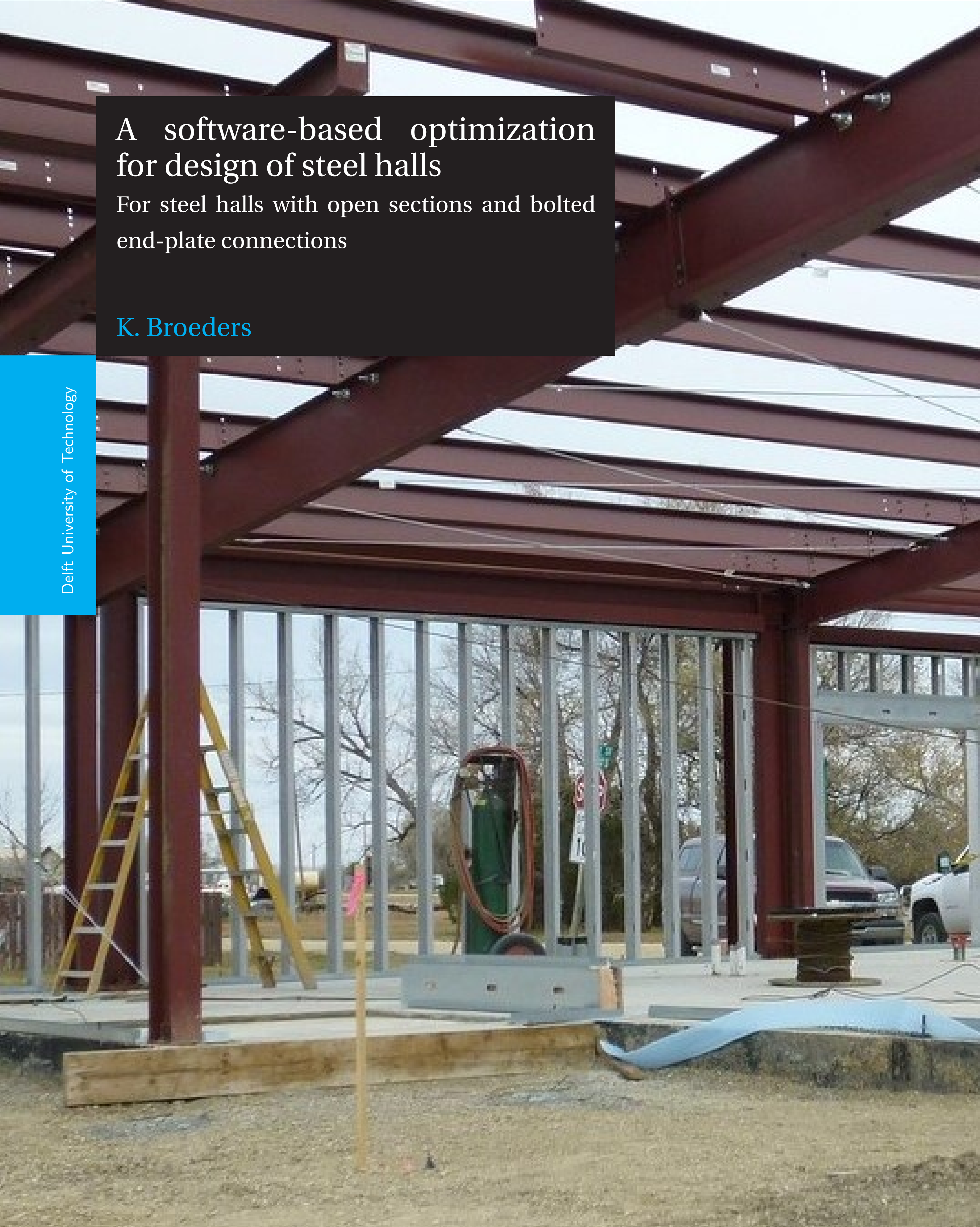
A software-based optimization for design of steel halls

For steel halls with open sections and bolted end-plate connections

K. Broeders

Delft University of Technology



TU Delft Delft University of Technology

Challenge the future

A SOFTWARE-BASED OPTIMIZATION FOR DESIGN OF STEEL HALLS

FOR STEEL HALLS WITH OPEN SECTIONS AND BOLTED END-PLATE CONNECTIONS

by

K. Broeders

in partial fulfillment of the requirements for the degree of

Master of Science
in Hydraulic Engineering

at the Delft University of Technology,
to be defended publicly on Monday March 1, 2021 at 13:00.

Supervisor:	Prof. dr. M. Veljkovic	TU Delft
Thesis committee:	Ir. P. A. de Vries,	TU Delft
	Ir. L. P. L. van der Linden	TU Delft
	Ir. D. Ntroumpis,	Temporary Works Design

An electronic version of this thesis is available at http://repository.tudelft.nl/.

Preface

This report is the result of my graduation research project. This is the final part of my master curriculum of Hydraulic engineering at Delft University of Technology.

During my bachelors of civil engineering I have always been interested in automating processes. This is why I always tried to automate all processes that are repetitive. I also noticed in my bachelors that during the design of a civil structure there are a lot of repetitive tasks, due to the iterative process of the design. In my minor project management I learned that it is difficult to estimate costs of structures. That is why in this research I wanted to focus on the cost optimization.

This research was performed at "Temporary Works Design" in Rotterdam, where I was guided by Dimitrios Ntroumpis. I would like to thank Dimitrios for helping me form the idea and for all the time and effort he put into the guidance of my thesis. I would like to thank George Tzanidakis for always being their to discuss my questions. I would like Bas Nederveen that helped me with all my programming questions and problems. And I would like to thank everybody else in the company for always taking the time to answer my questions.

I would like to thank my other supervisors, Milan Veljkovic, Lennert van der Linden and Peter de Vries, who guided me from the start of my project.Your comments and remarks helped me steer in the right direction throughout the graduation process.

Finally I would like to thank Arnout Klink from Crowners services for helping me with the practical questions I had about connections.

K. Broeders
Rotterdam, January 2021

Abstract

Engineering a structure in the civil world, could mean optimizing a structure to a certain variable. This optimization could be to weight, strength or costs. The most common way of optimizing steel halls is an optimization to weight. A limitation of this method is that the connection design is not considered in the optimization process. The connections can play a big role in the costs of a steel hall. The type of connection (hinged, semi-rigid, rigid) influences the profiles used in the structure. To create a more precise cost optimization of a steel structure, it is important to include the connection design from the start of the optimization process. To create this optimization process knowledge based engineering software can play a role, because this can help including all the engineering and design rules in the optimization process.

This thesis starts with a literature study. In the literature the following statements are found:

- The most cost-effective option should not always be the lightest option.
- The semi-rigid case beam-column connection should give the most cost-efficient total costs of the structure.
- The profile costs should be most cost-efficient for the rigid beam-column connection.
- The connection costs should be most cost-efficient for the hinged beam-column connection with the most frames.

To optimize steel halls for this thesis, an optimization tool is created. This tool creates a parametric model and optimizes this model to its costs. The connections in the tool are limited to beam-column connections. The optimization starts with the input variables of the model, the most important input variable is the type of the beam-column connection. This can vary between a hinged, semi-rigid or fully rigid connection. Other inputs are the list of profiles that need to be considered, the loads acting on the hall and the main dimensions and topology.

With all of these inputs the parametric model can be created. This model is then put into the finite element software RFEM, which calculates all the internal forces and deformations in the structure. Then with python code created for this thesis the strength and deformations for the structure are checked according to NEN-EN 1993. In case the structure is not sufficient the profile sizes are increased. This keeps increasing until the structure is sufficient.

For the connection design used in this study a database is created with all possible bolted end-plate connections of the four different connection designs used in this research. Of all these possible connections the stiffness and bending moment resistance were calculated with the component method. These values are added in the database. With the selected connection input a list is created from this database with all the connections that have a stiffness within the range of ± 10% of the wanted stiffness. Then for each connection in this list the cost is calculated. This list is then sorted on the costs from lowest to highest cost. After the complete connection list is created, the first connection of this list is checked in the component based finite element software IDEA StatiCa for the deformations and strength. In case the connection is sufficiently strong according to the Eurocode, the stiffness of the connection with the actual forces is checked with the component method. This is a python script created for this tool. In case the stiffness is not within the wanted range it checks the second connection from the list.

After the connection loop all the results of the optimized structure are saved. When all the results are saved, the loop is repeated for different topologies and different connection types. With these results the most cost-efficient structure design can be found including the connection design.

This tool could help answering the question on how knowledge-based engineering software influences the cost optimization of steel halls. The results of this tool are linked to the theories of cost optimization of steel structures including bolted connections. To get results from this tool a case study is used. This case study is based on a distribution hall. This limits the variables used in the parametric model. The variables that are used for the case study are:

Length	30 m
Width	10 m
Height	5 m
Steel grade	S355
Column profiles	HEA, HEB
Beam profiles	IPE
Purlin profiles	UPE
Loads	According to EN in the Netherlands snow zone 2 and wind area 1 unbuilt
Column-base connection	Hinged or Rigid
Number of purlins	5 to 9 with steps of 2
Number of frames	4 to 13 with steps of 1
Number of columns per frame	2
Beam-column connection	Hinged, low semi-rigid, medium semi-rigid, high semi-rigid or rigid

With the results of this case study the following observations are made:

- The optimization process proves for the cases used in this research, that the most cost-effective option is not always the same as the lightest option.
- The optimization process proves for the cases used in this research, that the semi-rigid case most often gives the most cost-efficient result.
- The optimization process proves for the cases used in this research, that the costs of the profiles are most cost-efficient for the rigid or high semi-rigid beam-column connection.
- The optimization process proves for the cases used in this research, that the connection costs are most cost-efficient for the beam-column connection with the lowest stiffness and the most frames.
- With the knowledge-based engineering approach used in this research it is possible to move the detailed connection design to the concept design phase.
- With the knowledge-based engineering approach used in this research it is possible to include the correct stiffness of a connection in the concept design phase.
- With knowledge-based engineering software it is possible to create an automated design process.

This tool has its limitation to bolted end-plate connections and single storey steel halls. Although this tool can be expanded with different types of connections. In this case a database needs to be created for these connection designs. Also the connection parameters needs to be added to the design parameters. The method used for cost optimizations can be used for all types of steel structures with open profiles, in this case the topology needs to be updated.

Contents

List of Figures

List of Tables

Nomenclature

API	=	Application Programming Interface
BIM	=	Building Information Modelling
CAD	=	Computer Aided Design
CAPEX	=	Capital Expenditures
CBFEM	=	Computer Aided Design
c.t.c.	=	Centre to centre
deg	=	Degree
FEM	=	Finite Element Method
KBE	=	Knowledge-Based Engineering
kg	=	kilogram
LCA	=	Life Cycle Analysis
m	=	meter
mm	=	millimeter
MPA	=	megapascal
NPV	=	Net Present Value
OPEX	=	Operating Expenses
PDE	=	Partial differential equation
OT	=	Optimization tool
VBA	=	Visual Basic

1

INTRODUCTION

In this chapter an introduction is written. First, a motivation for this research is discussed. Secondly, the research question and the sub-questions are stated. At the end, the structure of this research is described and a global overview of the optimization process used in this research is given.

1.1. MOTIVATION FOR RESEARCH

Engineering a structure in the civil world, could mean optimizing a structure to a certain variable. This variable changes depending on the type of structure and the client [42]. This variable could be for example:

- Weight
- Strength
- Costs

Optimization to weight of steel structures can be done from the start, because this depends mostly on the steel profiles. Steel structures could also be optimized more easily to its strength from the start of the design in case the type of connection is known (hinged/rigid), because this is mainly influenced by the main steel used in the structure. To optimize a structure to its costs is however more difficult because the costs depend on a lot of different factors that influence each other and influence the strength and weight of the structure. This is why a lot of engineering firms optimize a structure to its weight when a low-cost option is wanted, because the weight is directly linked to the costs. However the weight does not play the only role in the costs of a structure. For example when a structure has a lot of connections, decreasing the weight of a structure could increase the costs of the connection. Which could lead to higher total costs of the structure. When this is compared to a structure with a higher weight of the main steel and lower priced connections, the first option could result in higher total costs of the structure.

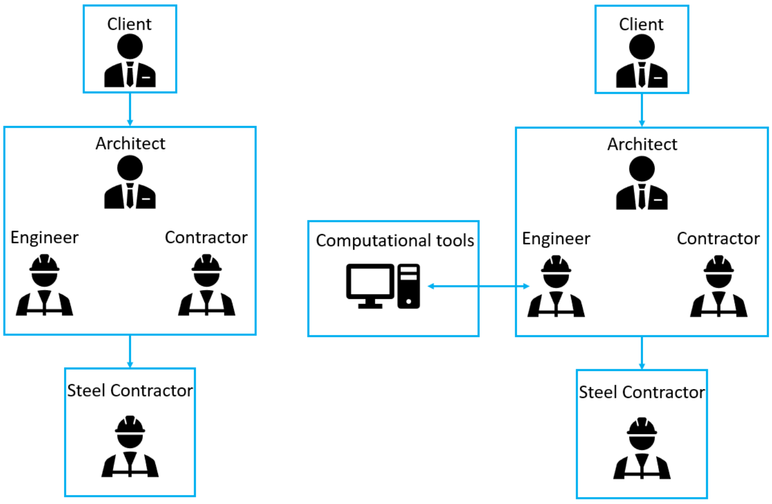
Despite the selected optimization variable, an optimization needs multiple iterations. This means that an optimization tool could help engineers a lot in the design process [1] [43].

In the design process of steel structures multiple parties can be involved. The different parties that could be involved are:

- The client
- The architect
- The engineer
- The contractor
- The steel contractor

The relation between these parties can be found in figure 1.1a.

In recent years engineers design their structures with increasing usage of computational tools that can help them with difficult calculations or with the iterations that need to be done to optimize a structure, see figure 1.1b. The usage of these computational tools help engineers do more complex calculations and more iterations in a shorter period of time. This makes it possible to optimize in more detail.



(a) old way of working (b) new way of working

Figure 1.1: Old vs new way of working

When looking at the smaller civil engineering projects, where there are multiple engineering firms competing on one project. The optimization variable of the structure is most often costs [44]. The more precise the cost estimations get the higher the chance to make a good and accurate estimation and the higher the chance on winning the bid.

Optimization to costs of steel structures is often done by estimating the weight of the main elements of the structure and adding a certain percentage for the rest of the costs [45] [46] [47] [48] [49]. However it can be found that the material cost of the main elements is less than half the total costs of the structure for a typical steel framed multi-storey commercial building in Europe, see figure 1.2. A big influence on the other parts of the costs as well as the material costs of the main elements of the structure are the connections [1]. For single storey steel structures the raw material costs are relatively bigger compared to the multi-storey building from figure 1.2. However the connections still have a significant influence on the structure [1].

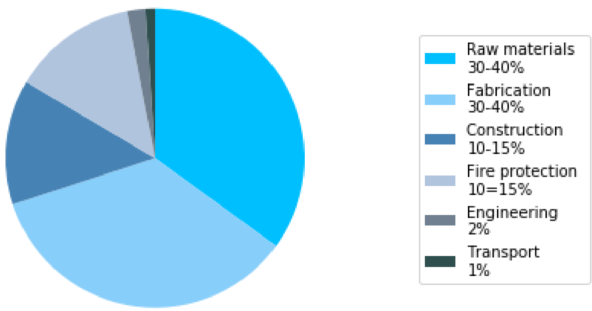


Figure 1.2: Breakdown of costs of steel frame for a typical multi-storey commercial building in Europe [1]

The best moment for a cost optimization in a project is in the beginning of the design phase, because in this phase an engineer has the most influence on the project cost as can be seen in figure 1.3.

This means that to do a more detailed cost optimization of a structure which could lead to cheaper structures, as much details as possible should be considered in the beginning of the design process. A detail that has a big influence on the costs of a structure is the connection design.

This is why the main purpose of this thesis is to introduce a knowledge-based method in order to optimize a steel frame structure including the bolted end plate connections.

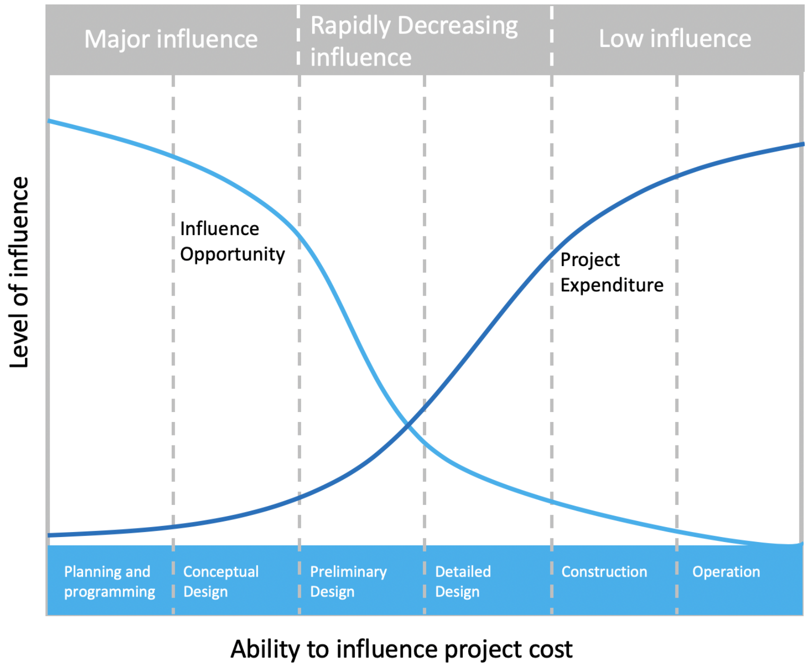


Figure 1.3: Influence and cost distribution over project time span [2]

1.2. RESEARCH QUESTIONS

In literature it can be found that cost optimization of steel structures can be done by changing one or multiple of the following aspects of the structure [46] [50]:

- The topology of the structure
- The shape of the structure
- The size of the profiles

In other studies of steel structure optimization including connections, multiple studies were found for optimizations to strength [4] [51] [52] [53]. In these studies the structure was optimized according to an assumed connection stiffness or the connection was optimized without considering the complete structure.

When cost optimizations that include connections are studied previous research showed that it is difficult to optimize for bolted connections due to all the variables like the type of bolted connection, the welds and the plate dimensions [52] [53]. There is a cost optimization model created by R. Ajouz that includes the overall cost of a structure including welded connections [54]. There is however no cost optimization model found that includes the overall cost of a structure including bolted connections. This leads to the following research question:

In what way can knowledge-based engineering software influence the cost optimization of steel halls with open sections and bolted end-plate beam-column connections?

The research question is answered using the following sub questions, these sub questions are divided into the subjects they relate to.

Theory

1. What is knowledge-based engineering and what are the advantages and disadvantages?
2. What methods exist to optimize steel structures to cost?
3. What cost model can be used to calculate the cost of steel structures?
4. What are the design steps made in the standard design of a steel hall and what aspects have the most influence on the strength and stiffness of the structure?

Optimization tool

5. How should the optimization tool be composed?
6. How does the optimization tool function and how does it differ compared to existing tools?
7. What variables are selected to be parametrized and what results are needed to make a cost comparison?

Connection

8. How is the connection design considered in the concept design phase?
9. What constraints are applied in the connection design and why these constraints?
10. How are the connections considered in the overall optimization?

Case study and results

11. What parameters have the most influence on the costs of the structure?
12. What are the results of this study?
13. Are the results as expected according to theory?
14. What are further possibilities of the methodology used in this research?

1.3. RESEARCH STRUCTURE

This thesis is divided into six chapters. The structure of the thesis is described below.

In chapter two, a literature study is presented. This literature study holds the theory that is applied in the research. The chapter starts with an introduction about knowledge-based engineering. Secondly, the existing methods to optimize steel structures to costs are described. Thirdly, the cost model that can be used to calculate the costs of steel structures is discussed. At last, the design steps used in the design of steel halls are described including the aspects that have the most influence on the strength and the stiffness of the hall.

In chapter three, the optimization model is described. The chapter starts with a description about the most important steps in the optimization tool. Secondly, a description of how the tool works is given in more depth, and a description is given about how it differs with existing tools. Thirdly, the variables that are parametrized and the results that are needed for a cost optimization are described. At the end of this chapter, the further possibilities of the method and tool used in this research are discussed.

In chapter four, the connection design in the concept phase is described in more detail. The chapter starts with how the connection design is considered in the concept phase. Subsequently, the constraints used in the connection design are discussed. At the end of the chapter, a description is given on how the connections are considered in the overall optimization to costs.

In chapter five, the case study is presented. The knowledge gained from the research in previous chapters are implemented in a case study. Firstly, the parameters that are selected for this case study are described. Secondly, the parameters that have the most influence on the costs of the structure are discussed. At the end of the chapter, there is reflected back on the results to see if the results of the case study are as expected according to theory.

In chapter six, a concise answer to all the sub-questions and the main-question are given and the conclusions of the total work and recommendations for further research are presented.

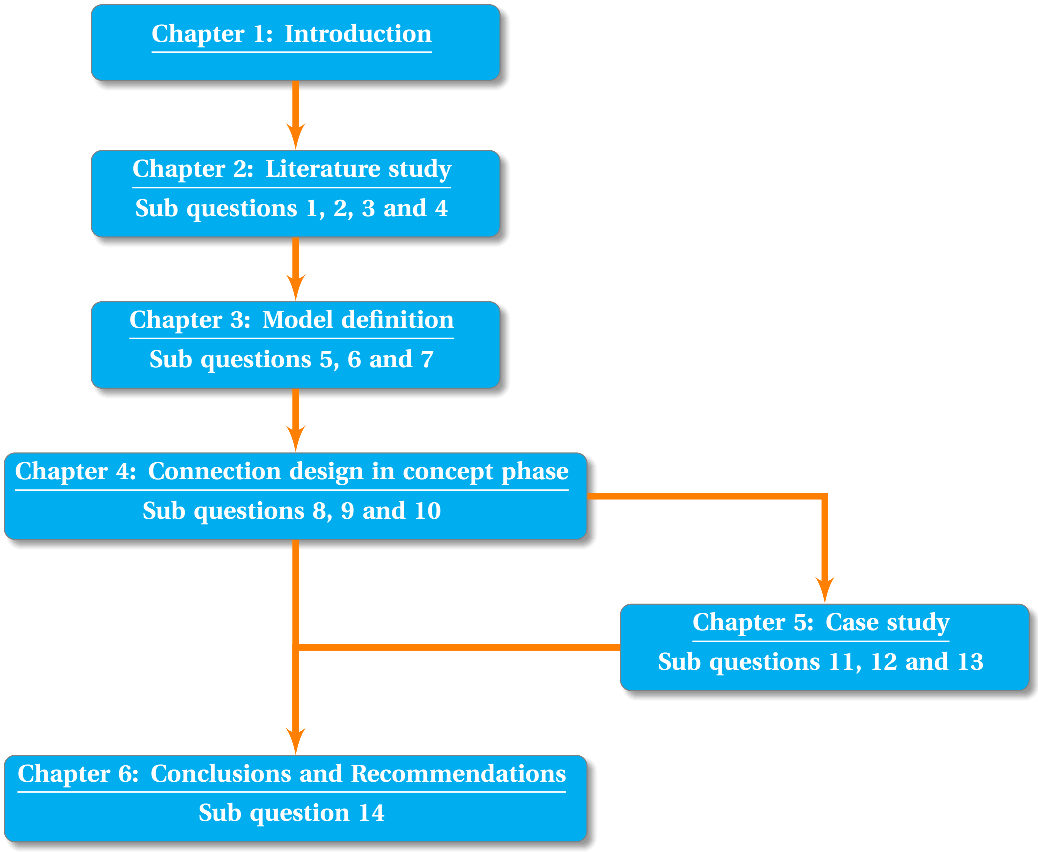


Figure 1.4: Overview of the thesis structure including sub questions

1.4. SCOPE

The scope of this thesis is on the preliminary design phase. The focus will be on the optimization model to include detailed connection designs in the concept design phase for steel structures with open sections.

Due to the time limitation of this research there will be looked at one case study in detail. There are two other cases studied with less variables. In later research this methodology can be tested on more cases.

Because of the many different connections that are possible in steel structures the optimization model used in this thesis will focus on the beam-column connection for open cross section profiles. After this research this model can be expanded to include other types of connections like the column-base or the beam-splice connections. This tool is set up as a general tool that can easily be expanded for different structures, load cases or connections after this research.

1.5. OPTIMIZATION PROCESS

To answer the main question a tool will be built that uses knowledge-based engineering software. This tool will link multiple programs and optimization steps. An overview of these steps can be found in figure 1.5 including the software used for each step.

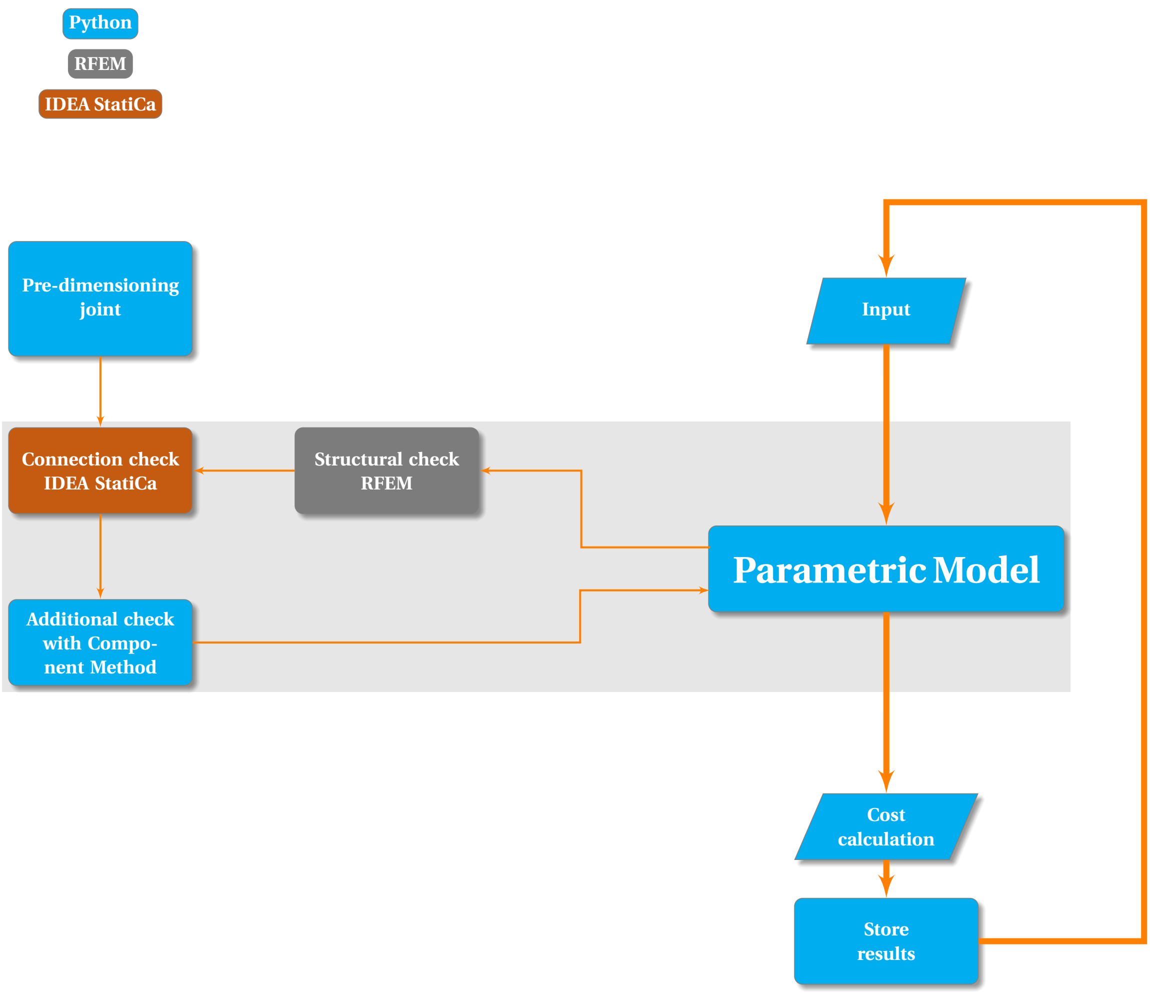


Figure 1.5: Flow diagram of optimization process

2

LITERATURE STUDY

In this chapter the literature study will be described. This literature study is done by researching articles, reports, websites and other studies. The chapter starts with an introduction about knowledge-based engineering and the parametric design process. Secondly, the existing methods to optimize steel structures and connections to costs are discussed. Thirdly, there is looked at different cost models that can be used to calculate the cost of a steel structure. At the end of the chapter, the design steps used in the design of steel halls are described. including the aspects that have most influence on the strength and stiffness of the steel hall.

2.1. PARAMETRIC DESIGN PROCESS AND KNOWLEDGE-BASED ENGINEERING

In this section an introduction is given into the parametric design process and knowledge based engineering (KBE). At the end of this section, the software that can be used for a parametric design including KBE will be discussed.

2.1.1. PARAMETRIC DESIGN PROCESS

Parametric design is getting more and more used in civil engineering. This is because parametric design makes it possible to compare more options in more detail in a shorter period of time [55]. This helps with the optimization of the structure.

