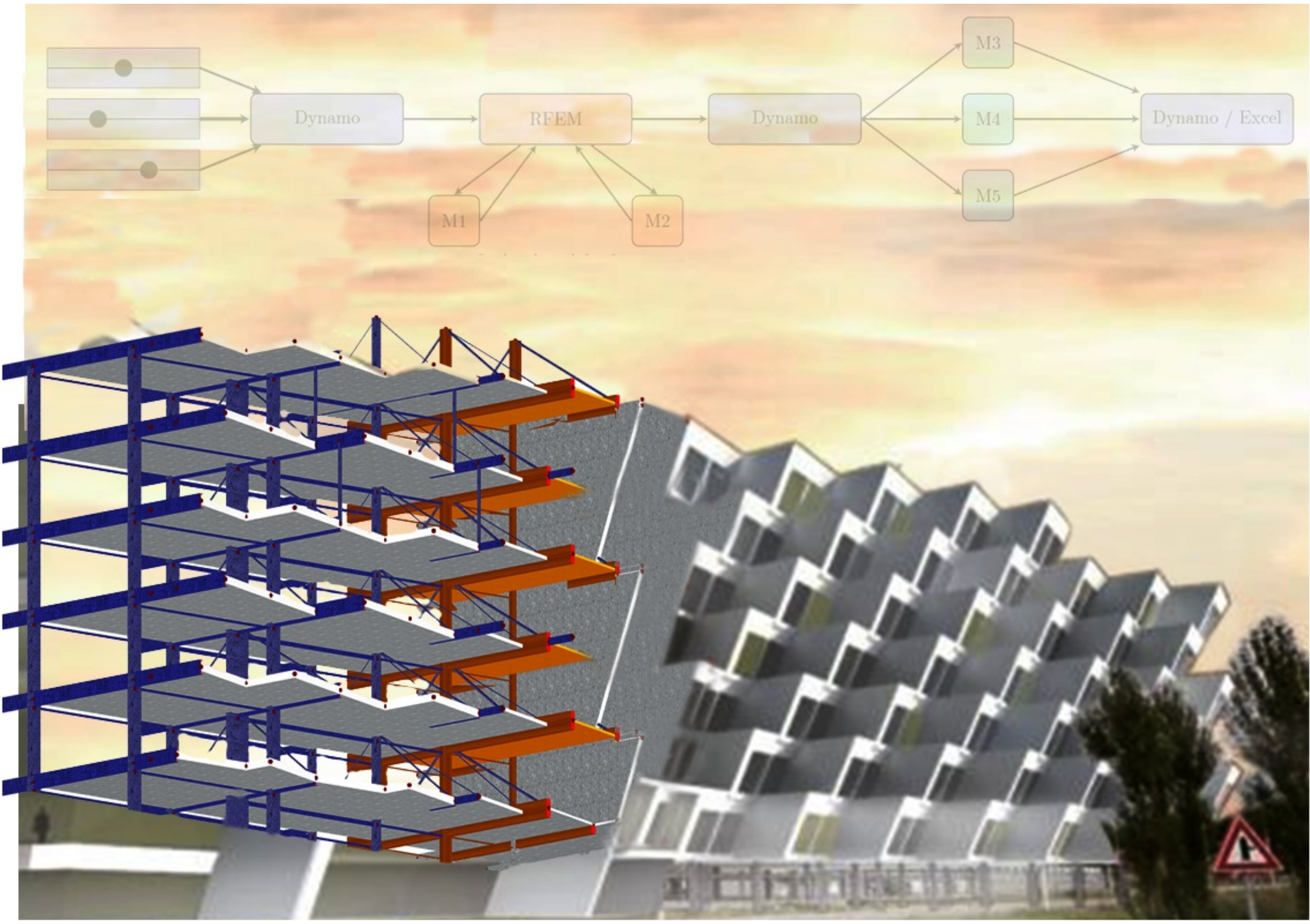
MSc Thesis report

Structural parametric design for variant studies in the preliminary design phase

by Igor Pečanac



TU Delft

ARCADIS Design & Consultancy for natural and built assets

Master Thesis

Structural parametric design for variant studies in the preliminary design phase

By

I. Pečanac

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at the Delft University of Technology,
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Preface

This report is the result of my master thesis project and is the final part of my Master curriculum of Structural Engineering at Delft University of Technology. During my bachelor of Civil Engineering, I had been taught that it is possible to automate certain problems and have repeatedly been able to do this for the remainder of my studies for laborious or time consuming work.

I always wondered if software with this functionality already existed. During my study for my Masters in Structural Engineering I was introduced to the workings of the finite element method and other frame work programs. At the time I thought that this was the answer to my questions. I quickly realized however, that this was not the case due to the many practical problems when using these programs.

Nonetheless, I always believed that it should be possible to modernize the construction industry. Automating the parts of the work that are laborious or time consuming should also be possible in practice as it was also possible for me during my studies.

Therefore, I am thankful for the opportunity that was given to me to work on this problem for my graduation project. I had the chance to learn what problems are faced when trying to automate the construction industry and I was able to have an active part in trying to solve these problems.

Igor Pečanac,

Rotterdam, June 2019

Acknowledgements

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I would like to thank ir. Lennert van der Linden for helping me with my research questions. His support helped me to find a focus for this thesis. His willingness to read trough my thesis and giving helpful advice were invaluable for the accomplishments in this thesis and is very much appreciated.

Special thanks go to ir. Michael van Telgen for introducing me to the concept of structural parametric design and giving me the opportunity to do a research about this topic at Arcadis. He taught me much about parametric design, practical structural engineering and coding. I would also like to thank him for his continued support throughout this thesis. It is his encouragements that motivated me to complete this thesis.

I want to thank my colleagues at Arcadis Rotterdam for welcoming me and their willingness to help me when needed. In particular I would like to thank André Martens for sharing his knowledge about the calculation of costs in the preliminary design phase and Charif Mounji for the provided information of the Bluebell Hotel.

Finally, I would like to thank my friends and family for their continued support during my thesis. Their believe and confidence in me helped me trough the final periods of the thesis.

Abstract

An organized process is necessary to ensure that the building is built on time and within budget. This does not always go well however and the following four major difficulties can arise: exceedance of the budget, design changes at the end of the design phase, limited available information and the increasing complexity of structures.
One of the current solutions is performing variant studies. Designing variants is costly however, which limits the amount of analysed variants. The focus is set on improving this process, which will indirectly affect the four named issues.

The advantages of structural parametric design (SPD) have the potential to increase the amount and complexity of variants. However, the disadvantage of large structures is the uncountable amount of possible calculations and parameters.
Current SPD-tools have the same functionality in general, but differ in key ways which influences the choice for a tool. These mostly focus on some form of optimisation. This research however, focuses on generating options. The connection between a visual programming software (Dynamo) and FEA software (RFEM) named the DRC is chosen to achieve this.

The demands of a building are found in the SoR and it is the task of the architect to implement it as well as possible. The structural engineer can help the architect achieve this by providing information about the support structure. This lead to the following research question:

What are the possibilities of informing the architect of the support structure using parametric design with RFEM and Dynamo in the preliminary design phase?

Attempting every possibility for the support structure is difficult. A method is therefore proposed called the "translation by component method" (TC-method), which gives the ability to more easily modify parametric models. This made it possible to divide the problem over several smaller scripts, called *variants*.
Each variant is able to produce models, called options, which provide information about the critical load combination, the type of failure and the unity checks for most eurocode demands. The eurocode checks and optimisation of cross sections were performed by modules in RFEM by using internal forces from the FEA. It has been verified that the modules apply the correct internal forces, use the correct eurocode formulae and choose the correct values of the Dutch national annex.

The structural engineer can learn from each variant and apply this knowledge on subsequent variants. The engineer's creativity is thus used to limit the amount of parameters and calculations. This process directs the engineer towards a solution.

Options provide information for the architect as well, namely: the geometry of the support structure, the optimized cross sections and the costs of the option. The architect can use this information to weigh options against each other to fulfil the demands of the SoR as well as possible. This process is named optioneering.

To test this method, a variant study is performed on a case study: The Bluebell Hotel. Only a part of this structure is analysed called the outer facade. The complexity of the geometry and the mechanical solution not being obvious makes this design a good candidate to test the DRC.

After the first variant was created, nine variants and 35 options were generated, analysed and reported within two weeks. Optimized options were obtained within 15 to 30 minutes. By learning from previous variants, it was possible to find a support structure which barely modifies the aesthetics of the structure. The options obtained during this process mostly have unique advantages and disadvantages which can be compared by the architect. A recommendation and summary is given for these results.

The applied method for the DRC enhanced the process of the variant study by increasing the amount and complexity of variants. More information can be obtained and by learning from previous variants more informed decisions can be made for subsequent variants by the structural engineer.
The architect has more freedom to choose the placement of structural elements and information of costs is earlier available. This can be used to better implement the demands of the SoR. The architect can take costs earlier into account as well which can be used to better estimate the costs-appreciation ratio of a certain choice.

It should be considered for each project whether the amount of information is worth the initial time investment. The case study can be used as an indication and it required approximately four weeks to construct the first variant and perform the variant study.
The size of the model will directly influence the time investment. It is therefore regarded as a limiting factor of the DRC as well. For the outer facade it was found that increasing the amount of members by a certain factor will approximately increase the computation time by the same factor.

Contents

1 Introduction

1.1 Problem definition

Contemporary structural engineering is a part of the larger general building process, which consists of the initiation stage, design stage, specifications stage and construction stage. This is a collaboration of many different professions and an organized process is necessary to ensure that the building is built on time and within budget (van Eekelen, Rip, & Wentzel, 2002).
Three design phases exist within the design stage: the conceptual design phase, preliminary design phase and detailed design phase. Within these phases a procedure is followed to ensure that the structure stays within budget throughout the design process.

This does not always go well however and four major difficulties can arise in the design stage. The first issue is that large scale or appealing projects commonly exceed the budget, which has caused promising structures to be delayed or prevented from being built at all. In many cases this is caused by insufficient management of costs in the design phase. The consequence is that changes will be necessary for the design (van Eekelen et al., 2002, p. 122).
The author continues to explain that this leads to a second issue. Design changes at the end of the design phase lead to time loss, extra costs and potential frustration. The quality and functionality of the final product may be reduced as well.
This problem is a consequence of the fact that decisions are made on limited information, which leads to the third issue: information is limited at the start of the

design stage. Most buildings are unique products and information must be gained for each one. This is limited at the start of the building process, but slowly grows until construction stage is reached. However, almost 80% of the final result is planned during the design stage while only approximately 50% of the information is available (van Eekelen et al., 2002, p. 166). The consequences of these decisions are therefore less clear.
The author continues with the final issue: many factors can influence the structural behaviour of a building. Designs are becoming more complex and are therefore difficult to predict.
Currently, variant studies are performed to increase the amount of available information. Designing variants is costly however, which greatly limits the amount of analysed variants (J. W. Kamerling & Kamerling, 2011, p. 8). The issue is further increased by the fact that a preferred variant usually does not exist. Most variants will have advantages and disadvantages which will have to be weighed against each other.

The advantages of parametric design have the potential to increase the effectiveness of the variant study by making more variants possible in a lower amount of time (J. Coenders, 2009) and by making more complex variants possible (van der Linden, 2018).
There are however disadvantages to structural parametric design. The uniqueness and complexity of current buildings make countless amounts of variations possible. It is difficult to build a parametric model which is able to include the numerous amounts of parameters in a single parametric model (Linssen, 2018).

1.2 Purpose of this research

The purpose of this research is to asses if the Dynamo-RFEM connection (DRC) can be used as a tool to increase the knowledge of the mechanical behaviour of the support structure for the architect. This is done because it is the role of the structural engineer to help the architect with the spatial planning in the preliminary design phase.

With this information the architect will have more design options, which could improve the efficiency and/or the quality of the building design. This is advantageous in this design phase, because there are fewer consequences to changing the structure compared to the next design phases.

The outcome of the thesis should show if parametric design with the DRC is able to improve the current practice of the structural engineer within the preliminary design phase.

1.3 Main research question

What are the possibilities of informing the architect of the support structure using parametric design with RFEM and Dynamo in the preliminary design phase?

1.4 Method

For each chapter, a goal will be stated and sub questions will be given in order to answer the main question. The intention is to make the thesis structure more clear.

Chapter 2: the literature study
The purpose of this chapter is to research: the current practice of the structural engineer without parametric design, general aspects of parametric design and currently available tools and how the DRC fits within parametric design. The following sub questions will be answered:

1. What is the role of the structural engineer in the preliminary phase?
2. How can parametric design be used within structural engineering and what are its advantages and disadvantages?
3. What tools currently exist for parametric design in structural engineering and what are their advantages and disadvantages?
4. How does the Dynamo-RFEM connection function? How does it differ compared to other tools?

Chapter 3: the parametric model
In the previous chapter the theoretical functioning of the DRC is explained. There is no guarantee that the in the previously mentioned chapter's advantages and disadvantages hold true. Therefore the Dynamo-RFEM connection will be tested on a case study: The façade of the Bluebell Hotel.
This chapter will be used to explain how the parametric model is set up with the Dynamo-RFEM connection. This will be done by answering the following questions:

1. What design aspects of the Bluebell Hotel should be taken into consideration for the variant study?
2. What variables are chosen to be parametrised and what results are needed of the parametric model?
3. Is it possible to implement the theoretical positive aspects of parametric design into the variant study? How can this be done while also reducing the impact of negative aspects of parametric design?

Chapter 4: the variant study of the support structure of the Bluebell Hotel's façade

The parametric model is completed, which makes it possible to start the variant study. The goal of the variant study is to find a support structure which requires the least amount of modifications to the design of the architect.
The variant study is done to analyse the behaviour of the tool and whether the positive aspects of parametric design were implemented.

1. Explain the results of the variant study. What parameters proved to have the most influence on the support structure?

2. What are the recommendations for the support structure of the Bluebell hotel based on this knowledge?

Chapter 5: Comparison

The Dynamo-RFEM connection has been tested and a practical example has been given. This connection can now be compared with the current design process without parametric design. Because the testing was restricted, limitations of this study will be considered. Other potentially interesting parameters will be mentioned.

1. Did the positive aspects of parametric design come forward in the variant study? Were there unexpected problems and how were they solved?

2. How does the DRC perform compared to the current design process? Does the theory of chapter 1 hold true in this case study?

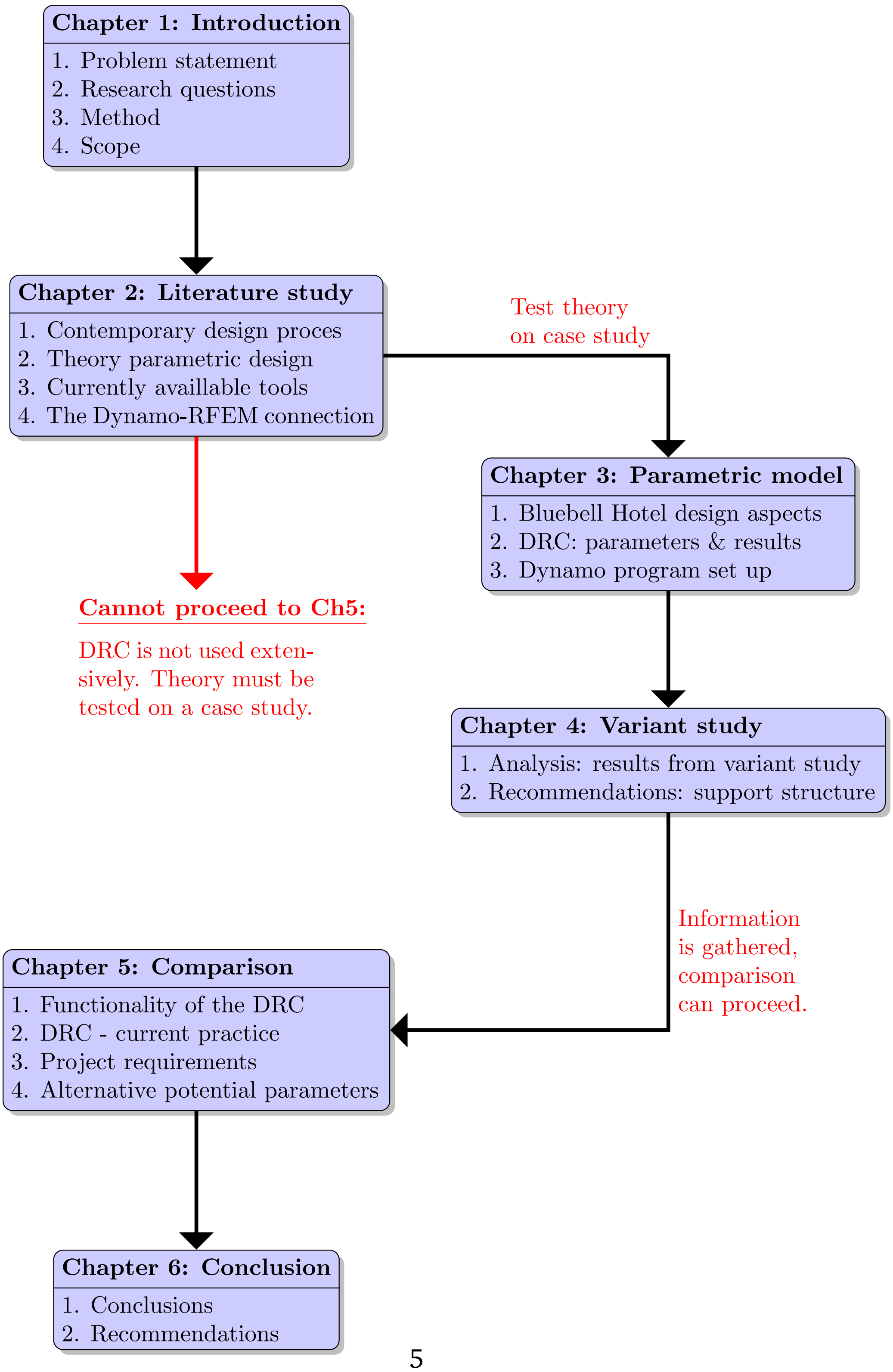
3. What are the requirements to be able to use parametric design as was done in this thesis? In what way could other projects differ?

4. What other aspects of parametric design could have potentially be analysed which were outside the scope of this thesis?

Chapter 6: Conclusion and recommendations

The final chapter will discuss the results of this thesis. Conclusions will be given about the main research question as well as the functionality of the DRC. The recommendations for the DRC shall be given as well.

Thesis structure



1.5 Scope and limitations

This research will be focused on the preliminary design phase. If certain aspects in this phase also affect the next phases, then it will be mentioned but not further researched. The research question makes less sense in the following design phases, since the role of the structural engineer will change as well.

The focus will stay on the functionality of the DRC, but its performance will be compared with other currently used tools with a connection between Visual programming software and finite element analysis software. It is not possible to test every combination, therefore this comparison will be limited to the outcome of the literature study. This is discussed in chapter 2.5.

Automatic optimisation techniques are possible, but will not be used within this thesis. The DRC will therefore not optimize *the design*, because this is not the goal in this design phase. The value comes from the knowing the consequences of design decisions on the supporting structure. However, this is different from optimising cross section, which will be needed to accurately inform the architect. This process is not automatic though.

The study will be limited to one structure, where one type of parametric modelling is applied and limited parameters and results are analysed. Nonetheless it should indicate the validity of the theory and will show potential unexpected problems. The “best” approach is not sought after within this thesis.

1.6 Terminology

1.6.1 Expected prior knowledge

It is advised to understand how the used programs work before continuing. Knowledge about finite element software and visual programming software is therefore recommended.
Terminology will be used which should be understandable if the reader has some experience with visual programming software, especially Dynamo. If this is not the case, then it is advised to read appendix A.1. Here, a short explanation is given of visual programming in Dynamo.

1.6.2 Terminology

A list of terms and abbreviations that are used within this thesis that need further clarification.

- **DRC:** Dynamo-RFEM connection, which is the tool used to do parametric designing in this thesis.
- **Dynamo, or Autodesk Dynamo Studio or Autodesk Dynamo sandbox**: A visual programming tool by Autodesk.
- **Dlubal RFEM:** A finite element analysis program.
- **API:** Application programming interface, an interface used for communication between Dynamo and RFEM.
- **C#:** Programming language.
- **FEA:** Finite element analysis.
- **FEM:** Finite element method.
- **Dynamo script:** the program made in Dynamo by connecting nodes in the workspace.
- **SoR:** Schedule of requirements.
- **TR-method:** Translation by room method: a proposed method to construct the model in Dynamo.
- **TC-method:** Translation by component method: the applied method to construct the model in Dynamo.

2

Present day structural engineering and parametric design

To effectively answer the main research question, an understanding will be needed of how structural engineers currently operate in practice. This requires an understanding of the building process and the actors within it. Finally complications in the building process and how the structural engineer currently deals with them will be researched.

Structural parametric design (SPD) could reduce the severity of the complications. Therefore it will be explained what SPD is, what its advantages and disadvantages are and what software currently is available. This information will be applied to the DRC to find if it's unique as a SPD tool and to find out how it could be applied to answer the main research question.

Each section in this chapter will be used to answer one of the following sub questions.

1. What is the role of the structural engineer in the preliminary phase? (2.1)
2. How can parametric design be used within structural engineering and what are its theoretical advantages and disadvantages? (2.2)
3. What tools currently exist for parametric design in structural engineering and what are their advantages and disadvantages? (2.3)
4. How does the Dynamo-RFEM connection function? How does it differ compared to other tools? (2.4)

2.1 Contemporary structural engineering

First a clear understanding is needed of the contemporary building process, because each phase may affect the other. Therefore the general building process will briefly be explained first. Next, the design process will be focused on and finally the role of the structural engineer in the preliminary phase will be explained. The focus will stay on the preliminary phase for the remainder of this thesis.
The building process is a collaboration of different specialisations. For this reason other professions have to be included before the specific role of the structural engineer can be made clear. This section also explains how the support structure influences the choices of the architect and vice versa, which will be simulated in the case study. Therefore the roles of both the architect and structural engineer will be focused on the most.

2.1.1 General building process

The design process will not always be the same in practice, therefore a basic model will be used as seen in figure 2.1 (van Eekelen et al., 2002, p. 7). According to them, the principles of this basic model should be applicable for any specific project and are also applicable for both new developments and altering/expanding existing structures.
Therefore, the Jellema series will be used throughout this section to model the building process. It is also possible that some phases are not needed for some structures. The full model will be used in this thesis to keep the process consistent.

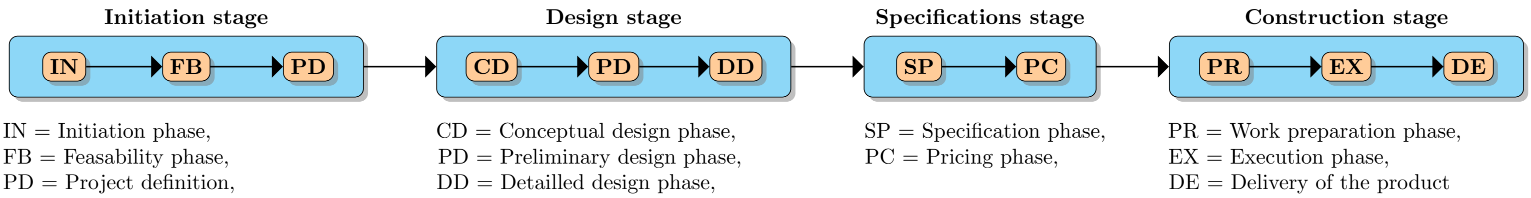


Figure 2.1: Building project phases from initiation to delivery. Amended from: (van Eekelen, Rip, & Wentzel, 2002, p. 14)

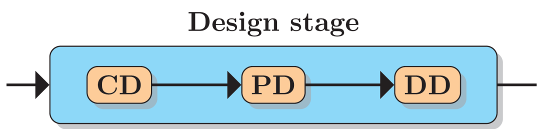
The building process briefly explained
The building process of figure 2.1 is separated in four stages. In the first stage, initiation, the current problems of the client are defined and feasible solutions are investigated. If the solution demands new development then a project definition will be made where the demands, budget and starting points are established (van Eekelen et al., 2002, pp. 21-29)
The authors then explain that the next stage, design, starts with choosing an architect and setting up the Schedule of requirements (SoR) with help of the architect. In the design stage it is the architect's task to develop a structure that incorporates

the demands of the client. The SoR will be refined depending on the needs of each design phase. The stage ends with the approval of the detailed design.
van Eekelen et al. (2002) finalize the building process with the specification stage, where the drawings and specifications are created and the pricing is calculated. Next the construction stage begins which ends with the delivery of the product.

Figure 2.1 seems to imply that all steps must be taken within a phase before initiating the next phase. This is not necessary, because some steps in a certain phase do not depend on the work of the previous phase. Sometimes it could even prove beneficial to start the next phase before finishing the previous phase (van Eekelen et al., 2002).

The structural engineer in the building process
The structural engineer will only be active for a part of the building process. Horix (2017) explains that in the initiation stage the functional requirements for the user needs are secured. At this stage structural requirements are not directly referred to.
According to the author structural engineering activities start at the design stage. A structure is designed and its performance/cost ratio is optimised. Cost optimisation and feasibility is secured by the end of this stage.
The author finally explains that specifications for construction are prepared in the specification stage. If design activities are postponed in the design phase, then structural site engineers will do it in the construction stage. This includes corrections when oversights in the design are discovered.
Beyond these stages structural engineering can be necessary for the operation/-maintenance stage, where the structure can be reconditioned or rebuild. For the final stage, demolition, structural action analysis is needed for a controlled demolition of the structure (Horikx, 2017, p. 115).

2.1.2 The design stage



CD = Conceptual design phase, PD = Preliminary design phase, DD = Detailled design phase

Figure 2.2: The design stage

The design stage briefly explained
The building process is lengthy. Therefore each phase will require basic steps to make the process manageable, which is visualised in figure 2.3. This procedure will be followed for each phase.
The design stage is separated in three phases and begins with setting up the SoR.

The SoR is necessary before the architect can start the conceptual design phase (*scheduling*). The architect then starts outlining the design based on the SoR and the construction site (*designing*). The output of this phase is a floor areas plan for which costs will be calculated (*budget estimation*). The design and its price are compared with the available budget and the SoR. If it is deemed necessary that the design needs adjustments, then (some of) the steps of this phase will have to be redone. This phase ends with the decision to develop the conceptual design into a preliminary design (*decision*) (van Eekelen et al., 2002, pp. 26-28).

They then explain that to advance the design in the preliminary design phase, multiple advisors will be necessary. Examples of advisors would be the structural engineer or advisors for installations. This phase starts with elaborating on the SoR by adding requirements relevant to this phase (*scheduling*). The design is further developed by making choices for the support structure and the space layout of the building(*design*). The price is calculated (*budget estimation*) and the phase ends with comparing the design and its price to the SoR and the budget. If adjustments are necessary, then all the steps of this phase have to be repeated. This phase ends when the decision is made to develop the preliminary design into a detailed design(*decision*).

van Eekelen et al. (2002) finalize the design stage with the detailed design. The purpose of this phase is to almost fully work out the design of the building. It starts by further elaborating the SoR relevant for this phase (*programming*). Choices are made for the support structure and the spatial layout (*designing*). The price calculation in this phase is done in an elaborated manner, since most information about the building is now accurate and available (*budget estimation*). This phase ends with comparing the structure and costs to the budget and SoE. if adjustments are deemed necessary then all steps of this phase must be repeated. The phase ends by accepting the detailed design and preparing for the specifications where the drawings, specifications and a time schedule are made (*decision*).

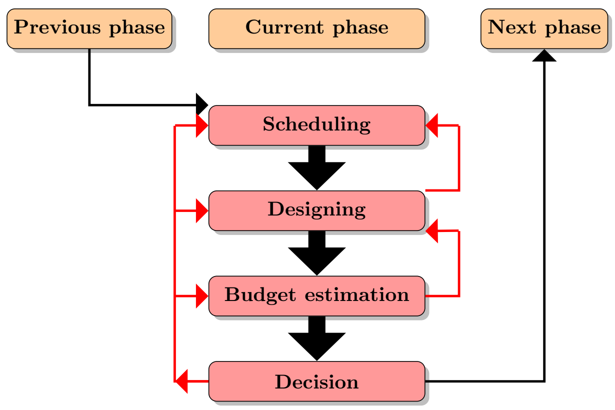


Figure 2.3: The design procedure. Amended from: (van Eekelen et al., 2002, p. 21)

The support structure design

According to van Eekelen et al. (2002, pp. 105-106, 89) the choice of support structure design can be made at the beginning, during or at the end of the architect's design process. This is a creative process for designing the building and is

different for every architect. The term "design process" should not be confused with the term "design stage". For the architect the most important aspects to analyse are the spans of the elements, the material and the height of the structure.
The authors explain that the support structure is usually chosen at the start of the design process for buildings with large spans. The reasoning is that the design of the structure will greatly depend on the support structure. In some cases the material of the building is deemed important to the architect. This will restrict the support structure and the design options.
The authors continue explaining that to design an efficient building, it is beneficial to repeat the same elements as much as possible. This is done during the design process by keeping the grid line size, and thus the spans, constant. The choice for grid line greatly depends on the rooms in the building.
The support structure can also be designed when the architect's design process is finished. The structural engineer will then have to react to the current design of the architect (van Eekelen et al., 2002).
The authors finalize by mentioning that the architect will make assumptions on the dimensions of the structure, which is based on experience. If this experience is lacking, then a structural engineer is asked for consultation. The structural engineer will in general produce basic calculations to inspect if the support structure fits in the design and checking its stability and stiffness requirements. The detailed calculations will not be done before the detailed design phase, where the final calculations are done and the geometry of the building is secured.

Complications in the design stage
1. Costs. The design of large scale or appealing projects commonly exceed the budget, which has caused promising structures to be greatly delayed or prevented from being build at all. In many cases this has been caused by insufficient management of the costs during the design stage (van Eekelen et al., 2002, p. 122).
A specific problem for design competitions, but also other projects, is that the design stage is usually led by the architect. The design will be based on his decisions with no significant considerations for the technical aspects. The consequence of this procedure is that changes will be necessary, which could change its appearance significantly. These issues will cost time and resources depending on the severity of the issues. This is especially problematic when these costs are imposed on the public for public works (Yeager, 2016).

2. Design changes are expensive at the end of the design phase. The authors continue explaining that if the costs are not properly managed during all phases, the budget could be exceeded in the specification phase. Cost savings must then be found which requires changes to the design. Changing the design at this stage of detail may lead to time loss, extra costs and potential frustration. The changes may also lead to a decrease in quality and functionality of the building.
This problem is a consequence of the fact that decisions are made on limited information (see problem 3). Clients are usually the ones who demand changes, but they will be the ones who pay for the additional costs. This is not a shortcoming

of the client or the designers, but a consequence for trying to produce an unique product (van Eekelen et al., 2002, p. 166).

3. Limited information at the start of the design stage. Figure 2.4 shows that the available information about the design is very limited at the start of the building process. This information gradually grows linearly when the design is worked out during the design stage. However, almost 80% of the final result is planned during the design stage while only approximately 50% of the information is available. (van Eekelen et al., 2002, p. 166). Decisions during this stage are made with limited information, which makes the consequences of decisions less clear.

4. Many factors influence structural behaviour. Complex designs make the structural behaviour difficult to predict. Variants are currently made to analyse how certain choices influence the structural design and if it is compatible with the demands from other specialisations. Designing variants is costly, which greatly limits the amount of variants made (J. W. Kamerling & Kamerling, 2011, p. 8). This again limits the amount of information that can be gained from the project.
A balance has to be found between the amount of variants, the detail of the variants and the information that is gained from the variants. After some point designing a variant will be too costly when considering the amount of information that will be gained from it. The problem is further increased by the fact that there usually is no "best variant". Each variant has its own advantages and disadvantages and a choice has to be made by weighing them against each other. Adding additional goals, such as sustainability, will only add more factors that have to be taken into consideration while designing the structure.

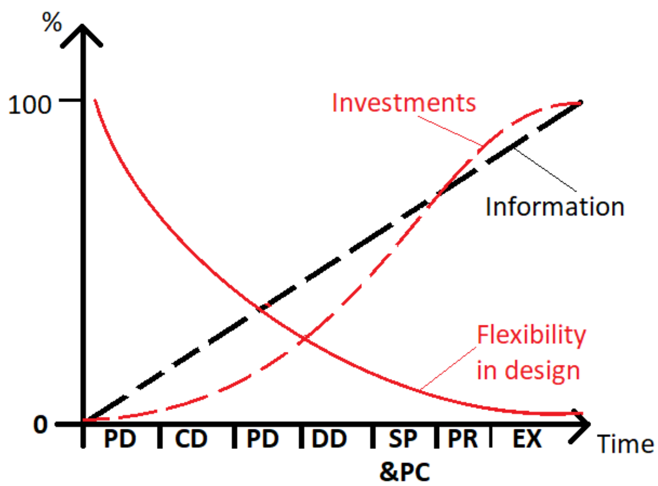


Figure 2.4: Influence of time on building process. Amended from: (van Eekelen, Rip, & Wentzel, 2002, p. 156)

Cost management
Currently, the risk of exceeding the budget can be reduced by managing the costs for each design phase. Proper management of costs in the initiation and design stages can prevent the risk of exceeding the budget during the specification stage. Figure 2.5 visualizes this process, which can be used for the complete building or just a part of it (van Eekelen et al., 2002).
Cost management starts by *estimating the price* for the design of that phase. One method to calculate costs is based on clusters of elements. This means that for each cluster of a type of element, for example beams, the total amount in m, m^2 or m^3 is calculated and then multiplied by the price per unit. Additional costs such as labour costs are included by adding a percentage of the total costs. The specific percentage is different per company. This estimation is then *compared* to the budget for the same phase. If corrections are necessary then this should be done by adjusting the design or the schedule. Finally the client *decides* if the changes are acceptable and sets the budget for the next phase.

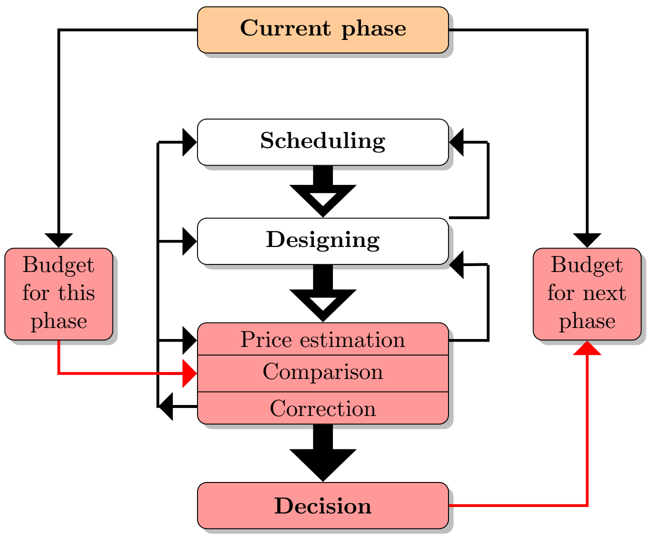


Figure 2.5: Cost management visualised. Amended from: (van Eekelen, Rip, & Wentzel, 2002, p. 122)

2.1.3 The preliminary design phase

The structural engineer's role
For the preliminary phase, the "Standaardtaakbeschrijving" (STB) will be used. This document explains what tasks need to be done at each phase to achieve a reliable design and execution and was created by BNA and NLingenieurs in co-operation with NVTL (BNA & NLingenieurs, 2014b). Using this document, the following tasks of the structural engineer are summarized: (BNA & NLingenieurs, 2014a)

- Designing the preliminary support structure

  1. **Setup starting points for the design**: for example the spatial plan, loading, available free area, fire safety, etc.
  2. **Develop variants of the support structure.** Show what factors were considered, what variant was chosen and why this choice was made.
  3. **Produce the preliminary design.** Show the following: the general design, material choice, general dimensions of the structure, foundation, stability outline, outline dilations, relevant cross sections and outline of the foundation and facade structure.

- **Set up policy document for structures.** The support structure is described and all relevant information for its design is reported.
- Risk evaluation for Structures
- Listing and advising if structure corresponds with the contract.
- Determine the influence of demolition
- Process the results and advise the geotechnics.

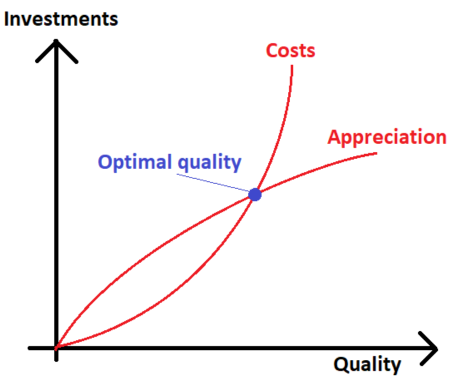


Figure 2.6: Appreciation for quality. Amended from: (J. W. Kamerling & Kamerling, 1997, p. 20)

The client's needs

Costs is not the only important aspect for businesses. The client will be content if the design is not too expensive, on time delivered and of expected quality. Quality in this context means that the design must follow regulations and the additional demands of the client for the suitability, indoor climate, aesthetics, sustainability and durability. Improving the quality beyond a certain threshold will bring additional costs while the benefits will be less appreciated. Therefore the quality is optimal when the highest quality is found for the lowest cost as visualised in figure 2.6. An economical building is one that costs the least but has all the demands and not the least costs per m^2 or m^3 of the building (J. W. Kamerling & Kamerling, 1997, p. 20).

They continue explaining that the designer starts with the SoR, more demands means more expenses. It is the task of the structural engineer to help the architect with this as much as possible. To deal with the limited information, consequences of choices are analysed by designing variants. Only few sensible variants are made due to the required amount of labour.

Cost efficiency for support structures

For the design of the support structure, cost efficiency is found in material or labour costs. This can be done by first finding the optimal material use, then finding a way to make it easily buildable. The opposite is also possible: find the optimal material use for a given (cost efficient) construction method. This must happen in cooperation with the architect, because concessions in design will usually have to be made (J. W. Kamerling & Kamerling, 1997, p. 24).

They continue by showing a few rules of thumb which can be followed for a cost efficient support structure. A few examples: shorter spans are more efficient, bending moments should be minimised and use cross sections which are efficient for a certain loading type (e.g. a truss for bending moments). For the labour costs the following rules of thumb are used: repeating identical elements as much as possible is cost efficient and joints are considered labour intensive.

The problem is that first of all these are rules of thumb and will not always be true. Furthermore, certain fist rules are contradictory. For example joints are labour intensive and thus costly, so minimizing joints would be advised. But for material efficiency, minimizing spans which increases the amount of joints is advised. This contradiction gives more rules of thumb, namely: use beams for short spans, trusses for longer spans and arches or hanging constructions for the longest spans. But these rules of thumb will also not be true for each project (J. W. Kamerling & Kamerling, 1997), which is why variants are needed to find answers to these questions.

2.2 Parametric design in structural engineering

The process of the previous section was traditionally done by drawing on paper, hand calculations and later with FEM programs or other analysis tools. Design changes force the structural engineer to redo his work which is a time consuming process. Thus it takes considerable time to give feedback for design options (van der Linden, 2018).

Complexity of structures and its demands are also increasing which makes it hard to find optimal quality for the support structure. One solution to these problems is the use of structural parametric design. This thesis will use the concept of parametric designing on topics within structural engineering, which will henceforth be called "structural parametric design" (SPD). This section will be used to show what structural parametric design is, how it currently is used and what its advantages and disadvantages are.

2.2.1 What is SPD?

Parametric design can be defined as using parameters for algorithms for calculations and can be done with specific software. What makes this special is the fact that the user has the ability to specify the type of input and its value. The user can for example choose between lengths, profiles, materials, etc. as input. In traditional software this choice is already made and the user can only change the values of the parameters.(Jeroen Coenders, 2018)

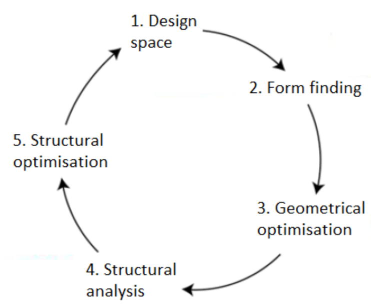


Figure 2.7: General workflow for SPD. Amended from: (van der Linden, 2018, p. 24)

General workflow of SPD
SPD consists of analysing geometry with software that is able to set it up parametrically. Afterwards the structural behaviour is analysed. It is not necessary to go through all the steps of the workflow of SPD, it depends on the problem that has to be solved and the design phase. A general workflow will be shown which consists of the following five steps (van der Linden, 2018):

1. **Set up the design space:** The structural problem is defined and the parametric model is set up.
2. **Form finding:** Explore different shapes to find an innovative or optimal shape. There are different methods to do this, but it will be based on elementary factors.
3. **Geometrical optimisation**: Form finding can produce a shape that is not buildable, affordable or optimal. It is the task of the engineer to interpret the results. An example of this concept is visualised in figure 2.8, where the red lines could be used to make a practical truss.
4. **Structural analysis:** Done with the finite element method. This can be done by exporting the geometry to the FEA software or by using packages within the visual programming software.
5. **Structural optimisation:** Done if optimisation is deemed necessary after structural analysis. It is key to clearly define what results are wanted and what parameters will be used.

For the conceptual design phase steps 1, 2 and 3 are generally followed (van der Linden, 2018). Step 4 is also possible, but this requires the structural engineer to be present at the conceptual design phase.
The information from the conceptual design is known and structural analysis can begin in the preliminary design phase. Therefore only steps 4, 5 and 1 are needed. Complex calculations are not necessary yet, therefore less comprehensive analysis packages can be used, for example Karamba (a FEA solver for Grasshopper) (van

der Linden, 2018).
The author continues explaining that for the detailed design elaborate structural analysis is necessary for which traditional FEM packages can be used, which are connected to the visual programming software through an API. An API is an interface through which software can communicate together. How it works will be explained in a coming subsection. Step 4 shall be repeated in this phase until a satisfactory design is found.
These are not absolute rules though. If certain information is wanted and SPD could be used to find the answer, there is nothing stopping the user from using the steps in the general workflow. One should realize that some form of investment will be necessary when using SPD, so the goal should be clearly defined.

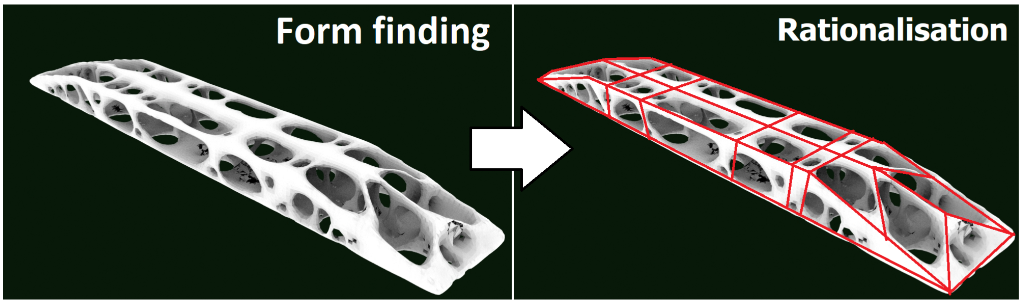


Figure 2.8: From form finding to optimisation. From: (Lee, 2010) and (Xtreme Quality, Service and Innovation, n.d.)

Development of SPD

The concept of parametric designing is not new and the same is true for SPD. Many companies have already been using SPD for approximately five years, while in ARUP it is used for almost 10 years (M. V. van Telgen, 2018).
In 2001 a breakthrough was reached by a non-profit organisation named SmartGeometry, where cooperation between professionals, researchers and students is stimulated for research on computer assisted tools. From this organisation a need for logic to design complex geometry was needed. This was achieved by Robert Aish who developed CustomComponents. This software works in Microstation, which is a CAD-application for Bentley, and was updated over the next ten years (Jeroen Coenders, 2018).
The author continues explaining that the second breakthrough was reached with Grasshopper, developed by David Rutten. He developed a similar application for Rhino called ExplicitHistory. This was later renamed to Grasshopper and was adopted by the active community of Rhino.
The final breakthrough was Dynamo. The engineer Ian Keough worked on a project to use parametric designing with Autodesk Software. Robert Aish was the Research Director there and from here Dynamo was developed. Dynamo is inspired by SmartGeometry but is linked with Autodesk software, for example Revit. Users of Autodesk software work on all aspects of constructions and through Dynamo the concept of parametric design was spread to many architectural firms

construction firms and engineering consultancies (Jeroen Coenders, 2018).
The author explains that presently parametric design is getting more attention and conferences focus on this subject more as well. There is always uncertainty in what the future holds, but change is going to happen. David Rutten still works on Grasshopper and many new platforms are in development. Parametric designing is growing immensely and will stay for the foreseeable future.
Currently the most used visual programming software is Grasshopper (van der Linden, 2018).

2.2.2 Advantages and disadvantages of SPD

Parametric design promises many potential advantages. Some are not fully realised yet while others are already shown to be partly true due to existing practical examples. But parametric design is not something which will solve every problem and will also have disadvantages.
The advantages and disadvantages are given for SPD in general and do not always have to hold true, it is therefore interesting to see if they will hold true for the DRC. By learning from previous experiences, one can try to use the advantages while trying to lessen the impact of the disadvantages as much as possible.

Reasons why SPD is useful
Changes are currently coming within structural engineering where parametric design could become more useful than before (Vissering & van Loenhout, 2018). These can be named as follows:

- Projects are becoming more complex, which makes analysing variants more laborious.
- There is more demand for optimisation. This does not only mean monetary costs, but also for example sustainability.
- Projects become bigger and more complex which requires more experts.
- 3D-models where multiple people can add information are becoming more popular .
- Iterative type of work becomes more common.
- Construction is led by the computer model, because of automation.
- The work of the structural engineer is expanding in the design and construction stage.

Advantages of parametric design
The following potential advantages of SPD from literature are shown:

1. Parametric design gives the ability to deal with complex geometries, boundary conditions and many load cases with linear or non-linear material behaviour (van der Linden, 2018).

2. Visual programming is intuitive and can be learned quickly. Automation, generation and optimisation of structures is now possible without extensive knowledge of programming languages (Jeroen Coenders, 2018).
3. Due to the previous point, the structural engineer can focus on his/her own profession and does not have to be a programmer as well (Jeroen Coenders, 2018).
4. Endless amounts of variants can be generated which can vary depending on the chosen parameters (Jeroen Coenders, 2018) (Veenendaal, 2018) (Linssen, 2018).
5. Because of the previous point, optimisation can be done accurately in a relatively simple manner. And is also possible for complex structures (Jeroen Coenders, 2018) (Linssen, 2018).
6. When a certain algorithm is set up for an input parameter, then a list of different values can also be input. This is called "replication" and results can be output more rapidly due to this property. Spreadsheets or programming languages usually do not have this ability.
7. Multi disciplinary optimisation is possible which increases the collaboration between professions (Veenendaal, 2018) (Linssen, 2018).
8. Parametric models can be reusable if properly set up and stored in a cloud. Models will not have to be remade from scratch if they are already stored on the cloud and over time more models will become available. If this could become open-source, then companies could benefit from each other (Veenendaal, 2018).
9. Adjustments in the design are possible during the whole construction process with little time loss (Linssen, 2018). Certain design choices and variations can now be done in a later stage of design, which gives the ability to wait until concrete choices have been made before making alterations (van der Ploeg, 2018).
10. Engineering mistakes will be lower, which could allow for a reduction of safety factor values. Human errors are made by individuals and will learn from their mistakes. Others will repeat the same mistakes though. But when an error in software is solved it will not be repeated (Linssen, 2018).
11. The role of the structural engineer will be made easier and it will not replace the engineer. The software will have to be written by the structural engineer (Linssen, 2018).
12. SPD is an excellent way to find a design that considers all demands in the beginning of the design stage. The support structure can be taken into consideration together with demands from other professions (van der Aa & van den Bos, 2018b).
13. Visual programming directly visualises the resulting data. It can immediately be seen if the script works properly (van der Aa & van den Bos, 2018b).

14. The ability to instantly see the results and thus the consequences of choices is especially useful for starting engineers. Experience could be gained in a rapid pace due to their exposure to many more structures compared to the traditional work process.

Disadvantages of parametric design

Some advantages can also be disadvantages depending on how it is used. For this reasons some topics from the previous paragraph will be repeated here. The following potential disadvantages are discussed:

1. **Time investments:** Setting up a parametric model from scratch is Laborious and thus time consuming work. At the same time there will hardly be any results until the model is finished, then huge amount of data is expected to be output. This workflow requires an investment at the start of the design which introduces a risk of not producing enough results. Therefore the use of SPD must be justified and preferably be used for difficult and/or repetitive tasks. (van der Linden, 2018):
2. **Suitability:** Some designs can be too difficult to parametrise, which can be especially true when exceptions in the design are present. The tool could end up costing more time and produce less results compared to traditional designing (van der Linden, 2018).
3. **Loss of flexibility:** The script for the parametric model starts being build with certain relations in mind. The model is then expanded based on this script. It is possible that the earlier relations will change which in turn could render the newer scripts unusable. Depending on the severity of these changes this would require adapting the script to the new relations which is time consuming, inconvenient and frustrating for the designer. The same problem could occur when more parameters have to be added (van der Linden, 2018).
4. **Individual work:** Visual programming gives the opportunity to solve the problem in many different ways. The individual will build the script based on what makes most sense to him/her. This problem can be compared to trying to read and understand someone else's script in any other programming language. It can be hard to read and difficult to understand someone else's logic (van der Linden, 2018). Therefore it is usually an individuals work, but not necessarily so if proper rules on how to set up the script are set beforehand.
5. **Knowledge of structural engineer is necessary:** This is mainly the case because the finite element method is used for analysis and it must be interpreted correctly. To do this the user must have extensive knowledge of mechanics. On the other hand, this does ensure that the structural engineer still has a role in the design process even with parametric design (van der Linden, 2018, p.32).

6. **Specialised tools limit functionality:** Continuing from the previous point, it is not always necessary to have a structural engineer. Sometimes special tools are made for structural analysis with its own UI. The UI makes sure that the inputs and outputs are correctly handled. But this requires the programmer to greatly limit the functionality for the user.

7. Generating multitudes of variants makes it **difficult to analyse all the data**. Therefore proper visualisation of data is needed before conclusions can be drawn, but this is not always possible (Veenendaal, 2018).

8. **Current payment methods** are based on providing service for construction documents and delivering the parametric model itself can cause problems with copyright infringement. A solution could be to only deliver relevant data, but it requires restructuring the building process (Veenendaal, 2018).

9. The previous point goes against the open-source mentality within the community. Construction companies should be united. Though current trends are that **commercial software is only available through licenses** (Veenendaal, 2018).

10. **It is not possible to use SPD to make a conceptual design of a complete building** and then fully analyse it structurally to a detailed design. This is due to the fact that there are almost infinite amounts of possible variables. Lowering the amount of variables is also no option because even with limited amount of variables the amount of required calculations is too large (Linssen, 2018).

11. Developing tools for parametric design **requires an investment without the guarantee of return on investments**. Parametric design was therefore not commonly used. The difference now is that standard software is developed more (Linssen, 2018).

2.3 Currently available tools

Other tools will also be analysed to see if or how the DRC is different compared to currently available tools. This will be limited to tools that use the combination of visual programming software with structural analysis software.
It is not possible to analyse every software combination and this is not the goal. The analysis is done to see what factors should be considered when choosing software and how the DRC is unique among its kind.

2.3.1 Currently used software for SPD

SPD is currently available in many different forms. They are usually developed with different purposes in mind and therefore have some unique qualities. Commonly used software will be listed in this section.

Parametric software will be shown first (figure 2.9). The most popular three visual programming software are: GenerativeComponents, Grasshopper and Dynamo.

Parametric software	
GenerativeComponents	Connected to Bentley's Microstation (an CAD application)
Grasshopper	Connected to Rhinoceros3D by Rhinocentre
Dynamo Studio, Dynamo Sandbox, Dynamo for Revit	Connected to software by Autodesk
Sverchok	Connected to Blender
Other examples of software/platforms with parametric properties	Fusion 360, Inventor, Marionette, Möbius, packhunt.io, paraPy, Shapediver and Viktor

Figure 2.9: Parametric software

New functions can become available for parametric software when packages or plug-ins are used (figure 2.10). There are numerous amounts available, which can be directly downloaded through the parametric software or through websites. For example www.food4rhino.com or www.dynamopackages.com (Jeroen Coenders, 2018).

Popular plug-ins for parametric software	
GeometryGym	Connects Grasshopper with BIM and analysis software. Example of FEA software are: Oasys GSA, Robot, ETABS, SCIA Engineer, SAP2000 and more.
Grasshopper-Tekla Live Link	Connects Grasshopper with Tekla Structures
Grevit	Connects Grasshopper to Revit
Hummingbird	Connects Grasshopper with Revit
VisualARQ	Connects Rhino and Grasshopper with Revit
Karamba3D	Integrated tool for analysis and engineering for Grasshopper
kangaroo	An engine to solve form-finding and optimisation problems within Grasshopper
Structural Analysis	Connects Dynamo with Robot Structural Analysis

Figure 2.10: Plug-ins for parametric software

Structural analysis calculation software (figure 2.11) can be directly connected with parametric software through plug-ins. It is also possible to program the connection if an API is available. Currently the following software is popular for SPD, this does not necessarily have to be FEA software, framework software is also possible for example.

Structural analysis software						
Ansys	AxisVM	Diamonds	DIANA	GSA	IDEA	Matrixframe
QEC	RFEM/RSTAB	Robot Structural Analysis	SCIA Engineer	Sofistik	Struct4U	Technosoft

Figure 2.11: Potential structural analysis programs connections

Developments towards BIM are happening in the building industry. Therefore interest has also sparked for a connection with BIM software (figure 2.12).

BIM Software					
Allplan	Civil3D	ArchiCAD	Infraworks	Revit	Tekla Structures

Figure 2.12: Potential BIM software connections

Finally, knowledge of programming languages (figure 2.13) is necessary to develop a connection with an API. It can also be used to develop other connections or user interfaces. It is also possible to use these languages to write additional functions in certain software.

Object orientated Programming languages for connecting software		
C#	Python	VB.net

Figure 2.13: popular languages

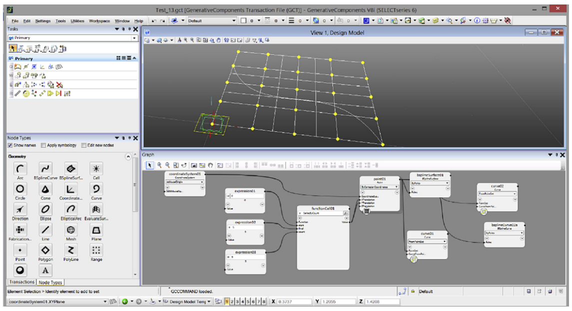
2.3.2 Comparison between visual programming software

The choice between different visual programming software can be difficult to make. Therefore a comparison will be made for the three most used software: Generative Components, Grasshopper and Dynamo. The research of Aish and Hanna (2017) was based on the experience of novice users. This is important because it is key to make visual programming as accessible as possible. The easier the software is to use, the higher the chance that a new user will pick it up.

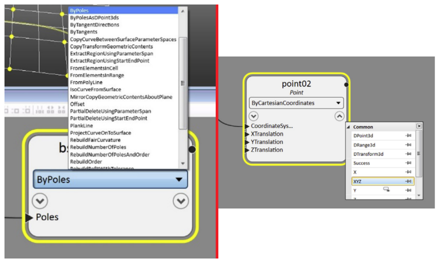
GenerativeComponents. The basic functionality of the software is well visualised such that little aid is necessary for novice learners. Though discovering more advanced functionality is problematic because it is not well documented (Aish & Hanna, 2017).
The authors continue showing that there is much flexibility for changes in the script, because nodes can be changed and functions can be added to nodes without deleting and remaking them. It also has the ability to select specific visualised geometry, which makes adapting the script even easier.
The result is that functions did not give unexpected results, workarounds are less needed and the script is not convoluted. This is mainly applicable for novice users (Aish & Hanna, 2017).

Grasshopper. This software uses data trees as a metaphor for lists or arrays to generate geometry, which is difficult to explain to novice users. Though data trees can be incredibly valuable if used correctly and it is anticipated that advanced users will learn this anyway. Also the nodes were presented well enough to not give results that were unexpected by the user (Aish & Hanna, 2017).
The authors continue explaining that due to the metaphoric use of terminology, the actual functioning is misrepresented. For example a node with the term "flattening" is used to combine arrays into one array. Changing the nodes not possible. They must be deleted and remade, losing all connections which have to be labo-

riously reconnected again. Additional functions can also not be added to nodes, which forces the user to add notes which makes the script larger.
Due to the use of data trees, multiple approaches are possible for a given problem. Depending on the user this can be convoluted or efficiently done. It is not possible to select specific geometry for editing purposes unless the geometry is "baked", but "baked" geometry cannot be re-generated (Aish & Hanna, 2017).

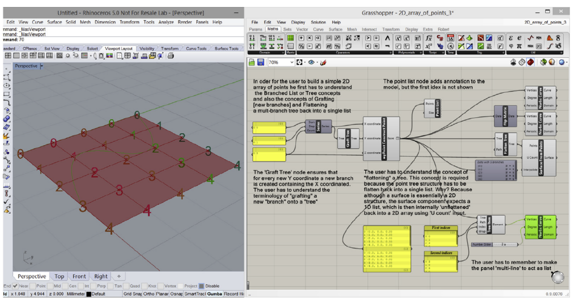


(a) Generative Components UI. From: (Aish & Hanna, 2017)

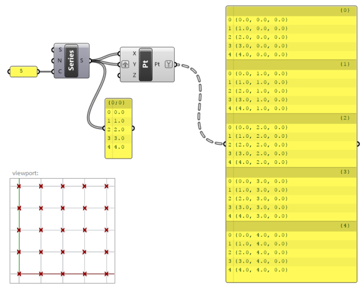


(b) Editing a node. From: (Aish & Hanna, 2017)

Figure 2.14: Generative Components



(a) Grasshopper UI. From: (Aish & Hanna, 2017)

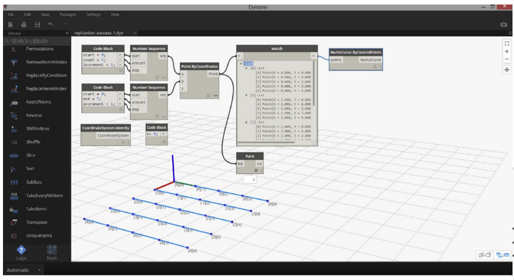


(b) Example of a data tree output from a node. From: (Ramsden, 2013)

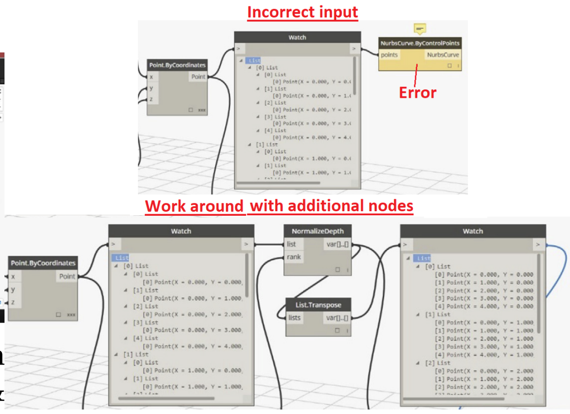
Figure 2.15: Grasshopper

Dynamo. The problem of using metaphors is more prevalent in Dynamo, because it is not only present in node naming, but also in its inputs, outputs and how it manipulates lists (called lacing). Terms are also inconsistent sometimes which adds to the confusion of using metaphors. Specific functionality can also be difficult to find because of the convoluted menu (Aish & Hanna, 2017).
They continue showing that instead of data trees, Dynamo uses lists to manipulate its data which could be less clear to novice users. The list must be set up correctly to get the correct functionality of the node, which is exemplified in figure 2.16b. To effectively use Dynamo, the user is forced to understand how the lists work and how they are manipulated by nodes. This can be confusing to new users. Lists can also force the user to use additional nodes to work around these problems which

makes the script larger.
Dynamo also does not have the capability to edit nodes and have to be recreated and rewired. More nodes are also necessary to get the same functionality compared to the other software.
Issues with lists have caused many unexpected results among new users, which makes the software less intuitive. Selecting geometry is not possible directly and additional nodes are necessary to select the correct list item (Aish & Hanna, 2017).



(a) Dynamo UI with a proper list shown in the watch node. From: (Aish & Hanna, 2017)



(b) Improper list with a workaround method. From: (Aish & Hanna, 2017)

Figure 2.16: Dynamo

According to Aish and Hanna (2017) the tests were performed to find a trajectory of concept learning over time and is represented in figure 2.17. The GenerativeComponents curve indicates gradual learning with a barrier in the middle. A barrier means that a difficult concept is reached which requires some time to learn.
Grasshopper users will experience many barriers, but they are consistently evenly spaced. This makes the learning process constant.
Novice Dynamo users will experience some barriers in the beginning, but after that there will be gradual learning until another barrier is reached for advanced users.

Conclusion
The research mainly analysed how accessible the three programs are to novice users. Based on this research, novice users will find Generative Components the most accessible because functions did not give unexpected results, workarounds are less needed, the learning curve is gradual and the script is not convoluted. More advanced uses are problematic because it is not well documented.
Advanced uses are deemed important for the DRC. Data trees in Grasshopper or lists in Dynamo can be considerably valuable for advanced users, but both have different issues for novice users. Based on this research it cannot be concluded that one is better than the other. According to figure 2.17 both programs require

the same amount of time to learn the given concepts. It will depend on what learning curve the user is more comfortable with.

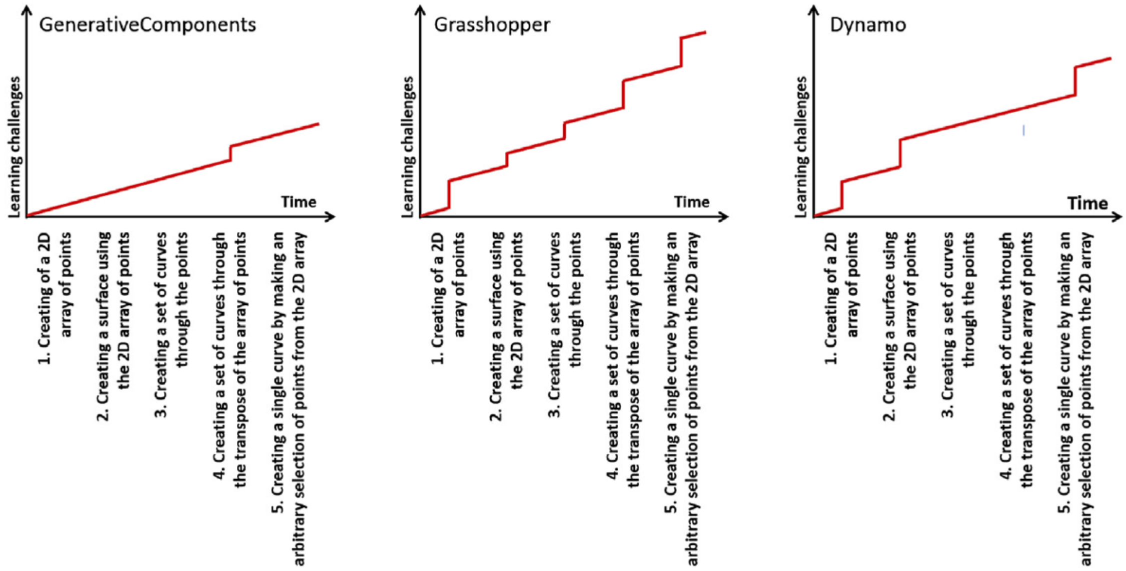


Figure 2.17: learning curves of three parametric design systems. From: (Aish & Hanna, 2017)

2.3.3 Examples of current application of SPD

The previous subsection showed that there are a lot of options for potential tools. The goal of this subsection is to state some currently used workflows to examine what already is done with SPD tools and why certain software was chosen. The functionality of several possible workflows will be analysed to show how SPD is currently done in practice.

