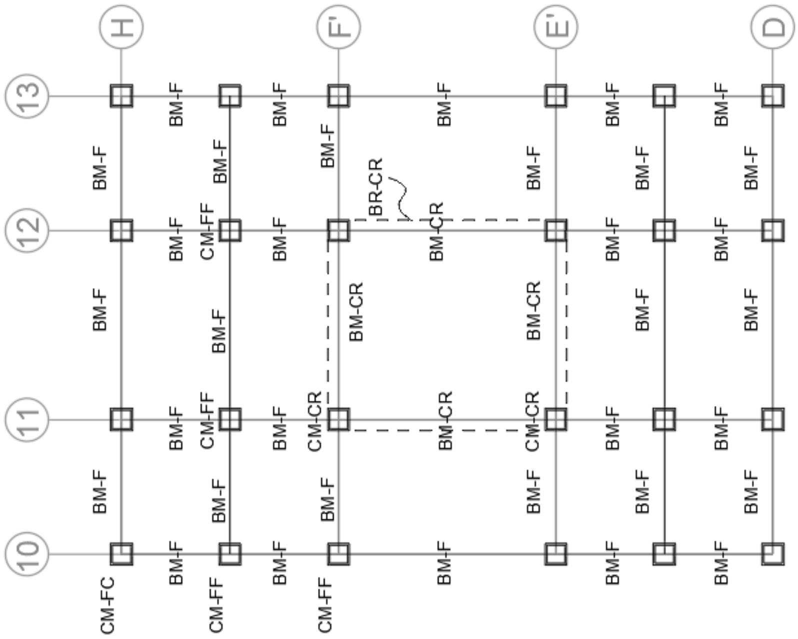
These cross sections were obtained by running the “optimize cross section” component of Karamba3D. After running the optimization component, the resultant elements were defined by several different cross sections, which resulted in a complex, non-realistic model. To simplify the model and to make it similar to a real structure, the building was divided in four sections along the height of the building. Subsequently, the cross sections, listed on the tables, were manually chosen and assigned to each of the 4 divisions of the model.



The cross sections of the columns are, in most of the cases, relatively big. This is the result of having 12 columns in the façade and 4 columns in the core. The small number of structural members makes it necessary to use bigger cross sections to satisfy the structural requirements. The large spans were defined to maximize the internal area. Hence, no columns were located between the core and the façade.

The elements are only made of steel. If composite action was considered, the size of these elements would be substantially smaller, however, this was neglected to account for elements made entirely of steel which would allow for an easy and efficient recycle at the end-of-life stage of the building.

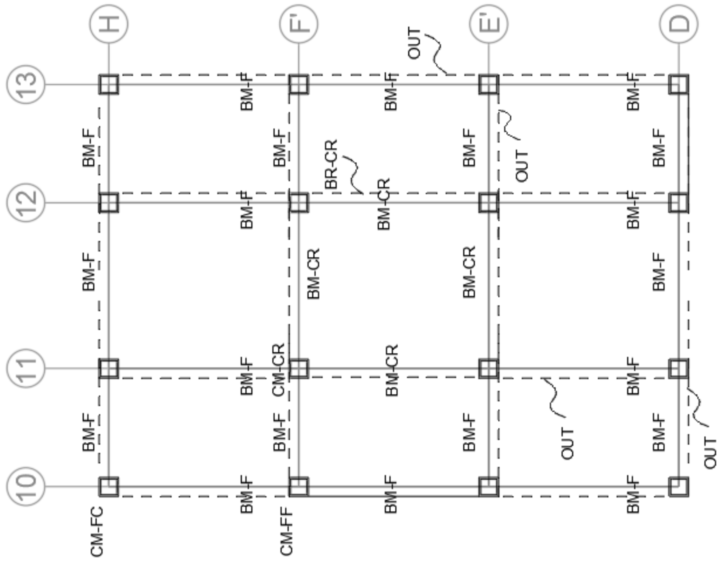
D. 1. MOMENT RESISTANT FRAME



	Element	Field 1	Field 2	Field 3	Field 4
11STMRF	Concrete Core Thickness - CCT				
	Core Beam - BMCR	HEM600	HEM600	HEM550	HEM550
	Façade Beam - BMF	HEA 600	HEA 600	HEA 600	HEA 600
	Core Column - CMCR	900x80	800x80	700x70	600x60
	Corner Façade Column - CMFC				
	Façade Column - CMFF	800x80	700x70	600x60	500x50
	Core Brace - BRCR	610x25	406x25	324x25	245x25
	Façade Brace - BRFF				
	Outrigger - OUT				

	Element	Field 1	Field 2	Field 3	Field 4
21CCMRF	Concrete Core Thickness - CCT	500	420	340	250
	Core Beam - BMCR				
	Façade Beam - BMF	HEA 600	HEA 600	HEA 600	HEA 600
	Core Column - CMCR				
	Corner Façade Column - CMFC				
	Façade Column - CMFF	800X70	600X60	500X50	400X40
	Core Brace - BRCR				
	Façade Brace - BRFF				
	Outrigger - OUT				

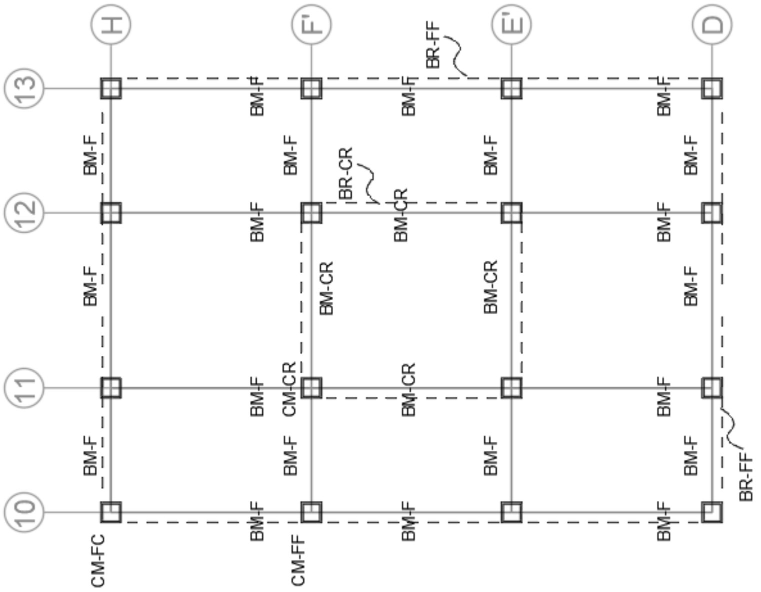
D. 2. OUTRIGGER



	Element	Field 1	Field 2	Field 3	Field 4
13STOUT	Concrete Core Thickness - CCT				
	Core Beam - BMCR	HEM550	HEM550	HEM550	HEM550
	Façade Beam - BMF	IPE550	IPE550	IPE550	IPE550
	Core Column - CMCR	1500X80	1300X80	1100X80	700X70
	Corner Façade Column - CMFC	700X70	600X60	500X50	300X25
	Façade Column - CMFF	1300X80	1000X80	700X60	400X40
	Core Brace - BRCR	760X25	610X25	508X25	324X25
	Façade Brace - BRFF				
	Outrigger - OUT	760X25	760X25	760X25	

	Element	Field 1	Field 2	Field 3	Field 4
23CCOUT	Concrete Core Thickness - CCT	500	420	340	250
	Core Beam - BMCR				
	Façade Beam - BMF	IPE550	IPE550	IPE550	IPE550
	Core Column - CMCR				
	Corner Façade Column - CMFC				
	Façade Column - CMFF	900X80	700X70	600X60	500X50
	Core Brace - BRCR				
	Façade Brace - BRFF				
	Outrigger - OUT	760X25	760X25	760X25	

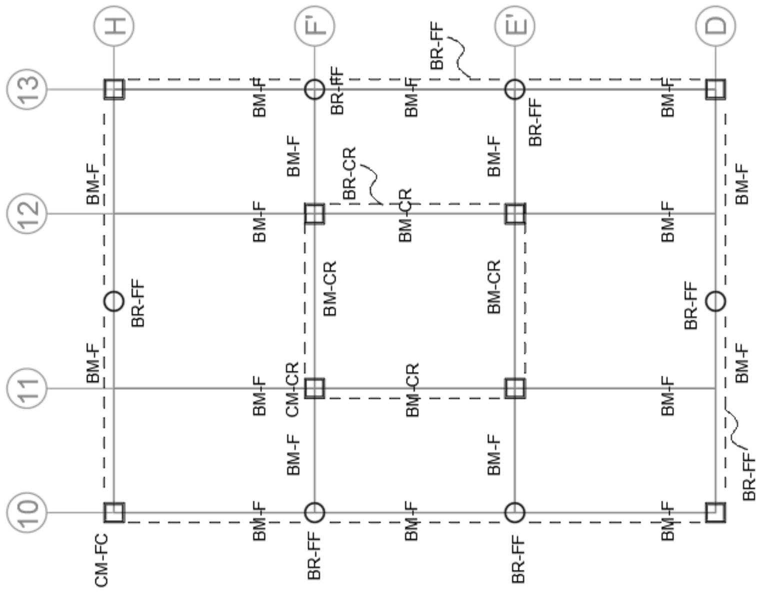
D. 3. MEGAFRAME



	Element	Field 1	Field 2	Field 3	Field 4
14STMGF	Concrete Core Thickness - CCT				
	Core Beam - BMCR	HEM550	HEM550	HEM550	HEM550
	Façade Beam - BMF	IPE550	IPE550	IPE550	IPE550
	Core Column - CMCR	1500X80	1300X80	1000X80	600X60
	Corner Façade Column - CMFC	800X80	700X60	600X50	400X30
	Façade Column - CMFF	800X70	700X60	600X50	300X30
	Core Brace - BRCR	406X25	356X25	324X25	245X25
	Façade Brace - BRFF	1016X25	914X25	760X25	610X25
	Outrigger - OUT				

	Element	Field 1	Field 2	Field 3	Field 4
24CCMGF	Concrete Core Thickness - CCT	500	420	340	250
	Core Beam - BMCR				
	Façade Beam - BMF	IPE550	IPE550	IPE550	IPE550
	Core Column - CMCR				
	Corner Façade Column - CMFC	800X80	700X60	600X50	400X30
	Façade Column - CMFF	800X70	700X60	600X50	300X30
	Core Brace - BRCR				
	Façade Brace - BRFF	1016X25	914X25	760X25	610X25
	Outrigger - OUT				

D. 4. DIAGRID



	Element	Field 1	Field 2	Field 3	Field 4
15STDGD	Concrete Core Thickness - CCT				
	Core Beam - BMCR				
	Façade Beam - BMF	IPE550	IPE550	IPE550	IPE550
	Core Column - CMCR	700X70	600X60	500X50	400X40
	Corner Façade Column - CMFC	800X80	700X60	600X50	400X30
	Façade Column - CMFF				
	Core Brace - BRCR	154X6	154X6	154X6	154X6
	Façade Brace - BRFF	508X25	406X25	356X25	324X25
	Outrigger - OUT				

	Element	Field 1	Field 2	Field 3	Field 4
25CCDGD	Concrete Core Thickness - CCT	500	420	340	250
	Core Beam - BMCR				
	Façade Beam - BMF	IPE550	IPE550	IPE550	IPE550
	Core Column - CMCR				
	Corner Façade Column - CMFC	700X70	600X60	500X50	300X30
	Façade Column - CMFF				
	Core Brace - BRCR				
	Façade Brace - BRFF	508X25	406X25	356X25	324X25
	Outrigger - OUT				