To make a parametric design, parametric engineering tools are used, more about these tools can be found in subsection 2.1.3. In all these tools the following steps need to be made [56]:

1. Formulation of the design problem
2. Creation of the logic to create a design space with a lot of variants
3. Investigation of the variants
4. Optimization of the design problem to derive a satisfying solution
5. Visualization of the results and the developed design

Steps 1 to 4 are described in more detail below, the first four steps in this process can be done with KBE software, more about KBE can be found in subsection 2.1.2.

The first step is to formulate the design problem, this is done with knowledge from all the different disciplines involved. It is important to involve all the disciplines to make sure that no essential variants are forgotten. The knowledge captured from these disciplines should then be transferred to a general design problem where all the variables are determined [56]. These variables will be the input values for the parametric model.

The second step is the creation of logic to create the design space. In this step the logic should be put in the model. This logic determines the data flow to get from your input variables to the results. Part of this logic are the mathematical equations that link the input variables. But this logic could also consist of the design constraints of the structure [45].

When the design problem and the logic is put in the parametric tool all the results can be calculated. The third step is then to investigate and compare all the different results to each other. This could lead to more constraints that can be put back into the model, or it could result in a final design [57].

The fourth step is to automatically compare all the different results to each other and optimize the model to different aspects. More about the optimization process can be found in section 2.2.

2.1.2. Knowledge-based engineering

The idea behind KBE is to capture the knowledge of engineers and put this in engineering and designing rules. This is done to reduce the time an engineer has to spend on repetitive tasks and designs. The advantages and disadvantages of KBE are described below [58], [59].

The advantages of KBE are:

- Reduced lead time
- Product optimization
- Extra time for innovation

The reduced lead time is realized due to the fact that the design process can be performed automatically if the KBE system includes the complete process and if all the needed inputs are given in the KBE system. KBE software also increases the re-usability of knowledge. If the knowledge is programmed in, the engineers do not have to look it up or include more senior engineers to gain the knowledge, which saves time and costs.

The product optimization is realized because if KBE software is used all the lessons learned from previous errors can be programmed in. It is also more effortless to find the optimum design in the given range, because the computer can do an infinite amount of iterations if wanted.

The extra time for innovation comes from the time saved on doing repetitive tasks, which is now done by the KBE software.

The disadvantages of KBE are:

- Building a KBE system takes a lot of time, skills and cost
- KBE software can become a black box

It can take a lot of time and costs to put all the engineering knowledge of a design into KBE software. Next to this some software packages also take a lot of programming skills, which not every engineer has.

Another disadvantage that can occur is that the KBE program becomes a black box. That the engineer only understands the input and the output, but has no knowledge about the design process that happens in between.

Because KBE software captures all the generic knowledge of a design, and it can help in the optimization process. It is ideal to use in the optimization of a common structure with the parametric design process.

2.1.3. Software

In this subsection, it is described what parametric design software is and what software choices there are. In the end there is described what software there will be used in this research.

Parametric design

"Parametric design is design of a structure where variables are used as input for a calculation or an algorithm." [57] According to this definition almost every engineer uses parametric design nowadays, because calculations in excel or designs in Computer Aided Design (CAD) programs can already be seen as parametric design.

Some of the possibilities of parametric design are that the design process can be optimized by connecting the design with calculations, meaning that changes in a later phase of the project can be easily altered because the calculations can be rerun without a lot of effort when the design changes. Parametric design can also bring more efficiency to the process when the models are connected to production processes [57].

Parametric software

Parametric software is used to make parametric designs. The uniqueness of parametric software is that the user is able to explicitly define the parameters, meaning that the user can choose more than just the value, but he can also define what values should be entered. There are also parametric software packages that can include a lot of engineering knowledge, like formulas and design choices. Besides using existing software another option is to create a KBE software packages that includes all the design choices, parameters and formulas needed for that specific design.

Parametric design software can be used in different phases of the design process. The more parametric design is used, the more can be automated in the design process. For example when the end product is known it is possible to use automated engineering. This is a method where multiple variables are put in and this can lead to a complete design where even cost, time and/or environment can be considered [60].

A different way of parametric design is to use it in the concept phase where there are a lot of unknown parameters meaning that it is impossible to calculate and consider each option without parametric design. Parametric design makes it possible to create and calculate multiple options if there are certain parameters known. This is very useful in the concept phase of a project where the final design is not yet known [61].

From the literature above it can be seen that parametric modelling is getting more and more to the stage where engineers are designing the process instead of the final product.

Used software

In this research the following software will be used:

- Visual Basic (VBA)
- Python
- C#
- Excel
- RFEM (FEM software)
- IDEA StatiCa (CBFEM software)

The link between the used programs can be found in figure 3.3.

2.1.4. FEM/ CBFEM

In this section FEM and CBFEM are explained in more detail, because these are used in the optimization model in this thesis.

Finite element method

The finite element method (FEM) is used to determine force distribution in the structure. This is used because the laws of physics for space and time dependent problems, like the reactions to forces, can only be mathematically expressed in partial differential equations (PDE's). These PDE's can only be solved with analytical methods for very simple problems and geometries. When the problem or the geometry gets more complex an approximation of the PDE's can be discretized with numerical model equations. These can be solved by numerical methods. Meaning that the finite element method is used to compute the approximations of these PDE's [62]. This means in general that the structure is broken down into many finite elements and the numerical methods describe the behaviour of every element with equations. These equations are summed up for all individual elements to predict the behaviour of the total structure.

Component based finite element method

Component based finite element method (CBFEM) is a method to analyse and design connections of steel structures. It is a combination of the component method and FEM. CBFEM splits the whole joint into separate components, steel plates, welds, bolts, anchors and concrete blocks. From each component the analysis model is created. All plates are meshed with shell elements. Bolts and anchors are represented by special non-linear springs and welds are modelled as constraints enabling the stress redistribution due to their plastification [3]. CBFEM is a quick way to validate the connections check. CBFEM is a design-oriented finite element analysis, which means that it is less accurate compared to a complete FEM simulation of the connection or experiments, see figure figure 2.1.

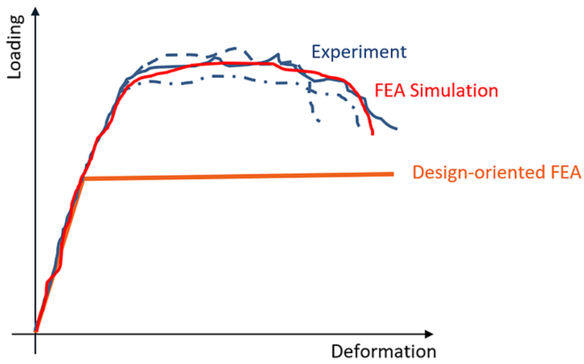


Figure 2.1: Design-oriented finite element analysis [3]

2.2. OPTIMIZATION

Optimization has a very broad meaning. That is why in this section it is narrowed down to the optimization used in this research. In this research there will only be looked at a cost optimization. In this section the state of the art of optimizing a steel frame design and the optimization of bolted joints are described. Secondly, the optimization of steel frames including joints is discussed. At the end of this section a small conclusion is given about what will be used in this research.

2.2.1. STEEL FRAME OPTIMIZATION

A lot of civil engineering firms use Building Information Modelling (BIM) methods and Life Cycle Analysis (LCA) methods to integrate the multi-disciplinary design information along the structure's lifetime. With these methods an iteration process is done to find an optimized solution [50]. There are three categories of structural optimization of steel frames that are often used [46] [50]:

- Topology optimization (number of elements)
- Shape optimization (total shape of structure)
- Size optimization (changing profile sizes)

These structural optimizations are an iterative process. The duration can be reduced by parametric design software that helps with the iterations, this is discussed in section 2.1.3.

In parametric design software a parametric model can be created. This can be connected with a calculation program to quickly calculate multiple different options and see their possibilities.

When looked at optimization of steel structures in literature it is often found that the optimization is an optimization to strength. There are some studies found of optimizing to costs. These are discussed in section 2.3. All of the literature found for this part is solely about the optimization of the frames. The connection design is not considered in these studies. In all of these studies is an assumption made for the connection stiffness at the beginning of the design process, where the connections are either fully rigid or hinged.

2.2.2. BOLTED JOINTS OPTIMIZATION

To optimize frames with bolted connections the difficulty is the additional angle deformation that is depending on multiple parameters such as: the type of bolted connection, elongation of bolts and plate dimensions [4] [52] [53]. Most often, in the concept phase the price is determined on the weight of the beams [52]. Meaning that to get the lowest costs there is looked for the lowest weight. When the optimization of the weight of the main elements of a structure is done, the connection design is optimized as far as possible without changing the beam cross-sections.

To consider the bolted connection at the beginning of the design process with semi-rigid joints, the stiffness of the joint needs to be determined before the detailed design phase. This can be done by using the "guess formula" that helps estimating the stiffness of the connection at the beginning of the design [4]. This formula can take both rigid and semi-rigid connections into account. It has the following limitations:

- The design is restricted to European H and I-sections

- The design must have two bolt rows in tension
- The bolt diameter needs to be approximately 1.5 times the thickness of the column flange
- The location of the bolt must be as close as possible to the column flange, the beam flange and the beam web
- The end plate thickness must be the same as the column flange thickness

This means that there is a formula that helps considering the stiffness of the joint at the beginning of the design process, but this is only for a specific type of joint. This formula cannot be used for all types of joints. Another way to estimate the stiffness of a connection at the beginning of the design process is by using the table from Steenhuis et. al. [4], see figure 2.2.

Both of these methods only give an estimation of the stiffness at the beginning of the design process if a pre-design of the connection is known. In the optimization of the bolted connection these formulas only help to reduce the iteration process. In the end of the design process these connections still need to be checked with more accurate methods.

2.2.3. STEEL FRAMES INCLUDING JOINT OPTIMIZATION

To consider the joint at the beginning of the optimization process, the failure modes that need to be considered are depending on the type of joint that is selected according to Eurocode 3 part 1-8 [41]. This code states that besides the structural integrity and weldability, mechanical rules need to be taken into account to validate the checks of the joint.

Optimizing steel frames including the joints makes it possible to design semi-rigid connections. This can lead to smaller beam sizes [4] [51] [52] [53]. It is however difficult to optimize for semi-rigid connections due to all the variables like the type of bolted connection, elongation of bolts and the plate dimensions [52] [53].

In the above mentioned literature there can be found a lot about optimizing steel frames for strength. This optimization is however not about the case where the detailed design phase is moved to the concept design phase. But the optimization process goes through the regular phases. This means that assumptions are made in the beginning, that are checked in the end. A cost optimization for a steel structure including the joints, is only found in literature with welded connections [54].

2.2.4. USED OPTIMIZATION

In this research a steel structure with open profiles will be optimized including the bolted connections. The optimization performed will be a cost optimization. This will include the costs of the joints. In this research a method will be presented where the detailed design phase is moved to the concept design, to consider the connection design from the beginning of the design. The optimization method used in this thesis will be based on the optimization to cost of only steel structures where at the beginning of the design a choice is made for the type of connection. However in this research this choice will also be including semi-rigid connections. The method to optimize the connection is based on a combination of the optimization of only joints and the optimization of structures. More about the optimization process used in this thesis can be found in chapter 3.

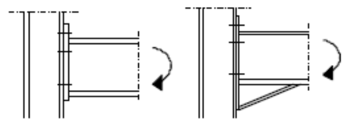
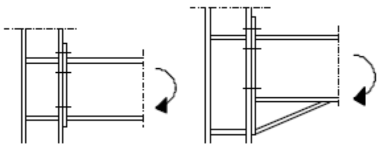
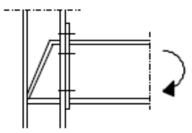
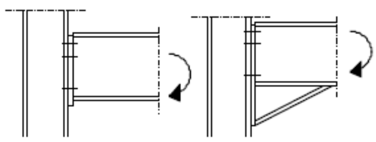
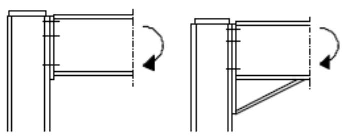
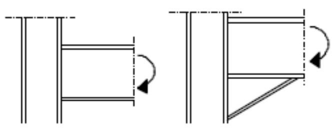
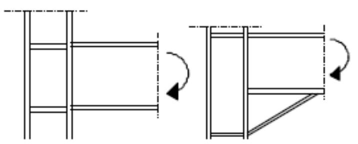
Configuration		S_j
Extended end-plate and unstiffened		$\frac{E z^2 t_{f,c}}{13}$
Extended end-plate, stiffened in tension and compression		$\frac{E z^2 t_{f,c}}{8{,}5}$
Extended end-plate and Morris stiffener		$\frac{E z^2 t_{f,c}}{3}$
Flush end-plate		$\frac{E z^2 t_{f,c}}{14}$
Flush end-plate and cover plate		$\frac{E z^2 t_{f,c}}{11{,}5}$
Welded joint and unstiffened		$\frac{E z^2 t_{f,c}}{11{,}5}$
Welded joint stiffened in tension and compression		$\frac{E z^2 t_{f,c}}{5{,}5}$

Figure 2.2: Formula to asses the stiffness of a joint taken from Steenhuis et. al. [4]

2.3. COSTS

The biggest part of the costs of steel structures are made in the last phases of the project, this can be seen in figure 1.3. To reduce these costs a lot of engineering firms try to optimize to costs by optimizing the structure to weight, because this is a quick approach to estimate the total costs [1]. If the total costs would only be calculated with a price per kg this leads to the lowest costs. However this weight optimization does not always give the actual lowest costs, the costs of a steel structure can be divided into multiple parts as can be seen in figure 1.2.

Figure 1.2 shows that most of the costs are in fabrication and construction for a multi-storey commercial building in Europe. In the case of single storey buildings the fabrication and construction costs will be relatively lower and the raw material costs will be higher. The design of connections plays a big part in the cost phases from figure 1.2, because the more complex the connection the more time is needed in the fabrication phase and the more time it can take in the construction phase. This can lead to an influence on 40% of the costs [63].

At a lot of engineering firms, it is seen that the optimization of costs is done by a weight optimization. Most engineering firms choose for this option because this can be done in the concept design phase by making simple assumptions about the connections. The optimization to weight is then completely focused on the weight of the main profiles (beams, columns, etc.) [1], because these profiles influence the weight of the structure the most. In a weight optimization an assumption needs to be made on the stiffness of the connection. Most often either a hinged or rigid connection is selected. This makes it possible to optimize without a design of the connection. However it does not always results in a realistic view on the costs of a structure. Another problem of this assumption is that often connections that are considered hinged, behave semi-rigid in reality and that connections that are considered rigid are sometimes not rigid enough [64].

2.3.1. DIRECT AND INDIRECT COST

When the actual construction costs without a time component are considered there are two types of costs, direct and indirect costs [65]. Direct costs can be defined as costs which can be accurately traced to a cost object. Indirect costs cannot be accurately traced to a costs object [66]. Multiple examples on direct and indirect costs can be found in table 2.1.

Direct	Indirect
Laborers' wages	Power consumption
Materials	Factory insurance
Paint and coatings	Supervisors' salaries
Transport	Factory depreciation
Installation	Factory manager's salary
	Machine depreciation
	Machine maintenance

Table 2.1: Direct and indirect costs examples

In this research there is looked at the direct costs of building the structure due to the traceability of the costs. The indirect costs are often project specific and the goal of this research is that the method used in this thesis can be used for multiple projects.

2.3.2. COSTS OF STEEL STRUCTURES

The total costs of steel structures are determined by both direct and indirect costs from multiple components. These components are: engineering, material, fabrication, fire protection, transport of the structure and the construction of the structure [1] [65]. The focus in this research is on how changes in the topology change the direct costs of a structure. The components that are affected by the change of topology are: the material needed, the fabrication, the fire protection layer, the transport and the construction of the structure.

MATERIAL

The direct cost of the material is determined by the amount of material needed.

FABRICATION
The direct cost of fabrication is determined by the design. For example, the number of elements and the number of connections that need to be made have a big influence on the direct costs of fabrication.

FIRE PROTECTION
The fire protection is not considered in this research, because the needed fire protection is very specific to the usage of the structure.

TRANSPORT
The direct cost of transport is the cost to transport the steel components to the building site. The transportation method limits the size of the elements that can be transported. The maximum dimensions of the trucks can be found in figure 2.3. Bigger dimensions are possible but need additional permits.

The transportation costs depend highly on the distance that need to be travelled. This is very project specific, this is why the transportation costs are not considered in this research.

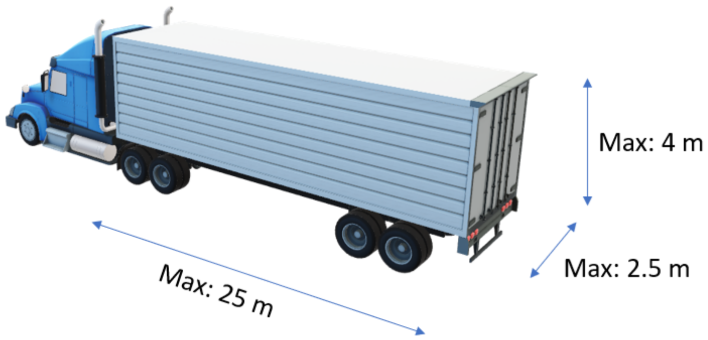


Figure 2.3: Maximum dimensions road transport in the Netherlands [5]

CONSTRUCTION
The direct cost of site assembly is largely depended on the total building time. Longer building time leads to higher costs, because of overhead cost, equipment rental, possible permits etc. [65]. During assembly, cranes are needed to put the parts in location. The size of the part that the crane needs to lift has no influence on the time it takes to lift this part [67]. Meaning that when doing a cost-optimization it can be more cost efficient to lift large parts so less parts need to be lifted, the size of the parts are however limited to the maximum transportation dimension.

2.3.3. COST INFLUENCES
According to multiple studies the material costs are a part of the total costs see figure 1.4 [1] [54] [68]. The material costs represent around 30-40% of the costs for multi-storey buildings in Europe. The other 60-70% is determined by multiple factors. These factors are: detailing, fabrication, coating, transportation and construction [1]. Some general principles that are developed to reduce the costs of the 60% are: welding in the fabrication shop, bolting on building site and adopt simply supported connections not fixed connections [68]. The problem with the last development is that the cost of material of the structure can go up while reducing these other costs, which can lead to a bigger total cost.

Besides all these components, costs are dominantly based on supply and demand [69]. High supply and low demand lead to low prices and low supply and high demand lead to high prices. Whether the demand and supply are high or low is time dependent. Meaning that the results of this research could be different within a couple of years.

The players on the market that have the most influence on the supply side are the steel producer, steel traders and steel contractors. The steel producers process the raw material into steel and process the steel into steel sections, the steel traders own stocks of steel for selling and the steel contractors process the beams to make them usable in a steel structure. All these players influence the prices based on competition, supply and demand.

Besides the demand and supply factor there are also other factors that play a role in the costs of a steel structure, these are: the number of repetitions, the bulk procurement of material the price of labour and the execution class [1].

The number of repetitions influences the cost of the production process. Repetition will result in a learning curve for the workers [65]. More repetition will create more efficiency in the process. Also, repetition will lower the risks on errors. A higher efficiency will lower the production time and thus costs [70].

The bulk procurement influences the costs by the volume procured. Buying large volumes of items lowers the price per item and buying small volumes of items give the highest price per item. However this changes per steel contractor and is dependent on the customer-supplier relationship.

The price of labour is determined by the number of man hours required, times the hourly rate that a worker gets paid. The number of man hours required is determined by the erection time and the time in the factory. The hourly rate is depending on the country where the labour takes place and the time it takes place. Below is the difference in hourly rate shown for a welder with 5 years of experience.

The average welder will earn a monthly gross salary of € 3500,- in the Netherlands [71]. In Poland the gross monthly salary is € 700,- [71]. The lowest salaries are in developing countries, for example in Angola the gross monthly salary is € 310,- for a welder with 5 years of experience [71].

Not only the salaries have significant differences around the world, also the price of steel differs a lot. For example the price of steel in China is 78% of that in the Netherlands [72]. Compared to the differences in salaries the steel price around the world can be seen as good as stable. This means that in countries where the salaries are high it can be cheaper to use more steel if it reduces the amount of labour, while in countries with low salaries it is more cost efficient to use as less steel as possible even if it takes a lot of labour.

The last factor that plays a role in the costs of a steel structure is the execution class. The execution class tells something about the quality of the steel structure, for example the quality level of welds. The execution classes are divided into 4 classes from EXC1 to EXC4 where EXC4 has the strictest requirements [73]. The execution class is determined by the following 4 steps [74]:

1. Define the consequence class
2. Select a service category
3. Select a production category
4. Select the execution class based on steps 1 to 3

The consequence class of the structure used in this research is CC1 according to table NB.5 from NEN-EN 1991-1-7+C1 [75], because only relative small structures are researched.

The service categories are divided into SC1 and SC2 where SC1 is for structures/components designed for quasi actions only and SC2 is for structures/components designed for fatigue actions to EC3 such as bridges or located in regions with medium/high seismic activity [73]. This means that for the structure used in this research SC1 is selected.

The production category is divided into PC1 and PC2, where PC1 is for components with a steel grade below S355 and PC2 is for components with a steel grade of S355 and above. Meaning that for the case study from chapter 5 PC2 is selected.

These different factors lead to the execution class 2, this can be found in table 2.2.

Consequence class		CC1		CC2		CC3	
Service category		SC1	SC2	SC1	SC2	SC1	SC2
Production category	PC1	EXC1	EXC2	EXC2	EXC3	EXC3	EXC3
	PC2	EXC2	EXC2	EXC2	EXC3	EXC3	EXC4

Table 2.2: Execution class based on Consequence class, service category and production category

2.3.4. Cost models

A cost estimation of a structure can be made with multiple different cost models. In appendix A the different cost estimation methods from literature could be found. In this appendix 5 different methods can be found. The cost model that will be used in this research is partly based on one of these models and can be found in section 3.3.1. The cost methods discussed in appendix A are:

- Weight concept
- Weight concept including rotational capacity of joint

- Weight concept including welds
- Activity based costs
- Reverse engineering

2.3.5. FEATURE BASED COSTS

Part of the cost that can be taken into account according to Haapio et al. [51] is the non productive time in the processes. The non-productive time is the time it takes for the preparation works which are required to execute the process flawless. This will not be considered in the cost formula used in this research, because this is very project specific.

2.4. DESIGN OF STEEL HALLS

In this section the standard design steps of a steel hall are described. At the end of this section, the aspects that have the greatest influence on the strength and the stiffness of the hall are described.

2.4.1. DESIGN STEPS

The design of a steel hall begins with the location of the steel hall this location will determine the loads on the structure that need to be considered [76]. Depending on the country of the hall the loads can be taken according to the Eurocode NEN-EN 1991-1-1 [77].

After the location of the hall the main dimensions of the hall need to be determined, these dimensions are depending on the function of the hall [76].

After these first two steps the steel hall can be designed. This will be designed for the loads that can be found in the Eurocode [77]. These loads will cause deformations and stresses in the structure. Depending on the allowed deformations and stresses a structural design of the hall can be created.

This design starts with the type of column base connection. Most often one of the following two options is selected:

- Hinged connection
- Rigid connection

A rigid column-base connection is more expensive than a hinged connection. Therefore, the hinged column-base connections are the main focus of this thesis.

After the type of column base connection a decision has to be made on the type of roof that will be used. This can differ between a flat roof, angled roof, gable roof or curved roof. This depends mostly on the rules of the municipality and the type of building. For industrial buildings most often a flat roof is selected and for agricultural buildings most often a hall with a gable roof is selected [78].

After the type of column-base connection, the design can vary in topology. This topology partly depends on the type of roof that is selected, the fire safety wanted in the structure, the function of the structure and the dimensions of the structure [76].

After the topology the beam column connection type needs to be selected.

With all the aspects that are selected above a design of the structure can be made which can determine the profile dimensions. These depend on the strength needed and deformations allowed in the structure. When this is known an estimation of the costs of the connection can be made this is often based on the weight [76].

After all the steps mentioned above the design of the connections can start. This is mostly depending on the beam profiles and the assumed stiffness of the connection.

2.4.2. DESIGN ASPECTS

The aspects that have most influence on the strength and the stiffness of a structure after the location and main dimensions are determined are:

- The type of column base connection (hinged / rigid)
- The type of beam column connections (hinged / semi-rigid / rigid)
- The topology of the structure

- The profile sizes

Because these aspects have the most influence on the strength and the stiffness of the hall, they will be variables in the design of the steel hall. More about this can be found in chapter 3.

3
MODEL DEFINITION

In this chapter the optimization process is described including the software used for this process. The chapter starts with a description about the most important steps in the optimization tool. Secondly, the parameters used in the tool and the results wanted from the tool are discussed. Thirdly, a description of how the tool works is given in more depth, including a description about the differences with existing tools. At the end of this chapter the expected results and further possibilities of the method and tool used in this research are discussed.

The structure used in this research is a single storey steel hall with a flat roof, see figure 3.1 for a sketch.

3.1. OPTIMIZATION TOOL

The most important steps in the optimization tool (OT) can be found below and in figure 1.5.

- The input variables
- Creation of parametric model
- Structural loop
- Connection loop
- Storage of results

The design steps considered in the OT are as described in section 2.4. In section 2.4 the aspects can be found that have the most influence on the strength and the stiffness of a structure. This is why these aspects are all considered as inputs in the OT. With these inputs the parametric model is created. More information about these inputs can be found in section 3.2. This model is put in the software RFEM, in this software the forces and deformations are calculated. These are then checked with the rules from the Eurocode. If the structure is strong and stiff enough the connections are checked with IDEA StatiCa. The connection is checked on strength, deformations and stiffness. In case all of these results are sufficient, the results are stored in an excel file. More details about all the steps can be found in section 3.3.

3.2. MODEL INPUTS

The input needed in this model can be divided into two types: fixed input and variable input. The fixed inputs are the profile types and the loads. These are considered fixed in this research because the profile types are fixed to a certain list of profile types that is set beforehand and will not change during the optimization process. This list can be changed for different structures. The loads are considered fixed because they are taken from the Eurocode, meaning that if the location is known and the function of the structure is known these loads are fixed and will not change during the optimization process. The variable inputs in the optimization process are the topology and the joint type. These are considered as variables because these inputs change to get the optimized structure.

3.2.1. TOPOLOGY

The topology of the structure influences the geometrical properties of the structure. The variables in the topology are:

- The height of one storey
- The width of the structure
- The length of the structure
- The number of storeys of the structure
- The number of columns in a frame
- The number of frames
- The number of purlins
- The number of spans the braces include in x direction
- The number of spans the braces include in y direction
- The location of the wind braces in x direction
- The location of the wind braces in y direction
- The shape of the braces in x direction
- The shape of the braces in y direction
- The column-base connection

See figure 3.1 for x, y and z directions in the model.

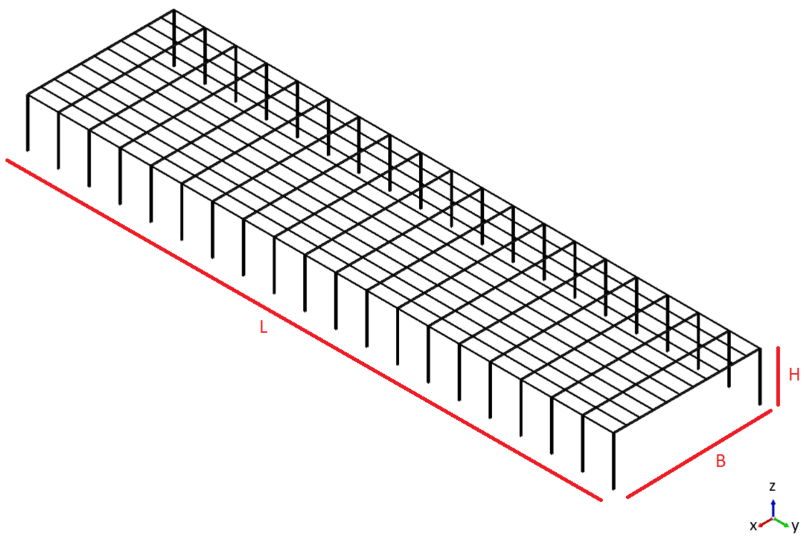


Figure 3.1: Coordinate system used in model

The dimensions of the structure: the length, width and height can be set to any value wanted. Meaning that different case studies can be performed for different structures. The dimensions are fixed during the optimization process. The height is the height of one storey meaning that if a structure has multiple storeys this height is the centre to centre (c.t.c) distance between two floors.

The number of floors of the structure can be set to any value. This makes it possible to check multiple types of structures with this process. In the optimization process of a structure this value is set beforehand and is fixed during the optimization.