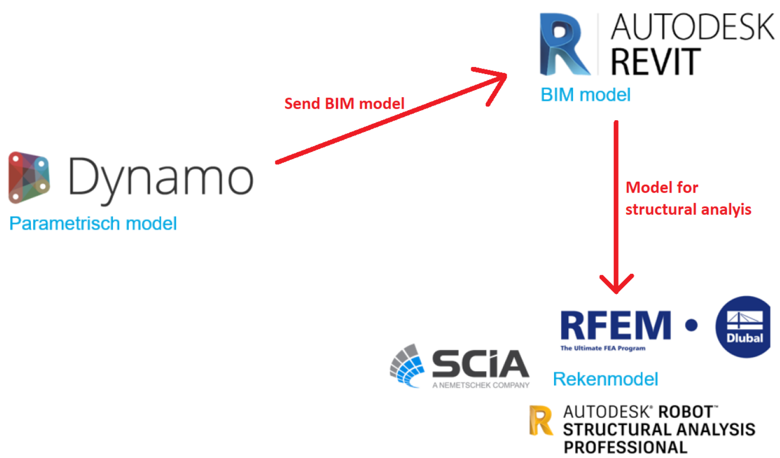


Figure 2.18: The workflow visualised. Amended from: (M. van Telgen, 2018)

Dynamo to Revit to FEA software (figure 2.18). Plug-ins exist for this type of connection, for example CADS for SCIA engineer and RFEM has the plug-in internally. This workflow is slow though and some data is lost when data is transferred to other software. Data also cannot be written back into Dynamo, which is why only one direction of flow is possible and thus no optimisation is possible (M. van Telgen, 2018). This connection was tested internally by Arcadis Nederland B.V. and is not available for public use.

Grasshopper and Dynamo can both be used for a **direct connection with FEA software** as seen in figure 2.19. This workflow is able to rapidly exchange data without loss of data, because no intermediate step is necessary. Data is able to flow back to Dynamo which makes automated optimisation possible. The advantage of Dynamo sandbox is that it does not require a license while Grasshopper does for Rhino (M. van Telgen, 2018). The connection between Dynamo and RFEM is available for internal use within Arcadis Nederland B.V. and not available for public use.

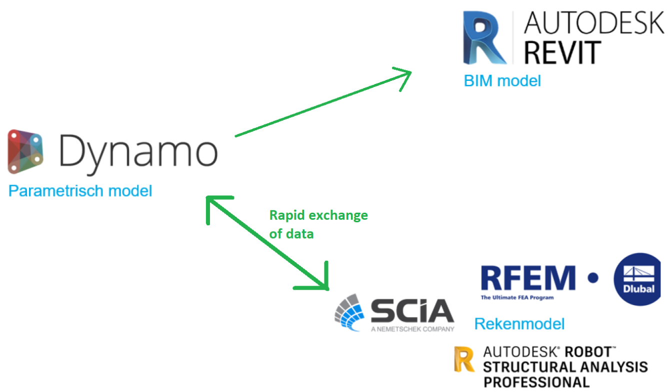


Figure 2.19: Potential workflows for Dynamo. Amended from: (M. van Telgen, 2018)

Direct connections between **Dynamo or Grasshopper with FEA software** is also possible with **plug-ins. GeometryGym** can be used for example in Grasshopper to connect with various FEA software. Dynamo can use the plug-in Structural Analysis to connect with Autodesk Robot (the same developer). The advantage of these plug-ins is that no knowledge of programming is necessary to use the tools. However, one cannot make changes in functionality of the plug-ins and is now dependent on the developer. Also plug-ins are not available for every FEA software, for example a plug-in for SCIA does not exist for Dynamo. An intermediate step just like the Dynamo-Revit-FEM workflow can be used for SCIA, but will yet again give data loss as stated before (M. van Telgen, 2018).

Direct connections between **Dynamo or Grasshopper with FEA software** can also be created from scratch with an **API**. Grasshopper and Dynamo have an API. The condition is that a working API is also needed for the FEA software which is able to communicate with the chosen visual programming software. For example SCIA engineer does not have an API yet, while RFEM and Robot do. The choice for FEA software is dependent on what exactly is wanted from the tool and should clearly be defined beforehand (M. van Telgen, 2018).

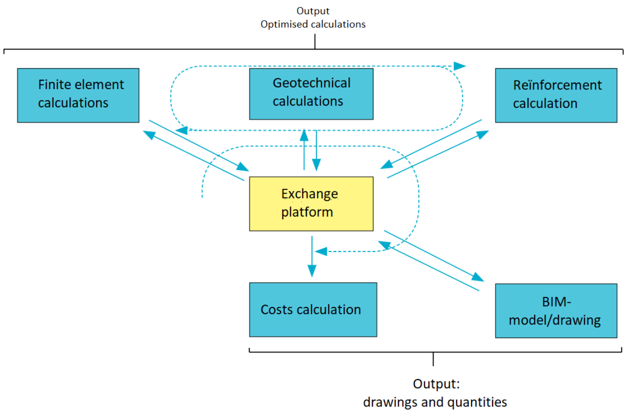


Figure 2.20: Exchange platform with multiple modules. Amended from: (Bongers & Mast, 2018)

A modular approach to parametric design is possible and done by the company VolkerInfra. It consists of an exchange platform which is the main parametric model. Design modules are then used to calculate and check specific problems by using parametric designing. The interaction between the different modules is done by the exchange platform. Modules can be added or removed depending on the needs of the user or the complexity of the project, which can save computation time (Bongers & Mast, 2018).

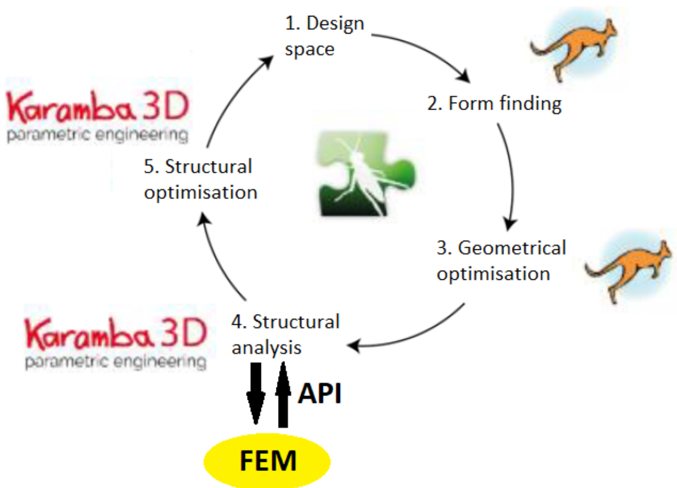
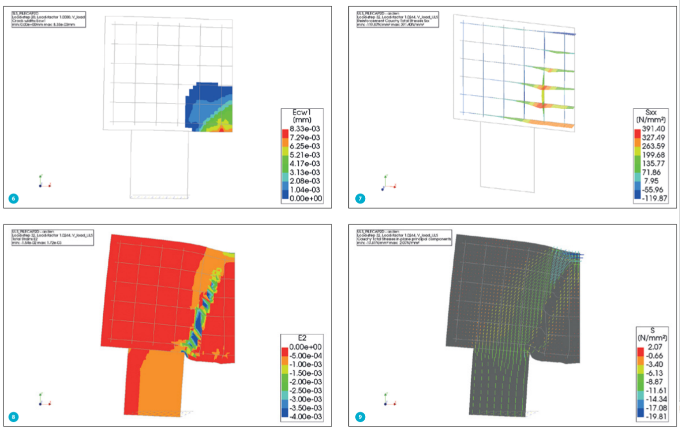


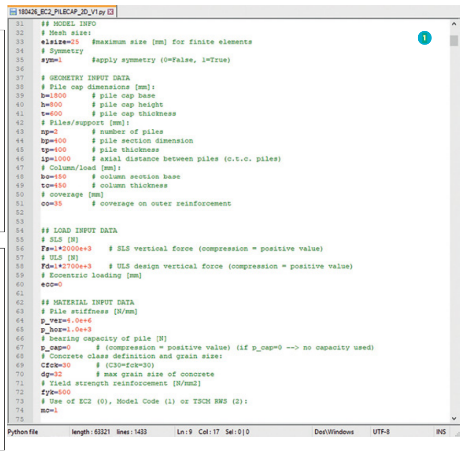
Figure 2.21: General workflow with potential software. Amended from: (van der Linden, 2018)

The general workflow will be used to show the capabilities of **Grasshopper**. For *step 1 design space* no software is necessary to define the structural problems. Form finding and geometrical optimisation can be done with form finding software, for example Kangaroo for Grasshopper(van der Linden, 2018).
The next steps structural analysis and structural optimisation can be performed with a plug-in, for example Karamba 3D. Karamba is a FEA plug-in which performs structural analysis within Grasshopper and is able to evaluate thousands of closely related design variations for optimisation purposes.
For detailed structural analysis, traditional FEA software is used. This can be done with GeometryGym which can connect to Oasys GSA, Robot, ETABS, SCIA Engineer, SAP2000 and more (van der Linden, 2018). It is also possible to use an API, but it requires the user to program the connection, which is extensive work.

SPD can also be used for smaller structural elements for more advanced analysis. An example is a **connection between Diana and 3D drawing software**, which can be automated with Python. Diana FEA BV automated the design of a footing with non-linear analysis (figure 2.22a). The goal was to design the reinforcement with non linear material properties, reinforcement and bonding/slipping of reinforcement. The footing was finally tested and optimised successfully for ULS and SLS. This was possible with a simple code as seen in figure 2.22b (van der Aa & van den Bos, 2018a)



(a) Analysis of the footing in Diana. From: (van der Aa & van den Bos, 2018a)



(b) The code used for automation. From: (van der Aa & van den Bos, 2018a)

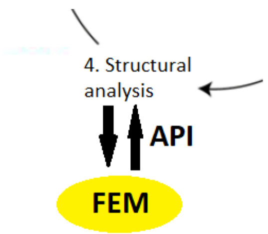
Figure 2.22: parametric design with Diana

There are numerous unique examples of workflows currently used in practice (Hordijk et al., 2018a) (Hordijk et al., 2018b) and it is shown that there are many different uses of SPD. Each with its unique purpose depending on what was originally wanted by the users.

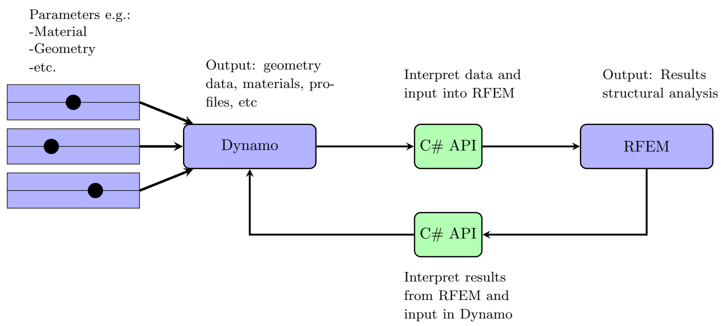
2.4 The Dynamo-RFEM connection (DRC)

The previous section showed that there are many different SPD tools already available. Most tools have the same basic functionality and usually use some form of optimisation. The difference is that most tools are made with a specific function in mind that gives them an unique quality.
Users of Autodesk software work on all aspects of construction (Jeroen Coenders, 2018) and they could therefore have relations with many construction firms. If their products are already used, then the decision for Dynamo could be useful because it can already connect to other Autodesk software (e.g. Revit). Grasshopper can also connect with Revit, but Rhino would have to be licensed, while Dynamo sandbox can be downloaded for free (M. van Telgen, 2018).
RFEM is able to exchange data with both Dynamo and Grasshopper through an API which is well documented. This is not the case for SCIA and Autodesk Robot. A connection between SCIA and Dynamo is difficult to set up and there is no direct connection available yet. Robot has packages which can connect with Dynamo, but if dependency on other developers is not wanted then it is useful to create a connection within the company (M. van Telgen, 2018).

2.4.1 How it functions



(a) DRC



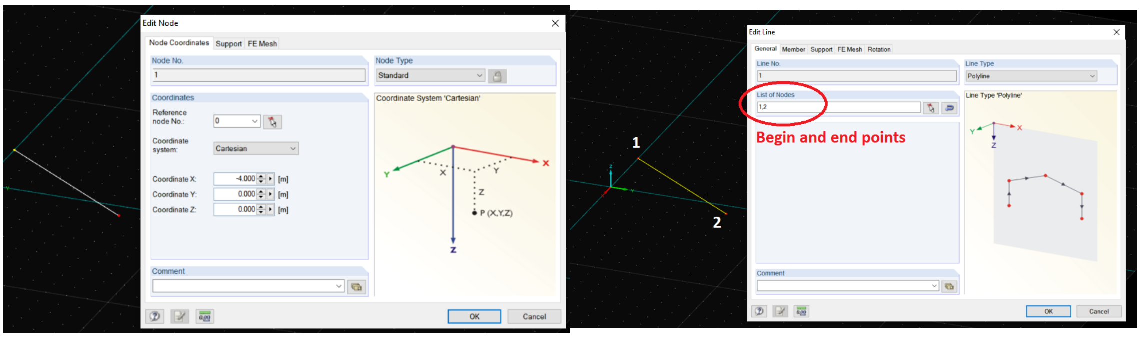
(b) Simple workflow of the DRC

Figure 2.23: The DRC

The Dynamo-RFEM connection (DRC) is a direct connection between Dynamo and RFEM by using an application programming interface (API). The purpose of Dynamo is to be able to set up (complex) geometry in a parametric manner and should also be able to specify the properties of the geometry (e.g. cross sections and materials). The API makes it possible for both RFEM and Dynamo to understand and exchange data contained in both programmes, which makes it possible to send all relevant data to RFEM for structural analysis. Finally, the data is send back to Dynamo for further analysis if needed.
Automatic optimisation techniques are possible if data can be send back to Dynamo from RFEM. The data can be analysed and the parameters changed accordingly. Automatic optimisation is not the only reason to send data back to Dynamo.

If certain features are lacking in RFEM, then this can potentially be scripted in the intuitive Dynamo environment. The data from RFEM can be directly used for this script.

The API connection explained



(a) Setting a node in RFEM

(b) Setting a line in RFEM

Figure 2.24: Setting item in RFEM

An API for Dynamo and RFEM is needed to be able to connect them together, which can be done in Microsoft Visual Studio. The goal of the code is to fill in data in RFEM as seen in figure 2.24. A line for example needs 2 nodes as input (figure 2.24b). Before this can happen, the nodes must be generated in Dynamo with the correct coordinates (figure 2.24a). Each node has its own unique number, which can then be used to select the correct nodes to generate a line. This is a simple example, but it rapidly becomes complex when many nodes are present, because it must be ensured that the correct node numbers are selected for a line. Advanced options in RFEM require substantially more complex code as well.

The Dynamo API is used to find geometrical data, for example the code "DynamoLine.StartPoint.X" gives the x-coordinate of the start point of a given line in Dynamo. This data can then be used for the line "RFEMnode.X" which sets the x-coordinate of a node in RFEM. An example code is shown in appendix A.2 for importing a line from Dynamo to RFEM. The result of this code is the "Elements.Line" node in figure 2.25, which generates the lines into RFEM that are input into the node.
This is done for every needed function until the complete model can be generated in RFEM. After that the structural analysis can begin and its results can be imported back into Dynamo in the same manner. This process is visualised in figure 2.23b

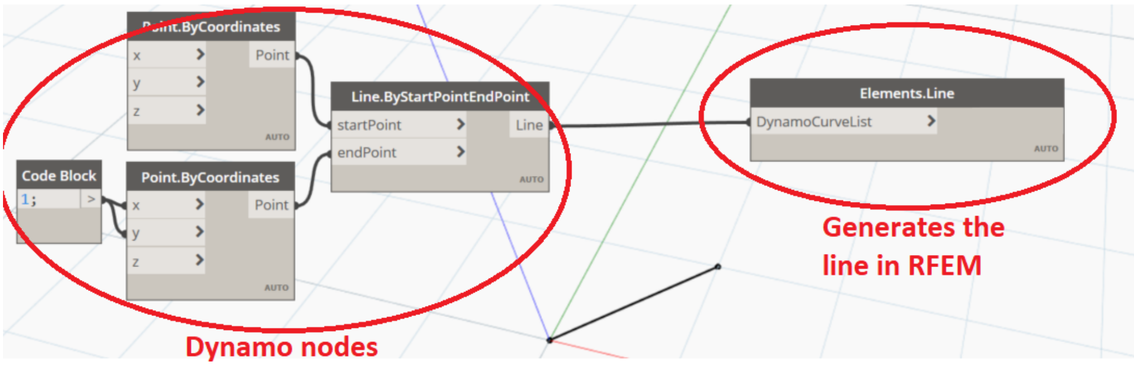


Figure 2.25: Result of the code: a Dynamo node

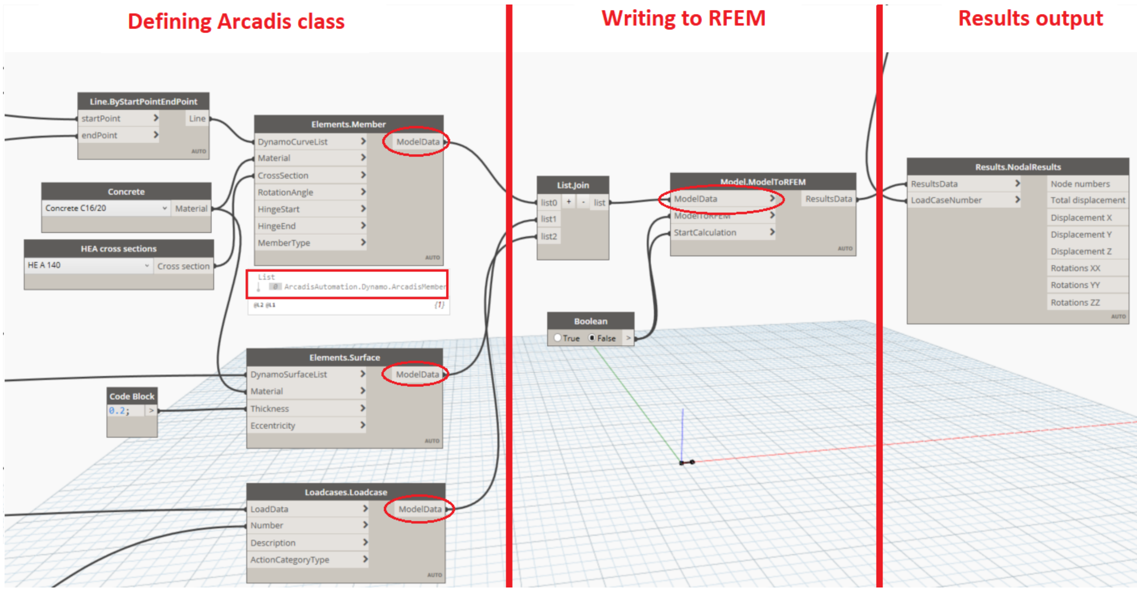


Figure 2.26: The DRC in Dynamo

The data flow of the DRC

The zero touch node of figure 2.25 is only able to import geometric information, but not material information or cross sections. No proper way was found to put this information into the geometry of Dynamo. Therefore additional nodes are created which can output an "Arcadis class" (figure 2.26). This class, in the figure called "Arcadisinformation.Dynamo.ArcadisMember", contains the geometrical data from Dynamo and the user can specify additional information. These nodes are called "Zero Touch nodes" and are programmed in C#.

The output from such a node is called "ModelData". This data can be collected within one list and then input in the node called "Model.ModelToRFEM". This node writes all the modelData to RFEM and is able to start the calculation. The results are then output as "ResultsData". Certain nodes, for example "results.NodelResults", can interpret the data and give the results of the structural analysis as an output in Dynamo. The user is now free to program his own script to analyse the data in Dynamo.

Available functions

The first version of the DRC was available on January 2019 and developed by Michael van Telgen for internal use in Arcadis Nederland B.V. Developing the DRC presents a risk, because the potential functionalities are not clear. There is also the risk that the DRC will not be used at all. A tool that does not work or one that is not used, would be a waste of investments.
Therefore it was decided to develop the tool in steps. In the first version the flow of data was set and a few functions were available, which are shown in figure 2.27. This is approximately 120 hours of work. How the nodes look like and their possible inputs is shown in appendix A.3.

Functions in the DRC (January 2019)						
Main	**Elements**	**Hinges**	**Loads**	**Cross sections**	**Materials**	**Results**
Data flow of DRC	Nodes	Supports	Load case	HEA, HEB, HEM, IPE	Concrete	Nodal displacements
Help file	Lines	Nodal hinges	Self-weight	HF RHS, HF SHS	Steel	
ModelToRFEM node	Members			UNP, UPE, L, LU	Timber	
	Surfaces			UB, UC		
				Timber sections		

Figure 2.27: Current functionality of the DRC

Critical elements are missing for a proper analysis of a structure, for example the loads. Therefore the author of this thesis has added functionality by reworking or adding Zero Touch nodes. For the loads it was necessary to program a new flow of data and reworking the load case and ModelToRFEM node. The new functions are shown in figure 2.28 and can also be seen in appendix A.3. Approximately 1500 lines of additional code were necessary to add these functions.

Added functions in the DRC						
Main	**Elements**	**Hinges**	**Loads**	**Cross sections**	**Materials**	**Results**
Data flow of loads	Spring constants for nodal supports	Line hinges	Load case (reworked)			Nodal displacements for load combinations
ModelToRFEM updated for new functions			Load combinations			Deformations and forces in beams
			Nodal loads			Deformations, internal forces and design forces for surfaces
			Line loads			Results from the RFEM STEEL_EC3 module
			Surface loads			Calculation of concrete surfaces

Figure 2.28: Functionality added by the author

2.4.2 Advantages and disadvantages of the DRC

In the previous section, the general advantages and disadvantages of parametric design were shown. These also apply to the DRC in addition to the ones stated in the introduction of this section. The biggest disadvantage of the DRC is the fact that many functions are lacking and have to be programmed.

RFEM as FEA software has plenty of potential. In short, the UI is intuitive and due to its efficient data input both simple and large structures can easily be created. The software is able to define structures, materials and loads for plates, walls, shells and members. Both solid and contact elements are possible and RFEM is able to do linear & non linear FEA (geometric and material), buckling and fatigue analysis. Finally it is also able to calculate the deformations, internal forces, stresses, support forces and soil contact stresses (Ram Caddsys, n.d.). Many more functions are possible, some of which are the modules.

Modules in RFEM
In addition to the mentioned functionalities RFEM also gives access to modules which enable more advanced analysis of the finite element results. For example the module "STEEL EC3" is able to do both ULS and SLS checks for steel members according to the eurocode (Dlubal, n.d.-b). Other FEA packages for example SCIA (SCIA, n.d.) and Robot (Autodesk, n.d.-b) also have modules which are able to analyse the FEA results.
RFEM is unique due to its functioning API which is able to connect to some modules as well. This gives the possibility to do advanced analysis with the DRC. It is not necessary any more to program the unity checks in Dynamo when RFEM is able to do it in a professional manner.
It is possible to connect with the following RFEM modules (Dlubal, n.d.-a):

- RF-/STEEL
- RF-/STEEL EC3
- RF-/ALUMINUM
- RF-/CONCRETE
- RF-STABILITY
- RX-TIMBER Glued-Laminated Beam
- RF-/TIMBER Pro
- RF-/DYNAM Pro
- SUPER-RC

Most modules have an unique API. It is laborious to program functionality of these modules into the DRC. For this reason it was decided to only program the "STEEL_EC3" and "TimberPRO" modules. This shows that it is possible to implement and use the modules effectively. How the modules function, how they perform the calculations and how the results are used will be explained in the next chapter.

The choice for Dynamo and RFEM
The API of RFEM gave the unique quality to add modules and the following advantages were discussed for Dynamo:

- Dynamo Sandbox is open source (M. van Telgen, 2018)
- It has the same learning time for novice users (Grasshopper as well) (Aish & Hanna, 2017)
- Advanced uses are better performed in Dynamo (Grasshopper as well) (Aish & Hanna, 2017)
- Users of Autodesk software work on all aspects of constructions (Jeroen Coenders, 2018). Dynamo can therefore be connected to other software (Grasshopper as well)

The code for the connection between Dynamo and RFEM has been made available for this thesis. By using this code it was highly likely that it would be possible to implement the functionality needed for the variant study. The cost is that additional code had to be written by the author, but there were no dependencies on other developers. This reduced the risk that certain functions would not be possible. Therefore the decision was made the expand the code of the DRC for this research.
It might be possible that better options exist. Grasshopper for example had many of the same functionality as Dynamo except for being open source. Grasshopper might be a better solution, but this will not be further researched and the focus will be set on the DRC.

2.5 Conclusion of the literature study

The goal of this chapter was to answer the following sub-questions.

1. What is the role of the structural engineer in the preliminary phase?
2. How can parametric design be used within structural engineering and what are its theoretical advantages and disadvantages?
3. What tools currently exist for parametric design in structural engineering and what are their advantages and disadvantages?
4. How does the Dynamo-RFEM connection function? How does it differ compared to other tools?

The role of the structural engineer
The building process can be different for each project. Therefore a general model was used which is applicable for both new developments and altering/expanding existing structures. In this model the architect is chosen in the conceptual design phase and the structural engineer gives consultation in the preliminary design phase.

Four complications are relevant to the structural engineer in the design stage. These are:

- The costs of large scale projects commonly exceeds the budget, which greatly delays or prevents its construction.
- Design changes are expensive at the end of the building process.
- Information is limited at the start (around 50%) of the design stage, while most decisions are made in this stage as well (around 80%).
- Many factors can influence the structural behaviour, which makes it difficult to predict the mechanical behaviour of the structure

These complication are currently controlled with proper cost management and by designing variants of the support structure. However, variant studies are costly, which is why only a few sensible variants are analysed.

Appreciation of the design is managed by the architect. It is therefore the goal of the structural engineer to inform the architect about possibilities for the support structure. This way the architect can make informed decision for the design, by being able to properly weigh options.

Structural parametric design
One solution to the above stated problems is the use of structural parametric design tools. This is not a new concept and has been in practical use for almost 10 years.
Changes are coming to structural engineering and seven reasons were given to use

SPD. The advantages of parametric design can be used to combat the complications in the building process.
However, there are also disadvantages that have to be considered. The most important one being that uniqueness and complexity of current buildings make countless amounts of variations possible. It is difficult to build a parametric model which is able to include the numerous amounts of parameters in a single parametric model. Lowering the amount of variables is also not an option because even with limited amount of variables the amount of required calculations is too large (Linssen, 2018).

Currently available tools
There are many options for software and the most popular ones were given. For visual programming software the three most popular ones are GenerativeComponents, Dynamo and Grasshopper. The latter being the most popular.
The same is true for SPD-tools. Choosing which tool to use or creating a new tool depends on what exactly is wanted by the user. Most tools have the same basic functions, but differ in some key ways. This gives each tool its own advantages and disadvantages. It is this uniqueness of the tool that influences the choice for a particular tool.
The literature study implies that most tools focus on some form of (automatic) optimisation. Whether structural or some other aspect, the end goal seemed to mostly be optimisation. But SPD could also potentially be used for other functions, for example rapidly generating or adjusting structural elements. This is laborious work in most FEA software.

The DRC
The connection between a visual programming software (Dynamo) and FEA software (RFEM) named the DRC is utilized for this research. The DRC is used for structural analysis and is programmed with the API of Dynamo and RFEM. The API allowed additional nodes for Dynamo to be written in C#.
No dependency on external developers is considered as the main advantage, because problems in the code can be solved by the the user. The API is very well developed as well which makes it possible to use most functions in RFEM as well as some RFEM modules. The modules can be used to perform ULS and SLS checks by utilizing professional code.
It should be noted however that Grasshopper for Rhino had many of the same functions as Autodesk Dynamo except for being open source software. Grasshopper might be a better solution, but this was not further researched. The focus will be set on the DRC, because it was more likely that the desired functionality could be implemented for the variant study.

3
The parametric model

In the previous chapter the theoretical functioning of the DRC is explained. This connection has not been used extensively, therefore research will be conducted to show if the previously mentioned advantages and disadvantages hold true. Therefore the Dynamo-RFEM connection will be tested on a case study: The façade of the Bluebell Hotel.

This chapter will be used to explain how the parametric model is set up with the Dynamo-RFEM connection. First the Bluebell Hotel will be analysed to find the factors that have to be considered in the parametric model. Using this information, the parameters can be set up for the parametric model. The wanted results will have to be clearly defined as well before a model is set up. Now all data is available and an explanation will be given of how the parametric model is set up.

Each section in this chapter will be used to answer one of the following sub questions.

1. What design aspects of the Bluebell Hotel should be taken into consideration for the variant study? (3.1)
2. What variables are chosen to be parametrised and what results are needed of the parametric model? (3.2)
3. Is it possible to implement the theoretical positive aspects of parametric design into the variant study? How can this be done while also reducing the impact of negative aspects of parametric design? (3.3)

3.1 Case study: The Bluebell hotel

"The story behind the design is based on a flock of birds." (Urban Climate Architects, 2017a)

The Bluebell hotel (previously known as De Zwerm) is a project in Aalsmeer close to Schiphol airport and is designed by Urban Climate Architects. Increasing amounts of traffic in Schiphol Airport is giving issues with parking places. The bluebell hotel functions as an answer to this growing problem by building a parking garage.
When entering Aalsmeer from Schiphol, the Bluebell Hotel will be the first structure that is seen. This will also be true for aeroplane passengers as they are about to land in Schiphol. The structure will be seen by many and should therefore be aesthetically pleasing. It was not possible to do this with the garage, which is why a hotel will be placed in front of the parking garage.
Just like a flock of birds, the Bluebell Hotel spreads out from a point as can be seen in figure 3.1. By using reflective glass together with the interesting façade, the building will almost move as one walks along the structure. These effects are still visible for passengers who are about to land in Schiphol Airport. (Urban Climate Architects, 2017a)

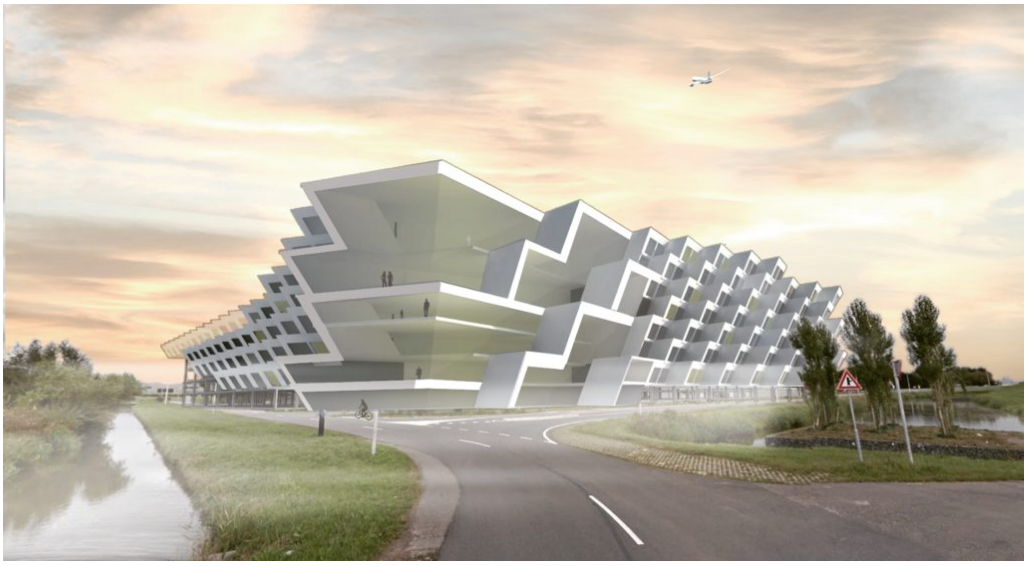


Figure 3.1: Early concept of the Bluebell Hotel. From: (Urban Climate Architects, 2017a)

3.1.1 Choosing the section to analyse

It is not possible to parametrise the whole structure within the scope of this thesis. For this reason it was chosen to analyse a section of the structure. The functionality of the DRC and its ability to inform the architect can then still be tested. Finding out what section would be best suited is done by judging each section with the following criteria.

- The section should be complex enough in a way that there is no obvious solution to the given problem of the structure.
- The section should be large enough to give an interesting structural analysis, but should be small enough to be achievable within the scope of this thesis.
- The section should be able to test the robustness of the tool.
- Variations should be meaningful and give interesting concepts to analyse.

There are a few sections which were considered for analyses, which are named: "the Corner façade", "the Outer façade", "the Inner façade", "garage façade" and "the structural core" (figure 3.2).

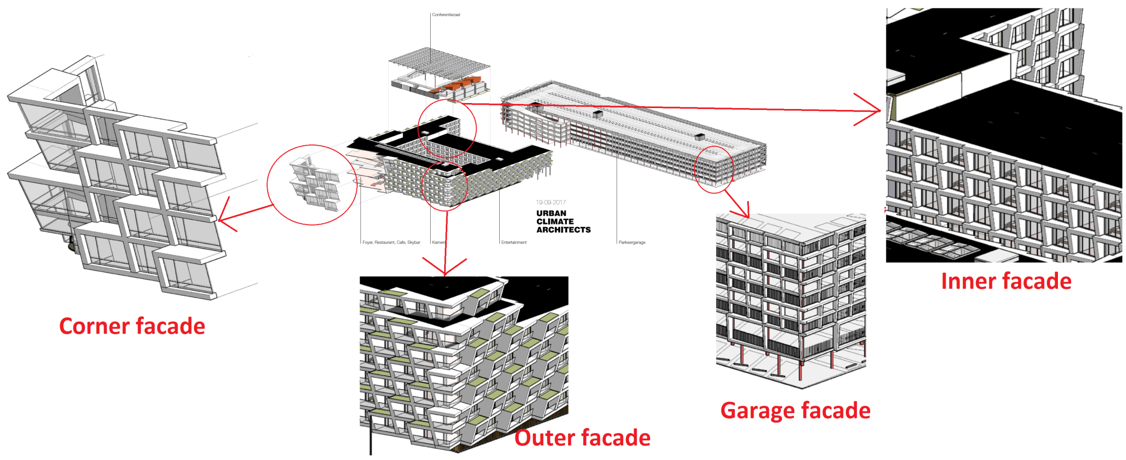


Figure 3.2: Bluebell Hotel parts. From: (Urban Climate Architects, 2017b)

The core structure, as seen in figure 3.3, was considered at first because it's a classical problem for structural engineers to solve. It could become an interesting problem when stability calculations are done in both directions and for torsion, while also considering the changing loads, geometries, materials and openings in the core structure.

The structure would have to be modelled as one large element with certain parameters as seen in figure 3.3. Due to the inexperience of using Dynamo, it is uncertain if the program is able to work with such a large model. If this is the case, then the model will have to become less complex. It would be risky to use this section, because of the potential that a meaningful structural analysis will not be possible. Though this section does follow most criteria and would be a great candidate for analysis.

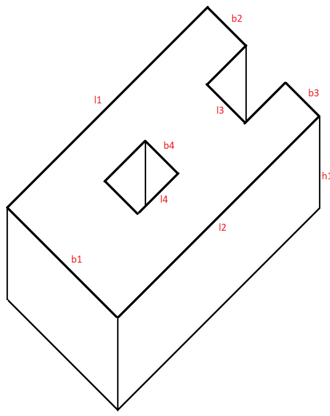


Figure 3.3: Core structure of the Bluebell Hotel

The complex, repeating elements in the **outer façade** makes it an interesting structure for analysis. It can be modelled by generating an element and then copying it over the whole façade. The result should be a complex structure which is made with relatively simple algorithms in Dynamo. The size of this section can be chosen freely when the parametric model is made. This gives the ability to test the complexity and size of a model before it becomes to intensive for the computer.
The balconies are designed in such a manner that it is difficult to predict how certain design changes will influence the support structure. The problem becomes worse when one realises that these balconies are cantilevered. Additional structural problems exist as well in the design of the outer façade.

When the **corner façade**, **garage façade** and **outer façade** are closely compared, one will notice that they essentially have the same shape. The differences are too minor to influence the decision. The *outer façade* could be combined with the *corner façade*, but the issue is that it could require more work to do but result in scant meaningful information for the main research question.

The *outer façade* is the only part that fits all criteria and will therefore be chosen for analysis. The problems that are solved will not always be the same as is done in practice. Nonetheless the DRC will be used in the same manner and can therefore indicate how it will function in practice.

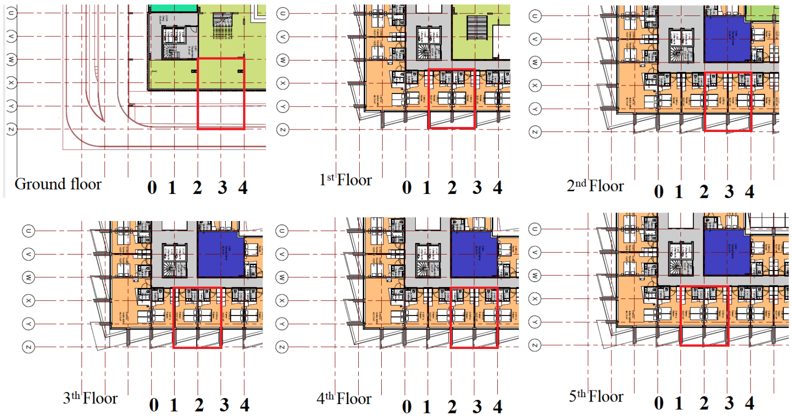


Figure 3.4: Floor plan of the Bluebell Hotel. From: Urban Climate Architects, 2017b

3.1.2 The design of the Outer façade

The shape of the outer façade looks complicated, but it has a simple repeating pattern. The most basic repeating shape is squared red in figure 3.4, which consists of two hotel rooms. Floor 1, 3 and 5 have the same layout and so do floor number 2 and 4. The only difference between the first and second floor is that the hotel rooms are horizontally translated by the width of one hotel room.

The grid lines

The only available information was a project documentation file which contains: the 3D model as seen in figure 3.2, all floor plans (figure 3.4) and a few detail drawings. Due to the lack of information, assumptions will have to be made where necessary.

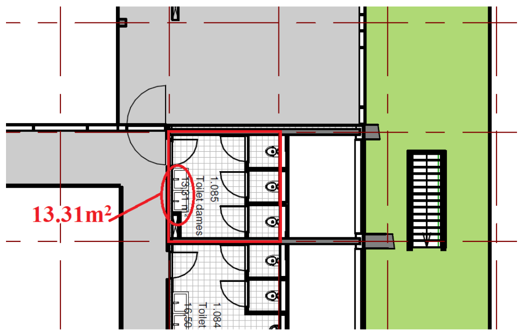
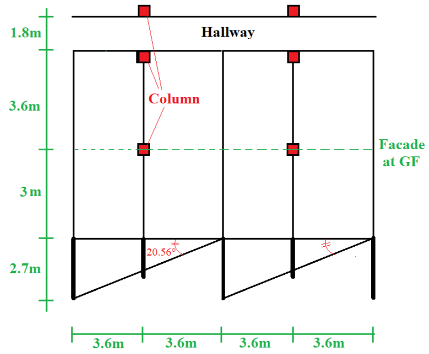
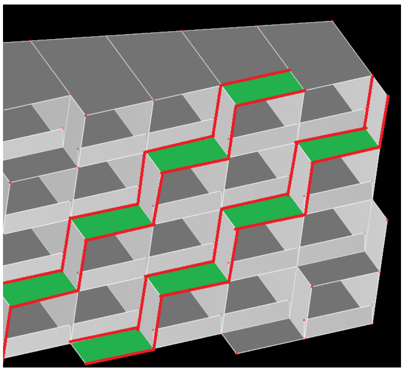


Figure 3.5: A room within grid lines, outlined in red. From: Urban Climate Architects, 2017b

The spacing of the grid lines for example had no value in the floor plans, but the area of rooms was given. A room was found that stays within the grid lines and has an area specified of $13.31m^2$ as seen in figure 3.5. The root of which is $3.648m$. Grid line values usually contain 1 decimal, it will therefore be assumed that the grid line is spaced at $3.6m$.
With this information, the geometries of the hotel rooms as indicated by the architect can be found (figure 3.6a) together with the columns and the wall of the façade at the ground floor level (GF). The columns beyond the hallway are used to understand how the *outer façade* connects to the rest of the structure.



(a) Floor plan of two rooms



(b) Staircase pattern of green roofs

Figure 3.6: Floor plan with geometry and the staircase pattern

Dependency in the geometry

The next issue was the staircase pattern as seen in figure 3.6b, which shows how the living (or green) roofs are layout in figure 3.2. The lack of information made it difficult to understand how the façade is designed due to its complex shape. Figure 3.7 shows where the living roofs are located and that the angled facade spans over two floors. This is how the illusion of the façade is achieved, but it requires certain parameters to be dependent on each other.

Algorithms will be necessary to be able to model this structure. Points are specified in figure 3.8 for the formulas below, these points will later be used to build the parametric model.

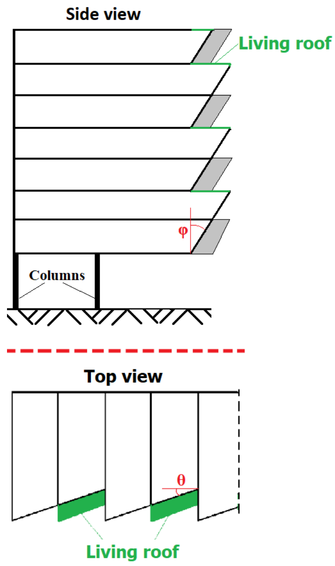
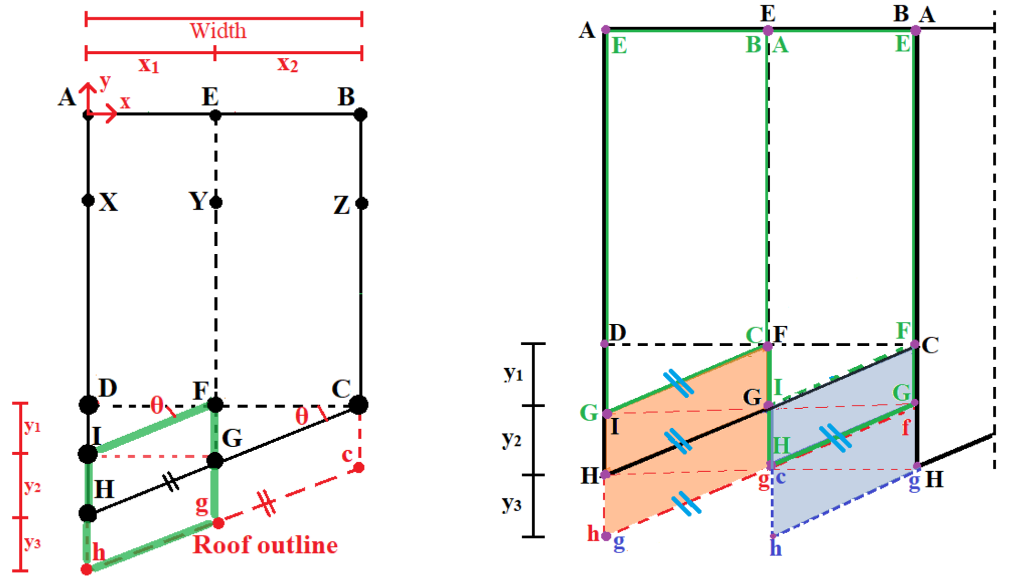


Figure 3.7: Schematisation of the views

To model the green roof correctly, certain lines must be parallel to each other as seen in figure 3.8a. The y-coordinates of point G and H, which are named y_G and y_H, can be found as follows using the parameters: x_1, x_2 and θ.

$$
\begin{aligned}
&tan(\theta) = \frac{y_1}{x_2} \longrightarrow y_1 = x_2 \cdot tan(\theta) \\
&y_C = y_D = y_F = length\ of\ room\ (known) \\
&y_c = y_I = y_G = y_C + y_1 \\
&y_g = y_H = y_C + y_1 + y_2 \\
&y_h = y_C + y_1 + y_2 + y_3
\end{aligned}
\tag{3.1}
$$

The architect chose 3.6m for x_1 and x_2 with $y_1 + y_2$ = 2.7m, which results in the angle $\theta = 20.56°$.



(a) Finding the points H and G

(b) Current floor (black) and underlying floor (green)

Figure 3.8: Floor plans of two hotel rooms with specified points

Figure 3.8b shows the current floor outlined in black and its living roof in orange. The floor below is outlined in green and its living roof in blue. The layout of figure 3.6b is only possible when one demands that the area outlined in blue is equal to the area outlined in orange in figure 3.8b. $y_1 = y_2 = y_3$ must be true as well if the angle over the height, named ϕ, is the same over the whole façade. A simple algorithm can be made as follows, where "h" is the height of a room:

$$
\begin{aligned}
& y_1 = y_2 = y_3 \qquad and \qquad y_1 = \frac{1}{2} \cdot Width \cdot tan(\theta) \\
& y_c = y_I = y_G = y_C + y_1 \\
& y_g = y_H = y_C + y_1 \cdot 2 \\
& y_h = y_C + y_1 \cdot 3 \\
& tan(\phi) = \frac{y_1}{h} = \frac{\frac{1}{2} \cdot Width \cdot tan(\theta)}{h}
\end{aligned}
\tag{3.2}
$$

Interestingly this means that the angle over the height must change depending on the widths of the rooms and the angle θ of the balcony.

A simple 3D model can be made as seen in figure 3.9. This is the most basic shape of the structure which can be copy-pasted to generate the façade.

Figure 3.10 shows the same structure at the left side which contains two hotel rooms. The second model is build from the first one by copying it and translating it by the width of one room. This model can then be copy pasted over the whole area to create the façade of the Bluebell Hotel.

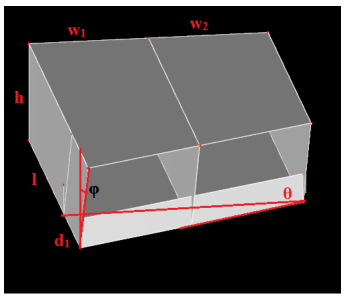


Figure 3.9: Model of two hotel rooms

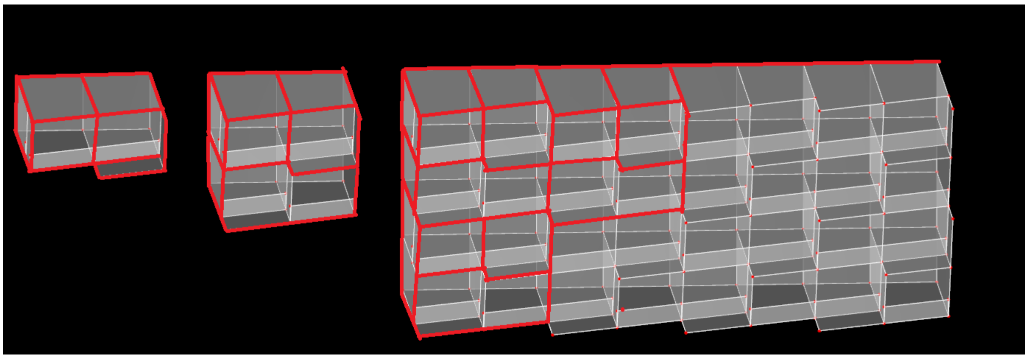


Figure 3.10: Crude 3D model of the façade

3.1.3 The support structure

The previous section explained that the geometric parameters within the hotel rooms have dependencies on each other. This results in complexity in geometry and therefore the underlying support structure as well. Complexity was searched for because an advantage of parametric design is that complex geometries should not be problematic. For this reason it was chosen to not model the Bluebell hotel beyond the hotel rooms. However to correctly model the behaviour of this part, knowledge is needed about how it connects to the rest of the structure.

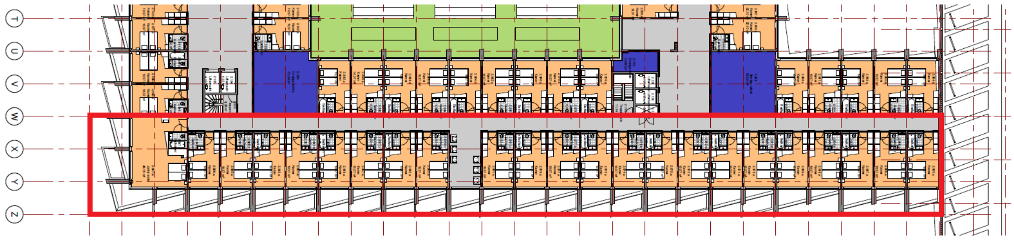


Figure 3.11: The outer façade of floor 2 outlined

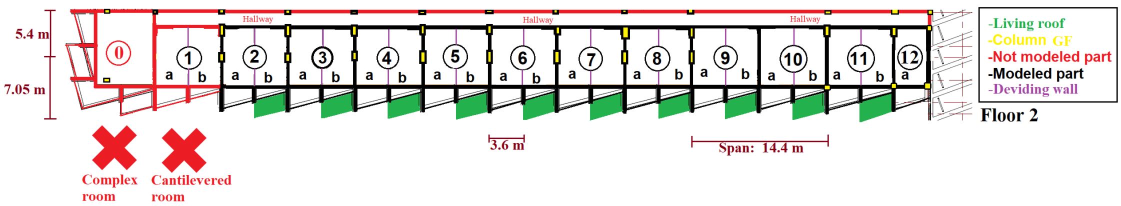


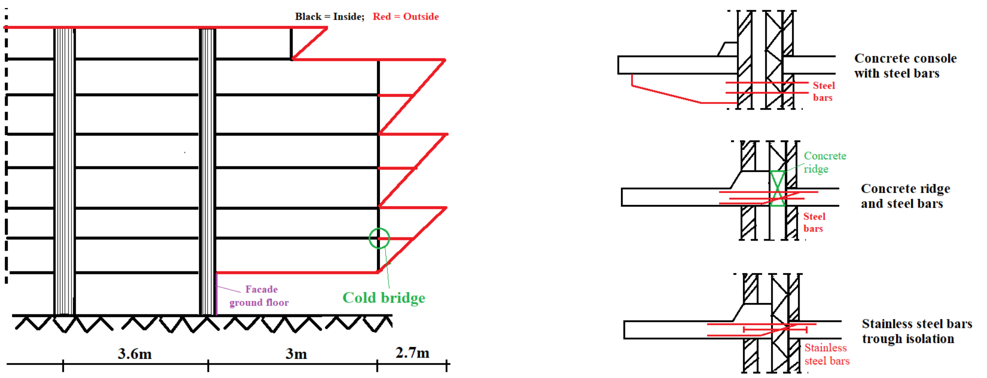
Figure 3.12: The outer façade's support structure schematised

The support structure as designed by the architect is schematized in figure 3.12. The ground floor supports the structure above with columns, and the rest of the structure is supported by walls (the thick black vertical lines).
It was decided to not combine the *corner façade* (room ⓪) with the *outer façade*. If this room is removed, then room ① will be cantilevered due to the inconvenient placement of the columns. This does not correctly model how the room actually

behaves mechanically, therefore it was decided to not include room ① as well. The main problems in this model are the façade's cantilever of 7.05 m and the floor span of 14.4 m in rooms ⑨ and ⑩. One could also pose the question whether the supporting walls should be set every 3.6 m, 7.2 m or 10.8 m. This is not easy to answer, because the design of many structural elements depend on this choice.

The cantilevered façade

A schematized side view is given in figure 3.13a with a column/beam skeleton. This is different from the architects design where columns stop at the first floor and are replaced by structural walls. A column/beam skeleton is chosen for now to exemplify the problems caused by the façade, namely: the large cantilever and the cold bridge.
It is presumed that the architect will strongly disagree with a column at the end of the cantilever for aesthetics reasons. But the floors will be loaded not only by the conventional variable and dead loads, but also the concrete "blocks" that form the façade, which will additionally be loaded by roof loads. This is alarming, because the support structure could become quite massive.



(a) Schematic side view

(b) Potential answers to the cold bridge

Figure 3.13: Side view of the Bluebell hotel. Amended from: (Spierings, van Amerongen, & Bakker, 1998)

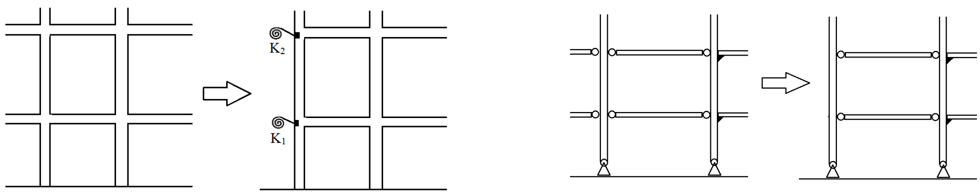
Figure 3.13a also shows what part of the structure can be considered to be in the inside environment and outside environment. To prevent cold bridging, a transition will have to be designed. A few examples are shown in figure 3.13b, where the steel will be used to carry the bending moment to the inside structure. The large loads and relatively large span of the structure will put significant strain on this steel.

Boundary conditions

To accurately model the outer façade the boundaries of the structure must be properly set. This is traditionally done with rotational and translational springs at the boundaries as seen in figure 3.14. Figure 3.14a shows a potential skeleton design for in-situ concrete. The rest of the structure can be simulated by using rotational

springs, which resist the deformation of the column just as the connecting beams would do. The value of the spring constant can be found by finding the relation $M_i = K_i \cdot \theta_i$ of the beam.
Horizontal translational springs should also be present, but its stiffness is difficult to calculate accurately. Assumptions will have to be made of the columns in the rest of the structure and then their stiffness to horizontal load would have to be analysed. The effect of the foundation should also be analysed. It is possible to replace the horizontal springs with roller supports, but this will cause the horizontal loads to go straight to these supports due their infinite stiffness. The influence of horizontal load on normal forces, shear forces or bending moments would be modelled inaccurately.
Therefore assumptions have been made about the horizontal deformations of the structure. With this assumption the horizontal spring stiffness of the supports was calculated in appendix B.1. This gives the possibility to add wind forces to the model.

It is not always necessary to use rotational springs. For example the steel structure of figure 3.14b. In this type of structure, the beams will not influence the rotation in the column due to the hinged connections. For this reason the beams on the uttermost left can be removed without having to add a rotational spring, which will simplify the parametric model. This does show that attention will have to be given to the springs each time the support structure is changed.



(a) Example of an in-situ concrete skeleton

(b) Example of a steel skeleton

Figure 3.14: Side view of the Bluebell hotel

The rotational spring stiffness is dependent on many factors (length of beams, cross sections, material, etc.), which would force the user to calculate the spring stiffness for almost each new variant. It was found to be much easier to include the (unloaded) beams connected to the outer façade with the ends fully clamped in the FE-model. This requires almost no user input due to the ease of generating geometry in Dynamo.

3.1.4 Relevant Euro Code

The following Eurocode documents were utilized: NEN-EN 1990 (Nederlands Normalisatie-instituut, 2011d), NEN-EN 1991-1-1 (Nederlands Normalisatie-instituut, 2011b) and NEN-EN 1991-1-3 (Nederlands Normalisatie-instituut, 2011a) and the Dutch national Annex (DNA). The results are shown in this subsection and a more in depth explanation is given in appendix B.1. Since the analysis is meant to simulate a variant study in the preliminary phase, not every formula has to be checked in depth. Therefore the following aspects are taken into consideration. Mind that the term "Snow1" was used for regular snow loads, while "Snow2" represents snow load on a roof below an adjacent roof.

Loads

- Floors (q_A): $q_k = 1.75kN/m^2$; $\quad Q_k = 3.0kN$
- Balconies (q_A): $q_k = 2.5kN/m^2$; $\quad Q_k = 3.0kN$
- Roof (q_R): $q_k = 1.0kN/m^2$; $\quad Q_k = 1.5kN$
- Living roof (G_LR): $G_LR = 1.7kN/m^2$
- Finishing layer: $G_{fl} = 1.875kN/m^2$
- Partitioning walls: $G_{PW} = 1.3kN/m$
- Sliding doors: $G_{Sl} = 0.9kN/m$
- Balcony walls: $G_B = 1.5kN/m$
- Snow 1 (q_s): $q_{snow} = 0.56kN/m^2$
- Snow 2 (q_s): $q_{snow} = 2.80kN/m^2$
- Wind in zone D ($q_{W,D}$): $q_{W,D} = 0.9kN/m^2$
- Wind in zone G ($q_{W,G}$): $q_{W,G} = -1.3kN/m^2$
- Wind friction ($q_{W,fr}$): $q_{W,fr} = 0.04kN/m^2$

The national annex mentions in 6.3.1.2 (11) that it is possible to set the full weight on two floors and use the factor ϕ_0 for the loads on the other floors.

Safety class and load combinations:

Hotels belong to the CC2 class and have a design lifetime of 50 years, which corresponds to design class 4.

$$\begin{array}{llll} \text{Class A:} & \psi_0 = 0.4 & \psi_1 = 0.5 & \psi_2 = 0.3 \\ \text{Class H:} & \psi_0 = 0 & \psi_1 = 0 & \psi_2 = 0 \\ \text{Snow:} & \psi_0 = 0 & \psi_1 = 0.2 & \psi_2 = 0 \\ \text{Wind:} & \psi_0 = 0 & \psi_1 = 0.2 & \psi_2 = 0 \end{array} \tag{3.3}$$

For the ULS check, the following load combinations are needed (DNA: Table NB.4 – A1.2(B)):

$$\begin{array}{llll} \textbf{Unfavourable} & & \textbf{favourable} & \\ \textbf{1} & 1.35G + 0.6q_A & \textbf{6} & 0.8G + 0.6q_A \\ \textbf{2} & 1.2G + 1.5q_A & \textbf{7} & 0.9G + 1.5q_A \\ \textbf{3} & 1.2G + 1.5q_R + 0.6q_A & \textbf{8} & 0.9G + 1.5q_R + 0.6q_A \\ \textbf{4} & 1.2G + 1.5q_s + 0.6q_A & \textbf{9} & 0.9G + 1.5q_s + 0.6q_A \\ \textbf{5} & 1.2G + 1.5q_w + 0.6q_A & \textbf{10} & 0.9G + 1.5q_w + 0.6q_A \end{array} \tag{3.4}$$

The SLS check requires the following load combinations:

Characteristic

11 $G + q_A$

12 $G + q_R + 0.4q_A$

13 $G + q_s + 0.4q_A$

14 $G + q_w + 0.4q_A$

Frequent

15 $G + 0.5q_A$

16 $G + 0.2q_s + 0.3q_A$

17 $G + 0.2q_w + 0.3q_A$

Quasi-permanent

18 $G + 0.3q_A$

(3.5)

The deflections:
The following demands for deflection will be used for each structure:

Floors with crack sensetive walls (frequent combination):

$$w_2 + w_3 \leq \frac{l_{rep}}{500}$$

Floors frequently walked on (frequent combination):

$$w_2 + w_3 \leq \frac{3 \cdot l_{rep}}{1000}$$

Roofs (characteristic combination)

$$w_2 + w_3 \leq \frac{l_{rep}}{250}$$

Viewable floors and roofs (Quasi-permanent combination):

$$w_1 + w_2 + w_3 \leq \frac{l_{rep}}{250}$$

(3.6)

3.2 Preparation: goals, parameters and results

In the previous chapter it was explained that before the parametric model is made, it is key to clearly consider the following topics:

- The goal of this parametric model
- The required parameters
- The desired results

This way it can be ensured that the amount of mistakes will be reduced. It also clears up what functionality is needed of the parametric model to output the desired results.

3.2.1 The goal of the parametric model

In this thesis the DRC is used for the structural engineer as a tool to help the architect. The previous chapter explained that large scale projects have problems with:

P1 Costs exceeding budgets

P2 Changing the design at the end of the design stage

P3 Limited information at the start of the design stage

P4 Mechanical behaviour of complex structures being hard to predict

The focus of the preliminary phase was to find the optimal quality of the design, which is mainly done by the architect. It is the job of the structural engineer to inform the architect as much as possible of the support structure. This is done by analysing different structural variants, which are currently expensive to make. Therefore the focus of the DRC is set to enhance the process of the variant study, because it will indirectly affect the four stated issues.
The Bluebell hotel is an example of a project where the structural engineer will have to react to a design. This could be considered as a worst case scenario because the engineer has less options and cannot alter the design too much.

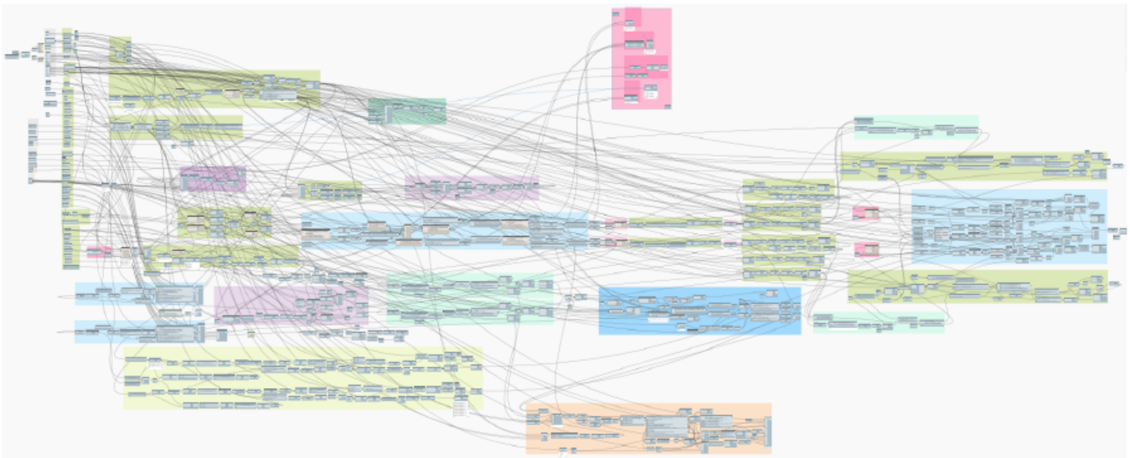


Figure 3.15: Complex Dynamo script. From (Gkioka, 2018)

Disadvantages
The disadvantages of SPD should be considered and their impact should be reduced as much as possible. The following disadvantages are relevant for this case study:

D1 *Individuals work.* Should be solved to make the model reusable. This can be achieved by making the script as readable as possible. A script similar to figure 3.15 should be avoided.

D2 *Loss of flexibility.* Should be avoided for problem P2.