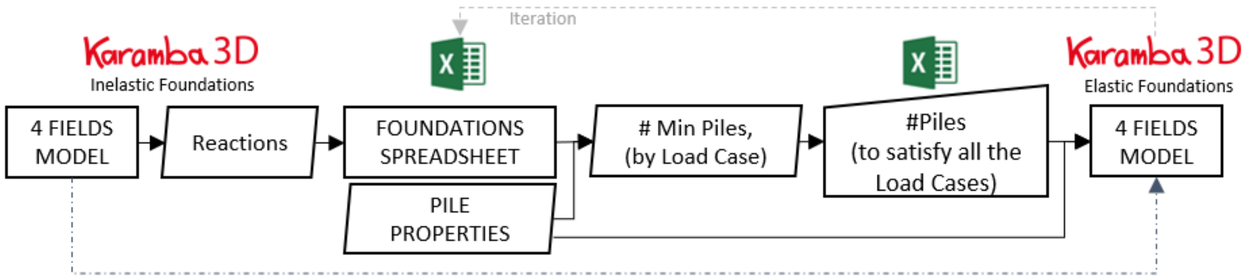
E. FOUNDATIONS

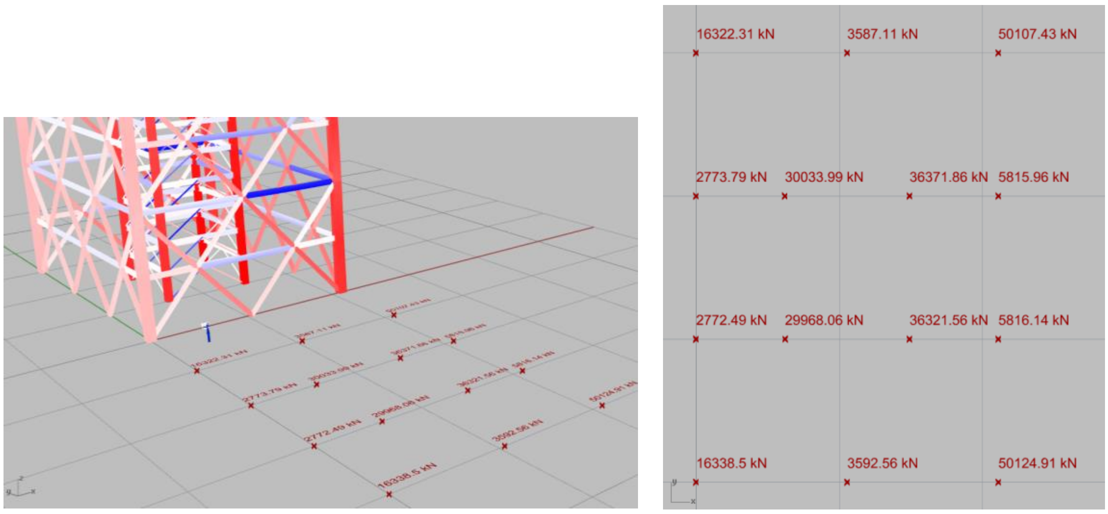
E. 1. CALCULATION OF NUMBER OF PILES

The use of lighter and stronger materials (steel instead of concrete) results in lighter structures. By decreasing the weight of the superstructure, the size of the substructure will be consequentially reduced. Furthermore, by using different structural systems with the main stability system located either internally (Moment Resistance Frames and Outrigger) or externally (Megaframe and Diagrid), the distribution of forces and reactions will vary from one system to another, and hence the corresponding piles on each system will also vary. And lastly, the inclusion of the piles on the FEM models provides a more accurate model by accounting for the contribution of the foundations to the horizontal displacement of the structure. Hence, the superstructure can be designed more efficiently.

To define the number of piles needed, a three steps algorithm was followed.

1. The Karamba3D model, with the optimized cross sections and the wind load distributed in four fields [Appendix C], was computed. The reactions at each support, resulting from the Load Combinations considered, were exported to an excel spread sheet. It is remarked that, on this first step, the supports were considered as inelastic.





a) b)

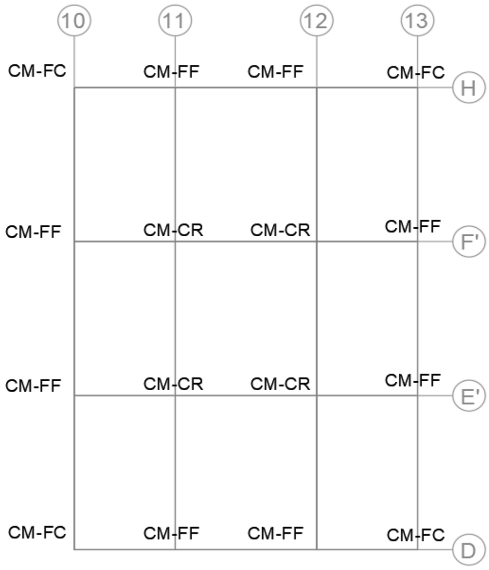
Fig. App. 36: Reactions of the Steel Diagrid 3D Model.
a) Reactions show where are the supports with the reference of the 3D model.
b) Value of the Reactions for a specific Load Case.

Additionally, the horizontal displacement at the top of the building was calculated (δ_{flex}). This displacement corresponds to the deflection of the building due to its flexural characteristics.

2. The minimum number of piles required was calculated for each of the load cases by using the pile properties provided by a supplier and the reactions obtained from the model. Then, the number of required piles was homogenized; i.e. in some supports the number of piles needed was lower from one Load Case to another, this was solved by using the Higher value of the minimum number of piles at each support. In this second step, three types of supports were defined: Core Columns, Façade Corner Columns, Façade Columns.

PILE PROPERTIES

Resistance	8,000	kN
Diameter	0.85	m
Length	29	m
E	600 x10^6	N/m
Density	2,500	kg/m^3
Area	0.57	m^2
c-t-c	1.87	m



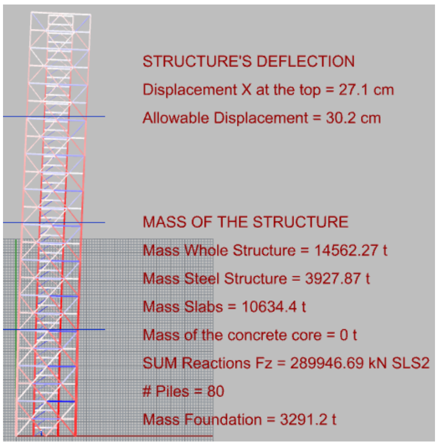
CM-FC	Corner Façade Column
CM-FF	Façade Column
CM-CR	Core Column

3. The minimum number of piles at each of the three support types was input in the Karamba3D model. These elements were modelled as springs with the structural properties provided by the supplier. For this step, only the contribution from the vertical component was considered, since the horizontal is relatively smaller.

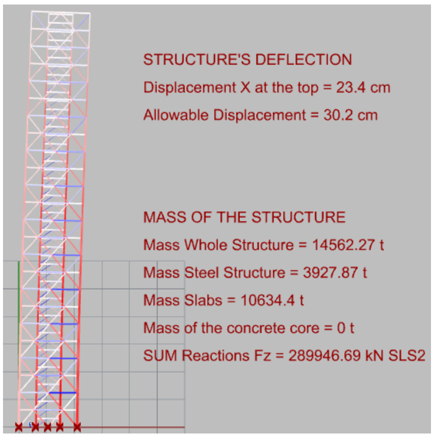
The new reactions were processed one more time in the excel spreadsheet to assure that the considered number of piles satisfied the elastic behaviour of the foundations. This iteration was repeated until the criterion was satisfied. (Usually 1 or 2 iterations and a variation of +/- 1 pile).

The results from the final structural models (Mass, Reactions, Displacement) were retrieved and processed for the Environmental Impact Assessment.

By implementing the elastic foundations, the horizontal displacements of the building could be differentiated as displacement due to flexural behaviour of the building (δ_{flex} from step 1), total displacement of the building (δ_{tot} from step 3) and rotational displacement ($\delta_{rot} = \delta_{tot} - \delta_{flex}$).



a) Total displacement



b) Flexural displacement

Fig. App. 37: Displacements of the Diagrid with Steel Core structure.

The following table contains a fraction of the spreadsheet used to calculate the number of piles. This table the contains the information for the Steel Diagrid, for the Load Combination 3 from the Karamba3D model. This was evaluated for the 8 structural systems and the 6 Load Combinations

The Vertical Reactions (Fz) were used to calculate the minimum number of piles required (green column). All the load cases were addressed simultaneously to verify the compliance of the system for all the load combination. The number of piles chosen (blue column) was exported to the Karamba3D model to evaluate the structure and the substructure together.

							[#]	[#]	[m2]	[mm]	[mm]	[m]	[mm/m]	[mm]
	X Coord	Y Coord	LC	Fz	Fx	Fy	#min	#	A	Un	Un - Umin	Xn - a	Phi	delta
FC	0.00	0.00	3	14,317	-81	1,031	1.79	2	1.13	11.93	1.42	-10.55	-0.13	-20.4
f	0.00	10.00	3	3,028	4	389	0.38	1	0.57	5.05	-5.46	-10.55	0.52	78.3
f	10.55	0.00	3	4,832	-3,215	2	0.60	1	0.57	8.05	-2.46	0.00	0.00	0.0
CR	6.20	10.00	3	38,844	-185	10	4.86	5	2.84	12.95	2.44	-4.35	-0.56	-84.7
CR	14.90	10.00	3	50,457	-200	10	6.31	7	3.97	12.01	1.50	4.35	0.35	52.3
f	0.00	20.00	3	3,030	4	-391	0.38	1	0.57	5.05	-5.46	-10.55	0.52	78.3
CR	6.20	20.00	3	38,930	-185	-10	4.87	5	2.84	12.98	2.47	-4.35	-0.57	-85.7
FC	21.10	0.00	3	75,056	-4,957	3,496	9.38	10	5.67	12.51	2.00	10.55	0.19	28.7
f	21.10	10.00	3	8,527	9	462	1.07	2	1.13	7.11	-3.40	10.55	-0.32	-48.8
CR	14.90	20.00	3	50,522	-200	-10	6.32	7	3.97	12.03	1.52	4.35	0.35	52.8
f	21.10	20.00	3	8,526	9	-464	1.07	2	1.13	7.11	-3.40	10.55	-0.32	-48.8
FC	0.00	30.00	3	14,297	-84	-1,029	1.79	2	1.13	11.91	1.40	-10.55	-0.13	-20.1
f	10.55	30.00	3	4,823	-3,215	-2	0.60	1	0.57	8.04	-2.47	0.00	0.00	0.0
FC	21.10	30.00	3	75,035	-4,954	-3,494	9.38	10	5.67	12.51	2.00	10.55	0.19	28.6

#min		#		[m2]	[mm]	[mm]	[m]	[mm / m]	[mm]	[cm]	[mm]
Piles		#	UC	A	Un	Un - Umin	Xn - a	Phi	delta	delta	diff delta
2	FC	10	5.0	5.67	2.39	-6.57	-10.55	0.62	94.1	9.4	12.8
1	f	2	2.0	1.13	2.52	-6.43	-10.55	0.61	92.2	9.2	14.7
1	f	2	2.0	1.13	4.03	-4.93	0.00	0.00	0.0	0.0	106.9
5	CR	7	1.4	3.97	9.25	0.29	-4.35	-0.07	-10.2	-1.0	117.1
7	CR	7	1.0	3.97	12.01	3.06	4.35	0.70	106.3	10.6	0.5
1	f	2	2.0	1.13	2.52	-6.43	-10.55	0.61	92.1	9.2	14.7
5	CR	7	1.4	3.97	9.27	0.32	-4.35	-0.07	-11.0	-1.1	117.8
10	FC	10	1.0	5.67	12.51	3.56	10.55	0.34	51.0	5.1	55.9
2	f	2	1.0	1.13	7.11	-1.85	10.55	-0.18	-26.5	-2.6	133.4
7	CR	7	1.0	3.97	12.03	3.08	4.35	0.71	106.9	10.7	0.0
2	f	2	1.0	1.13	7.11	-1.85	10.55	-0.18	-26.5	-2.6	133.4
2	FC	10	5.0	5.67	2.38	-6.57	-10.55	0.62	94.2	9.4	12.7
1	f	2	2.0	1.13	4.02	-4.93	0.00	0.00	0.0	0.0	106.9
10	FC	10	1.0	5.67	12.51	3.55	10.55	0.34	50.9	5.1	56.0

Furthermore, the differential settlements of the piles were calculated at each support to calculate the horizontal displacement at the top of the building. These displacements are not equal to the ones obtained from the Karambam3D model, because the imply individual behaviour of the supports. However, they provide a good estimate of the displacement due to the foundation rotation.