The span between the columns is always equally divided, meaning that the total width and the number of columns determine the c.t.c. distance between two columns, see equation (3.1). Per case that is studied an upper limit, lower limit and the step size can be set to decrease the number of topologies considered in the study. It is assumed that all the frames are the same.

$$c.t.c_{column} = \frac{width}{n_{columns} - 1} \tag{3.1}$$

The spans between the frames are always equally divided, meaning that the number of frames and the length of the structure determine the c.t.c. distance between two frames, see equation (3.2). Per case that is studied an upper limit, lower limit and the step size can be set to decrease the number of topologies considered in the study.

$$c.t.c_{frames} = \frac{length}{n_{frames} - 1} \tag{3.2}$$

The number of purlins can be varied with a minimum of two, the most outer purlins. The spans between the purlins are always equally divided. This means that the number of purlins together with the total width of the structure sets the c.t.c. distance between two purlins, see equation (3.3). Per case an upper limit, lower limit and the step size can be determined to decrease the number of topologies considered in the study.

$$c.t.c_{purlins} = \frac{width}{n_{purlins} - 1} \tag{3.3}$$

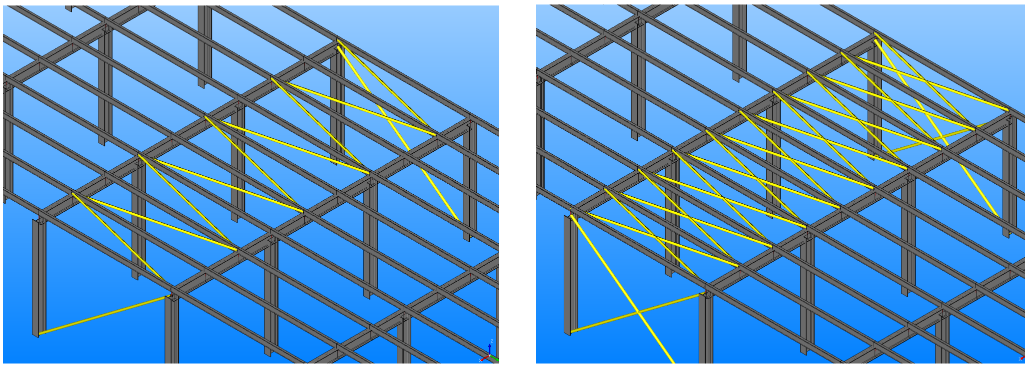
The number of spans the braces cover in x or y direction can be set to any number that can divide the total number of spans without having a remainder, see equation (3.4). It is assumed that all the braces in x or y direction are the same size.

$$Remainder\left(\frac{n_{spans} - 1}{n_{spans-covered}}\right) = 0 \tag{3.4}$$

The position of the braces in x or y direction can be set to any span number. If a span is selected that does not exist the brace will also not exist.

The brace shape can be determined for the braces in x and y direction separate. The shape can be varied from diagonal braces to x-shape braces. This is set for all the braces in the selected direction. See figure 3.2 for the different brace shapes.

At last the column-base connection type can be selected between rigid or hinged. This variable is set beforehand and is fixed during the optimization process.



(a) Diagonal braces

(b) X braces

Figure 3.2: Different brace shapes

3.2.2. PROFILE TYPES

The profile types that are available can be varied from any type of profiles mentioned in the list below. For each specific case a list can be created with the profiles and the dimensions that need to be considered.

- AB
- HEA
- HEB
- IPE
- UPE
- CHS
- RHS
- SHS

3.2.3. LOADS

In this section all the different load types from the model are mentioned. The loads are a fixed input in this optimization process. The formulas and more details on the loads are according to the Eurocode. An application of the loads in the RFEM model can be found in appendix B.

The loads considered in this research are permanent and variable loads according to the Eurocode for single storey buildings.

The permanent loads in the optimization process are:

- Permanent load due to roof and insulation
- Self weight

The variable loads in the optimization process are:

- Wind load
- Snow load
- Maintenance load

The ultimate limit state load combinations are used to check the strength of the structure and the serviceability limit state combinations are used to check the deformations in the structure.

3.2.4. JOINT TYPE

The beam-column connection is the variable connection used in this research, the other connections are fixed designs. The joint type of this connection variable is divided into a fully rigid joint, semi-rigid joints and a hinged joint. The semi-rigid connections are divided in connections with different stiffness factors. The three factors are varying from a closer to hinged to a closer to fully rigid connection. Fixed values are not an option because a fixed value can be rigid for one structure or semi-rigid for a different structure. Therefore, these values are calculated with a formula, which depends on the beam dimensions. The formula used is:

$$k\frac{EI_b}{L_b} \tag{3.5}$$

Where k differs between 6, 12 and 18 for the semi-rigid options. These k-factors are considered because a k-factor of 25 gives the lower limit for fully rigid connections and a k-factor of 0.5 gives the upper limit for hinged connections.

3.2.5. RESULTS NEEDED

The results needed to make a cost comparison are:

- Structural design
- Connection design
- Structural costs
- Connection costs
- Total costs

With these results a cost comparison can be made between all the different topologies. With this the difference in design can be found and the difference in costs. Because the wanted result of this tool is a comparison. The costs are normalized, this means that in the end there will not be an actual price given but a normalized price factor. The optimization tool is built in such a way that the above mentioned results can be produced.

3.3. MODEL PROCESS

In this section the process used in the model is explained in more detail. It starts with the cost model used in this research, which explains the variable for which is optimized. Secondly, the optimization process is explained. At the end of this section, the expected results are discussed.

3.3.1. COST MODEL

The cost model developed will be used to optimize production costs of steel structures. This model is developed by the following steps:

- Researching multiple existing cost models
- Comparing these existing models
- Researching steel prices

The results of these steps can be found in appendices A, C and D.

The aim of this thesis is to get the ratio between the costs of the different structures, not to get the exact costs. Therefore the average price from multiple companies is used for the variables in the cost formula.

The used cost equation is based on an activity based cost estimation, see appendix A. The equation used in this research includes material costs, sawing costs, blasting costs, and welding costs. This is because the other activity based costs are project specific, as mentioned in appendix A. The structure and connections are treated separate in the equation, see equations (3.6) and (3.8). These equations are for steel materials with standard steel grades.

$$Cost_{total} = \sum_{i=1}^{i=n} \left(L(i) \cdot \left(C_{mat,i} + C_{blast,i} \right) + C_{saw,i} \right) + \sum^{k} \left(m_k \cdot C_{connection,k} \right) \tag{3.6}$$

Where:

n	=	Number of elements in structure
m_k	=	Number of connections of type k
k	=	Type of connection, for example beam-column or purlin-beam
$L(i)$	=	Length of element i
$C_{mat,i}$	=	Cost of material per running meter of element i
$C_{saw,i}$	=	Sawing cost per beam
$C_{blast,i}$	=	Blasting cost per running meter of element i
$C_{connection,k}$	=	Cost per connection of type k

STRUCTURE COSTS

The costs of the structural part consists of the material costs, sawing costs and blasting costs. To determine the material costs, sawing costs and blasting costs there is looked at five different steel traders. The steel traders used in this research can be found in appendix C. From these steel traders the latest price lists available are used to determine the material costs, sawing costs and blasting costs [6] [7] [8] [9] [10] [11] [12] [13]. From these different costs an average cost per profile size is determined, see figures C.1 to C.12. The total price for a cut beam of one meter for each profile size can be found in tables C.1 till C.4.

CONNECTION COSTS

The costs of the connection are divided into the following parts:

- The number of bolts
- The bolt size
- The type of bolt
- The plate dimensions
- The weld size

The costs of the bolts are determined by the material costs and a factor for the labour costs. The material costs of bolts are divided into the costs of the bolt, the nut and the washers. The costs of these materials are estimated by researching multiple prices from different companies and averaging these prices. This can be found in the figures D.4 to D.29 and tables D.2 to D.4. The companies used for this cost estimation are can be found in appendix D. Not all of the companies had the same bolt sizes in stock, which causes a reduced number of companies considered for some of the bolts in figures D.4 to D.29. It is assumed in this research that a bolt assembly consists of one bolt, two washers and one nut.

The labour costs of the bolts depends on the time it takes to fasten the bolts. This time depends on whether or not the bolts are pre-tensioned or not. According to Crowners services, if it is assumed that the construction is prepared and ready it takes 3 minutes to install non pre-tensioned bolts. For hydraulic pre-tensioned bolts it could take around 10 minutes to install the bolt [79]. This time could then be multiplied by the hourly rate of a builder. It is assumed in this research that a Dutch builder with 5 years of experience assembles the connections. This gives a gross hourly rate of € 22.30 per hour [71]. In this research only non pre-tensioned bolts are considered in the connection design.

The costs of the plates are determined by the material costs, sawing costs and blasting costs [6] [14] [15] [7] [9]. These costs are determined by looking at multiple companies, the companies used for this cost estimation can be found in appendix D. From these costs an average price is determined for a plate of one m^2 for all the different plate thicknesses. This can be found in figures D.1, D.2 and D.3 and table D.1. The price for sawing the plates where only available at one company, meaning that there is no average price.

The last part of the connection costs is the welds. The costs for the welds is determined with a formula taken from R. Ajouz [54]. This formula includes the material costs, the labour costs and the equipment costs for the welds. These costs are all depending on the throat size a_w in mm. This equation gives the time it takes in minutes per meter weld of a given throat size. This leads to equation (3.7) [54].

$$T = 2.62 \cdot a_w^2 + 1.37 \cdot a_w + 0.09 \tag{3.7}$$

This time can be converted to costs by multiplying it with the hourly rate of a welder. In this research it is assumed that the welds are created by a welder with 5 years of experience in the Netherlands. The gross hourly rate of this welder will be around € 21.80 per hour [71].

This all leads to a cost formula of the connection that can be found in equation (3.8).

$$C_{connection} = n \cdot (C_{bolt} + 2 \cdot C_{washer} + C_{nut} + C_{builder} \cdot T) + \sum_{i=1}^{i=m} \left(A_{i,plate} \cdot \left(C_{i,mat} + C_{i,blast}\right) + C_{i,saw}\right) + C_{welder} \cdot T_{weld} \cdot L_{weld} \tag{3.8}$$

Where:

n	=	Number of bolts in the connection
C_{bolt}	=	Material costs of one bolt
C_{washer}	=	Material costs of one washer
C_{nut}	=	Material costs of one nut
$C_{builder}$	=	Hourly rate of a builder
T	=	Time it takes to install one bolt in hours
m	=	Number of plates
A_{plate}	=	Area of the plate in m^2
$C_{i,mat}$	=	Material costs of plate i per m^2
$C_{i,blast}$	=	Blasting costs of plate i per m^2
$C_{i,saw}$	=	Sawing costs of plate i
C_{welder}	=	Hourly rate of a welder
T_{weld}	=	Time it takes to weld in hours per meter from equation (3.7)
L_{weld}	=	Length of the welds in the connection in meters

The costs of the connection also determine the optimization of the connection. The connection is optimized to the most cost-efficient connection, with the use of the cost formula mentioned above.

3.3.2. OVERALL OPTIMIZATION PROCESS

The most general steps in the used optimization process are:

1. Generate geometry
2. Evaluate forces and deformations in structure
3. Generate detailing
4. Evaluate forces and deformations in connection
5. Calculate price
6. Store results
7. Repeat for different topology

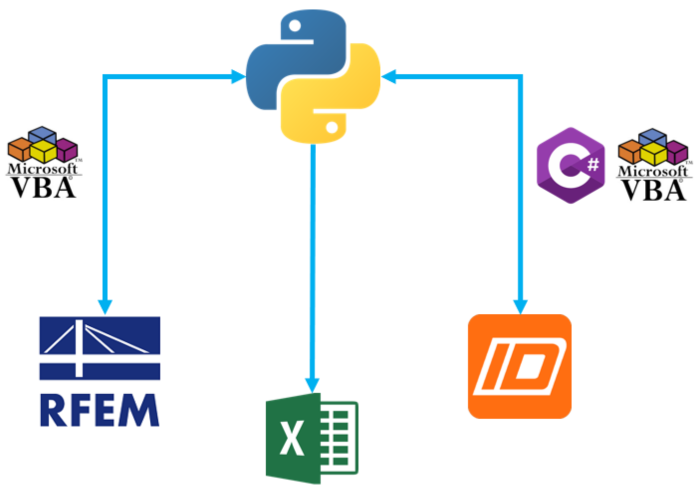


Figure 3.3: Software used including links between the software packages

The programs used to create this optimization model will be Python, C#, VBA, Excel, RFEM and IDEA StatiCa. Python will be the main software. With this the parametric model is built including the connections and this will include all the data.

RFEM will be used to calculate the forces and deformations in the structure. The link between Python and RFEM will be through the application programming interface (API) of RFEM which is created with Visual Basic (VBA) and Excel. IDEA StatiCa will be used to evaluate the forces and deformations in the connection. The link between Python and IDEA StatiCa can be created through the API of IDEA StatiCa this is created with C#. The result data will be stored in Excel. An overview of the software used can be found in figure 3.3.

Python will be used to create the main parametric model and link all different software packages with each other. It is also used to add extra tools like the cost model, the structural Eurocode check and the component method check.

All the steps in the complete optimization process including the programs needed per step and the input and results can be found in figure 3.9. A more detailed explanation of this figure can be found in the sections below including a reference to all the numbered items. A single loop of the optimization tool with all intermediate results can be found in appendix E.

GENERATE GEOMETRY

The first step of the optimization process is to generate the geometry. The geometry is generated with the inputs from section 3.2. The variable inputs are the topology (1) and the joint type (2). The fixed inputs are the profile types (3) and the loads (4).

The topology (1) should be set by multiple lists. These lists are:

- A list including the range for the number of columns considered
- A list including the range for the number of frames considered
- A list including the range for the number of purlins considered
- A list including the brace spans considered for both x and y direction

More information about these lists can be found in section 3.2.1. The other values mentioned in section 3.2.1 should be set to fixed values that belong to the structure.

The joint type (2) is set by a list of joint stiffness factors that should be considered in the case study. This can change for different case studies. More information about this list can be found in section 3.2.4.

The profile types (3) are considered a fixed input because these are lists that do not change during the optimization process. From these lists the profiles of the columns, beams, purlins and braces are selected. More information about the profile types can be found in section 3.2.2.

The last input variables are the loads on the structure (4). These loads are considered a fixed input because they are determined according to the Eurocode and differ only per project. More information about the loads can be found in section 3.2.3.

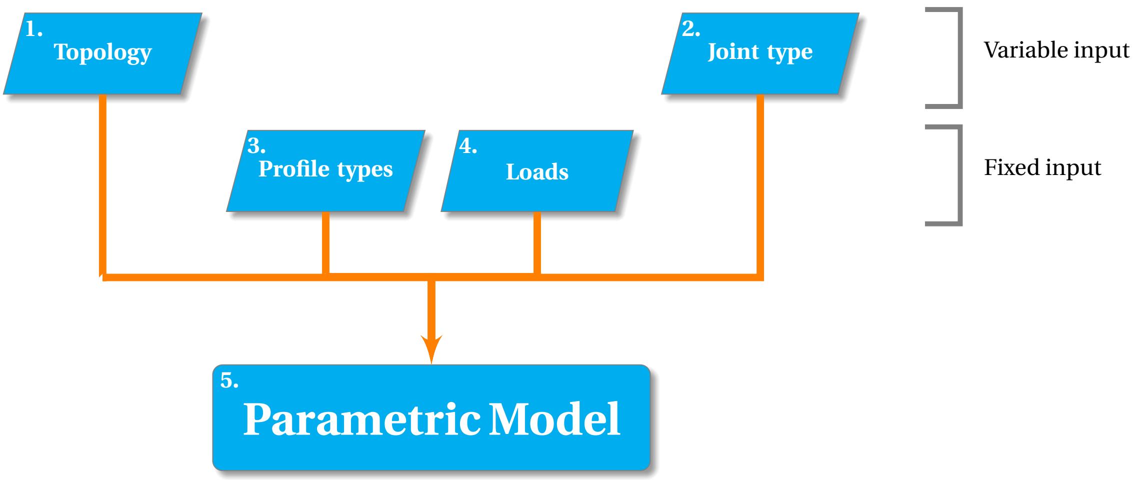


Figure 3.4: Geometry input taken from figure 3.9

All of the inputs mentioned above generate the geometry which creates the parametric model (5). These inputs and model are all created in Python. From this model (5) the optimization process can be started.

EVALUATE FORCES IN STRUCTURE

The forces in the structure are evaluated with both RFEM and the Eurocode checker. This Eurocode checker is created for this research as an add on to the parametric model. The evaluation process of the forces in the structure can be found in figure 3.5. This evaluation is done in multiple steps.

1. Load model into RFEM
2. Run RFEM
3. Read results from RFEM
4. Run results through Eurocode checker
5. Read results from Eurocode checker

The first step in this evaluation is to load the model into RFEM. This is done with the API of RFEM.

The second step in the evaluation of forces in the structure is to run RFEM (6). This is done through the API of RFEM. All the RFEM models are saved in a separate folder, to make it possible to visually check the models in a later stage.

The third step of this evaluation is to read the results from RFEM (7). The results from RFEM are the forces in the members and connections and the deformation of the structure (7). All these results are saved in an excel file. This excel file is used to read the results back into the parametric model.

The fourth step in the evaluation of the forces in the structure is to run the results through the Eurocode checker. This is modelled in the optimization flow chart by the question *"Is the structure sufficient?"* (8). This question is answered by taking the internal forces in the members and the deformations and run it through the Eurocode checker. The formulas that are used in this Eurocode checker are all according to EN 1993-1-1 [80].

The last step in the evaluation of the structure is to read the results from the Eurocode checker. This is done in python and these results are compared to an allowable design ratio. In this research this allowable design ratio is set to 1.0. This can be changed if wanted. By comparing the design ratio results from the Eurocode checker with the allowable design ratio the first loop can be created.

This loop starts with the cheapest option of the structural elements without considering the connections. This is done by creating a list with all the different combinations of beams, columns and purlins that are set in the input (3). Then, for each combination the price is calculated and coupled to this combination. The list that includes the prices is sorted from the lowest to the highest price, which gives the cheapest combination of beam, column and purlin profiles. This combination goes through all the steps mentioned above to evaluate the forces in the structure. When all the design ratios from the Eurocode checker are known they are compared to the allowable design ratio. In the case that the structure gives higher results than allowed the next profile combination is taken from the profile list (9). This goes on until the moment a profile combination is found that gives a design ratio that is lower than the allowable design ratio. This is done to make sure that the cheapest combination is used. In case the design ratio of all the different elements is above 2, the 2 cheapest profiles are increased one profile size to increase the speed of the optimization process.

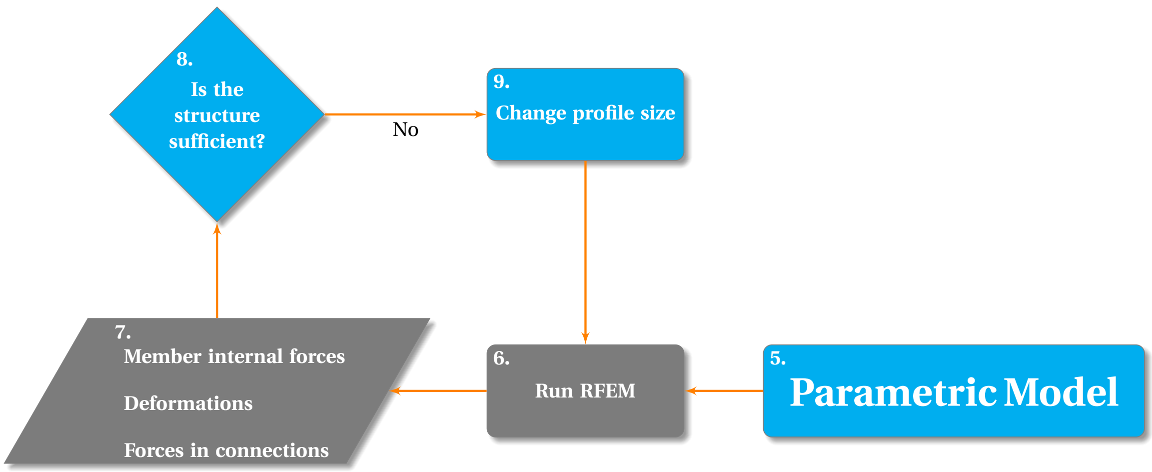


Figure 3.5: Evaluation of forces in the structure taken from figure 3.9

GENERATE DETAILING

After the evaluation of the structure the connection is automatically designed. This is done with the joint stiffness from the input. This stiffness is compared with the stiffness from a database that is created for this study. Where the stiffness and bending moment resistance is calculated for different end-plate connections. More on the design can be found in chapter 4. This database gives a list of connection designs that have a stiffness within ± 10% of the wanted stiffness for the selected joint type. For the list of designs that comes from the database, the costs are calculated for each connection with equation (3.8). Then the list is sorted from lowest to highest costs. The dimensions of the connections are selected from this list (11). This connection design is used to evaluate the forces in the connection, see figure 3.6.

EVALUATE THE FORCES IN CONNECTION

When a pre-design of the connection exists the forces in the connection can be evaluated. This is done in multiple steps:

1. Get the governing loads working on the connection
2. Put loads into IDEA StatiCa
3. Put pre-design into IDEA StatiCa
4. Run IDEA StatiCa
5. Read results from IDEA StatiCa
6. Additional check with component method

These steps can be found in figure 3.6

The first step in the evaluation of the connection is to get the governing loads working on the connection (10). This is done by reading the results from the RFEM output of the optimized structure. Because it is assumed in this research that all beam column connections are the same. The governing forces are taken from all the beam column connections. These forces are then used as input for the connection evaluation.

The second step for the connection evaluation is to get the loads into IDEA StatiCa, this is done by using the API of IDEA StatiCa.

The third step is to get the pre-design of the connection into IDEA StatiCa, This is done by using the API of IDEA StatiCa.

After the connection design and loads are in, IDEA StatiCa runs the connection model (12). The results of IDEA StatiCa are saved in an XML file.

The fifth step of the connection evaluation is to read the results from the XML file. The results are compared to the allowed design ratio and the maximum strain of 5%.This step is modelled in the flow chart by the question *"Is the connection sufficient?"* (13). This comparison creates the second loop in this process.

In this second loop the connection is checked according to the steps above. If the connection is not sufficient, meaning either that it is not strong enough or it deforms to much. Then the connection needs to be re-designed (14), this means it takes the next connection design from the sorted connection list. This happens over and over again until the connection is sufficient.

When this first connection loop is ended, the last step of the evaluation of the connection is taken. This is an additional check of the results with the component method (15) that is created into the python model. This second check is done to make sure that the connection design could resist the bending moment acting on the connection and is sufficiently stiff (16) considered the wanted stiffness (2). In the case that the connection is not sufficiently stiff or strong according to the component method, the connection will be changed and the second loop is started again.

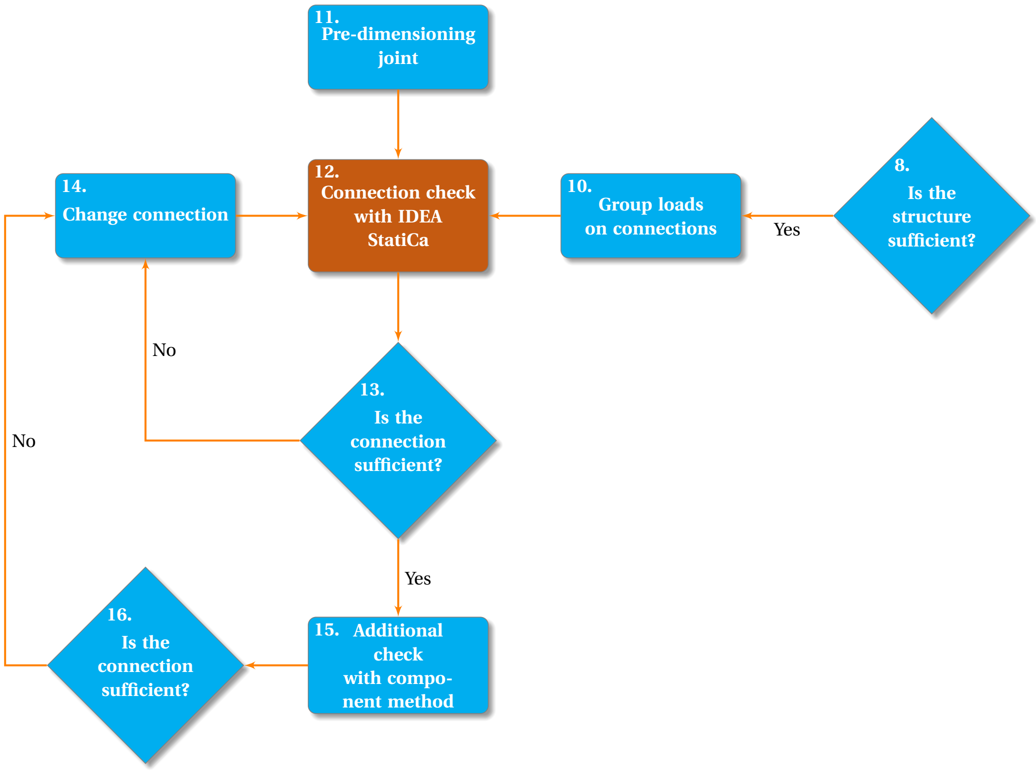


Figure 3.6: Evaluation of forces in the connection taken from figure 3.9

CALCULATE PRICE

Now that the complete structure including the connections is checked, the total cost of the structure can be calculated according to equation (3.6). All the variables taken from the parametric model are put into the cost equation which gives a total price as a result, see figure 3.7. This price is then transferred to a normalized cost factor to compare all the different results. The normalized cost factor is calculated by dividing the costs of the structure by the most cost-efficient option.

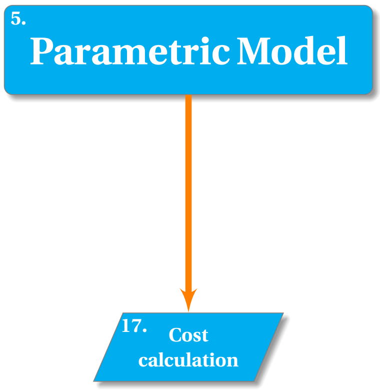


Figure 3.7: Costs of the structure taken from figure 3.9

STORE RESULTS

After the cost factors are calculated the results can be stored. These results will be stored in an excel file [(18)], see figure 3.8. The following results are stored:

- Costs
- Structural design
- Connection design

- Unity checks
- Stiffness of the beam-column connection
- Bending moment resistance of the beam-column connection

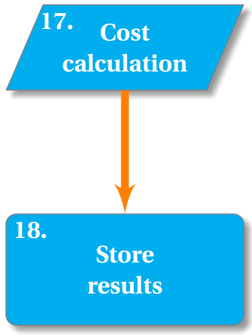


Figure 3.8: Results of the structure taken from figure 3.9

REPEAT FOR DIFFERENT TOPOLOGY

After the results are stored (18), the overall loop can be created. This loop can change the topology or the beam-column connection stiffness, depending on whether or not the lowest connection stiffness is selected (19) (20). This loop then calculates the results for multiple topologies see figure 3.9.

COMPLETE OPTIMIZATION PROCESS

Below, a recap is given of the complete optimization process mentioned above. This complete process can also be found in figure 3.9

The flow of this optimization process starts with the input values. The topology (1) and the joint type (2) are variable values and the profile types (3) and loads (4) are fixed values. The topology (1) consists of the number of frames, the number of columns per frame and the number of purlins. The joint type (2) includes the stiffness of the beam column connection. The profile types (3) are fixed to given lists of profiles. The loads (4) are fixed to the loads from the Eurocode.

These input values will be the start of the parametric model (5), and thus the start of the optimization process. In this optimization process the parametric model (5) is sent to RFEM and this model is checked (6). The results coming from RFEM are: internal forces in the members, deformations and forces in the joints (7). With these results it can be checked if the model is sufficient enough or not (8). This check is done according to the Eurocode EN 1993-1-1 [80]. If the model is not sufficient enough the profile sizes are changed (9). With these new profile sizes the loop starts again. this goes on until valid profiles are used.

When the model is sufficient enough (8), the loads on all the connections are grouped and the governing forces are selected (10). After the grouping of the loads there will be a pre-dimensioning of the connection (11) and this connection will be checked with IDEA StatiCa (12). The results from this check will tell if the connection is strong enough (13). In the case this connection is not strong enough, the connection is re-designed (14) and checked again in IDEA StatiCa (12). When the connection is strong enough the connection is checked by the component method (15) to check the stiffness. In case the connection is not stiff enough (16), the connection is re-designed (14) and checked again in both IDEA StatiCa (12) and the component method (15).

When all the checks are done and the connection is sufficiently strong and stiff, all the data is send back to the parametric model (5). After everything is in the parametric model, the cost formula is applied to calculate the total costs of the structure (17). The costs are then normalized, to make sure the costs can be easily compared to each other. This cost factor including the data of the structure and connection is stored in an Excel file (18). When the complete optimization is ended, the stiffness of the connection is changed and the whole process can be done again. In case the lowest stiffness of the list is used, the whole optimization process is redone with a different topology.