D3 *Time investment.* Time spend using SPD should be less than time spent with current practice, the only exception is if the quality of the structure is greatly increased.

D4 *Specialised tools limit functionality.* Full functionality of Dynamo and RFEM is wanted to give the structural engineer maximum creative freedom. Therefore no UI will be made.

D5 *Difficult to analyse all data of large models.* The results should be visualised as clearly as possible. The large amount of data should be ordered.

D6 *It is not possible to use SPD to make a conceptual design of a complete building.* A way must be found to manage the uncountable amount of possible parameters and calculations.

Advantages

To combat the problems directly or indirectly in the building process, advantages of SPD should be implemented. The following list shows what issues it should mitigate as well.

A1 Complex geometry can easily be taken into consideration. (P4)

A2 Endless amount of variants can be created. (P3 & P4)

A3 Optimisation is possible. (P1, P3, P4)

A4 Proper parametric models can be reusable. (D1, D3)

A5 Adjustments in the design are always possible. (P2)

A6 Software is written for the structural engineer, not to replace him.

A7 The ability to instantly visualise the consequences of design choices. (P4)

goals

The main goal is to increase the quality of the product and to save time for the structural engineer. This is done by improving the variant study with the DRC. Several additional goals are given as well.

- Give the parametric model the ability to generate as many variants of the support structure as needed in a timely manner.
- The variants should not only easily change geometrical dimensions, but also materials and placement of structural elements.
- The structural engineer should be informed as well as possible to ensure that proper decisions can be made for the next variant.
- The output of the variant study should contain information interesting to the Architect.
- Implement the seven named advantages of parametric design.
- Mitigate the six disadvantages of parametric design.

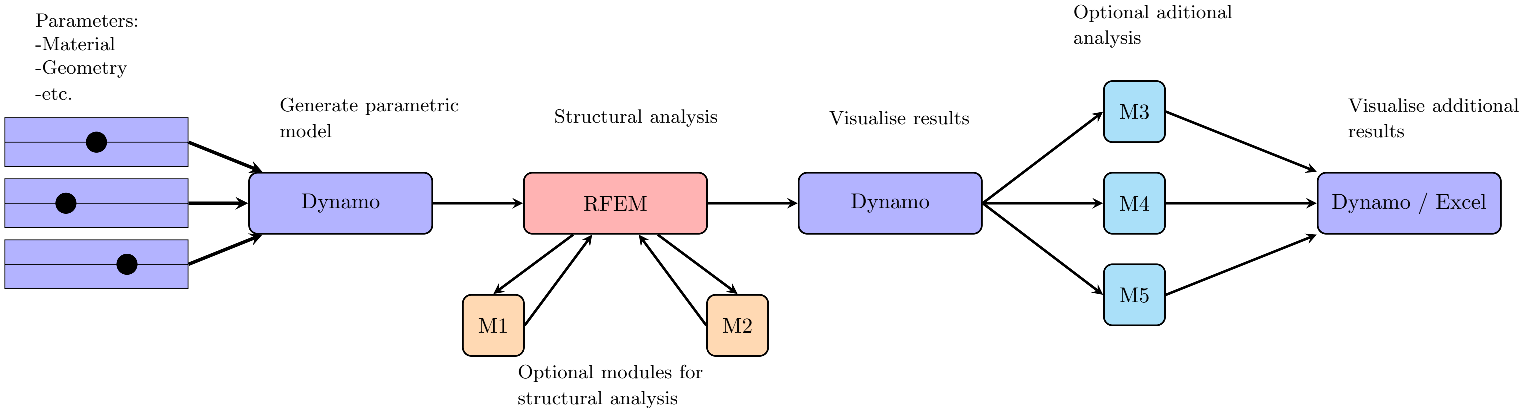


Figure 3.16: The workflow

The workflow
Every project will be unique and therefore the desired results will change as well. It depends on the client and the architect whether costs, sustainability, flexibility, etc. is deemed more valuable. Therefore the idea of a modular workflow (figure 2.20) was used for the workflow of this thesis (figure 3.16).
RFEM has the capability to use pre-made modules, which can be optionally chosen. This would be represented as a node in Dynamo. Module M1 could be an ULS and SLS check for steel members and M2 could be used for concrete floors. More modules are possible if they are programmed.
RFEM and its modules will output certain results in Dynamo which can be used for further analysis. This analysis could happen instantly if it is done with self made modules. These could be modules that perform ULS or SLS checks or modules that calculate costs.
The structural engineer can customize the Dynamo script to only analyse the topics that are deemed important for that specific project. The modules will look like Dynamo nodes which makes adding or removing them just as easy. The modules for additional analysis could be made from e.g. custom nodes or zero touch nodes.
The greatest advantage of this workflow is that the user will always use at most two programmes. One of which is an FEA program which should be known to a structural engineer. Both the interface, input and results are centralized in Dynamo.

3.2.2 The parameters

A few assumptions will have to be made, since this is an example of structural engineering at the end of the design process of the architect. The main one being that the dimensions of the hotel rooms will not be varied, because it's likely that rules apply for functionality. An exception will be made for the angle of the facade in height, which will influence the length of the rooms.

It shall also be assumed that the architect greatly values the aesthetics of the building. Therefore, attempts will be made to change the geometry of the outer facade as little as possible.

This does not include the placement of structural elements though, for example the placement of columns and structural walls will be varied. Their properties (cross section, material, etc.) should also be parametrised. Disadvantage 6 explained that there are uncountable amount of options for these parameters. It is therefore not possible to add all parameters into a single script.
To combat this problem, a way needs to be found to reduce the number of these parameters. The behaviour of the facade is complex, which makes it difficult to predict beforehand which parameters should be reduced. This information will be more available after a structural analysis is done, but the parametric model should be finished before the analysis.

3.2.3 Desired results

The desired results depend on what is actually wanted by the client or architect. It could be costs, durability, flexibility, height, etc. The costs are chosen to be the main factor for the case study. Subjective topics for example flexibility or height of the structure shall be considered though. The final result should be a large amount of variants of the support structure and a recommendation. The architect can then weigh his options to find a variant which is most desirable by the client.

To achieve this, a cost analysis must be done of the variants. The costs of a single variant should be output and clearly visualised. The costs can be calculated by multiplying the length or volume of the structural element to its price per unit. This would require a module for additional analysis in figure 3.16.
The profiles and lengths of the structural elements can be found with structural analysis and should conform to structural demands of the eurocode. RFEM modules could be used to do this automatically and even optimise some structural elements to increase the accuracy of the costs.

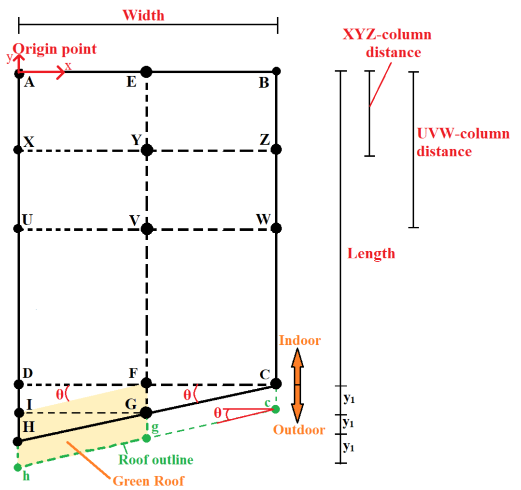
Detailed information about the performed structural analysis should be output for the structural engineer. The expectation is that having more information about the mechanical behaviour of the support structure would be beneficial for the variant study. The reason is that this knowledge can be used to make more informed decisions for the next variant. By using the creativity of the structural engineer, parameters could be filtered which would direct the variant study towards a solution.
To show that detailed information can be output as well from the DRC, cold bridge prevention will be included in the analysis. These checks require the bending moments and shear forces at the transition between inside and outside environment. A self made module is required to check if the internal forces are small enough to prevent cold bridging.

3.3 The parametric model

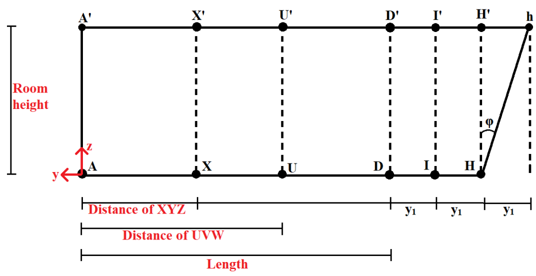
The goals and the Bluebell Hotel have been described. It is now possible to proceed to building the parametric model. This chapter will be used to explain how the parametric model was made and what was done to try to add the advantages while reducing the impact of disadvantages of parametric design.
Sometimes one of the complications (e.g. **D1 - D6**) will be shown between brackets. This means that this particular problem is attempted to be reduced.

3.3.1 Preparations

One of the explained disadvantages of parametric design was the loss of flexibility when the Dynamo script is expanded. It is therefore advised to not directly start in Dynamo when one wants to set up a model, especially for large scale structures. A large amount of time can be saved by planning the set-up of the script.



(a) Top view of the two hotel rooms



(b) Side view of the two hotel rooms

Figure 3.17: The points and parameters defined

This is achieved by clearly drawing and defining important geometry as seen in figure 3.17. The black points A-I and the green points c,g,h will be defined first and collected in a list. Points on the roof are indicated with an apostrophe. With these points all other geometry can be defined that is needed for the variants. For example a floor slab could be made from a surface with points ABCH or a beam can be made with a line from point X to Z. This is done because most geometry in Dynamo can easily be generated with points as input. Points XYZ and UVW can freely be moved in y-direction and will be used for the columns.

The two figures and the use of points make the script clearer, which makes it easier to understand and thus more reusable for other users (D1, D2, D3[1]). The most complicated piece in this Dynamo script is the part that defines the points. The rest of the script will simply select the correct points to generate new geometry (D1 & D2). This makes it easier to define new geometry if needed after the script is made (D3). For example, the script is finished and it is realised that a certain variant requires cross bracing at the right side. The lines that become the cross braces can easily be made in Dynamo by drawing lines from points B to C' and C to B'.

The greatest downside of this script is that adding new points will mean that a big part of the program must be modified (D2). This happens because the points are collected in a list and are retrieved by using their index numbers. When more points are added, these index numbers can change which means that each geometry that uses points must be modified. In this script this is the case for all geometry. So flexibility is only lost when new points are added to the parametric model. It is therefore key to think beforehand which points will be needed before writing the script.

3.3.2 The Dynamo model

There are two approaches to build the geometry of the structure in Dynamo, which will be called the modular and the integrated approach. A modular approach would build the geometry of the support structure by using separate components that can be connected together. The integrated approach means that the geometry of the whole structure is build as one whole without any clear divisions in the structure. The integrated approach will be possible for any structure, while the modular approach is limited to structures that have a repeating pattern.
In subsection 3.1.2 it was shown that the façade can be constructed by repeatedly using two hotel rooms in a certain pattern. This idea was used for the geometry of the structure in Dynamo. This was done because the following advantages were expected:

- The code can become smaller, because less geometry has to be defined.
- A smaller sized code makes the code easier to read and understand. It could also potentially make the calculations go faster, because less functions (nodes) are needed.
- A modular approach can add flexibility and control to the model. It should be easy to add or remove rooms.
- The code is more reusable due the modular approach. A completely different structure can be made with this code.

[1]D1: Individuals work, D2: Loss of flexibility, D3: Time investment

- It is easier to make changes in the geometry or to find and fix mistakes, because the modules will not affect each other.
- It is easy to add or modify geometry of the modules due to the fact that geometry can be defined with the points.
- Different structures will require different modules, which can be added to a cloud. The more this is used, the less time has to be spent building modules.
- The program is made in such a manner that other users can utilize it as well. It isn't individual work any more.

The question if these advantages actually hold true, could only truly be answered if the structure is remade as an integrated model and then compared. This will not be done, but it will be checked if these advantages were actually experienced while using the modular model.

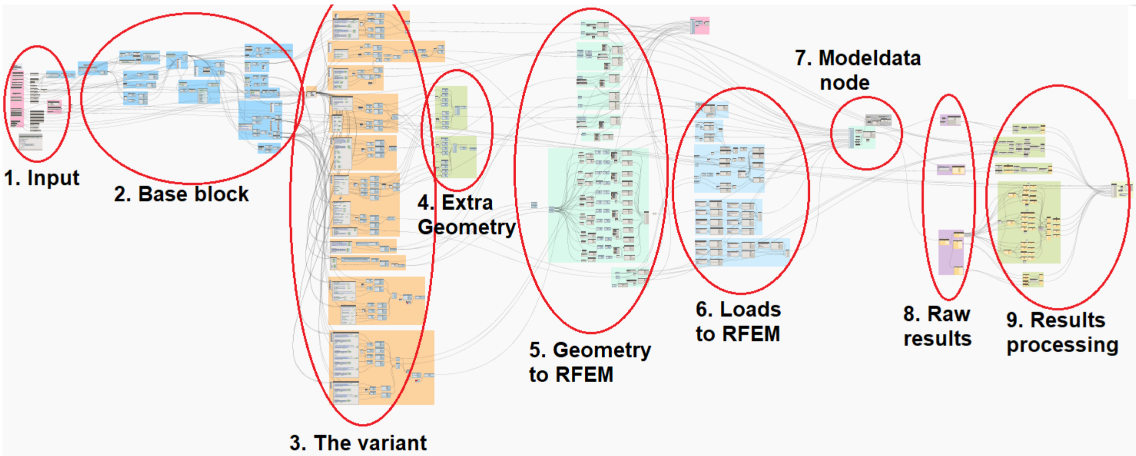


Figure 3.18: Set-up of the Dynamo code

The Dynamo program can be separated into six fields, each specified with their own colour in figure 3.18. 1.Input is where all parameters are defined and changed by mostly using sliders. Field 2.Base block is where the basic block is build, which contains all the necessary geometry for the next field: 3.The variant. Here, the structural variant is build by connecting the basic blocks together. In 4.Extra Geometry geometry is created from the orange blocks, this is useful for geometry that depends on the positioning of certain elements. 5.Geometry to RFEM collects the geometry from field 3 and 4 and assigns properties. 6.Loads to RFEM uses geometry from field 3 and 4 to generate loads, load cases and load combinations. 7.Modeldata node collects all data and sends it to RFEM for analysis. 8. Raw results outputs the results from RFEM. These are huge lists of data and therefore this data must be processed to find the wanted results. This is done in 9.Results processing and it will send the relevant data to an Excel sheet where the results will be summarized.
Each field shall be individually explained in the following paragraphs.

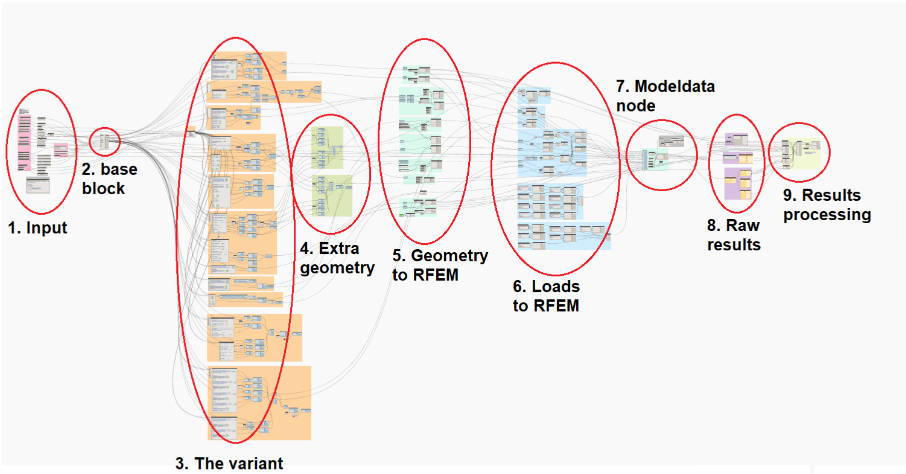


Figure 3.19: Simplified set-up of the Dynamo code

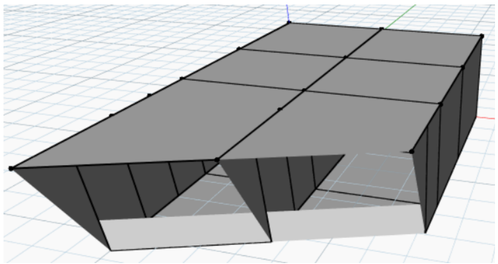


Figure 3.20: The basic block

The base block

The simplest way to set up the Dynamo script for purely structural analysis is shown in figure 3.19. Here, the unnecessary fields are removed and the base block field is turned into a custom node. In fields 6 and 9 some nodes were turned into custom nodes as well. Zooming in on the custom block (figure 3.21) shows the outputs. To effectively use this custom node one should have the drawings of figure 3.17 and the indexes should be clearly defined, which is done here with text blocks around the custom node. The output of the walls is shown as an example in the watch node.
By defining everything clearly, anyone could now use the custom node without having to know how this node is set up (D1[2]). If more elements are needed, the user will have to learn how the custom node works. It is possible to preview the base block, which is done for this custom block in figure 3.20. This block can be used to see how the parameters will influence the model.
It should be noted that the parametric behaviour of the structural elements is coded inside the basic block. It is therefore not necessary to add parametric behaviour for each element after it is translated.

Generating the variant

The variant itself will be generated by copying the geometry of the basic block and pasting it to the correct position. This should be done such that different geometries do not overlap each other. This is because geometry will be exported to RFEM as elements with properties such as weight and stiffness. If for example two lines overlap each other and become beams in RFEM, then they could give twice

[2] **D1:** Individuals work

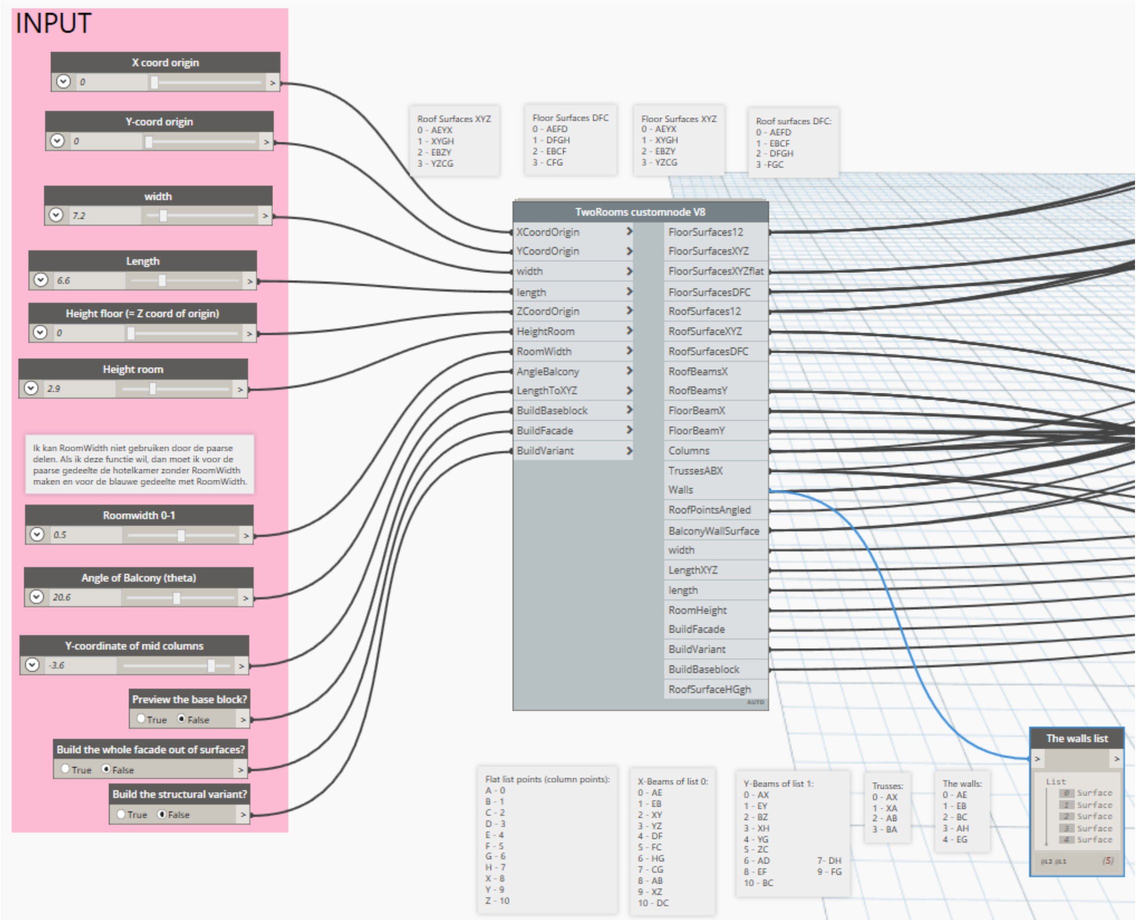


Figure 3.21: View of the custom node (more options were made possible in newer blocks)

the stiffness without the user noticing it. This can lead to dangerous situations and great attention should be given to avoid this. The DRC does partly check for overlapping geometry, but it is not guaranteed for all situations. Therefore it is good practice to build a model without overlapping geometry.
Two modular methods were attempted to achieve this. Whole rooms can be translated to their correct position or the variant could be build by translating structural elements. These techniques will be called the translation by room (TR-method) method and translation by component method (TC-method) respectively.

Translation by room method
The idea for the room-method is to find a room that repeats itself as much as possible. This room, called a "Unique room block", can be build from the elements in the basic block. Comparable to how one builds something from different LEGO Blocks, so too can the façade be build from different Room-blocks. Each unique Room-block can be given its own name and filled into the façade, which is shown in figure 3.22. The number of times these steps have to be repeated is equal to the number of unique rooms, which is equal to the number of orange blocks in figure

3.19
It is advised to make a drawing of how the structure will be build for similar reasons as was done for the custom node. The positions are defined by the the width of both rooms (W), and the height of the rooms (h). It is assumed for now that the height and width stay constant for each room, which makes it easier to define the positions of each Room-block.
This is not necessary though. If a single room with a larger width is wanted for example, then the coordinates can be changed accordingly. The coordinates can always be defined if a proper drawing is made beforehand. The structure starts at 2W, because the first two rooms are not included in the model.

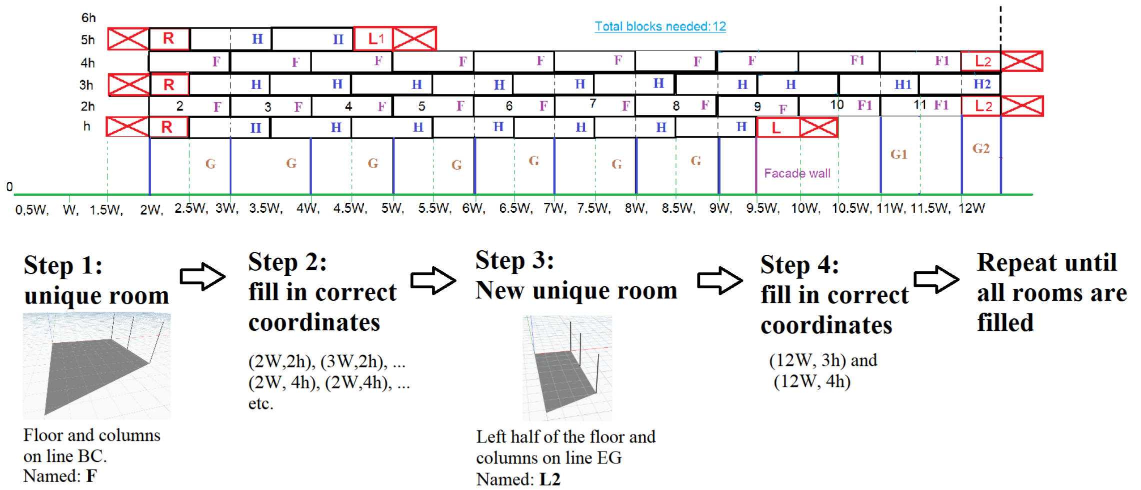


Figure 3.22: The room-method

This method works well for structures where only a few Room-Block are needed, but rapidly becomes problematic in cases such as this case study. Figure 3.22 states that twelve Room-blocks must be build to generate the structure. Twelve unique blocks will make the orange part in figure 3.19 approximately twice as large.
The 12 Room-blocks requirement is not true because it ignores the exception cases, which was at first overlooked by accident. For example the block "L2" is defined as having only a floor and columns, but no roof. Though block L2 in the uppermost right part should also have a roof. There are many more exceptions such as blocks with trusses or blocks at the edges, which makes this method labour intensive and error prone for this structure. Exceptions are thus still a problem. This would be an example of disadvantage 2: suitability.
The data flow of this method is disadvantageous as well. The data (surfaces, lines and points) is at first centralized at the Base block, then spread out per room in the orange part and finally must be collected again and ordered for field 4. data processing. A lot of nodes are needed for collecting and ordering the geometry for further processing, which seemed unnecessary because the data was already centralized. For these reasons another method was fashioned.

Translation by component method

Instead of defining rooms, the element-method defines structural elements such as: floors, columns, beams, roofs and trusses. The goal is to find where each elements repeats itself and then copy and paste it with the coordinates. These elements are still defined by the basic block, which is why figure 3.23 can be used to find the correct positions of the elements in the same manner as was done for the room-method.
This method is used in figure 3.19 and the size of field 3 is now independent of exceptions. But the size does depend on the amount of elements that are used from the basic block. Each orange block is used for a single type of structural element.

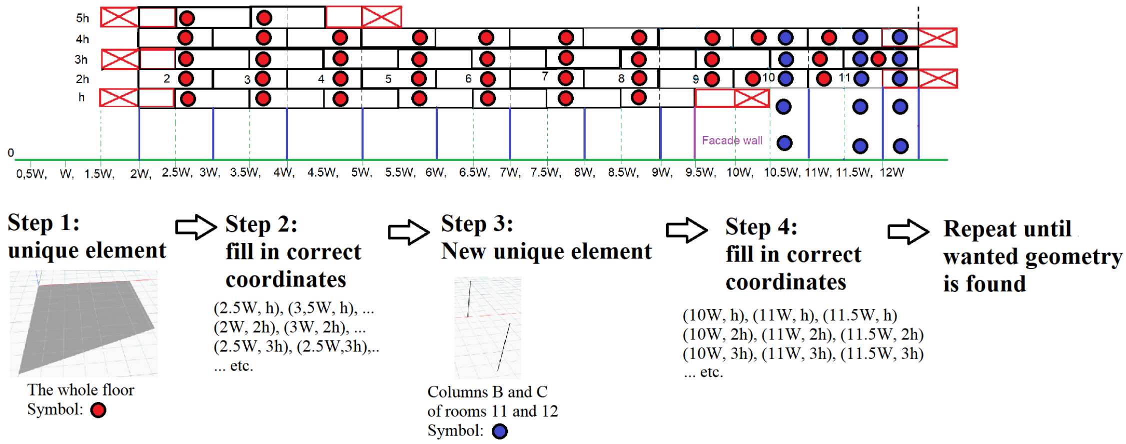


Figure 3.23: Drawing for the element-method

The part of field 3. where the columns at points B and C of rooms 11 and 12 are defined is shown in figure 3.24. This block looks complicated but the process is simple. The List.GetItemAtIndex node uses the output from "columns" in figure 3.21 and takes the columns at index 1 and 2, which corresponds to columns at point B and C. The node next to it named "Columns Type 2" is a renamed node that can copy and translate geometry and does so with little computation time. The basic block has its origin point in (0,0,0), which is why in the code block the coordinates of figure 3.23 can be used.
Similar data is not scattered any more and thus collecting data requires only one node. This also makes finding errors in the Dynamo program easier. The geometry for a variant is defined when this process is repeated for each element and is ready to be imported into RFEM with the DRC.

Errors can easily be made when working in Dynamo, though they are easy to spot due to the fact that the output can immediately be visualised. Correcting them can be done by editing the coordinates. Figure 3.25b shows the output of Field 3. for a steel skeleton with prefab floors. Field 5. is used to visualize the look of the façade (figure 3.25a) and can be used to see if the support structure has the correct geometries.

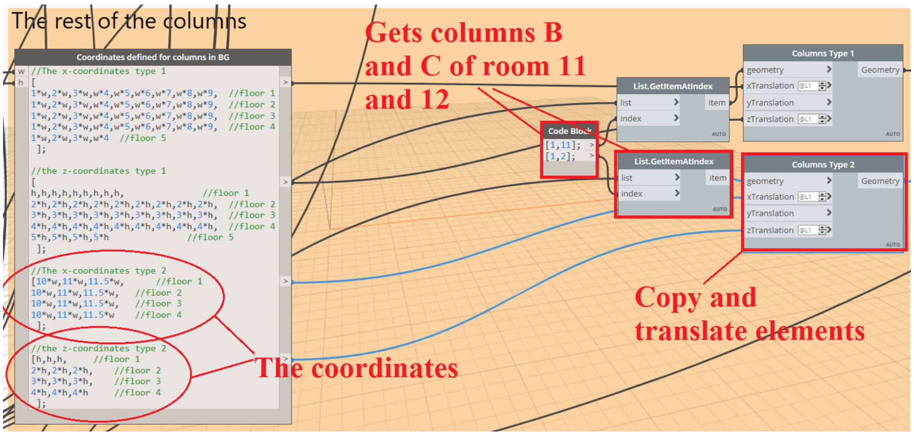
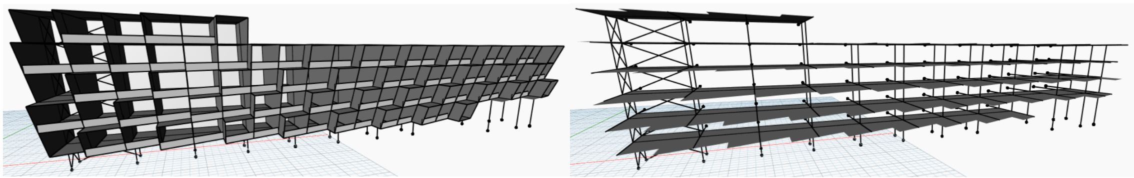


Figure 3.24: Field 3.The Variant, generating columns in rooms 11 and 12



(a) Visualisation of the façade (b) Visualisation of the structural variant

Figure 3.25: Example of a variant

Changing between variants

Changing between variants can be done in two manners: use sliders to vary existing geometry or select different structural elements and use the coordinates.
The sliders will mainly affect what is programmed in the basic block or the material properties and cross sections. Geometry can be varied as well, for example the angle over height ϕ or the width and height of the rooms. These variants require a small amount of input of the user.
Using the coordinate system to define new structural elements can be used to create a new type of support structure. For example varying from a steel skeleton structure with concrete floors to a concrete structure with supporting walls. Smaller adjustments are possible as well, such as adding an additional room or adding trusses to the steel structure variant.
The result is the possibility to significantly change the parametric model, while keeping the functionality of the basic block. It is not necessary to try to implement every possibility for this type of structure in a single parametric model, which makes the script smaller and more flexible for changes. One can build a single variant, analyse it and then use this knowledge to build a new variant. This does cost significantly more time compared to using sliders, but it was attempted to reduce this time investment by implementing the translation by component method.

Geometry and loads to RFEM
The beams in x direction were defined and stored in the node "Beams in x-direction" in figure 3.26. Because the data was already collected in the field 3., the data can directly be given a property. This was not the case for the room-method where data had to be filtered and collected.
The properties are given to geometry with the use of Arcadis Zero Touch nodes. The nodes "Steel", "IPE cross section" and "Hinges" make sure that the correct data entry is input into the Elements.Member node. This node is then able to create the Arcadis member object.

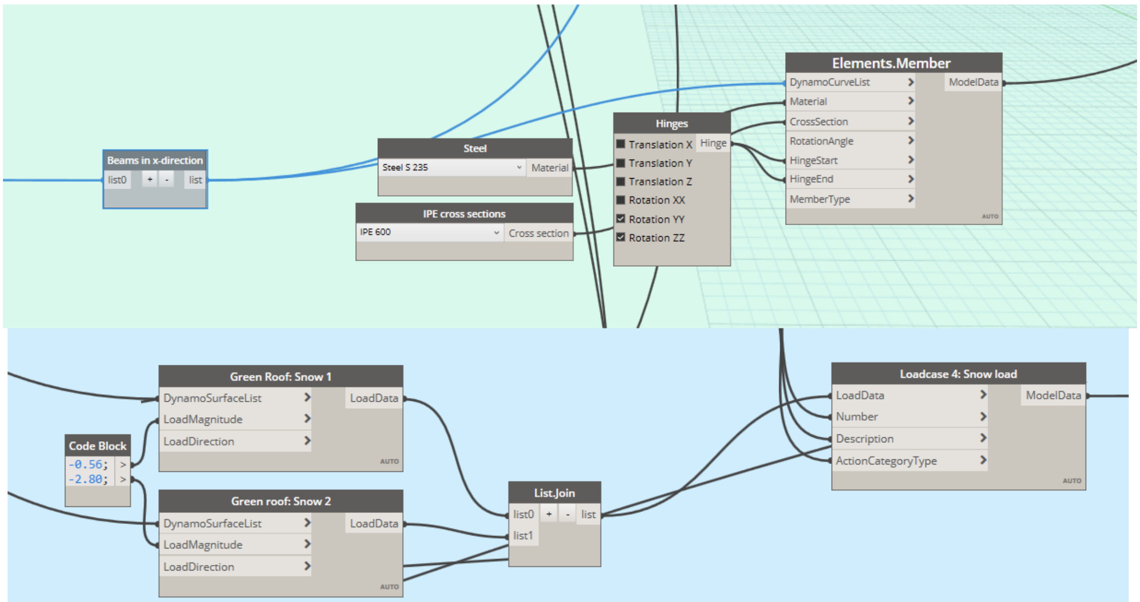


Figure 3.26: Giving properties to geometry (top) and Snow loads and snow load cases (bottom)

The same is true for the snow loads in 3.26 where the green roofs are already defined. The loads are then inserted into the correct load case which exports ModelData.
When all geometry is defined and given properties as Arcadis objects, the data is collected in the list.join node in figure 3.27. The list must be "flat" and can then be input into the Model.ModelToRFEM node. It is possible to automatically do the calculations in RFEM and collect the results. The error watcher is a self made node in RFEM which checks if any errors occurred in the DRC nodes. No errors have occurred if the list is empty in the node outlined in red.

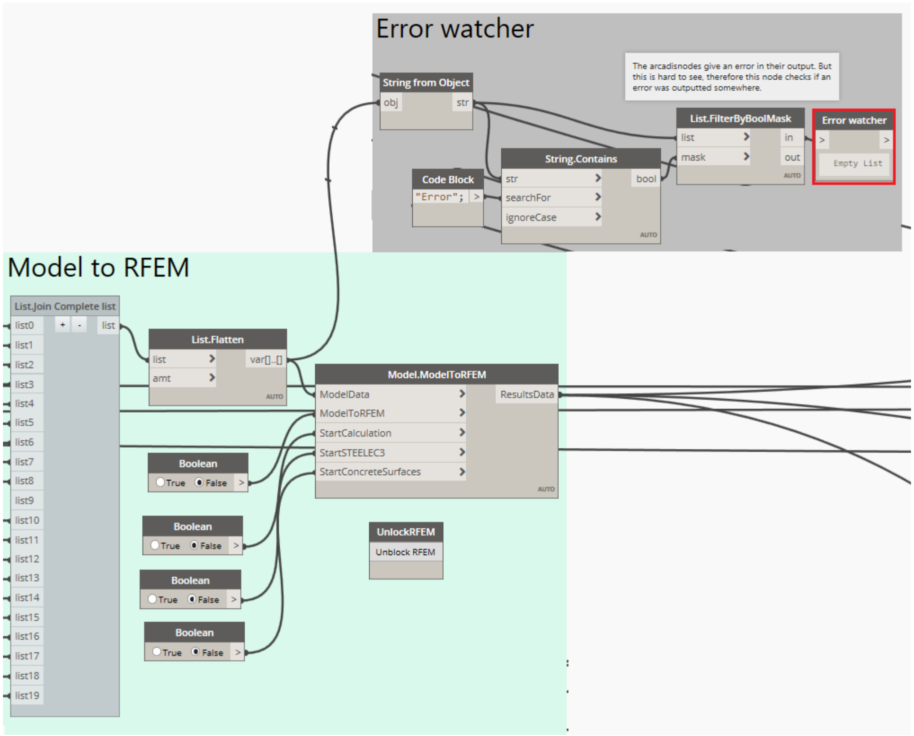
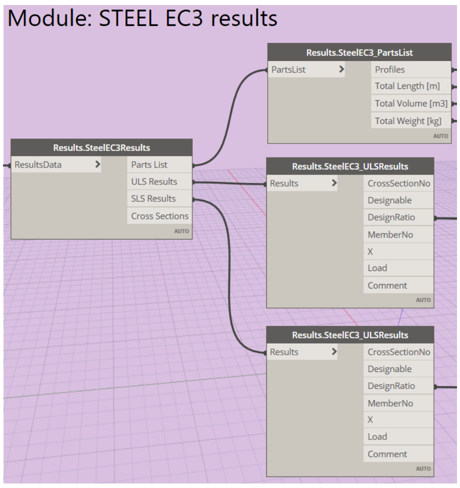
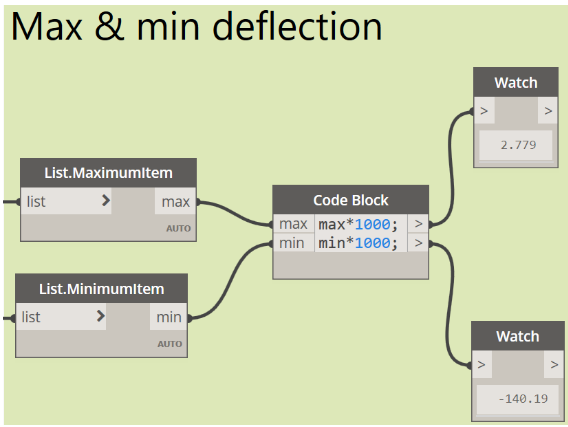


Figure 3.27: The ModelToRFEM node and an error watcher



(a) Raw results from the module STEEL EC3

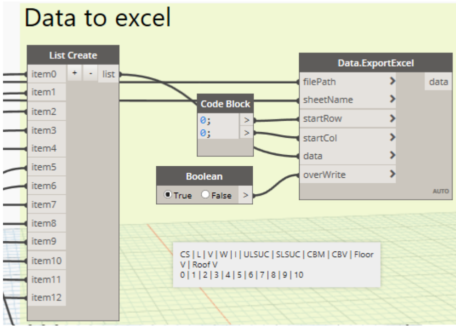


(b) Processing the results: a simple example

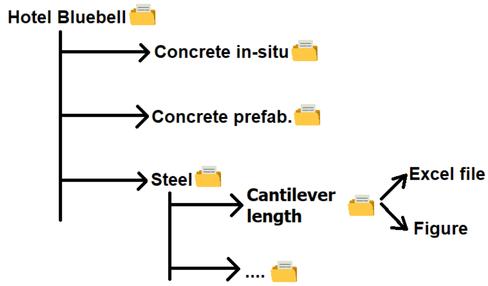
Figure 3.28: Processing the results

Analysis of the results

The node model.ModelToRFEM outputs "ResultsData" which works the same as "ModelData". Additional nodes are used to visualise the results as seen in figure 3.28a and can be used for further analysis. These nodes can output all the data present in RFEM. This is a large amount of data which must be visualised clearly. The maximum and minimum deflection of each load case could for example be calculated as seen in figure 3.28b. This would be a simple example of a reusable module for additional analysis that is made with a Custom Node (figure 3.16).



(a) Outputting the results to an Excel sheet



(b) Organising the data (Schematised)

Figure 3.29: Output to Excel

All the data is now calculated and analysed and can be exported to Excel (figure 3.29a). The most important data is organised and can be viewed as seen in figure 3.30. The data is ordered as follows (from top to bottom):

- Cells outlined in blue visualise the used profiles and their total length, volume and weight. This data is used to calculate the costs or shadow price of the variant.
- Cells outlined in grey visualise the parameter that was varied. In this case the cantilever length.
- Cells outlined in orange visualise the maximum and minimum ULS and SLS ratio's of the structure. These can be used to quickly see if the demands are met.
- Cells outlined in green visualise the unity check ratio for the Isokorbs. This is done for the bending moments and shear forces.
- Cells outlined in Purple visualise the maximum allowable translations. This is an approximated value and is therefore used to give estimations of the SLS requirement.
- Cells outlined in cyan visualise the upward and downward deflection of all elements in RFEM. This is done for each SLS combinations as well as the wind translations.
- Cells outlined in yellow visualise the same but only for slabs. A unity check is performed as well using the advised translations.
- Cells outlined in blue visualise the properties of the floor. How this is calculated will be explained in the next chapter. The total slab volumes can be used to calculate the costs of the slabs.
- Cells outlined in red visualise the output of the RFEM modules. This is done for each cross section and the design ratio is given for each demand. The

API of these modules contained no option to output the critical combinations however. These have to be checked in RFEM if this information is wanted.

The cross sections and their costs are interesting information for the architect. The rest is used to properly inform the structural engineer of the mechanical behaviour of the variant. This information can be used to make an informed decision for the next variant. The knowledge of the structural engineer increases for each variant which can be used to find a support structure for the design of the architect.

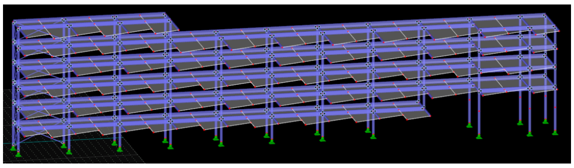
These results will be saved for each variant, which will result in a large amount of data. To organise the data, the results of figure 3.30 are saved on a new sheet each time the parameter is changed. In this example it would be the length of the cantilever. A new excel file is made when a new parameter is chosen. A simple figure will be added next to the excel file to visualise what is being varied in the given excel file. The ordering of the excel files is shown in figure 3.29b

Profiles	1 - HE A 450 | ArcelorMittal (2011)	2 - IPE 80	3 - IPE 500 | British Steel		
Total length	380,8	763,2	582,5		
Total volume	6,78	0,58	6,76		
Total weight	53209	4577	53047		
Lcantilever =	4,06				
	Maximum	Minimum			
Max & min ULS ratio	0,96	0,57			
Max & min SLS ratio	0,56	0,42			
	Bending moments	Shear forces			
IsoKorb U.C. ratio M / V	2,77	6,23			
	Characteristic	Frequent	Quasi-permanent	Wind	
Advised translations: Char. / Freq. / Quasi-perm. / wind	28,8	21,6	28,8	35,8	
Advised cantilever translations: Char. / Freq. / Quasi-perm. / wind	32,5	24,4	32,5	35,8	
Cantilever Unity check: Char. / Freq. / Quasi-perm. / wind	1,08	1,2	0,86	0,85	
	Up / negative	Down / positive			
Characteristic Maximum deflection up / down [mm]	0	-102,1			
Frequent Maximum deflection up / down [mm]	0	-102,1			
Quasi-permanent Maximum deflection up / down [mm]	0	-102,1			
Maximum characteristic horizontal translation negative / positive / U.C. [mm]	0	30	0,85		
	Up	Down	Unity check		
Characteristic Maximum SLAB relative eflection up / down / U.C. [mm]	0	-35	1,08		
Frequent Maximum SLAB relative deflection up / down / U.C. [mm]	0	-29	1,2		
Quasi-permanent Maximum SLAB relative deflection up / down / U.C. [mm]	0	-28	0,86		
Floor properties: **rho/rho0 , d slab , height slab**	0,60	224	246		
Slab shear U.C.: **floor, balcony, roof**	0,56	0,56	0,49		
Bending reinforcement area mm2: **floor, balcony, roof**	883	658	912		
		Continues for all cross sections			
Total floor slab volume	664,3				
Total roof slab volume	239,2				
Cross section number	1	1	1	1	
Design Ratio	0,00	0,01	0,82	0,82	
Demand	CS100) Negligible internal forces	CS101) Cross-section	CS102) Cross-section check -	ST301) Sta	ST302

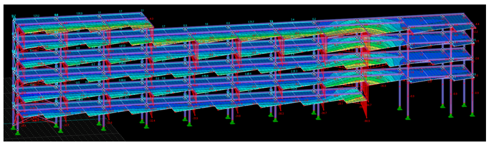
Figure 3.30: The results visualised in Excel

The model in RFEM

The model can be viewed and edited in RFEM after the process is completed as seen in figure 3.31. The structural engineer is always able to check the model if he does not trust the program. This eliminates the black box problem, because a structural engineer should have the knowledge to check if a finite element model behaves as expected or not. It can be argued that FEA software is a black box in itself. But even if one agrees to this statement, it should be realized that more detailed hand calculation will be done in the next design phases which check the results of the FEA software. The output of the RFEM modules can be viewed in detail in RFEM as well. An explanation is given in appendix B.2. The modules can be used to find the most efficient cross sections for a given variant.



(a) The structural variant in RFEM



(b) RFEM results for deflection in z-direction

Figure 3.31: Data processing

3.4 conclusion

The goal of this chapter was to answer the following sub-questions.

1. What design aspects of the Bluebell Hotel should be taken into consideration for the variant study?
2. What variables are chosen to be parametrised and what results are needed of the parametric model?
3. Is it possible to implement the theoretical positive aspects of parametric design into the variant study? How can this be done while also reducing the impact of negative aspects of parametric design?

The Bluebell Hotel
It was chosen to analyse a part of the Bluebell Hotel called the "Outer façade", which consists of the facade and the hotel rooms. The geometry of this structure is complex, but also recurrent. Thus, it is deemed suitable for parametric design. The first two rooms were not included in the model, because it would take considerable time to model, while not adding much relevant information.
Cold bridging will be a problem due to large expected bending moments that have to be carried over. Supports and additional beams will be used as boundaries. By making assumptions for the horizontal translations of the structure, wind forces could be included in the model. The loads, SLS and ULS demands drawn from the Eurocode are used for the analysis of the structure.

The parameters
The goal of the parametric model is to be able to quickly generate different variants of the outer façade's support structure. The costs of each variant will be calculated to give an objective value for comparison. The value of this model is that it provides lots of information for the architect. The result is more options and knowledge for the architect and thus a greater chance to ensure a high quality structure.
The structural engineer will perform the variant study for the support structure. It was therefore important to visualise the most important results from the structural analysis in the Excel sheet. The structural engineer will be able to react to this information and make an informed decision for the next variant. The creativity of the structural engineer is used to filter the uncountable possible parameters and calculations. This way, the variant study can be directed towards a solution without having to try every possible combination trough an algorithm.

For the parameters it was decided to keep the height and the width of the rooms constant, because these could be based on functional demands. However, it is still possible to vary them. Aesthetics are deemed important for this design, therefore attempts will be made to modify the design of the architect as little as possible. The angle over height (ϕ) is the only exception. A specific request was made for this parameter and its influence could prove interesting for the variant study, because other geometric parameters of the hotel rooms are dependent on this value. The uncountable amount of possibilities for the support structure should be available for this analysis. For example, it should be possible to change from a steel skeleton structure to a concrete structure with bearing walls. This requires that walls, columns, trusses, etc. should all be possible to add to the model. However not every possibility will be attempted. What is deemed as a sensible for the new variant will depend on the results of the structural analysis of the previous variant.

The parametric model

The main goal was to combat the four problems which are present in contemporary structural engineering. To do this effectively, it was attempted to add seven advantages[3] of parametric design. Mitigation of six disadvantages[4] was attempted as well. In short this was done as follows:

A1 The complex geometry of the Bluebell Hotel was modelled as well as the complex dependencies between geometrical parameters.

A2 Geometrical, cross sectional and material variations can be performed trough sliders. The translation by component method ensures that structural systems can be varied as well.

A3 Optimisation of the cross sections is possible by using the RFEM modules. However, this method does not guarantee that the *most* optimal design for the Outer facade is found.

A4 With proper documentation, the basic block could become reusable. Self made modules can be reused as well.

A5 Adjustments in the design are made possible with the translation by component method. The structural elements can be selected with the basic blocks and can be edited with the coordinate system.

A6 The DRC can only be used by structural engineers, because the finite element analysis should be properly interpreted.

A7 The consequences of the choices made by the structural engineer are instantly visualised in RFEM and the Excel sheet.

[3] A1:Complex geometry. A2: Endless amount of variants. A3: Optimisation is possible. A4: Reusability of parametric models. A5: Possible to adjust design. A6: Software written for structural engineer. A7: Instant visualisation of consequences of choices.

[4] D1: Individuals work. D2: Loss of flexibility. D3: Time investment. D4:Specialised tools limit functionality. D5:Difficult to analyse all data. D6: Uncountable amounts of calculations and parameters.

D1 The same actions will always be performed when a variant is build: select the wanted structural element and translate it to its correct position. The script is not based on the logic of the individual, except for the basic block.

D2 The translation by component method is partly used to avoid this problem. Because the structural engineer will change the model based on the information of the previous variant.

D3 This tool will require a time investment, which makes this disadvantage still true. However, an increase in quality of the product is expected with the amount of information that is made available of the variants.

D4 User interfaces will severely limit the flexibility of the tool. They are therefore not used.

D5 The most important results are visualised in the Excel sheet. More detailed information can be obtained from the model in RFEM if needed.

D6 Instead of attempting to try every possible combination, the creativity of the structural engineer is used to find a suitable support structure.

The variant study will be performed on a case study to verify whether these attempts were successful.

4
Variant study of the Bluebell Hotel's facade

The parametric model is finished and explained in the previous chapter. Advantages of parametric design were included to the model to try to lessen the difficulties in contemporary structural engineering. This chapter will be used to show how the variant study is performed and its results will be given. This information can be used to determine if the advantages of parametric design came forward and whether disadvantages were mitigated during the design process.

Numerous amounts of variants can be made with the DRC, but it is chosen to do a few in detail. Allowing more advanced capabilities of the connection to be shown. This is done to accurately answer the following sub questions:

1. Explain the results of the variant study. What parameters proved to have the most influence on the support structure? (4.2)

2. What are the recommendations for the support structure of the Bluebell hotel based on this knowledge? (4.3)

4.1 The method and assumptions

The results of the variant study will be shown in this section. First the design process using the DRC will be shown to explain how a viable support structure was found. Then a summary will be given of the assumptions and how the structure is modelled. Finally the most relevant information of the results will be shown.

Two concepts must be explained first, namely **Variants** and **Options**. A **variant** will be defined as a new Dynamo script where the parametric model underwent significant changes. The intention of the script as explained in chapter 3.3 is to be able to generate new variants rapidly.
An **option** is defined as producing models by using the Dynamo script of a variant. For example by using the sliders to change geometry or material properties.

The applied method is schematised in figure 4.1. First a variant is produced manually, which requires changing the Dynamo script. Then an option will be produced by using the sliders in the script. It is not yet known what the optimal cross sections are, therefore an educated guess will be done to try to get as close as possible. The script is then run and the results are visualised in an Excel sheet.
The RFEM modules will optimise the cross sections and show the results. The module does not recalculate the finite elements model with the new cross sections, but uses the internal forces of the original analysis. Therefore the advised cross sections must be input in the Dynamo script to be analysed again. If the resulting optimised cross sections are not the same as the cross sections that were input, then the cross sections are not yet optimised (NO) and the process has to be repeated by inputting the advised cross sections ("Educated guess for cross sections"). If the output contains the same cross sections as were input, then an optimised result is found (YES). Only these results will be shown in the report in tables and graphs. Most options required two or three iterations to produce an optimised option.
This process is not automated. Changing the cross sections and running the script had be done manually for each iteration. Attempts to automate this process with Dynamo Refinery failed because no method was found to do the optimisation part efficiently. Further research is required if this process is to be automated.

The variants will mainly be made for steel structures. This is done to show that it is possible to analyse a structure in depth with this tool. The same actions can be applied with other materials, which will be shown for a single variant in timber and concrete. RFEM modules for timber and steel have been implemented in the DRC, but are not yet available for concrete.
This method is chosen to show that an efficient design can be found with the DRC. It is also done to analyse whether the mentioned advantages came forward and if the disadvantages were lessened.

The goal in the variant study is to search for a support structure which is efficient but also requires the least amount of modifications to the architects design.

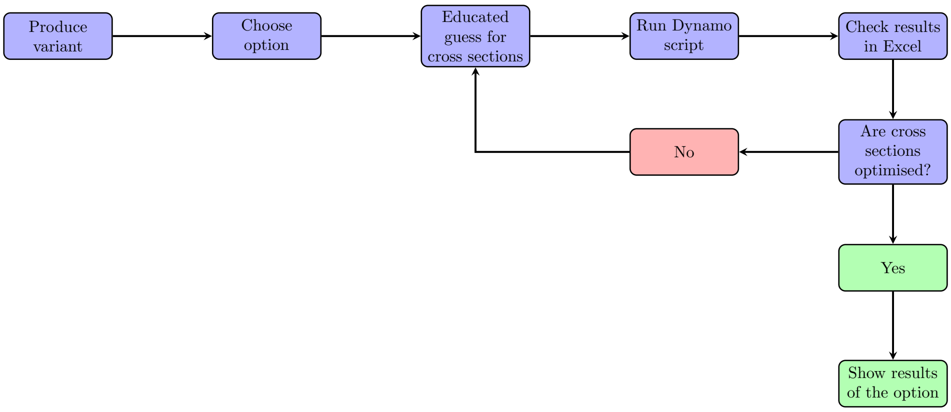


Figure 4.1: The applied method for the variant study

4.1.1 Steel skeleton: information and assumptions

A skeleton structure gives maximal flexibility in the use of floor plan. This quality is beneficial for utility structures because the floor plan can be repurposed if necessary. If the skeleton structure is made out of steel, then the following additional benefits can be expected.
A steel skeleton with bolted connections can rapidly be set up and gives flexibility for modifications. A hinged connection is also cheaper than a clamped connection for steel. This allows the elements to be reused. Mechanical properties include a low self weight and the ability to span large lengths while keeping the cross section small (Spierings, van Amerongen, & Bakker, 1998). Low weight will be important in this structure due to the large cantilever length.
Disadvantages of steel are mainly the sensitivity to corrosion (only in outside environments) and temperature (fire, expansions and cold bridging). Detailing steel connections will require more attention due to the temperature expansion of steel. Finally, additional attention is needed for detailing partitioning walls (Spierings et al., 1998).

Assumptions
Assumptions were made before the design process started, which are listed as follows:

- The loads in figure 4.2 represent the surface loads (G#) or line loads (L#). G1 is the permanent and variable load of the indoor floor plus the permanent load of the finishing layer. G2 is the same, but with a balcony variable load instead of the floor. It also contains the snow load. G3 is the same as the balcony, but is loaded by variable roof loads, snow loads for adjacent roofs and the additional permanent load for the living roof. G4 is the permanent and variable roof load. The line load L1 is the weight of the partitioning walls, L2 represents the load of the partitioning walls on the balcony and L3 the permanent load of the sliding door. More detail is found in appendix B.1.

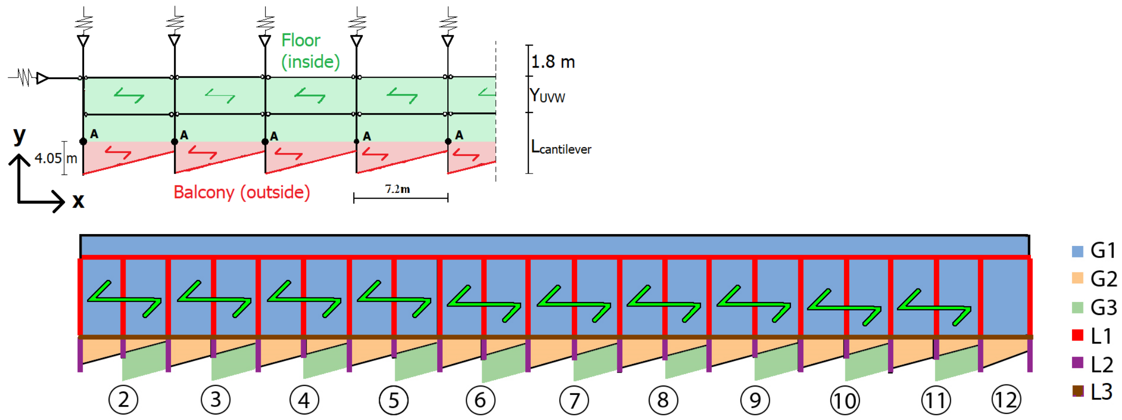


Figure 4.2: Top view of the floor system

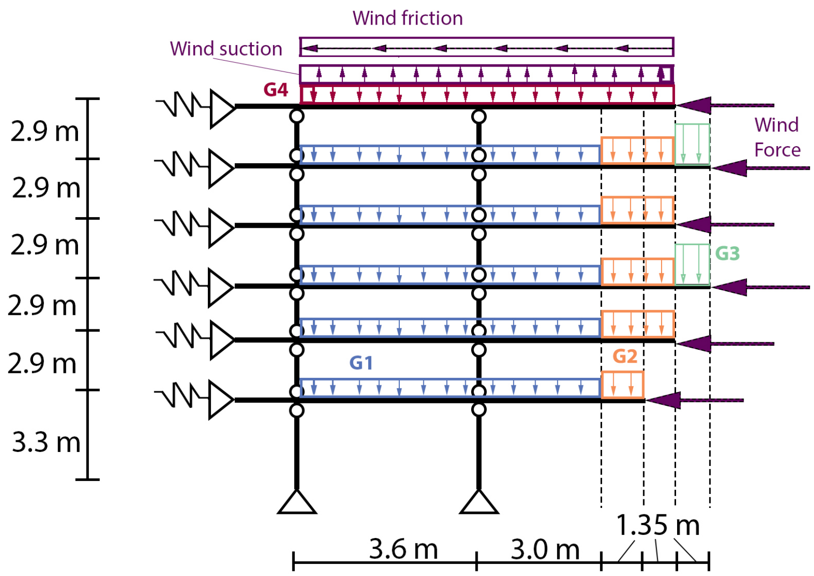


Figure 4.3: Side view of the floor system

- Floors are mainly added to accurately model the loads. The benefit is that loads on the floors and its self weight automatically change with the model. Another option is using an equivalent line load on the beams.

- The floors themselves will not be analysed in detail, because concrete modules are not added to the DRC yet. The floors in this model can still give an indication of potential problems which could occur, these will be described.

- Steel profiles beyond IPE600 and below IPE80 are not typically used in the Netherlands and will not be included. The steel strength will be kept at S235.

- For utility structures with more than 3 floors a braced support structure is advised (van Eekelen et al., 2002). Therefore the columns and beams in x-direction will be hinged. The plates are simply supported and span in one direction. The beams in y-direction have clamped connections, which will be necessary due to the large cantilever length.

- Bending moment and shear force resistance of Isokorbs are taken from tables of the company Schöck. A unity check is done by checking the bending moment at the boundary between inside and outside of the structure (point A in figure 4.2). A unity check larger than one for the Isokorb is not considered as failure. It does indicate that more attention will be required for these connections. A more detailed calculation is shown in appendix C.4.

- Since the floors will not be analysed in detail, a custom node is made which advises the required height of the floor (figure 4.4). How the calculation is performed is shown in appendix B.3. This node could be reused for future projects. The results are shown in figure 4.5 for the roof, balcony and indoor floor.

- The following costs of materials will be assumed: €$2.75/kg$ steel, €$350/m^3$ concrete and no value for timber. How these values were found is described in appendix C.3.

- The critical load combinations will be shown in the results. The numbering is kept the same as shown in chapter 3.1.4.

- The model is verified once for a structure with a floor span of 7.2m and a cantilever of 7.05m in appendix C.1.1. The final part of this verification includes the verification of the critical SLS check as indicated by the module STEEL EC3 for this variant. It shows that it is indeed not possible to build this structure for this cantilever length with only IPE and HEA beams.

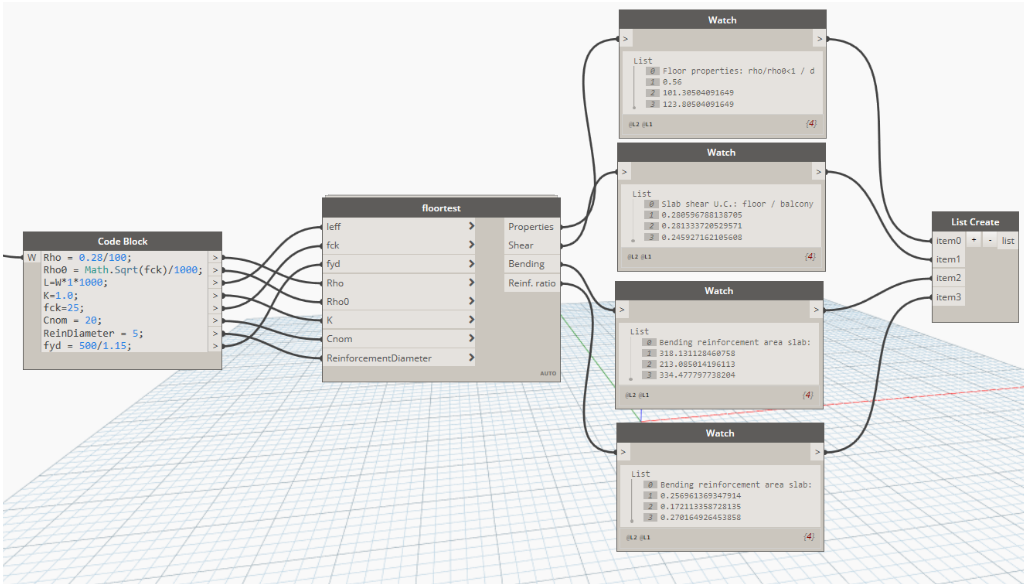


Figure 4.4: Custom node advising the floor height

	Floor span 7.2m			Floor span 3.6m		
	Roof slab	Floor slab	Balcony slab	Roof slab	Floor slab	Balcony slab
Reinforcement ratio: ρ	0.29	0.32	0.33	0.24	0.27	0.29
Reinforcement ratio check: $\rho/\rho_0 < 1$	0.58	0.64	0.66	0.48	0.54	0.58
Slab height reinforcement: d [mm]	213	244	255	80	96	107
Slab height: h [mm]	235	266	277	103	119	129
Minimal shear: U.C. <1	0.48	0.56	0.56	0.25	0.28	0.28
Reinforcement: A_s [mm^2]	666	855	911	236	328	372
Chosen slab height: h [mm]	**235**	**270**	**280**	**105**	**120**	**130**

Figure 4.5: The results from the custom node

Connections in the outer facade

To accurately model the behaviour of the structure in RFEM, several details are shown. The most important ones are the connections of the beams in x-direction and the beams in y-direction. These will be called x-beams and y-beams respectively to shorten their names. With this information the results can be better understood and verified as well.