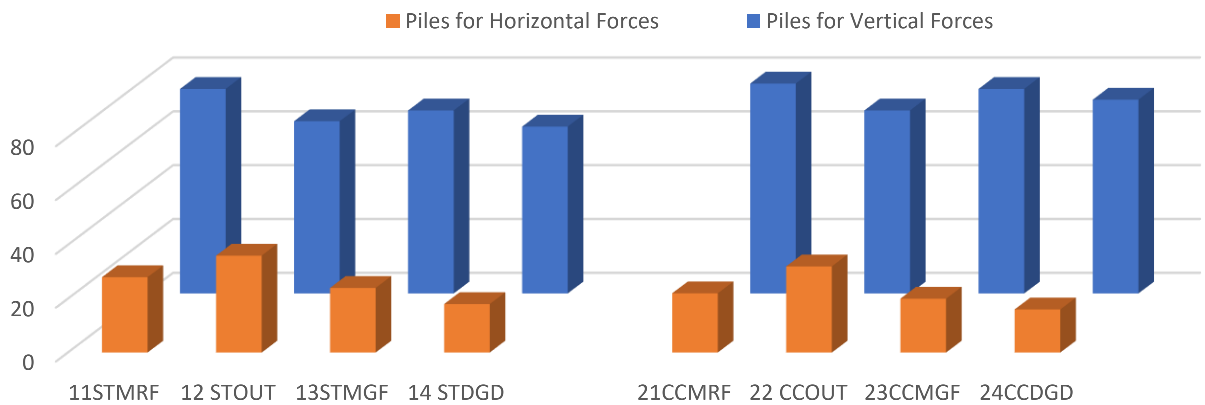
E. 2. CONTRIBUTION FROM BENDING MOMENT DUE TO HORIZONTAL FORCES.

In addition to the vertical forces due to gravity, the reactions generated in the supports due to the bending moment at the base of the building, resultant from the horizontal loads, contribute to the required resistance of the foundations. To analyse the contribution each of these contributors, their share to the total requirements was calculated.

First, the requirements due to the vertical force were obtained by using only the first two Load Combinations (the vertical loads were less onerous in the following load cases, thus non-governing). The number of piles to satisfy these criteria was then compared with the number of piles needed to fulfil all the Load Cases. The difference between these values correspond to the number of additional piles required to support the horizontal wind load.

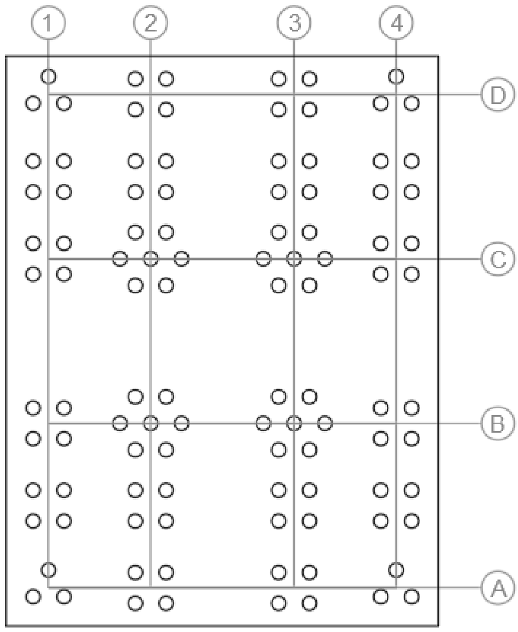
The following table contains the summary of the piles needed for the cases a) Considering all Load Combinations, and b) Considering only Load Case ULS1 (which correspond to the most onerous of the gravitational loads combination). The fourth column contains the difference between the first two rows, which corresponds to the number of piles required to support the horizontal load. Finally, the fifth and sixth column contain the percentage of the piles required for the vertical (gravitational) and horizontal (wind) requirements.

	Total Piles case a)	Piles for Vertical Loads case b)	Piles for Horizontal Loads	%VERT	%HOR
STMRF	104	76	28	73%	27%
STOUT	100	64	36	64%	36%
STMGF	92	68	24	74%	26%
STDGD	80	62	18	78%	23%
CCMRF	100	78	22	78%	22%
CCOUT	100	68	32	68%	32%
CCMGF	96	76	20	79%	21%
CCDGD	88	72	16	82%	18%

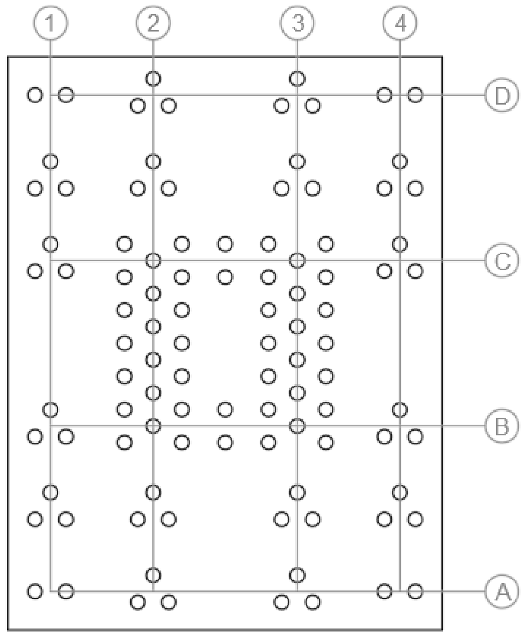


E. 3. PILE DISTRIBUTION SUMMARY

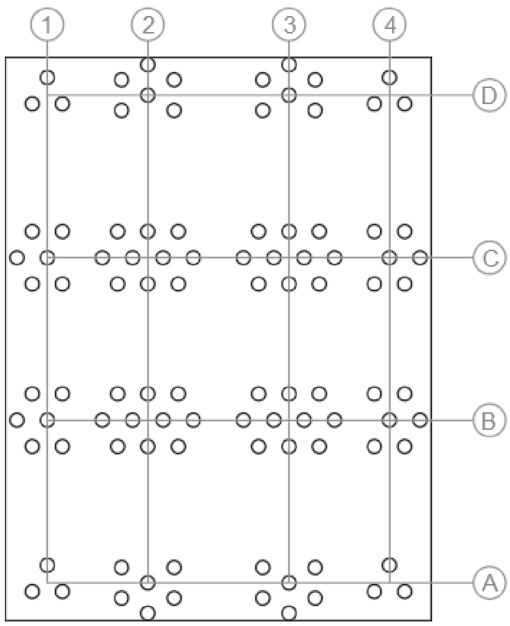
The following figures represent the location of the foundation piles for the different structural systems.