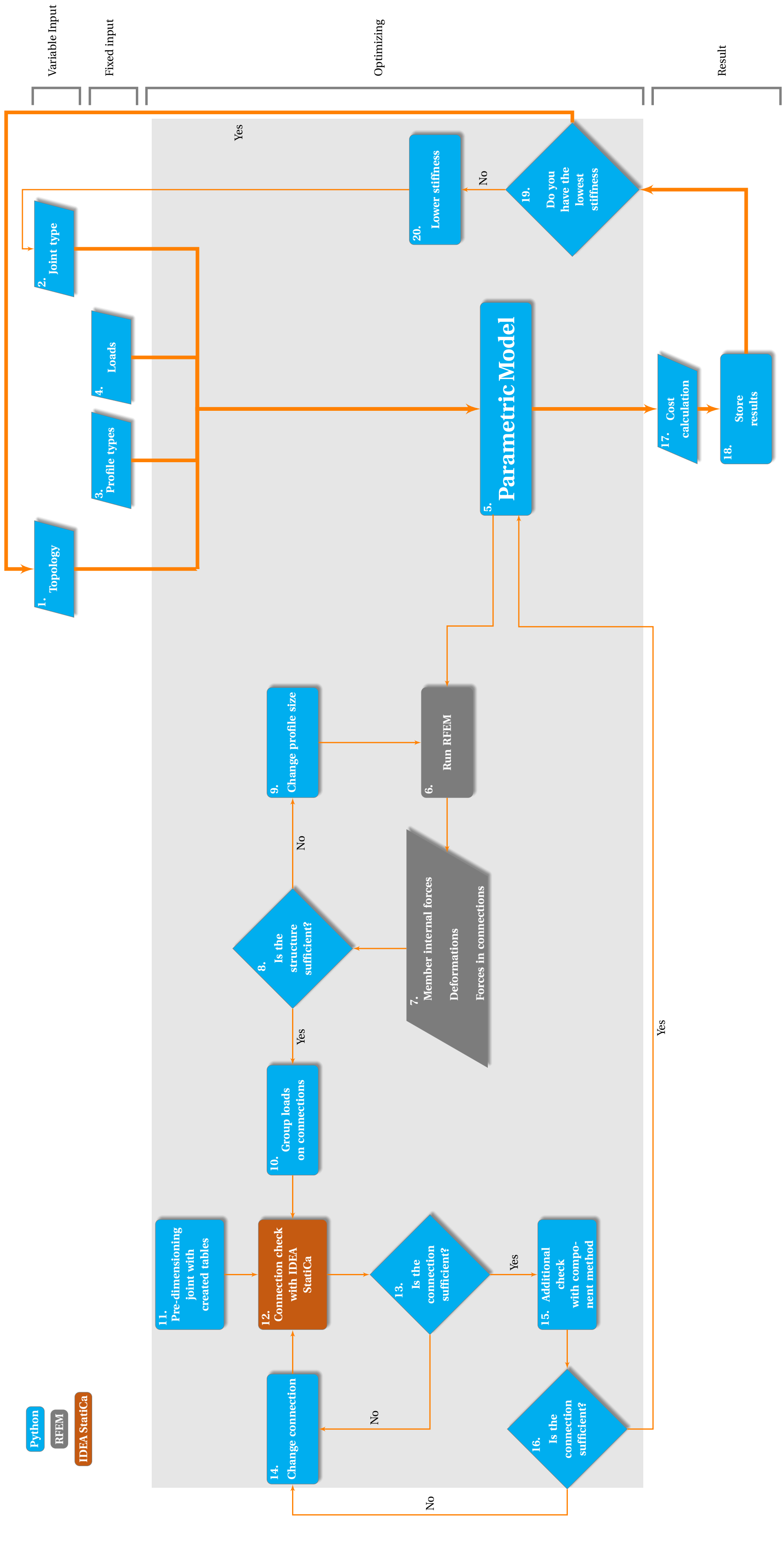


Figure 3.9: Flow diagram of process semi-rigid connections

3.3.3. DIFFERENCE WITH EXISTING TOOLS

The differences between the tool and methodology described above and other optimization tools and methodologies found in literature can be found in the table below.

	Used tool and methodology	Other tools and methodologies
Optimization part	Complete structure including bolted connections	Either connection, structure or complete structure including welded connections
Type of optimization	Cost optimization including connections	Weight optimization or cost optimization of part of structure
Connection design	Pre-design from a created database with multiple designs	One type of fixed connection design
Cost formula	Includes bolted connections	Only includes welded connections or uses factor for connections

One of the differences between the tool mentioned above and the optimization tools found in literature, see chapter 2, is that the tool used in this thesis optimizes the complete structure including connections to the costs. The existing tools found were tools that either optimizes the structure or connection. Another existing tool found used an optimization of the complete structure including bolted connections, but this was not a cost optimization. This was a weight optimization. In this case the semi-rigid connections are not considered. There was a case found where semi-rigid connections are considered but the methodology used was different. Multiple connections where designed and the structure was designed for these connections. The cheapest option would then be the optimized structure according to the study. There is however one tool found that did a cost optimization of the structure including the connections . But in this case all the connections are welded, and in this study bolted connections are considered.

Another big difference is the cost formula, because there is no tool that includes both the structure and the bolted connection. There is also no cost formula found that includes the structure and connection with the same precision. There were cost formulas found of the complete structure that included a factor for the costs depending on the stiffness of the connection. The biggest change due this new cost formula is that the semi-rigid connections can also be considered in more detail then with just a weight factor.

The biggest difference is the method used in this thesis to pre-dimension the connections. This is done by creating a database with all the different combinations possible for the given type of designs. The connection design is taken from this database based on the wanted stiffness. In other tools the connection is based on one design with a fixed number of bolts, stiffeners and a fixed plate thickness. One downside of the method used in this thesis is in case a new type of connection is wanted, the complete database for this design needs to be created.

3.4. POSSIBILITIES

In this section the possibilities of the tool and methodology used in the tool are described. To do this first the expected deliverables are discussed and then the possibilities are mentioned.

3.4.1. EXPECTED DELIVERABLES

The expected outcomes of this optimization process are:

- A process on how detailed bolt design can be considered at the beginning of the design phase
- A tool that incorporates this process
- A conclusion about what is the most cost-efficient variant for the given case study
- A conclusion about the difference between a cost optimization and a weight optimization for a given structure

The expectations for the difference between a weight and cost optimization according to theory can be found below in figure 3.10 and figure 3.11. These figures are a representation of the theories.

It is expected that for the weight optimization the structure with the lowest number of purlins and frames will have the lowest costs see figure 3.10. For the cost optimization it is expected that the structure with more frames and purlins but cheaper connections can have lower total costs, see figure 3.11. It is also expected that in a cost optimization the semi-rigid options are the cheapest when there is looked at the weight of the structure the fully rigid connections will give the lowest weight.

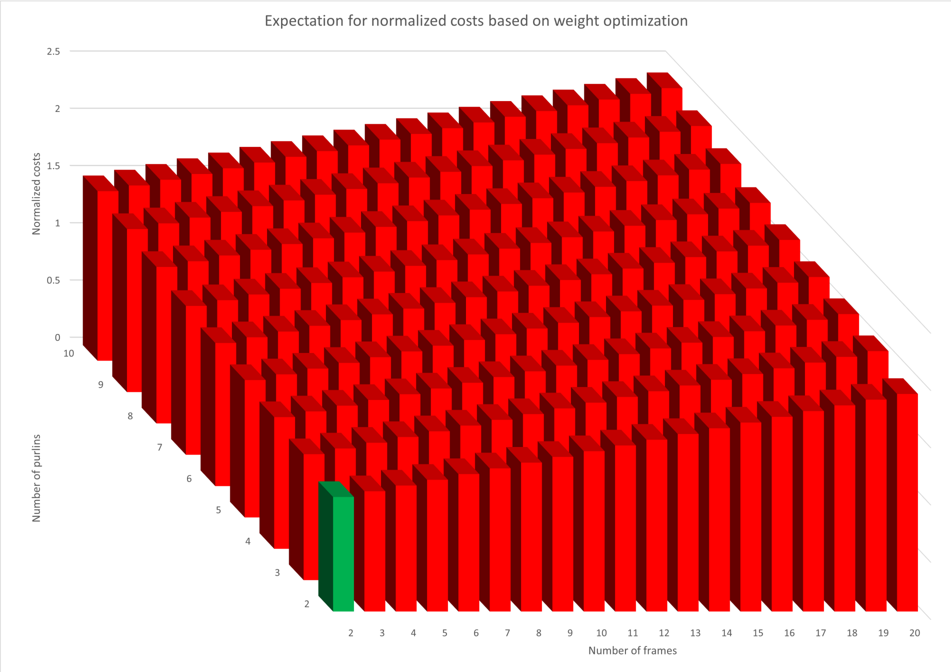


Figure 3.10: Expectation for normalized costs based on weight optimization

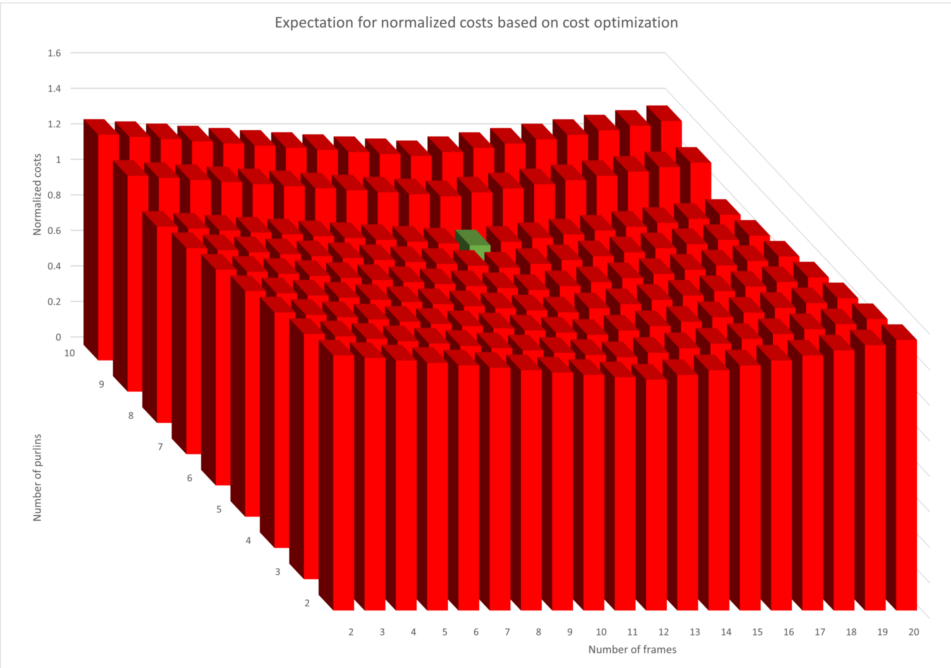


Figure 3.11: Expectation for normalized costs based on cost optimization

3.4.2. POSSIBILITIES AND LIMITATIONS

In this section the limitations and possibilities of the tool are discussed.

One of the limitations of the current tool is that it can only be used for structures in the Netherlands, because these are the only load cases programmed in the tool.This means that if the code is extended, the tool can be used for structures all around the world. This can be done by changing for example the load input from a fixed to a variable input.

The second limitation of the current tool is the type of connection. At the moment only end-plate connections are considered in the optimization tool. When additional connection types are added, a new database needs to be created for these connection types.

Another limitation is that the only connection in the steel hall that is optimized is the beam-column connection not the other connections, and that the assumption is made that all the connections in the structure are the same.

When the limitations mentioned above are considered and the possibilities they create are applied, all type of structures can be optimized including the connections with the optimization method used in this thesis. However for each type of connection design a database needs to be created once. The possibilities of the tool are the optimization of structures that include beam-column connections. This is not limited to steel halls. The type of optimization is fixed to a cost optimization. By changing the cost formula to a weight formula the optimization can be to weight. To create an optimization to strength the optimization tool needs some more alterations. However the same methodology can be used with the connection database.

4

CONNECTION DESIGN IN CONCEPT PHASE

In chapter 3 it can be seen that for the optimization process proposed it is necessary to include the detailed connection design in the process. To do this the connection design needs to be moved to the concept design phase. This is discussed in this chapter.

Firstly, the possibilities and challenges are discussed of moving the connection design to the concept design phase. Secondly, the influence of the different connection design variables on the stiffness and bending moment are discussed. Thirdly, the assumptions and constraints on the connection designs are discussed. Fourthly, the structural analysis of the connection design is discussed. At last the method is described how the connection design is considered in the overall optimization. The influence of the different connection design variables on the stiffness and bending moment are discussed.

The structure and connections are designed according to the Eurocode. All the formulas that are used for the analysis of the structure are according to Eurocode EN 1993-1-1 [80]. The connection is designed according to the component method of EN 1993-1-8 [41]. A benchmark check for the programmed code is done in appendices F and G.

4.1. POSSIBILITIES AND CHALLENGES OF MOVING THE DETAILED DESIGN PHASE

In this section the challenges and possibilities are discussed of moving the detailed connection design to the concept phase. A practical reason why the detailed connection design is not moved to the concept design phase is the workflow mentioned in figure 1.1a.

CHALLENGES

Moving the detailed design of joints forward can lead to numerous challenges. Some of which can be solved easily and others more difficult. One of the challenges that can occur when moving the detailed design forward is if there is not yet a good concept design. This could result in a lot of engineering time on concepts not used.

Another challenge can occur in the old way of working, see figure 1.1, in the case that there is no or not enough communication between the engineer, contractor and steel contractor. When this is the case some important factors can be forgotten in the concept phase, because there is a lack of knowledge. This could lead to failing designs that should be redone which costs a lot of time and money.

A third challenge can be the number of iterations that need to be done when taking the detailed design of the joints to the concept phase. In the old workflow these iterations can take a lot of extra time and money. This iteration process can be solved by using computational tools and parametric design programs, see figure 1.1b.

The three challenges above can be solved by using KBE software. This can include all the knowledge, which solves the second challenge. The number of iterations and time duration is solved by using computers.

POSSIBILITIES

When the connection design is taken into account at the start of the design process, the risk of having to increase the profile dimensions later in the design phase is avoided, this could save time and money. This risk is caused by the internal forces that in the concept design phase might be low enough for the profiles, but in the detailed design phase these forces may be too high for the connection that was assumed in the concept phase. Which means that the structure should be redesigned with different assumptions of the connection in the concept design phase. This could lead to different profile dimensions.

Another advantage that can be realized is the use of semi-rigid joints. With these joints the rotational stiffness of the joint can be considered. This will influence the moment distribution and deflections in the structure. This can lead to an extra cost optimization possibility in case the iterative procedure is used that can be seen in figure 4.1. This iterative process can lead to a cost efficient structure, due to an optimum usage of the material in the connection and beam.

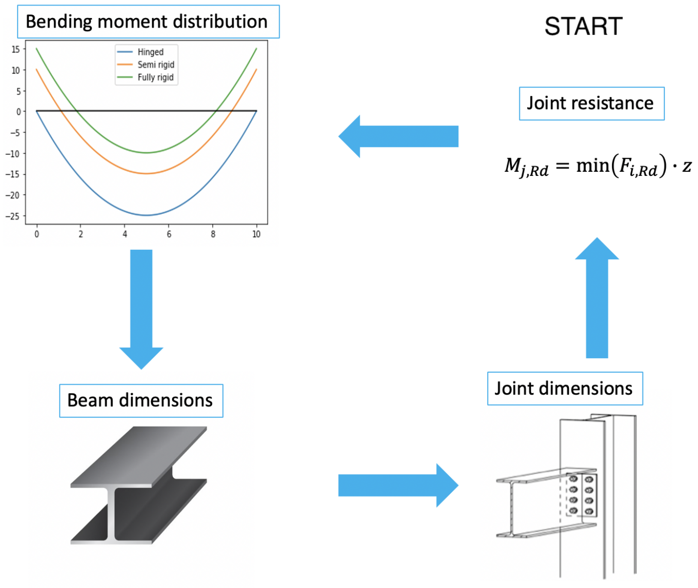


Figure 4.1: Iteration process

4.2. INFLUENCE OF CONNECTION VARIABLES

To use connection design in the optimization process limitations need to be made on the variables considered in the design. The first decision made, is that only end-plate connections are considered in the design.This is done to limit the types of connections and because end-plate connections can act like a hinged, semi-rigid and fully-rigid connections.

For the end-plate connection there is looked at the influence on the stiffness and the bending moment resistance of the connection for different variables. This study can be found in appendix H. The stiffness and bending moment are calculated with the component method.

- End plate length
- End plate length including only a top stiffener
- End plate length including top and bottom stiffener
- Haunch height/width ratio
- Haunch height
- Haunch height including only a top stiffener in the column
- Haunch height including a top and bottom stiffener in the column
- Haunch height including a top and bottom stiffener in the column and a beam stiffener
- Number of bolt rows
- Number of bolt rows including one bolt row above the beam

- Plate length above the beam
- Bolt size
- Bolt strength class
- End plate thickness

The different variables can be found in figure 4.2.

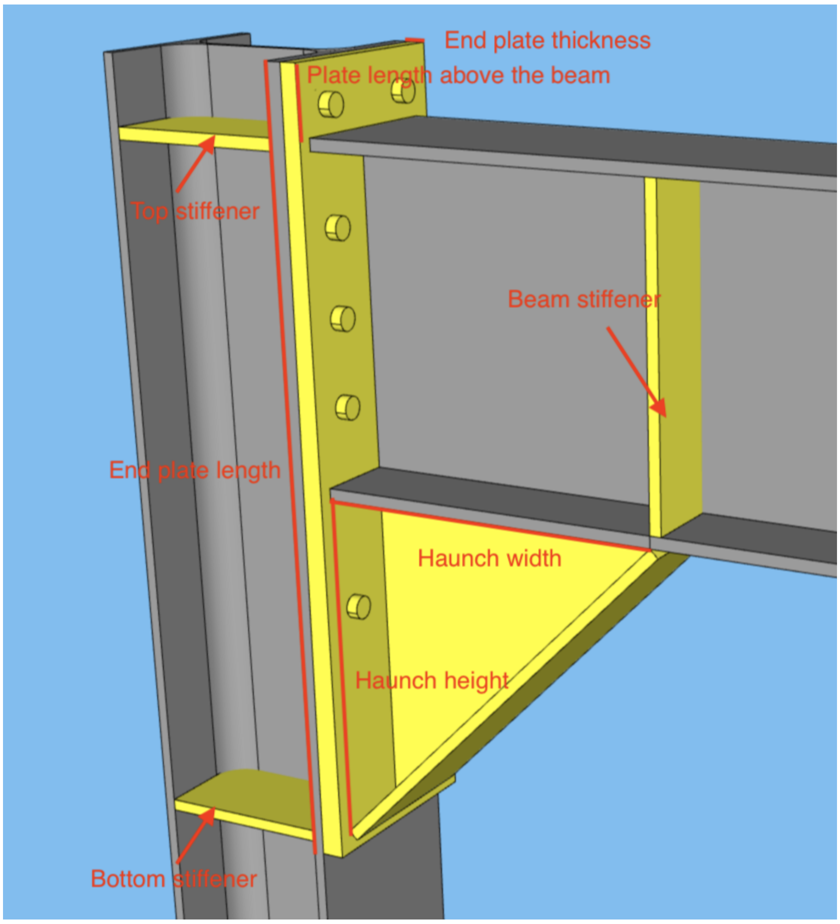


Figure 4.2: Connection variables

Because increasing the end plate length can increase the stiffness and bending moment resistance of the connection significantly up till a certain length, this is one of the variables that is considered in the designs used for the connections. To reduce the number of options in the connection designs there are 2 values in the main designs. 0.6 times the beam height and 1 times the beam height plus the haunch height (in case of a haunch) plus twice the end plate thickness to have space for fillet welds.

It can be seen that only a top stiffener has no influence on the stiffness of the connection and a top and bottom stiffener has a significant influence on the stiffness of the connection. That is why in the considered connection designs only the combination of both a top and bottom stiffener is considered. In case when a haunch is used there is also a stiffener added in the beam.

Only increasing the haunch length has no influence on the stiffness or bending moment resistance of the connection. It is also seen that increasing the haunch height increases the stiffness and bending moment resistance significantly. This is why a haunch that has a fixed haunch height over length ratio of 1 is selected. The haunch height is fixed to the height of the beam, this is to reduce the number of connections that need to be considered in the optimization process. This is why the haunch is a variable in the main design options.

Increasing the number of bolt rows increases the stiffness and bending moment resistance of the connection significantly. In the case of a bolt row above the beam flange in the tension zone. the number of bolt rows between the beam flanges have a reduced influence on the stiffness. This is why in the connection designs the bolt row above the flange will be considered in the main design difference and the number of bolt rows between the flanges will be considered within the main designs.

Increasing the plate extension above the flange reduces the stiffness and bending moment resistance. This is why in the connection designs considered the plate extension will be limited to the minimum distance needed, depending on the bolt hole diameter.

Increasing the bolt size increases the bending moment resistance of the connection significantly, but it increases the stiffness of the connection less. This is why in the connection designs this will be considered as a variable within the main designs.

It is also found that increasing the bolt strength the bending moment resistance of the connection is increased. The bolt strength class has no influence on the stiffness. This is why there are only 2 bolt strength classes considered as variable within the main designs.

Increasing the end plate thickness can increase both the stiffness and bending moment resistance significantly. This is why in the connection design all available standard plate thicknesses are considered in all the different designs.

4.3. DESIGN OF THE CONNECTION

In this research there is looked at 4 main types of designs for the connection. All of these designs are end-plate connections. These designs are discussed in more detail in the subsections below.

The idea of these designs is that these designs give the possibility for the connection to be hinged, semi-rigid or fully-rigid. In none of the designs is staggered spacing an option and in all the designs is the number of bolt-columns set to two, one on each side of the beam web. In all of the designs are the bolt-rows distributed evenly over the plate height if possible. The cases when it is not possible are if evenly distributed the minimum or maximum spacings of the bolt grid are higher or lower than the allowed values according to table 4.1. The variables in the designs are based on the influence of the variables from section 4.2.

The figures of the designs below are all for a column profile of HEA 180 and a beam profile of IPE 300. The dimensions that can be found in the figures differ for all the different connections. The bolt size considered in the figures below is an M20 bolt. The end-plate thickness in the figures below are all 10 mm.

4.3.1. DESIGN 1

The idea of design 1 is to create the possibility of hinged connections. Whether or not this can be considered as a hinged connection depends on the connection design and the beam and column dimensions. Design 1 can be found in figure 4.3.

As can be seen in the figure 4.3 design 1 is a partial end-plate. The design choices made in this design is that the height of the end-plate is 0.6 times the height of the beam. The width of the end-plate is the width of the column. The minimum and maximum width between the bolts are determined with the rules of EN 1993-1-8 chapter 3.5 [41]. These rules can also be found below in table 4.1

The variables in this design are the:

- Number of bolt rows
- Thickness of the end-plate
- Bolt type

The number of bolt rows is limited by the height of the end-plate. The thickness of the end-plate can vary between 8, 10, 12, 15, 20, 25 and 30 mm. These are standard end-plate thicknesses. The bolt types are limited to bolt classes 8.8 and 10.9. This is done to limit the options in the optimization process. The bolt sizes can vary between M10, M12, M14, M16, M18, M20, M22, M24, M27, M30, M36. This is however limited due to the number of bolt rows and the height of the plate, see table 4.1.

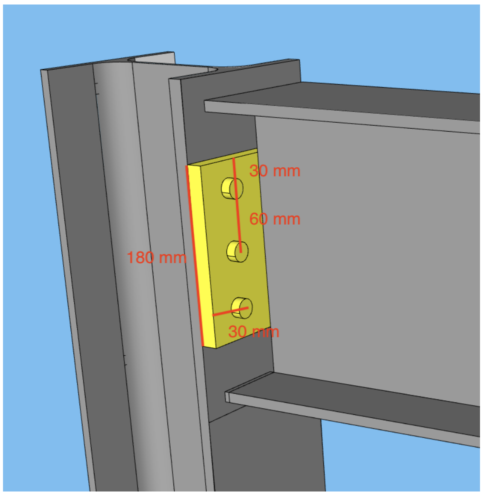


Figure 4.3: Isometric view of design 1

Distances and spacings, see Figure 3.5	Minimum	Maximum: Structures made from steels conforming to EN 10025 except steels conforming to EN 10025-5 — Steel exposed to the weather or other corrosive influences	Maximum: Structures made from steels conforming to EN 10025 except steels conforming to EN 10025-5 — Steel no exposed to the weather or other corrosive influences	Maximum: Structures made from steels conforming to EN 10025-5 — Steel used unprotected
End distance e_1	$1.2d_0$	$4t + 40mm$		$\max(8t; 125mm)$
Edge distance e_2	$1.2d_0$	$4t + 40mm$		$\max(8t; 125mm)$
Spacing p_1	$2.2d_0$	$\min(14t; 200mm)$	$\min(14t; 200mm)$	$\min(14t; 175mm)$
Spacing p_2	$2.4d_0$	$\min(14t; 200mm)$	$\min(14t; 200mm)$	$\min(14t; 175mm)$

Table 4.1: Minimum and maximum spacing, end and edge distance [41]

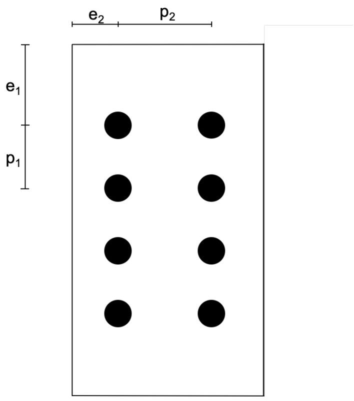


Figure 4.4: Distances and spacings of bolt grid

4.3.2. DESIGN 2

The idea of design 2 is to create the possibility of semi-rigid connections that are close to hinged connections. Whether this connection will act like a semi-rigid connection depends on the connection design and the beam and column dimensions. Design 2 can be found in figures 4.5 and 4.6. There are two combinations of design 2 one without stiffeners and one with stiffeners.

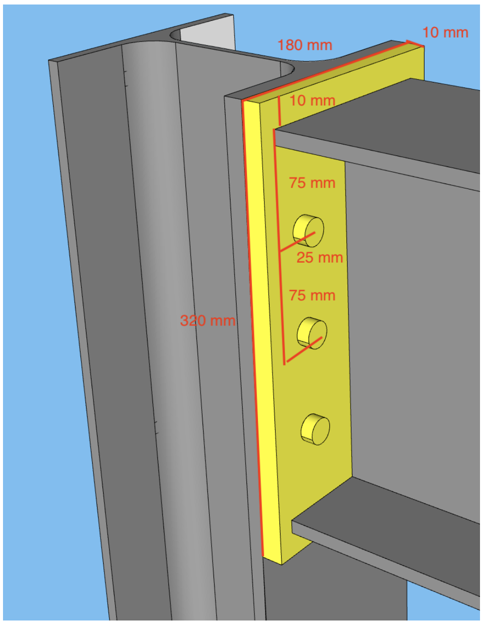


Figure 4.5: Isometric view of design 2 without stiffeners

For the design including stiffeners, the dimensions can be seen in the figure without stiffeners.

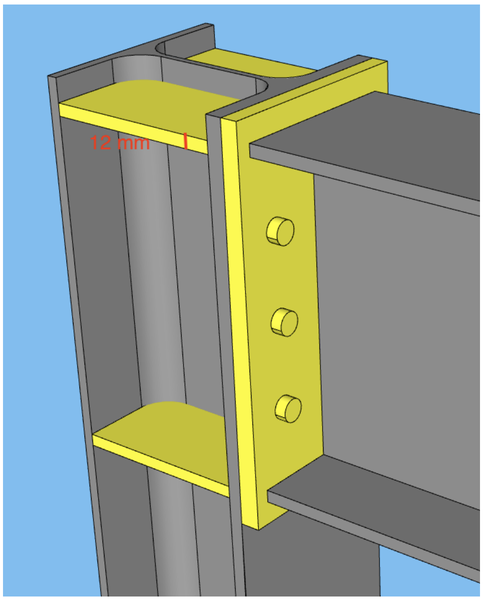


Figure 4.6: Isometric view of design 2 with stiffeners

Design 2 is a full end plate with a small extension at the top and bottom for the beam flange welds. In the design this extension is the same length as the end plate thickness. Meaning that when the plate thickness varies this extension length also varies. This is done to increase the strength of the column web in transverse compression as maximum as possible with increasing the plate length as minimum as possible

The height of the end-plate is the height of the beam plus the height of this extensions. The width of the end-plate is the same as in design 1, meaning that it is the width of the column.

In the case of the design with stiffeners in figure 4.6.The stiffeners are on the same height as the flanges of the beam. The stiffeners are both in the compression and tension zone. The thickness of the stiffeners is determined by taking the value of a standard plate that is higher and comes closest to the beam flange thickness. The list with standard plate thicknesses is 3, 4, 5, 6, 8, 10, 12, 15, 20, 25 and 30 mm. So for example when the beam flange thickness is 12.6 mm the stiffeners become 15 mm thick.

The variables of the connection are:

- With or without stiffeners
- Number of bolt rows
- Thickness of the end-plate
- Bolt type

The number of bolt rows is limited by the height of the end-plate, meaning the height of the beam. The thickness of the end-plate can vary between 8, 10, 12, 15, 20, 25 and 30 mm. These are standard end-plate thicknesses. This automatically determines the extension length below the beam. The bolt types are limited to bolt classes 8.8 and 10.9. This is done to limit the options in the optimization process. The bolt sizes can vary between M10, M12, M14, M16, M18, M20, M22, M24, M27, M30, M36. This is however limited due to the number of bolt rows and the height of the plate, see table 4.1.