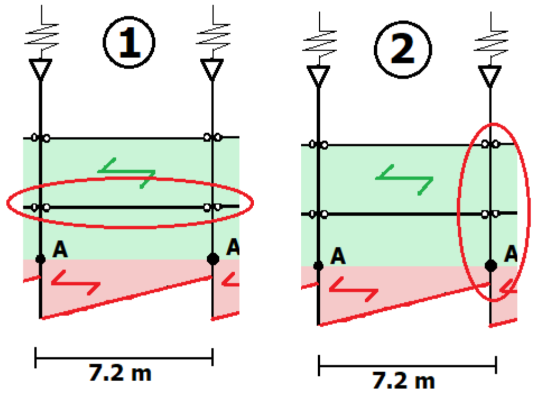


Figure 4.6: The positions of the details

The x-beams are shown in figure 4.6 under ①. The detail of this connection is shown in figure 4.7. The x-beams are used to connect the steel elements to unite the steel structure as a whole. These are mainly needed for stability in x-direction and will not be used to carry the floor, which spans in one direction. Therefore space must be left between these beams and the floor to ensure the model is still accurate. These beams are hinged as explained in the assumptions.

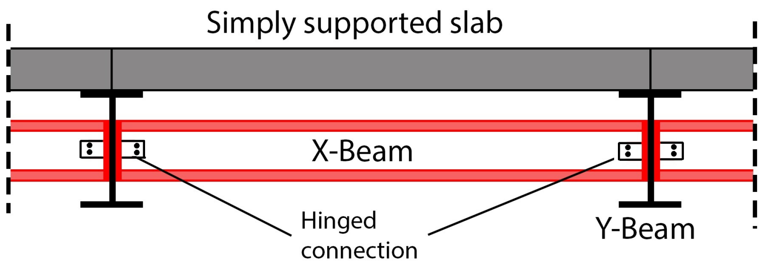


Figure 4.7: Schematisation of the x-beams (not to scale)

The y-beams shown in figure 4.6 under ② are schematised in figure 4.8. It shows that the y-beams have been made continuous to allow resistance to bending moments from the cantilever without having to resort to rigid joints. The columns and the beams in x-direction are both bolted to ensure a hinged connection. The line loads L2 and L3 will act on the indoor slabs and the balcony is disconnected from the indoor slab.
Isokorbs for steel beams are used to prevent cold bridging in the beams. These will be placed in the transitioning of indoor to outdoor (points A in figure 4.6 and 4.2).
To prevent cold bridging through the concrete slab, isolation is applied as intended by the architect (figure 4.9). The steel beams will carry the balconies and its loads to the inside structure. It is possible to use Isokorbs for slabs, but it will be assumed that the Isokorbs for steel beams is enough for this variant study.

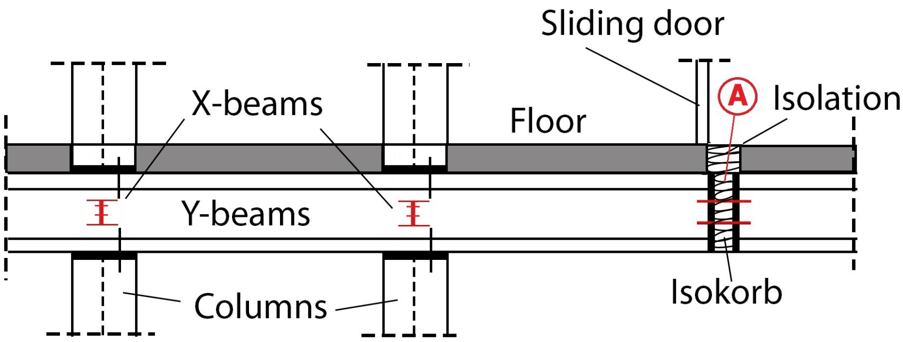


Figure 4.8: Schematisation of the y-beams (not to scale)

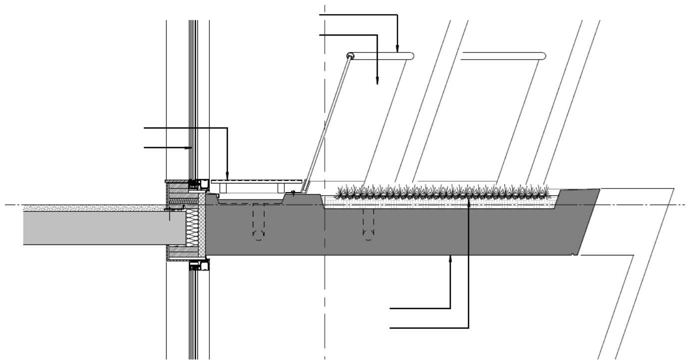


Figure 4.9: Detail of the balcony made by the architect. Amended from: (Urban Climate Architects, 2017b)

Cold bridging at the transitioning from floor or roof slab to balcony slab must also be prevented. This is already done by the architect as shown in figure 4.10. Small alterations will be necessary for an additional steel beam. This has been schematised for the roof and the same concept applies to the first floor. The balcony slab on the roof does not include a green roof.

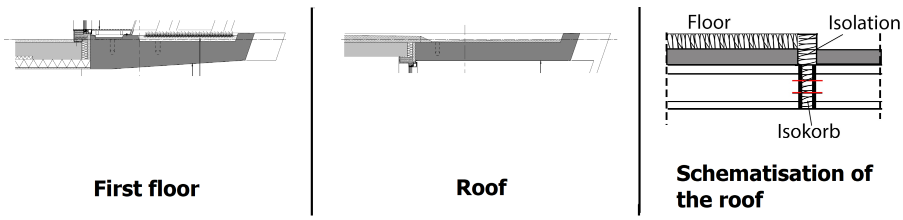


Figure 4.10: The details of the floor and roof. Amended from: (Urban Climate Architects, 2017b)

A steel structure with simply supported slabs spanning one way will be problematic at the first floor. These floors do not have living roofs which results in a sharp edge as visualised in figure 4.11. Even if a floor span of 3.6m is taken. This edge will be problematic, because the forces have to be transferred to the beam over a very small slab area. Therefore the balconies on the first floor are made rectangular.

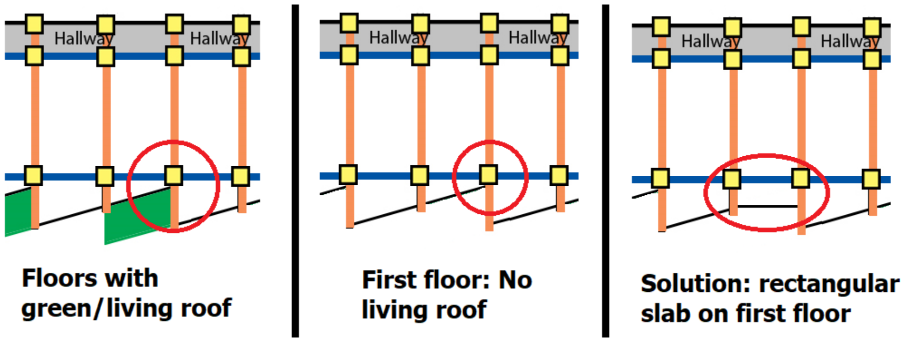


Figure 4.11: Visualisation of the first floor and subsequent higher floors

4.2 The results

The results of the variant study shall be shown in this section. If a solutions is possible for the support structure, then the results will be presented as follows.
The variant shall first be explained and the reason for analysing it will be given. The results interesting for the structural engineer is discussed next. This is done to show what kind of information can be extracted from the variant and how it will influence the choice for the next variant. Finally a conclusion shall be given for the variant where the information important to the architect will be discussed.

4.2.1 Results variants: Architect's design

The design of the architect will be analysed to observe where the main issues lie. Solutions can then be sought based on this information. The floor plan (bottom figure) and the frontal view (top figure) of the structure are schematised in figure 4.12. The structure is separated and numbered per 2 hotel rooms, which are separated by black lines in the frontal view. The placement of the columns is shown in blue.

A floor span of 14.4m is large and therefore IPE600 profiles are chosen for the x-beams and y-beams. The module STEEL EC3 is used to analyse the columns and beams, which is opened in RFEM. The output is visualised in figure 4.13 and shows that almost none of the demands are met for both ULS and SLS.
The problems mainly lie between rooms 9 and 10 where large deflections are observed (figure 4.14). Deflections up to 230mm are found for the serviceability load combinations. The deflections of the cantilevers in the left side of the structure are up to 105mm, which is also too large. Deflections on the right side stay below 30mm though, indicating that a solution may be possible.

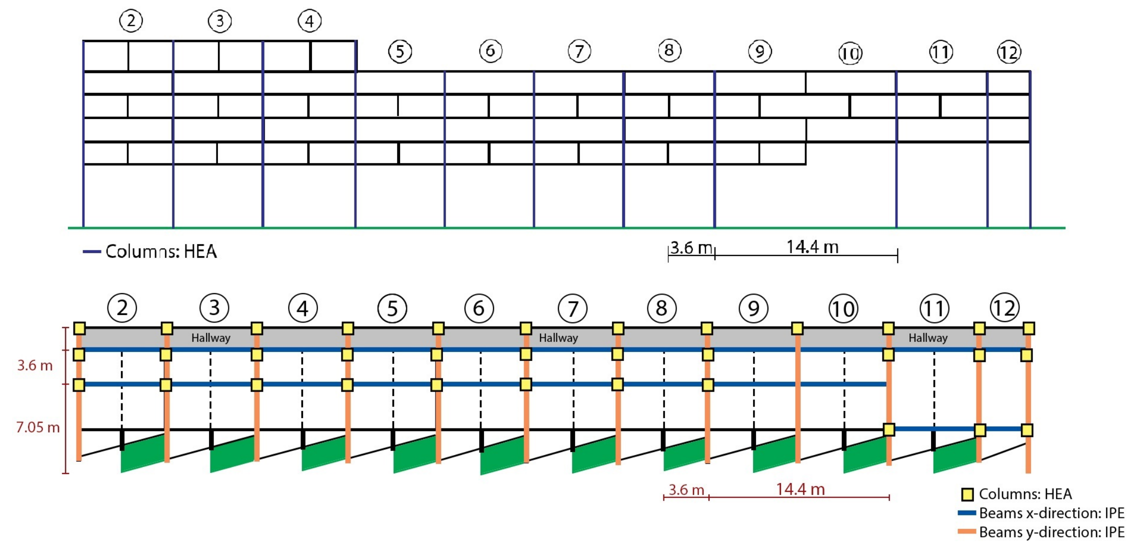


Figure 4.12: Schematisation of the architect's plan.

Load-ing	A Description	B Member No.	C Location x [m]	D Design Ratio	E	F Design According to Formula	G DS
	Ultimate Limit State Design						
CO1	1.35*LC1 + 0.6*LC2	202	0.000	1.71	> 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8	PT
CO2	1.2*LC1 + 1.5*LC2	202	0.000	1.68	> 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8	PT
CO3	1.2*LC1 + 1.5*LC3 + 0.6*LC2	202	0.000	1.50	> 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8	PT
CO4	1.2*LC1 + 1.5*LC4 + 0.6*LC2	202	0.000	1.68	> 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8	PT
CO5	1.2*LC1 + 1.5*LC5 + 0.6*LC2	202	0.000	1.57	> 1	CS221) Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.	PT
CO6	0.8*LC1 + 0.6*LC2	202	0.000	0.92	≤ 1	CS111) Cross-section check - Bending about y-axis acc. to 6.2.5 - Class 1 or 2	PT
CO7	0.9*LC1 + 1.5*LC2	202	0.000	1.19	> 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8	PT
CO8	0.9*LC1 + 1.5*LC3 + 0.6*LC2	202	0.000	1.08	> 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8	PT
CO9	0.9*LC1 + 1.5*LC3 + 0.6*LC2	202	0.000	1.08	> 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8	PT
	Serviceability Limit State Design						
CO11	LC1 + LC2	203	3.000	2.92	> 1	SE411) Serviceability - Combination of actions 'Characteristic' - z-direction, Cant	SC
CO12	LC1 + LC3 + 0.4*LC2	203	3.000	3.71	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO13	LC1 + LC4 + 0.4*LC2	35	2.706	3.98	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO14	LC1 + LC5 + 0.4*LC2	203	3.000	2.69	> 1	SE413) Serviceability - Combination of actions 'Quasi-permanent' - z-direction, C	SQ
CO15	LC1 + 0.5*LC2	203	3.000	3.63	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO16	LC1 + 0.2*LC4 + 0.3*LC2	203	3.000	3.61	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO17	LC1 + 0.2*LC5 + 0.3*LC2	203	3.000	3.53	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO18	LC1 + 0.3*LC2	203	3.000	2.66	> 1	SE413) Serviceability - Combination of actions 'Quasi-permanent' - z-direction, C	SQ

Figure 4.13: The SLS and ULS results.

Conclusion

Almost none of the SLS and ULS demands are met and very large deflections are observed at the 14.4m floor span. The results imply that significant changes to the design are necessary to find a suitable structure. A few proposed solutions are the use of castellated beams at the 14.4m span or to use a hanging structure. The decision is made to not explore these options further, because these solutions are costly.

The deflections on the right side of the structure indicated that a solution might be possible for a steel skeleton structure. The main problem in this structure lies with the 14.4m floor span. A suitable structure may be found if this span is reduced.

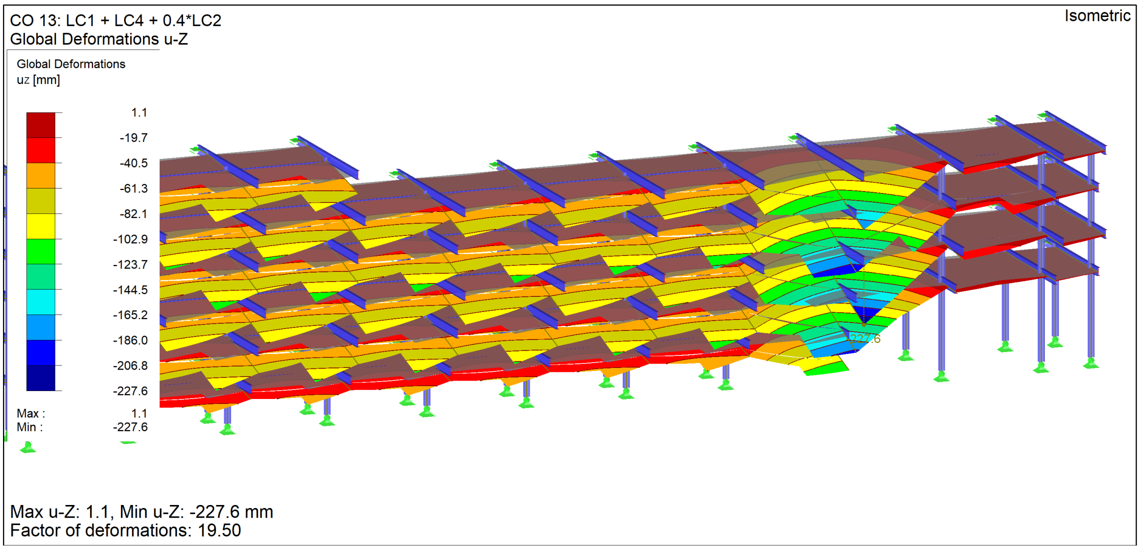


Figure 4.14: The deflections visualised in RFEM.

4.2.2 Results Variant: floor span 10.8m

Additional columns are used for this variant to reduce the floor span as shown in figure 4.15. The facade at the ground floor is positioned at the same location. The columns could therefore be hidden within the facade to preserve the aesthetics.

The results show progress because the ULS demands are met. The SLS demands however are still problematic. In the previous variant low deflections were observed in the right part of the structure with a lower cantilever. A solution could therefore be found if the cantilever is reduced.

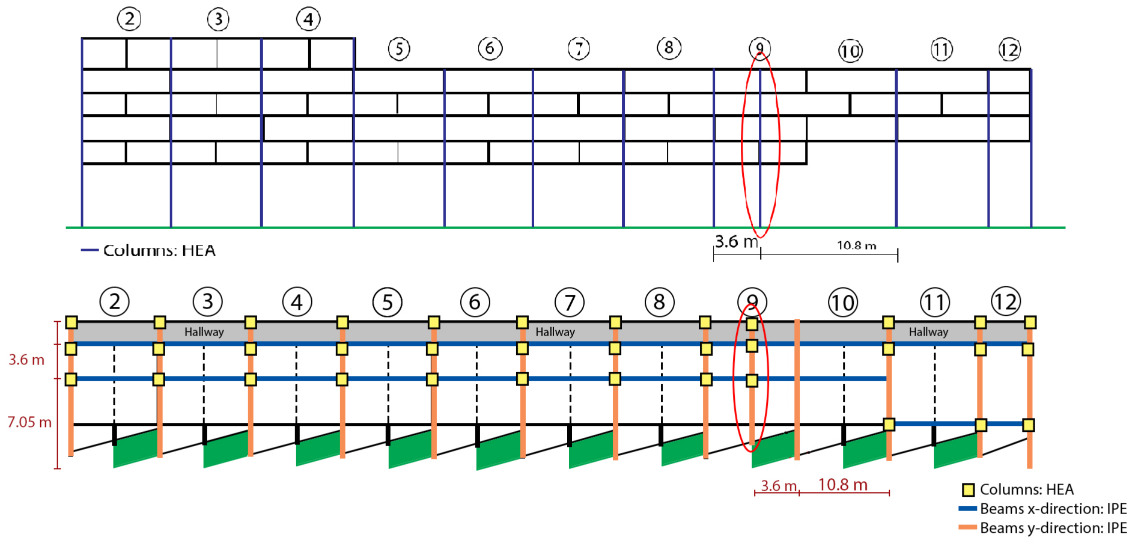


Figure 4.15: Schematisation of Variant: floor span 10.8m.

Loading	A Description	B Member No.	C Location x [m]	D Design Ratio		E	F Design According to Formula	G DS
	Ultimate Limit State Design							
CO1	1.35*LC1 + 0.6*LC2	526	0.000		0.97	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO2	1.2*LC1 + 1.5*LC2	526	0.000		0.95	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO3	1.2*LC1 + 1.5*LC3 + 0.6*LC2	526	0.000		0.91	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO4	1.2*LC1 + 1.5*LC4 + 0.6*LC2	526	0.000		0.95	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO5	1.2*LC1 + 1.5*LC5 + 0.6*LC2	526	0.000		0.84	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO6	0.8*LC1 + 0.6*LC2	526	0.000		0.60	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO7	0.9*LC1 + 1.5*LC2	526	0.000		0.75	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO8	0.9*LC1 + 1.5*LC3 + 0.6*LC2	526	0.000		0.71	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
CO9	0.9*LC1 + 1.5*LC3 + 0.6*LC2	526	0.000		0.71	≤ 1	ST314) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1	PT
	Serviceability Limit State Design							
CO11	LC1 + LC2	203	3.000		2.92	> 1	SE411) Serviceability - Combination of actions 'Characteristic' - z-direction, Cant	SC
CO12	LC1 + LC3 + 0.4*LC2	203	3.000		2.71	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO13	LC1 + LC4 + 0.4*LC2	35	2.706		2.98	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO14	LC1 + LC5 + 0.4*LC2	203	3.000		1.69	> 1	SE413) Serviceability - Combination of actions 'Quasi-permanent' - z-direction, C	SQ
CO15	LC1 + 0.5*LC2	203	3.000		2.63	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO16	LC1 + 0.2*LC4 + 0.3*LC2	203	3.000		2.61	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO17	LC1 + 0.2*LC5 + 0.3*LC2	203	3.000		2.53	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantileve	SF
CO18	LC1 + 0.3*LC2	203	3.000		1.66	> 1	SE413) Serviceability - Combination of actions 'Quasi-permanent' - z-direction, C	SQ

Figure 4.16: The SLS and ULS results.

Option 2: Cantilever span: 4.05 m

The positions of the columns and x-beams are set at the transition between inside and outside environment of the structure (circled in red in figure 4.17). This is true for all floors except the ground floor. A cantilever lower than 7m will result in columns being outside the structure at the ground floor level, which greatly affects the aesthetics of the structure. This options will analyse whether floor spans of 10.8m are a realistic possibility or not.

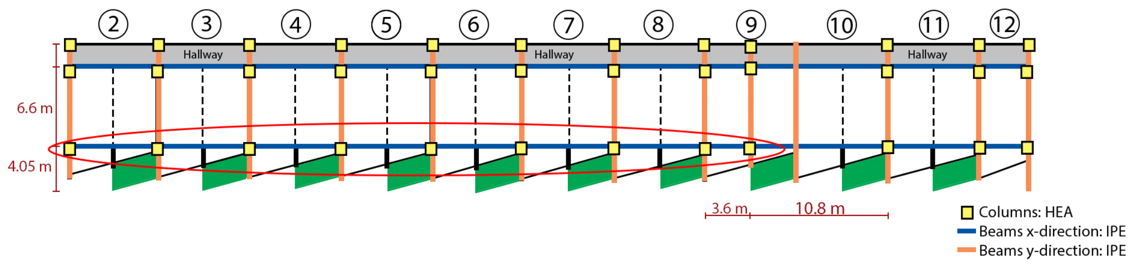


Figure 4.17: Schematisation (cantilever span 4.05 m).

Conclusion

The resulting deformations in figure 4.18 show that the deflections are still too large. The deflection of the beams show positive behaviour and indicate that the SLS demand can be reached if the floor span is kept below 7.2m. The unsupported corners of the balconies deflect considerably however and could become problematic.

The conclusion is that this floor plan could be possible if stiffer beams or a hanging structure is chosen at the 10.8m span. This is considered an expensive solution. It is therefore advised to add a column in the middle of the originally intended 14.4m floor span. The potential of these structures will be further explored.

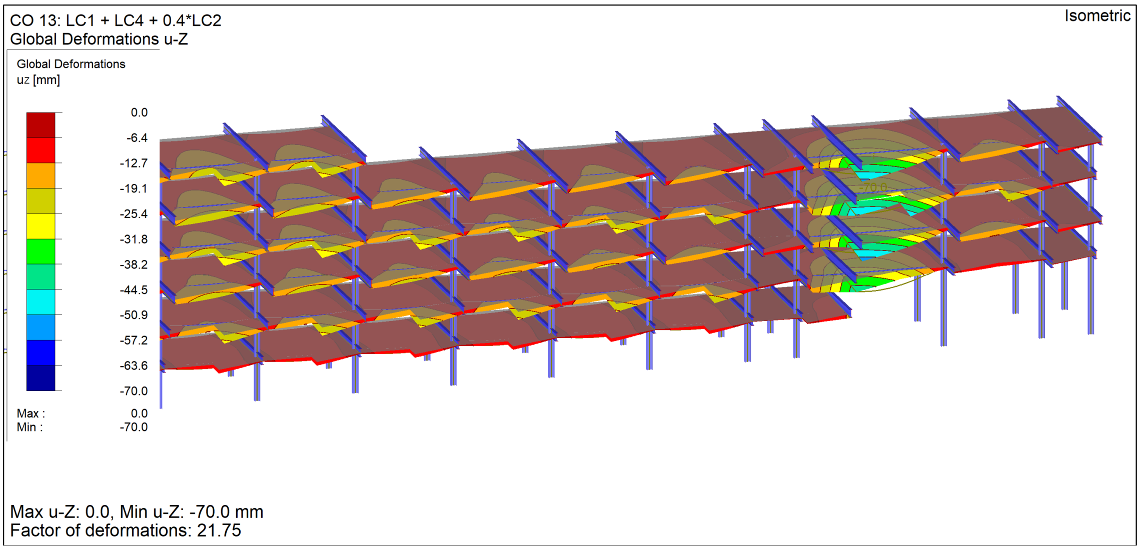


Figure 4.18: The deflections visualised

4.2.3 Results Variant: floor span 7.2m

The previous two variants made it clear that a column will be necessary between rooms 9 and 10 as shown in figure 4.20. The cantilever is set to 4.05m to analyse if a solution exists for this variant.
The most important results of the Excel sheet will be visualised in tables as seen in figure 4.21 and 4.22. The first table includes the maximum value of the ULS and SLS unity checks (U.C.) of each cross section as well as the load combination (CO.) it belongs to. The failure mode is shown as well. The resulting unity check (U.C.) of the Isokorbs for both bending moments (M) and shear (V) are shown in the same table.
The second table shows the deflections of the slabs and the calculated unity checks. The maximum horizontal translation is shown in this table as well and it is checked whether it stays within the demand (total height / 300).

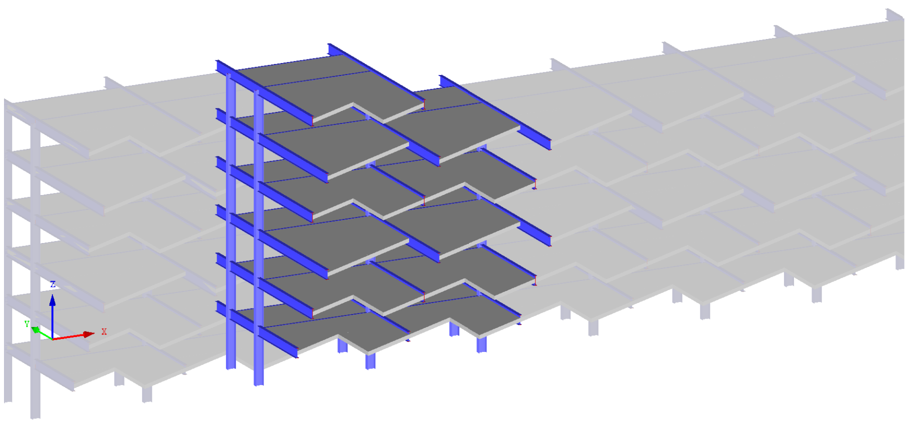


Figure 4.19: Impression of the RFEM model

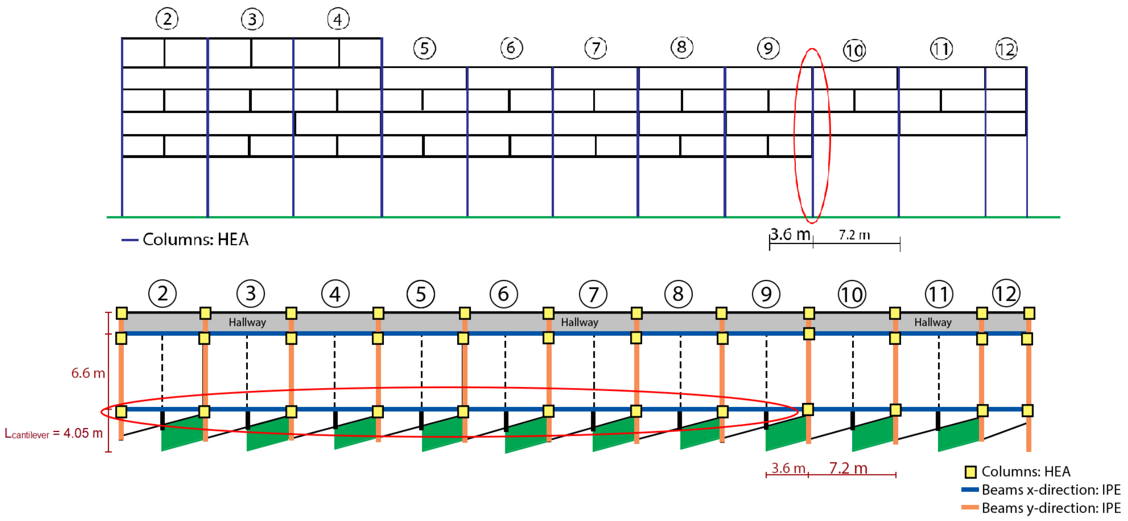


Figure 4.20: Schematisation of Variant: floor span 7.2 m.

Option	Isokorb connection		Structural element	Cross section	Max. ULS			Max. SLS		
	U.C. M	U.C. V			U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.
Option 1: $L_{cantilever}$ = 4.05 m	2,77	6,23	Column	HEA 450	0.92	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 500	0.96	CO4	-Bending and shear - Lateral torsional buckling	0.48	CO16	Frequent z-direction

Figure 4.21: Results: structural elements

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 1: $L_{cantilever}$ = 4.05 m	Characteristic combination	-	-35	1,08
	Frequent combination	-	-29	1,2
	Quasi-permanent combination	-	-28	0,86
	Max. horizontal translation	30		0,85

Figure 4.22: Results: Deflections

Results: Structural elements (figure 4.21)
The ULS and SLS demands are met for this structure, which means that a viable solution is found. The results for the structural elements are explained as follows.

Columns: A minimum cross section HEA 450 is needed for the column. Buckling about z-axis is expected due to the large normal loads on the columns at the ground floor.

X-beams: The lowest possible IPE cross section is found for the x-beams. These are used for lateral loads and to connect the steel structure together. Lateral loads are not included in the variant study, therefore these beams will only be loaded by their self weight.
Combination CO1[1] is critical for ULS because it has the largest self weight multiplier. For the SLS demand, the frequent combination is critical because it has the highest demand of $L_{rep}/333$ instead of $L_{rep}/250$.

Y-beams: The cantilever of the y-beams has become short enough for the SLS demand to not be critical for the beams in y-direction. It was possible to optimize the beams for the ULS demands, which resulted in an IPE500 profile. CO4[2] is critical due to the snow loads on the balconies and the living roof, which load the y-beams on the ends of the cantilever (the most disadvantageous point). For this same reason, CO16[3] is critical for the SLS demands due to its snow loads. This could still be considered a large cross section height, which will increase costs because it will affect the floor height. A detailed explanation is given in appendix C.5.1, which proposes the solution: set the floor between beams.

Results: The Isokorbs (figure 4.21)
The loads on the Isokorbs are far too large for both bending moments and shear forces. Two options are possible, lower the loads on the y-beams on the balcony or use Isokorbs for concrete slabs. Analysing the second options is not possible yet with the DRC, because no node exists which can find loads on specific points for a slab. Therefore this option will not be analysed in detail.
Lowering the loads can be done by shortening the floor span or lowering the self weight of structural elements. The permanent loads can be lowered for example by using hollow core slabs. The calculations in appendix C.5.1 show that the unity checks of the bending moment can be reduced to 1.66 (40% reduction) and 4.93 (21% reduction) for shear. This is not nearly enough and therefore Isokorbs for slabs are advised.

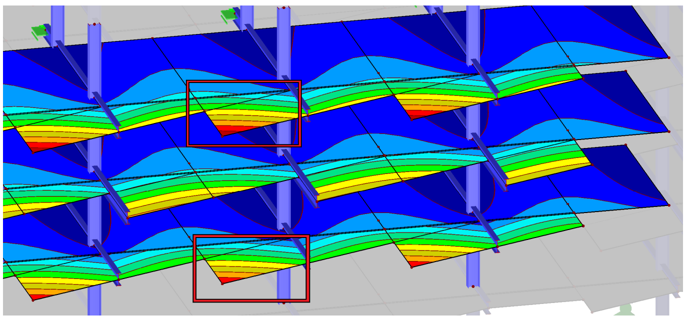


Figure 4.23: Deflection of the slabs

[1] $CO1 = 1.35LC_1 + 0.6LC_2$
[2] $CO4 = 1.2LC_1 + 1.5LC_4 + 0.6LC_2$
[3] $CO16 = LC_1 + 0.2LC_4 + 0.3LC_2$

Results: Maximum deflection (figure 4.22)

The deflections are too large for both characteristic and frequent combinations. The values for displacement upwards were too low (<1 mm) and are therefore not included.

The critical points on the slabs are shown in figure 4.23. It shows that the largest deflections are found at the the corner of the balcony (outlined in red). Every two floors there will be unsupported corners due to the geometry of the structure with a floor span of 7.2m.

Lowering the load on the balcony could significantly impact the deflection at these points. The effect of applying a hollow core slab will be analysed as was done with the Isokorbs in appendix C.5.1. It was calculated that the deflection may be reduced by 10.7mm if the deflection of the y-beams is considered as well. It should be noted that the reduction of the slab displacement is based on a simply supported floor. However, the corner of the balcony is somewhat cantilevered.

The allowable deflection for the frequent combination is 24mm. Even if the calculated effect is reduced by 50%, using hollow core slabs would reduce the maximum deflection to 29-10.7/2 = 23.7mm, which is within the SLS demand with a unity check of 0.99. Indicating that a viable solution for the floor is possible.

Conclusion

A structure is found which fulfils the demands for the steel elements. Though an IPE500 beam could still be considered a large cross section, because the height of the building will have to increase.

The unity check for the Isokorbs is too large for this structure and using hollow core slabs did not reduce the unity check enough. Therefore more actions will be required to prevent cold bridging. The loads will have to be lowered on the y-beams in some other manner or Isokorbs will have to be used for concrete slabs.

The floors will have to be analysed in more detail to ensure that the corners do not deflect too much. One option is using hollow core slabs to replace the currently used massive slabs. An additional solution could be to apply a larger cross section for the y-beams. The increase in stiffness will lower the deflection of the y-beams and therefore the floors as well. The trade off is that a larger cross section is required for the beams. For this option this is disadvantageous, because an IPE500 is already considered a large cross section.

Options: Varying the cantilever

The previous option showed that it is possible to use a steel structure for the outer facade. The difficulties were mainly in the Isokorbs and the corners of the balcony slabs. For both these difficulties a solution was given.

If the architect wants a cantilever for aesthetics, then it would be interesting to analyse how large the cantilever can become and how it influences the structure. This is done in increments of 0.5 m for each option. Only the positioning of the columns and the x-beams will be varied for these options (the value $L_{cantilever}$).

It should be noted that placing columns between the rooms will influence its flexibility in floor plan. The columns will be in the way if certain rooms are combined or widened.

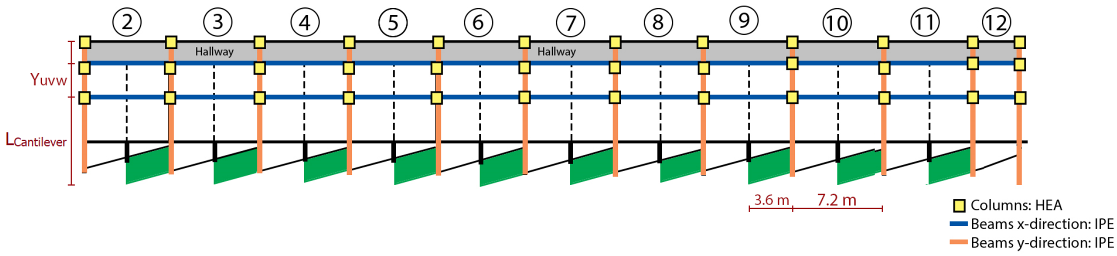


Figure 4.24: Schematisation (varying L_{span}).

Option	Isokorb connection		Structural element	Cross section	Max. ULS			Max. SLS		
	U.C. M	U.C. V			U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.
Option 2: $L_{cantilever}$ = 4.55 m	2,44	3,63	Column	HEA 450	0.97	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 500	0.82	CO4	-Bending and shear - Lateral torsional buckling	0.84	CO16	Frequent z-direction
Option 3: $L_{cantilever}$ = 5.05 m	2,45	3,38	Column	HEA 500	0.98	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 550	0.66	CO4	- Bending and shear - Lateral torsional buckling	0.99	CO16	Frequent z-direction
Option 4: $L_{cantilever}$ = 5.55 m	2,46	3,19	Column	HEA 550	1.01	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 600	0.69	CO4	- Bending and shear - Lateral torsional buckling	1.12	CO16	Frequent z-direction

Figure 4.25: Results: structural elements

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 2: $L_{cantilever}$ = 4.55 m	Characteristic combination	-	-47	1,28
	Frequent combination	-	-40	1,45
	Quasi-permanent combination	-	-38	1,04
	Max. horizontal translation	30		0,85
Option 3: $L_{cantilever}$ = 5.05 m	Characteristic combination	-	-53	1,31
	Frequent combination	-	-46	1,51
	Quasi-permanent combination	-	-44	1,09
	Max. horizontal translation	30		0,85
Option 4: $L_{cantilever}$ = 5.55 m	Characteristic combination	-	-60	1,34
	Frequent combination	-	-52	1,56
	Quasi-permanent combination	-	-50	1,13
	Max. horizontal translation	30		0,85

Figure 4.26: Results: Deflection and Isokorb connection

Results: Structural elements (figure 4.25)

Y-beams: Three additional options were analysed as seen in figure 4.25. Increasing the cantilever by half a meter had no impact on the y-beams cross section. It stayed an IPE500 from option 1 to 2.
For the subsequent options, the y-beams increase to an IPE600 as expected due to the larger cantilever. This is because the SLS demand has become critical instead of the ULS demand from option 2 onward. CO4[4] is critical due to the large snow loads on the ends of these beams, which cause large bending moments and shear forces.

Columns: The resulting additional weight from larger y-beams caused the height of the columns to increase as well. Increasing the cantilever span also causes a larger portion of the floor weight to act on the columns as seen in figure 4.27. This weight increase is significant enough that larger cross sections are needed for the columns.

X-beams: The x-beams stay constant for all options as expected. Both the ULS and SLS checks do not vary because the loading stays the same on the x-beams. CO1 is also still critical as expected.

Results: Isokorb checks (figure 4.25)

The unity checks for bending moments vary slightly. This is expected, because an increased cantilever should not increase the bending moments at the same point in a rigidly supported cantilever (figure 4.28). the bending moments are still too large as was the case for options 1, therefore alternative solutions have to be found. The shear forces should vary slightly as well. Instead, the shear forces decrease for larger cantilever spans. Appendix C.5.1 explains that this happens due to the stiffness of the column.

[4] $CO4 = 1.2LC_1 + 1.5LC_4 + 0.6LC_2$

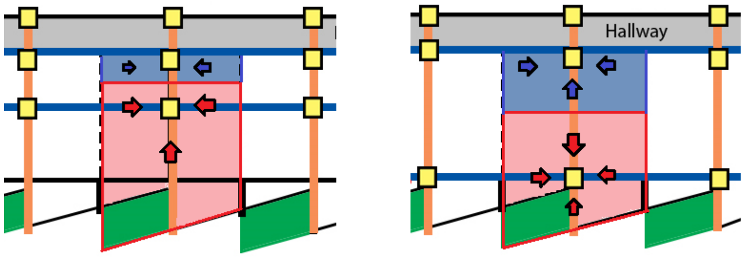


Figure 4.27: Schematising the loads of the floors acting on the columns

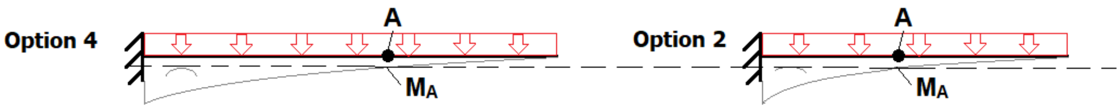


Figure 4.28: The bending moments for different cantilever spans

Results: Maximum deflection (figure 4.26)
The deflections of the slabs increase when the cantilever is increased. This happens because the SLS unity check increases as well, which means that the y-beams deflect more for each subsequent option. The slabs are simply supported on these beams and will therefore have an additional deflection as well. it is a coincidence that the SLS unity check increases for increasing cantilevers.
There were already difficulties with the floors in option 1 and the issue increases for larger cantilever spans. Other solutions besides a hollow core slab will have to be sought if larger cantilevers are wanted.

Conclusion
It is possible to increase the cantilever span for this variant, but there will be issues with the corner of the balcony and the Isokorbs. A hollow core slab will probably not be enough to mitigate these issues for larger cantilever spans and different solutions will have to be sought. The height of the structure will also have to be increased if floors are set on top of beams.
The final results are visualised with graphs. Figure 4.29 shows the profile number for each option within the ULS and SLS demands for the steel elements. For this variant three structures are possible, though large IPE profiles are needed and solutions will have to be found for the floor and Isokorbs. Therefore only a cantilever of 4m is realistic with hollow core slabs and Isokorbs for concrete slabs.

The costs of the structures have been calculated as seen in figure 4.30. The costs vary slightly and are therefore shown relative to option 1 (cantilever 4.05 m) in figure 4.31. A positive percentage should be interpreted as a cost increase and a negative percentage as a cost decrease. It shows that the total costs of the support structure will increase by 6% from lowest to largest cantilever.

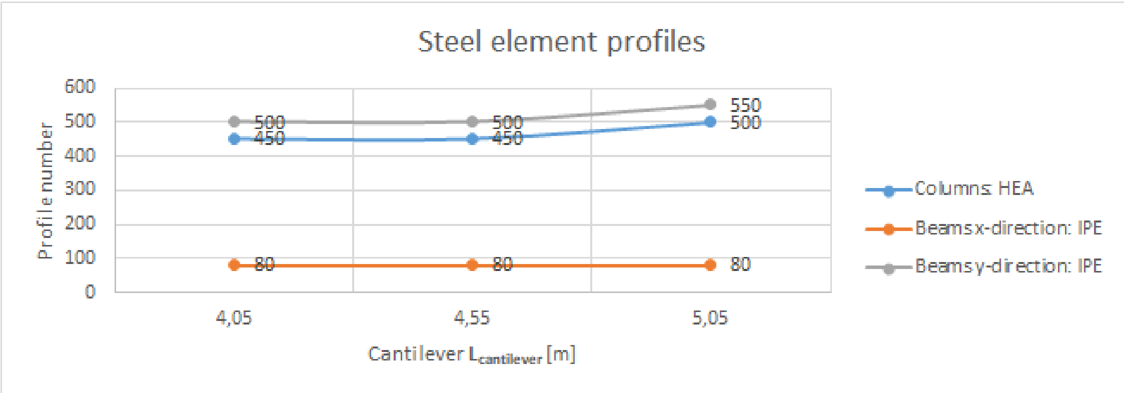


Figure 4.29: Profile heights for varying cantilever ($L_{cantilever}$)

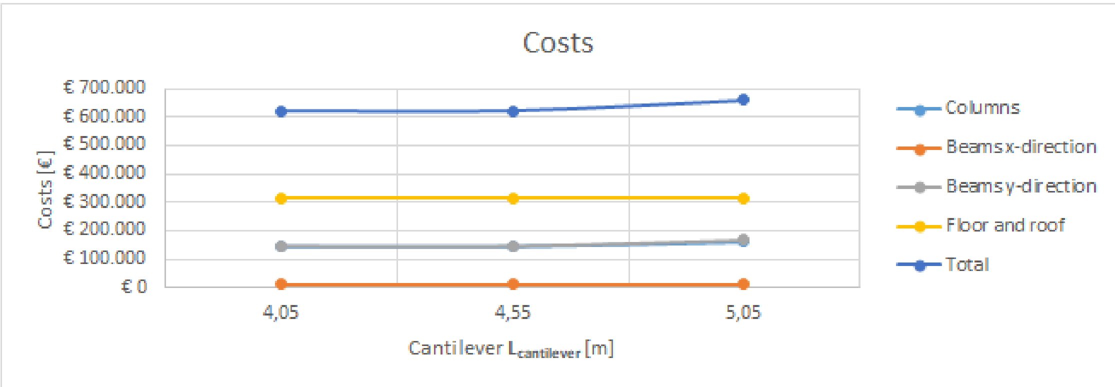


Figure 4.30: Costs for varying cantilever ($L_{cantilever}$)

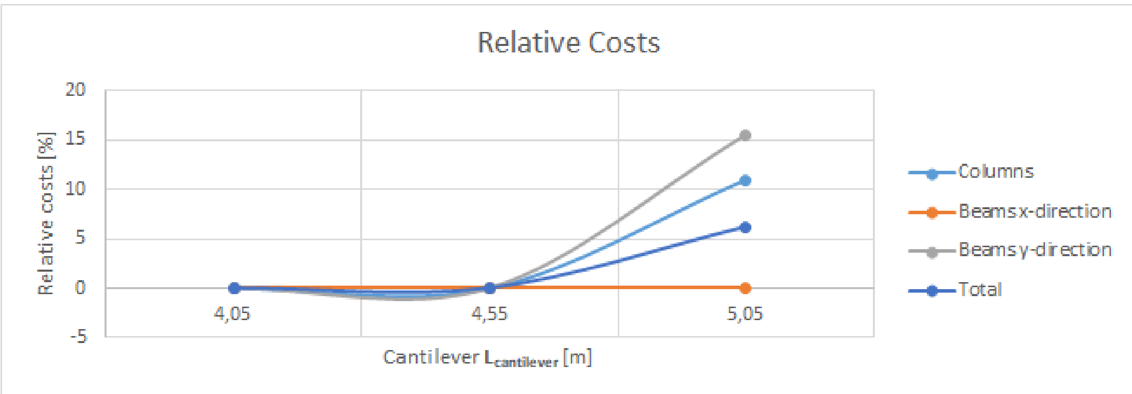


Figure 4.31: Relative Costs for varying cantilever ($L_{cantilever}$) compared to $L_{cantilever} = 4.05m$

Options: Varying the angle over height ϕ
A request was made to analyse the effects of varying ϕ on the support structure. The cantilever is set back to 4.05 m (option 1), because it was the most viable structure. Only parameter ϕ shall be varied during this analysis. Chapter 3.1.2 explained that it mainly influences the length of the hotel rooms, thus the cantilever increases when ϕ is increased. The expected results are that the profile height for beams in y-direction should increase when ϕ increases.
A simplified table is given for these options to visualise the large amount of data. Therefore only the critical beams are given, which are the beams in y-direction for this analysis. The full results can be viewed in appendix C.6.1.

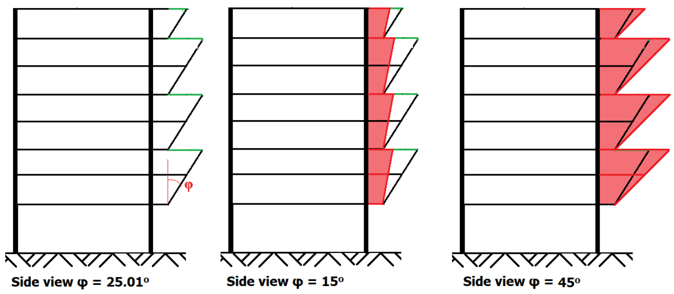


Figure 4.32: Side view of the structure for varying phi

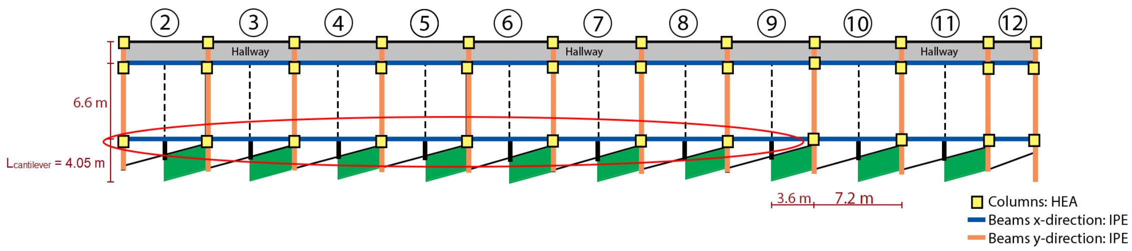


Figure 4.33: Schematisation (cantilever span 4.05 m).

Results: Structural elements
Y-beams: The beams in y-direction behave as expected. A larger angle ϕ gives an increased cross section height. Combination CO4 is still the critical load combination as was the case when the column position was varied. The bending moments and shear forces caused by the these loads are critical for this analysis which explains why lateral torsional buckling and shear and bending are critical demands.
Mechanical behaviour:
• The critical combinations for the SLS demand change between options 1 to 3 and options 4 to 6.
• Both CO13[5] and CO16[6] are loaded by additional roof and variable loads and the

[5] $CO13 = LC1 + LC4 + 0.4LC2$
[6] $CO16 = LC1 + 0.2LC4 + 0.3LC2$

unity check of both combinations were very close. Both frequent and characteristic combinations were critical, but the frequent demand becomes slightly more critical for larger cantilever spans.
• Combination CO15[7] is critical for option 1 because of the short cantilever. The roof loads at the ends of the beams are therefore less effective.

Option	Critical Cross section	Max. ULS			Max. SLS			Isokorb connection		Max. slab Displacement U.C.
		U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.	U.C. M	U.C. V	
Option 1: ϕ = 15°	IPE 360	0.75	CO4	- Lateral torsional buckling	0.30	CO15	Frequent z-direction	0,8	3,64	1,23 (Fr.)
Option 2: ϕ = 18.7°	IPE 400	0.90	CO4	-Shear and bending - Lateral torsional buckling	0.22	CO13	Characteristic z-direction	1,51	4,19	0.97 (Fr.)
Option 3: ϕ = 22.5°	IPE 500	0.76	CO4	- Lateral torsional buckling	0.33	CO13	Characteristic z-direction	2,18	5,08	1.12 (Fr.)
Option 4: ϕ = 26.2°	IPE 550	0.84	CO4	- Lateral torsional buckling	0.50	CO16	Frequent z-direction	3,04	6,31	1,14 (Fr.)
Option 5: ϕ = 30°	IPE 600	0.88	CO4	- Shear and bending - Lateral torsional buckling	0.64	CO16	Frequent z-direction	4,12	7,58	1,27 (Fr.)
Option 6: ϕ = 33.7°	IPE 600	1.18	CO4	- Bending and shear force - Lateral torsional buckling	0.77	CO16	Frequent z-direction	-	-	-

Figure 4.34: Results: varying the angle over height ϕ

Results: Isokorb checks

The unity checks of the Isokorbs increase incrementally for larger values of ϕ. This can be explained if the beams in y-direction are simplified as a rigidly supported cantilever beam. Increasing ϕ increases the length of the y-beams, which will produce larger bending moments and shear forces at the transition between the inside and outside environment of the support structure. Thus both unity checks increase for increasing ϕ.

Noticeable is that none of the options are viable, therefore it is still necessary to find a solution to prevent cold bridging (e.g. Isokorbs for slabs). This is also the case if hollow core slabs are used to reduce the loads (as calculated for option 1 for varying column positions).

[7] $CO15 = LC1 + 0.5LC2$

Results: Maximum deflection

The deflections of the slab increase for larger values of ϕ, because the length of the y-beams increase. For this analysis the lowest possible unity checks for the steel elements was sought after without considering floor deflections. The deflections of the slabs can therefore be decreased by increasing the cross section of the y-beams.

Using hollow core slabs could be applied as well. By assuming a 10.7mm decrease of slab deflection, steel profiles can be kept the same and the unity checks stay below 1 (using appendix C.6.1).

Conclusion

The support structure behaves as expected when the angle over height ϕ is increased and a maximum angle of 30^o is possible for this variant. However, options 3 to 5 require a large cross section, which will increase the floor height. Option 1 is the best solution for efficiency, but it comes at a cost of aesthetics.

Figure 4.37 shows the relative costs compared to option $\phi = 25^o$, which is the angle chosen by the architect. It shows that it is possible to decrease the total costs of the structure by 22% if an angle of 15^o is chosen. Relative costs for options below $\phi = 25^o$ are below 0, which indicates that the analysis went well.

It was chosen to not go below 15^o because much of the geometry of this structure is dependent on the angle ϕ. A lower value will affect the functionality and aesthetics of the structure too significantly.

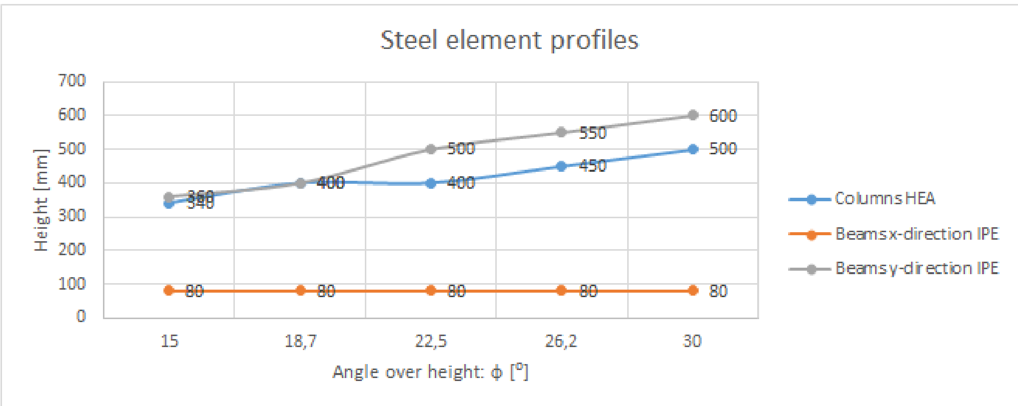


Figure 4.35: Cross section profiles for varying angle over height (ϕ)

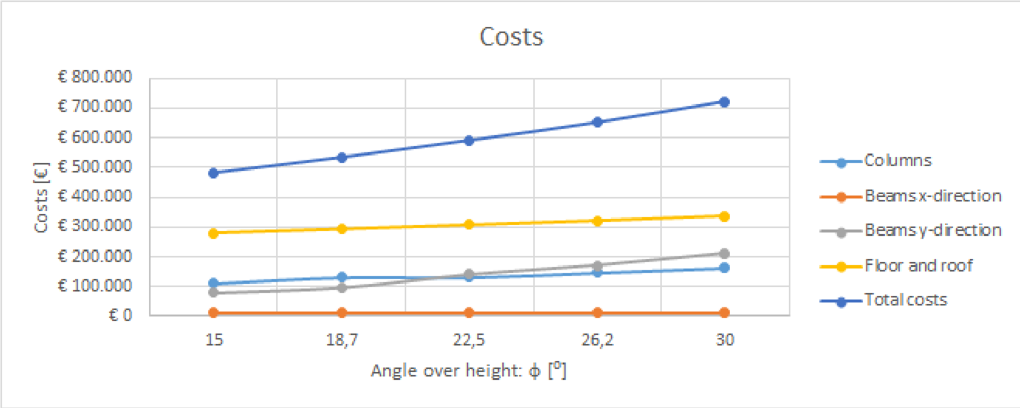


Figure 4.36: Costs for varying angle over height (ϕ)

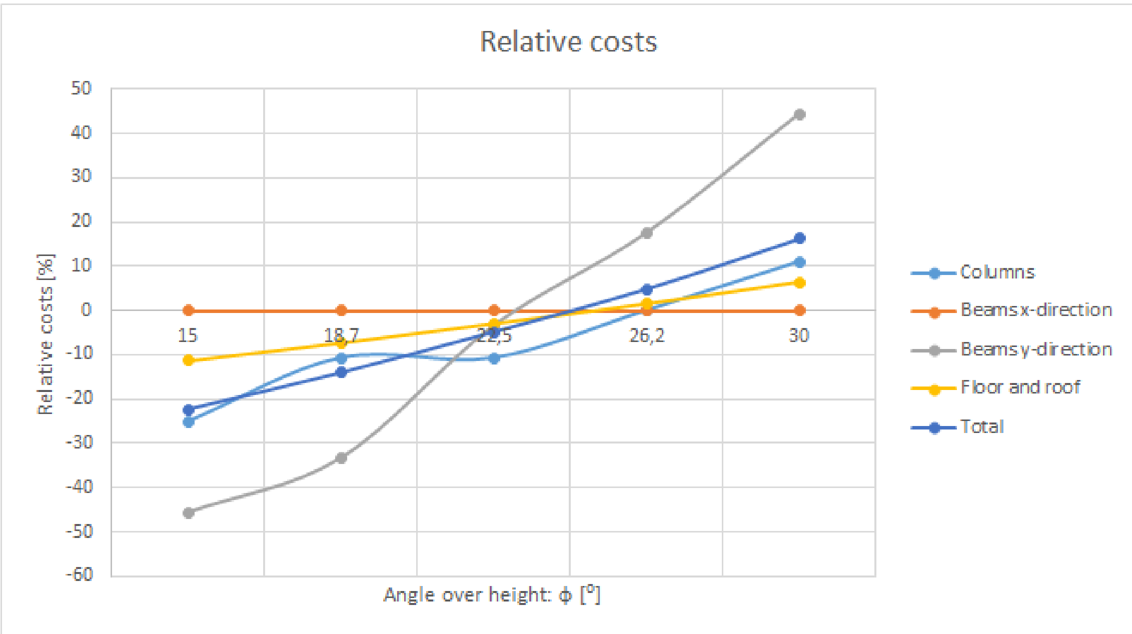


Figure 4.37: Relative costs for varying angle over height compared to $\phi = 25^o$

4.2.4 Results Variant: floor span 3.6m

The previous section showed that only limited amount of options were available for a 7.2m floor span. It was mostly the beams in y-direction that were critical. Two solutions are possible: increase the strength of these beams by using larger profiles (e.g. castellated beams) or reduce the loads on the y-beams. This variant is made to analyse how the structure behaves when the loads are significantly reduced by reducing the floor span by half.
Labour costs will increase significantly compared to the 7.2m floor span, but this analysis will show if this increase can be partly mitigated by a reduction of cross sections of both steel and concrete elements.
Another added benefit of this variant is that there will be no unsupported corners for the balcony slabs. Thus the expectation is that the floor deflection will stay within the SLS demand.

An analysis will be done for this structure for varying column positions. This is done to investigate whether a larger cantilever is possible and to answer how large it can become. If a cantilever of 7.05m can be reached as wanted by the architect, then a successful structure has been found.

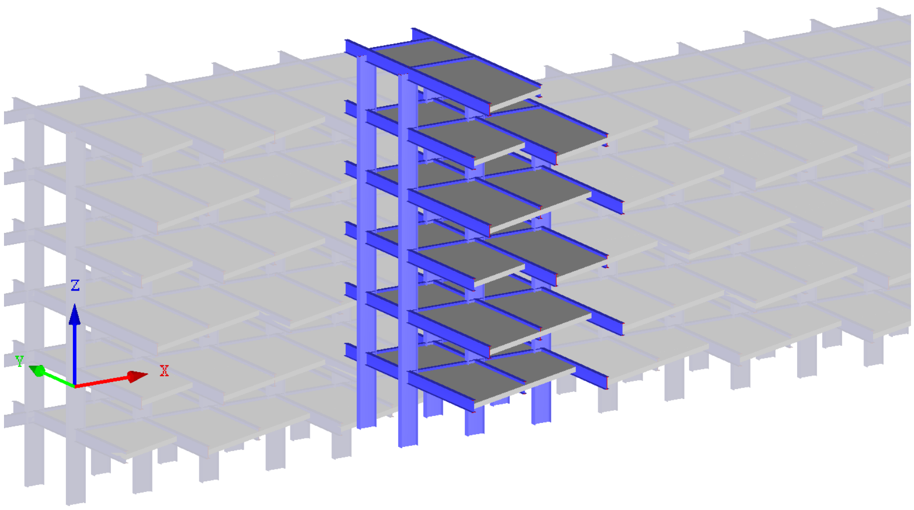


Figure 4.38: Impression of the RFEM model

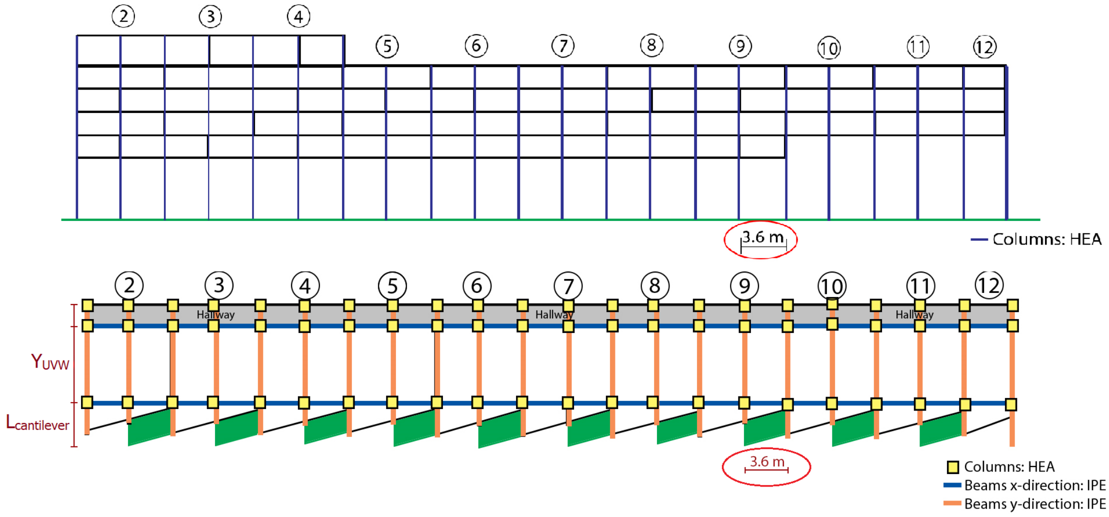


Figure 4.39: Schematisation of Variant: floor span 3.6 m.

Option	Critical Cross section	Max. ULS U.C.	Max. ULS CO.	Max. ULS Design according to EC.	Max. SLS U.C	Max. SLS CO.	Max. SLS Design according to EC.	Isokorb connection U.C. M	Isokorb connection U.C. V	Max. slab Displacement U.C.
Option 1: $L_{cantilever}$ = 4.05 m	IPE 330	0.95	CO2	- Lateral torsional buckling	0.75	CO13	Characteristic z-direction	0,45	1,23	0.83 (Char.)
Option 2: $L_{cantilever}$ = 4.55 m	IPE 400	0.59	CO4	- Bending and shear - Lateral torsional buckling	0.80	CO13	Characteristic z-direction	0,44	1,05	0.91 (Char.)
Option 3: $L_{cantilever}$ = 5.05 m	IPE 450	0.59	CO4	- Bending and shear - Lateral torsional buckling	0.94	CO16	Frequent z-direction	0,44	1,02	1.06 (Fr.)
Option 4: $L_{cantilever}$ = 5.55 m	IPE 550	0.49	CO4	- Bending and shear - Lateral torsional buckling	0.82	CO16	Frequent z-direction	0,45	1	0.92 (Fr.)
Option 5: $L_{cantilever}$ = 6.05 m	IPE 600	0.48	CO4	- Bending and shear - Lateral torsional buckling	0.87	CO16	Frequent z-direction	0,45	1	0.97 (Fr.)
Option 6: $L_{cantilever}$ = 6.55 m	IPE 600	-	-	-	1.07	CO16	Frequent z-direction	-	-	-

Figure 4.40: Results: varying the cantilever span $L_{cantilever}$

Results: Structural elements

Y-beams: The expected critical beams are the y-beams and the results are shown in figure 4.40. The elaborate results are found in appendix C.6.2. Reducing the loads on the y-beams by decreasing the floor span resulted in additional two more options compared to the previous variant.

Mechanical behaviour:
• The cross sections of the y-beams increase when the cantilever increases as expected.
• The SLS demand is critical for all options except for option 1, because the cantilever for this option is not large enough to cause large deflections.
• Therefore a low cross section can be used for option 1 which will be critical for the ULS demand. This happens for combination CO2[8] because large permanent and variable loads are present.
• The snow loads are still critical for the SLS demands though indicated by CO13[9]. The larger loads of the characteristic combinations result in larger deflections than the frequent combination for lower cantilever lengths.
• From options 3 onward the SLS demand for the frequent combination becomes critical. The unity check difference between the characteristic and frequent increased for options 3 to 5 from 0.02 to 0.07.
• Therefore both combinations are influential, but the frequent combination becomes more critical the larger the cantilever becomes due to its larger SLS demand.

Results: Isokorb checks
The unity checks for the bending moments stay constant as expected from the previous variant. The unity check for bending moments is far below the demand and should therefore not pose problems for this design. The shear forces lower a lot less than the previous variant when the cantilever is increased, because the loads on the beams have lowered significantly.
The unity check for the shear forces are not within the demands, but additional actions could lower it to acceptable levels. Hollow core slabs could be used for example which significantly decreased the shear force unity check in the previous variant. Another option is contacting the company that makes the Isokorbs, because they indicated that larger values are potentially possible. A unity check of 1.23 implies that the shear force resistance of the Isokorb must increase to $72 * 1.23 = 88.6kN$.