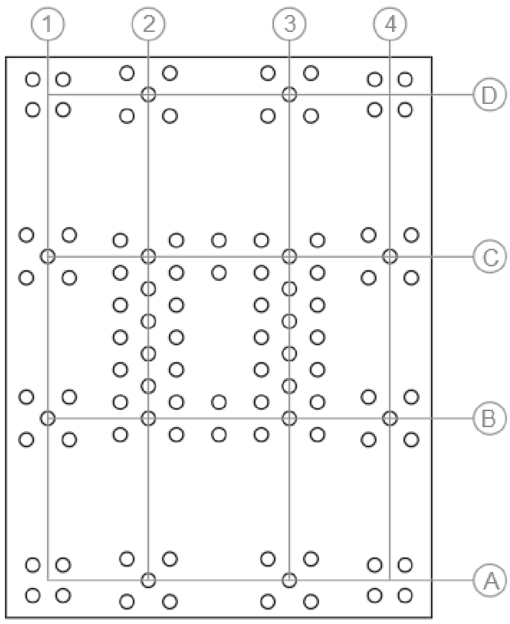
a) STMRF pile distribution



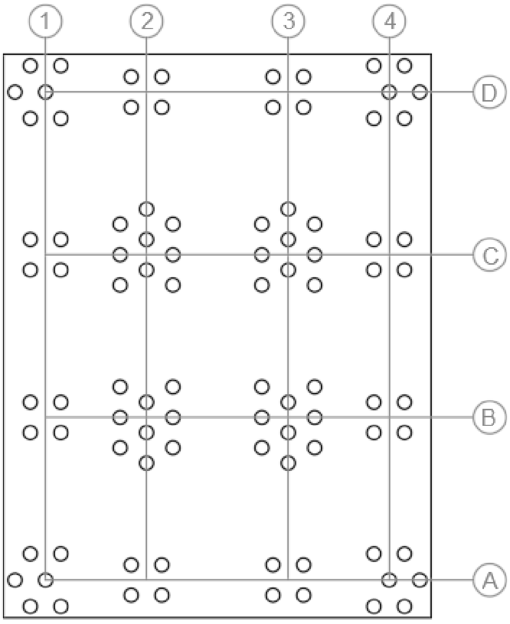
b) CCMRF pile distribution



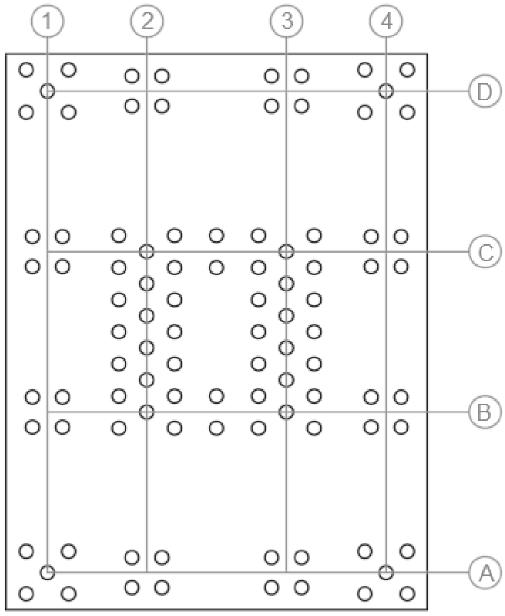
a) STOUT pile distribution



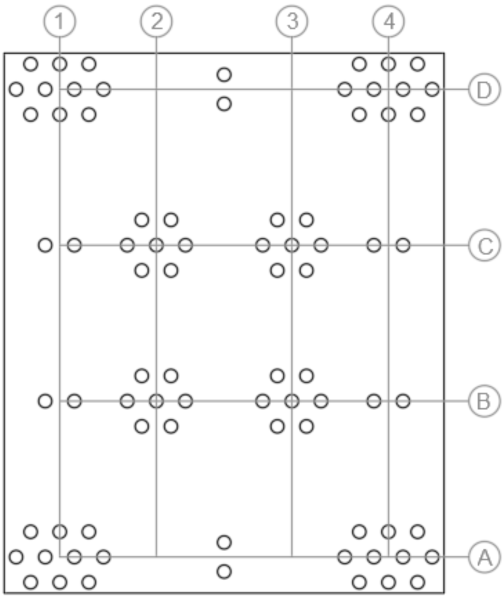
b) CCOUT pile distribution



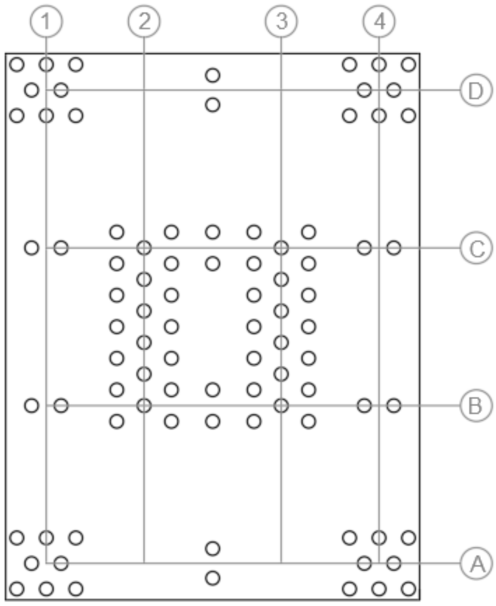
a) STMGF pile distribution



b) CCMGF pile distribution



a) STDGD pile distribution



b) CCDGD pile distribution

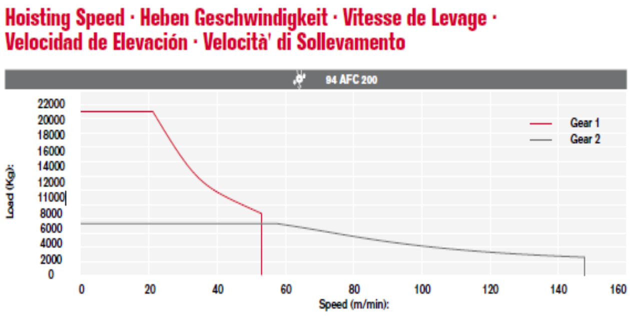
F. HIGH-RISE FACTOR

The consideration of high-rise buildings on the research implies the use of high cranes. Besides the structural challenges that arise from increasing the height of the building, more areas are affected from this. One of them is the construction, and consequently the de-construction, of the building. This section addresses the fact that, when the height increases the vertical transportation becomes more important, as the time to assemble one element becomes larger due to the time spent on the crane. This affects the cost of the construction (i.e. more crane operation time) and the time of the construction per se. The following calculation presents the derivation of a factor to account for the additional time increased.

If it is considered that the lifting speed is the same for all the elements, and that there are the same elements per floor. The total time it takes to assemble one element can be calculated as:

$T = T_{load} + T_{assembly} + T_{travel}.$ Where: $T_{travel} = T_{horizontal} + T_{up} + T_{down}$

For this study the fixed times were considered as: Tload =4min, Tassemby=5.5min, Thorizontal=0.5min. The variable times (Tup and Tdown) are dependent on the height of the building and on the speed of the crane. (T = height / Speed). The speed of the crane was obtained by considering a Terex Crane CTT 721-40 HD23.



Source: http://www.stknl.com/site/wp-content/uploads/CTT-721-40-HD23.pdf

The following table was elaborated to calculate the time increment as a function of the height.

The middle section shows the time it takes to assemble a floor at a certain height. However, to build it, the previous floors should've been built beforehand, for this reason the las section shows the accumulated times. These last ones represent the time it would take to build up to a certain level (for instance floors 1,2,3…48, instead of only the floor 48).

These values were interpolated to derive the factor for a 48 floors building compared to a 10 floors building. To account for the fact that not all the buildings considered on the JRC study were high rise buildings but low-rise buildings. However, as it is discussed on the conclusions section, even if this factor may influence heavily on the construction phase (on this case it is increased by 17%), this factor does not represent a relevant contribution to the whole Life Cycle of the Building, because the construction phase has a small share of the total GWP.

or height	3.15	m/min																
V	50	m/min	Ths is highly dependant on the weight lifted															
Vdown	150	m/min	Taken as the maximum value															
								per element per floor					accumm per element at the floor X					
Load	Assembly	Rad	rad/v	# floors	H	H/v	H/vdown	fixed tim	var time	total	% fix	%var	fix	var	tot	%fix	%var	
min	min	m	min	#	m	min	min											
4	5.5	25	0.5	1	3.15	0.063	0.021	10.0	0.1	10.1	99.2%	0.8%	10.0	0.1	10.1	99.2%	0.8%	
4	5.5	25	0.5	2	6.3	0.126	0.042	10.0	0.2	10.2	98.3%	1.7%	20	0.252	20.252	98.8%	1.2%	
4	5.5	25	0.5	3	9.45	0.189	0.063	10.0	0.3	10.3	97.5%	2.5%	30	0.504	30.504	98.3%	1.7%	
4	5.5	25	0.5	4	12.6	0.252	0.084	10.0	0.3	10.3	96.7%	3.3%	40	0.84	40.84	97.9%	2.1%	
4	5.5	25	0.5	5	15.75	0.315	0.105	10.0	0.4	10.4	96.0%	4.0%	50	1.26	51.26	97.5%	2.5%	
4	5.5	25	0.5	6	18.9	0.378	0.126	10.0	0.5	10.5	95.2%	4.8%	60	1.764	61.764	97.1%	2.9%	
4	5.5	25	0.5	7	22.05	0.441	0.147	10.0	0.6	10.6	94.4%	5.6%	70	2.352	72.352	96.7%	3.3%	
4	5.5	25	0.5	8	25.2	0.504	0.168	10.0	0.7	10.7	93.7%	6.3%	80	3.024	83.024	96.4%	3.6%	
4	5.5	25	0.5	9	28.35	0.567	0.189	10.0	0.8	10.8	93.0%	7.0%	90	3.78	93.78	96.0%	4.0%	
4	5.5	25	0.5	10	31.5	0.63	0.21	10.0	0.8	10.8	92.3%	7.7%	100	4.62	104.62	95.6%	4.4%	prod 10 stor
4	5.5	25	0.5	11	34.65	0.693	0.231	10.0	0.9	10.9	91.5%	8.5%	110	5.544	115.544	95.2%	4.8%	
4	5.5	25	0.5	12	37.8	0.756	0.252	10.0	1.0	11.0	90.8%	9.2%	120	6.552	126.552	94.8%	5.2%	
4	5.5	25	0.5	13	40.95	0.819	0.273	10.0	1.1	11.1	90.2%	9.8%	130	7.644	137.644	94.4%	5.6%	
4	5.5	25	0.5	14	44.1	0.882	0.294	10.0	1.2	11.2	89.5%	10.5%	140	8.82	148.82	94.1%	5.9%	
4	5.5	25	0.5	15	47.25	0.945	0.315	10.0	1.3	11.3	88.8%	11.2%	150	10.08	160.08	93.7%	6.3%	
4	5.5	25	0.5	16	50.4	1.008	0.336	10.0	1.3	11.3	88.2%	11.8%	160	11.424	171.424	93.3%	6.7%	
4	5.5	25	0.5	17	53.55	1.071	0.357	10.0	1.4	11.4	87.5%	12.5%	170	12.852	182.852	93.0%	7.0%	
4	5.5	25	0.5	18	56.7	1.134	0.378	10.0	1.5	11.5	86.9%	13.1%	180	14.364	194.364	92.6%	7.4%	
4	5.5	25	0.5	19	59.85	1.197	0.399	10.0	1.6	11.6	86.2%	13.8%	190	15.96	205.96	92.3%	7.7%	
4	5.5	25	0.5	20	63	1.26	0.42	10.0	1.7	11.7	85.6%	14.4%	200	17.64	217.64	91.9%	8.1%	20 story
4	5.5	25	0.5	21	66.15	1.323	0.441	10.0	1.8	11.8	85.0%	15.0%	210	19.404	229.404	91.5%	8.5%	
4	5.5	25	0.5	22	69.3	1.386	0.462	10.0	1.8	11.8	84.4%	15.6%	220	21.252	241.252	91.2%	8.8%	
4	5.5	25	0.5	23	72.45	1.449	0.483	10.0	1.9	11.9	83.8%	16.2%	230	23.184	253.184	90.8%	9.2%	
4	5.5	25	0.5	24	75.6	1.512	0.504	10.0	2.0	12.0	83.2%	16.8%	240	25.2	265.2	90.5%	9.5%	
4	5.5	25	0.5	25	78.75	1.575	0.525	10.0	2.1	12.1	82.6%	17.4%	250	27.3	277.3	90.2%	9.8%	
4	5.5	25	0.5	26	81.9	1.638	0.546	10.0	2.2	12.2	82.1%	17.9%	260	29.484	289.484	89.8%	10.2%	
4	5.5	25	0.5	27	85.05	1.701	0.567	10.0	2.3	12.3	81.5%	18.5%	270	31.752	301.752	89.5%	10.5%	
4	5.5	25	0.5	28	88.2	1.764	0.588	10.0	2.4	12.4	81.0%	19.0%	280	34.104	314.104	89.1%	10.9%	
4	5.5	25	0.5	29	91.35	1.827	0.609	10.0	2.4	12.4	80.4%	19.6%	290	36.54	326.54	88.8%	11.2%	
4	5.5	25	0.5	30	94.5	1.89	0.63	10.0	2.5	12.5	79.9%	20.1%	300	39.06	339.06	88.5%	11.5%	30 story
4	5.5	25	0.5	31	97.65	1.953	0.651	10.0	2.6	12.6	79.3%	20.7%	310	41.664	351.664	88.2%	11.8%	
4	5.5	25	0.5	32	100.8	2.016	0.672	10.0	2.7	12.7	78.8%	21.2%	320	44.352	364.352	87.8%	12.2%	
4	5.5	25	0.5	33	103.95	2.079	0.693	10.0	2.8	12.8	78.3%	21.7%	330	47.124	377.124	87.5%	12.5%	
4	5.5	25	0.5	34	107.1	2.142	0.714	10.0	2.9	12.9	77.8%	22.2%	340	49.98	389.98	87.2%	12.8%	
4	5.5	25	0.5	35	110.25	2.205	0.735	10.0	2.9	12.9	77.3%	22.7%	350	52.92	402.92	86.9%	13.1%	
4	5.5	25	0.5	36	113.4	2.268	0.756	10.0	3.0	13.0	76.8%	23.2%	360	55.944	415.944	86.6%	13.4%	
4	5.5	25	0.5	37	116.55	2.331	0.777	10.0	3.1	13.1	76.3%	23.7%	370	59.052	429.052	86.2%	13.8%	
4	5.5	25	0.5	38	119.7	2.394	0.798	10.0	3.2	13.2	75.8%	24.2%	380	62.244	442.244	85.9%	14.1%	
4	5.5	25	0.5	39	122.85	2.457	0.819	10.0	3.3	13.3	75.3%	24.7%	390	65.52	455.52	85.6%	14.4%	
4	5.5	25	0.5	40	126	2.52	0.84	10.0	3.4	13.4	74.9%	25.1%	400	68.88	468.88	85.3%	14.7%	40 story
4	5.5	25	0.5	41	129.15	2.583	0.861	10.0	3.4	13.4	74.4%	25.6%	410	72.324	482.324	85.0%	15.0%	
4	5.5	25	0.5	42	132.3	2.646	0.882	10.0	3.5	13.5	73.9%	26.1%	420	75.852	495.852	84.7%	15.3%	
4	5.5	25	0.5	43	135.45	2.709	0.903	10.0	3.6	13.6	73.5%	26.5%	430	79.464	509.464	84.4%	15.6%	
4	5.5	25	0.5	44	138.6	2.772	0.924	10.0	3.7	13.7	73.0%	27.0%	440	83.16	523.16	84.1%	15.9%	
4	5.5	25	0.5	45	141.75	2.835	0.945	10.0	3.8	13.8	72.6%	27.4%	450	86.94	536.94	83.8%	16.2%	
4	5.5	25	0.5	46	144.9	2.898	0.966	10.0	3.9	13.9	72.1%	27.9%	460	90.804	550.804	83.5%	16.5%	
4	5.5	25	0.5	47	148.05	2.961	0.987	10.0	3.9	13.9	71.7%	28.3%	470	94.752	564.752	83.2%	16.8%	
4	5.5	25	0.5	48	151.2	3.024	1.008	10.0	4.0	14.0	71.3%	28.7%	480	98.784	578.784	82.9%	17.1%	48 story
4	5.5	25	0.5	49	154.35	3.087	1.029	10.0	4.1	14.1	70.8%	29.2%	490	102.9	592.9	82.6%	17.4%	
4	5.5	25	0.5	50	157.5	3.15	1.05	10.0	4.2	14.2	70.4%	29.6%	500	107.1	607.1	82.4%	17.6%	50 story
4	5.5	25	0.5	51	160.65	3.213	1.071	10.0	4.3	14.3	70.0%	30.0%	510	111.384	621.384	82.1%	17.9%	
4	5.5	25	0.5	52	163.8	3.276	1.092	10.0	4.4	14.4	69.6%	30.4%	520	115.752	635.752	81.8%	18.2%	
4	5.5	25	0.5	53	166.95	3.339	1.113	10.0	4.5	14.5	69.2%	30.8%	530	120.204	650.204	81.5%	18.5%	