4.3.3. DESIGN 3

The idea of design 3 is to create a connection that is semi-rigid and close to fully rigid. Whether this connection will act like a semi-rigid or fully-rigid connection depends on the connection design and the beam and column dimensions. Design 3 can be found in figures 4.7 and 4.8. There are two combinations of design 3 one without stiffeners and one with stiffeners.

The remaining dimensions of design 3 can be found in figure 4.5.

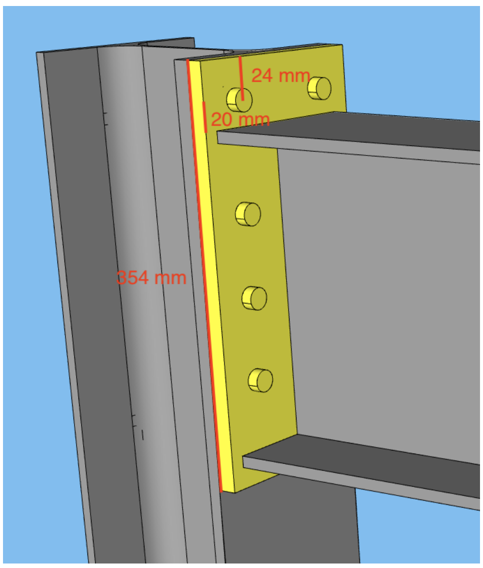


Figure 4.7: ISO view of design 3 without stiffeners

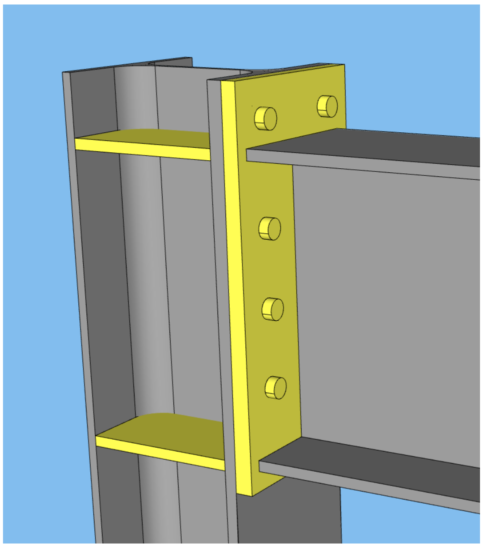


Figure 4.8: ISO view of design 3 with stiffeners

Design 3 is a full end plate with a small extension at the bottom and an extra bolt row at the top above the beam. The extension at the bottom is for the bottom beam flange weld. In the design this extension is the same length as the end plate thickness. Meaning that when the plate thickness varies this extension length also varies. This is done to increase the strength of the column web in transverse compression as maximum as possible with increasing the plate length as minimum as possible.

The length of the extension at the top to include the top bolt row is the minimum length necessary depending on the bolt size, see table 4.1. In the design only one bolt row above the beam is considered. This already increases the stiffness of the connection significantly compared to design 2. The bolts above the beam has the same size as the other bolts in the connections.

The height of the end-plate is the height of the beam plus the height of the top and bottom extensions. The width of the end-plate is the same as in design 1, meaning that it is the width of the column.

In the case of the design with stiffeners in figure 4.8.The stiffeners are on the same height as the flanges of the beam. The stiffeners are both in the compression and tension zone. The thickness of the stiffeners is determined by taking the value of a standard plate that is higher and comes closest to the beam flange thickness. The list with standard plate thicknesses is 3, 4, 5, 6, 8, 10, 12, 15, 20, 25 and 30 mm. This is the same as with design 2.

The variables of the connection are:

- With or without stiffeners
- Number of bolt rows
- Thickness of the end-plate
- Bolt type

The number of bolt rows is limited by the height of the end-plate, meaning the height of the beam. The number of bolt rows above the beam is always 1 in this design. The thickness of the end-plate can vary between 8, 10, 12, 15, 20, 25 and 30 mm. These are standard end-plate thicknesses. This automatically determines the extension length below the beam. The bolt types are limited to bolt classes 8.8 and 10.9. This is done to limit the options in the optimization process. The bolt sizes can vary between M10, M12, M14, M16, M18, M20, M22, M24, M27, M30, M36. This is however limited due to the number of bolt rows and the height of the plate, see table 4.1. The bolt size determines the length of the extension above the beam. To make sure that the plate is as short as possible which saves costs.

4.3.4. DESIGN 4

The idea of design 4 is to create a design that can act as a fully-rigid connection. Whether this connection will act like a fully-rigid connection depends on the connection design and the beam and column dimensions.

Design 4 can be found in figures 4.9 and 4.10. There are two combinations of design 4 one without stiffeners and one with stiffeners.

The remaining dimensions of design 4 can be found in figures 4.5 and

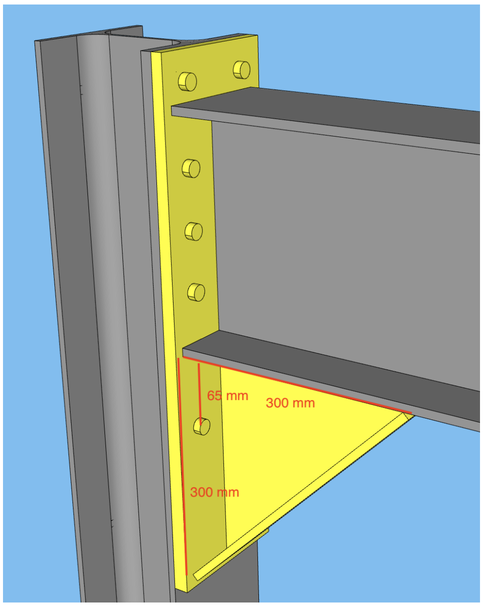


Figure 4.9: ISO view of design 4 without stiffeners

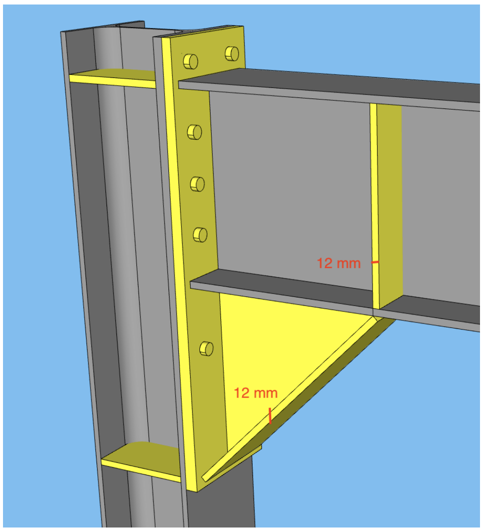


Figure 4.10: ISO view of design 4 with stiffeners

Design 4 is a full end plate including a haunch with a small extension at the bottom below the haunch and an extra bolt row at the top above the beam. The extension at the bottom is for the bottom beam flange weld. In the design this extension is the same length as the end plate thickness. Meaning that when the plate thickness varies this extension length also varies. This is done to increase the strength of the column web in transverse compression as maximum as possible with increasing the plate length as minimum as possible.

The length of the extension at the top to include the top bolt row is the minimum length necessary depending on the bolt size, see table 4.1. In the design only one bolt row above the beam is considered. This already increases the stiffness of the connection significantly compared to design 2.

The haunch height is the same as the beam height. The haunch height over length ratio is 1. The haunch web thickness and the haunch flange thickness are determined in the same way as the stiffener thickness in design 2 and 3. Where the haunch web thickness is compared with the beam web thickness and the haunch flange thickness with the beam flange thickness.

The height of the end-plate is the height of the beam plus the height of the haunch plus the height of the top and bottom extensions. The width of the end-plate is the same as in design 1, meaning that it is the width of the column.

In the case of the design with stiffeners in figures 4.10.The stiffeners are on the same height as the flanges of the beam. The stiffeners are both in the compression and tension zone. The thickness of the stiffeners is determined by taking the value of a standard plate that is higher and comes closest to the beam flange thickness. The list with standard plate thicknesses is 3, 4, 5, 6, 8, 10, 12, 15, 20, 25 and 30 mm. So for example when the beam flange thickness is 12.6 mm the stiffeners become 15 mm thick. This is the same as with design 2.

The variables of the connection are:

- With or without stiffeners
- Number of bolt rows
- Thickness of the end-plate
- Bolt type

The number of bolt rows is limited by the height of the end-plate, meaning the height of the beam. The number of bolt rows above the beam is 1 in this design. The number of bolt rows below the beam is 1 and is on one third of the height of the haunch. This bolt has no influence on the stiffness or bending moment capacity of the connection. This is only added for practical reasons. The thickness of the end-plate can vary between 8, 10, 12, 15, 20, 25 and 30 mm. These are standard end-plate thicknesses. This automatically determines the extension length below the beam. The bolt types are limited to bolt classes 8.8 and 10.9. This is done to limit the options in the optimization process. The bolt sizes can vary between M10, M12, M14, M16, M18, M20, M22, M24, M27, M30, M36. This is however limited due to the number of bolt rows and the height of the plate, see table 4.1. The bolt size determines the length of the extension above the beam. To make sure that the plate is as short as possible which saves costs.

4.4. STRUCTURAL ANALYSIS CONNECTION

The structural analysis of the connection is checked with both the Component Method from the Eurocode and the software IDEA StatiCa. It is done in both because the current version of the software IDEA StatiCa cannot give the initial stiffness and the bending moment resistance of a connection through the API. This is why the Component Method is used to calculate the stiffness and the bending moment resistance of a connection. IDEA StatiCa is used to check the strength of the different connection elements more precisely.

The Component Method is programmed in python, this is according to Eurocode EN 1993-1-8 [41].

The programmed component method is verified by checking 6 examples. These examples are verified by comparing them in different programs. The comparison between three of these examples are done between IDEA StatiCa, Excel, Programmed code and an pre-calculated example from books or lectures. The other three examples are compared between IDEA StatiCa, Excel and the Programmed code. The examples including the results of the comparison can be found in appendix G.

4.5. CONNECTIONS IN OVERALL OPTIMIZATION

This section describes how the connection design is considered in the overall optimization. This starts with the input of the joint. Then the database for the pre-design of the connections is described. And at last how a connection is taken from this database and used in the optimization loop.

4.5.1. INPUT

The connection is considered in the input by giving the wanted joint type. This can be selected from a list of 5 options.

- Hinged
- Low semi-rigid

- Medium semi-rigid
- High semi-rigid
- Rigid

This input gives a wanted stiffness for the connection, more about this stiffness can be found in section 3.2.4. When this input is selected it checks the database for a stiffness that is in the range of plus or minus 10% of the wanted stiffness.

4.5.2. CONNECTION DATABASE

The connection database is created by looking at all the different combinations from the 4 designs mentioned above. This is done for all different beam column combinations. The columns can be any size from the profiles HEA or HEB and the beams can be of any size of the IPE profile. For this beam column combination 4 tables are created, one for each design. An example of a table for design 1 and the column HEA180 and beam IPE270 can be found in figure 4.11. In this example it can be seen that for each end plate thickness (8, 10, 12, 15, 20, 25 and 30) the stiffness and bending moment resistance is calculated for all different numbers of bolt rows and bolt sizes that are possible with the number of rows.

This database then gives all the possible design options including their stiffness and bending moment resistance. The loads used for the calculation of the stiffness and bending moment resistance is a bending moment of 70 kNm and a shear force of 50 kN. These loads are not based on anything.

In the database the stiffness is divided in 3 colours green (hinged), yellow (semi-rigid), red (rigid). These colours change depending on the length of the beam. Meaning that the colours in the database are an indication and the stiffness value describes whether or not the connection is hinged, semi-rigid or rigid. The stiffness and bending moment resistance are calculated with the component method according to EN 1993-1-8.

4.5.3. DATABASE IN OPTIMIZATION LOOP

When the database is created the joint input gives a range of stiffnesses. When this is known there is looked in the database tables of the given beam-column combination. From these tables a list is created with all the designs that have a stiffness that falls in the input range. For each design in this list the cost of the connection is calculated with equation (3.8). This list is then sorted on the costs, from low to high costs. In the connection optimization part there is looped through this list and at the end of the check the actual stiffness of this connection is checked with the actual acting loads. When this actual stiffness is known it is checked if this stiffness is within -10% and the wanted stiffness. This is done because in the case the stiffness of the connection is a bit lower than the calculated stiffness, the connection gets lower loads working on it. This means that the connection does not have to be recalculated for these new loads.

S = Stiffness

M = Bending moment resistance

HEA180 + IPE270 Design 1														
End plate thickness														
	8		10		12		15		20		25		30	
2 bolt rows														
Bolt Size	S 8	M 8	S 10	M 10	S 12	M 12	S 15	M 15	S 20	M 20	S 25	M 25	S 30	M 30
M10:8.8	858	7	1041	7	1142	7	1212	7	1238	7	1226	7	1203	7
M12:8.8	885	10	1084	10	1199	10	1284	10	1325	10	1325	10	1311	10
M14:8.8	905	14	1116	14	1241	14	1336	14	1391	14	1399	14	1392	14
M16:8.8	919	19	1138	19	1271	19	1375	19	1439	19	1455	19	1455	19
M18:8.8	824	22	1033	23	1161	23	1265	23	1333	23	1353	23	1358	23
M20:8.8	657	21	844	24	965	24	1067	24	1138	24	1163	25	1172	25
M22:8.8	503	19	666	24	777	24	875	24	946	24	974	24	985	24
M24:8.8	434	16	572	19	666	19	749	19	809	19	832	19	843	19
M27:8.8	331	15	432	16	500	16	558	16	601	16	618	16	625	16
M30:8.8	274	16	354	17	407	17	451	17	484	17	496	17	502	17
M10:10.9	858	9	1041	9	1142	9	1212	9	1238	9	1226	9	1203	9
M12:10.9	885	12	1084	12	1199	12	1284	12	1325	12	1325	12	1311	12
M14:10.9	905	17	1116	17	1241	17	1336	17	1391	17	1399	17	1392	17
M16:10.9	919	22	1138	23	1271	23	1375	23	1439	23	1455	23	1455	23
M18:10.9	824	22	1033	24	1161	24	1265	24	1333	24	1353	25	1358	25
M20:10.9	657	21	844	24	965	24	1067	24	1138	24	1163	25	1172	25
M22:10.9	503	19	666	24	777	24	875	24	946	24	974	25	985	25
M24:10.9	434	18	572	21	666	21	749	21	809	21	832	21	843	21
M27:10.9	331	16	432	18	500	18	558	18	601	18	618	18	625	18
M30:10.9	274	18	354	18	407	18	451	18	484	18	496	18	502	18
3 bolt rows														
Bolt Size	S 8	M 8	S 10	M 10	S 12	M 12	S 15	M 15	S 20	M 20	S 25	M 25	S 30	M 30
M10:8.8	1110	11	1330	13	1452	13	1540	13	1581	13	1579	13	1562	13
M12:8.8	1137	14	1371	16	1505	16	1605	16	1661	16	1670	16	1663	16
M14:8.8	1156	17	1401	20	1543	20	1653	20	1720	20	1737	20	1737	20
M16:8.8	1170	21	1422	24	1570	24	1687	24	1762	24	1786	24	1792	24
M18:8.8	1101	27	1345	26	1492	26	1610	26	1689	26	1717	26	1727	26
M20:8.8	942	25	1170	29	1310	29	1426	29	1508	29	1539	29	1552	29
M10:10.9	1110	13	1330	14	1452	14	1540	14	1581	14	1579	14	1562	14
M12:10.9	1137	16	1371	18	1505	18	1605	18	1661	18	1670	18	1663	18
M14:10.9	1156	21	1401	23	1543	23	1653	23	1720	23	1737	23	1737	23
M16:10.9	1170	26	1422	26	1570	26	1687	26	1762	26	1786	26	1792	26
M18:10.9	1101	27	1345	28	1492	28	1610	28	1689	28	1717	29	1727	29
M20:10.9	942	25	1170	29	1310	29	1426	29	1508	29	1539	29	1552	29
4 bolt rows														
Bolt Size	S 8	M 8	S 10	M 10	S 12	M 12	S 15	M 15	S 20	M 20	S 25	M 25	S 30	M 30
M10:8.8	1202	12	1421	13	1543	13	1632	13	1677	13	1678	13	1665	13
M12:8.8	1228	15	1459	16	1591	16	1691	16	1749	16	1761	16	1756	16
M14:8.8	1285	19	1528	20	1668	20	1776	20	1844	20	1863	20	1866	20
M16:8.8	1393	24	1647	24	1794	24	1909	24	1985	24	2011	24	2019	24
M10:10.9	1202	14	1421	15	1543	15	1632	15	1677	15	1678	15	1665	15
M12:10.9	1228	17	1459	19	1591	19	1691	19	1749	19	1761	19	1756	19
M14:10.9	1285	22	1528	23	1668	23	1776	23	1844	23	1863	23	1866	23
M16:10.9	1393	30	1647	30	1794	30	1909	31	1985	31	2011	31	2019	31
5 bolt rows														
Bolt Size	S 8	M 8	S 10	M 10	S 12	M 12	S 15	M 15	S 20	M 20	S 25	M 25	S 30	M 30
M10:8.8	1211	12	1426	13	1547	13	1638	13	1687	13	1692	13	1682	13
M12:8.8	1296	16	1523	16	1655	16	1758	16	1820	16	1835	16	1834	16
M10:10.9	1211	14	1426	15	1547	15	1638	15	1687	15	1692	15	1682	15
M12:10.9	1296	19	1523	19	1655	19	1758	19	1820	19	1835	19	1834	19
6 bolt rows														
Bolt Size	S 8	M 8	S 10	M 10	S 12	M 12	S 15	M 15	S 20	M 20	S 25	M 25	S 30	M 30
M10:8.8	1378	15	1590	15	1713	15	1808	15	1866	15	1877	15	1874	15
M10:10.9	1378	17	1590	17	1713	17	1808	17	1866	17	1877	17	1874	17
7 bolt rows														
Bolt Size	S 8	M 8	S 10	M 10	S 12	M 12	S 15	M 15	S 20	M 20	S 25	M 25	S 30	M 30
8 bolt rows														
Bolt Size	S 8	M 8	S 10	M 10	S 12	M 12	S 15	M 15	S 20	M 20	S 25	M 25	S 30	M 30

Figure 4.11: Database example

5
Case Study

In this chapter the case study performed to test the optimization model is described. The chapter starts with the inputs and parameters of the case study, and at the end of the chapter the results are presented and discussed.

5.1. Introduction

The research that is performed is a study about the cost optimization of a steel frame hall in the concept design phase of a project. In this case study the topology will be changed by:

- The number of frames
- The number of purlins

Besides the topology also the type of connection for the beam column joint is a variable, this will change from fully rigid to semi-rigid to hinged. The change in topology will automatically influence the number of connections. To make a more realistic cost estimation of the structure including the connections, the detailed design phase of the connections will be moved forward to the concept design phase and is considered during this case study.

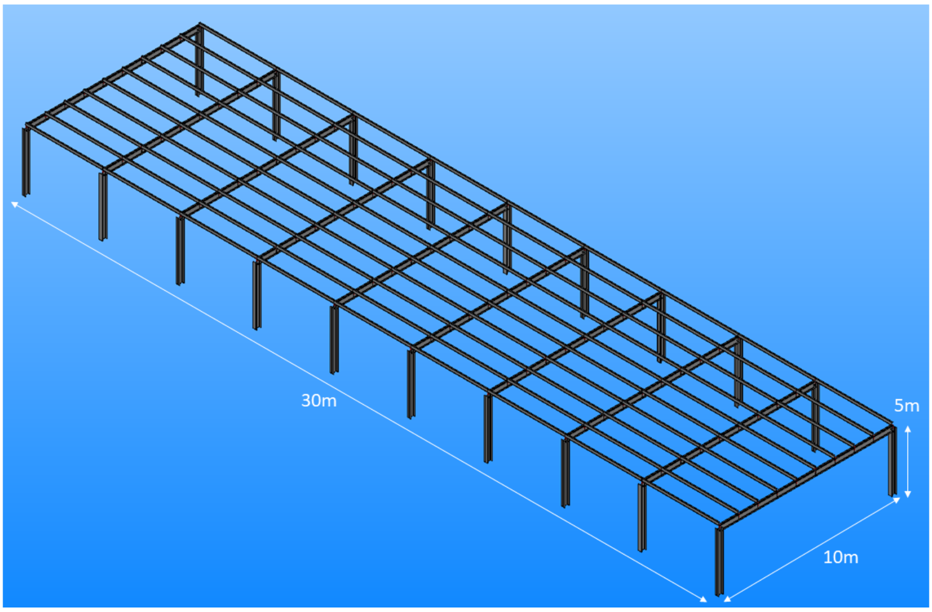


Figure 5.1: Isometric view of case study

For the case study a fictional case that fits within the boundaries of the tool is selected. This is to ensure that a lot of different topologies can be checked and therefore a lot of different results can be created. This makes it possible to compare the results with the theory and this could show more about the limitations of the tool.

The case study that will be used in this research is that of a steel hall that is used for distribution. This choice is made to have a very generic hall but still have some global boundary conditions to base the case study on. The dimensions of the steel hall can be found in figure 5.1 and in the section below.

5.2. CASE STUDY

In this section the inputs and the structural analysis of the case study are discussed.

5.2.1. INPUTS

The fixed variables of the steel hall are:

Height	5 m
Width	10 m
Length	30 m
Steel grade	S355
Elastic modulus	210,000 MPa
Shear modulus	81,000 MPa

Because the area of the hall 300 m^2 is smaller than 1000 m^2 there are no compartments necessary for fire safety [76]. The choice for a distribution hall limits the number of columns that can be used in the hall [81], because a big open space is preferred. Therefore, the span between two columns is limited to the full width of the structure 10 m, which means that the number of columns per frame used in this case study is 2. Another limitation of a distribution hall is the need for loading docks. These truck openings need to be 2.5 m wide with at least a c.t.c. distance of 4 m [82] [83]. Meaning that in the length the minimum distance between the frames is 2.5 m. This way there is space for truck openings. The steel structure considered in this cost optimization contains the steel frames including the columns and beams and the purlins on top of the frames that carry the roof , see figure 5.2.

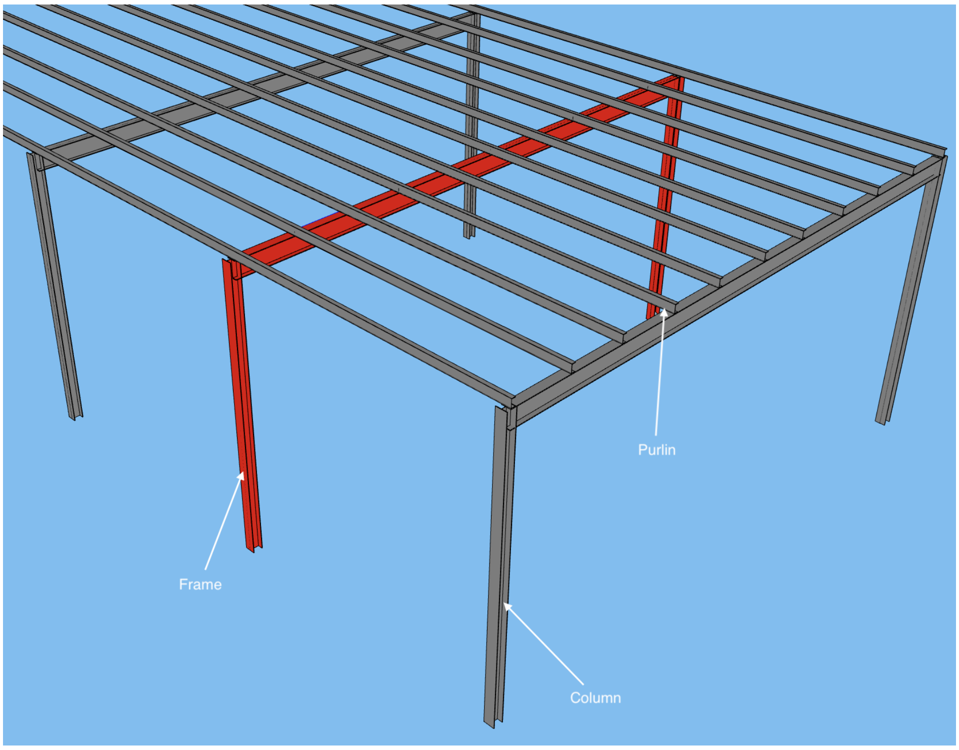


Figure 5.2: Structural elements in case study

COLUMN-BASE CONNECTION

The first step of designing a hall is to determine the type of column base connections. This connection will be assumed to be either fully rigid or hinged. The fully rigid option is needed to create a stable structure in the case of hinged beam column connections without braces. The fully rigid column base connection is more expensive, but it could lead to smaller deflections and smaller column profiles of the structure, which could reduce the costs of the structure. The detailing and the cost of this connection will be outside the scope of this research, because in this research the focus is on steel connections and not steel/concrete connections. This is why in this thesis both cases are considered but they can not be compared to each other.

PURLINS

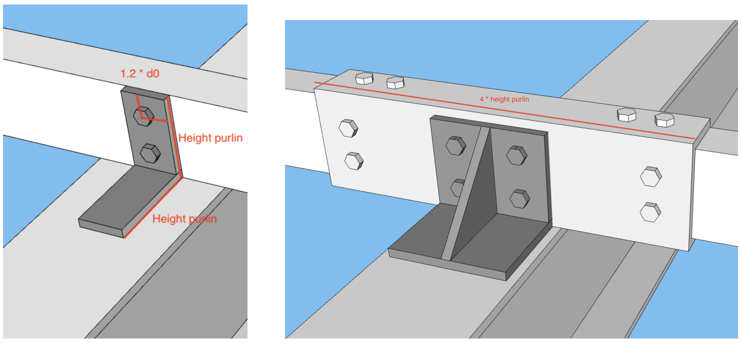
After the column base connection a roof type and insulation need to be selected, to determine part of the static load on the structure. The choice of roof and insulation also limits the distance between the purlins. Steel plates with a dimension of 2.5 m by 1 m and a weight of 6 kg/m^2 are selected for the roof [84] [85]. The insulation has the dimensions of 2.5 m by 1.2 m and a thickness of 0.2 m, the weight of the insulation is 30 kg/m^3 [86]. It is assumed that the plate beneath the insulation has the same dimensions as the roof plates. Besides the weight and dimensions, the roof is not considered in the cost optimization. This is done because the roof and insulation needed is the same area for all different topologies, meaning that the costs of the roof for each option is the same.

Now that the dimensions of the roof are known the limits for the number of purlins can be determined. The purlin profile is a UPE-profile because this is an often used profile for steel purlins. Due to the dimensions of the roof and the insulation, the distance between the purlins is maximum 2.5 m.

The minimum number of purlins is determined by the roof and insulation sheet lengths, see equation (5.1). Meaning that the number of purlins will vary from 5 to 9 purlins. This maximum number of purlins is to check the influence of the purlins.

$$n_{purlin,min} = \frac{B}{L_{roof,max}} + 1 = \frac{10}{2.5} + 1 = 5 purlins \tag{5.1}$$

The purlins are connected to the beam with an angle cleat. This angle cleat is welded on one side and bolted to the other side. Examples of the purlin-beam connections can be found in figure 5.3. The purlin-splice connection will be with a sleeve, this is a plate between the purlin and the angle cleat insuring that the purlins are connected together, see figure 5.3b. The dimensions of these connections depend on the profiles they are connecting. The type of joint for these purlin splice and purlin-beam connections are not variable because this is out of the scope for this research.



(a) Purlin-beam connection (b) Purlin-splice connection

Figure 5.3: Purlin connections

The number of purlins will be varied from 5 to 9 limited by the limitations mentioned above, they will vary with steps of two. This is because steps of 2 limits the number of options considered. This gives 3 options for the number of purlins. The purlins are available in the standard lengths of 10, 12, 15 and 18 m [87]. In this research purlin lengths of 10 are selected. This means that the purlins do not need to be cut, which saves money. The best option for the truck space would be the 12 m but this means cuts are needed and still the same amount of purlin splices are needed. This is why the 10 m long purlins are used.

FRAMES

The beam and column profiles will be limited to the classes HEA, HEB and IPE-profiles.

The length of the purlins restricts the lowest number of spans between the frames which will be 30/10 = 3, because all the purlin connections need to be on the beams. The maximum number of frames is limited due to the truck openings that are necessary for the distribution hall. This will give 30/2.5 = 12 spans. This means that the number of frames will vary from 4 to 13. These will be varied with steps of one. This gives 9 options for the number of frames. The beams are available in the following standard lengths: 6, 10, 11, 12, 13, 14, 15, 18 and 24 m [88] [89] [90]. In this study the beam lengths of 10 m is selected, this means that for the total width of 10 m there is one beam needed which can be between two columns. This also means that there is no beam splice connection needed.

The number of columns per frame will be set to 2 columns. The columns are available in HEA and HEB profiles and the beams in IPE profiles. For the columns a length of 5 m is needed. Meaning that a standard length of 6 m is selected where 1 m will be cut off. This means that there will be no column splice connections.

BRACES

Because in this thesis the focus is on the effect of the beam column connections. There will be no braces used in this case study. This means that for the hinged column-base connection the hinged column-beam connection is not considered because this creates a mechanism.