Results: Maximum deflection
The maximum deflections have lowered significantly compared to the previous variant, because the corners in the balcony are now supported by y-beams. Therefore, if the y-beams are within the SLS demand, then there will be a solution for the balcony slabs. This is true because the additional deflection of the slab will be low due the the lower floor span of this variant.
This is the case for all options except for option 3, because the SLS unity check of the y-beams (0.94) is too close to 1. For this option a hollow core slab could be used or the cross section of the y-beams could be increased.

[8] $CO2 = 1.2LC1 + 1.5LC2$
[9] $CO13 = LC1 + LC4 + 0.4LC2$

Conclusion

Lowering the floor span from 7.2m to 3.6m had significant positive effects on the structure. The cross sections of the columns and y-beams lowered, the floor slabs mostly stay within the SLS demand and using Isokorbs for steel beams becomes a realistic option. The cantilever could also be increased by one meter for this variant. Figure 4.41 shows the optimised cross section per cantilever span.

Even though the cross sections of both the floors and the steel elements decreased, the amount of steel elements almost doubled. Therefore the total costs of the previous variant have been included in figure 4.42 to analyse how it impacts the costs. The costs for options 1 decreased by approximately 33% and 25% for options 2 and 3 respectively.

Not only did the material costs decrease, but lower cross sections are also possible for variant: floor span 3.6m. This would lower costs because the height of the structure could be decreased if wanted. Though one should consider that labour costs will increase significantly because the amount of structural elements almost doubled.

As expected the relative costs in figure 4.43 show that increasing cantilevers impacts the beams in y-direction the most, where the price increases by 149%. This is not 100% (doubled costs for doubled height) because the costs are calculated by weight and not by height of the IPE profile. The total material costs increase by 56% if the largest cantilever span is chosen instead of the lowest one.

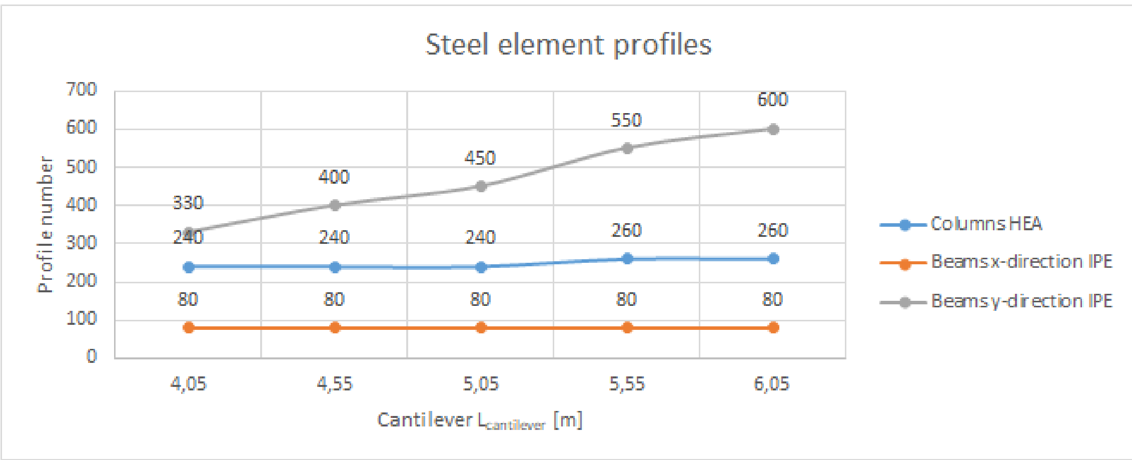


Figure 4.41: Profile heights for varying cantilever span

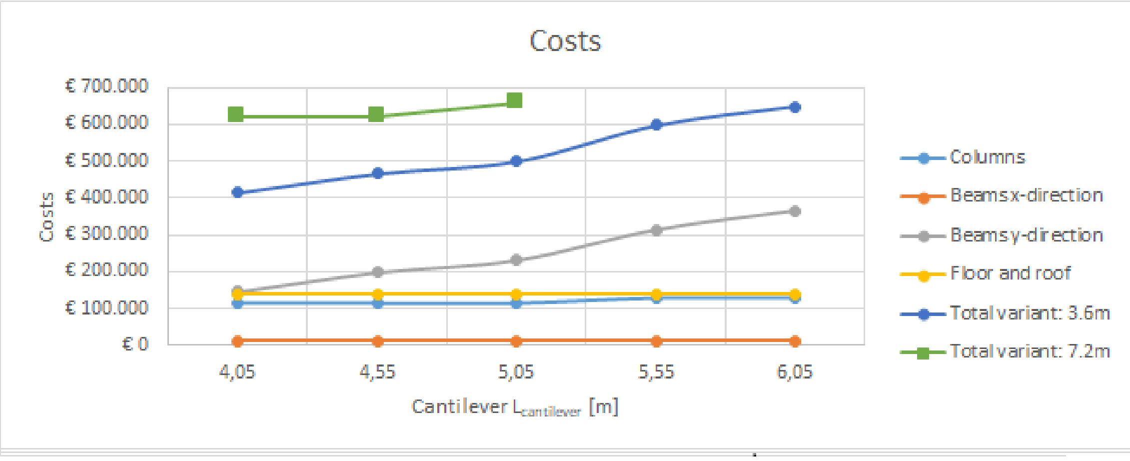


Figure 4.42: Costs for varying cantilever span (L_{span})

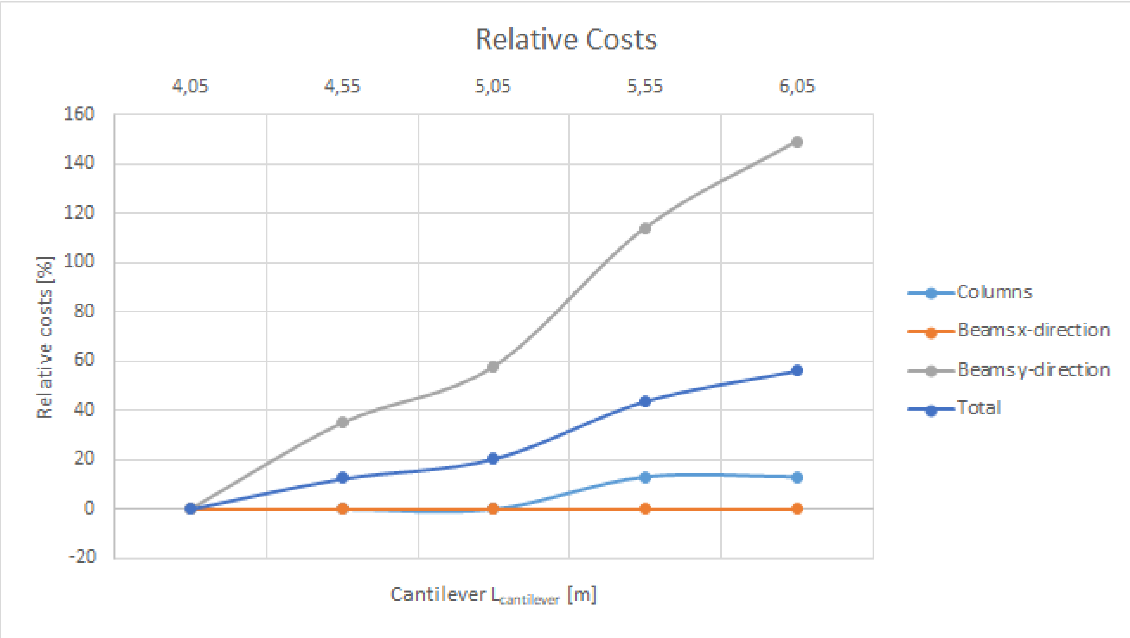


Figure 4.43: Relative costs relative to $L_{span} = 4.05m$

Options: Varying the angle over height ϕ

If it is decided that a floor span of 3.6m is more desirable, then it would also be convenient if the behaviour of varying ϕ is known. This will be analysed in this section and the same behaviour as was seen for the previous variant is expected: increasing ϕ should increase the cross sections. The results are shown in figure 4.46 and the full results are found in appendix C.6.3

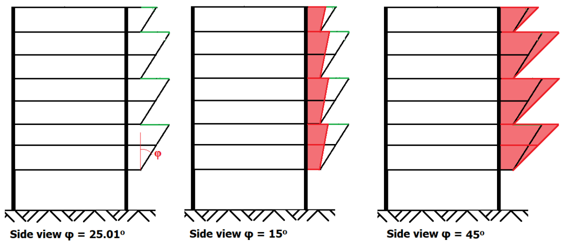


Figure 4.44: Side view of the structure for varying phi

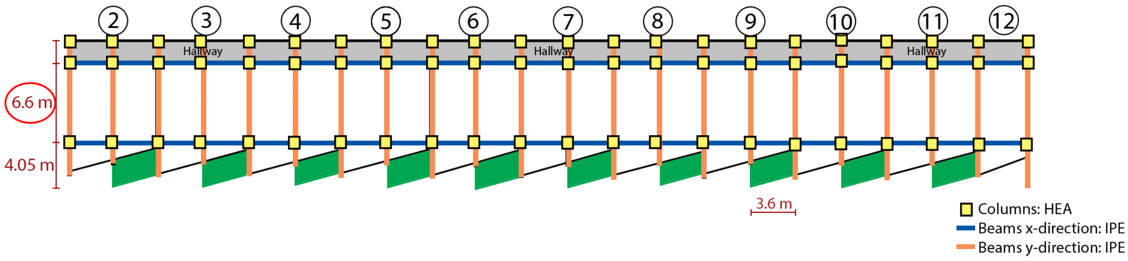


Figure 4.45: Schematisation of the floor plan (varying ϕ)

Option	Critical Cross section	Max. ULS			Max. SLS			Isokorb connection		Max. slab Displacement U.C.
		U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.	U.C. M	U.C. V	
Option 1: ϕ = 15°	IPE 330	0.98	CO2	- Lateral torsional buckling	0.70	CO15	Frequent z-direction	0,15	0,55	1.07 (Fr.)
Option 2: ϕ = 18.7°	IPE 330	0.90	CO2	- Lateral torsional buckling	0.61	CO15	Frequent z-direction	0,24	0,75	0.8 (Fr.)
Option 3: ϕ = 22.5°	IPE 330	0.89	CO2	- Lateral torsional buckling	0.48	CO15	Frequent z-direction	0,36	1	0.61 (Fr. & Char.)
Option 4: ϕ = 26.2°	IPE 330	0.98	CO2	- Lateral torsional buckling	0.91	CO13	Characteristic z-direction	0,5	1,33	0.97 (Char.)
Option 5: ϕ = 30°	IPE 400	0.79	CO4	- Bending and shear - Lateral torsional buckling	0.90	CO13	Characteristic z-direction	0,7	1,44	1.05 (Char.)
Option 6: ϕ = 33.7°	IPE 500	0.63	CO4	- Bending and shear - Lateral torsional buckling	0.80	CO13	Characteristic z-direction	0,94	1,52	0.91 (Char.)
Option 7: ϕ = 37.2°	IPE 550	0.65	CO4	- Bending and shear - Lateral torsional buckling	0.91	CO13	Characteristic z-direction	1,25	1,76	1.03 (Fr.)
Option 8: ϕ = 41.2°	IPE 600	0.68	CO4	- Bending and shear - Lateral torsional buckling	1.001	CO16	Frequent z-direction	1,64	1,99	1.12 (Fr.)

Figure 4.46: Results: varying the angle over height ϕ

Results: Structural elements

Y-beams: The behaviour of the y-beams changes significantly compared to variant: span 7.2m. The cross sections stay constant for the first four options and then increase rapidly. The unity checks decrease and then increase again for the first four options. This behaviour is explained as follows.

Mechanical behaviour:

• The y-beams are schematised as seen in figure 4.47. The critical bending moment is M_1 for the first three options.

• For these options, combination CO2[10] is critical because it produces the largest loads between points A and B.

• The cantilever length (BC) increases when ϕ increases. The added load on the cantilever decreases the value of M_1, but increases M_2.

• M_1 is critical for the first three options. Therefore when the length of the y-beams increases (BC), M1 decreases which explains the decrease of the ULS unity check.

• For options 4-8 the value of M_1 becomes low enough to not be critical any more. M_2 becomes critical for option 4.

• The cantilever is not yet large enough for snow loads to be more influential than the permanent and variable loads for option 4, which is why CO2 is still critical.

• After this option, snow loads (CO4[11]) become influential and the cross sections increase rapidly when the cantilever increases.

• For these options both the frequent and characteristic combinations were critical, but the characteristic demand was slightly larger by a difference of 0.09 to 0.01 from options 5 to 8. This is why the frequent combination becomes critical for option 8.

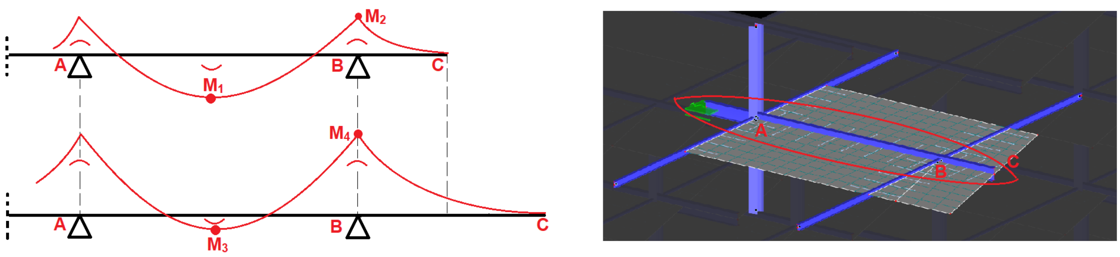


Figure 4.47: Schematisation of the y-beams

[10] $CO2 = 1.2LC1 + 1.5LC2$

[11] $CO4 = 1.2LC1 + 1.5LC4 + 0.6LC2$

Results: Isokorb checks

The Isokorbs react as expected, both shear forces and bending moments increase when ϕ increases due to the resulting larger cantilever. Options 1 and 2 are possible without any additional actions. The other options do not stay within the demands and additional actions are required.

Using hollow core slabs reduced the unity check by 1.3 for shear and 1,1 for bending moments. The floor span is halved however and the reduction is estimated to 0.6 for shear and 0.5 for bending moments. For option 8 this would mean that the unity checks for shear and bending moments become 1.14 and 1.39 respectively. The company that produces Isokorbs indicated that larger forces are possible. Therefore an angle over height up to $\phi = 41.2^o$ could be possible.

Results: Maximum deflection

Beam AB in figure 4.47 has the largest deflection for options 1 to 3. This behaviour is shown in figure 4.48. The deflection will decrease when ϕ is increased, which is why the unity check decreases for options 1 to 3.

This behaviour can also be observed in the results in the appendix (C.6.3). In this table, options 1 to 4 have a deflection upwards which decreases when ϕ is increased.

For options 4 to 8 the deflection of the cantilever becomes critical. Therefore the unity check of the floor is dependent on the deflection of the y-beams. When the SLS demands of the y-beams is too close to 1, the additional deflection of the floor will be larger than the SLS demand.

The unity check is very close to 1 and therefore a solution should be possible for the floors. For example, by increasing the cross section of y-beams or using hollow core slabs.

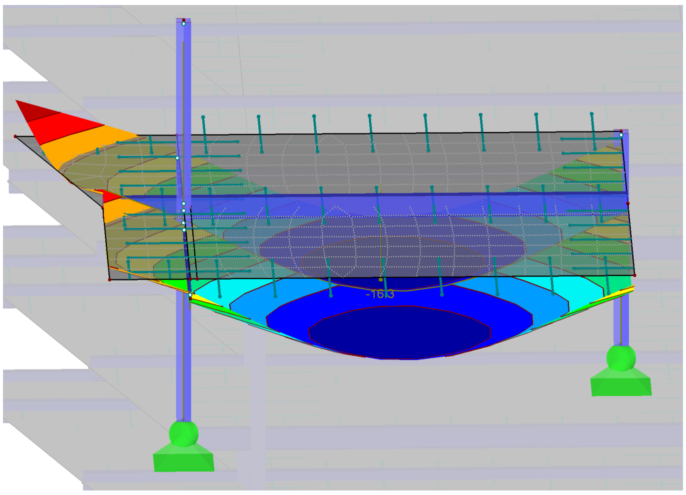


Figure 4.48: The deflections for $\phi = 22.5^o$

Conclusion
This variant allowed three more options which is expected. However, varying ϕ gave unexpected results for this variant and its influence is significantly different from the previous variant. Lowering ϕ below the current value of 25^o will not be beneficial for the structure as seen in figure 4.49, because the beams inside the structure will become critical for these options. Increasing the angle to 26.2^o would increase the costs only slightly. Larger angles however will rapidly increase the cross sections. The total material costs have decreased by 25% compared to varying ϕ for variant: span 7.2m.
The cross sections have also decreased significantly. For the angle $\phi = 26.2^o$ the profiles decreased from IPE550 to IPE330. However, labour costs will be larger, because twice the amount of structural elements are required.
The costs in figure 4.51 are compared to the currently used angle (25^o). The largest contributor to the costs are the y-beams, whose costs increase by 220% if the angle 41.2^o is chosen.
The y-beams change from IPE330 to IPE600 as was the case when $L_{cantilever}$ was varied, but for the latter variant an increase of 149% was found for costs. This difference comes from the fact that increasing ϕ increases the total length of the y-beams. Therefore the weight of these beams and thus the costs increased more. Choosing 41.2^o will increase the total material costs by 96% compared to the currently used angle.

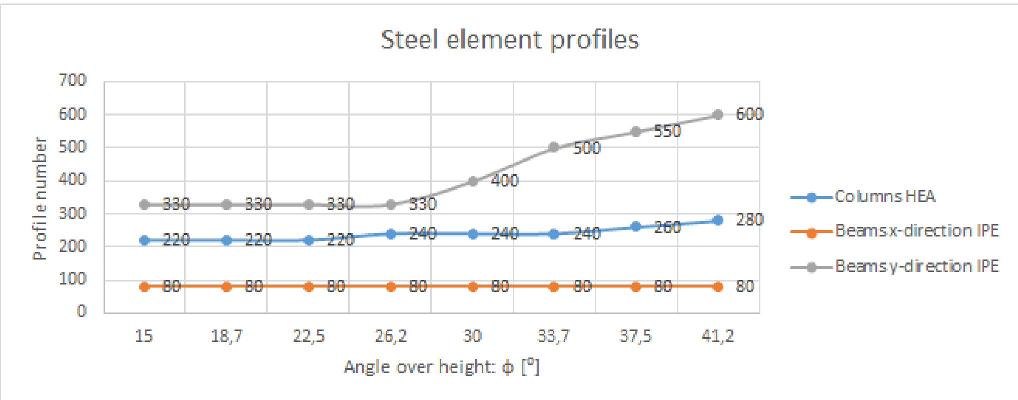


Figure 4.49: Profile height for varying angle over height (ϕ)

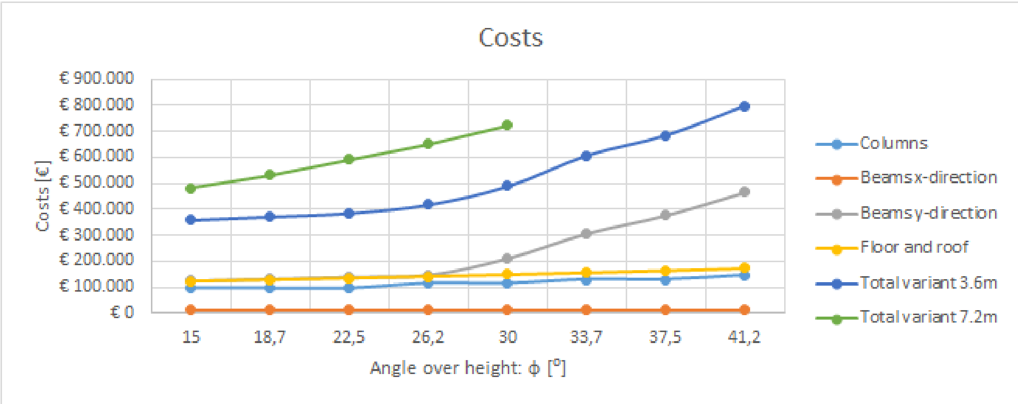


Figure 4.50: Costs for varying angle over height (ϕ)

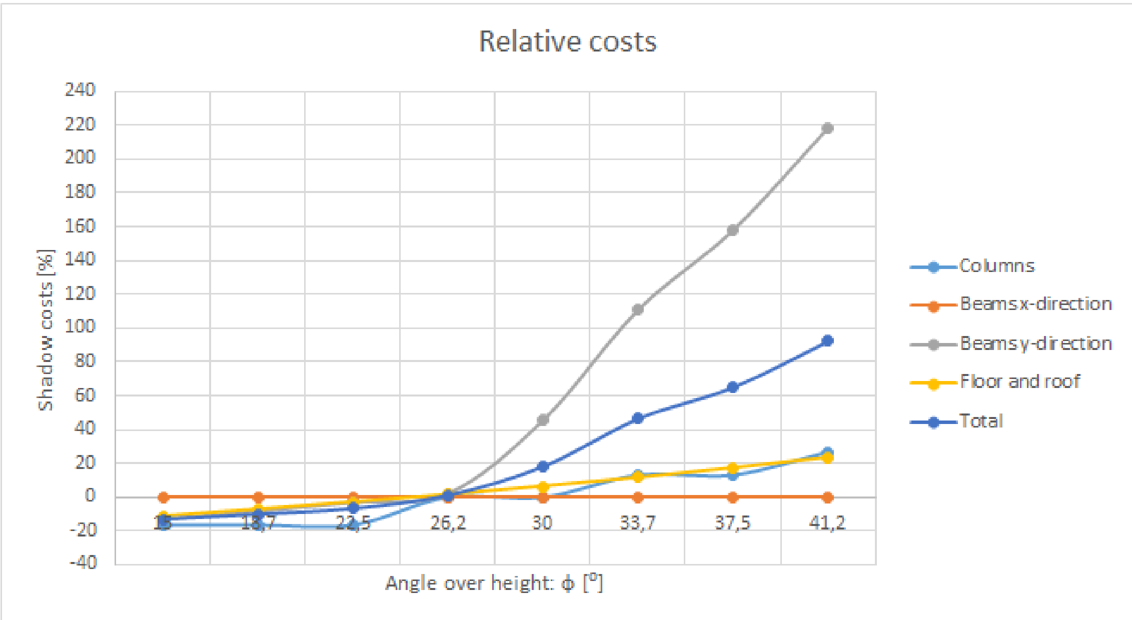


Figure 4.51: Relative costs for varying angle over height compared to $\phi = 25^o$

4.2.5 Results variant: Tie rods

It is clear from the previous two variants that only limited amount of options are available when only steel beams and columns are used. Additional structural elements could improve the mechanical behaviour of the structure.

For this variant tie rods (round bars, called RD) have been chosen to partially carry the floor. The connections are set at the ends of the cantilever beams except for the green roofs. This is done, because the tie rods can be hidden within the walls as seen in figure 4.55b. Inside the structure, the tie rods could be hidden between the partitioning walls between hotel rooms. The aesthetics as intentioned by the architect are thus preserved. Though it does require additional columns with tie rods on the roof.

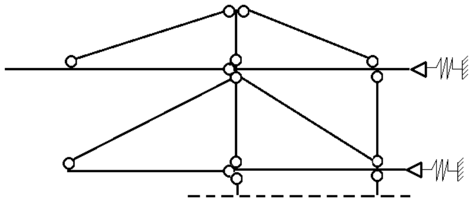


Figure 4.52: Schematisation of the connections for the roof and the floor below

A few alterations to the finite element model had to be applied(figure 4.52). The tie rods are hinged at both ends, because no shear or bending moments are wanted. These internal forces will still be present in the finite element results due to the self weight of the rods. The cantilevered y-beams are hinged as well to ensure a statically determined design. The results are shown in figure 4.56 and the full results are found in appendix C.6.4.

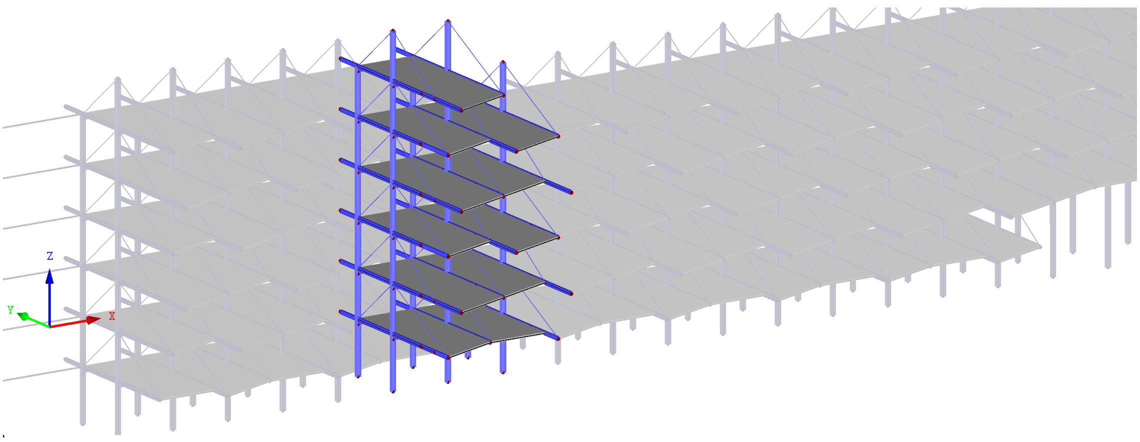


Figure 4.53: Impression of the RFEM model

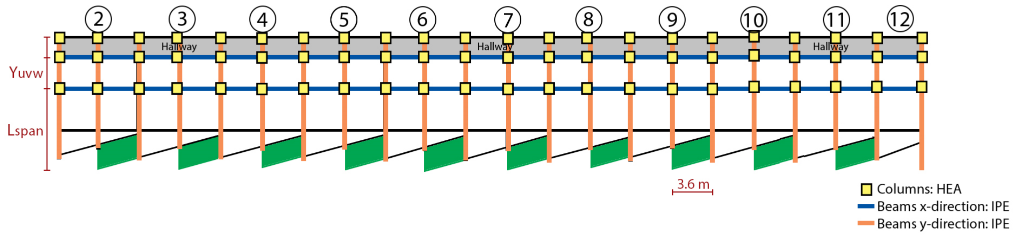
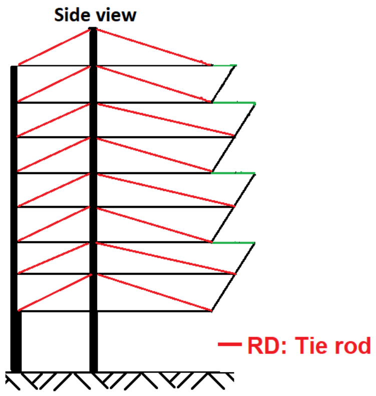
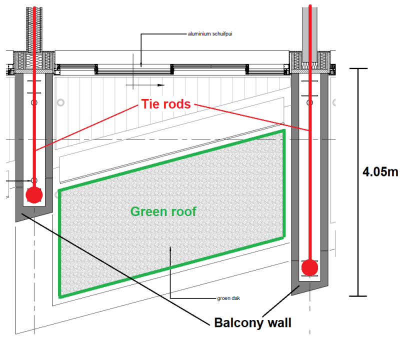


Figure 4.54: Schematisation of the floor plan (varying $L_{cantilever}$)



(a) Side view of the support structure



(b) The placement of the tie rods on the balcony

Figure 4.55: Schematisation of the support structure of variant: Tie rods

Results: Structural elements
A structure is found with a cantilever of 7m. It requires an IPE300 profile and a round bar with a diameter of 45mm. More options were made for this variant to analyse how this variant differs from the previous two variants. Both y-beams and tie rods are critical, therefore both results are shown in figure 4.56
Mechanical behaviour: The y-beams and tie rods however behave unexpectedly. When the cantilever is increased, both y-beams and tie rod cross sections decrease until option 6 where it increases again. This behaviour is explained as follows:
• The internal tension forces in the tie rods depend on the cantilever length. When the cantilever increases, more loads will have to be carried by the tie rods. Thus the internal tension will increase.
• When the cantilever decreases, so too will the loads. The result is a lower cross section for tie rods and y-beams. This is the case for options 7 to 5.
• Combinations CO2 and CO4 are both critical due to the large permanent and variable loads for these options. The y-beams are not truly cantilevered for this variant, therefore snow loads on the end of the y-beams are less critical.
• For options 1 to 4 the cantilever is low and therefore the tension in the tie rods lowers. The consequence is that compressive forces occur in the tie rods due to the wind loads.
• For this reason CO5[12] becomes critical. The compressive normal forces increase for lower cantilevers, therefore the cross sections of the tie rods increase.

[12] 1.2LC1+1.5LC5+0.6LC2

Option	Critical Cross section	Max. ULS U.C.	Max. ULS CO.	Max. ULS Design according to EC.	Max. SLS U.C	Max. SLS CO.	Max. SLS Design according to EC.	Isokorb connection U.C. M	Isokorb connection U.C. V	Max. slab Displacement U.C.
Option 1:	RD 65	0.94	CO5	Bending and compression	-	-	-	0,57	1,38	0.46 (Fr.)
$L_{cantilever}$ = 4.05 m	IPE 360	0.45	CO2	Bending, shear and axial force	0.92	CO16	Frequent z-direction			
Option 2:	RD 70	0.79	CO5	Bending and compression	-	-	-	0,19	0,85	0.52 (Fr.)
$L_{cantilever}$ = 4.55 m	IPE 270	0.84	CO2	Bending and compression	0.47	CO15	Frequent z-direction			
Option 3:	RD 55	0.86	CO5	Bending and compression	-	-	-	0,05	0,5	0.58 (Fr.)
$L_{cantilever}$ = 5.05 m	IPE 240	0.89	CO2	Bending and compression	0.51	CO16	Frequent z-direction			
Option 4:	RD 35	0.82	CO4	Bending, shear and axial force	-	-	-	0,07	0,08	0.71 (Fr.)
$L_{cantilever}$ = 5.55 m	IPE 200	0.88	CO4	Bending and compression	0.68	CO16	Frequent z-direction			
Option 5:	RD 35	0.90	CO4	Bending, shear and axial force	-	-	-	0,17	0,41	0.72 (Fr.)
$L_{cantilever}$ = 6.05 m	IPE 180	0.97	CO4	Bending and compression	0.89	CO15	Frequent z-direction			
Option 6:	RD 40	0.85	CO4	Bending, shear and axial force	-	-	-	0,31	0,43	0.76 (Fr.)
$L_{cantilever}$ = 6.55 m	IPE 240	0.49	CO2	Bending and compression	0.83	CO15	Frequent z-direction			
Option 7:	RD 45	0.90	CO4	Bending, shear and axial force	-	-	-	0.45	0.4	0,77 (Fr.)
$L_{cantilever}$ = 7.05 m	IPE 300	0.64	CO2	Bending and compression	0.98	CO15	Frequent z-direction			

Figure 4.56: Results: varying the cantilever span $L_{cantilever}$

Results: Isokorb checks

The unity checks of the Isokorbs are all within the demand except for option 1. Options 1 to 3 are not recommended due to the compression in the tie rods. Therefore, no additional measures have to be taken for preventing cold bridging in this variant. The manner in which the unity checks behave is unexpected and an explanation is given in appendix C.5.2.

Results: Maximum deflection

The deflections of the floors are all within the SLS demand as expected, because the corners are supported. Therefore, no additional action is needed for the balcony floors of this variant.

Conclusion

A structure is found with a cantilever length of 7m. Though it comes at a cost of total loss of flexibility. It will be difficult to change the floor plan, because the tie rods will be in the way. Therefore, the decision lies between aesthetics and flexibility of the floor plan. This can be decided by the architect.
The cross sections of the x-beams and the columns are visualised in figure 4.57. The costs in figure 4.58 show that a cantilever length of 6.05m is the most efficient for this structure. If the aesthetics of the design of the architect are wanted ($L_{cantilever} = 7.05$), then the costs will increase by 23%.
The total costs of the previous two variants where the cantilever was varied are shown as well. Variant 7.2m has the most material costs, but does have half the amount of x-beams, y-beams and columns and none of the tie rods. This will significantly decrease the labour costs.
The same is true for variant 3.6m where no tie rods are present. However the material costs decrease by 47% for $L_{cantilever} = 6.05$, which might compensate the labour costs.

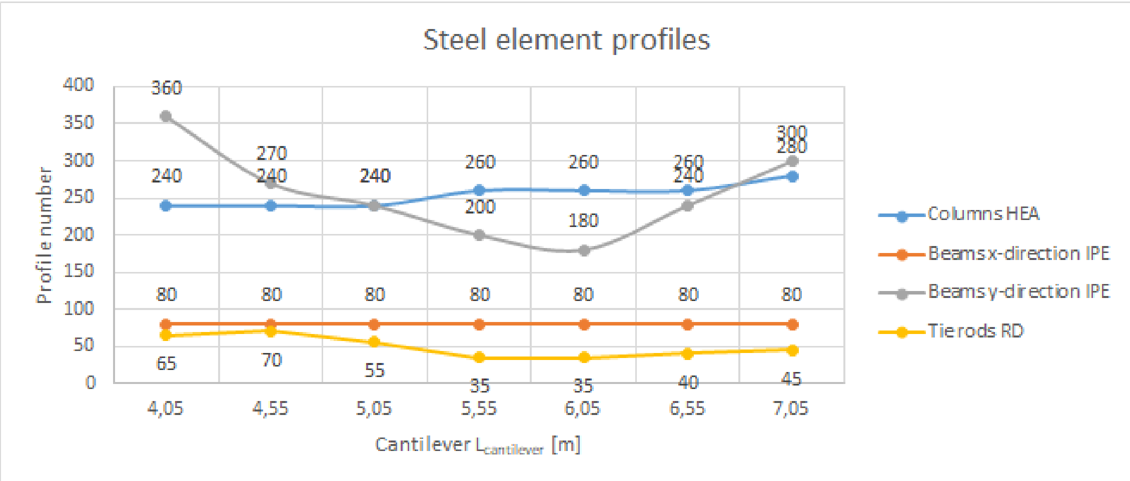


Figure 4.57: Profile heights for varying cantilever span

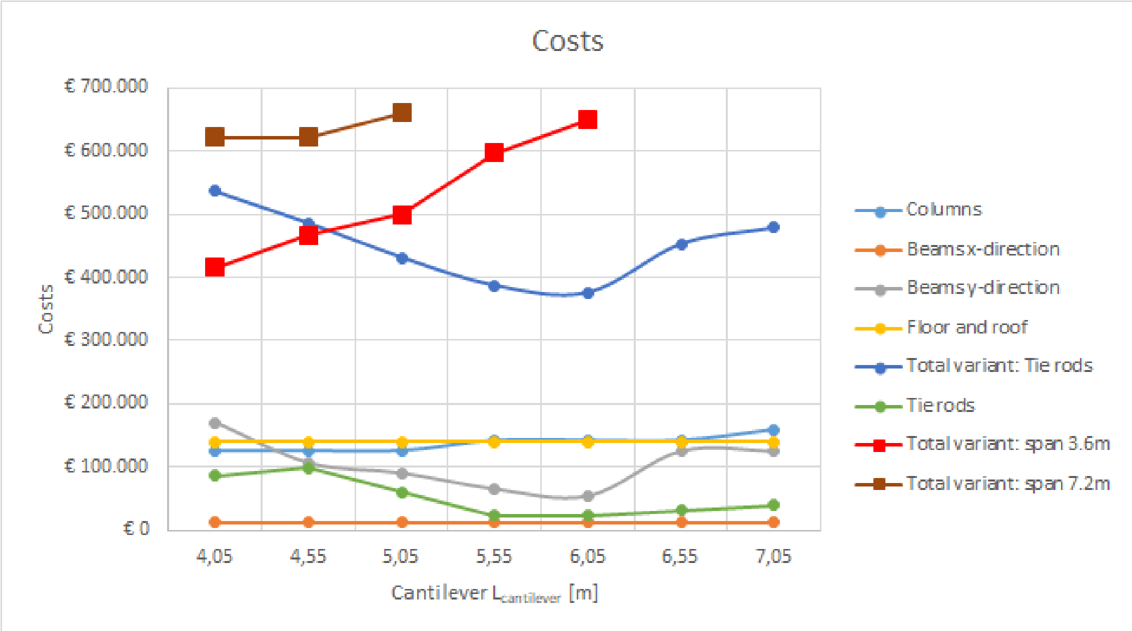


Figure 4.58: Costs for varying cantilever span (L_{span})

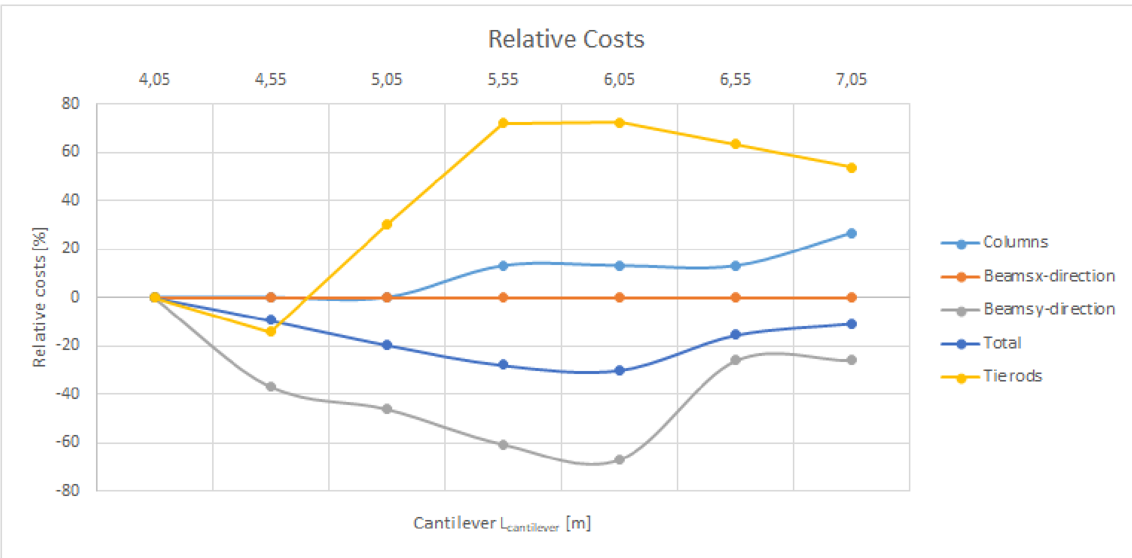


Figure 4.59: Relative costs relative to $L_{span} = 4.05m$

4.2.6 Results: Miscellaneous variants

The previous variants have shown how the DRC can be used as a design tool and that a valid solution can be found for a given problem. It also showed that additional solutions can be advised based on the information output from the finite element model.
A valid solution for a steel structure is found for the outer facade. The next step could be to try to find additional solutions for steel structures or the same analysis for different materials can be performed.
This chapter will be used to exemplify three other solutions for steel structures, which shows that it is possible to rapidly build new variants. Finally variants will be made for a timber structure and a concrete structure. The goal of this section is to show that these options are possible with the DRC. Therefore the structural analysis will be less elaborate.

The miscellaneous variants will only be analysed for a cantilever of 7m, because it stays closest to the design of the architect and because a viable structure was found for this cantilever.
Three more steel variants shall be analysed. The tie rods variant will be analysed for a floor span of 7.2m. This increases the flexibility and it might save costs.
The tie rod variants require a column on the roof, which significantly affects the aesthetics of the outer facade. Therefore additional columns are placed at the top floor, which will carry the roof (figure 4.62a). The columns will be hinged on both sides to produce a statically determined model. To preserve the aesthetics of the model, the columns will be hidden between the balcony walls as was done for the tie rods. The consequence is that the columns cannot be too wide, because it will affect the balcony wall if it does not fit within it.

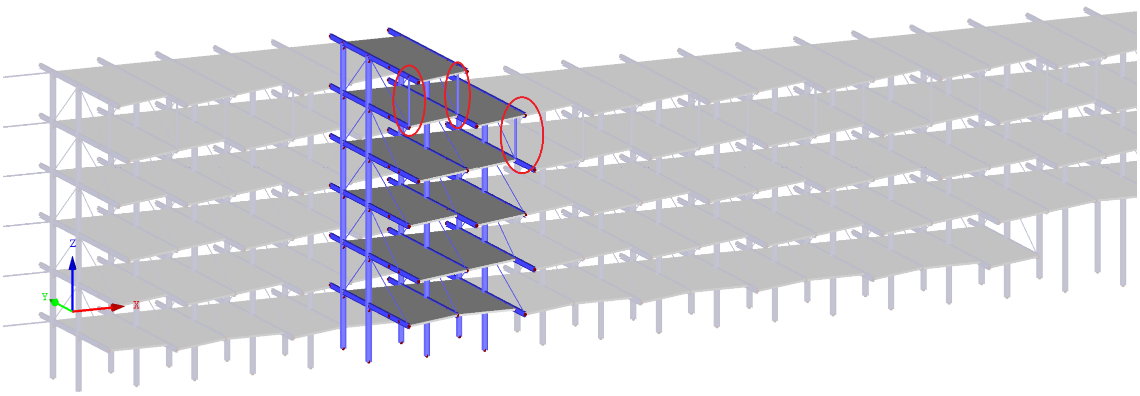


Figure 4.60: Impression of the RFEM model: Tie rods V2 3.6m

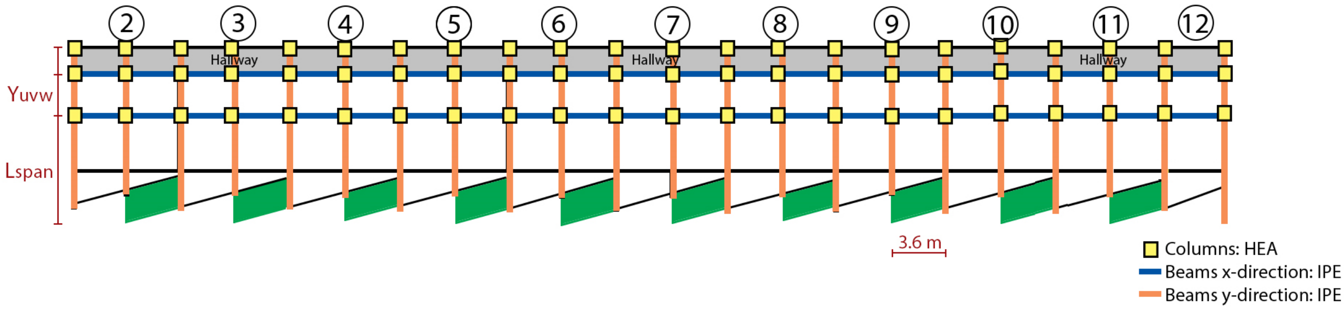
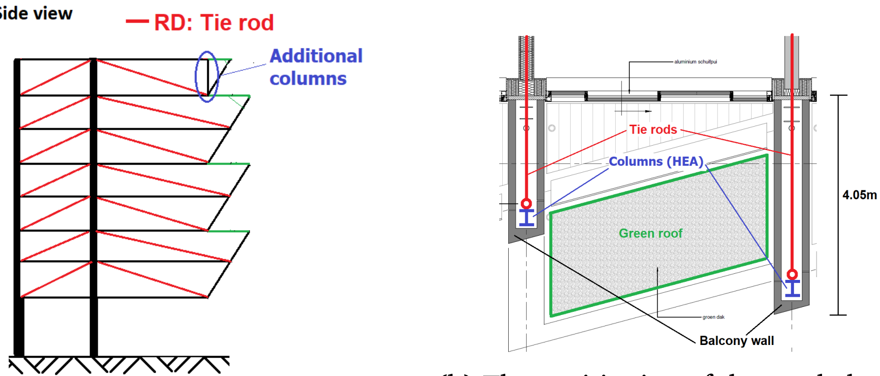


Figure 4.61: Schematisation of the floor plan for all variants



(a) Side view of the support structure with an additional column

(b) The positioning of the steel elements within the walls

Figure 4.62: Schematisation of the support structure

Conclusion

The results of the structural analysis can be viewed in appendix C.6.5. The tie rod variants with columns below the roof instead of tie rods on the roof are called variant: "Tie rods V2". The additional columns that support the roof for this variant are called: "columns below roof".
The results in the appendix shows that the structural elements are all within the ULS and SLS demands. There are some issues with shear forces for the Isokorbs, but this could be remedied with hollow core slabs. It is therefore possible to hide the support structure to preserve the aesthetics of the structure. The unity check is low for HEA profile of *variant tie rods V2 3.6m*, which gives the possibility to explore smaller cross sections.

The variants are compared in figure 4.63 and 4.64. By replacing tie rods on the roof with columns below the roof, the tie rod cross section increased. The effects on the material costs are not seen in the graphs, because the costs increased by approximately 1%. Tie rods and columns on the roof were removed which compensate the costs of the additional columns below the roof. This is the case for both 3.6m and 7.2m floor spans.
A cost increase of 52% is observed between a floor span of 3.6m and 7.2m. This is

a significant increase in material costs, however it does give more flexibility in the floor plan. The labour costs will decrease, because the total amount of elements is halved. This could lower the cost difference enough to make a floor span of 7.2m more preferable if more flexibility in floor plan is desired.

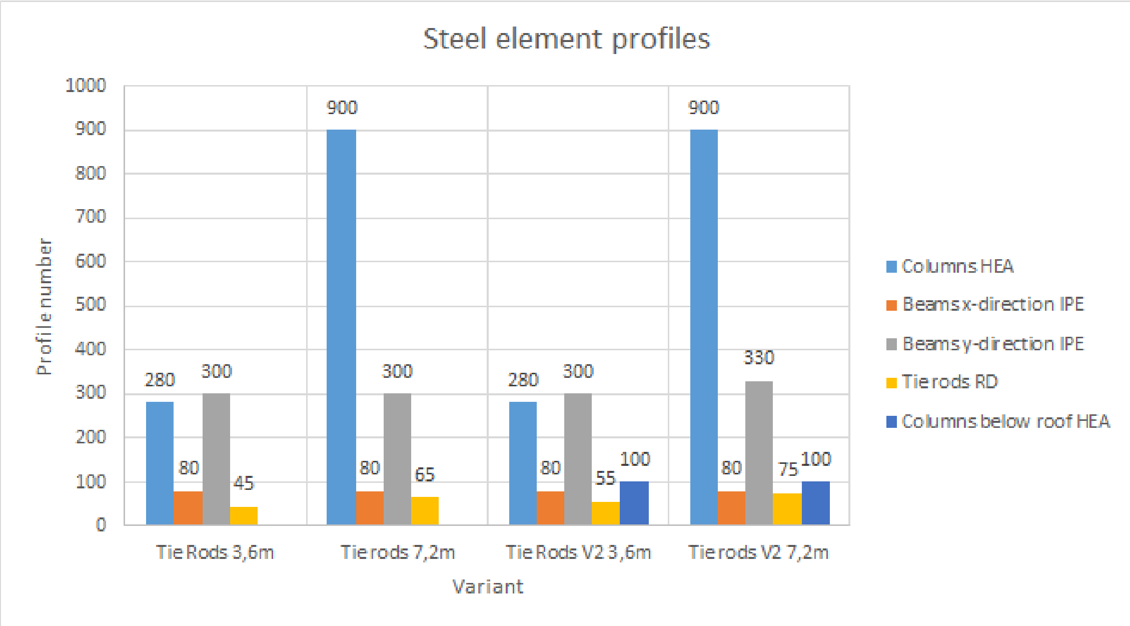


Figure 4.63: Costs of the variants

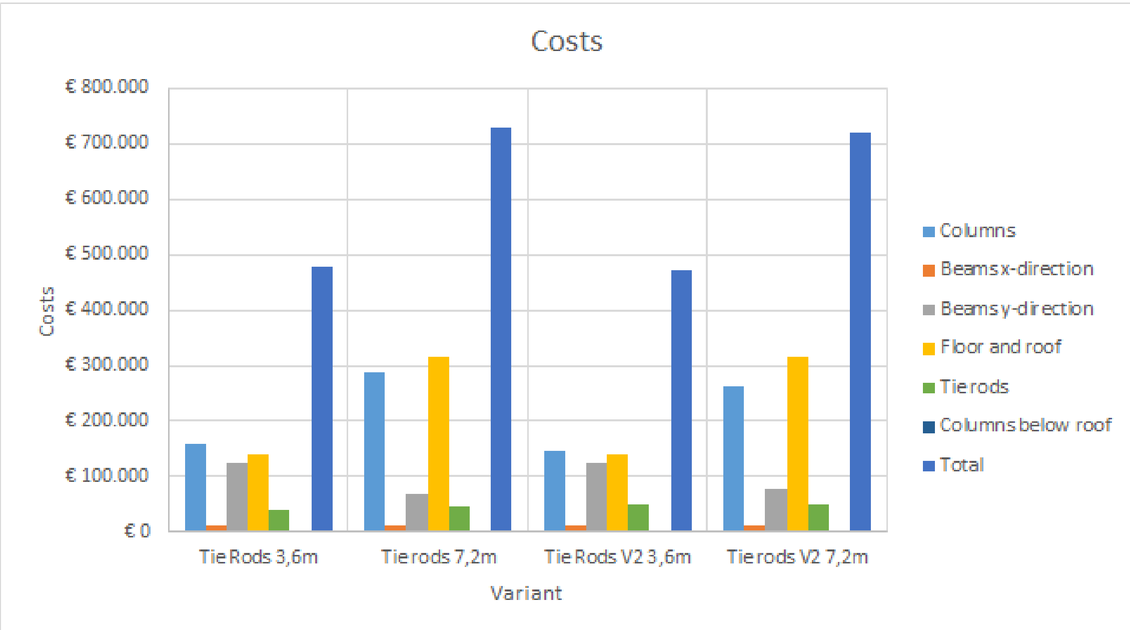


Figure 4.64: Costs of the variants

Changing materials
The choice can be made to analyse different materials, which would follow the same method as was done for the steel variants. The **variant: timber structure with steel tie rods** is analysed to show that ULS and SLS demands can also be checked for timber (using the module: TimberPro) and that it also possible for a structure with mixed materials. The floor plan of *variant: Tie rods 3.6m* is used.

A timber skeleton structure is chosen with glulam 36h T-rectangle profiles for the columns, x-beams and y-beams. A prefabricated timber floor is chosen for which a reference floor will be used by MetsäWood (MetsäWood, n.d.). A floor with Kerto-S beams of 51x300 is advised in their documents whose weight is calculated as $2.2kN/m^2$. This is an orthotropic material, which is not supported in the DRC yet. Therefore a concrete floor (thickness: 88mm) is chosen with an equivalent weight. The deflection of the floor will not be analysed. It should be noted that the stiffness of the concrete floor will influence the results, but an estimation of the timber cross sections should be achievable.
This variant is attempted because the low self weight of timber could be beneficial for the structure. Though the stiffness of timber is low compared to steel.

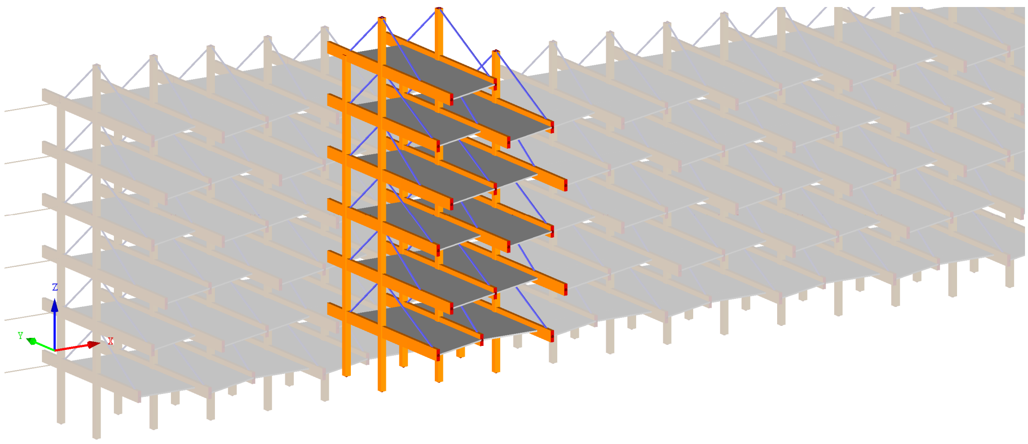


Figure 4.65: Impression of the RFEM model

Conclusion
The results in figure 4.66 show that a structure is possible for a cantilever of 7m. The tie rods and the y-beams require large cross sections, which is mainly caused by the shear ULS demand of timber. Hiding tie rods within the partitioning walls will be more difficult due to their large cross section. Y-beams 200/650 (width/height) also produce a significant height increase compared to steel variants.
Costs could not be calculated for timber variants as was done for concrete and steel as explained in appendix C.3. Shadow costs are calculated instead to show that it is also possible to optimize for sustainability if it is desirable. Appendix C.2 shows how this is calculated for timber and steel. The results in figure 4.67 can be used to see the impact of each element. This information can be used to analyse

which elements have the highest impact. Modifying these would produce the most benefits.
The conclusion is that it is possible to build this structure using timber. However, a large increase in cross section height is needed and therefore timber is not recommended.

Option	Isokorb connection		Structural element	Cross section	Max. ULS			Max. SLS		
	U.C. M	U.C. V			U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.
Variant: Timber structure	0,56	0,08	Column	330/330	0.91	CO2	Stability: axial compression, both axes	-	-	Negligible deformation
			Tie rods	RD 100	0.27	CO1	Bending, shear and axial force	-	-	-
			Beams – Y	200/650	0.93	CO2	Shear due to shear force	0.62	CO13	Characteristic z-direction
			Beams - X	30/50	0.77	CO2	Bending and compression	-	-	-

Figure 4.66: Results: structural elements

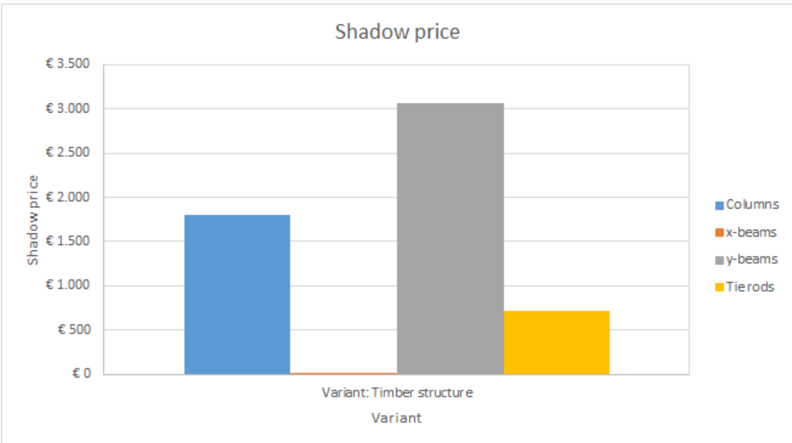


Figure 4.67: Shadow price of the timber variant

The **Variant: in-situ concrete structure** is shown to demonstrate that it is possible to rapidly change the structural scheme and materials of a variant. The floors are rigidly connected to the walls and the beams are replaced by concrete walls 200 mm thick.
Concrete modules exist within RFEM, but are not yet available to the DRC. Therefore simple checks are done to analyse the potential of concrete variants.

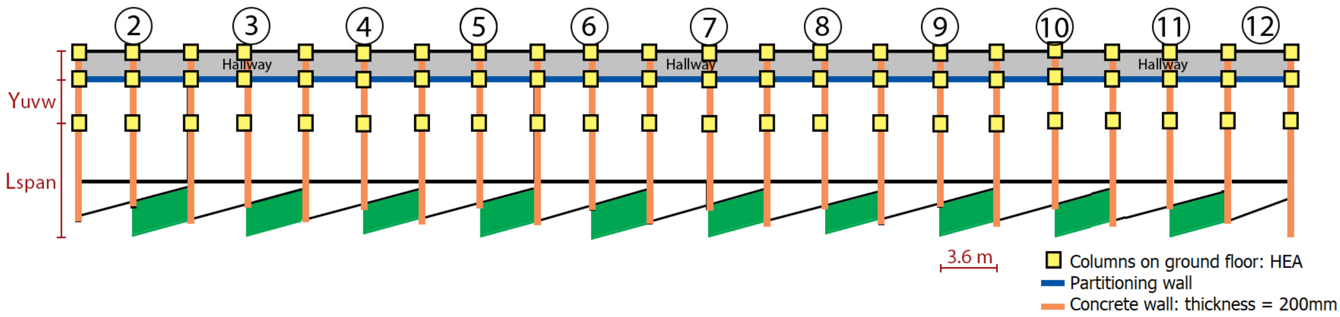


Figure 4.68: Schematisation of the floor plan

Conclusion
SLS demands require the maximum deflection to be less than 28mm for the frequent load combination. Figure 4.69 shows a maximum deflection of 4.5mm at the end of the cantilever. The SLS demand is met.
The columns (250x250mm) at ground floor level are also checked for normal forces. A maximum normal force of 1374kN was found, which gives a stress of $22N/mm^2$. This is within the compressive strength of concrete. The normal force in the columns could pose issues with stability, but this can be remedied with larger column cross sections.
These are promising results. The conclusion is that concrete variants could prove to be interesting and an analysis is recommended.

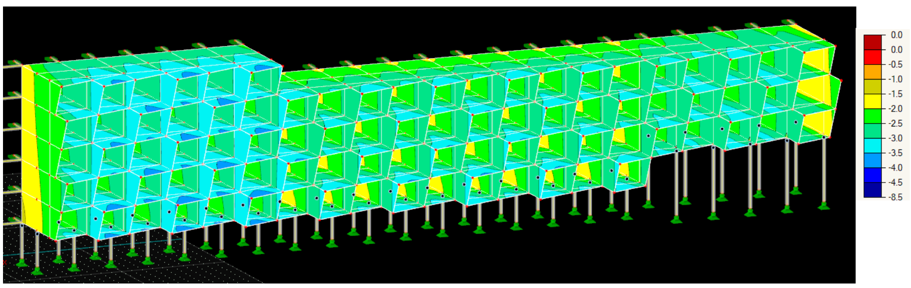


Figure 4.69: Global deflections of the concrete variant

4.3 Recommendations for the outer facade

Data has been collected for several variants and conclusions have been made for each one. This section will be used to collect and analyse the most relevant data and finalize with a recommendation for the outer facade of the Bluebell Hotel.

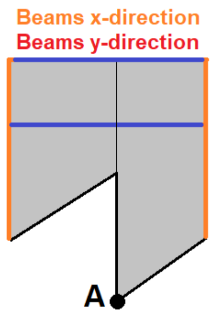


Figure 4.70: The floor of two hotel rooms supported by beams

In summary, the following can be said:

- A support structure for the architects plan will be costly due to the 14.4m floor span. Decreasing the span to 10.8m did not work as well. Therefore it is recommended to use additional columns in the middle of the 14.4m span. A hanging structure or castellated beams could be possible as well but could be considered costly solutions.
- Two options were possible for a span of 7.2m. The cantilever will have to be reduced significantly to 4m however, with a maximum value of 5m. There will be issues with the Isokorbs however and Isokorbs for concrete slabs will probably be necessary. The corners of the balconies (point A figure 4.70) are unsupported as well which causes issues with deflections. Using hollow core slabs could be a solution for the deflections.
- Varying the angle over height ϕ is possible between 15^o and 30^o. Decreasing the angle to 15^o results in a material cost reduction of 22% compared to the current angle. Increasing the angle to 30^o costs up to 18% more and greatly increases the height of the cross sections.
- Further decreasing the floor span to 3.6m had significant positive effects. Both the Isokorb connection and the floor deflections pose little difficulties and the cantilever could be increased up to 6m. The material costs decreased by 25% compared to a floor span of 7.2m with the same cantilever lengths and the cross sections decrease ass well. However labour costs will increase significantly because twice the amount of elements will be present.
- The angle over height can be increased up to 41.2^o for this floor span, but it will make the structure 91% more expensive compared to the current one. Decreasing the angle had little effect on the costs and is therefore not recommended for this variant.

- If a cantilever of 7m is wanted to preserve the current aesthetics, then using tie rods could be a solution. It will come at a total loss of flexibility however. The most efficient solution is a cantilever of 6m and has the lowest total material costs for a large cantilever. A cantilever of 7m is 23% more expensive. Low cantilevers are not recommended for this variant. Variant 3.6m would be the cheaper solution for cantilevers between 4m and 5m.
- The aesthetics of the structure can be further preserved if the tie rods and columns on the roof are replaced by columns on the balcony below the roof. The support structure can be hidden and the costs hardly increase. Flexibility is still lost, but it is possible to increase the floor span to 7.2m for these variants. The material costs increase by 55%, but this will be lower in reality due to the decrease in labour costs. If flexibility is wanted, then these variants could produce viable solutions.
- Using timber instead of steel is not recommended due to the resulting large cross sections. Especially if a cantilever of 7m is wanted, due to the low stiffness of timber which is not compensated by the lower self weight.
- The concrete structure produced positive results and could prove to be an interesting solution. Therefore it is recommended to explore different variants in concrete as was done for the steel variants.

The recommendation based on this information is as follows:

- If a cantilever of 7.05m is wanted, then a steel structure with tension rods with columns below the roof (Variant: Tie rods V2) is recommended. If costs are problematic, decrease the cantilever to 6.05m.
- This will affect the flexibility of the floor plan. If this is deemed important than the same structure with a floor span of 7.2m is recommended.
- If there are no issues with setting columns below the balconies, then variant: floor span 3.6m is advised. It will give total flexibility in floor plan and is a cheaper solution than using tie rods.
- Do not use timber if a large cantilever is wanted. Also do not set stone walls on the balcony, because it will increase the frequent SLS demand significantly. The consequence being a more costly support structure.
- Concrete variants are promising and could be more efficient than a steel structure. An analysis should be done for in-situ and prefabricated concrete variants

4.4 conclusion

The goal of this chapter was to answer the following sub-questions.

1. Explain the results of the variant study. What parameters proved to have the most influence on the support structure?
2. What are the recommendations for the support structure of the Bluebell hotel based on this knowledge?

The variant study and recommendations
Variants were made and analysed for the variant study and options were created as well. The positioning of the columns and the beams in y-direction proved to be the most influential on the results, due to the large cantilever in the design of the architect. The floor spans were critical as well. A 14.4m floor span had to be replaced by a 7.2m or 3.6m floor span.
The following recommendation was given. The architect's plan with a 14.4m floor span is not recommended and caused many issues. The cantilever of the structure also posed significant problems, but a suitable support structure was found for a steel skeleton structure with tie rods. The support structure can be hidden within the building to preserve the aesthetics. If a steel structure is wanted then this would be the best option based on the variant study. It is highly recommended to replace the concrete walls on the balconies with non-stony materials. This would severely decrease the SLS demand and the loads on the balconies and could significantly reduce the sizes of the cross sections.
A concrete support structure with walls showed a lot of potential as well and additional analysis for concrete variants is advised.

5
The comparison

The variant study was successfully performed, which gave insight about the behaviour of the DRC. This chapter will be used to analyse the results.
In the first section it will be verified whether the advantages of parametric design were implemented and the disadvantages mitigated.
The second section will analyse how the DRC performed compared to the current design process. Its effects on the four complications in structural engineering will be shown as well.
The DRC was tested on one structure, but each building is a unique product. The requirements to use the proposed method will therefore be explained.
Finally additional uses of the DRC shall be explained which were outside the scope of this research.