G. FIRE RESISTANCE

To consider that the structural elements satisfy the fire safety requirements, a simplified analysis was performed [52]. The following steps describe how the analysis was performed:

1. A critical time of 550ºC and the fire resistance time of 150min was chosen.

2. Fire proofing by means of intumescent paint was chosen. The effective thermal conductivity at 550º is $\lambda = 0.02\ [W/m * K]$ [53].

3. The Cross-Section Area and the Perimeter of the steel profiles that conform the structure was obtained for all the steel elements. Subsequently, the value $\frac{A_p}{V}$ was computed.

4. A fire resistant coat thickness of $d = 2mm$ was selected.

5. With the spreadsheet, the values $\frac{A_p}{V}\frac{\lambda}{d}$ were calculated for all the cross sections.

		Structural Element					Fire Proofing Intumescent Paint							Ratio paint/steel	
		Width	thickness	W-2th	ρ steel	Mass/m	Ap	V	Ap/V	λ	d	A/V*λ/d	ρ paint	Mass pain	ratio
COLUMNS		m	m	m	kg/m3	kg/m	m2/m	m2	-	W/m.K	m	-	kg/m3	kg/m	kg/kg
11STMRF	Façade	0.90	0.08	0.74	7800	2047	3.6	0.262	13.72	0.02	0.002	137.2	1450	10.44	0.0051
	Façade	0.80	0.08	0.64	7800	1797	3.2	0.230	13.89	0.02	0.002	138.9	1450	9.28	0.0052
	Façade	0.70	0.07	0.56	7800	1376	2.8	0.176	15.87	0.02	0.002	158.7	1450	8.12	0.0059
	Façade	0.60	0.06	0.48	7800	1011	2.4	0.130	18.52	0.02	0.002	185.2	1450	6.96	0.0069
	Core	0.80	0.08	0.64	7800	1797	3.2	0.230	13.89	0.02	0.002	138.9	1450	9.28	0.0052
	Core	0.70	0.07	0.56	7800	1376	2.8	0.176	15.87	0.02	0.002	158.7	1450	8.12	0.0059
	Core	0.60	0.06	0.48	7800	1011	2.4	0.130	18.52	0.02	0.002	185.2	1450	6.96	0.0069
	Core	0.50	0.05	0.4	7800	702	2	0.090	22.22	0.02	0.002	222.2	1450	5.8	0.0083
21CCMRF	Façade	0.80	0.07	0.66	7800	1594	3.2	0.204	15.66	0.02	0.002	156.6	1450	9.28	0.0058
	Façade	0.60	0.06	0.48	7800	1011	2.4	0.130	18.52	0.02	0.002	185.2	1450	6.96	0.0069
	Façade	0.50	0.05	0.4	7800	702	2	0.090	22.22	0.02	0.002	222.2	1450	5.8	0.0083
	Façade	0.40	0.04	0.32	7800	449	1.6	0.058	27.78	0.02	0.002	277.8	1450	4.64	0.0103
12STOUT	Façade	1.50	0.08	1.34	7800	3544	6	0.454	13.20	0.02	0.002	132.0	1450	17.4	0.0049
	Façade	1.30	0.08	1.14	7800	3045	5.2	0.390	13.32	0.02	0.002	133.2	1450	15.08	0.0050
	Façade	1.10	0.08	0.94	7800	2546	4.4	0.326	13.48	0.02	0.002	134.8	1450	12.76	0.0050
	Façade	0.70	0.07	0.56	7800	1376	2.8	0.176	15.87	0.02	0.002	158.7	1450	8.12	0.0059
	Core	1.30	0.08	1.14	7800	3045	5.2	0.390	13.32	0.02	0.002	133.2	1450	15.08	0.0050
	Core	1.00	0.08	0.84	7800	2296	4	0.294	13.59	0.02	0.002	135.9	1450	11.6	0.0051
	Core	0.70	0.06	0.58	7800	1198	2.8	0.154	18.23	0.02	0.002	182.3	1450	8.12	0.0068
	Core	0.40	0.04	0.32	7800	449	1.6	0.058	27.78	0.02	0.002	277.8	1450	4.64	0.0103
														0	
22CCOUT	Façade	0.90	0.08	0.74	7800	2047	3.6	0.262	13.72	0.02	0.002	137.2	1450	10.44	0.0051
	Façade	0.70	0.07	0.56	7800	1376	2.8	0.176	15.87	0.02	0.002	158.7	1450	8.12	0.0059
	Façade	0.60	0.06	0.48	7800	1011	2.4	0.130	18.52	0.02	0.002	185.2	1450	6.96	0.0069
	Façade	0.50	0.05	0.4	7800	702	2	0.090	22.22	0.02	0.002	222.2	1450	5.8	0.0083

Table App. 1:Fire Proofing of Columns

		Structural Element					Fire Proofing Intumescent Paint							Ratio paint/steel	
		Width	thickness	W-2th	ρ steel	Mass/m	Ap	V	Ap/V	λ	d	A/V*λ/d	ρ paint	Mass pain	ratio
	COLUMNS	m	m	m	kg/m3	kg/m	m2/m	m2	-	W/m.K	m	-	kg/m3	kg/m	kg/kg
13STMGF	Façade	1.50	0.08	1.34	7800	3544	6	0.454	13.20	0.02	0.002	132.0	1450	17.4	0.0049
	Façade	1.30	0.08	1.14	7800	3045	5.2	0.390	13.32	0.02	0.002	133.2	1450	15.08	0.0050
	Façade	1.00	0.08	0.84	7800	2296	4	0.294	13.59	0.02	0.002	135.9	1450	11.6	0.0051
	Façade	0.60	0.03	0.54	7800	534	2.4	0.068	35.09	0.02	0.002	350.9	1450	6.96	0.0130
	Core	0.80	0.08	0.64	7800	1797	3.2	0.230	13.89	0.02	0.002	138.9	1450	9.28	0.0052
	Core	0.70	0.06	0.58	7800	1198	2.8	0.154	18.23	0.02	0.002	182.3	1450	8.12	0.0068
	Core	0.60	0.05	0.5	7800	858	2.4	0.110	21.82	0.02	0.002	218.2	1450	6.96	0.0081
	Core	0.40	0.03	0.34	7800	346	1.6	0.044	36.04	0.02	0.002	360.4	1450	4.64	0.0134
23CCMGF	Façade	0.80	0.08	0.64	7800	1797	3.2	0.230	13.89	0.02	0.002	138.9	1450	9.28	0.0052
	Façade	0.70	0.06	0.58	7800	1198	2.8	0.154	18.23	0.02	0.002	182.3	1450	8.12	0.0068
	Façade	0.60	0.05	0.5	7800	858	2.4	0.110	21.82	0.02	0.002	218.2	1450	6.96	0.0081
	Façade	0.40	0.03	0.34	7800	346	1.6	0.044	36.04	0.02	0.002	360.4	1450	4.64	0.0134
14STDGD	Façade	0.80	0.08	0.64	7800	1797	3.2	0.230	13.89	0.02	0.002	138.9	1450	9.28	0.0052
	Façade	0.70	0.06	0.58	7800	1198	2.8	0.154	18.23	0.02	0.002	182.3	1450	8.12	0.0068
	Façade	0.60	0.05	0.5	7800	858	2.4	0.110	21.82	0.02	0.002	218.2	1450	6.96	0.0081
	Façade	0.40	0.03	0.34	7800	346	1.6	0.044	36.04	0.02	0.002	360.4	1450	4.64	0.0134
	Core	0.70	0.07	0.56	7800	1376	2.8	0.176	15.87	0.02	0.002	158.7	1450	8.12	0.0059
	Core	0.60	0.06	0.48	7800	1011	2.4	0.130	18.52	0.02	0.002	185.2	1450	6.96	0.0069
	Core	0.50	0.05	0.4	7800	702	2	0.090	22.22	0.02	0.002	222.2	1450	5.8	0.0083
	Core	0.40	0.04	0.32	7800	449	1.6	0.058	27.78	0.02	0.002	277.8	1450	4.64	0.0103
24CCDGD	Façade	0.70	0.07	0.56	7800	1376	2.8	0.176	15.87	0.02	0.002	158.7	1450	8.12	0.0059
	Façade	0.60	0.06	0.48	7800	1011	2.4	0.130	18.52	0.02	0.002	185.2	1450	6.96	0.0069
	Façade	0.50	0.05	0.4	7800	702	2	0.090	22.22	0.02	0.002	222.2	1450	5.8	0.0083
	Façade	0.30	0.03	0.24	7800	253	1.2	0.032	37.04	0.02	0.002	370.4	1450	3.48	0.0138