JOINTS

The type of beam-column connections will be changed between one fully rigid option, three semi-rigid options and one hinged option.

This leads to different cross sections of the purlins, the beams and the columns, which will lead to different costs per option.

The beam-column connection are end-plate connections and can vary between the four designs mentioned in section 4.3.

LOADS

The loads on the structure according to the Eurocode consists of the self-weight of the roof + insulation, self weight of the beams, columns and purlins, snow, wind and a maintenance load on the roof. When there is wind, the snow is blown of the roof. This gives five load cases:

- Permanent load
- Wind load x-direction
- Wind load y-direction
- Snow load
- Maintenance load

The application of these load cases in the RFEM model can be found in appendix B.

The wind consists of two components, one horizontal and one vertical. The vertical component can be either uplift or downforce. It is assumed for the wind load that the building is built in an unbuilt surrounding with a wind area of 1. The vertical wind force is changing over the length of the structure. The formulas and values for the wind can be found in NEN-EN 1991-1-4 [91].

The self weight of the roof is 6 kg/m^2 which is $\frac{6 \cdot 9.81}{1000} = 0.06 kN/m^2$. The self-weight of the insulation is 30 kg/m^3 which lead to $\frac{30 \cdot 0.2 \cdot 9.81}{1000} = 0.06 kN/m^2$. The plates underneath the insulation have the same dimensions as the roof, meaning that this has a self-weight of 0.06 kN/m^2.

It is assumed for the snow load that the steel hall is built in the middle of the Netherlands, which means snow zone 2. This gives a weight of the snow of 0.42 kN/m^2 according to the equations given in NEN-EN 1991-1-3 [92].

The load combinations checked are:

- Permanent load + Maintenance load
- Permanent load + Snow load
- Permanent load + Wind load y-dir
- Permanent load + Wind load x-dir

INPUT SUMMARY

In the table below a summary of the fixed input parameters is given.

Column profiles	HEA and HEB
Beam profiles	IPE
Purlin profiles	UPE
Loads	According to EN in the Netherlands snow zone 2 and wind area 1

In the table below a summary of the variable input parameters are given.

Column-base connection	Hinged or Rigid
Number of purlins	5 to 9 with steps of 2
Number of frames	4 to 13 with steps of 1
Number of columns per frame	2
Beam-column connection	Hinged, low semi-rigid, medium semi-rigid, high semi-rigid or rigid

5.2.2. STRUCTURAL ANALYSIS

The structural analysis of the steel frame is done with the use of the program RFEM. RFEM gives the internal forces in the structure. These forces are used to calculate all the checks from the Eurocode. The results that come out of these checks will tell if the structure is strong enough and if it doesn't deform too much according to the Eurocode. This check determines whether or not the profile dimensions should be altered.

The loads in the connections are taken from RFEM. With the different load combinations mentioned above, a load envelope is created and the connections are checked for the maximum forces in this envelope.

The structural analysis of the joints will be done in IDEA StatiCa and will be additionally checked with the component method. This component method check will be done in Python. For the semi-rigid connections the beam dimensions influence the stiffness of the connection, for more information see section 3.2.4. The connection designs used in the case study can be found in section 4.3.

The results of IDEA StatiCa are design ratios of the bolts, the plates and the welds. The welds are all assumed to be double fillet welds in this study. This is because fillet welds are mostly used and they do not require edge preparation which saves costs. Above a certain throat thickness penetration welds are cheaper but this is not taken into account in this case study [54]. More about the optimization process can be found in chapter 3

5.3. VERIFICATION AND RESULTS

In this section the check is presented that verifies if the tool results the most cost-efficient combination and the results of the case study are presented.

5.3.1. TOOL VERIFICATION

To check if the optimization process returns the cheapest structure a check is done for a steel hall, with the following inputs:

The variable inputs of this check are:

Number of columns per frame	2
Number of frames	9
Number of purlins	7
Beam-column connection	Medium semi-rigid
Column-base connection	Hinged
Beam profile:	IPE300 - IPE500
Column profile:	HEA180 - HEA220
Purlin profile:	UPE100 - UPE360

The hall is checked for multiple profile sizes with their optimized connection. Only the profile size combinations are considered that have a unity check below 1.0. A table with the profile combinations that have a normalized cost factor below 1.2 can be found in table 5.1 and a graph of the normalized costs per combination is presented in figure 5.4. The normalized costs are the costs divided by the lowest cost. In this table and figure the combinations that have a normalized cost factor above 1.2 are not displayed. In figure 5.4 is above each point the profile combination and the maximum design ratio of the structure presented, with this it can be seen that a high design ratio does not automatically mean cost-efficient. In this graph the normalized total costs are presented on the y-axis, the profile combination number is presented on the x-axis, see table 5.1 for all the profile combinations. The orange line is normalized cost value of the most cost efficient combination. The normalized costs increase if the purlin size increases. The normalized costs increase if the beam profile increases and the normalized costs increase if the column profile increases. It can also be seen in this graph that for this case it does not mean that the highest unity check returns in the most cost-efficient structure.

Results that are found with the optimization tool can be found in table 5.2. As can be seen in table 5.1 and figure 5.4 is the cheapest option combination 6 which has a beam profile of IPE300, a column profile of HEA200 and a purlin profile of UPE100. This is the same combination that results from the optimization as can be seen in table 5.2. In figure 5.4 it can also be seen that the lightest option is not the cheapest option.

	Beam profile	Column profile	Purlin profile	UC Beam	UC Column	UC Purlin
1	IPE300	HEA180	UPE120	0.45	0.88	0.93
2	IPE330	HEA180	UPE100	0.40	0.88	0.95
3	IPE330	HEA180	UPE120	0.40	0.87	0.81
4	IPE360	HEA180	UPE100	0.40	0.87	0.94
5	IPE360	HEA180	UPE120	0.38	0.86	0.81
6	IPE300	HEA200	UPE100	0.42	0.72	0.97
7	IPE300	HEA200	UPE120	0.42	0.72	0.83
8	IPE330	HEA200	UPE100	0.38	0.71	0.90
9	IPE330	HEA200	UPE120	0.38	0.71	0.77
10	IPE360	HEA200	UPE100	0.36	0.71	0.89
11	IPE360	HEA200	UPE120	0.35	0.70	0.76
12	IPE300	HEA220	UPE100	0.40	0.60	0.93
13	IPE300	HEA220	UPE120	0.40	0.60	0.83
14	IPE330	HEA220	UPE100	0.36	0.59	0.85

Table 5.1: Profile combinations

Figure 5.4: Total cost factor for different profile combinations

General		Frame		Connection	
Stiffness [kNm/rad]	23302	**Beam**	IPE300	**Shortplate**	False
Bending moment Resistance [kNm]	60	**Column**	HEA200	**Shortplate factor**	1
UC columns	0.72	**Purlin**	UPE100	**Extension length [mm]**	8
UC beams	0.42	**Brace**	[-]	**Extension top length [mm]**	59.5
UC purlins	0.97	**Number of stories**	1	**Haunch**	False
UC braces	0	**Number of columns**	2	**Haunch Stiffeners**	False
UC welds	0.64	**Number of frames**	9	**Stiffeners**	Both
UC bolts	0.37	**Number of purlins**	7	**Haunch height [mm]**	0
Plate stress [MPa]	308	**Covered spans x-dir**	[-]	**Number of bolt rows**	2
Plate strain [%]	0	**Covered spans y-dir**	[-]	**Number of extra bolt rows**	0
Cost factor frame	16.4	**Brace location x-dir**	[-]	**Number of top bolt rows**	1
Cost factor beam-column connection	3.59	**Brace location y-dir**	[-]	**Number of bolt columns**	2
Cost factor total	23.02	**x-shape x-dir**	False	**Plate thickness [mm]**	8
		x-shape y-dir	False	**Bolt**	M16:8.8
		Beam column connection	Med-semi rigid	**Weld size end-plate/ beam web [mm]**	6
		Column base connection	Hinged	**Weld size end-plate/ beam flange [mm]**	8
				Weld size stiffener/ column web [mm]	6
				Weld size stiffener/ beam web [mm]	0
				Weld size stiffener/ column flange [mm]	8
				Weld size haunch web/ column [mm]	0
				Weld size haunch web/ haunch flange [mm]	0
				Weld size haunch web/ beam [mm]	0
				Weld size haunch flange/ beam [mm]	0
				Weld size haunch flange/ column [mm]	0

Table 5.2: Optimized structure results

5.3.2. CASE STUDY RESULTS

The results of the case study are presented below. The results are presented in 3D-graphs. All the result graphs are given in appendix I.

The results are divided into 2 types of graphs, the type of graphs are explained below. Both type of graphs are created for two cases. The first case is a hall with a hinged column-base connection. The second case is a hall with a rigid column-base connection.

In the first type of graph the number of purlins is fixed, the number of frames is varied on the x-axis and the beam-column connection type is varied on the y-axis. On the z-axis the normalized costs are presented. The normalized costs are normalized by dividing the costs of the combination by the lowest costs in the graph. An example of this graph can be found in figure 5.5.

In the second type of graph the type of beam-column connection is fixed, the number of frames is varied on the x-axis and the number of purlins is varied on the y-axis. On the z-axis the cost factor is shown. The cost factor is factorized by dividing the cost of the combination by the cheapest costs of all the 5 connection types. An example of this graph can be found in figure 5.8

Of both type of graphs there are three versions. In the first one the normalized costs of the profiles are presented, see figures 5.5 and 5.8. In the second version of the graph the normalized costs of the connections are presented, see figures 5.6 and 5.9. This is the normalized cost for the summation of all the beam-column connections. In the last version the normalized total costs are presented, see figures 5.7 and 5.10. In all graphs is the combination with the lowest normalized cost of that graph presented in orange.

FIXED NUMBER OF PURLINS

Below the three graphs of type 1 are presented. All these graphs are for the hinged column-base connection and 5 purlins. In these graphs it can be seen that the most cost-efficient profile costs will be for the medium semi-rigid, high semi-rigid or rigid beam-column connection with 7 frames. The most cost-efficient connections are for the low semi-rigid beam-column connection case with the 8 frames. The total costs of the structure is the most cost-efficient for the medium semi-rigid beam-column connection with 7 frames.

In figure 5.5, the normalized costs of the profiles are presented for a structure with 5 purlins. The number of frames is variable and is plotted on the x-axis, the type of beam-column connection is variable and is plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective option is presented in orange. It can be seen that the most cost effective option is for a medium semi-rigid, high semi-rigid or rigid beam-column connection. According to theory the rigid beam-column connection should result in the most cost-efficient option, so the results are according to theory. The stresses in the beam reduce when a rigid beam-column connection is used instead of a semi-rigid or hinged beam-column connection, which means that smaller profiles can be used. In the graph it is also shown that the most cost-effective option is for the case with 7 frames. This is because if more frames are used, the profile sizes can be decreased. At a certain point the extra material added plays a bigger part compared to the decrease in profile sizes.

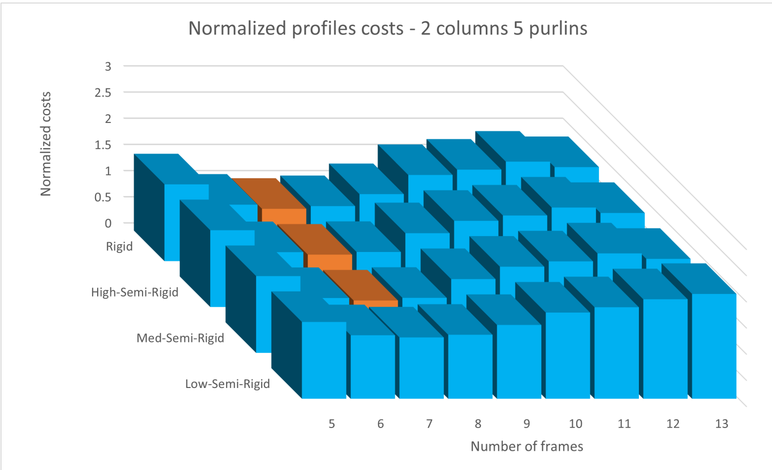


Figure 5.5: Normalized profile costs for 5 purlins

In figure 5.6, the normalized costs of the connections are presented for a structure with 5 purlins. The number of frames is variable and is plotted on the x-axis, the type of beam-column connection is variable and is plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective options are presented in orange. It can be seen that the most cost effective option is for a Low semi-rigid beam-column connection this is as expected according to theory. The stresses in the connection are lowest, when a Low semi-rigid beam-column connection is used. Smaller stresses in the connection means that there is less material needed which makes the connection cheaper. In the graph it is also shown that the most cost-effective option is for the case with 8 frames. If the beam and column profiles are the same, the beam-column connection is the same. A jump in the normalized connection costs means that a different combination of beam and column profiles are used. A difference between the 5 and 6 frames with a rigid beam-column connection is an increase in the beam profile and a reduction in the column-profile, due to the different load distribution. This different combination of beam and column profile cause a more expensive rigid beam-column connection.

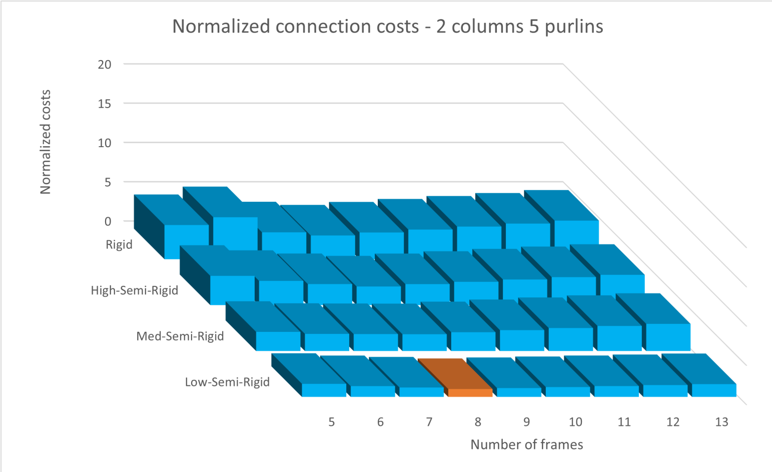


Figure 5.6: Normalized connection costs for 5 purlins

In figure 5.7, the normalized total costs are presented for a structure with 5 purlins. The number of frames is variable and is plotted on the x-axis, the type of beam-column connection is variable and is plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective option is presented in orange. It can be seen that the most cost-effective option is for a medium semi-rigid beam-column connection, this is as expected according to theory. Because this result is different then the graph in figure 5.5, the influence of the connections in the cost calculation can be seen.

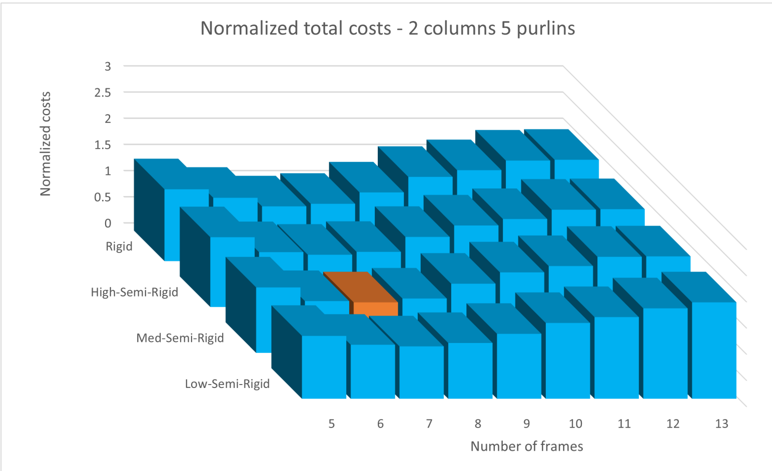


Figure 5.7: Normalized total costs for 5 purlins

FIXED NUMBER OF BEAM-COLUMN CONNECTION

Below the three graphs of type 2 are presented. All these graphs are for the hinged column-base connection and the medium semi-rigid beam-column connection. In these graphs it can be seen that the most cost-efficient profile costs will be for a combination of 5 purlins and 7 frames. The most cost-efficient connections are for the case with 7 frames and 5, 7 or 9 purlins. The most cost-efficient total costs are for the same combination as the cheapest frame.

In figure 5.8, the normalized costs of the profiles are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of purlins is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective option is presented in orange. It can be seen that the most cost-effective option is for 7 frames and 5 purlins. According to theory it is expected that more frames and purlins can give a more cost-effective option. This is because if the number of frames and purlins are increased the profile sizes can be decreased. This means that it is possible to get a lower price of the profiles while there are more profiles. It is expected however according to theory that this optimized option is with more frames and purlins.

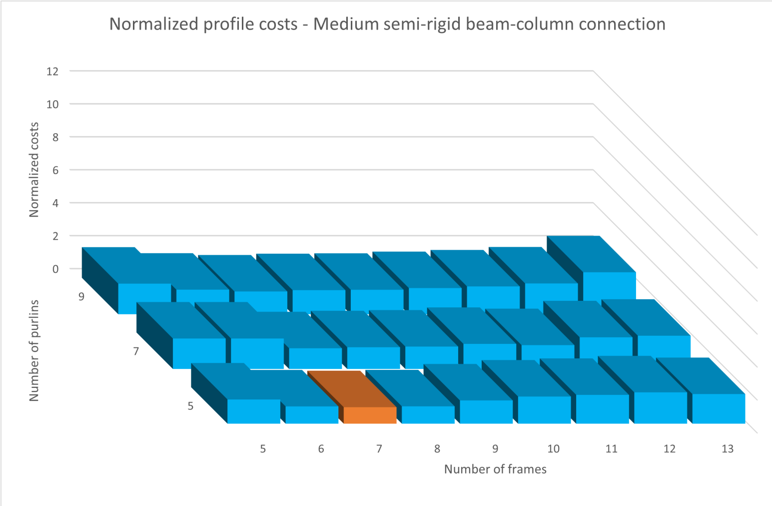


Figure 5.8: Normalized profile costs for medium semi-rigid beam-column connection

In figure 5.9, the normalized costs of the connections are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of purlins is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective options are presented in orange. It can be seen that the most cost effective options are for 7 frames and 5, 7 or 9 purlins. According to theory it is expected that the biggest number of frames would result in the most cost-efficient beam-column connection. But because in this graph the costs of all the beam-column connections are presented, the more frames the more connections, the bigger the total connection costs. This is why in this graph the most cost effective connection is not for the most amount of frames. A jump in the normalized connection costs means that a different combination of beam and column profiles are used.

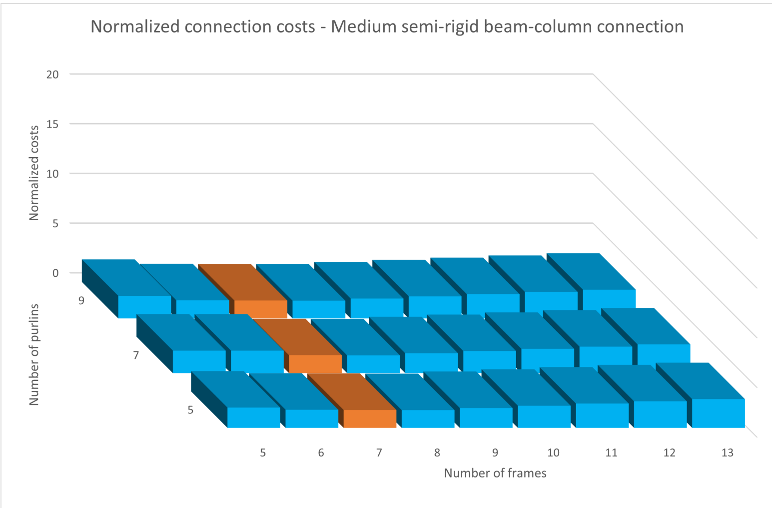


Figure 5.9: Normalized connection costs for medium semi-rigid beam-column connection

In figure 5.10, the normalized total costs are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of purlins is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective options are presented in orange. It can be seen that the most cost effective option is for 7 frames and 5 purlins. According to theory it is expected that more frames and purlins can give a more cost-effective option. This is because if the number of frames and purlins are increased the profile sizes can be decreased, meaning that also the connection costs can be decreased. This means that it is possible to get a lower total costs of the structure while there are more profiles. It is expected however according to theory that this optimized option is with more frames and purlins.

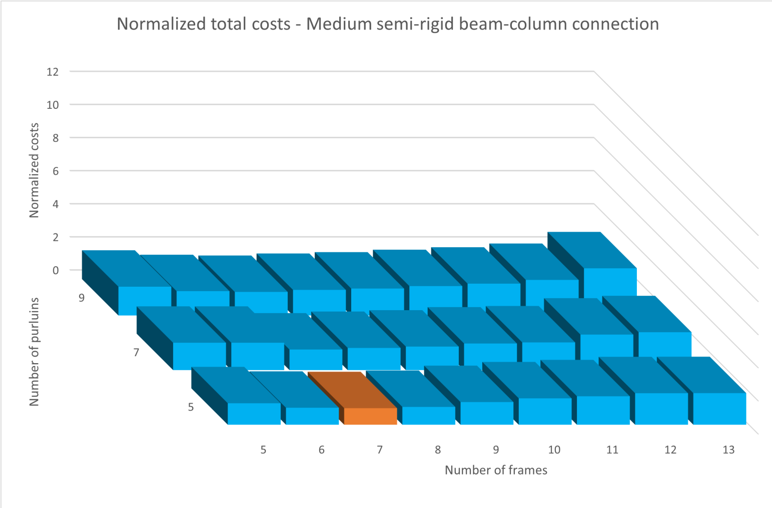


Figure 5.10: Normalized total costs for medium semi-rigid beam-column connection

In figure 5.11 the cost distribution can be seen for the optimized structure of 7 frames, 5 purlins and a medium semi-rigid connection.

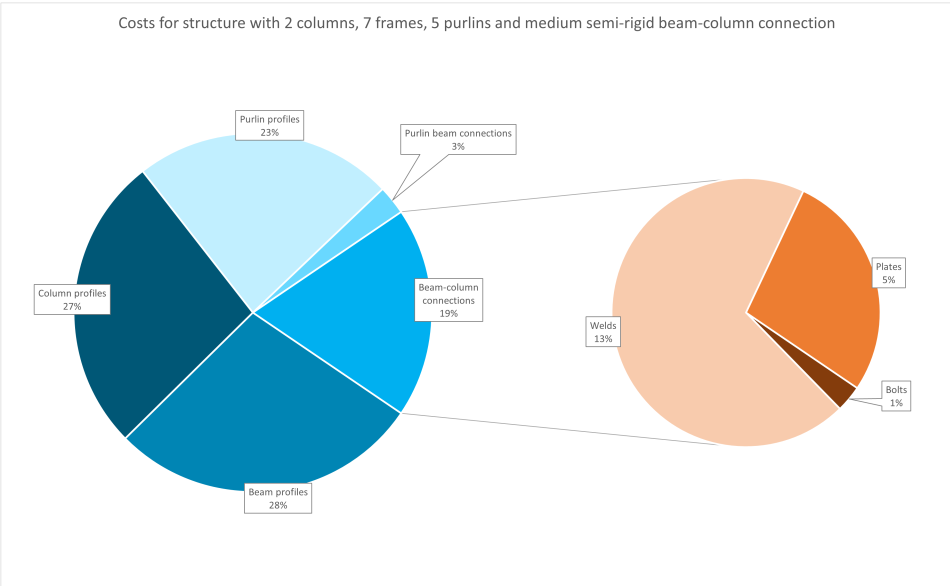


Figure 5.11: Cost distribution

5.3.3. ADDITIONAL CASES

In addition to the cases described above and in appendix I there are 2 extra cases checked.

ADDITIONAL CASE 1

The first case is for the medium-semi rigid beam-column connection and the rigid column-base connection with the same topology as the main case-study and twice the loads on the structure. This is done to increase the influence of the number of frames in the structure. The results for this case can be found in the figures below. In these figures it can be seen that a bigger number of frames and purlins can create a more cost-effective option.

In figure 5.12, the normalized costs of the profiles are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of purlins is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective option is presented in orange. It can be seen that the most cost effective option is for 10 frames and 9 purlins. According to theory it is expected that more frames and purlins can give a more cost-effective option. This is because if the number of frames and purlins are increased the profile sizes can be decreased. This means that it is possible to get a lower price of the profiles while there are more profiles.

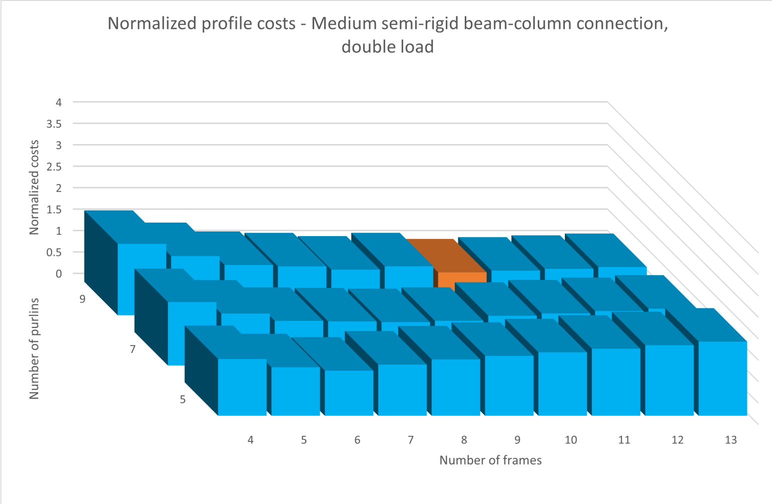


Figure 5.12: Additional case 1 profiles costs

In figure 5.13, the normalized costs of the connections are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of purlins is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective options are presented in orange. It can be seen that the most cost effective options are for 4 frames and any number of purlins. According to theory it is expected that the biggest number of frames would result in the most cost-efficient beam-column connection. But because in this graph the costs of all the beam-column connections are presented, the more frames the more connections, the bigger the total connection costs. This is why in this graph the most cost effective connection is for the least amount of frames. The number of purlins change the load distribution in the structure, which can change the beam and column profiles. In the case of 4 frames all the beam and column profiles are the same and the only difference is in the purlin profile. This is why the connection costs are all the same. A jump in the normalized connection costs means that a different combination of beam and column profiles are used.

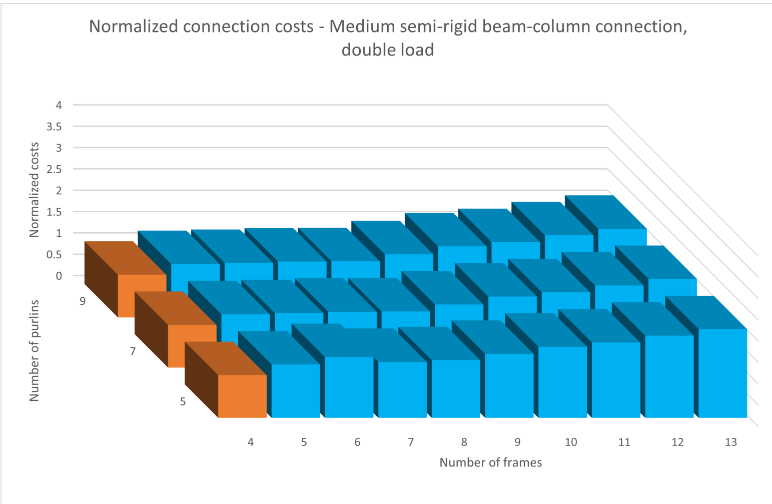


Figure 5.13: Additional case 1 connection costs

In figure 5.14, the normalized total costs are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of purlins is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective option is presented in orange. It can be seen that the most cost effective option is for 8 frames and 7 purlins. According to theory it is expected that more frames and purlins can give a more cost-effective option. This is because if the number of frames and purlins are increased the profile sizes can be decreased, meaning that also the connection costs can be decreased. This means that it is possible to get a lower total costs of the structure while there are more profiles. As can be seen between the difference between figure 5.14 and figure 5.12 the connection costs play a significant role in the total costs of the structure. Otherwise the graphs would have the same most cost-effective option.

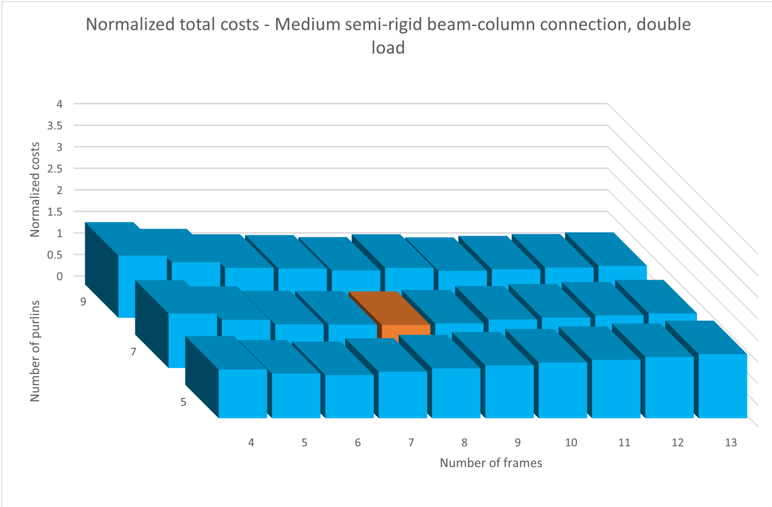


Figure 5.14: Additional case 1 total costs

In figure 5.15 the cost distribution can be seen for the optimized structure of 8 frames, 7 purlins and a medium semi-rigid connection.