This will be done by answering the following questions:

1. Did the positive aspects of parametric design come forward in the variant study? Were there unexpected problems and how were they solved? (5.1)
2. How does the DRC perform compared to the current design process? Does the theory of chapter 1 hold true in this case study? (5.2)
3. What are the requirements to be able to use parametric design as was done in this thesis? In what way could other projects differ? (5.3)
4. What other aspects of parametric design could've potentially be analysed which were outside the scope of this thesis? (5.4)

5.1 Functionality of the DRC

By testing the DRC on the case study, more information is available about its functionality. This section will be used to explain if the advantages of parametric design came forward, if disadvantages were (partly) mitigated, what unexpected problems were encountered and if unexpected positive aspects came forward.

5.1.1 Advantages and mitigation of disadvantages

The main goal of the variant study was to test whether the advantages came forward and if the disadvantages were lessened. These are listed as follows and an explanation will be given for each item.
The expected advantages:

A1 Complex geometry can easily be taken into consideration.

A2 Endless amount of variants can be created.

A3 Optimisation is possible.

A4 Proper parametric models can be reusable.

A5 Adjustments in the design are always possible.

A6 Software is written for the structural engineer, not to replace him.

A7 The ability to instantly visualise the consequences of design choices.

The disadvantages to be mitigated:

D1 Individuals work.

D2 Loss of flexibility.

D3 Time investment.

D4 Specialised tools limit functionality.

D5 Difficult to analyse all data of large models.

A1: Complex geometry
Complex geometry can be separated in two aspects for the DRC: generating complex geometry in Dynamo and generating a complex finite element model.

Dynamo
Designing a complex model in Dynamo proved to be possible for the complex geometry of the Bluebell hotel. Changing parameters such as the angle over height ϕ greatly affected other parameters of the outer facade. Once the relations between the geometry were set up in Dynamo, these dependencies posed little problem. New geometry can be generated by using sliders.
Positioning the structural elements correctly in Dynamo with the TC-method was laborious and error prone when setting up the first variant (the architects design).

This was the case due to the many exceptions which required a complex coordinate system as seen in figure 5.1. This was less the case for subsequent variants. The coordinates for the structural elements required minor alterations to produce new variants and the geometry of the structure was better understood.

RFEM

Using the complex geometry in Dynamo to generate a finite element model was possible. Complex relations between parameters posed no issues in RFEM. However generating a proper finite element model which accurately models reality becomes more difficult the more complex the model becomes.

A large part of the labour will consist of verifying and interpreting the results. The more complex the structure the more time will be needed to ensure that a correct model is generated.

Conclusion

Generating complex geometry with dependencies between parameters was possible an complex relations can be taken into account in Dynamo. Using the coordinate system was difficult at first, but after gaining experience with both the method and the Bluebell hotel the process became considerably easier. Therefore this advantage was successfully implemented for Dynamo.

The RFEM part however requires significant knowledge with of the finite element method. Not only interpreting results is important, but one should know how to turn a real structure into a proper model. This rapidly becomes more difficult the more complex the model is made and requires an experienced finite element method user.

Therefore it is suggested to keep the finite element model simple. For example in the variant study, the floors could be removed and replaced by line loads on the steel beams to remedy the effects of the stiffness of the floors.

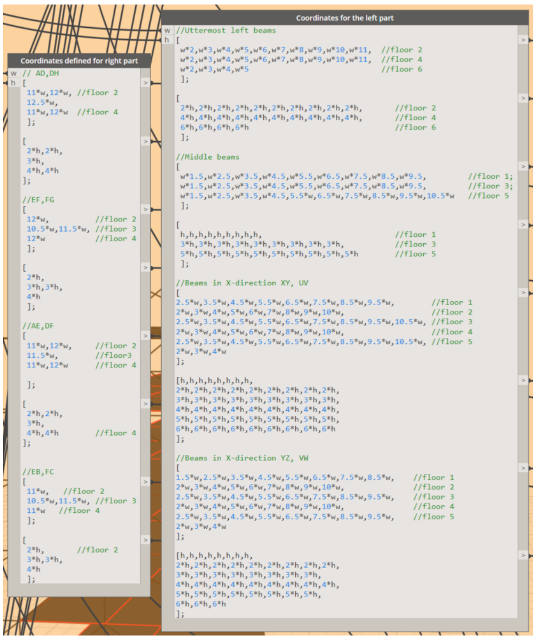


Figure 5.1: Coordinates for the line hinges

A2: Endless variants
The variant study showed that it is possible to rapidly produce options for the architect. Variants required manual work, which slowed down this process. It was shown with the steel variants that the complexity of the supporting structure can be increased to find a suitable solution. The behaviour of potential variants could be analysed and the newly gained knowledge was used to generate new interesting variants.
The timber and concrete variants show that this method is not limited to only steel and that the same approuch is possible for any material which has a module in RFEM. The timber variant showed that it is possible to analyse a structure with mixed materials and that for each material an ULS and SLS check can be performed.

It is possible to vary the structure in endless manners with the basic block and the TC-method. It is therefore concluded that this advantage was successfully implemented. Though the process is slower than traditional parametric design due to the more advanced calculations and because of the required manual labour.

A3: Optimisation
Optimisation of cross sections was partly possible by using the RFEM modules. However, it required manually changing the cross sections and recalculating the structure. An optimal structure was found by iterating until the "optimised" results of the modules were equal to the structure in RFEM. Doing this manually was laborious and time consuming.
Automatic optimisation using modules would be a solution. The API documentation of Dynamo describes a function which can run the dynamo script (Autodesk, n.d.-a). A script could be written which follows the steps which are now done manually. This way an optimal cross section can be found automatically. This function can be further extended to vary parameters automatically as well.

Optimisation with ULS and SLS checks can be done manually with the benefit that one can use the professional code of the RFEM modules. This process is to laborious and time consuming to conclude that this advantage is implemented. It is therefore advised to search for options with automatic optimisation.

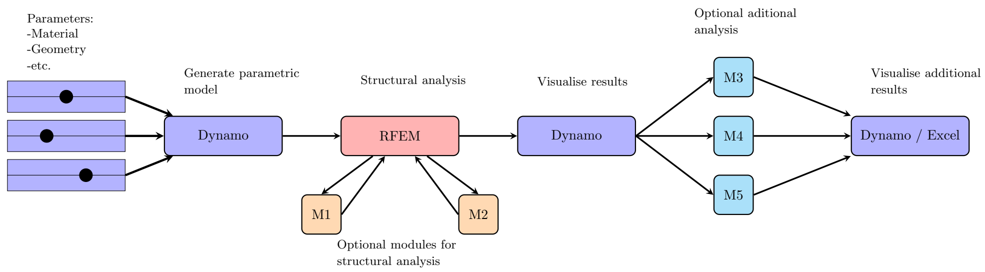


Figure 5.2: The workflow discussed in chapter 3.2

A4: Reusable models
The basic blocks could become reusable if the method of this thesis is adopted by other users. One could search for a block which fits in the current structure and use its elements to set up the parametric model. The more general the basic block, the more reusable it becomes.
The basic block used for the outer facade has low potential due to the shape of the elements being unique (e.g. the shape of the balcony). A simpler basic block using a rectangular balcony would be more reusable.
The modules as shown in figure 5.2 (M1-M5) are reusable as well. For example the custom node which estimated the floor thickness could be used for any structure with a one way spanning simply supported floor. Other projects might require different functions. These can be made and added to the DRC to further increase its functionality. Proper documentation is needed to make it reusable and to increase the trust in the custom node.

This advantage is implemented, but heavily depends on using the basic blocks. Custom nodes or zero-touch nodes are reusable however and functionality of the DRC will increase the more nodes are made. Due to the modularity of the DRC, the user can decide what functions are included in the analysis.

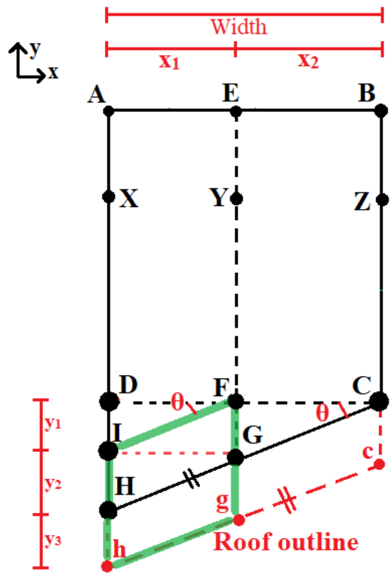


Figure 5.3: The floor plan of a basic block

A5: Adjusting the design
Using the TC-method with basic blocks made it possible to quickly adjust the design in specific manners. The modularity of the basic block made it possible to rapidly add new elements, for example the tie rods in the steel variants. This is only possible if the elements can be defined by using the points in the basic block (A-h in figure 5.3). Otherwise new points would have to be made, which is time consuming.

Each individual component could be adjusted in the outer facade. If different parametric behaviour is desired per hotel room, then this can be achieved by adding additional basic blocks. This is done in the variant study for the architects plan (figure 5.3). The structure was separated for columns with a varying cantilever span (left part of the structure) and the columns with a fixed favourable cantilever span (right part of the structure).
One variant had to be fully set up, after that it could be reused to generate new variants by adding or removing structural elements. Each additional variant made it easier to create a new variant. For example building a variant with a floor span of 3.6m was time consuming to make from a 7.2m variant, but less than making it from scratch. however when these variants were available, the tie rods variants for both 3.6 and 7.2m spans could rapidly be generated.

These features allowed unexpected adjustments to be quickly implemented. The architect can therefore change the design and the parametric model can be adjusted accordingly. Thus this advantage is deemed to be successfully implemented.

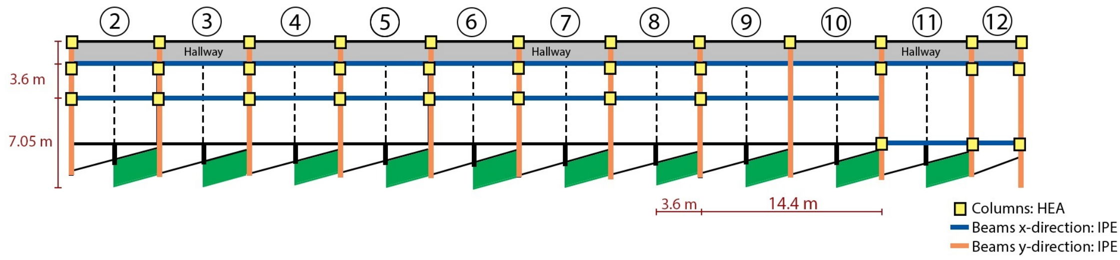


Figure 5.4: Schematisation of the architect's plan.

A6: Software for structural engineer
The DRC is build specifically for the structural engineer. An architect should not use this tool, because it relies too heavily on the ability to model a finite element model and interpreting its results. The average architect does not posses the knowledge to use this software directly.
If the DRC is expanded, it could potentially be possible to build pre-made models with sliders for the architect to use. Though it will severely limit the functionality and flexibility of the tool to guarantee the correctness of the results. In these situations a structural engineer is still necessary to ensure that a proper model is generated.

The DRC is made for the structural engineer to allow for maximal flexibility in its use. It will require a structural engineer to build a correct model and interpret the results.

A7: Instant visualisation
Instant visualisation helps prevent errors while building the parametric model and gives the ability to rapidly get feedback for design choices from the finite element analysis.

The model
Slow feedback for big structures makes building a parametric model arduous. Mistakes were often made due to the complex geometry and are only seen after the script is run. It is too time consuming to run the script after each change, but the more changes are made the harder it is to detect the error. If smaller structures are analysed, this problem will rapidly be lessened and one can visualise the model each time a change is made to the script.
Visualisation works well to prevent errors until a certain element is not properly visualised in both RFEM and Dynamo. For example hinges are shown in RFEM, but their conditions (hinged, rigid or spring) are not visualised. This error is easy to overlook and could lead to a faulty model being analysed.

The results
After the parametric model is finished, one can visualise the results of the finite element analysis after the script is run. One can rapidly learn about the mechanical behaviour of the structure, because the consequences of decisions is visualised. Not only are internal forces shown, but the modules gave the ability to show what the critical demand was. This advantage works well with advantage A5: adjusting the design, because one can see what the main issues are and react accordingly by adjusting the design. The structural engineer can make decisions for new variants based on this knowledge.

This advantage is implemented and helps to detect errors and gives feedback to design decisions. However this process is not instantaneous and one has to wait for the feedback. The larger the model, the more time consuming this process becomes. Therefore visualisation is implemented, but the size of the structure will be a limiting factor. More information about time requirements are given in D3.

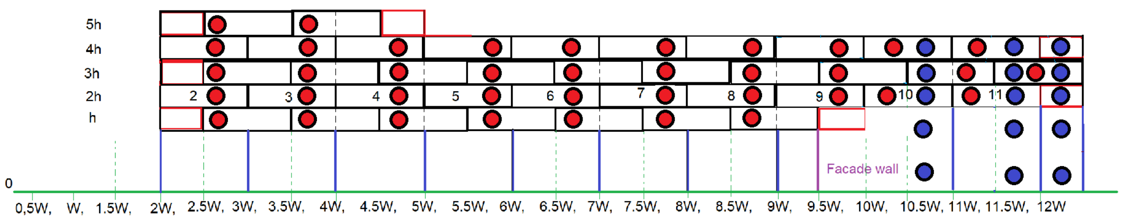


Figure 5.5: The TC-method discussed in chapter 3.3

D1: Individuals work

If the use of basic blocks is adopted for other projects, then the Dynamo scripts become usable by others. Even large scripts could become understandable because it always performs the same action: choose an element and translate it to the proper coordinate. The basic block can be visualised in Dynamo and its functions can be known with proper documentation. Using custom nodes can further simplify the script making it more understandable. These require documentations as well.

The coordinate system as seen in figure 5.1 can be confusing to use and understand. Therefore it is recommended to use an external program which can output them. one such solution could be an user interface that looks similar to figure 5.5. By clicking on the blocks one can choose where the elements are translated. The program can then output the coordinates, which can be copied to the Dynamo script.

This method worked well for the author of this thesis after some practice. New variants could easily be created and there was little confusion when adjustments were necessary in the script. If this is also the case for other users, then this method could mitigate this disadvantage.

D2: Loss of flexibility

Paragraph A5 discussed that it was possible to change the parametric model even if unexpected additional functionality was required. The modularity of the script made it possible to add changes rapidly. This is dependent on the functionality of the basic block. If changes are wanted outside the scope of the basic block, then the basic block has to be adjusted or new basic blocks have to be build. These changes are considerably less flexible.

The script must be properly organized to make this achievable. Additional time is necessary to properly set up the script to make it readable and flexible to change. It is advised to not make geometry too dependent on each other. Generating new geometry with coordinates would be a better option for flexibility, but it will make the Dynamo script larger. A large script is less problematic though because the same actions are performed: choose an element and translate it accordingly.

Therefore, the method used to build the Dynamo script was effective in reducing this disadvantage. But this is less effective if changes are necessary for the basic block. Therefore one should still clearly define the wanted functionality of the basic block before the parametric model is made.

D3: Time investment

The time invested into the variant study should be less than the contemporary process unless enough results are produced of sufficient quality. Therefore timings are given for running the script and building the variants.

Variant	Span 7.2m	Span 3.6m	Tie rods 3.6m
Dynamo script	36 s.	39 s.	45 s.
RFEM geometry	50 s.	1:05 min.	1:24 min.
FEA calculation	1:02 min.	1:23 min.	1:34 min.
Saving results	1:43 min.	2:01 min.	2:17 min.
Importing results	58 s.	1:25 min.	1:50 min.
Total time	5:08 min.	6:30 min.	7:53 min.
Laptop specifications	Dell precision M4800. Processor: Intel(R) Core(TM) I7-4810MQ CPU @ 2.80GHz 2.80GHz. Installed RAM: 16.0GB. Windows 10 64-bit operating system, x64-based processor.		

(a) Timings of a single run per variant

Variant	Span 7.2m	Span 3.6m	Tie rods 3.6m
Nodes	570	642	699
Members	763	910	1206
Total elements	3521	4233	4836
Load information	5 load cases 18 load combinations 1287 loads (line, nodal and surface)		

(b) Amount of relevant elements per variant

Figure 5.6: Timings and number of elements

Figure 5.6 shows the timing for each process when a script is run. First the Dynamo script is run until data is transferred to RFEM, then the geometry is generated in RFEM. This is followed by the finite element calculation in RFEM and the results are then saved. Finally the data is imported and visualised in Dynamo and Excel. The number of the most relevant elements are shown as well. For this variant study the beams were most critical because the ULS, SLS and isokorb checks were dependent on member results. Other elements such as surfaces were mostly constant and their results were less computationally heavy for this analysis.
Waiting eight minutes before one can edit and run a new script is time consuming. The code can potentially be improved to efficiently do the importing and saving of results. Five minutes could still be considered too long, therefore optimisation is advised as discussed in A3. The variants would be made manually, while the options can be generated automatically.

Figure 5.7 shows the time spent to make new variants by using available variants. No accurate time is available for the first variant, because it was used to develop the proposed method over a longer period of time. Before the first variant can be made, time is needed to understand the geometry of the structure, choosing functionality of the basic block and analysing how complex the finite element model should be made. The required amount of time investment for the first variant will depend on the capabilities of the user. Especially his ability to accurately model and verify an finite element model.
Timings given did not include verification of the FE-model, extra programming, error adjustment, etc. In reality errors and exceptions are expected. It is possible that issues arise, which considerably lengthens the time it takes to generate a variant.
There is a large potential to produce interesting data with the DRC, but a considerable time investment is required. It is therefore still necessary to justify the use of the DRC and it should be reserved for complex projects. Therefore this disadvantage is not mitigated.

Utilized variant → Created variant	Timing
Variant: floor span 7.2m → Variant: floor span 3.6m	3 hours
Variant: floor span 3.6m → Variant: Tie rods 3.6m	1 hour and 15 minutes
Variant: floor span 7.2m → Variant: Tie rods 7.2m	45 minutes
Variant: Tie rods 7.2m → Variant: Tie rods 7.2m V2	10 minutes
Variant: Tie rods 3.6m → Variant: Timber structure	30 minutes
Variant: floor span 3.6m → Variant: Concrete structure	1 hour

Figure 5.7: Time spent making variants

D4: Specialised tools limit functionality
Advantage A6 discussed the option to make specific models for architects by limiting functionality and flexibility of the DRC. This shows that this disadvantage still persists. Therefore any UI is not recommended for the structural engineer. The exception is the proposed UI for generating the coordinates for structural elements, which can be done outside the Dynamo environment.

D5: Difficulty in analysing large models
The difficulty of variant studies for a building is the fact that almost infinite amount of variables are possible and the required amount of calculations is too large. New variants require significant changes as well. Making a parametric model which is able to generate all the different variants would require a large and complex script which would also run slowly. This will come at a cost of flexibility of the script as well.
The proposed method manages this problem by giving the ability to quickly generate different parametric models for each variant. Options are then produced by using the unique parametric model for each variant. this way the dynamo scripts becomes smaller, more readable and run quicker.
This method could partly circumvent this disadvantage. However, making new variants requires heavy manual input and a structural engineer who is able to do this effectively. This part will be difficult to automate since it depends on the creativity of the user. Producing options however can possibly be fully automated. This could lead to optimisation of the options themself.

The issue of having uncountable amount of calculations and parameters is partly mitigated with this method. By testing the structure variant by variant new knowledge is gained after each analysis. With this knowledge informed decisions can be made for new variants. The structural engineer will know what variables and calculations are interesting and which are not. This way the amount of variables and calculations can be reduced by using the creativity of the structural engineer.

Optimising results based on large amount of data is not done for this method. Instead the experience and creativity of the structural engineer is amplified by giving him the options to rapidly generate results and then react to them accordingly. This way it is not necessary to analyse all the possible parameters and calculations. The structural engineer acts as a filter for these possibilities. This disadvantage is deemed to be partly successfully mitigated for these reasons.

5.1.2 Unexpected positive and negative aspects

Advantage: demands considered at the start of the design stage (A8)
Advantage 12 in the literature study explained that SPD is an excellent way to consider all demands in the beginning of the design stage. By using the modules in RFEM it was possible to analyse every element in the finite element model according to the eurocode with the Dutch annex. The cost being a small amount of computation time.
By performing such complex calculations, one can prevent issues with demands in the later design stages. It is therefore less likely that changes are necessary for the support structure due to a certain ULS or SLS demand.
This advantage is important and is therefore called advantage A8.

Disadvantage: Difficult to analyse all data
Generating multitudes of variants makes it difficult to analyse all the data. This was mitigated by visualising the most important data in an Excel sheet. Detailed information could be found in RFEM. For the architect the data was shortened to costs and cross sections, which can be visualised in graphs.
The proposed method requires the structural engineer to analyse the data per variant. It is therefore not necessary to analyse the data of all variants at the same time.

Disadvantage: Suitability of the design
The TC-method was manages exceptions in the design. Exceptions in the variant study required additional nodes and more coordinates. This enlarges the script, but does not make it more complex, because structural elements are only selected and translated. This is mostly discussed in advantage A5. Therefore this disadvantage was unexpectedly partly mitigated.

Other small unexpected positive aspects are listed as follows:

- A new Dynamo script was used for each variant. The problem was that if certain changes are wanted in all the past scripts then one would have to edit all the previous files. This would be the case if an error was found after some variants were already made.
  Custom nodes are therefore advised because they only have to be edited once and will automatically be updated in each script. Though it was explained that custom nodes are computationally heavy in Dynamo. Therefore a balance will have to be found between time spent editing Dynamo files and time spent running the script.
- The DRC allows development towards specific functions. This gives the option to develop tools within the DRC. The RFEM modules are a complex example, but simpler analysis methods are possible as was done for the concrete floors in the variant study. The benefit is that there is no dependence on a developer and the user can choose his own functionality for the module.

- Information is given for each type of element, which is useful for optimizing designs. One can see which structural elements impacts the costs the most and react appropriately. Efficiency in the design can be focused on these elements, which can be used to more effectively reduce monetary costs and shadow prices of the support structure.
- Instantly showing the consequences of certain choices can be a great learning tool for structural engineers with little experience. The potential to gain experience is increased because the consequences of actions are visualised and more types of structures can be analysed.

5.2 DRC compared to contemporary design process

Chapter 2.1 explained the current general building process. The most important topic shall be stated in this chapter and compared to the current method using the DRC. The abbreviations of the advantages of parametric design (A1-A8) shall be included.

The architect's design process

The structural engineer may offer consultation at the start, during or end of the architect's design process. Basic calculations are performed to check the stiffness and stability requirements of the support structure.

Required information

For the architect the most important aspects to analyse are the spans of the elements, the materials and the height of the structure.
The DRC provided information about the possible options for the support structure. The architect will be able to choose the materials and spans of the structure by analysing different variants. The height of the structure will partially depend on the chosen cross sections. Advantage A8 allowed this information to be more detailed than the contemporary method.

Start:

Information may be required of the support structure for certain design choices at the start of the design process. Examples are a large hall without columns or the decision to only use a certain material.
Advantages A2 and A8 can give the architect more options and more detailed information. This can ensure that a proper decision is made at the start of the design process. Future issues can be anticipated and reduced as well by weighing the advantages and disadvantages of the variants.

During

Less experienced architects may need consultation during the design process and may want to try different options.
The DRC makes it possible to give the architect multiple options as well as their

advantages and disadvantages (A8). If none are deemed suitable, then adjustments to the design should not be problematic (A5).

End
For some projects a small amount of attention is given for the support structure during the design process. This requires the structural engineer to react to the design which may require alterations after structural analysis.
The DRC can be used as was done in the variant study to find a suitable design and to provide numerous solutions for the support structure (A2). The complexity of the design should pose little issues for the analysis (A1). Multiple alterations may be necessary during the time that a solution is found which suit both the architect and the structural engineer. These are possible (A5) without having to sacrifice the amount of information given (A8). More informed decisions can be made and more options are given, which may lead to more quality or efficiency of the structure.

However, for all these advantages one must not forget that the disadvantage of time investment still applies. It must be judged for each project whether this amount of information is necessary and whether it is worth the significant time investment (D3).

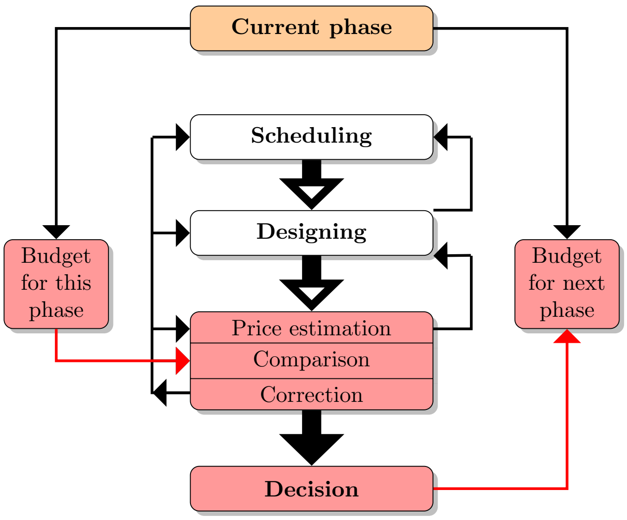


Figure 5.8: Cost management visualised. Amended from: (van Eekelen, Rip, & Wentzel, 2002, p. 122)

Cost management
Budget estimation is not only done for the support structure. Other topics are included as well such as terrain, installations, labour or building elements without structural purposes (van Eekelen et al., 2002, p. 147). The costs can be calculated with the cluster of elements method. If corrections are necessary than this is done by adjusting the design until the decision is made that the design is acceptable.

Cost management in its current form is still necessary, because The DRC only considers the support structure. However, the DRC directly affects the step "Designing" in figure 5.8, where information about material costs is given for each variant in this step. The ability is given to consider costs earlier when comparing options (A8). This may prevent issues when costs are compared to the budget.
If corrections are deemed to be necessary, then adjustments to the design are possible in the DRC (A5). Redoing the steps can rapidly be done because of this. If a large amount of options are generated (A2), then these can be considered instead. This prevent having to generate new variants or options.

Tasks of the structural engineer
The tasks of the structural engineer in the preliminary design phase consist of: setting up starting points of the design, developing variants of the support structure and producing a preliminary design. The structural engineer will help the architect to implement the requirements of the SoR and the consequences of choices are analysed by designing variants. Designing variants is costly though, which limits the amount of variants analysed and their complexity.

The focus was mainly set on enhancing the process of developing variants. The same tasks will therefore have to be done by the structural engineer. The amount of variants (A2) and their complexity (A1) can be increased however. More information can be extracted as well from the variants (A8). The additional amount of information can help the architect implement the SoR more efficiently or of more quality.

Cost efficiency
Cost efficiency is found in material and labour costs, this must be done in cooperation with the architect because it may lead to concessions in the design. Currently rules of thumb are used to find a cost efficient design. The issue is that these are not true for every project and can contradict themselves. These results in even more rules of thumb, which also do not have to be true for each project. Therefore analysing variants is still important.

Material costs are directly output by the DRC (A8) for each option. It is possible to generate numerous options and variants (A2), which can then be compared by the architect. Rules of thumb can still be applied to the variants, but contradicting rules of thumb can be analysed for each project to find the most efficient option. One could for example compare increasing span widths to the amount of joints and find the best solution in the middle. Less dependency on rules of thumb is therefore achieved allowing for more cost efficient structures.

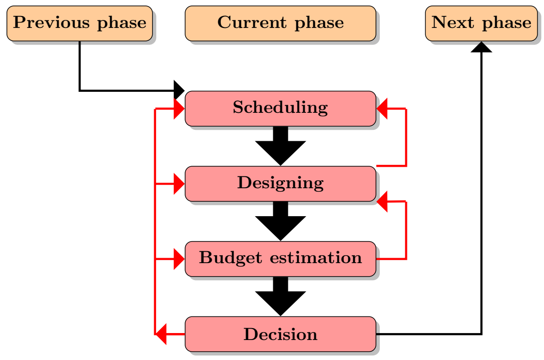


Figure 5.9: The design procedure. Amended from: (van Eekelen, Rip, & Wentzel, 2002, p. 21)

The preliminary design phase

To make the process in the preliminary design phase more manageable, the basic steps in figure 5.9 were introduced. The SoR is elaborated (*scheduling*), the support structure is developed (*designing*), the price is calculated (*budget estimation*) and finally a decision is made. If adjustments are deemed necessary, then these steps have to be followed again

The proposed method mainly influences the *designing* and *budget estimation* parts. Decisions have to be made for the support structure and variants will be weighed against each other. By using the DRC, it was possible to give the architect more options (A2) and more information (A8) about them by adding detailed information about costs and cross sections. This way the architect will know the consequences on the costs from his actions as well as if the support structure fits within his design.

Proper budget estimation will still be necessary because information was only given about the material costs, which was an estimation. The DRC is able to output more detailed information about the cross sections (A8), which can can make the cost estimation of the structural part more accurate.

The steps in the preliminary design phase will therefore stay the same. Advantages in the design part and budget estimation are expected due to the increase of available information (A2&A8). If the decision is made to adjust the design, then the model can be edited with little effort (A4).

5.2.1 The four complications

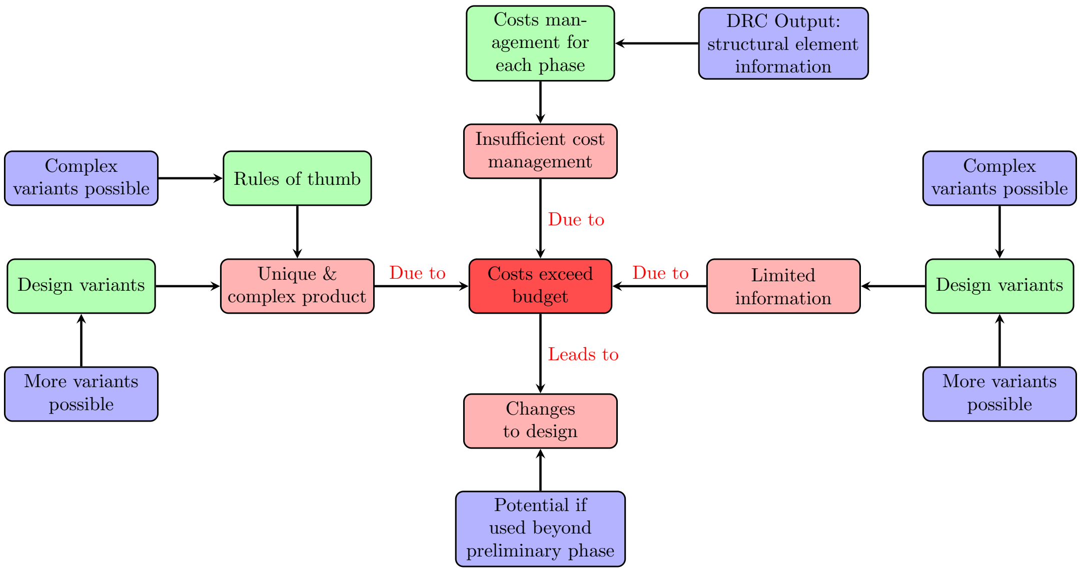


Figure 5.10: Complications in contemporary structural engineering and solutions

Chapter 2.1 discussed that costs commonly exceed the budget causing projects to be delayed or not build at all. This is mainly caused by: insufficient cost management, planning a design based on limited information and that structures are unique complex products. Costs exceeding the budget makes it necessary to make alterations to the design, which is costly at the end of the design stage. These issues are coloured red in figure 5.10. The current solutions to these issues is given in green and the influence of the DRC is shown in blue.

Limited available information
Variant studies in the preliminary design phase are done to find information about the project. Only limited amounts of variants are designed however, because designing and analysing variants is a costly process.
The DRC allows more variants to be produced while increasing their complexity as well.

More variants
More information can be gained of the support structure which gives the architect more options. More options give the architect the freedom to pursue more interesting designs (enhancing quality). It gives the architect more solutions as well, which can be used to find a more optimal design (enhancing efficiency).

More complex variants
The DRC allowed for more complex geometries for the variants. More complex calculations were also made available which made more information available for both the structural engineer as the architect.

The structural engineer
The DRC was able to inform the structural engineer about the following topics: the critical load combination, the type of failure and the unity checks. The modules made this information available for each structural element. A summary of these results was given to manage the large amount of data, but it is possible to check detailed results in RFEM.
More knowledge of the structure is available which can be used to make an informed decision for the next variant. By understanding what the critical aspects are, one can focus the next variant on solving these issues. Repeating this process results in a focused approach which leads the structural engineer towards a solution for the support structure.

The Architect
The DRC was able to inform the architect of the following topics: costs of options, available cross sections of the support structure and possible positions of structural elements. The architect has more options to choose from and an informed decision can be made by better weighing the advantages and disadvantages.

Buildings are unique and complex products
Increasing the amount of information was needed, because structure are becoming more complex. This makes their mechanical behaviour difficult to predict.
Rules of thumb are used as well to lower the costs of complex structures. These are not always true and can contradict each other though.

Variants
Complex geometry and dependencies between parameters can be included in the Dynamo script. These posed little problems when generating new variants or options. It is therefore possible to check the influence of parameters more in depth than before. For example, calculating maximum deflections in the Bluebell Hotel for varying parameter ϕ would be laborious by hand, because the dimensions of the rooms change based on ϕ.
Complex structures make it difficult to predict what the critical demand is and for what structural element it applies. The critical demand and structural element can change as well when parameters are varied. With the DRC however, one is able to perform such an analysis with little required input, especially if making options and optimisation is automated.

Rules of thumb
Rules of thumb can still be applied to the variants. For example varying between cross sections as little as possible was applied in the variant study, because it reduces labour costs. Contradicting rules of thumb can be analysed however.

Insufficient cost management
Proper cost management at each phase of design can prevent the risk of exceeding the budget during the specification stage. This can be done with basic calculations using mass and volume or more accurately with clusters of elements, which requires the total amount of length, area or volume of a certain structural element.

This information is output by the DRC in an excel file for each unique cross section. The data can be used to make material costs sooner available in the design process. By outputting this for each cross section, the largest contributor to the costs can be targeted. More attention is given to the costs, which may lead to a more efficient structure and less chance that the budget is exceeded.

Changes to the design
The proposed method is made purposely to allow for changes in the design. However the variant study simulated a design at the start of the preliminary design phase. A detailed design at the end of the design phase was not tested. The complexities required for a detailed design and the place of the DRC in this design phase is not discussed. Therefore no conclusions can be given for this issue. However many of the positive aspects incorporated into the model could prove to be beneficial.

Enhancing the process
Using the DRC to mitigate the current issues resulted in enhancing the current solutions instead of finding new solutions. The benefit is that the current workflow of the structural engineer is disrupted less, which can help give the DRC a place within the current building process and make it more likely to be used.

5.3 Requirements for SPD

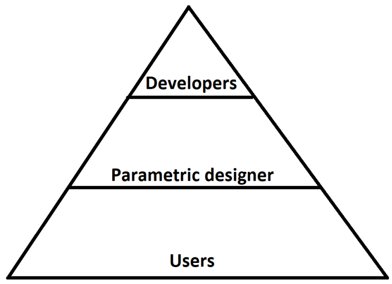


Figure 5.11: The pyramid of users

The pyramid of users
The proposed method is made specifically for the structural engineers to properly use the DRC. They can be grouped in three categories named: the developer, the parametric designer and the user (figure 5.11).
The developer can code zero touch nodes and create new scripts with C#. New functionality can be developed by this person to aid the parametric designer.
The parametric designer can build parametric models with the DRC and use it to perform an analysis. This person will be able to create new variants.
The user has little knowledge of the DRC. If a parametric model is made, then this person can create new options by using the sliders for example.

The knowledge requirement of the user can differ and it is therefore not necessary to know everything about the DRC. Though knowledge of the finite element method will always be needed to correctly interpret the results.

Time investment
Disadvantage 1 of the literature study explained that setting up a parametric model from scratch is laborious and thus time consuming. The time investment introduces the risk of not producing enough results.
This is still the case for the DRC. One should consider for each project if the increased amount of information would lead to more quality or a more efficient design. The extra costs of SPD must be compensated by the increase of quality or efficiency.
Due to the uniqueness of buildings it is difficult to give criteria for such considerations. It does not only depend on the complexity of the structure, but also the wants of the architect and the client.
However, this is the largest issue of the DRC and will prove to be the main factor for not using the tool for most projects.

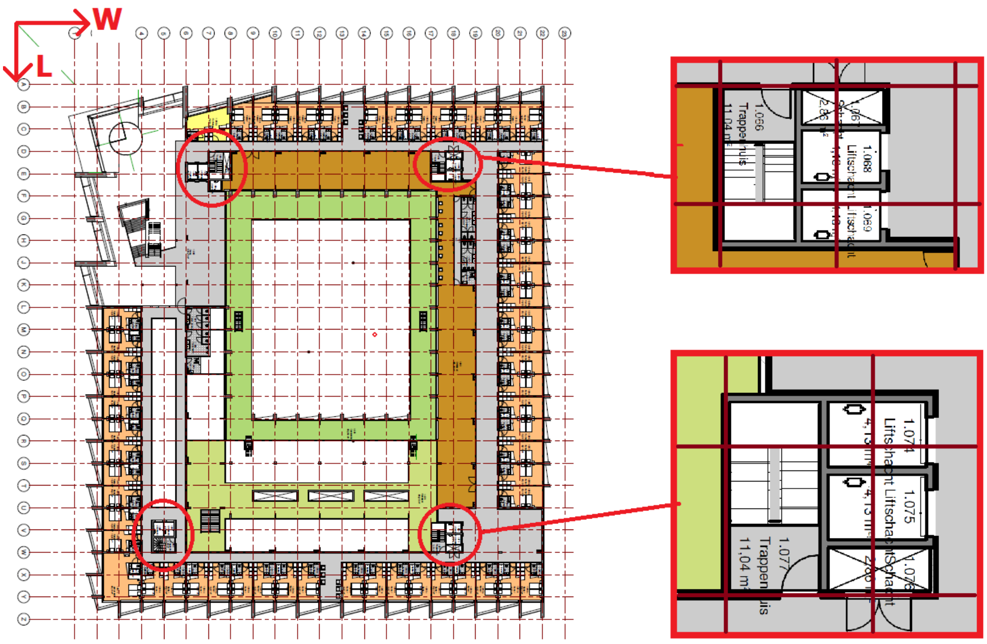


Figure 5.12: The ground floor plan of the Bluebell Hotel

Grid lines

The proposed method heavily depended on a coordinate system. In the variant study this was done by dividing the outer facade. A coordinate point was available for each two hotel rooms.

The method relies on the repetition of structural elements. The literature study explained that architects currently keep grid lines constant in their design to ensure that spans remain constant. This ensures that many structural elements are kept the same size and thus repeatable.

The ground floor of the Bluebell hotel in figure 5.12 shows that most elements stay within the grid lines. These can be defined by the coordinates. However some parts, for example the stairwells, do not stay within the grid. These walls will have to be added to the basic block or a new basic block can be made.

An example of applying the proposed method on a 2D grid is shown in figure 5.13, where elements were added and removed per variant. Basic block 1 contains a single column at the top left part and four walls along its edges (which can be individually selected). The manners in which the columns can vary is programmed into the basic block ($\Delta X, \Delta Y$). The columns do not necessarily have to stay within the basic block, but it is important to clearly definite what parameters are varied and how they should be varied.

It is recommended to not use too many custom nodes, which were used to create the basic block. The proposed method relies on repeating elements. More unique elements come at a cost of a larger script, which results in more computation time and more time to produce a new variant. This method is therefore not suitable if the floor plan contains to many exception cases.

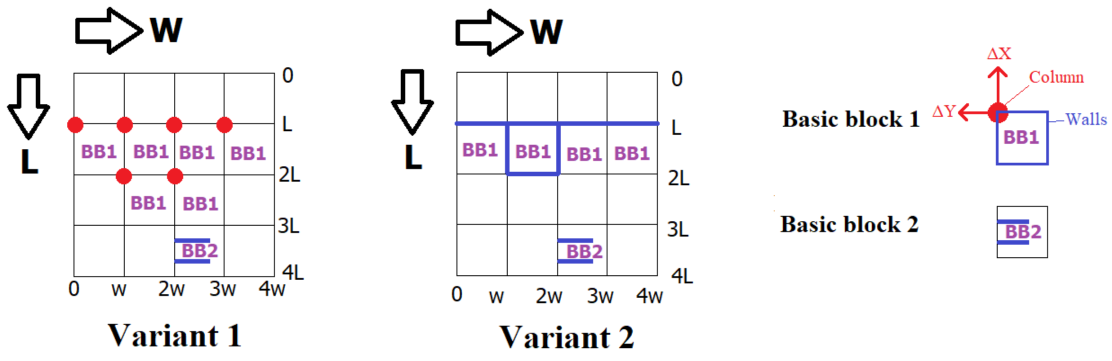


Figure 5.13: Using the basic block on a grid

Size of the model
The size of the model should be limited when using the DRC. Large models require significant computation time, more time to create a new variant and visualisation of the model (to check for errors) takes more time. These issues grow the larger the model becomes.
It should be possible to further optimize the code of the DRC to decrease the computation time. Automatic generation of options could reduce the significance of this problem as well, because other tasks can be performed by the user while the computer generates the options. With the current functionality of the DRC it is advised to not analyse structures larger than the Outer facade.
One possible solution is to divide the structure and analyse the parts individually. The Outer facade for example was a critical part of the Bluebell Hotel due to its cantilever. Other critical parts may be analysed as well in a similar manner. However, this requires an even larger time investment.

5.4 Additional potential functionality of the DRC

Additional functionality in RFEM
A large amount of additional RFEM functions can be added to the DRC according to the documentation of the API (Dlubal Software GmbH, 2018). Therefore only the most important additional functions shall be described.

- **Results:** Solids can be analysed in RFEM, though it will cost significant computation time for large finite element models.
  Surface results can be expanded to include a function which can find results on a specific point in a chosen surface.

- **Calculation parameters:** Non-linear calculations can be performed including large deformation and post critical analysis.
  Load increments, number of load iteration and bending theory type can be set as well.

- **Mesh options:** mesh options could become necessary for certain results, for example to mitigate singularities. This can be done for 1D, 2D and 3D elements.

- **Loads:** varying surface or line loads can be added. More advanced functions such as displacements, imperfections or temperature loads are possible as well.

- **Model data:** simple inputs were used to classify structural elements, but RFEM allows more advanced options as well. Mesh refinements, modifying stiffness, eccentricities, elastic foundations, orthotropic materials and many more functions are available.

- **RFEM Modules:** chapter 2.4.2 showed that many more RFEM modules can be implemented.

This is only a fraction of the possible functionality. If a certain function is wanted then it can be added for the project by the developer, whereafter it will be available for subsequent projects. This way the DRC can grow while reducing the risk that functions will not be used.

(Shadow)Costs module
The costs in the variant study were approximated by using the cost value per volume or weight. This resulted in speculation when comparing variants, because other cost aspects such as cost of joints or labour costs were not included. A possible solution could be to link the output of the DRC to a database.

Linking the output of the DRC to a database would give the advantage that the costs are instantly visualised correctly. Shadow costs could be calculated in this manner as well if sustainability is important for the project. Databases exist for the life cycle analysis as was done in appendix C.2. The "Nationale Milieudatabase" would be an example for the Netherlands (https://milieudatabase.nl/).

The result is a better cost estimation and no time has to be spent calculating the amount of joints and structural elements. Further research is recommended to analyse how this can be done in practice and if it is better than the current method. It should also be analysed whether this information is needed in this design phase.

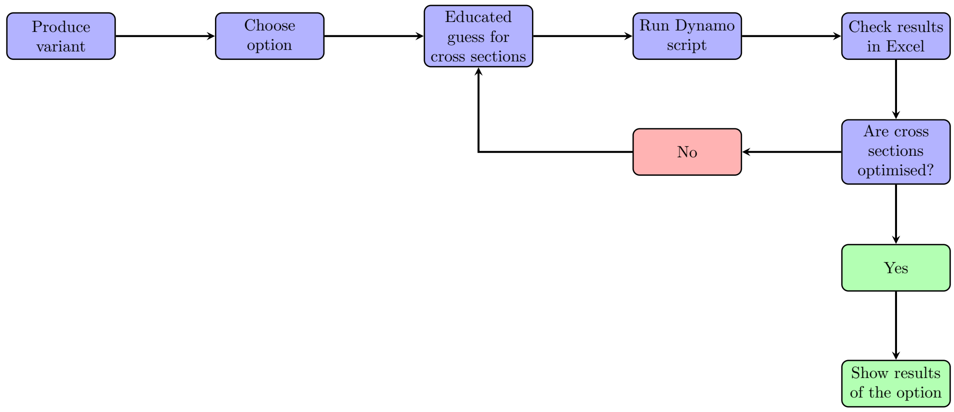


Figure 5.14: The applied method for the variant study as shown in chapter 4

Automatic generation of options
The possibility exists to automate the process of the variant study with the DRC (figure 5.14). There are two main issues to solve: the optimisation of cross sections and varying the parameters to generate new options.
The RFEM modules output the unity checks of the structural analysis. An algorithm could be developed in C# which is able to find the lowest possible cross section by using the unity checks of the SLS and ULS demands. This could be done in an iterative manner in a Zero Touch Dynamo node. However, the problem is simplified in this paragraph. Many issues will have to be solved for such a code. Automatic generation of options requires a function that can run the Dynamo script after a parameter is varied. According to the API documentation of dynamo, such a function exists (Autodesk, n.d.-a). A zero-touch node could be developed which is able to vary parameters and then run the script for a specified amount of times.

5.4.1 Additional aspects of parametric design

The literature study presented 14 different advantages of parametric design. Not all aspects have been added, but they could prove interesting for the DRC. The most interesting ones which could not be included in this research will be specified.

Multi disciplinary optimisation

Informing the architect of the support structure requires collaboration between the architect and the structural engineer. More professions are needed however to successfully design every aspect of a building. The DRC offers potential to include needs of other professions, which might lead to less conflict of interest. The architect would be better informed of the restrictions in the structure and can weigh the advantages and disadvantages more accurately.

What professions and what parts of these professions can be included should be further researched. More collaboration could result in a structure that is efficient for multiple disciplines.

SPD beyond the preliminary design phase

Advantage A5 gave the ability to adjust the parametric design when needed. This was performed for only the preliminary design, but the DRC could prove useful beyond it. Especially since the parametric model is already available.

One of the advantages in the literature study (#9) discussed that adjustments in the design could be possible throughout the whole construction process. This gives the ability to wait until certain choices have been made until alterations are made.

5.5 Conclusion

Positive and negative aspects
The goal of this chapter was to answer the following sub-questions.

1. Did the positive aspects of parametric design come forward in the variant study? Were there unexpected problems and how were they solved?
2. How does the DRC perform compared to the current design process? Does the theory of chapter 1 hold true in this case study?
3. What are the requirements to be able to use parametric design as was done in this thesis? In what way could other projects differ?
4. What other aspects of parametric design could've potentially be analysed which were outside the scope of this thesis?

Advantages and disadvantages
The following advantages were (partly) successfully implemented: Complex geometry (A1), endless variants (A2), reusable models (A4), adjustments to the design (A5), software for the structural engineer (A6).
Optimisation of cross sections (A3) was deemed to be unsuccessfully implemented due to the required amount of labour. However an unexpected advantage was implemented, namely that many demands can be considered at the start of the design stage.

The following disadvantages were mitigated: individuals work (D1), loss of flexibility (D2) and difficult to analyse all data of large models (D5).
Both time investment (D3) and specialised tools limit functionality (D4) are deemed unsuccessfully mitigated. However, the following two disadvantages were unexpectedly mitigated: difficult to analyse all data and suitability of the design.

The DRC compared to the contemporary process
The DRC was applied to improve the variant study in the preliminary design phase. This was done to mitigate the complications in contemporary structural engineering. Instead of finding new solutions, the DRC enhanced the current processes in the preliminary design phase. The benefit is that the current workflow of the structural engineer is less disrupted, which can help give the DRC a place within the current building process and thus make it more likely to be used.

The DRC is able to inform the architect of the following topics: costs of options, available cross sections for the support structure and possible positions of structural elements. The architect has more options to choose from which gives more freedom to choose the placement of structural elements and more information is made available. This may be used to better implement the demands of the SoR. The architect can take costs into account as well which may be used to better estimate the costs-appreciation ratio for design choices.

The DRC was able to inform the structural engineer about the following topics: the critical load combination, the type of failure and the unity checks for most eurocode demands. The modules made this information available for each structural element, which decreases the reliance on rules of thumb. More variants can be created for the variant study, more information can be obtained from them and by learning from previous variants more informed decisions can be made for subsequent variants.
This is an improvement compared to the hand calculations in appendix C, which required a few hours of labour to calculate. The geometry and mechanical models were simplified, cross sections were not optimised, eurocode demands were not yet considered, costs were not calculated and the effect of the rest of the structure was not taken into account. Altering parameters will require repeating most calculations as well.

Requirements of the DRC
It is not required to know everything about the DRC to use it and three types of users were described. However, knowledge of the finite element method is required.
A significant time investment is required. Therefore it should be considered for each project whether the additional information is needed and if it is worth the time investment.
Every structure is a unique product, but they do contain grid lines in the design. These can be used for the coordinate system of the translation by component method. However, this method heavily relies on repeating structural elements. A small amount of exceptions can be handled by the DRC, but too many unique elements can enlarge the Dynamo script considerably.
The size of the model should be limited and it is not recommended to use the DRC for a whole building. Instead it is advised to divide the structure and analyse the parts individually. This will require more time investment however.

Other aspects of parametric design
A large amount of additional functions can be added to the DRC. The following possible functions were shown: RFEM functions, RFEM modules, an advanced cost module and automatic generation of options. Other interesting aspects of parametric design were: multi disciplinary optimisation and SPD beyond the preliminary design phase.

6
Conclusion and recommendations

6.1 Conclusions

6.1.1 The main research question

The following main research question was defined for this research:

What are the possibilities of informing the architect of the support structure using parametric design with RFEM and Dynamo in the preliminary design phase?

The applied method for the DRC enhanced the process of the variant study by increasing the amount and complexity of variants. More information can be obtained and by learning from previous variants more informed decisions can be made for subsequent variants by the structural engineer.
The architect has more freedom to choose the placement of structural elements and information of costs is made available. This can be used to better implement the demands of the SoR. The architect can take costs earlier into account as well which can be used to better estimate the costs-appreciation ratio of design choices.

6.1.2 The DRC

Time investment

Time investment is the main limiting factor of the DRC. Therefore, it should be considered for each project whether the amount of information is worth the initial time investment. An indication can be given for the required time investment, which can be separated in five categories, namely: the initial time investment, modelling the first variant, modelling the subsequent variants, computation time of the script and performing the variant study.

- **Initial time investment:** Before the parametric model can be generated, information of the building relevant to the structural engineer should be collected. The goal, required parameters and desired results of the parametric model are to be defined and new functionality should be programmed for the DRC if required. The results of this research cannot be used to give a proper conclusion for the required time investment of this process.

- **The first variant:** The required geometry and functions are added to the script to create the first variant. The results of the finite element model should be verified next. This process took one to two weeks for the outer facade.

- **Subsequent variants:** The flexibility of the script allowed for modifications of the design. It is therefore possible to use available variants to create new variants. Figure 6.1a shows the time spent creating the subsequent variants for the Outer facade.

- **Script computation time:** Figure 6.1b shows the required computation time to produce an option. However, optimizing cross sections required three iterations on average. An option with optimized cross sections was generated in approximately 15 to 30 minutes.

- **The variant study:** The variant study can start after the first variant is created. the results of chapter 4 were obtained within two weeks.

Utilized variant → Created variant	Timing
Variant: floor span 7.2m → Variant: floor span 3.6m	3 hours
Variant: floor span 3.6m → Variant: Tie rods 3.6m	1 hour and 15 minutes
Variant: floor span 7.2m → Variant: Tie rods 7.2m	45 minutes
Variant: Tie rods 7.2m → Variant: Tie rods 7.2m V2	10 minutes
Variant: Tie rods 3.6m → Variant: Timber structure	30 minutes
Variant: floor span 3.6m → Variant: Concrete structure	1 hour

(a) Time spent creating variants

Variant	Span 7.2m	Span 3.6m	Tie rods 3.6m
Total time [minutes]	5:08	6:30	7:53
Members	763	910	1206
Total elements	3521	4233	4836
Load information	5 load cases 18 load combinations 1287 loads (line, nodal and surface)		

(b) Required computation time

Figure 6.1: Time investment

Model size
The size of the model will directly influence the time investment. It is therefore regarded as a limiting factor of the DRC as well.
The amount of members is included in figure 6.1b, because these influence the computation time the most for the Outer facade. Increasing the amount of members by a certain factor will approximately increase the computation time by the same factor.
This is not necessarily true for other scripts, because the computation time greatly depends on what nodes are utilized in Dynamo. It is therefore not possible to produce a general rule for the required computation time. However, the amount of loads and elements for each variant in figure 6.1b can be used to estimate the required computation time.

Additional requirements
Using the DRC has certain requirements, namely:

- **Knowledge of the finite element method** will always be necessary. However, full knowledge of the DRC is not needed to use the connection and the users can be separated into three groups. The *developer* can add functionality to the DRC, the *parametric designer* can use existing functionality to construct a parametric model and the *user* can generate options by using sliders.

- **Grid lines** can be used for the coordinate system of the TC-method. This method relies on repeating elements. More unique structural elements result in larger scripts and makes the proposed method less effective.

Functionality
The following can be concluded about the functionality of the DRC.

- It is possible to model complex geometry in the DRC, but setting up the first variant was error prone and laborious. For this reason it may take weeks to produce the first variant. This issue was reduced for subsequent variants by using previously made variants, which can be created within hours or less.

- Options can be produced with sliders while variants require manual labour. It is possible to create numerous variants with varying geometrical parameters, structural systems, materials and cross sections. Options however, are limited to the available sliders.

- Optimisation of cross sections is possible but the manual labour was laborious and time consuming. Professional code is used for the optimisation of cross sections. However, the TC-method does not necessarily optimise the design of the support structure.

- Basic blocks and modules are both reusable with proper documentation. It is possible to choose what functions to add due to the modular set-up of the script.

- The Dynamo script is flexible to changes due to the TC-method. This made it possible to make adjustments to the design. This does not apply when new points are needed in the basic block.

- Visualisation helps to mitigate errors and gives feedback of design decisions. This is less true for elements which are not directly viewable, such as hinges or internal forces. A larger model will make this process more time consuming and difficult as well, because results are not instantly visualised but depend on the computation time.

- It is possible to consider significantly more demands from the eurocode at the start of the design phase. This is done with professional code, which uses spreadsheets for calculations. The verification showed that the correct formulae and national annex are applied. Having this information early may prevent issues with ULS and SLS demands in future design phases.

- The TC-method simplifies building a variant to: select an element and translate it accordingly. By performing the same action, the script may become more understandable for other users.

- By outputting a summary on an Excel sheet, a large amount of data was visualised in a clear manner. Detailed information can be found in RFEM if necessary.

6.1.3 Influence on structural engineering

The following can be concluded about the influence of the DRC on contemporary structural engineering.

- **The architects design process:** the DRC allowed for detailed information, geometry dependent on other geometry, adjustments in the design and numerous amounts of options. This can be used to advise the architect at the start, during or end of the design process.

- **Cost management** will still be necessary. However, the DRC allows costs to be considered before the budget estimation takes place. Costs exceeding the budget can therefore be anticipated. Other options can be chosen if changes are deemed necessary, which may prevent the need to design new options or variants.

- **The tasks of the structural engineer** will not change when the DRC is applied. The DRC enhances the variant study by increasing the amount of variants as well as their complexity. More information can be extracted as well.

- **Rules of thumb** will be less necessary for cost efficiency. They can still be applied to the variants. Contradicting rules of thumb can be analysed in depth with the variant study.

- **In general:** The DRC improves the designing and budget estimation part of the preliminary phase. More options are given with more detailed information for the architect. The consequences of certain choices are therefore better known. Relevant information of the structural elements is available for the costs specialist as well. If the decision is made to adjust the design, then other options can be considered or new variants can be made if necessary.

6.1.4 The four complications

Four complications in structural engineering were discovered. The following can be concluded about mitigating these complications with the DRC.

Limited information
Limited amount of information at the start of the design stage was mitigated by allowing more variants and more complex variants to be generated at the start of the design stage. Thus, both the architect and the structural engineer were more informed.
The architect was informed of: the costs of options, the possible placements of structural elements and the cross sections. The advantages and disadvantages of each option can be compared, which may lead to enhanced quality and efficiency of the design.
The structural engineer was informed of: the critical load combination, the type of failure and the unity checks for each element. A summary is given for these checks, but detailed information can be found in RFEM. Therefore knowledge is increased about the mechanical behaviour of the structure, which may lead to more informed decisions for the support structure.

More complex buildings
Complex geometry can be considered with the DRC. When the geometry of the structure is set in the first variant, generating the subsequent new variants was less problematic because the first variant can be reused. Parameters which are laborious to analyse can now be analysed in depth with the DRC.
Complex projects can make it difficult to predict what structural element and what demand becomes critical. This issue is mitigated with the DRC by analysing every structural element and demand. Rules of thumb will therefore become less necessary.

Insufficient cost management
The DRC introduced material costs for each option. This information is made available during the design part of the preliminary design phase. The consequences of certain design choices on material costs are therefore visualised. The architect can anticipate on costs exceeding budget. Giving more attention to costs may lead to more efficiency or quality of the structure.

6.2 Recommendations

Recommendations can be given based on the outcome of this research. These shall be divided into recommendations for users and recommendations for improving the DRC.

Users

- The RFEM model should be kept simple to ensure that it correctly models reality. Complex geometries can be taken into account with a simpler finite element model.
- Proper documentation should always be made of the modules or basic blocks to make it reusable. This data should be well organized to ensure other users can find the needed information.
- What parameters are wanted and how they should vary should be known before the Dynamo script is made. The basic block greatly depends on this information.
- Models larger than the outer facade are not recommended. An alternative option could be to divide the structure in smaller parts. Making scripts for each part will require an even larger time investment though.
- Using the DRC should be properly justified. One should verify whether the amount of information output is necessary for the project and if it is worth the time investment.

DRC improvements

- Develop automatic optimisation of cross sections to greatly reduce the required manual labour.
- The research of this thesis was done for the preliminary design phase. Analysing whether the functions of the DRC can be applied in subsequent phases may prove to be useful.
- Developing an external UI for the coordinate system of the TC-method is advised. This program can help make the most difficult concept and error prone part of the TC-method more understandable.
- The code of the Zero Touch nodes was not optimised. It is therefore recommended to increase the efficiency of the code to reduce runtime.
- A large amount of additional RFEM functions can be added to the DRC. It is not necessary to add all functions at the same time. It could be done per project to ensure that the additional functionality will be used.

- A more in depth cost calculation module is recommended to better inform the architect of the costs. Currently only material costs were approximated. Other costs, such as joints or labour costs, give the architect the ability to better weigh his options. A more in depth calculation of the shadow price is possible as well.

- Time investment is a considerable issue for the DRC. Automating the generation of options is possible and it is recommended to implement it before using the DRC in practice. It will significantly reduce the manual labour required, which will then only be necessary for making new variants.

- The DRC offers the potential to include the demands of other professions. This may lead to less conflicts of interest and a structure that is efficient for multiple disciplines. It is advised to research for which professions this applies and how their demands can successfully be implemented.

Bibliography

Aish, R., & Hanna, S. (2017). (in press). comparative evaluation of parametric design systems for teaching design computation. *Design Studies*. Retrieved from http://dx.doi.org/10.1016/j.destud.2017.05.002

Autodesk. (n.d.-a). Dynamo api documentation. Retrieved from http://dynamods.github.io/DynamoAPI/

Autodesk. (n.d.-b). Structures and design modules. Retrieved from https://knowledge.autodesk.com/support/robot-structural-analysis-products/learn-explore/caas/CloudHelp/cloudhelp/2017/ENU/RSAPRO-UsersGuide/files/GUID-19198EA8-EDD6-49E2-B9AE-DD0B38261C11-htm.html

BNA, & NLingenieurs. (2014a). *Standaardtaakbeschrijving - complete inhoud fase voorontwerp*.

BNA, & NLingenieurs. (2014b). *Standaardtaakbeschrijving dnr-stb 2014 toelichting*.

Bongers, G., & Mast, R. (2018). Modulair parametrisch ontwerpen: Praktische benadering van een bouwbedrijf. *Cement 2018/6 parametrisch ontwerpen (1)*.

BRIS BV. (2018). *Bouwbesluit 2012*.

Circular Ecology. (n.d.). Embodied energy and carbon - the ice database. Retrieved from http://www.circularecology.com/embodied-energy-and-carbon-footprint-database.html#.XN6lB-SP4fs

Coenders, J. [J.]. (2009). Parametric and associative design as a strategy for conceptual design and delivery to bim. (2009).

Coenders, J. [Jeroen]. (2018). Parametrisch ontwerp? wat is dat? *Cement 2018/6 parametrisch ontwerpen (1)*.

Dlubal. (n.d.-a). Programmable com interface for rfem/rstab. Retrieved from https://www.dlubal.com/en/products/rfem-and-rstab-add-on-modules/others/rf-com

Dlubal. (n.d.-b). Rf-/steel ec3 add-on module for rfem/rstab. Retrieved from https://www.dlubal.com/en/products/rfem-and-rstab-add-on-modules/steel-and-aluminium-structures/rf-steel-ec3

Dlubal Software GmbH. (2018). *Rf-com*.

Gemeente Amsterdam. (2004). Handleiding daktuinen. Retrieved from https://www.publicspaceinfo.nl/bibliotheek/handleiding-daktuinen/

Gkioka, M. (2018). *Knowledge model for the parametric design of concrete viaducts* (master thesis, Delft University of Technology).

Gyproc SAINT-GOBAIN. (n.d.). 6 scheidingswanden op houten frame. Retrieved from http://www.gyproc.be/webobjbpb/PDF/Scheidingswanden/houtenframeNL.pdf

Hammond, G., & Jones, C. (2011). Inventory of carbon & energy (ice) version 2.0. Retrieved from http://www.circularecology.com/embodied-energy-and-carbon-footprint-database.html#.XN6lB-SP4fs

Hordijk, D., Braam, R., Lagendijk, P., van Loenhout, M., Linssen, J., Sterken, R., ... Wapperom, H. (Eds.). (2018a). Cement 2018/6 parametrisch ontwerpen (1).

Hordijk, D., Braam, R., Lagendijk, P., van Loenhout, M., Linssen, J., Sterken, R., ... Wapperom, H. (Eds.). (2018b). Cement 2018/7 parametrisch ontwerpen (2).

Horikx, M. (2017). *A methodical approach on conceptual structural design* (Ph.D. dissertation, Delft University of Technology).

IJzermagazijn. (n.d.). Draagkracht tabellen. Retrieved from http://www.ijzermagazijn.nl/content/11-draagkracht-tabellen

Kamerling, J. W., & Kamerling, M. W. (1997). *Jellema hogere bouwkunde 9 utiliteitsbouw*. Leiden: SPRUYT, VAN MENTGEM & DE DOES BV.