Table App. 2: Fire Proofing of Columns

6. These values were analysed in the following graph.

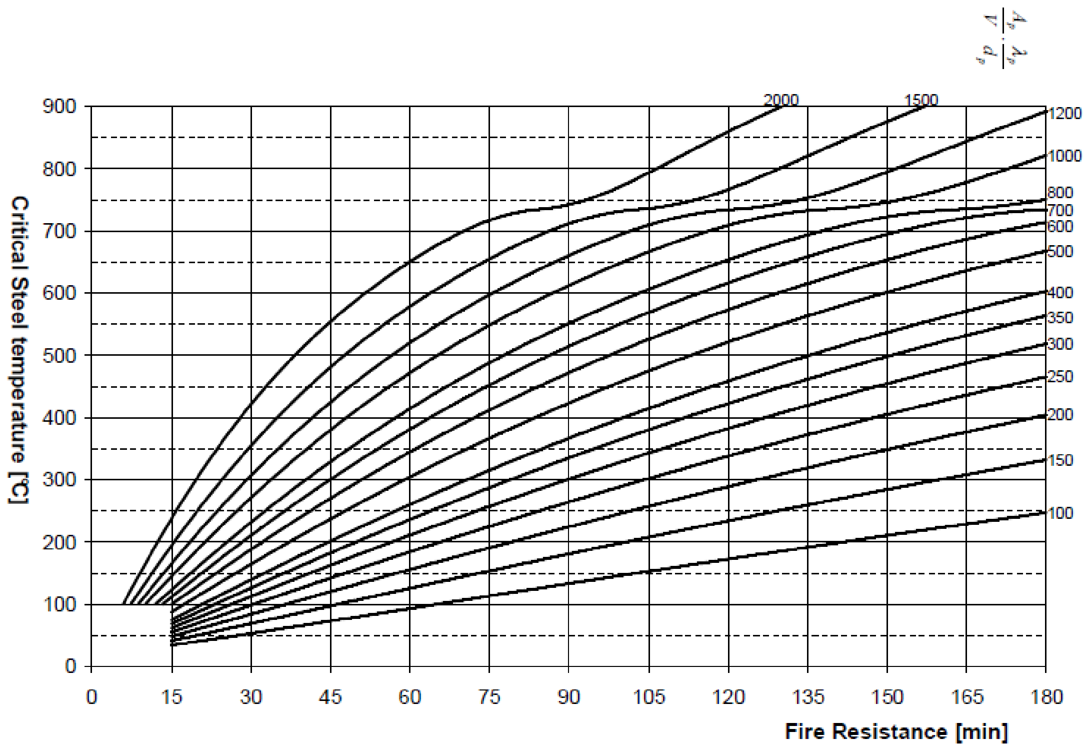


Fig. App. 38: Nomogram for Protected Steel Members [52]

7. The highest value, among the values obtained for all the cross sections, is $\frac{A_p}{V}\frac{\lambda}{d} = \mathbf{370}$, which satisfies the 150 min fire resistance requirement at a temperature of 550º. Additionally, the lower values of $\frac{A_p}{V}\frac{\lambda}{d}$ will result in longer resistance time.

8. This thickness (2mm) was used to compute the amount of coat used [kg of coat] per mass of steel member [kg of steel]. The averaged value (0.73%) corresponds to a representative and simplified percentage.

The previous computation was performed to consider the amount of coat used in the whole building. A further and more thorough analysis of the local behaviour of the structural elements should be performed to verify that the steel elements fulfil the fire resistance solicitations.

This result (0.73%) is higher than the proportion of material used in other studies (0.55%) [24]. However, due to the uncommon size of the structural elements, the value obtained from the analysis will be considered on this thesis.

H. LIFE CYCLE ANALYSIS

H. 1. SECONDARY MATERIALS

To avoid detailed calculations and due to time constraints, the values used for the secondary materials were obtained from projects with similar conditions and requirements. These values are averaged values and may results in different values than the real requirements, however this percentages were used to provide a point of departure in the overall comparison.

Secondary Material	%	% of	Reference
Steel Reinforcement	2.23%	Kg of concrete Core	[25], IMd
Steel Reinforcement	1.19%	Kg of Concrete Foundation Pile	[25], IMd
Prestressing Steel	1.22%	Kg of Concrete Hollow Core Slab	[25], IMd
Intumescent Paint	0.73%	Kg of Steel Structural Element	Appendix G

Table App. 3: Secondary materials as a percentage of the main material

H. 2. LCA CRADLE TO GATE (C2Gt) DATA

As it is presented on Table App. 4, the databases were not complete, thus it was necessary to extrapolate or average the information to fill some values. The values taken and used for this study are listed in the Table App. 4 below. It is remarked that, at the time this thesis was written, these values were taken from the free versions available on the websites.

GWP [kgCO2eq / kg]	Source						
	NMD	NMD+MRPI		CTBUH	BCSA	JRC	
Material	[36]	[36],[49]		[16]	[50]	[13]	
Structural Steel	1.82E+00	9.08E-01	**	*1.17E+00*	2.01E+00	1.69E+00	
Reinforcement Steel (Feb500)	1.49E+00	1.49E+00	*	1.24E+00	1.32E+00	2.13E+00	
Prestressing Steel (PT)	1.79E+00	1.79E+00	*	*1.50E+00*	1.76E+00	*2.13E+00*	
Concrete C50/60	*1.12E-01*	*1.12E-01*	*	*1.70E-01*	1.70E-01	*1.19E-01*	
Concrete C55/67	1.18E-01	1.18E-01	*	1.70E-01	*1.70E-01*	*1.18E-01*	
Fire Proofing (*)	1.89E+00	2.40E+00	**	2.60E-01	2.91E+00	2.40E+00	*

Table App. 4: Global Warming Potential values.

The values in *italic font* were averaged or extrapolated from the available information from each source.
* Values from NMD
** Values from MRPI
(*) Fire Proofing Material was variable according to the different sources:
NMD – does not have available information, so Paint with acrylate dispersion was used.
MRPI – does not have available information, so Spray Painting was used.
CTBUH – Fire-Proof Spray is used.
BCSA – Intumescent Paint is used.

H. 3. LCA CRADLE TO CRADLE (C2Cr) DATA

Structural Element	Material	A1-A3	A4	A5 (6)	C1 (6)	C2	C3	C4	D
Foundations	C55/67 (1)	*0.19000*	0.00140	0.00070	0.01500	0.02840	0.00189	0.00481	-0.00192
Foundations	FeB500	2.13000	0.02840	0.00070	0.05250	0.02840	0.00189	0.00481	-0.58600
Floor Slabs	C50/60 (1)	*0.18028 (8)*	0.02840 (4)	0.01989 (5)	0.00702	0.02840	0.00189	0.00481	-0.00192
Floor Slabs	Prestress Steel (2)	2.13000	0.02840	0.01989 (5)	0.02457	0.02840	0.00189	0.00481	-0.58600
Concrete Core	C50/60 (1)	*0.18000*	0.00140	0.00033	0.00702	0.02840	0.00189	0.00481	-0.00192
Concrete Core	FeB500	2.13000	0.02840	0.00033	0.02457	0.02840	0.00189	0.00481	-0.58600
Structural Steel	S355	1.94350 (7)	0.02840	0.01989	0.02457	0.02840	0.00242	0.00160	-0.40200
Structural Steel	Paint	2.40000 (3)	0.02840	0.01989 (3)	0.02547	0.02840	0.00242	0.00160	0.00000

Table App. 5: C2Cr characterization factors [kgCO2eq/kg]

The values in *italic font* were averaged or extrapolated from the available information from each source.
(1) The values for C25 available were extrapolated to derive the values of C50/60 and C55/67
(2) The values for Reinforcement Rebar were used.
(3) Values from MRPI
(4) The value for A4 of the slabs was increased to account for the fact that hollow core slabs, as an element, are being transported, instead of only concrete.
(5) The values from MRPI for steel construction were used to account for the assembly of hollow core slabs, as an element.
(6) Values considering a multiplying factor to account for the works below ground level (2.5) and for the works at height (1.17) [Appendix F].
(7) A factor (1.15) to account for production of the structural elements is considered [13].
(8) The value is considered as the production value for concrete plus the value of the hollow core slab production. This last one was considered as the same as A5 for concrete (0.00028).

By using the MRPI values for the construction, there is no risk of double counting benefits or loads arising from the recycling at the end of life, thus, it is possible to use these values.

H. 4. LCA CRADLE TO CRADLE WITH 100% REUSE (C2Cr100) DATA

Structural Element	Material	A1-A3	A4	A5	C1	C2	C3	C4	D
Foundations	C55/67	*0.19000*	0.00140	0.00070	0.01500	0.02840	0.00189	0.00481	-0.00192
Foundations	FeB500	2.13000	0.02840	0.00070	0.05250	0.02840	0.00189	0.00481	-0.58600
Floor Slabs	C50/60	*0.18028*	0.02840	0.01989	0.00702	*0.03800*	*0.00000*	*0.00000*	*-0.17839*
Floor Slabs	Prestress Steel	2.13000	0.02840	0.01989	0.02457	*0.03800*	*0.00000*	*0.00000*	*-1.31943*
Concrete Core	C50/60	*0.18000*	0.00140	0.00033	0.00702	0.02840	0.00189	0.00481	-0.00192
Concrete Core	FeB500	2.13000	0.02840	0.00033	0.02457	0.02840	0.00189	0.00481	-0.58600
Structural Steel	S355	1.94350	0.02840	0.01989	0.02457	*0.03800*	*0.00000*	*0.00000*	*-0.69966*
Structural Steel	Paint	2.40000	0.02840	0.01989	0.02547	*0.03800*	*0.00000*	*0.00000*	*2.40000*

Table App. 6: GWP values [kgCO2eq] for the 100% Reuse Scenario

Most of the values were considered with the same values and criterion that for the analysis without 100% reuse. The differences are the values, from the Table App. 6, written on *italic font* which were derived with the following expressions:

MODULE	EXPRESSION	COMMENTS
MODULE A1-A3	$[(1 - R_C)E_V + R_C * E_R]$	The same precedence of the material will be considered Therefore, E_V, E_R *and* R_C remain the same. Thus, the value remains the same
MODULES C1-C4	$[(1 - RR)E_D]$	The reuse rate (RR) will be considered as 100% Additionally, the impacts E_D for the phases C3 and C4 are avoided thus $E_D = 0$ (on those modules) and $E_D = E_D$ on modules C1 and C2
MODULE D	$[(RR - R_C)(E_R^* - E_V^* * C_f)]$	The reuse rate (RR) will be considered as 100% Additionally, the impact of recycling is avoided, but an additional impact due to storage and sorting of the element must be applied, thus $E_R^* \neq 0$. The impacts from E_V^* are considered as the impacts to fabricate a structural element, thus, they are equal to the values of module A1-A3

In the case of hollow core slabs (Concrete and Prestressing steel):

RR 100%, due to the fully reusable scenario

R_C 0% for the concrete and 38% for the prestressing steel; obtained from the data of the research [13] to avoid double counting from module A1-A3.

E_D For modules C1 and C2 $E_D = E_D$ of the structural steel, since the hollow core slabs will be considered as structural elements.
For C1, a factor that accounts for the height of the building and is considered.