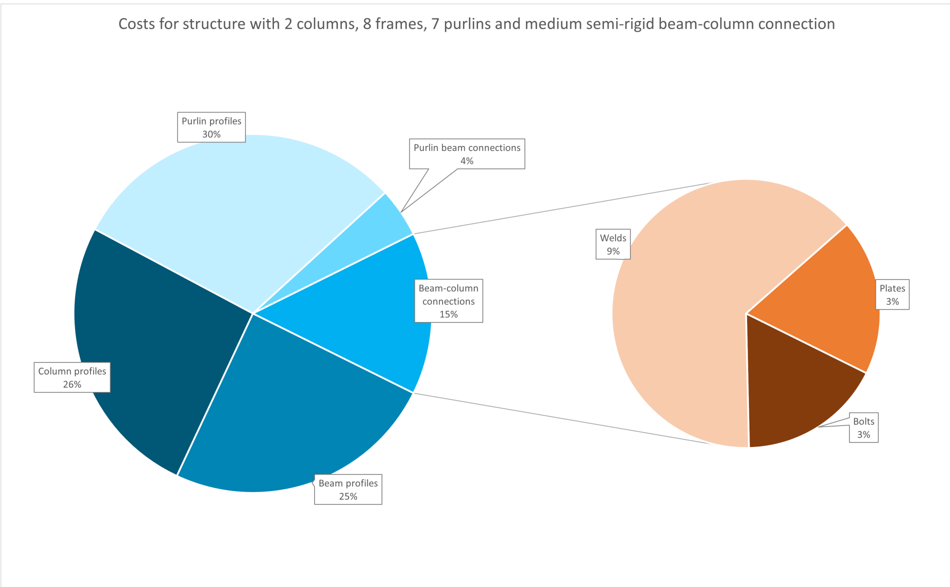


Figure 5.15: Cost distribution additional case 1

ADDITIONAL CASE 2

The second case is for a structure with a length of 100m a width of 16m and a height of 5m. The structure will have 8 purlins and the number of frames will vary between 17, 21 and 26. The number of columns per frame will vary between 2 and 3. The beam-column connection is medium semi-rigid and the column-base connection is hinged. This is done to check if the tool works for bigger halls and to check the influence of the number of columns per frame. The results for this case can be found in the figures below. In these figures it can be seen that a bigger number of frames and columns can create a more cost-effective option.

In figure 5.16, the normalized costs of the profiles are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of columns per frame is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective option is presented in orange. It can be seen that the most cost effective option is for 21 frames and 3 columns per frame. According to theory it is expected that more frames and columns can give a more cost-effective option. This is because if the number of frames and columns are increased the profile sizes can be decreased. This means that it is possible to get a lower price of the profiles while there are more profiles.

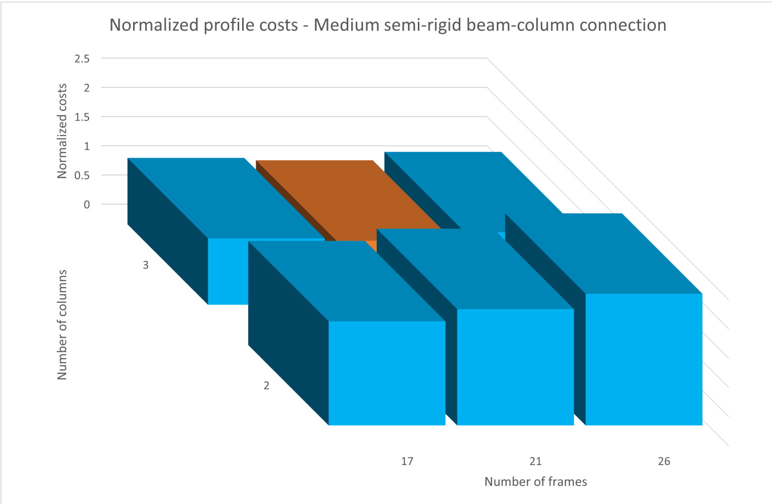


Figure 5.16: Additional case 2 profiles costs

In figure 5.17, the normalized costs of the connections are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of columns per frame is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective options are presented in orange. It can be seen that the most cost effective options are for 17 frames and 2 columns per frame. According to theory it is expected that the biggest number of frames would result in the most cost-efficient beam-column connection. But because in this graph the costs of all the beam-column connections are presented, the more frames the more connections, the bigger the total connection costs. This is why in this graph the most cost effective connection is for the least amount of frames.

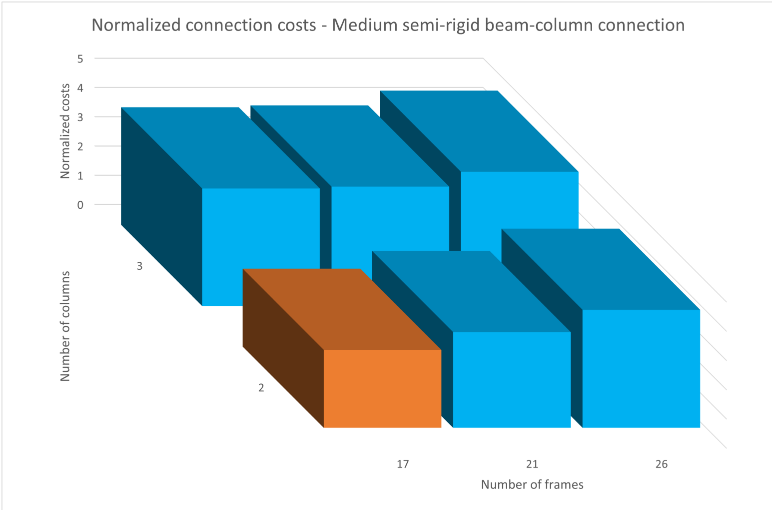


Figure 5.17: Additional case 2 connection costs

In figure 5.18, the normalized total costs are presented for a structure with a medium semi-rigid beam-column connection. The number of frames is variable and is plotted on the x-axis, the number of columns per frame is variable and plotted on the y-axis. On the z-axis are the normalized costs plotted. The most cost-effective option is presented in orange. It can be seen that the most cost effective option is for 17 frames and 3 columns per frame. According to theory it is expected that more frames and columns can give a more cost-effective option. This is because if the number of frames and columns are increased the profile sizes can be decreased, meaning that also the connection costs can be decreased. This means that it is possible to get a lower total costs of the structure while there are more profiles. As can be seen between the difference between figure 5.18 and figure 5.16 the connection costs play a significant role in the total costs of the structure. Otherwise the graphs would have the same most cost-effective option.

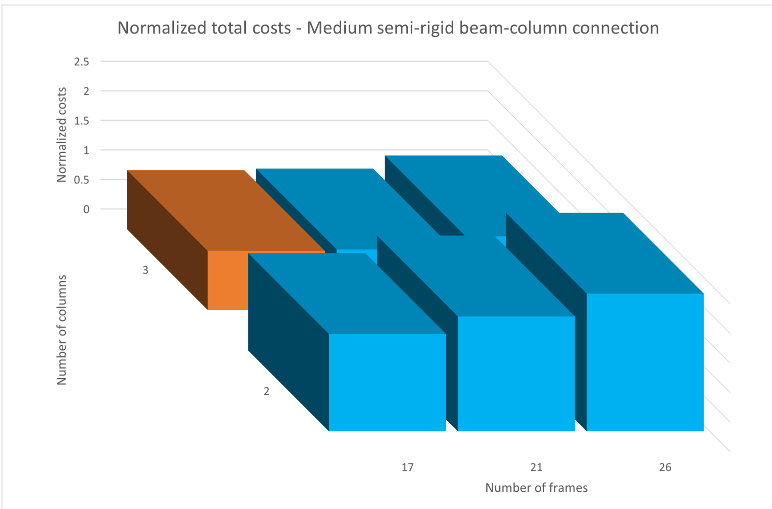


Figure 5.18: Additional case 2 total costs

In figure 5.19 the cost distribution can be seen for the optimized structure of 17 frames, 3 columns and a medium semi-rigid connection.

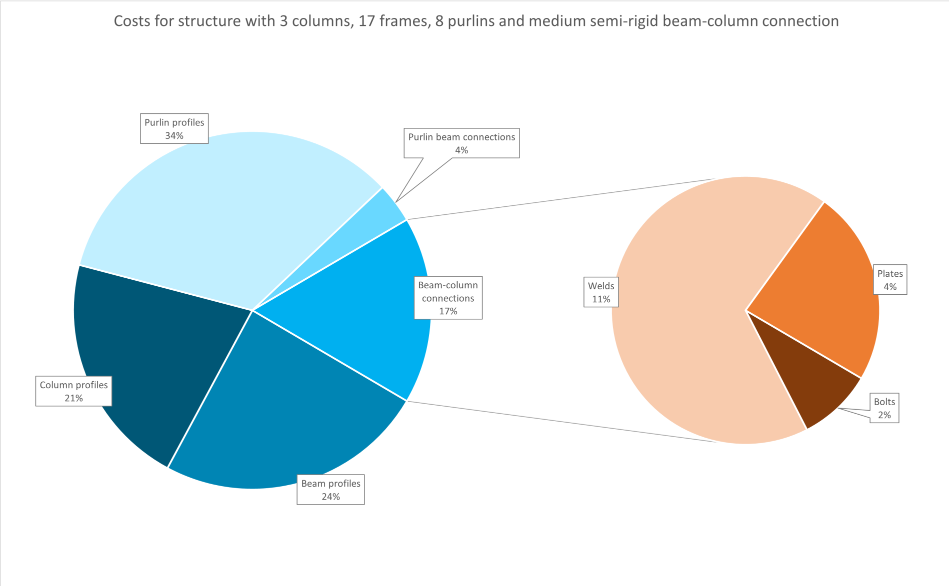


Figure 5.19: Cost distribution additional case 2

5.4. DISCUSSION

In this section the results and the link between the theory and the results are discussed. This starts with a discussion of the results that can be found in appendix I. At the end of this section, a recap is given about what is expected according to theory and then this is linked to the results.

5.4.1. RESULTS

The results in appendix I are of the main case study and are divided in the following four parts:

- Hinged column-base connection and a fixed number of purlins
- Hinged column-base connection and a fixed type of beam-column connection
- Rigid column-base connection and a fixed number of purlins
- Rigid column-base connection and a fixed type of beam-column connection

In the case of a hinged column-base connection it can be seen that in almost all results the semi-rigid option gives the most cost-efficient total costs of the structure, see figures I.3, I.6 and I.9. It is found in the results of the case study that more frames and purlins then the least amount of purlins and frames results in the most cost-efficient structure in spite of the beam-column connection type that is selected, see figures I.12, I.15, I.18 and I.21. In the case that there is looked at the normalized costs of the profiles it can be seen that the most cost-efficient option is not always the lightest option, see figures I.10, I.13, I.16 and I.19. The most cost-efficient option is the combination with more then 5 frames. In the case that there is looked at the normalized connection costs it can be seen that in almost all results the most cost-efficient connection is for more than 5 frames, see figures I.11, I.14, I.17 and I.20. Because there is a difference in the most cost-efficient option for the frames, the connections and the total costs. It can be seen that the connections influence the total costs of a structure.

It can also be seen in the hinged column-base connection results that the least amount of purlins is the most cost-efficient option. This applies to the frame costs, connection costs and total costs, see figure I.10 to figure I.21.

For the cases with a rigid column-base connection, it can be found that in almost all results the semi-rigid beam-column connection is the most cost-efficient option for the total costs of the structure, see figures I.24, I.27 and I.30. It is also found in the results of the case study, in contrast to the hinged column-base connection results, that the option with the least amount of purlins and frames does not result in the most cost-efficient structure in spite of the beam-column connection type that is selected, see figures I.33, I.36, I.39, I.42 and I.45. For example, the low semi-rigid beam column connection gives the most cost-efficient combination with 5 frames and 7 purlins. The lightest option is with 4 frames and 5 purlins. Meaning that compared to the hinged column-base connection results, the rigid column-base connections most cost-efficient total costs do not always result is the lightest option. In the case that there is looked at the normalized costs of the profiles it can be seen that the most cost-efficient option is never the lightest option, see figures I.31, I.34, I.37, I.40 and I.43. In all the results is the most cost-efficient option the case with 5 frames and at least 5 purlins. In the case that there is looked at the normalized connection costs it can be seen that in almost all results the most cost-efficient connection costs is not for the least amount of frames and purlins, see figures I.32, I.35, I.38, I.41 and I.44. Because there is a difference in the most cost-efficient option for the frames, the connections and the total costs. It can be seen that the connections influence the total costs of a structure.

5.4.2. COMPARISON WITH THEORY

According to theory the results need to show that:

- The most cost-effective option is not always the lightest option
- The semi-rigid case should give the most cost-efficient total costs of the structure
- The profile costs should be most cost-efficient for the rigid beam-column connection
- The connection costs should be most cost-efficient for the hinged beam-column connection with the most frames

THE MOST COST-EFFECTIVE OPTION IS NOT ALWAYS THE LIGHTEST OPTION

In the results presented in this thesis it can be seen that the lightest option (least amount of frames and purlins) will not always be the cheapest option. To get more clear results a case is ran with loads that are twice as big, see the results in figure 5.14. In this figure it is seen that the most cost-efficient option has 8 frames and 7 purlins while the lightest option has 4 frames and 5 purlins. This means that this case study shows that the connections influence the costs of a structure, even in a simple structure like a steel hall. The bigger the structure and the more connections a structure has the more influence the connections will have on the costs.

THE SEMI-RIGID CASE SHOULD GIVE THE MOST COST-EFFICIENT TOTAL COSTS OF THE STRUCTURE

Another thing that can be concluded from the results of the case study, is that the semi-rigid options are for all the results more cost-efficient than the rigid or hinged beam-column connections. This can be found for both a hinged or rigid column-base connection. From these results no conclusion can be made about whether a low, medium or high semi-rigid connection is most optimal. It can be seen that for the rigid column-base connection the connection costs are most cost-efficient for the hinged beam-column connection. For the hinged column-base connection the low semi-rigid beam-column connection is most cost-efficient.

THE PROFILE COSTS SHOULD BE MOST COST-EFFICIENT FOR THE RIGID BEAM-COLUMN CONNECTION

It can also be seen that the profile costs are most cost-efficient for the high semi-rigid options for the rigid column-base connection and either medium semi-rigid, high semi-rigid or rigid for the hinged column-base connection. The result of the hinged column-base connection is as expected. But the results of the rigid column-base connection are not as expected, the expected most cost-efficient beam-column type would be the rigid beam-column connection. For the rigid column-base connection case the difference between the profiles for the most cost-efficient (high-semi rigid) beam-column connection and the rigid beam-column connection can be found in table 5.3.

	Beam	Column	Purlin
High-semi rigid	IPE300	HEA200	UPE240
Rigid	IPE300	HEB160	UPE270

Table 5.3: Profiles semi-rigid vs rigid beam-column connection

Due to a different force distribution the optimization tool needed to increase the purlin to get an allowable design ratio below 1.0 which causes the costs of the frame to increase. The column is decreased, but because the length of the purlins is larger than the length of the columns, they have a bigger influence on the cost calculation. Which means that the total costs of the frame is increased.

For the hinged column-base connection case the difference between the profiles for the cheapest (high-semi rigid) beam-column connection and the rigid beam-column connection can be found in table 5.4.

	Beam	Column	Purlin
High-semi rigid	IPE330	HEA180	UPE300
Rigid	IPE330	HEA200	UPE300

Table 5.4: Profiles semi-rigid vs rigid beam-column connection

Due to the bigger bending moment in the connection. The column needs to take a bigger bending moment, which increases the column profile. Because there is only an increase in the column profile and no decrease in one of the other profiles the costs of the frame are increased.

THE CONNECTION COSTS SHOULD BE MOST COST-EFFICIENT FOR THE HINGED BEAM-COLUMN CONNECTION WITH THE MOST FRAMES

With the case study results in appendix I it can be seen that the normalized connection costs are most cost-efficient for the hinged beam-column connections for the rigid column-base connection, and the low semi-rigid beam-column connection for the hinged column-base connection. In both cases the most cost-efficient option in the graphs is not the option with the most amount of frames, but this is because in the graphs the normalized costs are presented as a sum of all the beam-column connections in the structure, see figures I.11, I.14, I.17, I.20, I.32, I.35, I.38, I.41 and I.44. In figure 5.20 a graph is shown for a hinged column-base connection and a low semi-rigid beam-column connection where the connection costs are the costs for only 1 beam-column connection. In this figure it can be seen that the most cost-efficient connection is the connection with the biggest number of frames.

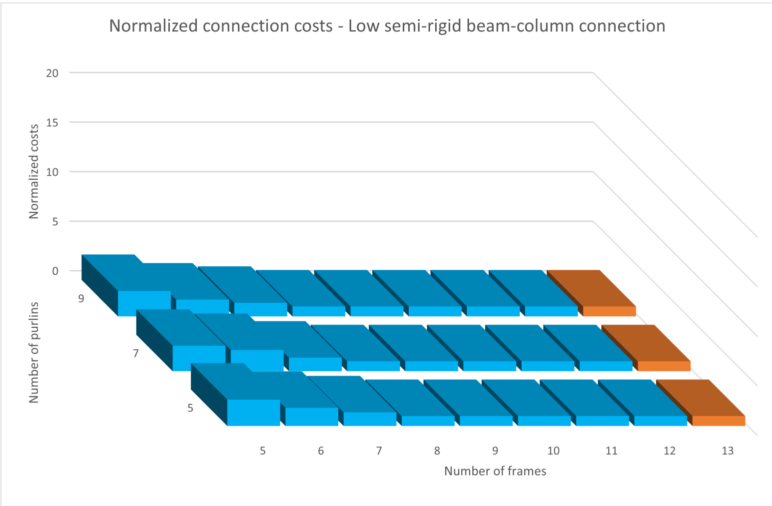


Figure 5.20: Hinged connection costs

6

CONCLUSIONS AND RECOMMENDATIONS

In this final chapter conclusions about the optimization process and tool are given, as well as the obtained results. Furthermore, the sub-questions and main research question are answered and at the end recommendations are given regarding improvements and further research.

6.1. SUB-QUESTIONS

In this section a concise answer is given to the sub-questions that can be found in section 1.2.

6.1.1. WHAT IS KNOWLEDGE-BASED ENGINEERING AND WHAT ARE THE ADVANTAGES AND DISADVANTAGES?

Knowledge based engineering is to capture knowledge of engineers in engineering and design rules. Knowledge based engineering software allows the user to capture all of these rules in the software package. The advantages of KBE are:

- Reduced lead time
- Product optimization
- Extra time for innovation

The disadvantages of KBE are:

- Building a KBE system can take a lot of time, skills and cost
- KBE software can become a black box

6.1.2. WHAT METHODS EXIST TO OPTIMIZE STEEL STRUCTURES TO COST?

The methods that could be found in literature that exists to optimize steel structures to costs are:

- Weight optimization with a cost factor
- Cost optimization with a fixed connection design
- Cost optimization with welded connections

6.1.3. WHAT COST MODEL CAN BE USED TO CALCULATE THE COST OF STEEL STRUCTURES?

The available cost models to calculate the cost of steel structures can be found in appendix A and are:

- Weight concept
- Weight concept including rotational capacity of joint
- Weight concept including welds
- Activity based costs
- Reverse engineering

6.1.4. What are the design steps made in the standard design of a steel hall and what aspects have the most influence on the strength and stiffness of the structure?

The design steps that are made in the standard design of a steel hall are:

1. Location of the hall (this determines the loads)
2. Main dimensions need to be determined
3. Type of column base connection
4. The type of roof that will be used
5. Topology needed
6. Type of column base connection that will be used
7. Profiles used
8. Connection design

The design aspects that have the most influence on the strength and the stiffness of the structure after the location and main dimensions are selected are:

- The type of column base connection
- The type of beam column connection
- The topology of the structure
- The profile sizes

6.1.5. How should the optimization tool be composed?

The optimization tool should have input variables which create a parametric model. The inputs that determine the parametric model are:

- Topology
- Joint type
- Profile types
- Loads

This parametric model is then checked structurally, with the FEM loop. This loop optimizes the structural profiles. With the profiles and loads from the structural check the connection is pre-designed. This is done by using a database of connection variables. This starts the connection loop. This pre-designed connection is checked for strength and stiffness and if both are sufficient the results are stored and the overall loop is redone.

6.1.6. How does the optimization tool function and how does it differ compared to existing tools?

A detailed description on how the optimization tool functions can be found in section 3.1. The differences between the tool and methodology described above and other optimization tools and methodologies found in literature can be found in the table below.

	Used tool and methodology	Other tools and methodologies
Optimization part	Complete structure including bolted connections	Either connection, structure or complete structure including welded connections
Type of optimization	Cost optimization including connections	Weight optimization or cost optimization of part of structure
Connection design	Pre-design from a created database with multiple designs	One type of fixed connection design
Cost formula	Includes bolted connections	Only includes welded connections or uses factor for connections

6.1.7. WHAT VARIABLES ARE SELECTED TO BE PARAMETRIZED AND WHAT RESULTS ARE NEEDED TO MAKE A COST COMPARISON?

The variables that are selected to be parametrized are:

- The number of frames
- The number of purlins
- The type of beam-column connection
- The type of column-base connection
- The profile dimensions of the beam, column and purlin
- The braces location, shape and design.

The results needed to make a cost comparison are the design variables of both the frame and the connection and the normalized costs.

6.1.8. HOW IS THE CONNECTION DESIGN CONSIDERED IN THE CONCEPT DESIGN PHASE?

The connection design is considered in the concept design phase by creating a database with all types of connections. The assumed stiffness is then checked in this database and this returns a detailed pre-design of the connection. This is how the detailed connection design is considered in the concept design phase.

6.1.9. WHAT CONSTRAINTS ARE APPLIED IN THE CONNECTION DESIGN AND WHY THESE CONSTRAINTS?

The constraints that are applied in the connection design are:

- The connection has to be an end-plate connection
- The end-plate is in all cases the width of the column
- In the partial end-plate design, is the height of the end-plate 0.6 times the height of the beam
- The number of bolt rows is limited by the height of the end-plate
- The bolt rows are evenly distributed over the plate height if possible
- In the full end-plate design is the extension of the end plate above and below the beam always the thickness of the end-plate
- The haunch height is the same as the haunch width
- The haunch height is the same as the beam height
- The haunch flange thickness is the same as a standard plate thickness that comes closest and is bigger than the beam flange thickness
- The stiffener thickness is the same as a standard plate thickness that comes closest and is bigger than the beam flange thickness

The reason for why these constraints are chosen can be found in section 4.2

6.1.10. HOW ARE THE CONNECTIONS CONSIDERED IN THE OVERALL OPTIMIZATION?

In the overall optimization only the beam-column connections are considered as variable. These are designed after the structure is designed. It is assumed that all beam-column connections are the same.

6.1.11. WHAT PARAMETERS HAVE THE MOST INFLUENCE ON THE COSTS OF THE STRUCTURE?
The items that have a significant influence on the costs of the structure are:

- Number of frames
- Number of purlins
- Profile dimensions
- Connection type
- Beam-column connection

The variables in the connection design that have a significant influence on the costs of the connection are:

- Plate thickness
- Welds
- Haunch
- Stiffeners
- Number of bolts

6.1.12. WHAT ARE THE RESULTS OF THIS STUDY?
The results of this study can be found in section 6.3 below.

6.1.13. ARE THE RESULTS AS EXPECTED ACCORDING TO THEORY?
Yes the results are as expected according to theory. The only result that was not as expected was that not in all cases the rigid beam-column connection would create the most cost-efficient profile costs.

6.1.14. WHAT ARE FURTHER POSSIBILITIES OF THE METHODOLOGY USED IN THIS RESEARCH
The recommendations of this thesis can be found in section 6.4 below.

6.2. MAIN RESEARCH QUESTION
In this section the main research question will be answered. The main question is:

In what way can knowledge-based engineering software influence the cost optimization of steel halls with open sections and bolted end-plate beam-column connections?

From the case studies in this thesis it can be seen that the knowledge-based engineering software used in this research, makes it possible to include the connection design in detail in the concept phase of a project. Due to all the knowledge that can be included and the iterative process is done by a computer, it is possible to do a cost optimization that includes more factors in the cost calculation. Another benefit of this method is that the joints that are considered rigid can be checked to see if the connection is rigid enough, and the joints that are considered hinged in the global analysis model can be checked to see if the connections are not actual semi-rigid. This is because with knowledge-based engineering software the connection design can be coupled to the global analysis model.

A limitation of using a knowledge-based engineering approach in the optimization of steel halls is that beforehand, all the boundaries of the design need to be known. Because it can take a lot of time to optimize a structure if incorrect boundaries are given or it can skip important options. Another limitation of this knowledge-based approach is that it takes a lot of time to create this tool and apply this process, which means that it is not profitable to use for small projects that are created one time. Although, when the tool is already created it is easily adjustable to different projects.

All of this means is that the knowledge-based engineering software used in this thesis makes it possible to include the connection design in the concept phase of the design. Which makes it possible to do a cost optimization of a steel hall with open sections and bolted end-plate connections. It is however important to consider all the parameters that need to be included in the process from the start.

6.3. CONCLUSIONS

In this thesis a knowledge-based cost optimization model was created for steel halls with open sections and bolted end-plate connections. The optimization model was created in the programming language Python. The structure was modelled in the finite element software RFEM and the connection was modelled in the component based finite element software IDEA StatiCa. The optimization model was assessed by comparing the results of a case study with existing theories.

From the results of the case study the following conclusions can be drawn:

- The optimization process proves for the cases used in this research, that the most cost-effective option is not always the same as the lightest option.
- The optimization process proves for the cases used in this research, that the semi-rigid case most often gives the most cost-efficient result.
- The optimization process proves for the cases used in this research, that the costs of the profiles are most cost-efficient for the rigid or high semi-rigid beam-column connection.
- The optimization process proves for the cases used in this research, that the connection costs are most cost-efficient for the beam-column connection with the lowest stiffness and the most frames.
- With the knowledge-based engineering approach used in this research it is possible to move the detailed connection design to the concept design phase.
- With the knowledge-based engineering approach used in this research it is possible to include the correct stiffness of a connection in the concept design phase.
- With knowledge-based engineering software it is possible to create an automated design process.

6.4. RECOMMENDATIONS

For improvements and further research, the following recommendations are stated.

6.4.1. TOPOLOGY

To make a more diverse tool that can be used for multiple types of structures. The input topology values could be increased, to make structures possible where not all the frames or columns are equally distributed over the length or width. This is why it is recommended to adjust this in the tool, so more structures can be checked with this optimization process.

6.4.2. LOADS

To make a more diverse tool, the loads input could be changed to an input where the loads are not calculated according to the Eurocode. The loads can be created to have an input of the actual load values and the beam or node number. This creates the possibility to create all kinds of loads on a structure that are not in the Eurocode. This also creates the possibility to use the optimization tool for more types of structures. This is why it is recommended to adjust the tool in this way so all loading types can be added.

6.4.3. CONNECTION OPTIMIZATION

Because in the connection optimization only 4 connection designs are considered there is a limitation in the connection designs that can be offered by this optimization process. The connections are limited to bolted end-plate connections on rolled open cross sections. This means that the profiles used for the beams and columns are also limited to rolled open cross sections. This can be changed by adding more connections in the connection designs. To do so, more databases need to be created for different connection types. Creating these databases can take a week or more depending on the options and variables in considered. The databases only have to be created once. This makes it possible to add different types of connections, meaning that also structures with other connections can be optimized to their costs with this method. This also gives the opportunity to optimize a structure and all its connections, not only the beam-column connection. This is why it is recommended to create more databases for different connections, to make it possible that more structures can be checked with this optimization process.

6.4.4. FURTHER RECOMMENDATIONS

Further recommendations for this study would be to optimize more types of steel halls and check if similar results are found. An additional recommendation would be to check if the optimization process is optimized and if it can be improved. The last recommendation would be to do a study in the semi-rigid connections and see if a optimum semi-rigid connection can be found between the low, medium and high semi-rigid connection options.

A

Cost estimation methods

There are multiple cost models found in literature for steel frame structures, these differ in the complexity of the cost model. The cost models are used to calculate the costs of steel frame structures. These cost models consider different joint types from fully rigid to semi-rigid to hinged. From the literature it is found that often the most cost-efficient option when looked at a fixed frame topology is the semi-rigid joint [49] [93]. The cost models that are found are explained in more detail below from least complex to most complex.