Kamerling, J. W., & Kamerling, M. W. (2011). *Jellema hogere bouwkunde 9 utiliteitsbouw*. Amersfoort: ThiemeMeulenhoff.

Lee, C. (2010). Woven space truss and morphogenetic structural flow/form models. Retrieved from https://biosarch.wordpress.com/2010/01/04/woven-space-truss-and-morphogenetic-structural-flowform-models/

Linssen, J. (2018). De scope van parametrisch ontwerpen. *Cement 2018/6 parametrisch ontwerpen (1)*.

MetsäWood. (n.d.). *Kerto for load-bearing structures*. Retrieved from https://www.metsawood.com/nl/producten/Kerto/toepassingen/Pages/Prefab%5C-houtelementen.aspx

Nederlands Normalisatie-instituut. (2011a). *Eurocode 1: Actions on structures - part 1-1: General actions - densities, self-weight, imposed loads for buildings*.

Nederlands Normalisatie-instituut. (2011b). *Eurocode 1: Actions on structures - part 1-3: General actions - snow loads*.

Nederlands Normalisatie-instituut. (2011c). *Eurocode 1: Actions on structures - part 1-4: General actions - wind actions*.

Nederlands Normalisatie-instituut. (2011d). *Eurocode: Basis of structural design*.

Nederlands Normalisatie-instituut. (2016). *Eurocode 3: Design of steel structures - part 1-1: General rules and rules for buildings*.

Ram Caddsys. (n.d.). Rfem - powerful, easy to use and intuitive software. Retrieved from https://ramcadds.in/software/rfem/

Ramsden, J. (2013). Data trees and c# in grasshopper. Retrieved from http://james-ramsden.com/data-trees-and-c-in-grasshopper/

Rocel Nederland BV. (n.d.). *Schuifpuisystemen mb-59hs*. Retrieved from https://rocel.nl/aannemer-kozijnspecialist/aluminium/hefschuifpui/?gclid=EAIaIQobChMItZ3Sq8XZ4gIV6Z3tCh3w4AvDEAAYASAAEgK6wPD_BwE

Saint-Gobain Gyproc. (n.d.-a). Gyproc adviseert voor hotel - gangwanden. Retrieved from https://www.gyproc.nl/systemen/zoeken?f[0]=field_toepassingen_taxonomie%3A2896

Saint-Gobain Gyproc. (n.d.-b). Gyproc adviseert voor hotel - wanden hotelkamerscheidend. Retrieved from https://www.gyproc.nl/systemen/zoeken?f[0]=field_toepassingen_taxonomie%3A2893

Schöck Ltd. (2014). Schöck isokorb type kst user selection guide. Retrieved from https://www.schoeck.co.uk/en%5C-gb/isokorb-kst%5C#category

SCIA. (n.d.). Scia engineer - edities. Retrieved from https://www.scia.net/nl/software/scia-engineer/editions

Spierings, T. G. M., van Amerongen, R. P., & Bakker, W. J. (1998). *Jellema hogere bouwkunde 3 draagstructuur*. Leiden: SPRUYT, VAN MENTGEM & DE DOES BV.

Stichting Bouwkwaliteit. (2014). *Bepalingsmethode milieuprestatie gebouwen en gww-werken*.

Urban Climate Architects. (2017a). A building that moves with you. Retrieved from http://www.ucarchitects.com/architecture/de-zwerm-aalsmeer/

Urban Climate Architects. (2017b). *Projectdocumentatie hotel de zwerm*. Aalsmeer.

van der Aa, P., & van den Bos, A. (2018a). De keuze voor parametrisch ontwerpen: Afwegingen met veel invloedsfactoren. *Cement 2018/7 parametrisch ontwerpen (2)*.

van der Aa, P., & van den Bos, A. (2018b). Parametrisch ontwerpen in diana: Niet-lineaire berekeningen automatisch aangestuurd. *Cement 2018/6 parametrisch ontwerpen (1)*.

van der Linden, L. P. L. (2018). *Parametric engineering*. Lecture Notes TU Delft version 1.0.

van der Ploeg, C. (2018). De keuze voor parametrisch ontwerpen: Afwegingen met veel invloedsfactoren. *Cement 2018/7 parametrisch ontwerpen (2)*.

van Eekelen, A. L. M., Rip, J. J., & Wentzel, P. L. (2002). *Jellema hogere bouwkunde bouwproces 10 ontwerpen*. Amersfoort: ThiemeMeulenhoff.

van Telgen, M. V. (2018). *Computation at sd&e*. Unpublished internal document, Rotterdam.

van Telgen, M. (2018). Parametrisch model gekoppeld aan rekenprogramma: Voorbeeld van koppeling tussen dynamo en rfem. *Cement 2018/6 parametrisch ontwerpen (1)*.

VBI. (n.d.). *Kanaalplaatvloer 260*. Retrieved from https://vbi.nl/downloads/

Veenendaal, D. (2018). De rol van parametrisch modelleren in het ontwerpproces: Een nieuwe manier van denken. *Cement 2018/6 parametrisch ontwerpen (1)*.

Vissering, C., & van Loenhout, M. (2018). Stuurt constructeur software of acteert deze zelf? *Cement 2018/8*.

Walraven, J. C., & Fennis, S. A. A. M. (2013). *Gewapend beton*. Lecture Notes TU Delft.

Xtreme Quality, Service and Innovation. (n.d.). Triangle truss. Retrieved from https://www.xsftruss.com/triangle-truss

Yeager, A. (2016). *The role of the structural engineer in the contemporary design process* (bachelor thesis, Syracuse University). Retrieved from https://surface.syr.edu/honors_capstone/922

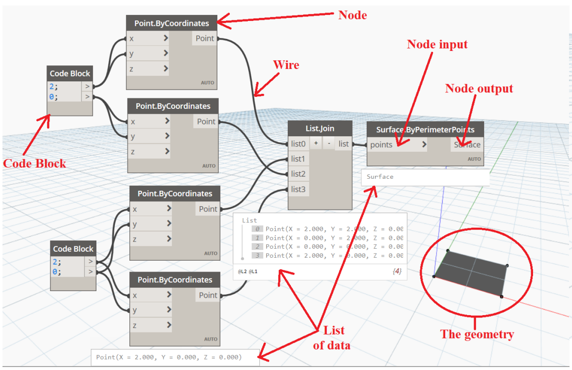
Appendices

A

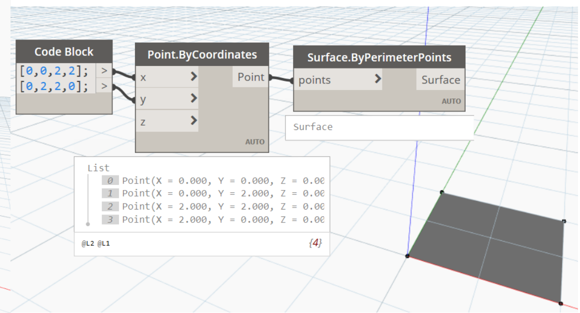
The DRC

A.1 An introduction to Autodesk Dynamo

Dynamo is a visual programming tool by Autodesk similar to Grasshopper 3D. Instead of typing code, one can create programs by connecting graphical elements called "nodes". Nodes usually contain an input and output and are connected by using wires. Each node has its own functionality and will store the data within a list. For example: in figure A.1a the Point.ByCoordinates node creates a point at a given X, Y and Z coordinate. This point is stored within a list inside this node and can then be output through a wire. The coordinates can easily be changed by modifying the values in the code block. A code block is a node where one can type code, which can make the script clearer if used correctly.



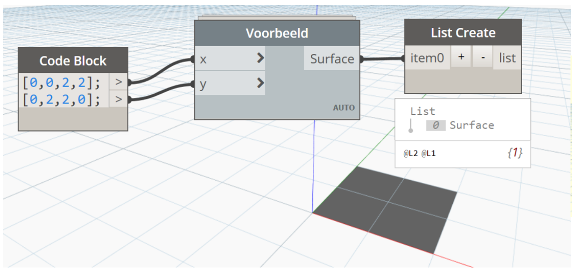
(a) Definitions in dynamo



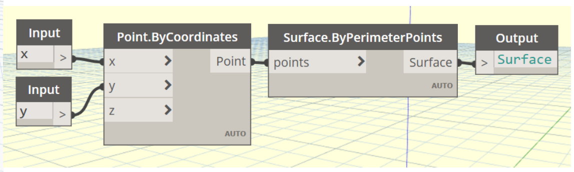
(b) Simplified way to build a surface

Figure A.1: Building a surface in Dynamo

The geometry build with the nodes will be displayed in the background almost instantly, which makes it easy to check if mistakes were made. Because of the instant visualisation of the output, one can use visual programming without the need of knowing how to code. The interface is intuitive enough that a functional program can be build by experimenting with the nodes.
Through experience one will be able to set up programs more efficiently by using less nodes. Figure A.1a shows how a beginner would build a surface. Eight nodes were needed to build one surface. The same code with different coordinates can be used to build more surfaces, but this would mean that for each surface an additional 8 nodes are needed. An overview can still be maintained for a small number of surfaces, but one can imagine this will not be the case if 250 surfaces are needed.
Figure A.1b shows a program that builds the same geometry, using only three nodes. There are usually different ways to create the same geometry, but as a rule of thumb one should strive to use the least amount of nodes possible. This makes the code easier to read and understand, which becomes important when huge amounts of data flow through the nodes as lists. These lists must be carefully monitored and to truly use the full capability of Dynamo one must understand how to manage lists and how nodes influence them.
Another reason for the rule of thumb is that each node does a calculation which costs computation time. This differs per node and what its input is. It is difficult to predict which node is more efficient or how large the program must be before the computation time becomes noticeable.



(a) Custom node called "Voorbeeld"



(b) The functionality of the custom node

Figure A.2: Building a surface in Dynamo

Sometimes a problem will occur which cannot be solved with the use of standard nodes. Extra functionality can be obtained with a different type of node called "custom node". In this thesis the following custom nodes are used: "code node", "Dynamo Custom Node" and "Zero Touch Node". A code node can be seen in figure A.1 which is called a code block. This block can be used to write in Design Script, which are functions that performs the same actions as the standard Dynamo nodes. These are mainly used in this thesis to shorten the program or to make lists.
A Dynamo Custom Node, named "Voorbeeld", can be seen in figure A.2a. This node behaves just like any other node, but its functionality can be build within dynamo as seen in figure A.2b. This custom node generates the same surface as

shown before. One should be careful with these nodes because overusing them can cost considerable computation time. Sometimes it is more efficient to keep the original script instead of dividing it into different Dynamo Custom Nodes.
The Zero Touch Nodes can be build with C# code for more complex functions. In this thesis these nodes are used to communicate with RFEM. An explanation of how this works is given in section 2.4.

A.2 Zero Touch node Code example

The following code gives an example of how a line is exported to RFEM (M. van Telgen, 2018).

```
using System;
using System.Collections.Generic;
using System.Runtime.InteropServices;
using Dlubal.RFEM5;



namespace StructureToRFEM

{

public static class Cement_Line_Export

{
//DynamoLineList contains the information for geometry of lines present in
    Dynamo
public static void CementLijnRFEM(List DynamoLineList)

{

// 1) Start sending data to RFEM

IModel model = Marshal.GetActiveObject("RFEM5.Model") as IModel;

model.GetApplication().LockLicense();

IModelData data = model.GetModelData();

data.PrepareModification();


// 2) Make a new variabel for nodes and lines in RFEM

Node RFEMnode = new Node();

Line RFEMline = new Line();


// 3) Write the lines and nodes to RFEM

int StartPoint = 1;
```

```
int EindPoint = 2;
int LineNumber = 1;

foreach (Autodesk.DesignScript.Geometry.Curve DynamoLine in DynamoLineList)
{
//Startpoint
RFEMnode.No = StartPoint;
RFEMnode.Type = NodeType.Standard;
RFEMnode.RefObjectNo = 0;
RFEMnode.CS = CoordinateSystemType.Cartesian;
RFEMnode.X = DynamoLine.StartPoint.X;
RFEMnode.Y = DynamoLine.StartPoint.Y;
RFEMnode.Z = DynamoLine.StartPoint.Z;
data.SetNode(RFEMnode);
StartPunt += 2;

//Eindpoint
RFEMnode.No = EindPoint;
RFEMnode.Type = NodeType.Standard;
RFEMnode.RefObjectNo = 0;
RFEMnode.CS = CoordinateSystemType.Cartesian;
RFEMnode.X = DynamoLine.EndPoint.X;
RFEMnode.Y = DynamoLine.EndPoint.Y;
RFEMnode.Z = DynamoLine.EndPoint.Z;
data.SetNode(RFEMnode);
```

```
EindPunt += 2;


//Line

RFEMline.Type = LineType.PolylineType;

RFEMline.No = LineNumber;

RFEMline.NodeList = Convert.ToString(StartPoint) + "," +
    Convert.ToString(EindPoint);

data.SetLine(RFEMline);

LineNumber += 1;


}


// 4) Finish sending data to RFEM

data.FinishModification();

model.GetApplication().UnlockLicense();

model = null;

System.GC.Collect();

System.GC.WaitForPendingFinalizers();

}
}
}
```

A.3 Dynamo Zero Touch nodes

The following two figures show the available Zero Touch nodes and their inputs to show what functions are available to the tool. The nodes in red are modified by the author.

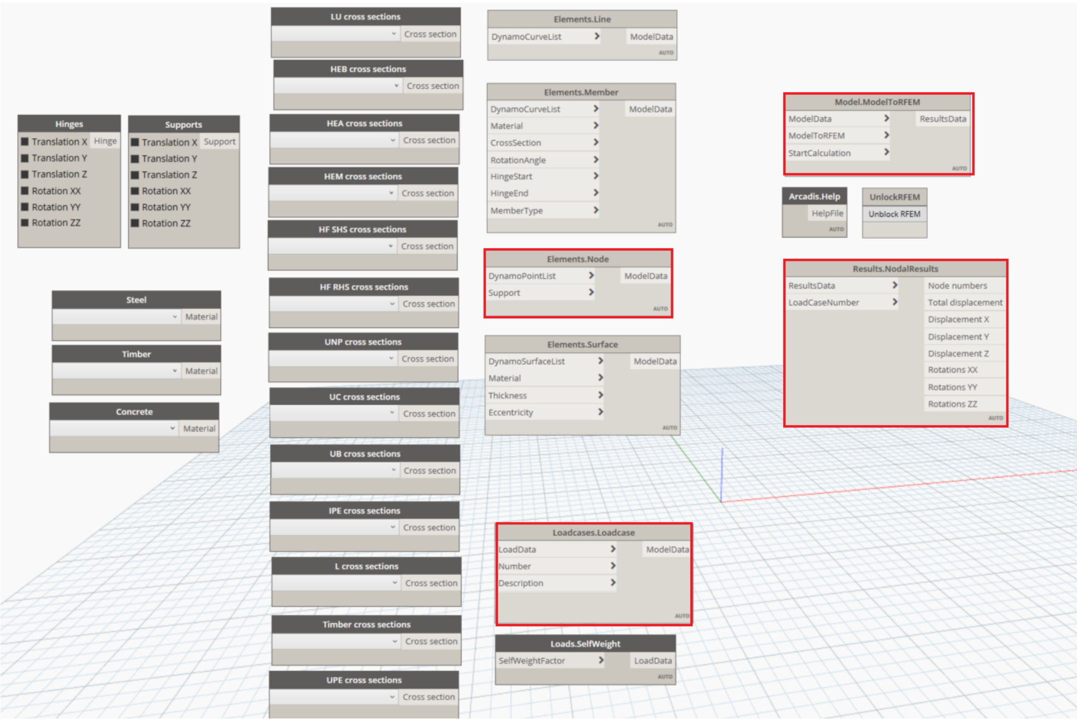


Figure A.3: Available Zero Touch Nodes in the DRC (January 2019)

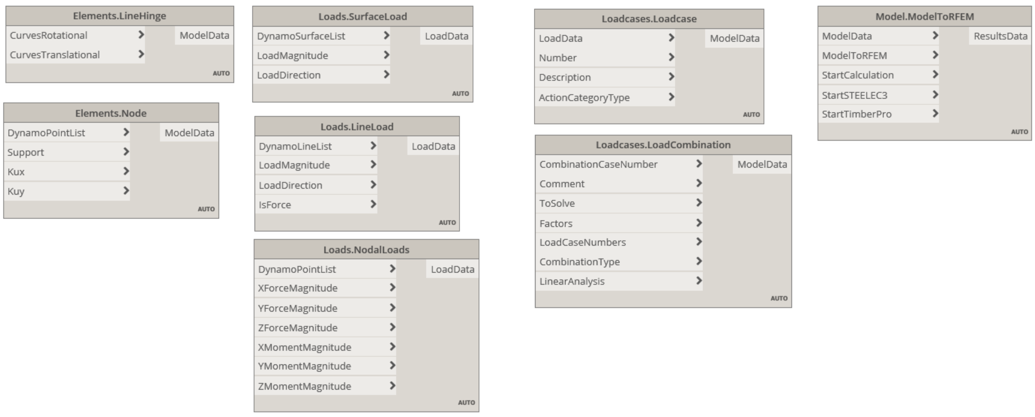


Figure A.4: Zero Touch nodes (elements and loads) reworked or added by author of this thesis

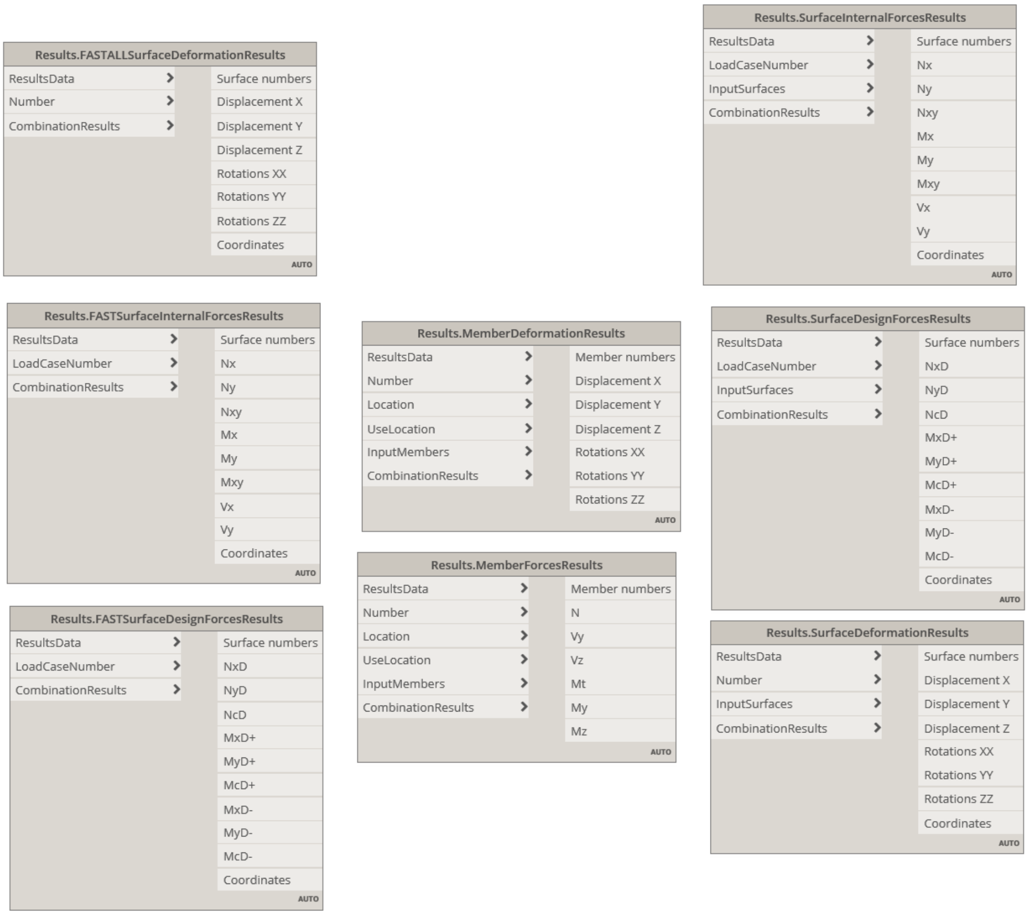


Figure A.5: Zero Touch nodes (results) reworked or added by author of this thesis

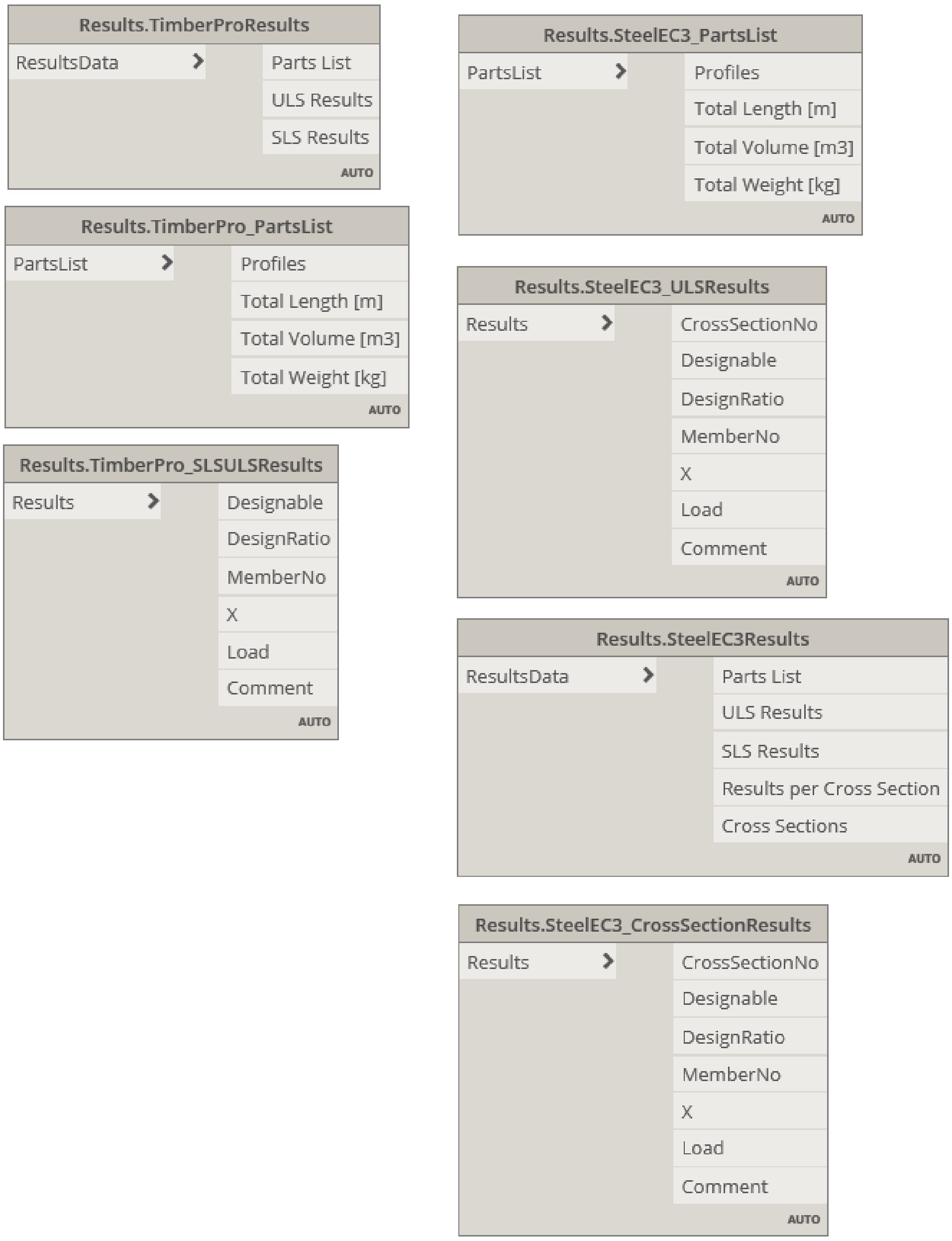


Figure A.6: Zero Touch nodes (modules) reworked or added by author of this thesis

B
The Eurocode

B.1 Eurocode

The Eurocode is used in this thesis for the analysis of the structure. This subsection is used to show the Eurocode that can be used for any general structure. These are the loads, SLS criteria, safety class and its $\psi - factor$ and load combinations. NEN-EN 1990 (Nederlands Normalisatie-instituut, 2011d), NEN-EN 1991-1-1 (Nederlands Normalisatie-instituut, 2011b) and NEN-EN 1991-1-3 (Nederlands Normalisatie-instituut, 2011a) and the Dutch national Annex (DNA) are used in this subsection. Eurocodes relevant to a structure with a specific materials will be shown in Appendix B. Since the analysis is meant to simulate the preliminary phase, not everything in the Eurocode has to be checked yet, therefore the following aspects are taken into consideration.

Loading: (NEN-EN1991-1-1)
According to table 6.1 hotels have the use class A, which corresponds to the following imposed loads:

- Floors (q_A): $q_k = 1.75kN/m^2$ $Q_k = 3.0kN$
- Balconies (q_A): $q_k = 2.5kN/m^2$ $Q_k = 3.0kN$

The national annex mentions in 6.3.1.2 (11) that it is possible to set the full weight on two floors and use the factor ϕ_0 for the loads on the other floors. This is only true for structures with more than two floors and does not apply to roofs.

It will be assumed that all roofs will not be accessible and are not sloped. This coincides with class H in table 6.9 and the load(Dutch annex table 6.10):

- Roof (q_R): $q_k = 1.0kN/m^2$ $Q_k = 1.5kN$

The densities for common materials are given as: $\rho_{concrete} = 25kN/m^3$ and $\rho_{steel} = 77 - 78.5kN/m^3$. This coincides with the values used in RFEM. The following is found by comparing steel profiles in practice to profiles in RFEM (IJzermagazijn, n.d.), two are shown:

- HEA 500: Practice:$158kg/m$ RFEM:$155.04kg/m$
- IPE 240: Practice:$31.3kg/m$ RFEM:$30.69kg/m$

These values are deemed close enough to be realistic.

Finishing layer
A finishing layer is usually necessary for floors. These are 50mm for thin pipes and 100mm for thick pipes (Spierings et al., 1998, p.128). An assumption will be used that the finishing layer of 75mm is used for all floors. Its weight can be calculated with:

$$G_{fl} = \rho_{concrete} \cdot h = 25 * 0.075 = 1.875kN/m^2 \tag{B.1}$$

Green/living roof
The balconies have an additional green roof that is not walkable. This part of the balcony will be considered as a roof for the variable loads. The permanent loads will be the same as the rest of the balcony except for the additional load of the grass roof. This weight can vary between 130 and 220 kg/m^2 for a grassy roof with 10-15mm substrate, which includes a 10mm water weight (Gemeente Amsterdam, 2004). An average value of 175 kg/m^2 is used which results in a **surface load of** $1.7\ kN/m^2$.

Partitioning walls
The weight of the partitioning walls will be considered as line loads on the support structure. An estimation of the weight has been made by taking the average weight of partitioning walls for hotels (Saint-Gobain Gyproc, n.d.-b). The walls separating the hotel room from the hallway has approximately the same weight (Saint-Gobain Gyproc, n.d.-a). The weight ranges from 45 to 50 kg, therefore a **line load of 1.3 kN/m** is assumed. This value is calculated by assuming that the wall will be the same height of a floor (2.9m). This is not true due to the fact that the wall will not penetrate the floor, but it will be a conservative value.
The boundary between balcony and indoor floor is separated by a sliding door. A reference sliding door is used which weighs $32.5/kg/m^2$ (Rocel Nederland BV, n.d.). A floor height of 2.9 m results in a **line load of 0.9 kN/m**.

balcony walls
It is assumed that the walls on the balcony are not used for structural purposes due to the fact that the balcony is cantilevered. Figure B.1 was therefore used to find a wall thickness of 200mm. Using the Eurocode density of reinforced concrete ($25kN/m^3$) and a height of 2.9m, one will find a line load of 14.5 kN/m. This is too large especially for the required cantilever of the structure.

Therefore another option is analysed using a timber frame instead of concrete plates. Figure B.1 shows that 500 mm space is left for the partitioning wall on the balcony. A timber frame structure is therefore chosen for its light weight properties. A reference timber frame is chosen to estimate the loads of such a structure with a thickness of 115mm which weighs 26 kg/m^2 (Gyproc SAINT-GOBAIN, n.d.). Two such frames can be used per partitioning wall to get a shape similar to the concrete wall in figure B.1. This preserve the aesthetics of the architect. This results in a weight of 52 kg/m^2, resulting in an additional **line load of 1.5 kN/m on the balcony**.

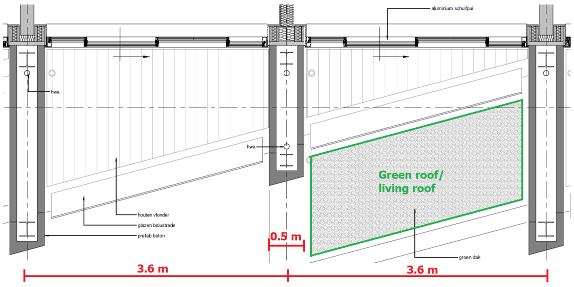


Figure B.1: Balcony design by the architect. From: (Urban Climate Architects, 2017b)

Snow load: (NEN-EN1991-1-3)
This document describes snow loads, which will be present on all roofs. Two types of snow loads are considered: snow on roofs and roofs adjacent to a higher roof.

$$\begin{aligned}
&\alpha = 0 \longrightarrow \mu_i = 0.8 && \text{((Table 5.2))}\\
&C_e = 1.0 && \text{(DNA: 5.2 (7))}\\
&C_t = 1.0 && \text{(DNA: 5.2 (8))}\\
&S_k = 0.7kN/m^2 && \text{(DNA: 4.1 (1))}\\
&q_{snow} = \mu_1 \cdot C_e \cdot C_t \cdot S_k = 0.56kN/m^2 && \text{(eq. 5.2)}
\end{aligned} \tag{B.2}$$

For roofs adjacent to other roofs the following applies:

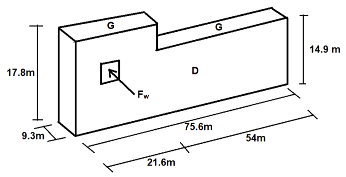
$$\begin{aligned}
&\mu_2 = \mu_s + \mu_w && \text{(eq. 5.7)}\\
&\alpha < 15^o \longrightarrow \mu_s = 0 && \text{(eq. 5.8)}\\
&\mu_w = 4.0\ conservative && \text{(5.3.1 (1))}\\
&q_{snow} = \mu_2 \cdot C_e \cdot C_t \cdot S_k = 2.80kN/m^2 && \text{(eq. 5.2)}
\end{aligned} \tag{B.3}$$

Wind loads: (NEN-EN1991-1-4)
The extreme wind pressure (q_p) was calculated for the Netherlands area II, in an undeveloped site for a structure with a height of 23.6m (the highest point in the building). A value of $q_p = 1.07kN/m^2$ was found which coincides with table NB.5 (Nederlands Normalisatie-instituut, 2011c).
The structure is schematised as a block to model the wind loads (figure B.2a). This is only a part of the whole structure, therefore only the wind force in one direction

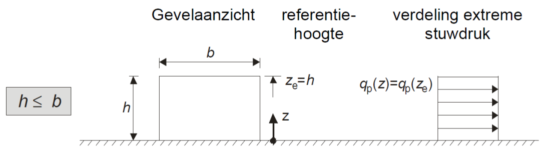
on zone D will be considered and the roof will be schematised to a zone G. The roof should also have a zone F and H, but the complex shape of the roof makes it difficult to implement these zones. The following loads are found:

$$\begin{aligned}
&c_s c_d = 1 \longrightarrow 6.2(1)c \\
&\text{Zone D: } q_{p,D} = C_{pe,10D} \cdot q_p = 0.8 \cdot 1.07 = 0.856 kN/m^2 \longrightarrow 7.2.2(DNA) \\
&\text{Zone G: } q_{p,G} = C_{pe,10G} \cdot q_p = -1.2 \cdot 1.07 = -1.284 kN/m^2 \longrightarrow 7.2.3(DNA) \\
&\text{friction } q_{p,fr} = c_f \cdot q_p = 0.04 \cdot 1.07 = 0.043 kN/m^2 \longrightarrow 7.5
\end{aligned} \tag{B.4}$$

The loads in zone D will be schematised into point loads on the beams of the support structure by multiplying the pressure by the area that will affect the beam. This force is called F_W. The friction on the green roofs will not be considered due to the fact that these are adjacent to a wall.



(a) The simplified structure



(b) The wind forces on the structure. From: (Nederlands Normalisatie-instituut, 2011c)

Figure B.2: Placement of the loads

Spring stiffness

According to NEN-EN1990, the horizontal deflection must not be more than the height of the structure divided by 300 for the characteristic combination. The stiffness of the spring will be calculated assuming the structure will be just stiff enough to hold to this demand. This could be considered a conservative assumption. The stiffness of the spring can be calculated (as seen in figure B.3) with the following formula:

$$\begin{aligned}
&k_i = \frac{F_{w,i}}{u_i} \\
&F_{W,floor1} = q_{p,D} \cdot b \cdot h = 0.856 \cdot 57.6 \cdot 2.9 = 143.0 kN \\
&F_{W,floor234} = q_{p,D} \cdot b \cdot h = 0.856 \cdot 75.6 \cdot 2.9 = 187.7 kN \\
&F_{W,floor5} = q_{p,D} \cdot b_{3rooms} \cdot h + c_f \cdot A_{roof} + q_{p,D} \cdot b_{9rooms} \cdot 0.5 \cdot h = \\
&0.856 \cdot 21.6 \cdot 2.9 + 0.04 \cdot 9.3 \cdot 54 + 0.856 \cdot 54 \cdot 0.5 \cdot 2.9 = 140.7 kN \\
&F_{W,floor6} = q_{p,D} \cdot b_{3rooms} \cdot 0.5 \cdot h + c_f \cdot A_{roof} = \\
&0.856 \cdot 21.6 \cdot 0.5 \cdot 2.9 + 0.04 \cdot 9.3 \cdot 21.6 = 34.8 kN \\
&H = 23.6 m
\end{aligned} \tag{B.5}$$

The following stiffness is found for floors 1 to 5. The stiffness should be divided by the amount of springs on the given floor to find the spring stiffness of a single spring, which is a variable.

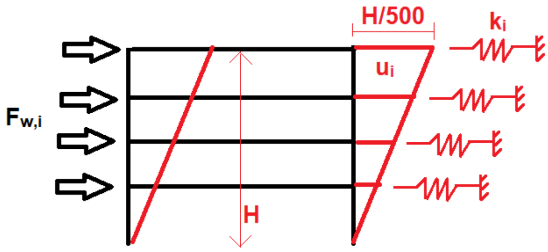


Figure B.3: Schematisation of the horizontal deformations

$$\begin{aligned} k_1 &= 21667kN/m \\ k_2 &= 15137kN/m \\ k_3 &= 10313kN/m \\ k_4 &= 7821kN/m \\ k_5 &= 4722kN/m \\ k_6 &= 1024kN/m \end{aligned} \tag{B.6}$$

Safety class and load combinations: (NEN-EN1990)

The safety class and the ψ factors are found in this document. Table NB.20-B1 (DNA) states that hotels belong to the CC2 class. Table 2.1 states that the structure should have a design lifetime of 50 years, which corresponds to design class 4. The ψ factors from table NB.2 -A1.1 state the following relevant values:

$$\begin{array}{llll} \text{Class A:} & \psi_0 = 0.4 & \psi_1 = 0.5 & \psi_2 = 0.3 \\ \text{Class H:} & \psi_0 = 0 & \psi_1 = 0 & \psi_2 = 0 \\ \text{Snow:} & \psi_0 = 0 & \psi_1 = 0.2 & \psi_2 = 0 \\ \text{Wind:} & \psi_0 = 0 & \psi_1 = 0.2 & \psi_2 = 0 \end{array} \tag{B.7}$$

For the ULS check, the following load combinations are needed (DNA: Table NB.4 – A1.2(B)):

$$\begin{aligned} &1.35G_{k,j} + 1.5\psi_{0,i}\, Q_{k_i} \longrightarrow \text{(unfavourable G)} \\ &0.8G_{k,j} + 1.5\psi_{0,i}Q_{k_i} \longrightarrow \text{(favourable G)} \\ &1.2G_{k,j} + 1.5Q_{k_i} + 1.5\psi_{0,i}Q_{k_i} \longrightarrow \text{(unfavourable G)} \\ &0.9G_{k,j} + 1.5Q_{k_i} + 1.5\psi_{0,i}Q_{k_i} \longrightarrow \text{(favourable G)} \end{aligned} \tag{B.8}$$

These coincide with the following ULS load combinations.

Unfavourable		**favourable**	
1	$1.35G + 0.6q_A$	**6**	$0.8G + 0.6q_A$
2	$1.2G + 1.5q_A$	**7**	$0.9G + 1.5q_A$
3	$1.2G + 1.5q_R + 0.6q_A$	**8**	$0.9G + 1.5q_R + 0.6q_A$
4	$1.2G + 1.5q_s + 0.6q_A$	**9**	$0.9G + 1.5q_s + 0.6q_A$
5	$1.2G + 1.5q_w + 0.6q_A$	**10**	$0.9G + 1.5q_w + 0.6q_A$

(B.9)

The SLS check requires the following load combinations:

$$\begin{array}{ll} \text{Characteristic:} & G_{k,j} + Q_{k_1} + \psi_{0,i}\, Q_{k_i} \\ \text{Frequent:} & G_{k,j} + \psi_{1,1}\, Q_{k_1} + \psi_{2,i}\, Q_{k_i} \\ \text{Quasi-permanent:} & G_{k,j} + \psi_{2,i}\, Q_{k_i} \end{array} \tag{B.10}$$

These coincide with the following SLS load combinations.

Characteristic

11 $G + q_A$

12 $G + q_R + 0.4q_A$

13 $G + q_s + 0.4q_A$

14 $G + q_w + 0.4q_A$

Frequent

15 $G + 0.5q_A$

16 $G + 0.2q_s + 0.3q_A$

17 $G + 0.2q_w + 0.3q_A$

Quasi-permanent

18 $G + 0.3q_A$

(B.11)

The deflections: (NEN-EN1990)

There are no additional codes for deflection applicable to this structure according to NEN-EN1993-1-1 and NEN-EN1992-1-1. For this reason the following demands for deflection will be used for each structure:

Simply supported: $l_{rep} = l$ Cantilevered: $l_{rep} = l * 2$

Floors with crack sensetive walls (frequent combination):

$w_2 + w_3 \leq \frac{l_{rep}}{500}$ (DNA: A1.4.3(3))

Floors frequently walked on (frequent combination):

$w_2 + w_3 \leq \frac{3 \cdot l_{rep}}{1000}$ (DNA: A1.4.3(3))

Roofs (characteristic combination)

$w_2 + w_3 \leq \frac{l_{rep}}{250}$ (DNA: A1.4.3(3))

Viewable floors and roofs (Quasi-permanent combination):

$w_1 + w_2 + w_3 \leq \frac{l_{rep}}{250}$ (DNA: A1.4.3(4))

Horizontal translation whole structure (characteristic):

$w \leq \frac{h}{500}$ (DNA: A1.4.3(7))

(B.12)

B.2 Steel EC3 Eurocode check

One of the modules in RFEM used in this thesis is called: RF STEEL EC3. This module is used to automatically check and optimise the steel columns and beams of a structure according to the Eurocode of a given country. It can take the national annex into consideration as well. Eurocode 1993-1-1 and Eurocode 1990 with the Dutch national annex is used (Nederlands Normalisatie-instituut, 2016) (Nederlands Normalisatie-instituut, 2011d).

This chapter of the national annex shows what formulas are used by the module in figure B.4 and B.5. The module mainly uses chapter 6 of NEN-EN 1993-1-1. Checking all of the formulas will not be done due to the number of necessary checks which will change depending on certain factors. For example, the cross section classification class heavily influences the checks.
Therefore a few results will be given and the relevant Eurocode formulas will be shown. These were filled in by the author of this thesis and compared to the given table results in RFEM in figure B.6. A more complex calculation was checked as well, namely the lateral torsional buckling in figure B.7. The calculations of the module matched the hand made calculations. The correct formulae were taken from the Eurocode in RFEM as well. It took the correct values from the Dutch national annex.
The SLS check in STEEL EC3 was compared to the SLS check in section 3.1.4. The module uses the correct criteria for the given load combinations (figure B.8). The limit value criteria for the characteristic combination and frequent combination is the same as stated in the Eurocode. For the quasi-permanent combination, a more conservative value is used. These values can be changed if needed as seen in figure B.9.

According to the Dutch national annex 6.1 (1):

$$\begin{aligned} &\gamma_{M0} = \gamma_{M1} = 1.00 \\ &\gamma_{M2} = 1.25 \end{aligned} \tag{B.13}$$

Section No.	A Member No.	B Location x [m]	C Loading	D Design Ratio	E	F Design According to Formula
1	IPE 240 | British Steel - Cross section created in Dynamo					
	7	1.000	CO3	0.03	≤ 1	CS101) Cross-section check - Tension acc. to 6.2.3
	8	1.000	CO3	0.02	≤ 1	CS102) Cross-section check - Compression acc. to 6.2.4
	1	1.000	CO5	0.28	≤ 1	CS111) Cross-section check - Bending about y-axis acc. to 6.2.5 - Class 1 or 2
	6	0.000	CO3	0.84	≤ 1	CS121) Cross-section check - Shear force in z-axis acc. to 6.2.6
	5	0.000	CO3	0.01	≤ 1	CS123) Cross-section check - Shear force in y-axis acc. to 6.2.6
	5	0.000	LC1	0.00	≤ 1	CS126) Cross-section check - Shear buckling acc. to 6.2.6(6)
	1	1.000	CO5	0.28	≤ 1	CS141) Cross-section check - Bending and shear force acc. to 6.2.5 and 6.2.8
	2	1.000	CO3	0.31	≤ 1	CS161) Cross-section check - Biaxial bending and shear force acc. to 6.2.6, 6.2.7 and 6.2.9
	8	0.500	CO3	0.76	≤ 1	CS181) Cross-section check - Bending, shear and axial force acc. to 6.2.9.1
	6	0.500	CO3	0.64	≤ 1	CS221) Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.10 and 6.2.9
	1	1.000	CO3	0.40	≤ 1	ST331) Stability analysis - Lateral torsional buckling acc. to 6.3.2.1 and 6.3.2.3 - I-Section
	5	0.000	CO3	0.68	≤ 1	ST363) Stability analysis - Biaxial bending acc. to 6.3.3, Method 2
	8	1.000	CO3	0.78	≤ 1	ST364) Stability analysis - Bending and compression acc. to 6.3.3, Method 2
	1	0.000	LC1	0.00	≤ 1	SE400) Serviceability - Negligible deformations
	8	0.500	CO3	0.52	≤ 1	SE401) Serviceability - Combination of actions 'Characteristic' - z-direction
	8	0.500	CO7	0.30	≤ 1	SE402) Serviceability - Combination of actions 'Frequent' - z-direction
	8	0.500	CO9	0.14	≤ 1	SE403) Serviceability - Combination of actions 'Quasi-permanent' - z-direction
	6	0.500	CO3	0.02	≤ 1	SE406) Serviceability - Combination of actions 'Characteristic' - y-direction
	6	0.500	CO7	0.01	≤ 1	SE407) Serviceability - Combination of actions 'Frequent' - y-direction
	6	0.500	CO9	0.00	≤ 1	SE408) Serviceability - Combination of actions 'Quasi-permanent' - y-direction

Figure B.4: Eurocode checks for a beam

Section No.	A Member No.	B Location x [m]	C Loading	D Design Ratio	E	F Design According to Formula
2	HE A 500 | ArcelorMittal (2011) - Cross section created in Dynamo					
	9	0.000	CO3	0.67	≤ 1	CS102) Cross-section check - Compression acc. to 6.2.4
	9	1.400	CO3	0.15	≤ 1	CS121) Cross-section check - Shear force in z-axis acc. to 6.2.6
	9	1.400	CO3	0.07	≤ 1	CS123) Cross-section check - Shear force in y-axis acc. to 6.2.6
	9	0.000	LC1	0.00	≤ 1	CS126) Cross-section check - Shear buckling acc. to 6.2.6(6)
	10	1.400	CO7	0.02	≤ 1	CS201) Cross-section check - Bending about z-axis, shear and axial force acc. to 6.2.9.1
	9	1.000	CO3	0.51	≤ 1	CS221) Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.10 and 6.2.9
	9	0.000	CO3	0.67	≤ 1	ST301) Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	10	0.000	CO5	0.45	≤ 1	ST311) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	9	0.000	CO3	0.69	≤ 1	ST312) Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
	9	0.000	CO5	0.48	≤ 1	ST321) Stability analysis - Torsional buckling acc. to 6.3.1.4 and 6.3.1.2(4)
	9	0.000	CO3	0.68	≤ 1	ST322) Stability analysis - Torsional buckling acc. to 6.3.1.4 and 6.3.1.2
	9	0.200	CO3	0.88	≤ 1	ST364) Stability analysis - Bending and compression acc. to 6.3.3, Method 2
	9	0.000	LC1	0.00	≤ 1	SE400) Serviceability - Negligible deformations
	9	0.600	CO3	0.07	≤ 1	SE401) Serviceability - Combination of actions 'Characteristic' - z-direction
	10	0.600	CO7	0.04	≤ 1	SE402) Serviceability - Combination of actions 'Frequent' - z-direction
	10	0.600	CO9	0.02	≤ 1	SE403) Serviceability - Combination of actions 'Quasi-permanent' - z-direction
	9	0.600	CO3	0.13	≤ 1	SE406) Serviceability - Combination of actions 'Characteristic' - y-direction
	9	0.600	CO7	0.06	≤ 1	SE407) Serviceability - Combination of actions 'Frequent' - y-direction
	9	0.600	CO9	0.03	≤ 1	SE408) Serviceability - Combination of actions 'Quasi-permanent' - y-direction

Figure B.5: Eurocode checks for a column

Axial tension

$$N_{pl,Rd} = \frac{A\,f_y}{\gamma_{M0}} \quad (6.6)$$

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1{,}0 \quad (6.5)$$

⊟ Design Internal Forces					
Axial Force	N_{Ed}	23.13	kN		
⊟ Design Ratio					
Tension Force	$N_{t,Ed}$	23.13	kN		
Cross-Sectional Area	A	39.10	cm^2		
Yield Strength	f_y	23.50	kN/cm^2		3.2.1
Partial Factor	γ_{M0}	1.000			6.1
Design plastic resistance to normal forces	$N_{pl,Rd}$	918.85	kN		(6.6)
Axial Force Resistance	$N_{t,Rd}$	918.85	kN		
Design Ratio	η	0.03		≤ 1	(6.5)
⊟ Design Formula					
$N_{t,Ed} / N_{t,Rd} = 0.03 \leq 1$ (6.5)					

Axial compression

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1{,}0 \quad (6.9)$$

$$N_{c,Rd} = \frac{A\,f_y}{\gamma_{M0}} \quad (6.10)$$

$$N_{c,Rd} = \frac{A_{eff}\,f_y}{\gamma_{M0}} \quad (6.11)$$

⊟ Design Internal Forces					
Axial Force	N_{Ed}	-21.23	kN		
⊞ Cross-Section Classification - Class 1					
⊟ Design Ratio					
Compression Force	$N_{c,Ed}$	21.23	kN		
Cross-Sectional Area	A	39.10	cm^2		
Yield Strength	f_y	23.50	kN/cm^2		3.2.1
Partial Factor	γ_{M0}	1.000			6.1
Axial Force Resistance	$N_{c,Rd}$	918.85	kN		Eq. (6.10)
Design Ratio	η	0.02		≤ 1	(6.9)
⊟ Design Formula					
$N_{c,Ed} / N_{c,Rd} = 0.02 \leq 1$ (6.9)					

Bending and shear

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1{,}0 \quad (6.12)$$

klasse 1 of 2

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl}\,f_y}{\gamma_{M0}} \quad (6.13)$$

$$V_{pl,Rd} = \frac{A_v\left(f_y / \sqrt{3}\right)}{\gamma_{M0}} \quad (6.18)$$

Also checks bending + shear (6.2.8)

⊟ Design Internal Forces					
Axial Force	N_{Ed}	0.00	kN		
Shear Force	$V_{y,Ed}$	0.00	kN		
Shear Force	$V_{z,Ed}$	0.00	kN		
Torsional Moment	T_{Ed}	0.00	kNm		
Moment	$M_{y,Ed}$	24.06	kNm		
Moment	$M_{z,Ed}$	-0.03	kNm		
⊞ Cross-Section Classification - Class 1					
⊟ Design Ratio					
Moment	$M_{y,Ed}$	24.06	kNm		
Plastic Section Modulus	$W_{pl,y}$	367.00	cm^3		
Yield Strength	f_y	23.50	kN/cm^2		3.2.1
Partial Factor	γ_{M0}	1.000			6.1
Moment Resistance	$M_{pl,y,Rd}$	86.25	kNm		Eq. (6.13)
Shear Force	$V_{z,Ed}$	0.00	kN		
Effective Shear Area	$A_{v,z}$	19.13	cm^2		6.2.6(3)
Shear Force Resistance	$V_{pl,z,Rd}$	259.52	kN		Eq. (6.18)
Criterion $V_{z,Ed} / V_{pl,z,Rd}$	v_z	0.000		≤ 0.5	6.2.8(2)
Moment Resistance	$M_{c,y,Rd}$	86.25	kNm		
Design Ratio	η	0.28		≤ 1	(6.12)
⊟ Design Formula					
$M_{y,Ed} / M_{c,y,Rd} = 0.28 \leq 1$ (6.12)					

Bending+shear

Details - Member 2 - x: 1.000 m - CO3

⊞ Cross-Section Classification - Class 1					
⊟ Design Ratio					
Moment	$M_{y,Ed}$	46.72	kNm		
Yield Strength	f_y	23.50	kN/cm^2		3.2.1
Partial Factor	γ_{M0}	1.000			6.1
Moment Resistance	$M_{pl,y,Rd}$	86.25	kNm		Eq. (6.13)
Shear Force	$V_{z,Ed}$	0.00	kN		
Effective Shear Area	$A_{v,z}$	19.13	cm^2		6.2.6(3)
Shear Force Resistance	$V_{pl,z,Rd}$	259.52	kN		Eq. (6.18)
Criterion $V_{z,Ed} / V_{pl,z,Rd}$	v_z	0.000		≤ 0.5	6.2.10(2)
Moment	$M_{z,Ed}$	0.31	kNm		
Plastic Section Modulus	$W_{pl,z}$	73.92	cm^3		
Moment Resistance	$M_{pl,z,Rd}$	17.37	kNm		Eq. (6.13)
Shear Force	$V_{y,Ed}$	0.00	kN		
Effective Shear Area	$A_{v,y}$	24.83	cm^2		6.2.6(3)
Shear Force Resistance	$V_{pl,y,Rd}$	336.95	kN		Eq. (6.18)
Criterion $V_{y,Ed} / V_{pl,y,Rd}$	v_y	0.000		≤ 0.5	6.2.10(2)
Interaction Constant	α	2.000			6.2.9.1(6)
Interaction Constant	β	1.000			6.2.9.1(6)
Design Component for M_y	η_{My}	0.29		≤ 1	(6.41)
Design Component for M_z	η_{Mz}	0.02		≤ 1	(6.41)
Design Component for M	η_M	0.31		≤ 1	(6.41)
⊟ Design Formula					

Figure B.6: ULS checks according to EN1993-1-1

Lateral torsional buckling

Design Ratio					
Section Height	h	240.0	mm		
Section Width	b	120.0	mm		
Criterion	h/b	2.00		≤ 2	Tab. 6.5
Buckling Curve	BC_{LT}	b			Tab. 6.5
Imperfection Factor	α_{LT}	0.340			Tab. 6.3
Modulus of Elasticity	E	21000.00	kN/cm²		
Shear Modulus	G	8076.92	kN/cm²		
Length Factor	k_z	1.000			
Length Factor	k_w	1.000			
Length	L	1.000	m		
Moment of Inertia	I_z	284.00	cm⁴		
Warping Constant of Cross-Section	I_w	37400.00	cm⁶		
Torsional Constant	I_t	13.00	cm⁴		
Elastic Critical Moment for Lateral-Torsional Buckling	M_{cr}	1029.81	kNm		
Section Modulus	W_y	367.00	cm³		
Yield Strength	f_y	23.50	kN/cm²		3.2.1
Slenderness	λ_LT	0.289			6.3.2.2(1)
Parameter	λ_LT,0	0.400			6.3.2.3(1)
Parameter	β	0.750			6.3.2.3(1)
Auxiliary Factor	Φ_{LT}	0.513			6.3.2.3(1)
Reduction Factor	χ_{LT}	1.000			Eq. (6.57)
Correction Factor	k_c	0.829			6.3.2.3(2)
Modification Factor	f	0.959			6.3.2.3(2)
Reduction Factor	$\chi_{LT,mod}$	1.000			Eq. (6.58)
Partial Factor	γ_{M1}	1.000			6.1
Design Lateral-Torsional Buckling Resistance Moment	$M_{b,Rd}$	86.25	kNm		Eq. (6.55)
Moment	$M_{y,Ed}$	34.75	kNm		
Design Ratio	η	0.40		≤ 1	(6.54)
Design Formula					
$M_{y,Ed} / M_{b,Rd} = 0.40 \leq 1$ (6.54)					

6.3.2.2 Kipkrommen – Algemeen

(1) $\bar{\lambda}_{LT} = \sqrt{\dfrac{W_y f_y}{M_{cr}}}$

6.3.2.3 Kipkrommen voor gewalste profielen of equivalente gelaste profielen

(1) — de waarde van $\bar{\lambda}_{LT,0}$ moet gelijk zijn genomen aan 0,4; $\Rightarrow$ **$\lambda_{LT,0}$ =0.4**

— de waarde van β moet gelijk zijn genomen aan 0,75; $\Rightarrow$ **β = 0.75**

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} \text{ maar } \chi_{LT} \leq 1{,}0 \quad (6.56) \qquad \Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}2\right) + \bar{\lambda}_{LT}^2\right]$$

(2) $f = 1 - 0{,}5 \times (1 - k_c) \times \left(1 - 2{,}0\left(\bar{\lambda}_{LT} - 0{,}8\right)^2\right)$ $\quad f \leq 1{,}0$ en k_c volgens 6.6.

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} \text{ maar } \begin{cases} \chi_{LT,mod} \leq 1 \\ \chi_{LT,mod} \leq \dfrac{1}{\bar{\lambda}_{LT}^2} \end{cases} \quad (6.58)$$

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \quad (6.55) \qquad \frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0 \quad (6.54)$$

Figure B.7: More complex example: Lateral torsional buckling

Characteristic combination

⊞ Material Properties - Steel S 235 | DIN EN 1993-1-1:2010-12				
⊞ Cross-Section Properties - HE A 500 | ArcelorMittal (2011)				
⊟ Deflections				
Direction x	w_x	-1.6	mm	
Direction y	w_y	1.8	mm	
Direction z	w_z	1.0	mm	
⊟ Design Ratio				
Deflection	$w_{max,z}$	0.5	mm	
Refer. Length	l	2.000	m	
Limit Value Criterion	$l / w_{limit,z}$	250.00		
Limit Value of Deflection	$w_{limit,z}$	8.0	mm	
Design Ratio	η	0.07		≤ 1
⊟ Design Formula				
$w_{max,z} / w_{limit,z} = 0.07 \leq 1$ EN 1990 (6.13)				

$w_2 + w_3 = l_{rep}/250$

Frequent combination

⊞ Material Properties - Steel S 235 | DIN EN 1993-1-1:2010-12				
⊞ Cross-Section Properties - HE A 500 | ArcelorMittal (2011)				
⊟ Deflections				
Direction x	w_x	-0.7	mm	
Direction y	w_y	0.6	mm	
Direction z	w_z	0.0	mm	
⊟ Design Ratio				
Deflection	$w_{max,z}$	-0.2	mm	
Refer. Length	l	2.000	m	
Limit Value Criterion	$l / w_{limit,z}$	333.00		
Limit Value of Deflection	$w_{limit,z}$	6.0	mm	
Design Ratio	η	0.04		≤ 1
⊟ Design Formula				
$w_{max,z} / w_{limit,z} = 0.04 \leq 1$ EN 1990 (6.13)				

$w_2 + w_3 = 1000 * l_{rep}/3$
(frequently walked on floors)

Quasi-permanent combination

⊞ Material Properties - Steel S 235 | DIN EN 1993-1-1:2010-12				
⊞ Cross-Section Properties - HE A 500 | ArcelorMittal (2011)				
⊟ Deflections				
Direction x	w_x	-0.5	mm	
Direction y	w_y	0.5	mm	
Direction z	w_z	0.0	mm	
⊟ Design Ratio				
Deflection	$w_{tot,z}$	-0.2	mm	
Precamber	w_c	0.0	mm	
Deflection Minus Precamber	$w_{tot,z} - w_{c,x}$	-0.2	mm	
Refer. Length	l	2.000	m	
Limit Value Criterion	$l / w_{limit,z}$	200.00		
Limit Value of Deflection	$w_{limit,z}$	10.0	mm	
Design Ratio	η	0.02		≤ 1
⊟ Design Formula				
$w_{max,z} / w_{limit,z} = 0.02 \leq 1$ EN 1990 (6.13)				

$w_{max} = l_{rep}/200$
More conservative than EC

Figure B.8: SLS checks for deflection

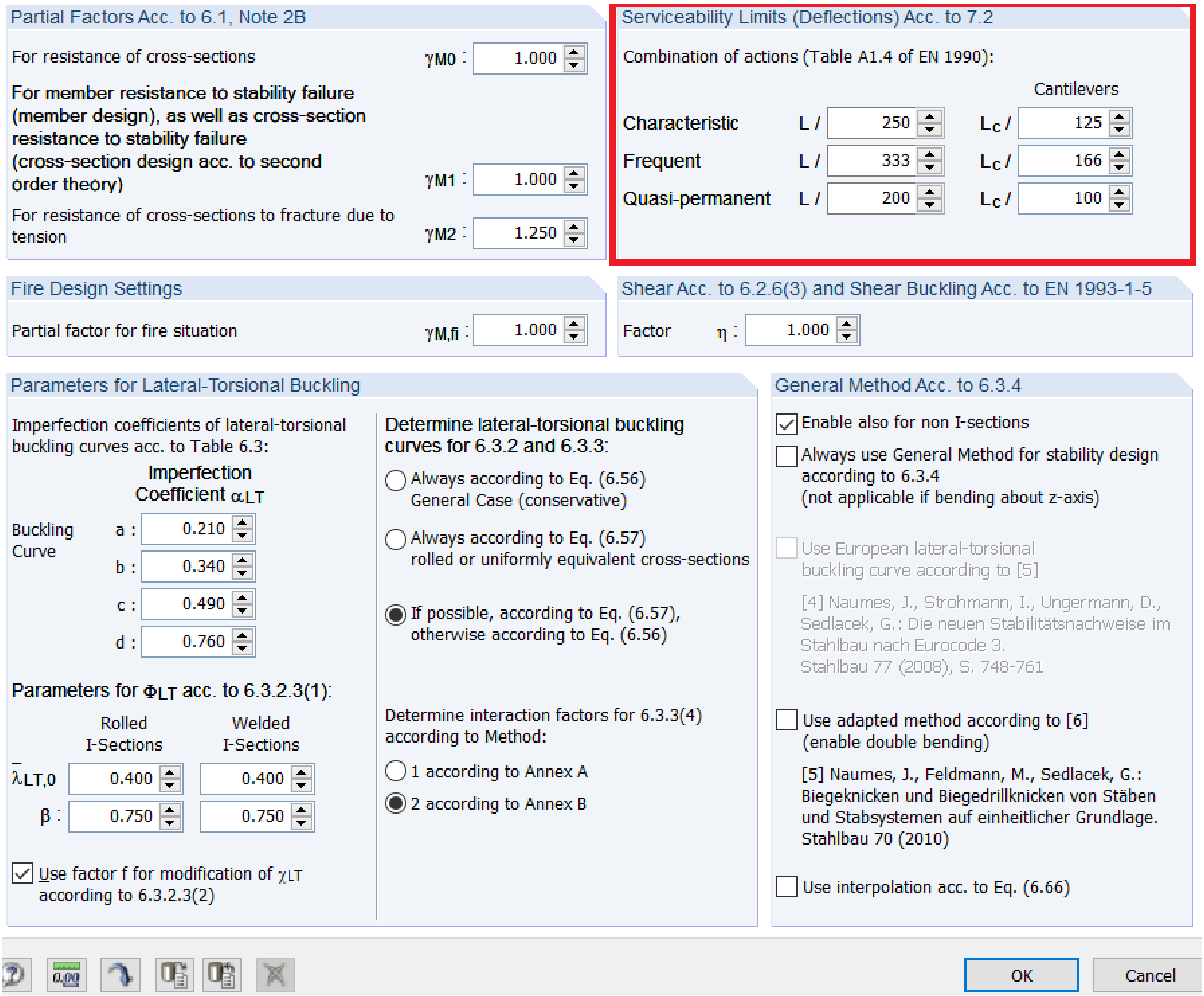


Figure B.9: Options in STEEL RF3

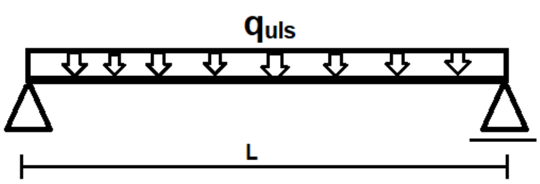
B.3 Concrete slab height calculation

A few assumptions will be given before the calculation of the height of the slab is shown. The reinforcement ratio (ρ) of floors is usually between 0.15% and 0.3% (Walraven & Fennis, 2013) and the correct value will be found trough iteration. The concrete cover is assumed to be 20mm, the reinforcement strength is $500N/mm^2$ and the concrete strength class C25/30 is chosen. With this information the following Eurocode formulae can be used which include the reinforcement ratio, strength class and tension stiffening (Walraven & Fennis, 2013):

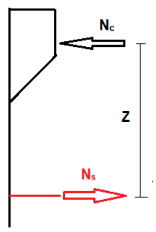
$$\begin{aligned}
&\text{EN 1992-1-1 eq. 7.16a and 7.16b}\\
&\frac{l_{eff}}{d} = K \cdot [11 + 1.5\sqrt{f_{ck}} \cdot \frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}(\frac{\rho_0}{\rho} - 1)^{\frac{3}{2}}] \qquad \text{for } \rho \leq \rho_0\\
&\frac{l_{eff}}{d} = K \cdot [11 + 1.5\sqrt{f_{ck}} \cdot \frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho}}] \qquad \text{for } \rho > \rho_0\\
&\rho_0 = \sqrt{f_{ck}} \cdot 10^{-3}\\
&h = d + C_{nom} + \frac{1}{2}\phi
\end{aligned} \tag{B.14}$$

Where the value of K is dependent on the static scheme (K = 1 for simply supported beams) and ρ' is the compression reinforcement ratio. The effective depth (d) can be calculated from these formulae.
Shear reinforcement can be expensive in floors and therefore it was chosen to not include it. The following demand has to be be true:

$$\begin{aligned}
&v_{Ed} < v_{Rd} \longrightarrow v_{Rd} = v_{min}\\
&v_{min} = 0.035k^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} \longrightarrow \text{EN1992-1-1 (6.3N)}\\
&k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \longrightarrow \text{EN1992-1-1 (cl. 6.2.2)}
\end{aligned} \tag{B.15}$$



(a) Floor schematisation: simply supported beam



(b) The cross sectional balance

Figure B.10: Building a surface in Dynamo

The required reinforcement area has to be calculated as well to check if the assumed reinforcement ratio is correct. Using the loads from chapter 3.1.4 the following formulae are found:

$$\begin{aligned}
&M_{max} = \frac{1}{8}qL^2 \longrightarrow z = 0.9d \longrightarrow N_s = \frac{M_{max}}{0.9d} \longrightarrow A_s = \frac{N_s}{f_{yd}} \\
&A_s = \frac{M_{max}}{0.9 \cdot d \cdot f_{yd}} = \frac{q \cdot L^2}{8 \cdot 0.9 \cdot d \cdot f_{yd}} = \frac{q \cdot L^2}{7.2 \cdot d \cdot f_{yd}}
\end{aligned} \tag{B.16}$$

The following values for q can be found, which will differ for the floor, balcony and roof slabs. Filling these values into the previous formulae gives one formula which directly calculates the required reinforcement area.

$$\begin{aligned}
&q_{uls,F} = 1.35G + 0.6Q_A \longrightarrow \text{Highest load for the floor} \\
&q_{floor} = 1.35 \cdot (1.875 + h \cdot 1 \cdot 25) + 0.6 \cdot (1.75) = 3.6 + 33.8 \cdot h \\
&q_{uls,B} = 1.2G + 1.5Q_S + 0.6Q_A \longrightarrow \text{Highest load for the balcony} \\
&q_{balcony} = 1.2 \cdot (1.875 + h \cdot 1 \cdot 25) + 1.5 \cdot 2.8 + 0.6 \cdot (2.5) = 5.7 + 30 \cdot h \\
&q_{uls,R} = 1.2G + 1.5Q_R \longrightarrow \text{Highest load for the roof} \\
&q_{roof} = 1.2 \cdot (1.875 + h \cdot 1 \cdot 25) + 1.5 \cdot (1.0) = 1.5 + 30 \cdot h
\end{aligned} \tag{B.17}$$

These loads can also be used to find the shear stress for the slabs. These can then be used to check whether shear reinforcement is required.

$$V_{Ed} = \frac{1}{2}qL \longrightarrow v_{Ed,c} = \frac{V_E d}{b \cdot d} \longrightarrow b = 1m \tag{B.18}$$

The concrete area and the reinforcement area are both known. The assumed reinforcement ratio can now be checked. One can reiterate these formula in Dynamo until the calculated and assumed reinforcement ratio's are equal to each other.