For C2, the travel distance is considered as 200km.

For modules C3 and C4 $E_D = 0$

E_R^* There is no recycle process, however, there are impacts generated from the storage and handling of the element, thus, for this study this factor is considered as $E_R^* = C3$ (from the previous assessment C3 = 0.00189)

E_V^* The impacts from E_V^* are considered as the impacts to fabricate a structural element, thus, they are equal to the values of module A1-A3.

C_f Considered as 1, since the element is intended for the same use.

For the steel elements (Structural steel and Paint):

RR 100%, due to the fully reusable scenario

R_C 64%, obtained from the data of the research [13]

E_D For modules C1 and C2 $E_D = E_D$ of the structural steel.

For C1, a factor that accounts for the height of the building and is considered.

For C2, the travel distance is considered as 200km

For modules C3 and C4 $E_D = 0$

E_R^* There is no recycle process, however, there are impacts generated from the storage and handling of the element, thus, for this study $E_R^* = C3$. Additionally, the application of a new fire-proof coat is considered, thus E_R^* is equal to the production values, for the paint.

E_V^* The impacts from E_V^* are considered as the impacts to fabricate a structural element, thus for structural steel, these are equal to the values of module A1-A3.

C_f Considered as 1, since the element is intended for the same use.

For the rest of the materials, the characterization factors are the same as the ones used on the previous assessment.

H. 5. LCA CRADLE TO GATE (C2Gt) RESULTS APPENDIX

The results that arise from using the different databases for the LCA C2Gt are plotted in the following graphs:

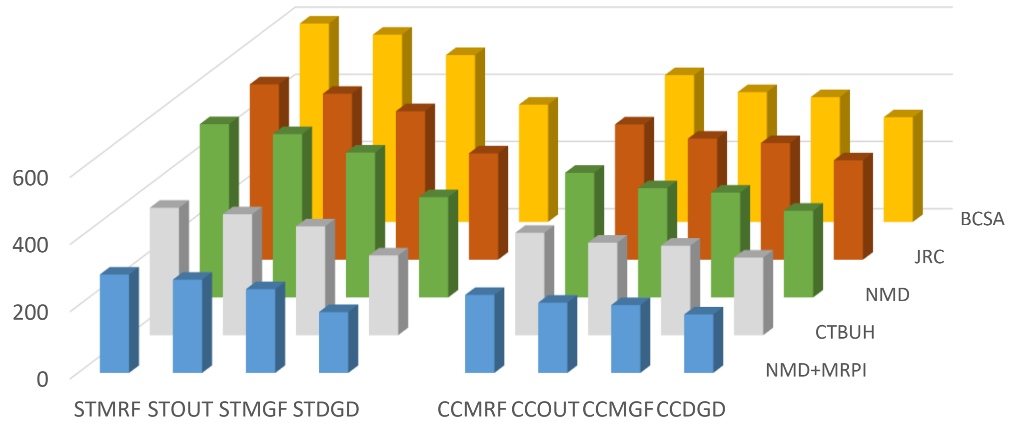
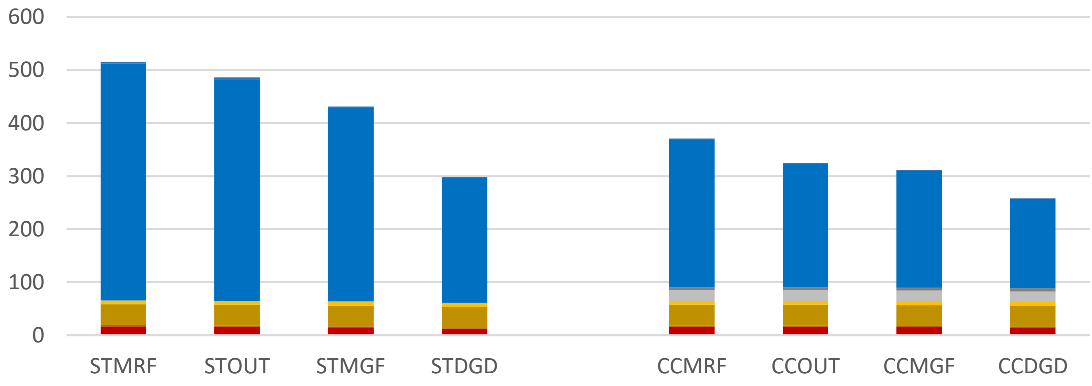


Fig. App. 39: Total GWP by structural system, using different databases.

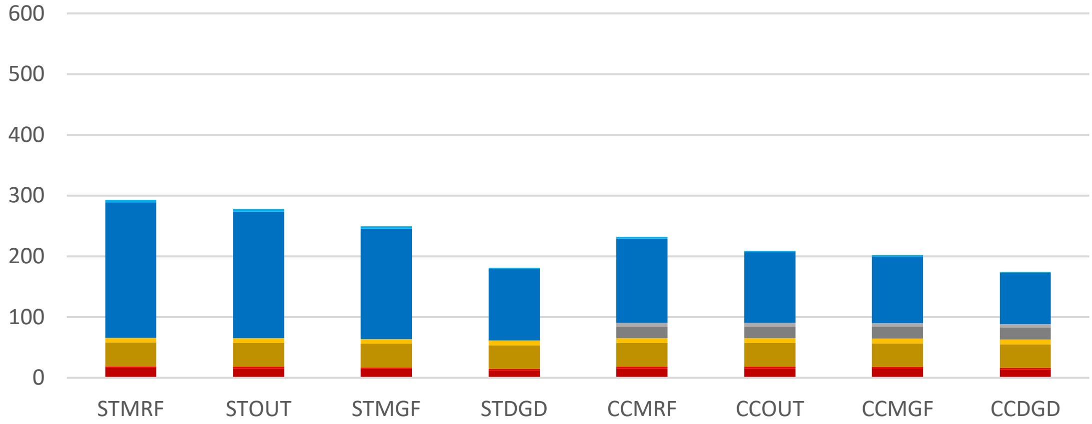
Fig. App. 39 presents the results of each of the systems, when different databases are used.

The resultant values from the evaluation with the database NMD+MRPI results in the lower set of results. The remaining databases are 1.63 (NMD), 1.31 (CTBUH), 1.91 (BCSA) and 1.74 (JRC) times higher. These differences arise mainly from the dominant difference of steel's characterization factors.

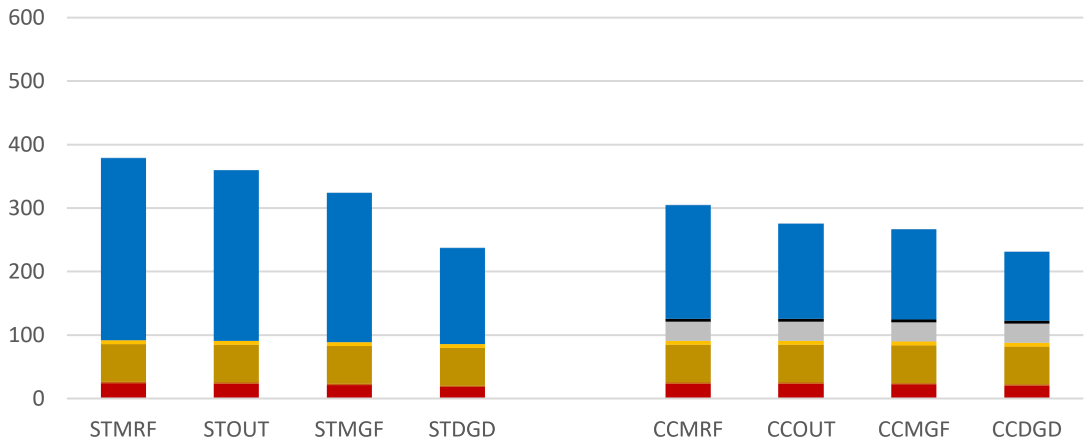
The results of the Global Warming Potential of the different structural systems, according to the different databases used, are plotted on the following graphs. Additionally, each graph bar has a division to represent the contribution of each material. This value represents the impact per square meter of building.



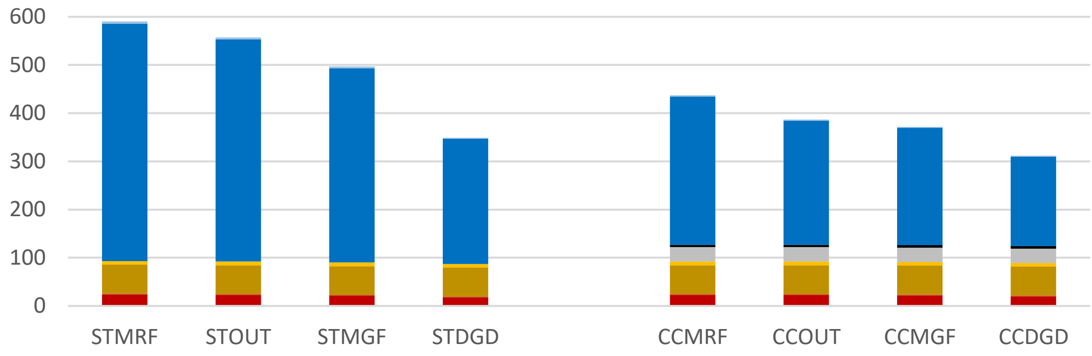
a) GWP (A1-A3) with data from NMD [kg CO2 eq/m2]



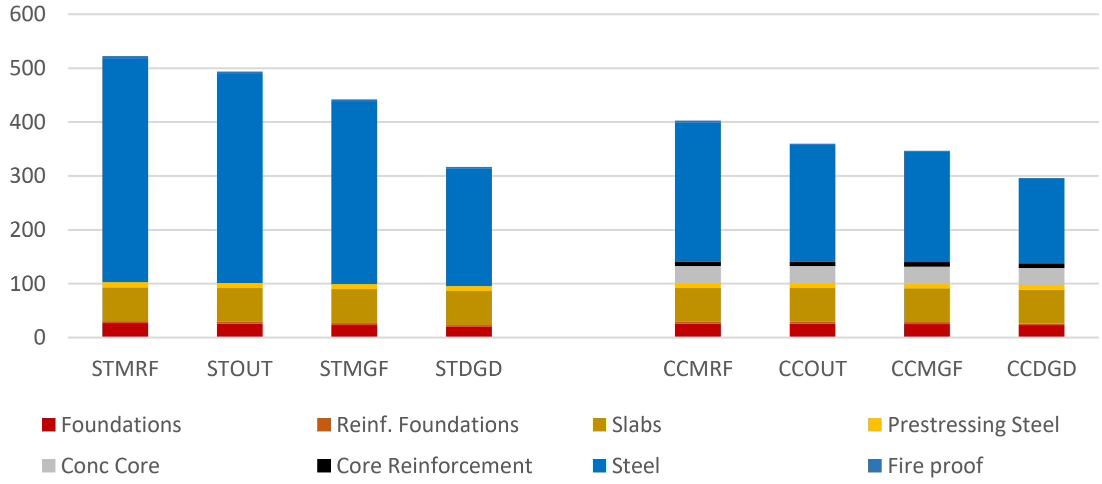
b) GWP (A1-A3) with data from NMD+MRPI [kgCO2eq/m2]



c) GWP (A1-A3) with data from CTBUH [kg CO2 eq/m2]



d) GWP (A1-A3) with data from BCSA [kgCO2eq/m2]



e) GWP (A1-A3) with data from JRC [kgCO2eq/m2]

Fig. App. 40: Global Warming Potential of the eight structural systems studied, with different databases.

From the analysis of the results, the following conclusions are drawn:

Steel is the material that contributes the most to the environmental impact. This is expected since the GWP of steel elements is several times higher than that of concrete, for instance. the GWP of steel S355 is 15 times larger than the GWP of concrete on the NMD database; 7.7 times larger on the MRPI; and 12 times larger on the BCSA database. The same applies to reinforcement and prestressing steel, however, due to the small amounts of these materials used, their contribution is not as big as the one of S355.

The use of a certain database heavily affects the output. The biggest difference on the material's characterization values are the ones that correspond to the fire proofing material. However, the volume required of this material is very small (less than 0.25% of the total), and thus, do not affect significatively the results. Additionally, the difference on the characterization factor is a result of the different materials considered as fire proofing material.

In the case of steel S355, its contribution to the total weight is highly relevant (from 12% to 33% of the weight, depending on the structural system considered), and its GWP factor, among the different databases, is substantially variable (1.82, 0.91, 1.17 and 2.01). The effect of these two aspects results in large differences in the results. For instance, the total GWP of the structures evaluated with the BCSA is on average 90% higher than the correspondent to the structures evaluated with the NMD+MRPI database. The enormous variation of the steel's GWP is the main factor for the large variation of the results of the whole structure.

Finally, for the remaining materials this situation is less significative than in the case of steel elements, thus, it will not be addressed further.

H. 6. SHADOW COSTS OF LCA CRADLE TO GATE (C2Gt)

The GWP indicator will be used to perform the cradle to cradle assessment, since it is the indicator with more studies and available information. However, it must be highlighted that using only one impact indicator may result on incomplete assessments. According to the SBK Bepalingsmethode (version 3.0), there are ten impact categories from the EN15894 that must be weighed and added to a single indicator of environmental impact. The option to address it, according to this method, is by means of the Shadow price[54]. The shadow price (or shadow cost) consists on assigning a monetary value to each of the ten impact categories and adding up the total monetary value from all the categories. The monetary value represents the costs that it would take to prevent the corresponding emission[55]. By using this indicator, several impacts are considered on the analysis. However, it is highlighted that there is less consensus about weighting factors than about characterization factors, and thus, the method may bring uncertainties[54].

The values of the shadow cost(Table App. 7), with information from the NMD and the MRPI, were calculated (Table App. 9) to evaluate the entire environmental impact of the Cradle-to-Gate phase.

NMD+MRPI	Shadow Cost (ten indicators) [€/kg]	Shadow Cost (of only GWP) [€/kg]
Structural Steel	0.0675	0.04540
Reinforcement Steel	0.2471	0.07440
Prestressing Steel	0.6568	0.08960
Concrete C50/60	0.0086	0.00562
Concrete C55/67	0.0090	0.00590
Fire Proofing	0.9798	0.12000

Table App. 7: Summary of the Shadow Cost of the materials used.

A comparison of the shadow cost applied to ten indicators against the shadow cost applied only to GWP was elaborated. Fig. App. 41 shows the results of this analysis.

From this comparison, it is concluded that: By using only GWP the whole picture is not considerably disregarded. There are other impacts, besides the emission of greenhouse gases, that heavily influence the result. The whole environmental impact of a structural system can be from 1.76 (Steel Moment Resistant Frame) to 1.91 (Concrete Diagrid) times higher.

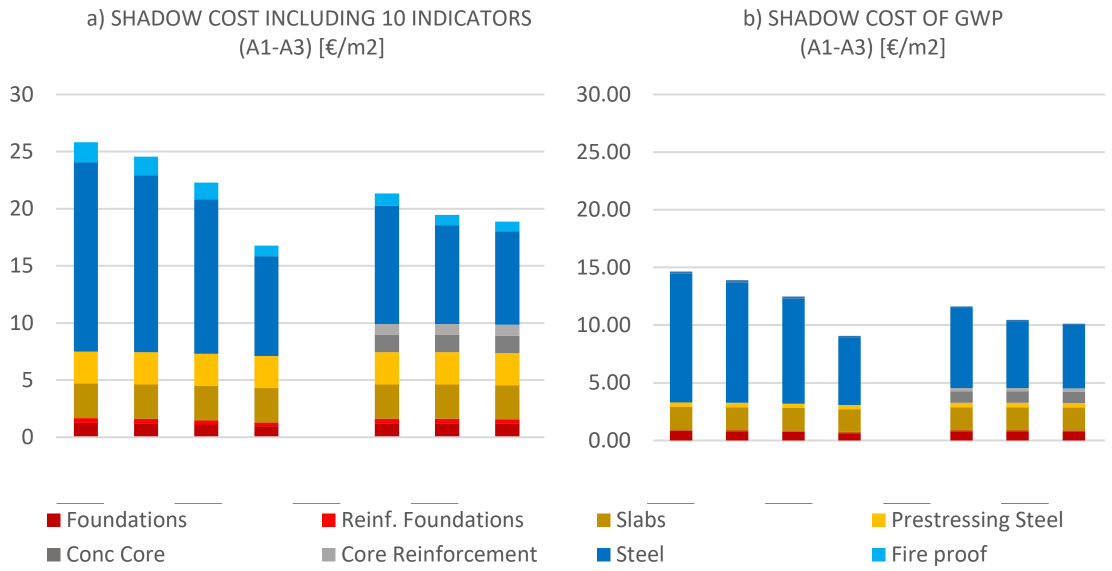


Fig. App. 41: Shadow Cost Comparison.
a) Shadow cost applied to GWP. b) Shadow cost applied to ten indicators.

Additionally, some materials become more relevant to the environmental impact when more impact categories are included. I.e. the impact of the considered prestressing steel, of the hollow core slabs, has almost the same value as the slabs themselves, even with a very small ratio in volume. The same holds for the concrete core and the foundation piles and their corresponding steel reinforcement.

Finally, steel is still the biggest contribution to the impact, however in less proportion. This means that, by only considering GWP, steel may look like a "more pollutant material" than it is. This is summarized on Table App. 8.

For instance, when looking at Concrete Core with Steel Diagrid, if only the Global warming potential is considered, it looks like the steel structure is responsible of almost half of the impact, when its contribution is only 42%.

STRUCTURAL SYSTEM	Considering € of 10 indicators	Considering € of GWP	Impact difference ($\%_{€GWP}$ / $\%_{€10indicators}$)
Steel Core Moment Resistant Frame	71%	77%	1.09
Steel Core Outrigger	70%	76%	1.10
Steel Core Megaframe	67%	74%	1.11
Steel Core Diagrid	58%	66%	1.15
Concrete Core Moment Resistant Frame	54%	61%	1.14
Concrete Core Steel Outrigger	49%	57%	1.15
Concrete Core Megaframe	48%	55%	1.16
Concrete Core Diagrid	42%	49%	1.18

Table App. 8: Contribution of Steel and Fire protection to the total impact.

	Impact Category	Global warming (GWP100)	Abiotic depletion	Ozone layer depletion (ODP)	Human toxicity	Fresh water aquatic ecotox.	Marine aquatic ecotoxicity	Terrestrial ecotoxicity	Photochemical oxidation	Acidification	Eutrophication
	Unit	kg CO2 eq	kg Sb eq	kg CFC-11 eq	kg 1,4-DB eq	kg 1,4-DB eq	kg 1,4-DB eq	kg 1,4-DB eq	kg C2H4	kg SO2 eq	kg PO4--- eq
Shadow price (Euro) per kg equivalents	€/kg	0.05	0.16	30	0.09	0.03	0.0001	0.06	2	4	9

Source	Material	Unit	Global warming (GWP100) kg CO2 eq	Abiotic depletion kg Sb eq	Ozone layer depletion (ODP) kg CFC-11 eq	Human toxicity kg 1,4-DB eq	Fresh water aquatic ecotox. kg 1,4-DB eq	Marine aquatic ecotoxicity kg 1,4-DB eq	Terrestrial ecotoxicity kg 1,4-DB eq	Photochemical oxidation kg C2H4	Acidification kg SO2 eq	Eutrophication kg PO4--- eq
MRPI	Steel production (A1-A3)	ton	9.08E+02	5.21E+00	1.55E-05	3.33E+01	3.02E+00	6.34E+03	4.68E-01	3.30E-01	3.38E+00	3.74E-01
NMD	Steel reinforcement net FeB 500 HKN	kg	1.49E+00	1.27E-02	5.65E-08	6.59E-01	6.33E-01	5.90E+02	2.75E-02	8.47E-04	5.16E-03	1.05E-03
NMD	Prestressing steel (average)	kg	1.79E+00	1.54E-02	7.17E-08	3.81E+00	1.49E+00	1.32E+03	3.18E-02	9.27E-04	7.38E-03	1.34E-03
NMD	Concrete Mortar C50/60 (CEM I-CEM III) averaged	kg	1.12E-01	3.18E-04	5.04E-09	1.16E-02	2.32E-03	3.88E+00	2.09E-04	8.55E-06	2.52E-04	4.41E-05
NMD	Concrete mortar C55/67 (CEM I-CEM III)	kg	1.18E-01	3.31E-04	5.19E-09	1.19E-02	2.37E-03	3.96E+00	2.17E-04	8.86E-06	2.60E-04	4.53E-05
MRPI	Spray Painting (A1-A3)	kg	2.40E+00	3.00E-02	1.40E-07	5.70E+00	8.30E-01	4.50E+01	3.60E-02	1.20E-01	1.40E-02	1.60E-03

Source	Material	TOTAL [€ / kg]	Global warming (GWP100)	Abiotic depletion	Ozone layer depletion (ODP)	Human toxicity	Fresh water aquatic ecotox.	Marine aquatic ecotoxicity	Terrestrial ecotoxicity	Photochemical oxidation	Acidification	Eutrophication
MRPI	Steel production (A1-A3)	0.0675	4.54E-02	8.34E-04	4.65E-07	3.00E-03	9.06E-05	6.34E-04	2.81E-05	6.60E-04	1.35E-02	3.37E-03
NMD	Steel reinforcement net FeB 500 HKN	0.2471	7.44E-02	2.04E-03	1.70E-06	5.93E-02	1.90E-02	5.90E-02	1.65E-03	1.69E-03	2.06E-02	9.47E-03
NMD	Prestressing steel (average)	0.6568	8.96E-02	2.46E-03	2.15E-06	3.43E-01	4.46E-02	1.32E-01	1.91E-03	1.85E-03	2.95E-02	1.21E-02
NMD	Concrete Mortar C50/60 (CEM I-CEM III) averaged	0.0086	5.62E-03	5.08E-05	1.51E-07	1.04E-03	6.97E-05	3.88E-04	1.25E-05	1.71E-05	1.01E-03	3.97E-04
NMD	Concrete mortar C55/67 (CEM I-CEM III)	0.0090	5.90E-03	5.29E-05	1.56E-07	1.07E-03	7.10E-05	3.96E-04	1.30E-05	1.77E-05	1.04E-03	4.08E-04
MRPI	Spray Painting (A1-A3)	0.9798	1.20E-01	4.80E-03	4.20E-06	5.13E-01	2.49E-02	4.50E-03	2.16E-03	2.40E-01	5.60E-02	1.44E-02

Table App. 9: Derivation of Shadow Cost per material.