C
The variant study

C.1 Verification of results

Some verifications are already done in appendix B. This is done for the internal forces and deflections and the module STEEL EC3.

C.1.1 Forces and deflections

A verification has to be done for the finite element model to check if the resulting internal forces and deflections are realistic. This is laborious and it will therefore only be done once for the beams.
The room to be analysed can be seen in the figure to the right. The two IPE80 beams in x-direction and the IPE600 beam in y-direction will be checked. A schematisation of this room is shown in figure C.2.

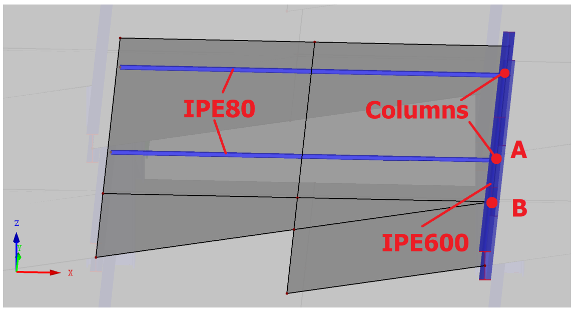


Figure C.1: The x-beams, y-beams and floor slabs of two hotel rooms

In the top left part of this figure (shown as **(0)**), two hotel rooms are schematised. The complex shape of the room and the shape of the slabs make it a laborious problem.
The loads have therefore been simplified in **(1)**. These figures only show the parts of the floor that will load the beam in y-direction. **(3)** shows how the simplification is made, which is done to keep the line loads constant. In **(2)** the mechanical model is shown as well as the loads.
G1 and G2 are the self weight of the floors/balconies and the beams. L1 is the self weight of the partitioning walls, L2 the self weight of the walls on the balcony and L3 is the load of the sliding door. The variable loads are given as q_a and the snow load q_{s1} and q_{s2}. These values are given in chapter 3.1.4.

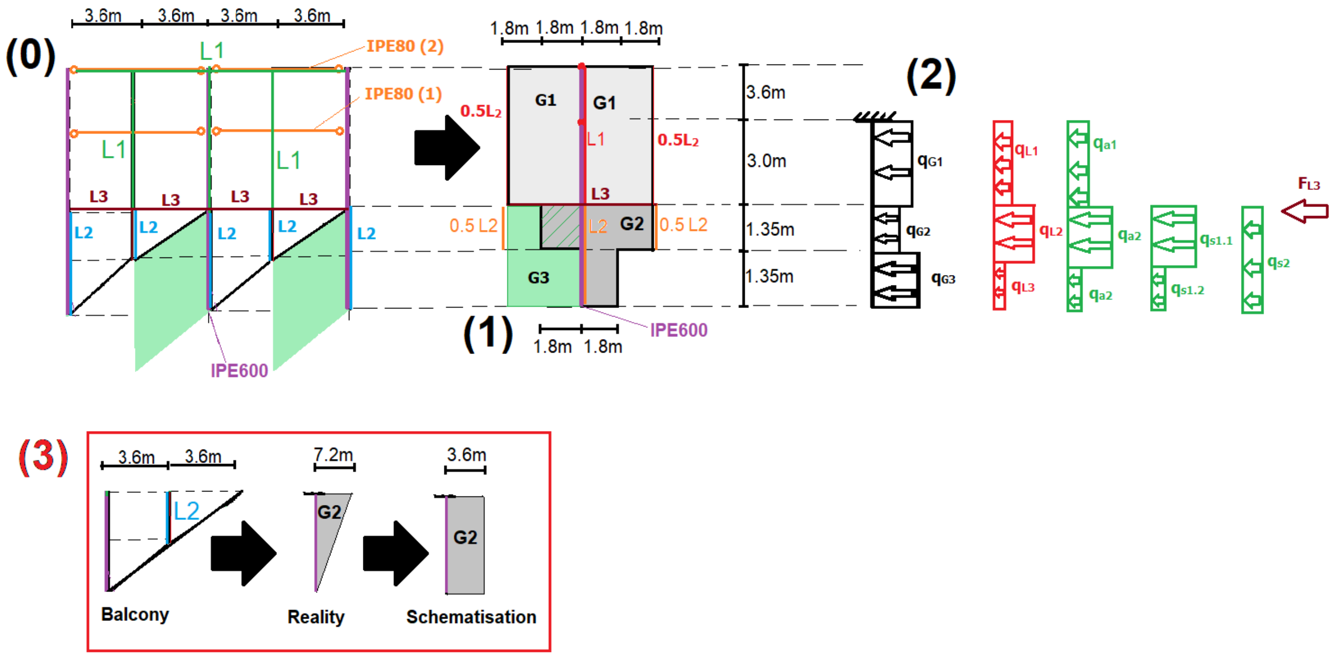


Figure C.2: Schematisation of the loads

Schematisation **(1)** will be calculated first. The value of the loads are:

Frequent combination: $G + 0.3Q_A + 0.2Q_s$

$$q_{G1} = G_{floor} + G_{beam} + G_{fin.layer} = 0.27 \cdot 25 \cdot 7.2 + 1.25 + 1.875 \cdot 7.2$$
$$q_{G1} = 63.4kN/m$$
$$q_{G2} = G_{floor} + G_{beam} + G_{fin.layer} + G_{livingroof} = 0.28 \cdot 25 \cdot 9.0 + 1.25 + 1.875 \cdot 5.4 + 3.6 \cdot 1.7$$
$$q_{G2} = 80.5kN/m$$
$$q_{G3} = G_{floor} + G_{beam} + G_{fin.layer} + G_{livingroof} = 0.28 \cdot 25 \cdot 5.4 + 1.25 + 1.875 \cdot 1.8 + 3.6 \cdot 1.7$$
$$q_{G3} = 48.5kN/m$$
$$q_{L1} = (1 + 0.5 + 0.5) \cdot G_{part.Wall} = 2 \cdot 1.3 = 2.6kN/m$$
$$q_{L2} = (1 + 0.5 + 0.5) \cdot G_{balc.Wall} = 2 \cdot 1.5 = 3kN/m$$
$$q_{L3} = 1 \cdot G_{balc.Wall} = 1 \cdot 1.5 = 1.5kN/m$$
$$F_{L3} = 7.2 \cdot 0.9 = 6.5kN$$
$$q_{a1} = 0.3 \cdot Q_{floor} \cdot 3.6 = 0.3 \cdot 1.75 \cdot 3.6 = 1.9kN/m$$
$$q_{a2} = 0.3 \cdot Q_{balcony} \cdot 5.4 = 0.3 \cdot 2.5 \cdot 5.4 = 4.1kN/m$$
$$q_{a3} = 0.3 \cdot Q_{balcony} \cdot 1.8 = 0.3 \cdot 2.5 \cdot 1.8 = 1.4kN/m$$
$$q_{s1.1} = 0.2 \cdot Q_{snow1} \cdot 5.4 = 0.56 \cdot 5.4 = 0.6kN/m$$
$$q_{s1.2} = 0.2 \cdot Q_{snow1} \cdot 1.8 = 0.56 \cdot 1.8 = 0.2kN/m$$
$$q_{s2.1} = 0.2 \cdot Q_{snow2} \cdot 3.6 = 2.8 \cdot 3.6 = 2.0kN/m$$

Gives:

$$q_1 = q_{G1} + q_{L1} + q_{a1} = 67.9kN/m$$
$$q_2 = q_{G1} + q_{L1} + q_{a1} + q_{s1.1} + q_{s2.1} = 90.2kN/m$$
$$q_3 = q_{G1} + q_{L1} + q_{a1} + q_{s1.1} + q_{s2.1} = 53.6kN/m$$
$$F_{L3} = 6.5kN$$

(C.1)

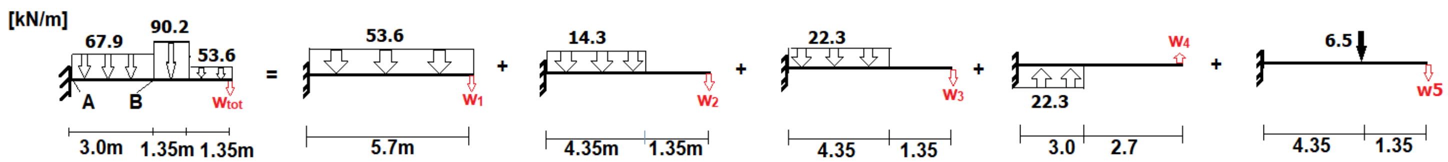


Figure C.3: Schematisation of the loads (superposition)

The bending moments and shear forces are calculated for points A (the column location) and point B (the transition from outside to inside) as follows:

$$M_A = 67.9 \cdot 3.0 \cdot 1.5 + 90.2 \cdot 1.35(3 + 0.5 \cdot 1.35) + 1.35 \cdot 53.6 \cdot (1.35 + 3 + 0.5 \cdot 1.35) + 6.5 \cdot 4.35$$
$$M_A = 1145kNm$$
$$M_B = 90.2 \cdot 1.35 \cdot 0.5 \cdot 1.35 + 53.6 \cdot 1.35(1.35 + 0.5 \cdot 1.35) = 229kNm$$
$$V_A = 67.9 \cdot 3 + 90.2 \cdot 1.35 + 1.35 \cdot 53.6 = 404kN$$
$$V_B = 90.2 \cdot 1.35 + 53.6 \cdot 1.35 = 194kN$$

(C.2)

Using superposition for calculation of the deflections gives:

$$
\begin{aligned}
&w = \frac{ql^4}{8EI} \qquad\qquad \theta = \frac{ql^3}{6EI} \qquad\qquad w = \frac{Fl^3}{3EI} \qquad\qquad \theta = \frac{Fl^2}{2EI}\\
&\text{IPE600: } EI = 210000 \cdot 67117 \cdot 10^{14}\\
&w_1 = \frac{53.6 \cdot 5700^4}{8 \cdot 1.94 \cdot 10^{14}} = 36mm\\
&w_2 = \frac{14.3 \cdot 4350^4}{8 \cdot 1.94 \cdot 10^{14}} + \frac{14.3 \cdot 1350^3}{6 \cdot 1.94 \cdot 10^{14}} \cdot 1350 = 10.2mm\\
&w_3 = \frac{22.3 \cdot 4350^4}{8 \cdot 1.94 \cdot 10^{14}} + \frac{22.3 \cdot 1350^3}{6 \cdot 1.94 \cdot 10^{14}} \cdot 1350 = 15.8mm\\
&w_4 = -\frac{22.3 \cdot 3000^4}{8 \cdot 1.94 \cdot 10^{14}} - \frac{22.3 \cdot 3000^3}{6 \cdot 1.94 \cdot 10^{14}} \cdot 2700 = -4.5mm\\
&w_5 = \frac{6.5 \cdot 4350^3}{3 \cdot 1.94 \cdot 10^{14}} + \frac{6.5 \cdot 1350^2}{2 \cdot 1.94 \cdot 10^{14}} \cdot 1350 = 0.02mm\\
&w_{tot} = w_1 + w_2 + w_3 + w_4 + w_5 = 57.5mm
\end{aligned}
\tag{C.3}
$$

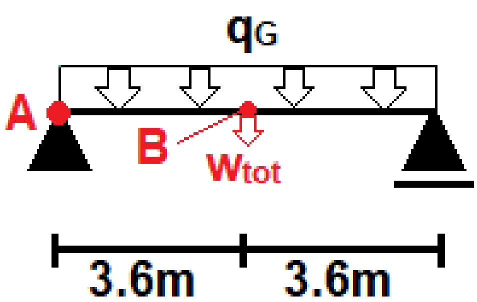


Figure C.4: Schematisation of the loads

Next, the beams in x-direction will be checked starting with an IPE80. It will only be loaded by its self weight, because it is disconnected from the floor. Its loads can be calculated as (same combination):

$$q_{G1} = q_1 = G_{beam} = 6.11 \cdot 9.81 \cdot 10^{-3} = 0.06kN/m \tag{C.4}$$

The stiffness of an IPE80 beam is: $EI = 1.68 \cdot 10^{11}$. The internal forces in point A and B can be calculated as:

$$
\begin{aligned}
M_B &= \frac{1}{8} q_1 l^2 = 0.39kNm\\
V_A &= \frac{1}{2} q_1 \cdot l = 0.22kN
\end{aligned}
\tag{C.5}
$$

The deflections are calculated as:

$$\text{Line loads: } w = \frac{5ql^4}{384EI} \qquad \theta = \frac{ql^3}{24EI}$$
$$w_{tot} = \frac{5 \cdot 0.06 \cdot 7200^4}{384 \cdot 1.68 \cdot 10^{11}} = 12.5mm \tag{C.6}$$

	V_x kN	M_x [kN/m	w_x [mm]	M_{yA} [kN/m]	M_{yB} [kN/m]	V_{yA} [kN]	V_{yB} [kN]	w_y [mm]
Calculated results	0.22	0.39	12.5	1145	229	404	194	57.5
RFEM results	0.22	0.39	12.7	635	277	171	107	58.4
Error [%]	0,0	0,0	1,6	44,5	21,0	57,7	44,8	1,6

Figure C.5: Comparison between calculated results and RFEM results

All the loads are known and the internal forces in RFEM are visualised in figure C.5 for the same frequent load combination. M stands for the bending moment, V is the shear force and w is the deflection. The figure shows that for the beams in x-direction everything went as expected and almost no error is found. The same is true for the deflection of the beam in y-direction. Though for the bending moments and shear forces the error is too large and cannot be explained with the simplified schematisation of the loads.
The internal forces in points A and B are visualised in RFEM in figure C.7. Point B marks the transition between outside and inside where the slabs are disconnected.
At first an error was sought in the finite element model, but no obvious mistakes were found. The fact that a surface is present with a sharp corner also does not cause this effect, because the same effect happens at points without a sharp corner. Therefore it is not considered a problem with the mesh.

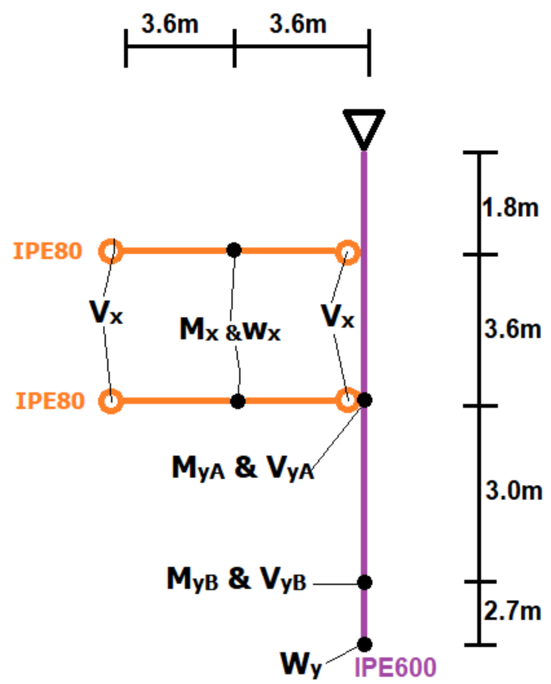


Figure C.6: The errors at each calculated point (top view)

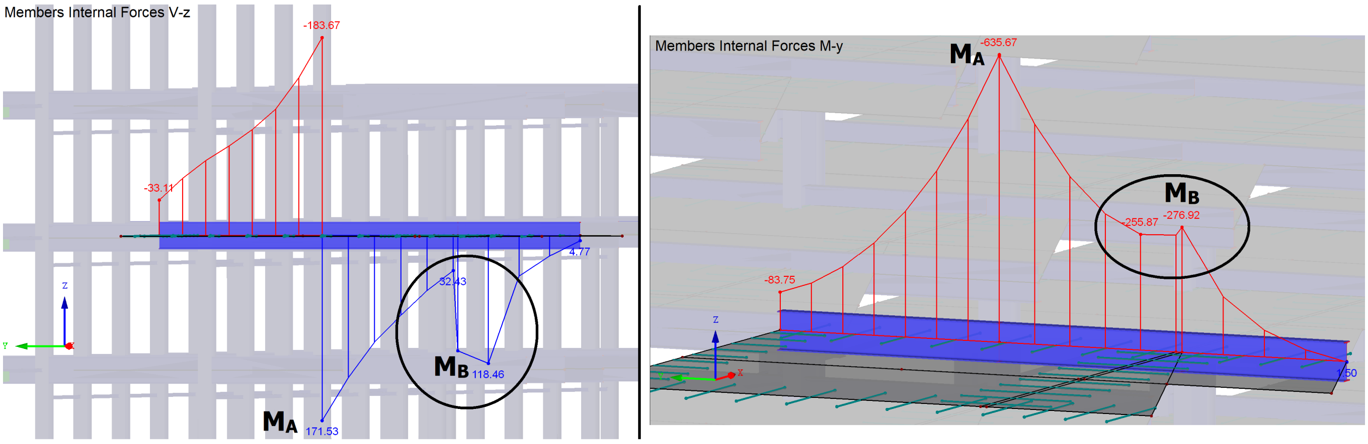
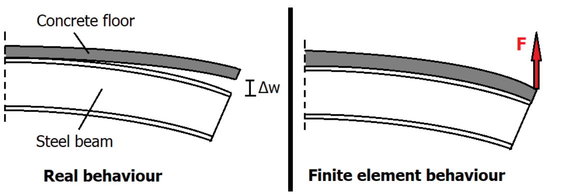
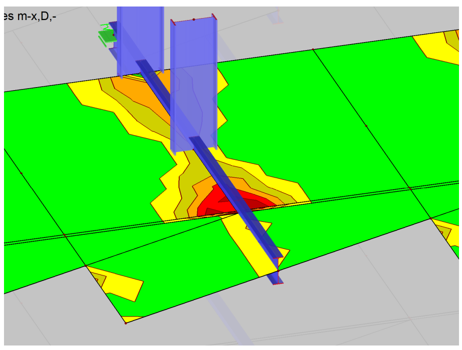


Figure C.7: The shear force and bending moment in RFEM visualised

This behaviour is explained as follows. In reality the simply supported floor should not be fully connected to the floor. Due to the difference in stiffness the floor will deflect slightly less than the steel beam as seen in the left part of figure C.8a.
However in the finite element model the floor will be connected to the beam and will deflect the same amount. It does not matter if a line hinge is used, both elements will always stay together as seen in the right part of figure C.8a. The floor will still want to translate upwards and will therefore cause a force instead.
This effect is observable in the results of the slabs as seen in figure C.8b. The value $m_{x,D-}$ is shown, which intensifies near the transition between outside and inside. This effect is largest at the transition and decreases slowly the further away the points are from the transition.
Therefore this effect is explained and is the reason for the large differences in internal forces between the calculated results and the finite element results. It is currently not possible to remedy this effect with the DRC. Due to the accurate prediction of the deflection, it is assumed that these effects mostly influence the internal forces of this model. It will be assumed that the model produces results which are accurate enough for the variant study.



(a) Side view of the concrete floor on the cantilevered beam



(b) Results of $M_{X,D-}$ for a floor

Figure C.8: Building a surface in Dynamo

Checking STEEL EC3 SLS
The SLS demand for the frequent combination (combination 16) was the critical demand as seen in figure C.9. The demand for the frequent combination is $2 \cdot L_{rep}/333$. L_{rep} is the cantilever length, which gives a maximum allowable deflection of 34.2 mm. The calculated value for the deflection was 57.5mm, which gives a unity check of $57.5/34.2 = 1.68$.
The deflection of the column was not included in the verification, which is 5.5mm according to RFEM (this difference was already corrected in the table). This gives $63/34.2 = 1.84$. There is still a difference of 0.14, because long term effects were not considered in the verification. STEEL EC3 does that this into consideration.

	782	3.600	CO18	0.31	≤ 1	SE403) Serviceability - Combination of actions 'Quasi-permanent' - z-direction
	30	2.706	CO13	1.69	> 1	SE411) Serviceability - Combination of actions 'Characteristic' - z-direction, Cantilever
	30	2.706	CO16	1.98	> 1	SE412) Serviceability - Combination of actions 'Frequent' - z-direction, Cantilever
	30	2.706	CO18	1.45	> 1	SE413) Serviceability - Combination of actions 'Quasi-permanent' - z-direction, Cantilever
	90	1.353	CO14	0.02	≤ 1	SE416) Serviceability - Combination of actions 'Characteristic' - y-direction, Cantilever

Figure C.9: The SLS results in STEEL EC3

C.2 Calculation of the shadow price

Materials have an impact on the environment. This is not only the impact on climate change, but also includes other impacts for example depletion of resources or acidification. The shadow price is a way to visualise the impact on the environment and can be seen as the price of restoring the damage that has been done to produce the material.
An estimation of the shadow price is calculated using the Inventory of Carbon and Energy (ICE). This is an embodied energy and carbon database created by researchers at the University of Bath. The database contains a summary which states the equivalent kg CO2 for producing a kilogram of a certain material (Circular Ecology, n.d.). This database is made for materials in the UK. The Dutch database, named "Nationale Milieudatabase", can be used as well but requires additional calculation for the CO2 equivalent. A more detailed calculation is possible with both databases, but will not be done.
One of the outputs of the DRC is the total weight for each type of structural element, which makes it possible to find the equivalent kg CO2 for the whole structure with a simple multiplication. The following values were relevant for this thesis (Hammond & Jones, 2011):

Materials	CO2 equivalent [kgCO2e / kg]	Shadow price [€ / kg]
Engineering steel (Recycled)	0,72	0,036
Reinforced concrete 25/30	0,198	0,0099
Glue Laminated timber	0,87	0,0435

Figure C.10: Equivalent kg CO2 per kg material

An elementary way to calculate the shadow price is to simply multiply the CO2 equivalent value by €0.05, which is the weight factor in €/ kg equivalent CO2 (Stichting Bouwkwaliteit, 2014, p. 37).
A more advanced analysis can be performed to calculate the shadow price. The result would be a certain value with the unit [€/ kg], which can be used in the same manner as the price in the table.

C.3 The costs of materials

According to A. Martens (personal communication, May 21, 2019), a senior costs specialist for Arcadis, the following is done for structures in the preliminary design phase.
Costs per unit length or mass are mainly calculated by costs specialist using the most recent quotations. These vary significantly over time, which is why detailed calculations should be made with the most recent information. There is not enough information available in the preliminary design stage yet to justify a detailed cost calculation. If details such as connections are missing, then the detailed cost calculation would be too inaccurate.
For the preliminary phase an indication of the costs is calculated by using data from previous cost calculations. From this data an estimation can be made for the calculation of the price of the support structure. The following values were advised:

$$\begin{aligned}
&\text{Steel: } Costs = €2.75/kg \qquad \text{(Cheaper for large profiles in high bulk)}\\
&\text{Concrete: } Costs = €350/m^3\\
&\text{Timber: } Costs = No\ value\ available
\end{aligned} \tag{C.7}$$

Using these values would approximate the costs with enough accuracy at this stage of design. The costs of timber products are difficult to approximate in this manner and were therefore not available. More detailed information is required before these costs can be approximated.

C.4 The IsoKorb connection for steel beams

Cold bridging should be prevented in the structure. The balcony concrete slabs are disconnected from the indoor slabs with thermal isolation. This is not possible for the steel beams, because internal forces must still be carried inside. Therefore Isokorbs are used for steel beams to prevent cold bridging through the steel beams. For reference Schöck Isokorbs type KST are used (Schöck Ltd, 2014). According to this company the following can be said about their product.

- Isokorbs type KST are used to connect cantilevered steel girders together for new constructions or renovations.
- Isokorbs are modular and can therefore be adapted to all profile sizes and load bearing capacity requirements.

- The universal beam connections strength can be increased for both shear force and bending moments, but it requires contact with the technical department of the company.

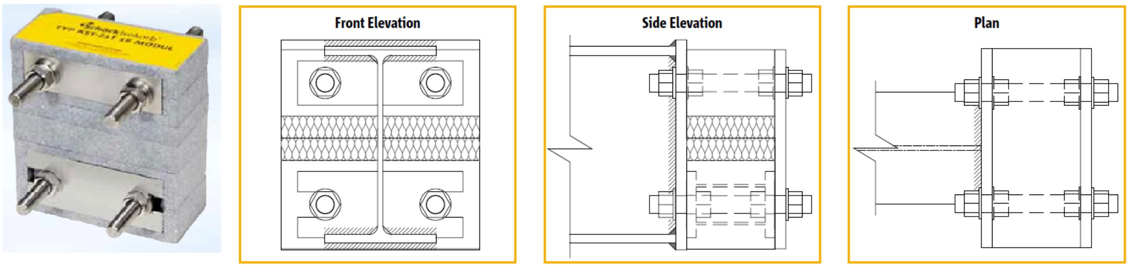


Figure C.11: The Isokorb type KST for beams. Amended from: (Schöck Ltd, 2014)

The resistance of the Isokorbs for bending moments and shear forces can be seen in figure C.12. For the variant study the maximum possible shear force and bending moments are used for the unity check:

$$\begin{aligned} &\text{Bending moments:} \quad U.C._{Isokorb} = \frac{M_{Ed}}{M_{Rd}} = \frac{M_{Ed}}{173} < 1 \\ &\text{Shear forces:} \quad U.C._{Isokorb} = \frac{V_{Ed}}{V_{Rd}} = \frac{Ed}{72} < 1 \end{aligned} \tag{C.8}$$

The values V_{Ed} and M_{Ed} are the respective internal forces found in the finite element model at the position of the Isokorb. These values can be retrieved for each cantilever beam and the unity check can then be automatically calculated and represented in the output.

Schöck Isokorb® type KST Selection Table for Universal Beam Sections

Con. No.	KST Modules	152 UB (B1)		178 UB (B2)		203 UB (B3)		254 UB (B4)		305 UB (B5)	
		Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear
C1	ZST+QST 16	-10kNm	+30kN	-13kNm	+30kN	-15.5kNm	+30kN	-22kNm	+30kN	-28kNm	+30kN
C2	ZST+QST 22	-17.5kNm	+36kN	-23kNm	+36kN	-28kNm	+36kN	-40.5kNm	+36kN	-52kNm	+36kN
C3	ZST+QST+QST 16	-	-	-	-	-	-	-21kNm	+60Kn	-26kNm	+60Kn
C4	ZST+QST+QST 22	-	-	-	-	-	-	-39.5kNm	+72kN	-48.5kNm	+72kN
C5	ZST+ZST+QST+QST 16	-	-	-	-	-	-	-	-	-31kNm	+60Kn
C6	ZST+ZST+QST+QST 22	-	-	-	-	-	-	-	-	-57kNm	+72kN

Schöck Isokorb® type KST Selection Table for Universal Beam Sections

Con. No.	KST Modules	356 UB (B6)		406 UB (B7)		457 UB (B8)		533 UB (B9)		610 UB (B10)	
		Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear
C1	ZST+QST 16	-32.5kNm	+30kN	-38.5kNm	+30kN	-44kNm	+30kN	-52kNm	+30kN	-61kNm	+30kN
C2	ZST+QST 22	-61kNm	+36kN	-72kNm	+36kN	-83kNm	+36kN	-99kNm	+36kN	-115kNm	+36kN
C3	ZST+QST+QST 16	-30.5kNm	+60Kn	-35.5kNm	+60Kn	-41kNm	+60Kn	-49kNm	+60Kn	-57kNm	+60Kn
C4	ZST+QST+QST 22	-57kNm	+72kN	-67kNm	+72kN	-77kNm	+72kN	-93kNm	+72kN	-109kNm	+72kN
C5	ZST+ZST+QST+QST 16	-39.5kNm	+60Kn	-49kNm	+60Kn	-60kNm	+60Kn	-75kNm	+60Kn	-92kNm	+60Kn
C6	ZST+ZST+QST+QST 22	-72kNm	+72kN	-92kNm	+72kN	-110kNm	+72kN	-142kNm	+72kN	-173kNm	+72kN

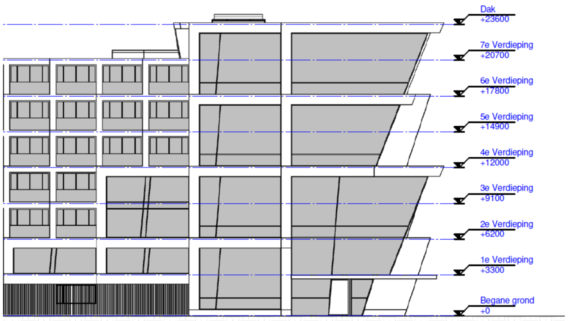
Figure C.12: The allowable bending moments and shear forces for the Isokorbs. From: (Schöck Ltd, 2014)

C.5 Additional information of the variant study

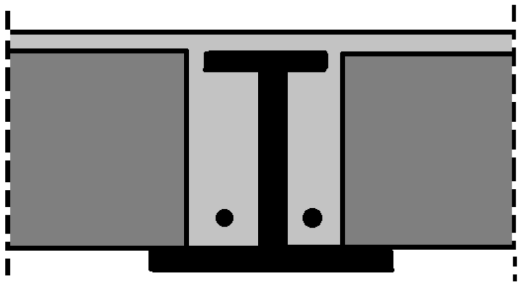
C.5.1 Variant: floor span 7.2m

The floor height

An IPE500 profile can be considered a too large profile. The architect set the height to 2.9 m as seen in figure C.13a. The steel beam and the floor would have a height of 500+270+75=845mm, which leaves 2.05m. The requirement for rooms with lodging functions in the Netherlands is between 2.1m and 2.6m for new structures according to the Dutch code "Bouwbesluit" (BRIS BV., 2018). Therefore the floor height has to be increased or the floor slabs have to be set between the IPE beams as seen in figure C.13b. The latter will require a new calculation, because different steel profiles are used.



(a) The side view with floor heights as intentioned by the architect



(b) An example of a floor between beams

Figure C.13: Visualisation of RFEM results

Effect of hollow core slabs on the Isokorb unity checks

Lowering the loads can be done by shortening the floor span or lowering the self weight of structural elements. The permanent loads can be lowered for example by using hollow core slabs. The permanent load of the massive 270mm slab is $6.75kN/m^2$. A reference hollow core slab of 260mm weighs $3.76kN/m^2$ (VBI, n.d.), a reduction of 44%. For a floor span of 7.2m this leads to a reduction of the line load on the beams by 23.3 kN/m. The reduction of the unity check is estimated with formula's of a rigidly supported cantilever beam:

$$
\begin{aligned}
&M = \frac{1}{2}ql^2 \qquad\qquad V = ql \\
&M_{reduction} = \frac{1}{2} \cdot 23.3 \cdot 4.05^2 = 191kNm \\
&V_{reduction} = 23.3 \cdot 4.05 = 94kN \\
&M_{isokorb} = U.C._M \cdot M_{Rd,Isokorb} = 173 \cdot 2.77 = 479kNm \\
&V_{isokorb} = U.C._V \cdot V_{Rd,Isokorb} = 72 \cdot 6.23 = 449kN \\
&U.C._{M,new} = \frac{M_{isokorb} - M_{reduction}}{M_{Rd,Isokorb}} = \frac{479 - 191}{173} = 1.66 \\
&U.C._{V,new} = \frac{V_{isokorb} - V_{reduction}}{V_{Rd,Isokorb}} = \frac{449 - 94}{72} = 4.93
\end{aligned}
\tag{C.9}
$$

This estimation shows that a significant decrease in unity check for the Isokorbs is found. The unity check is lowered by 40% for bending moments and 21% for shear forces. This is not enough to lower the unity check to acceptable levels.

Effect of hollow core slabs on the Isokorb unity checks

The reduced load was 23.3kN/m. The stiffness EI of the hollow core slab is calculated as $EI = 1.72 \cdot 10^{13}$ which includes cracking of concrete. The stiffness of the currently used massive slab is $EI = 1.86 \cdot 10^{13}$. The floors are schematised as a simply supported beam. The difference in deflection w between the current slab and the hollow core slab can be estimated as:

$$
\begin{aligned}
&\text{Permanent load massive slab:} P_{massive} = 25 \cdot 0.28 = 7kN/m^2 \\
&q_{massive} = 7 \cdot 7.2 = 50.4kN/m \\
&\text{Permanent load hollow core slab:} P_{HCS} = 3.76kN/m^2 \\
&q_{HCS} = 3.76 \cdot 7.2 = 27.1kN/m \\
&q_{HCS} = 0.54q_{massive} \qquad\qquad EI_{HCS} = 0.92EI_{massive} \\
&w_{floor} = \frac{5ql^4}{384EI} = \frac{5 \cdot (q \cdot 0.54) \cdot l^4}{384 \cdot (EI \cdot 0.92)} = 0.6 \cdot \frac{5ql^4}{384EI} \\
&\text{Permanent contribution to the floor deflection: } w_{perm,HCS} = 0.6w_{perm,massive} \\
&w_{y-beams} = \frac{ql^4}{8EI} = \frac{\cdot(q \cdot 0.54) \cdot l^4}{8 \cdot (EI)} = 0.54 \cdot \frac{5ql^4}{384EI} \\
&\text{Permanent contribution to the y-beam deflection: } w_{perm,HCS} = 0.54w_{perm,massive}
\end{aligned}
\tag{C.10}
$$

The floor was schematised as a simply supported beam for a span of 7.2m. By finding the differences in stiffness and permanent load, the difference in deflection is found. This is a simplified calculation and should be considered a conservative estimate. Using a hollow core slab would also decrease the loads on the y-beams which would decrease their deflection as well. This is calculated by schematising the y-beams as rigidly supported cantilevers.

The deflection of the floor due to permanent loads according to the finite element model is shown in figure C.14. The difference in deflection between the middle of the floor and its sides is estimated as 8mm. Using the above formula gives a deflection of $0.6 \cdot 8 = 4.8mm$, a reduction of 3.2mm. The permanent contribution of the permanent load to the deflection of the y-beams is 11mm. This gives a reduction of $0.54 \cdot 11 = 5.94mm$. These values do not consider the complex geometry of the structure, but they will be used to estimate if a hollow core slab could produce a viable solution.

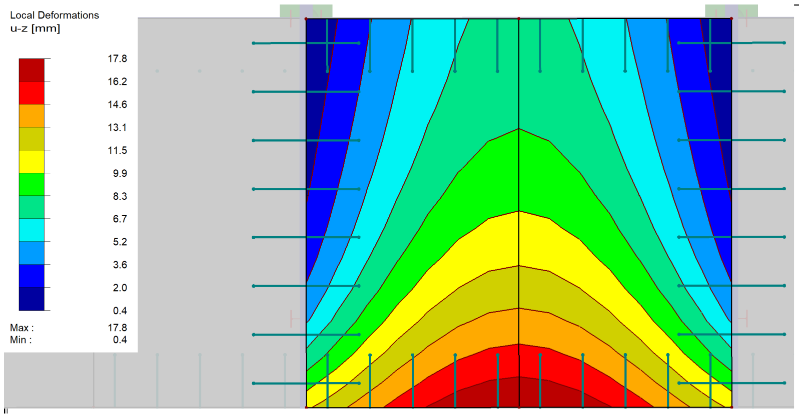


Figure C.14: Deformations caused by permanent loads in a single indoor floor slab

Effects on shear force explained

The shear forces should also vary only slightly. Instead the shear forces decrease for increasing cantilever length. The shear forces are visualised for a single y-beam for different spans in figure C.15. The difference between a cantilever of 7.05m and 4.55m can not be explained with a rigidly supported cantilever beam. In reality the support will have a stiffness, which is affected by the positioning of the column. This change causes lower shear forces the further away the column is from the Isokorb, thus the shear unity check will lower.

The difference from option 1 to option 2 is significant and can also be explained with figure C.15 between a cantilever of 4.05m and 4.55m. When the column is set at the edge of the transitioning between inside and outside, the effect of the floors on the shear force is gone. This behaviour was explained in the verification and is the reason why the shear forces change significantly between these two options. The reason why it does not happen for cantilever of 4.05m is that the balcony could be supported by the columns, therefore one should be sceptical about the results of this option.

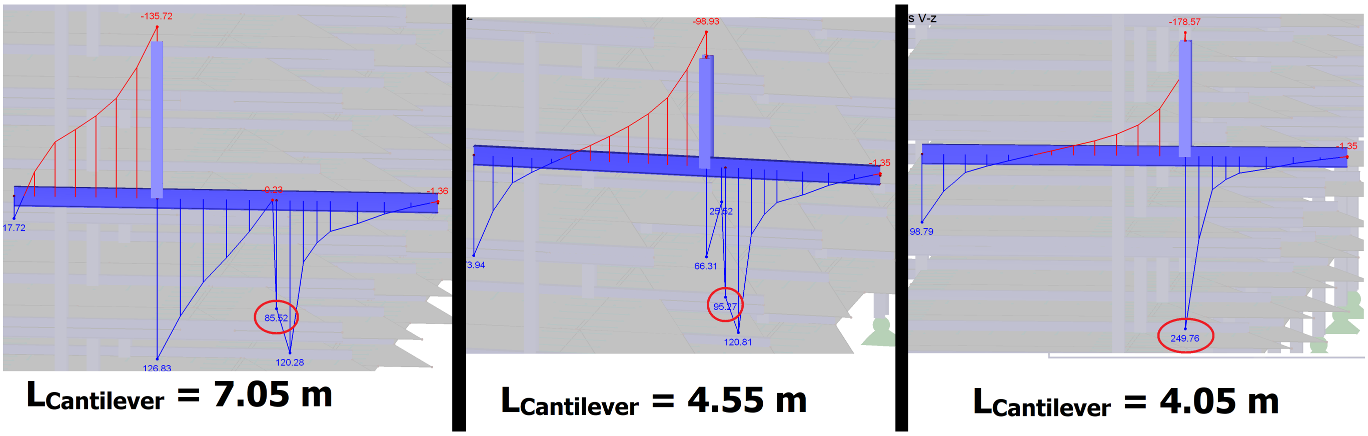


Figure C.15: Internal shear forces for varying cantilever spans (Load case 1)

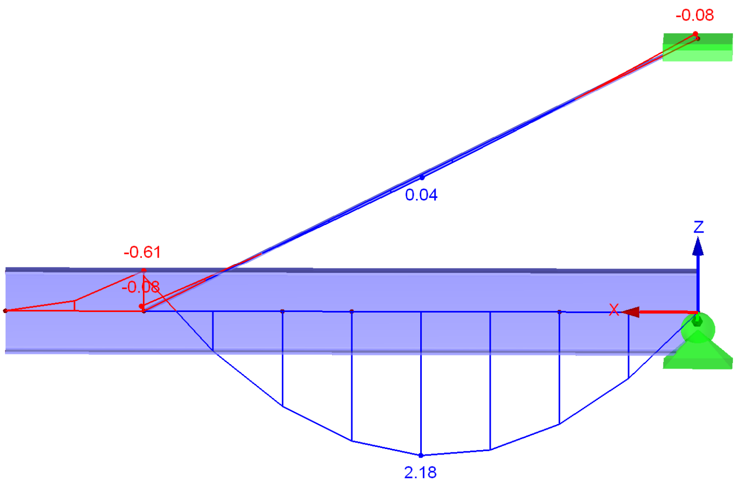


Figure C.16: Simplified version of the tie rod variant

C.5.2 Variant: Tie rods 7.2m

Explanation of the Isokorb unity checks The behaviour of the unity checks for the Isokorbs is explained as follows. The floor will still exert an upwards force at the transition between outside and inside. This does not happen when the column is placed at this transition point which explain the larger unity checks of option 1. For options 2 to 4 both bending moment and shear force unity checks decrease due to the column being further away from the Isokorbs as was the case for the previous variants.
For options 5 to 7 the bending moments increase while the shear forces stay constant. Figures C.17 and C.18 show the complex behaviour of the bending moments and shear forces at the Isokorb. Increasing the value of $L_{cantilever}$ has little effects on the shear force, because it is not a cantilever any more. The effects of the stiffness at the columns is gone, because now the y-beam is connected with a hinge. The bending moment however become more positive closer to the column. This

is unexpected and a simplified model was made without floors in figure C.16 to show the expected behaviour. The conclusion is that the stiffness of the floors affect the bending moments near the column. This happens becomes the floors are still continuous at this point while the y-beams are hinged.

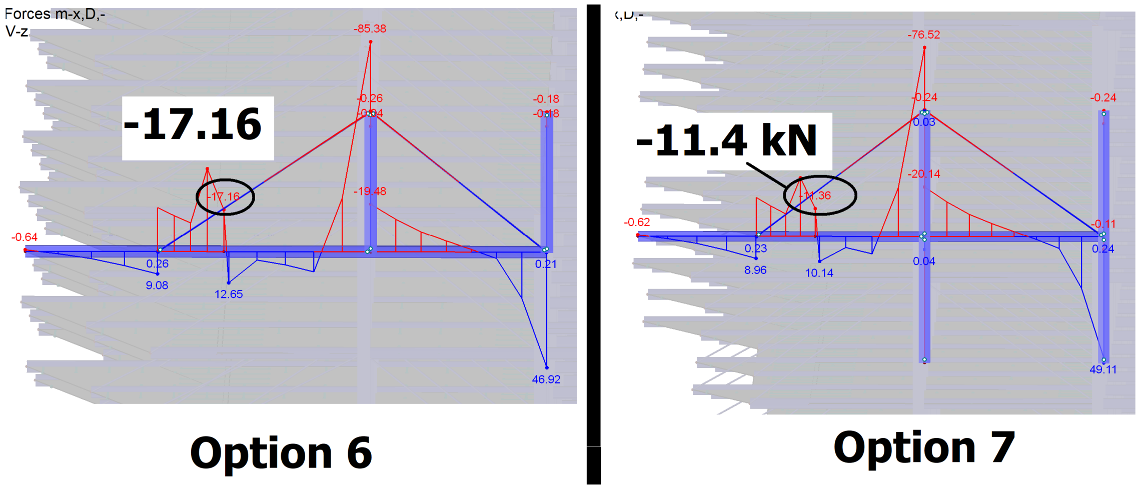


Figure C.17: Internal shear forces for varying cantilever spans

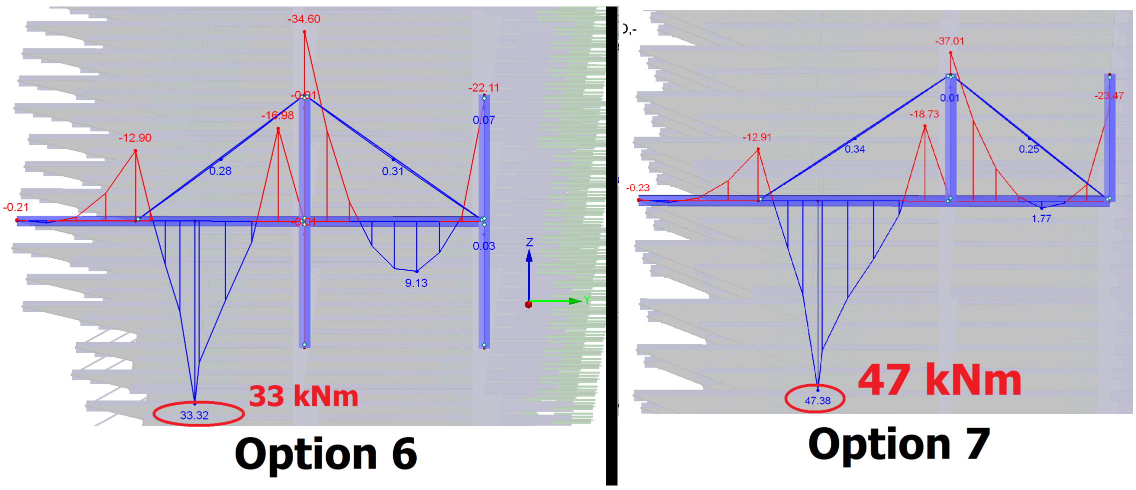


Figure C.18: Internal bending moments for varying cantilever spans

C.6 The results of the variant study

C.6.1 Variant span 7.2m: Results for varying ϕ

Option	Isokorb connection U.C. M	Isokorb connection U.C. V	Structural element	Cross section	Max. ULS U.C.	Max. ULS CO.	Max. ULS Design according to EC.	Max. SLS U.C	Max. SLS CO.	Max. SLS Design according to EC.
Option 1: ϕ = 15°	0,8	3,64	Column	HEA 340	0.99	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 360	0.75	CO4	- Lateral torsional buckling	0.30	CO15	Frequent z-direction
Option 2: ϕ = 18.7°	1,51	4,19	Column	HEA 400	0.88	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 400	0.90	CO4	-Shear and bending - Lateral torsional buckling	0.22	CO13	Characteristic z-direction
Option 3: ϕ = 22.5°	2,18	5,08	Column	HEA 400	0.95	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 500	0.76	CO4	- Lateral torsional buckling	0.33	CO13	Characteristic z-direction
Option 4: ϕ = 26.5°	3,04	6,31	Column	HEA 450	0.95	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 550	0.84	CO4	- Lateral torsional buckling	0.50	CO16	Frequent z-direction
Option 5: ϕ = 30°	4,12	7,58	Column	HEA 500	0.95	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.35	CO1	Lateral torsional buckling	0.56	CO17	Frequent z-direction
			Beams – Y	IPE 600	0.88	CO4	- Shear and bending - Lateral torsional buckling	0.64	CO16	Frequent z-direction
Option 6: ϕ = 33.7°	5,48	9,77	Column	HEA 500	-	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	—	CO1	Lateral torsional buckling	—	CO17	Frequent z-direction
			Beams – Y	IPE 600	1.18	CO4	- Lateral torsional buckling	-	CO16	Frequent z-direction

Figure C.19: Results: structural elements and Isokorb connection

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 1: ϕ = 15°	Characteristic combination	-	-19	1,03
	Frequent combination	-	-17	1,23
	Quasi-permanent combination	-	-17	0,91
	Max. horizontal translation	30		0,85
Option 2: ϕ = 18.7°	Characteristic combination	-	-24	0,84
	Frequent combination	-	-21	0,97
	Quasi-permanent combination	-	-21	0,72
	Max. horizontal translation	30		0,85
Option 3: ϕ = 22.5°	Characteristic combination	-	-29	1
	Frequent combination	-	-24	1,12
	Quasi-permanent combination	-	-24	0,82
	Max. horizontal translation	30		
Option 4: ϕ = 26.2°	Characteristic combination	-	-35	1,02
	Frequent combination	-	-29	1,14
	Quasi-permanent combination	-	-28	0,82
	Max. horizontal translation	30		0,85
Option 5: ϕ = 30°	Characteristic combination	-	-46	1,13
	Frequent combination	-	-38	1,27
	Quasi-permanent combination	-	-36	0,91
	Max. horizontal translation	30		0,85
Option 6: ϕ = 33.7°	Characteristic combination	-	-62	1,34
	Frequent combination	-	-52	1,5
	Quasi-permanent combination	-	-50	1,07

Figure C.20: Results: Deflections

C.6.2 Variant span 3.6m: Results for varying cantilever

Option	Isokorb connection		Structural element	Cross section	Max. ULS			Max. SLS		
	U.C. M	U.C. V			U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.
Option 1: $L_{cantilever}$ = 4.05 m	0,45	1,23	Column	HEA 240	0.77	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 330	0.95	CO2	- Lateral torsional buckling	0.75	CO13	Characteristic z-direction
Option 2: $L_{cantilever}$ = 4.55 m	0,44	1,05	Column	HEA 240	0.86	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 400	0.59	CO4	- Bending and shear - Lateral torsional buckling	0.80	CO13	Characteristic z-direction
Option 3: $L_{cantilever}$ = 5.05 m	0,44	1,02	Column	HEA 240	0.95	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 450	0.59	CO4	- Bending and shear - Lateral torsional buckling	0.94	CO16	Frequent z-direction
Option 4: $L_{cantilever}$ = 5.55 m	0,45	1	Column	HEA 260	0.88	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 550	0.49	CO4	- Bending and shear - Lateral torsional buckling	0.82	CO16	Frequent z-direction
Option 5: $L_{cantilever}$ = 6.05 m	0,45	1	Column	HEA 260	0.96	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 600	0.48	CO4	- Bending and shear - Lateral torsional buckling	0.87	CO16	Frequent z-direction
Option 6: $L_{cantilever}$ = 6.55 m	-	-	Column	HEA 280	-	CO	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	-	CO	Lateral torsional buckling	-	CO	Frequent z-direction
			Beams – Y	IPE 600	-	CO	- Lateral torsional buckling	1.07	CO16	Frequent z-direction

Figure C.21: Results: structural elements and Isokorb connection

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 1: $L_{cantilever}$ = 4.05 m	Characteristic combination	3	-27	0,83
	Frequent combination	3	-17	0,7
	Quasi-permanent combination	2	-14	0,45
	Max. horizontal translation	33		0,91
Option 2: $L_{cantilever}$ = 4.55 m	Characteristic combination	-	-33	0,91
	Frequent combination	-	-25	0,9
	Quasi-permanent combination	-	-22	0,61
	Max. horizontal translation	33		0,91
Option 3: $L_{cantilever}$ = 5.05 m	Characteristic combination	-	-41	1,02
	Frequent combination	-	-32	1,06
	Quasi-permanent combination	-	-30	0,75
	Max. horizontal translation	33		0.91
Option 4: $L_{cantilever}$ = 5.55 m	Characteristic combination	-	-38	0,85
	Frequent combination	-	-31	0,92
	Quasi-permanent combination	-	-29	0,66
	Max. horizontal translation	33		0.91
Option 5: $L_{cantilever}$ = 6.05 m	Characteristic combination	-	-42	0,87
	Frequent combination	-	-35	0,97
	Quasi-permanent combination	-	-34	0,69
	Max. horizontal translation	33		0.91
Option 6: $L_{cantilever}$ = 6.55 m	Characteristic combination	-	-	-
	Frequent combination	-	-	-
	Quasi-permanent combination	-	-	-
	Max. horizontal translation	-		-

Figure C.22: Results: Deflections

C.6.3 Variant span 3.6m: Results for varying ϕ

Option	Isokorb connection U.C. M	Isokorb connection U.C. V	Structural element	Cross section	Max. ULS U.C.	Max. ULS CO.	Max. ULS Design according to EC.	Max. SLS U.C	Max. SLS CO.	Max. SLS Design according to EC.
Option 1: ϕ = 15°	0,15	0,55	Column	HEA 220	0.79	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 330	0.98	CO2	- Lateral torsional buckling	0.70	CO15	Frequent z-direction
Option 2: ϕ = 18.7°	0,24	0,75	Column	HEA 220	0.87	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 330	0.90	CO2	- Lateral torsional buckling	0.61	CO15	Frequent z-direction
Option 3: ϕ = 22.5°	0,36	1	Column	HEA 220	0.96	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 330	0.89	CO2	- Lateral torsional buckling	0.48	CO15	Frequent z-direction
Option 4: ϕ = 26.2°	0,5	1,33	Column	HEA 240	0.80	CO4	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 330	0.98	CO2	- Lateral torsional buckling	0.91	CO13	Characteristic z-direction
Option 5: ϕ = 30°	0,7	1,44	Column	HEA 240	0.91	CO4	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 400	0.79	CO4	- Bending and shear - Lateral torsional buckling	0.90	CO13	Characteristic z-direction

Figure C.23: Results: structural elements and Isokorb connection

Option	Isokorb connection		Structural element	Cross section	Max. ULS			Max. SLS		
	U.C. M	U.C. V			U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.
Option 6: ϕ = 33.7°	0,94	1,52	Column	HEA 260	0.87	CO4	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 500	0.63	CO4	- Bending and shear - Lateral torsional buckling	0.80	CO13	Characteristic z-direction
Option 7: ϕ = 37.5°	1,25	1,76	Column	HEA 260	0.99	CO4	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 550	0.65	CO4	- Bending and shear - Lateral torsional buckling	0.91	CO13	Characteristic z-direction
Option 8: ϕ = 41.2	1,64	1,99	Column	HEA 280	0.98	CO4	Flexural buckling about z-axis	-	-	Negligible deformation
			Beams - X	IPE 80	0.05	CO1	Lateral torsional buckling	0.06	CO15	Frequent z-direction
			Beams – Y	IPE 600	0.68	CO4	- Bending and shear - Lateral torsional buckling	1.001	CO16	Characteristic z-direction

Figure C.24: Results: structural elements and Isokorb connection

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 1: ϕ = 15°	Characteristic combination	5	-17	0,91
	Frequent combination	5	-15	1,07
	Quasi-permanent combination	4	-14	0,76
	Max. horizontal translation	33		0,91
Option 2: ϕ = 18.7°	Characteristic combination	4	-16	0,69
	Frequent combination	4	-14	0,8
	Quasi-permanent combination	3	-13	0,57
	Max. horizontal translation	33		0,91
Option 3: ϕ = 22.5°	Characteristic combination	4	-18	0,61
	Frequent combination	3	-13	0,61
	Quasi-permanent combination	3	-12	0,43
	Max. horizontal translation	33		0.91
Option 4: ϕ = 26.2°	Characteristic combination	2	-33	0,97
	Frequent combination	2	-22	0,84
	Quasi-permanent combination	1	-19	0,55
	Max. horizontal translation	33		0.91
Option 5: ϕ = 30°	Characteristic combination	0	-42	1,05
	Frequent combination	0	-30	0,99
	Quasi-permanent combination	0	-27	0,67
	Max. horizontal translation	33		0.91
Option 6: ϕ = 33.7°	Characteristic combination	-	-42	0,91
	Frequent combination	-	-31	0,9
	Quasi-permanent combination	-	-29	0,62

Figure C.25: Results: Deflections

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 7: ϕ = 37.5°	Characteristic combination	-	-54	1,02
	Frequent combination	-	-41	1,03
	Quasi-permanent combination	-	-38	0,71
	Max. horizontal translation	-		-
Option 8: ϕ = 41.2°	Characteristic combination	-	-67	1,1
	Frequent combination	-	-51	1,12
	Quasi-permanent combination	-	-47	0,78
	Max. horizontal translation	-		-

Figure C.26: Results: Deflections

C.6.4 Variant Tie rods span 3.6m

Option	Isokorb connection U.C. M	Isokorb connection U.C. V	Structural element	Cross section	Max. ULS U.C.	Max. ULS CO.	Max. ULS Design according to EC.	Max. SLS U.C	Max. SLS CO.	Max. SLS Design according to EC.
Option 1: $L_{cantilever}$ = 4.05 m	0,57	1,38	Column	HEA 240	0.76	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 65	0.94	CO5	Bending and compression	-	-	-
			Beams – Y	IPE 360	0.45	CO2	Bending, shear and axial force	0.92	CO16	Frequent z-direction
Option 2: $L_{cantilever}$ = 4.55 m	0,19	0,85	Column	HEA 240	0.84	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 70	0.79	CO5	Bending and compression	-	-	-
			Beams – Y	IPE 270	0.84	CO2	Bending and compression	0.47	CO15	Frequent z-direction
Option 3: $L_{cantilever}$ = 5.05 m	0,05	0,5	Column	HEA 240	0.91	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 55	0.86	CO5	Bending and compression	-	-	-
			Beams – Y	IPE 240	0.89	CO2	Bending and compression	0.51	CO16	Frequent z-direction
Option 4: $L_{cantilever}$ = 5.55 m	0,07	0,08	Column	HEA 260	0.82	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 35	0.82	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 200	0.88	CO4	Bending and compression	0.68	CO16	Frequent z-direction
Option 5: $L_{cantilever}$ =6.05 m	0,17	0,41	Column	HEA 260	0.90	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 35	0.90	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 180	0.97	CO4	Bending and compression	0.89	CO15	Frequent z-direction

Figure C.27: Results: structural elements and Isokorb connection

Option	Isokorb connection		Structural element	Cross section	Max. ULS			Max. SLS		
	U.C. M	U.C. V			U.C.	CO.	Design according to EC.	U.C	CO.	Design according to EC.
Option 6: $L_{cantilever}$ =6.55 m	0,31	0,43	Column	HEA 260	0.86	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 40	0.85	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 240	0.75	CO2	Bending and compression	0.97	CO15	Frequent z-direction
Option 7: $L_{cantilever}$ =7.05 m	0,45	0,4	Column	HEA 280	0.97	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 45	0.90	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 300	0.64	CO2	Bending and compression	0.98	CO15	Frequent z-direction

Figure C.28: Results: structural elements and Isokorb connection

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 1: $L_{cantilever}$ = 4.05 m	Characteristic combination	-	-13	0,39
	Frequent combination	-	-11	0,46
	Quasi-permanent combination	-	-11	0,33
	Max. horizontal translation	15		0,42
Option 2: $L_{cantilever}$ = 4.55 m	Characteristic combination	-	-17	0,48
	Frequent combination	-	-14	0,52
	Quasi-permanent combination	-	-14	0,37
	Max. horizontal translation	15		0,43
Option 3: $L_{cantilever}$ = 5.05 m	Characteristic combination	-	-22	0,54
	Frequent combination	-	-18	0,58
	Quasi-permanent combination	-	-17	0,41
	Max. horizontal translation	16		0,45
Option 4: $L_{cantilever}$ = 5.55 m	Characteristic combination	-	-30	0,68
	Frequent combination	-	-24	0,71
	Quasi-permanent combination	-	-22	0,5
	Max. horizontal translation	18		0,49
Option 5: $L_{cantilever}$ = 6.05 m	Characteristic combination	-	-34	0,69
	Frequent combination	-	-26	0,72
	Quasi-permanent combination	-	-25	0,51
	Max. horizontal translation	19		0,53

Figure C.29: Results: Deflections

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Option 6: $L_{cantilever}$ = 6.55 m	Characteristic combination	-	-36	0,69
	Frequent combination	-	-29	0,74
	Quasi-permanent combination	-	-27	0,52
	Max. horizontal translation	20		0,56
Option 7: $L_{cantilever}$ = 7.05 m	Characteristic combination	-	-39	0,7
	Frequent combination	-	-33	0,77
	Quasi-permanent combination	-	-31	0,55
	Max. horizontal translation	21		0,59

Figure C.30: Results: Deflections

C.6.5 Miscellaneous variants

Option	Isokorb connection U.C. M	Isokorb connection U.C. V	Structural element	Cross section	Max. ULS U.C.	Max. ULS CO.	Max. ULS Design according to EC.	Max. SLS U.C	Max. SLS CO.	Max. SLS Design according to EC.
Variant: Tie rods span 3.6m	0,45	0,4	Column	HEA 280	0.97	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 45	0.90	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 300	0.64	CO2	Bending and compression	0.98	CO15	Frequent z-direction
Variant: Tie rods span 7.2m	0,74	1,74	Column	HEA 900	0.94	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 65	0.92	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 300	0.98	CO2	Bending and compression	0.97	CO15	Frequent z-direction
Variant: Tie rods V2 span 3.6m	0,46	0,42	Column	HEA 280	0.97	CO2	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 55	0.80	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 300	0.65	CO2	Bending and compression	0.98	CO15	Frequent z-direction
			Columns below roof	HEA 100	0.28	CO3	Flexural buckling about z-axis	-	-	-
Variant: Tie rods V2 span 7.2m	0,85	1,55	Column	HEA 900	0.94	CO1	Flexural buckling about z-axis	-	-	Negligible deformation
			Tie rods	RD 75	0.91	CO4	Bending, shear and axial force	-	-	-
			Beams – Y	IPE 330	0.89	CO2	Bending and compression	0.95	CO15	Frequent z-direction
			Columns below roof	HEA 100	0.88	CO3	Flexural buckling about z-axis	-	-	-

Figure C.31: Results: structural elements and Isokorb connection

Option	Combination	Max. slab Displacement [mm]		Displacement down: Unity check (U.C.<1)
		Up	Down	
Variant: Tie rods span 3.6m	Characteristic combination	-	-39	0,7
	Frequent combination	-	-33	0,77
	Quasi-permanent combination	-	-31	0,55
	Max. horizontal translation	21		0,59
Variant: Tie rods span 7.2m	Characteristic combination	-	-50	0,89
	Frequent combination	-	-44	1,05
	Quasi-permanent combination	-	-43	0,76
	Max. horizontal translation	24		0,66
Variant: Tie rods V2 span 3.6m	Characteristic combination	-	-39	0,7
	Frequent combination	-	-33	0,78
	Quasi-permanent combination	-	-32	0,56
	Max. horizontal translation	20		0,57
Variant: Tie rods V2 span 7.2m	Characteristic combination	-	-57	1,01
	Frequent combination	-	-52	1,23
	Quasi-permanent combination	-	-51	0,9
	Max. horizontal translation	23		0,65

Figure C.32: Results: Deflections