A.1. Weight concept

Most of the time for steel structures the costs of a structure are calculated by the weight of the structure. The total weight of the structure is multiplied by a price per kilogram of steel. The additional costs like the joints and the fabrication and erection costs etc. are called the shadow costs. To consider these shadow costs an additional percentage is added per kg of steel [45] [46] see equation (A.1). This percentage depends on the type of connection and can be varied from 10% for simple joints to 50% for complex joints. The different prices for the structure can be found in table A.1. Assumptions for this structure are:

- Column profile: HEB240
- Beam profile: IPE400
- Purlin profile: UPE240
- Specific weight: 7.85 ton/m^3
- Price steel: 560 €/ton
- Length structure: 50 m
- Width structure: 11 m
- Height structure: 4 m
- Variable connections are the column beam connections

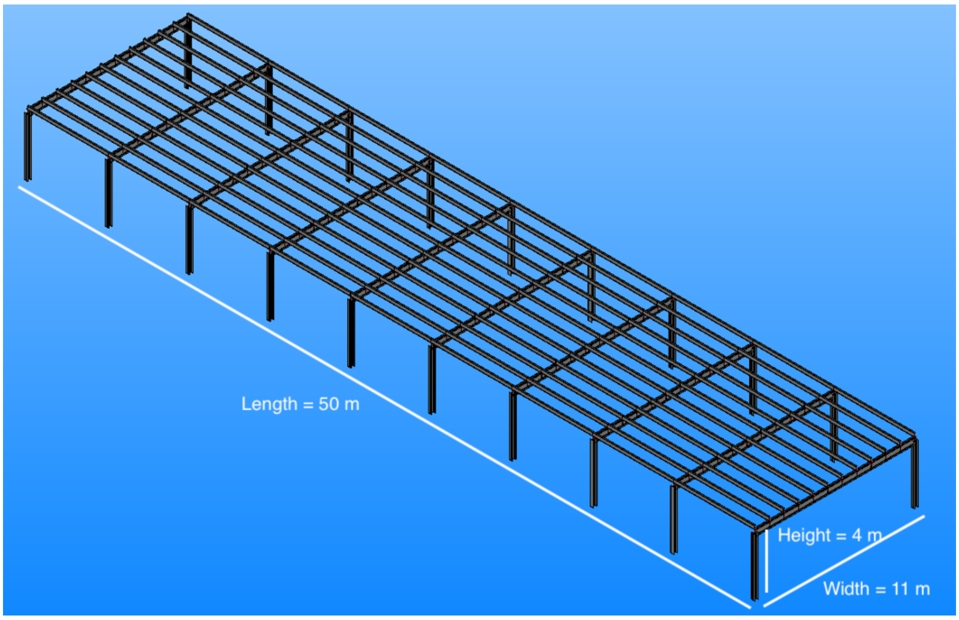


Figure A.1: Comparing cost estimation methods

Type of connection	Hinged	Fixed
Price [€]	19858	27079

Table A.1: Cost method 1 prices

$$€ = ton \cdot €_{ton,steel} \cdot (1 + \%_{additional}) \tag{A.1}$$

This formula is very easy to use especially in the concept phase of a design, because the only thing necessary for this formula is the main dimensions of the structural elements used and the additional percentage that depends on the shadow costs. The downside of this formula is that the more slender a structure is the lower the total costs of a structure are. While in practice this is not always the case. There are cases where due to slender profiles the joints need a lot of extra stiffeners or plates. This can become very expensive which leads to a higher total cost.

A.2. WEIGHT CONCEPT INCLUDING ROTATIONAL CAPACITY OF JOINT

In the second method a bit more complexity is considered by not only looking at the weight of the main material and give a percentage depending on the type of joint, but by including the rotational capacity of the joint. Two different methods are found in literature, one that is based on rotational stiffness and one that is based on a fixity factor. Both methods are based on bolted semi-rigid connections. The results of these formulas are both in kilograms instead of a price. The total weight is then determined by the weight of the structure including a weight depending on the rotational capacity of the joints. This total weight can be multiplied by a price per kilogram to give a price just like in method 1. The prices for this second method based on the fixity factor applied to the structure in figure A.1 can be found in table A.2.

Type of connection	Hinged	Fixed
Price [€]	21114	22748

Table A.2: Cost method 2 prices

A.2.1. ROTATIONAL STIFFNESS

The formula that uses the rotational stiffness of a joint is created by Xu and Grierson [47], and can be found in equation (A.2). This formula gives the total costs of a structure depending on the rotational stiffness. Equation (A.2) is used multiple in multiple studies to calculate costs of a structure. [48][49].

$$Z(x) = \sum_{i=1}^{nm} W_i A_i + \sum_{i=1}^{nbm} \sum_{j=1}^{2} \left(\beta_{ij} R_{ij} + \beta_{ij}^0\right) + \sum_{i=1}^{nco} \left(\beta_i R_i + \beta_i^0\right) \tag{A.2}$$

where:

nm			=	Total number of members in the system
nbm			=	Total number of beams
nco			=	Total number of columns
W_i	=	$\rho \cdot L$	=	Weight coefficient
A_i			=	Cross sectional area
$\beta_{ij}\&\beta_i$	=	$\frac{(0.225 W_i A_i)}{k_i}$	=	Connection cost coefficient
$\beta_{ij}^0\&\beta_i^0$	=	$0.125 W_i A_i$	=	Cost coefficient of pinned connection
$R_{ij}\&R_i$			=	Connection rotational stiffness
k_i			=	Estimated rotational stiffness

In this formula the estimated rotational stiffness k_i is constrained to be between $2.26 \cdot 10^2$ kNm/rad and $5.65 \cdot 10^5$ kNm/rad depending on the stiffness of a connection. The first part of this formula gives the weight of the structure. The second part describes the cost of the beam to column connection. The third part describes the cost of the column base connection. In the second and third part, the costs are calculated with a rotational stiffness and an estimated rotational stiffness. In this formula the actual rotational stiffness of the connection is divided through the estimated rotational stiffness. Meaning that a factor is included on how well your actual connection behaves as the estimated connection. If the actual rotational stiffness is higher this factor will be bigger than 1 and if the actual rotational stiffness is lower this factor will be between 0 and 1.

A.2.2. FIXITY FACTOR

"The fixity factor α defines the stiffness of the connection relative to the attached beam and is perhaps the most powerful and important concept for the analysis of frames with semi rigid joints. It relates quite closely to how the structure will behave in the context of the connection, far more so than the absolute value of S_j." [93] The fixity factor can be calculated with equation (A.3).

$$\alpha = \frac{1}{1 + 3EI/LS_j} \tag{A.3}$$

where:

S_j = Rotational stiffness

The fixity factor has a range from 0 to 1. A fixity factor of 0 means that it is a hinged connection and a fixity factor of 1 means that it is a fully rigid connection. Simões states: *"Published data suggests that the cost of a steel member with IPE section is increased by 20% if it has pin-jointed end-connections, and by 60% if its end-connections are bolted or welded."* [93] This is why the costs of a beam including connections should be between 1.2 and 1.6 times the costs of the beam. The formula that is used in this research to calculate the total cost of a member with two end-connections can be seen in equation (A.4).

$$Z_i = W_i A_i + \sum_{k=1,2} \left(V_{ik}^0 + V_{ik}^1 \alpha_{ik} V_{ik}^2 \alpha_{ik}^2\right) \tag{A.4}$$

where:

W_i = $\rho_i \cdot L_i$ = Weight coefficient

A_i = Cross sectional area

When the boundary conditions of the type of end-connections are considered equation (A.5) is found.

$$Z_i = W_i A_i + \sum_{k=1,2} \left(0.1 W_i A_i - 0.4 W_i A_i \alpha_{ik} + 0.6 W_i A_i \alpha_{ik}^2\right) \tag{A.5}$$

The first part in equation (A.5) describes the own weight of the beam and the rest of the formula gives the cost of the connection. It can be seen that if both connections are hinged $\alpha_{ik} = 0$ the total costs will be 1.2 times the cost and if both connections are fully rigid $\alpha_{ik} = 1$ the total costs will be 1.6 times the cost of the beam.

A.3. WEIGHT CONCEPT INCLUDING WELDS

This method determines the cost by using the weight of the structure and the volume of the welds. There are two different options found in literature when wanting to include welds to the total cost. The first one determines the weld volume in kg and adds this to the total weight [94]. Like method 1 and 2 the result is in kg, but this can be converted to a price by multiplying it by a price per kilogram of steel. The second option checks the number of weld beads, the length of the weld and the welding speed. With this a price per weld can be determined and added to the cost of the structure [95].

A.3.1. WELDING VOLUME

To include labour and anti-corrosion protection costs a factor f is included in the formula that is the cost ratio between 1 kg welding to 1 kg of steel. This ratio depends on the country of fabrication and construction, the automation grade of the company and the labour costs [94]. If this is applied for a connection with a haunch then a conversion constant can be determined that converts the height of the haunch to an estimate of the welding length [4]. Steenhuis et al. [4] assumes this conversion constant to be 4.3. This gives equations (A.6) to (A.8) for the total weight of a beam-column connection with a haunch and without any other stiffeners.

$$W_{material} = A \cdot L \cdot \rho \tag{A.6}$$

$$W_{weld,haunch} = 4.3 \cdot a^2 \cdot h_h \cdot f \tag{A.7}$$

$$W_{total} = W_{material} + W_{weld,haunch} \tag{A.8}$$

Where:

A	=	Cross section of beam or column
L	=	Length of beam or column
ρ	=	Density of beam or column
a	=	Throat of the weld
h_h	=	Height of the haunch
f	=	Cost ratio factor

In table A.3 a cost estimate is made for a rigid joint and a semi-rigid joint. In the table the semi-rigid joint possesses bigger beam cross section and a smaller haunch height. In this calculation it is assumed that the haunch is welded around with 12 mm fillet welds. The length of the haunch is assumed to be 1.5 time its height. This table shows that semi-rigid connections are for this case cheaper than fully rigid connections even if the column size increases. The rigid joint has a column profile of HEB340 and the semi-rigid option has a column profile of HEB400. It is assumed for this calculation that the column has a length of 3.5 m.

Cost category	Rigid joints	Semi-rigid joint	Savings when semi-rigid joint is used
Column material [kg] $A l_c \rho$	$171 \cdot 10^{-4} \cdot 3.5 \cdot 7850 = 470$	$198 \cdot 10^{-4} \cdot 3.5 \cdot 7850 = 544$	$-74kg$
Weld material haunch [kg] $a^2 \cdot 4.3 \cdot h_h \cdot \rho \cdot f$	$0.012^2 \cdot 4.3 \cdot 0.75 \cdot 7850 \cdot 100 = 364kg$	$0.012^2 \cdot 4.3 \cdot 0.35 \cdot 7850 \cdot 100 = 170kg$	$+194kg$
Total			$+120kg$

Table A.3: Cost comparison method 3.1

A.3.2. WELD BEADS

Welds that have a throat thickness bigger than 6 mm will have multiple weld beads. The literature states that the cost of a weld is calculated with equation (A.9) where a welding speed of 4 meter per hour per weld bead was assumed [95].

$$C_{total} = L \cdot n \cdot v \cdot R \tag{A.9}$$

Where:

L	=	Length of the weld
n	=	Number of bead layers
v	=	Welding speed
R	=	Hourly labour rate

This weld beads option is more specific than the weld volume option, which can lead to more realistic cost estimation. However, the welding speed depends very much on the experience of the welder, and the hourly rate of the labour depends a lot on the company and country where the welding is done. This means that this can get better results then the welding volume method, but it also needs more input to get a realistic estimate.

A.4. ACTIVITY BASED COSTS

The last method is the most complex one. This method has a lot of variables that need to be determined, but if all these variables can be determined this method can also give the most accurate cost estimation. Activity based costs differs from the other methods on the fact that this method is not based on products but on activities. This also means that only direct costs can be considered without adding factors for the indirect costs. When the profit margins are relatively low, the price is mostly determined by the direct cost. In the case of the building industry the profit margins are often low in case there is a lot of competition during tender offers [96]. This means that in the building industry for buildings that are often build, the activity based cost method gives a realistic estimation.

In literature big differences can be found in the number of activities that are taken into account. In some literature only a couple of stages are considered [97] while in other studies all the activities in these stages are considered separate [98].

A.4.1. MULTIPLE STAGES

In this method performed by S. Kravanja and T. Zula [97], the stages in the process considered are:

- Material cost
- Fabrication cost
- Erecting cost
- Painting cost

In this study the fabrication costs are assumed to be 40% of the material cost, this can be seen in equation (A.10).

$$Cost = V_{structure} \cdot \rho \cdot C_{mat} \cdot (1+0.4) + A_{structure} \cdot C_{paint} + n \cdot C_{erect} \tag{A.10}$$

Where:

$A_{structure}$	=	Surface area of the structure
$V_{structure}$	=	Volume of the structure
n	=	Number of structural elements
ρ	=	Density of the steel
C_{mat}	=	Price of structural steel per kg.
C_{paint}	=	Price of paint per m^2
C_{erect}	=	Erection price per element

A.4.2. SEPARATE ACTIVITIES PER STAGE

In this study performed by Flyvbjerg et al. [96] the cost are determined for all the activities separate and added together in the end. This gives the equation for the total cost as can be seen in equation (A.11).

$$C_{total} = C_{SM} + C_B + C_{CU} + C_{BW} + C_S + C_D + C_{CO} + C_{PF} + C_{PA} + C_{PT} + C_P + C_T + C_E \tag{A.11}$$

Where:

C_{SM}	=	Material cost
C_B	=	Blasting cost
C_{CU}	=	Cutting cost
C_{BW}	=	Beam welding cost
C_S	=	Sawing cost
C_D	=	Drilling cost
C_{CO}	=	Coping cost
C_{PF}	=	Part fabrication cost
C_{PA}	=	Part assembling cost
C_{PT}	=	Post treatment cost
C_P	=	Painting cost
C_T	=	Transporting cost
C_E	=	Erecting cost

The material cost includes the number of profiles, plates, bolts, nuts and washers. The direct cost of the blasting is the labour and the consumables which in this case is the steel shot. The direct cost of cutting is the labour and the cost of gasses, plasma electrodes and nozzles. The direct cost of beam welding is labour and welding material. The biggest cost influences on sawing, drilling and coping are the labour costs. The same yields for the fabrication and assembling of parts and the post treatment. For these activities the labour costs are more than 60% of the total cost. The cost of the painting is for more than 90% the paint [96]. The transporting and erecting again are mostly determined by the labour cost, because these activities take time.

A.5. Reverse engineering

Besides the methods discussed in appendix A an effective method to analyse the cost of a structure is by doing a project that is already done a lot of times and reverse engineer the costs. A problem with this is that most companies only have their own projects to look into, because this data is not publicly shared. Due to the differences in projects it is hard to find projects in the same company that are similar enough. Another problem that can arise when you can get the data from different companies is that it is unknown what the profit rate is on the project. This makes it impossible to know what part of the price the actual cost was and what part profits or losses were.

A.6. Conclusion

In this chapter multiple cost estimation methods are discussed. The methods differ most in the time that is required and the precision of the result. The more time a method takes the more precise the results will be.

Besides time you also need a high level of detail in the model to get an accurate cost estimation, this is for example for method 4 needed. Besides the direct costs used in the process also indirect costs can be taken into account. If it is needed to get a very precise cost estimation even the non-productive time can be considered in the indirect costs. If not all the details are known and general values are used, it has no use to do a very time-consuming cost estimation, because the result will be untrustworthy. In conclusion there is a balance in making a cost model which is to detailed and specific and cannot be used for multiple cases and making it too general and thus not precise. That is why in this thesis a simplified version of the activity based method is used where only the non-project specific activities are considered. Because in this thesis a comparison is important average values are used and the result will be factorized in the end.

B

LOADS IN RFEM

In this appendix the loads that work on the structure are presented.

The loads considered in this research are permanent and variable loads according to the Eurocode for single storey buildings.

B.1. PERMANENT LOADS

The permanent loads consists of two parts, the permanent load G and the permanent load P.

B.1.1. PERMANENT LOAD G

Load G is the self weight of the structure. This is variable depending on the structures topology and the profiles of the elements in the structure.

B.1.2. PERMANENT LOAD P

Load P is the load from the roof and insulation. A permanent load is programmed in the model with a variable input, which will represent the roof and insulation on the structure. In this model it is assumed that the purlins that are on the beams carry the roof and insulation. The c.t.c. distance between these purlins is variable. The load on the purlins varies with the number of purlins. An example of the permanent load on 9 purlins can be found in figure B.1

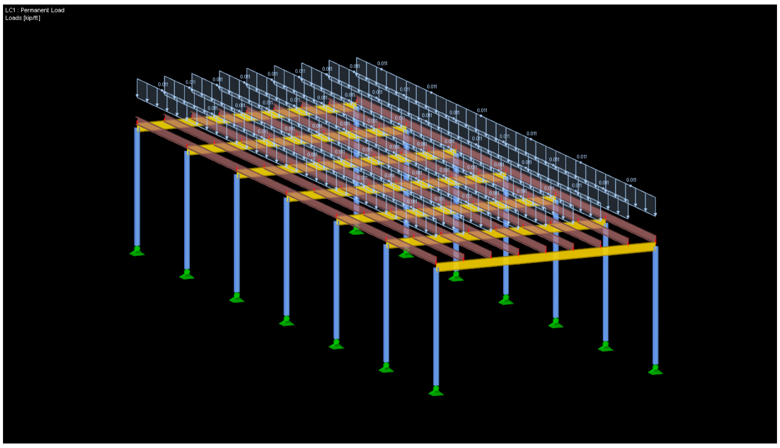


Figure B.1: Permanent load P

B.2. VARIABLE LOADS

The variable loads in this research are wind, snow and maintenance according to the Eurocode NEN-EN 1991-1-1 [77].

B.2.1. THE WIND LOAD

is determined according to the Eurocode NEN-EN 1991-1-4 [91] and NEN-EN 1991-1-4+A1+C2/NB [99].

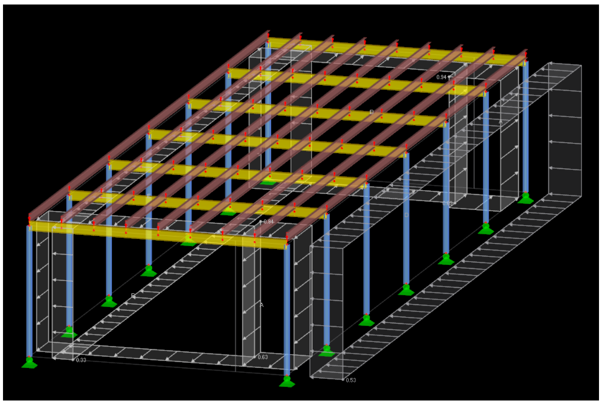


Figure B.2: Wind load x-dir horizontal

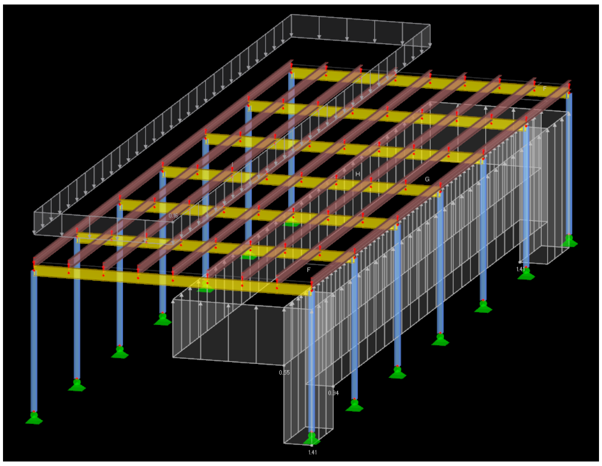


Figure B.3: Wind load x-dir downward vertical

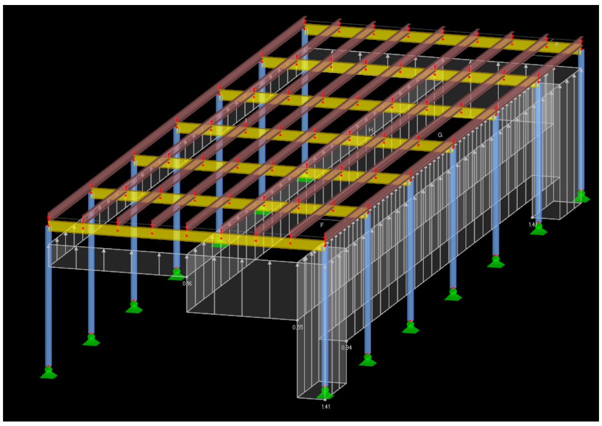


Figure B.4: Wind load x-dir uplift vertical

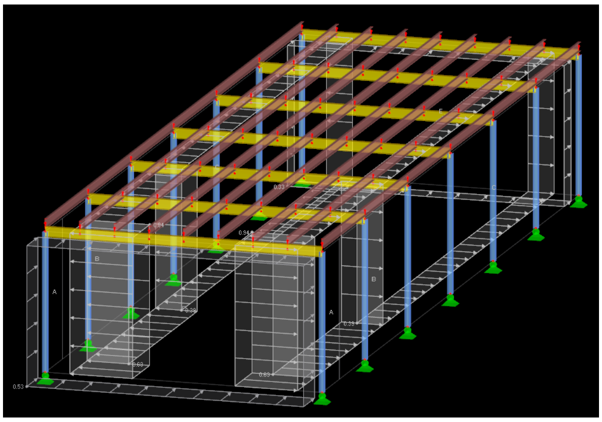


Figure B.5: Wind load y-dir horizontal

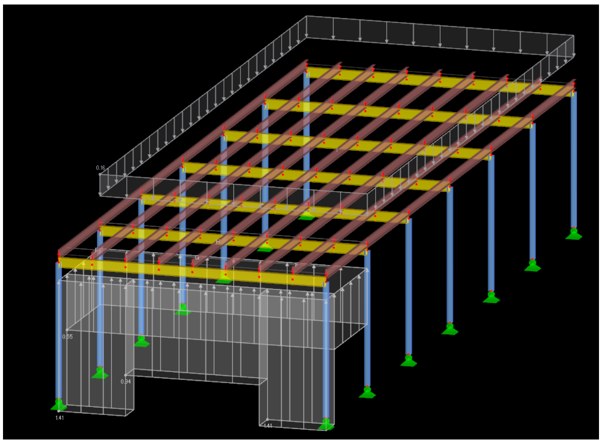


Figure B.6: Wind load y-dir downward vertical

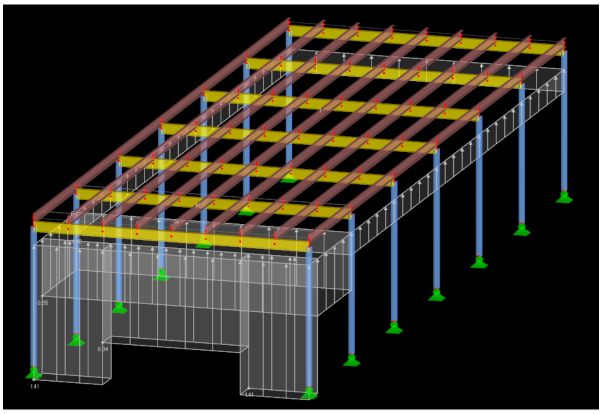


Figure B.7: Wind load y-dir uplift vertical

B.2.2. THE SNOW LOAD

depends on the environment, the zone number, the height above sea level, the roof angle and the temperature coefficient. The zone number can be determined with figures C2 to C10 from NEN-EN 1991-1-3 [92].

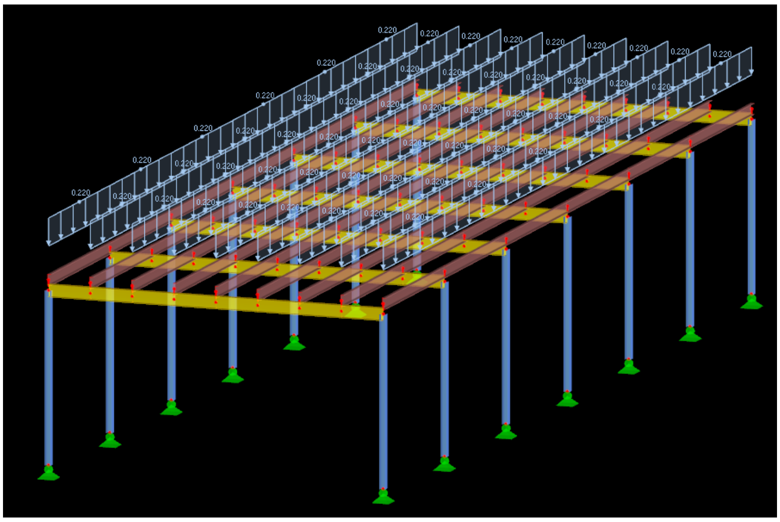


Figure B.8: Snow load

B.2.3. THE VARIABLE LOAD FOR MAINTENANCE

is a load of $1kN/m^2$ on 10 m^2 of the roof or a point load of 1.0 kN according to NEN-EN 1991-1-1 [77]. This load represents people and material standing on the roof to perform maintenance on the roof.

The most unfavourable location of the maintenance load is the middle of the purlins between two frames. Because in this optimization model the number of purlins and the number of frames is variable the location that gives the biggest bending moment changes per topology. It is assumed that the 10 m^2 is divided in a strip where the width is always the length between two purlins and the length is the full length of the structure. This gives in all cases an area bigger then 10 m^2 meaning that it is conservative. This conservative approach is used because it is impossible to program a load with the API in RFEM that is not over the full length.

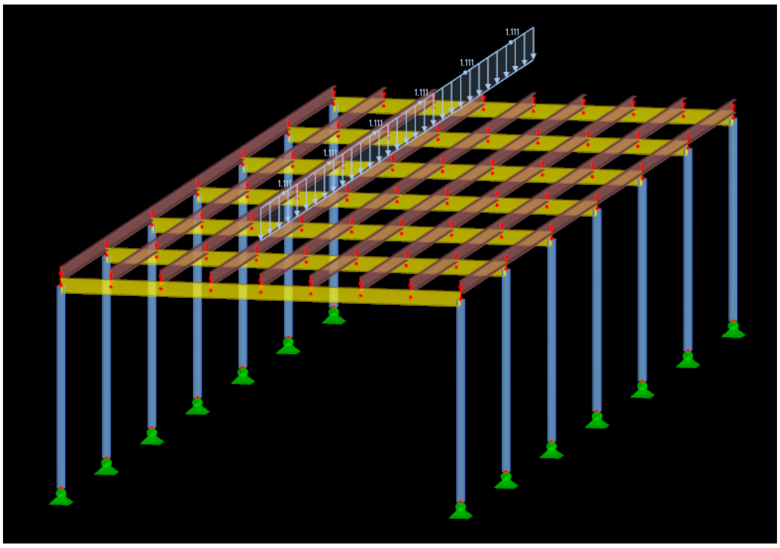


Figure B.9: Maintenance load

C

PRICE COMPARISON STEEL BEAMS

C.1. HEA BEAMS

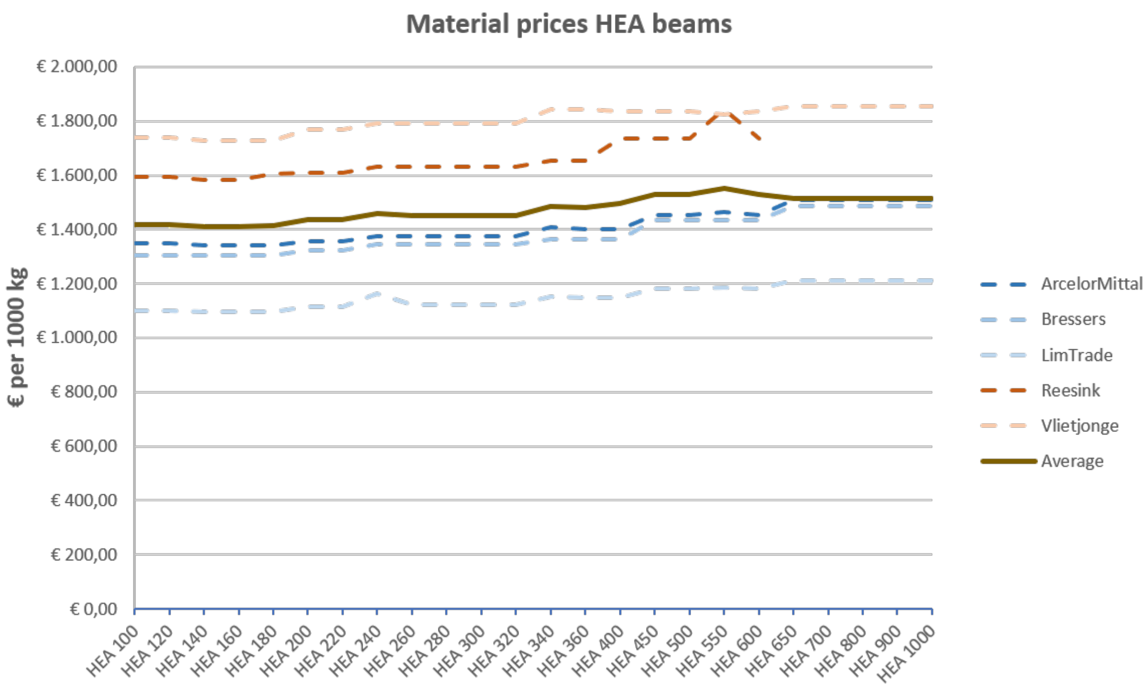


Figure C.1: Material prices HEA beams [6] [7] [8] [9] [10]

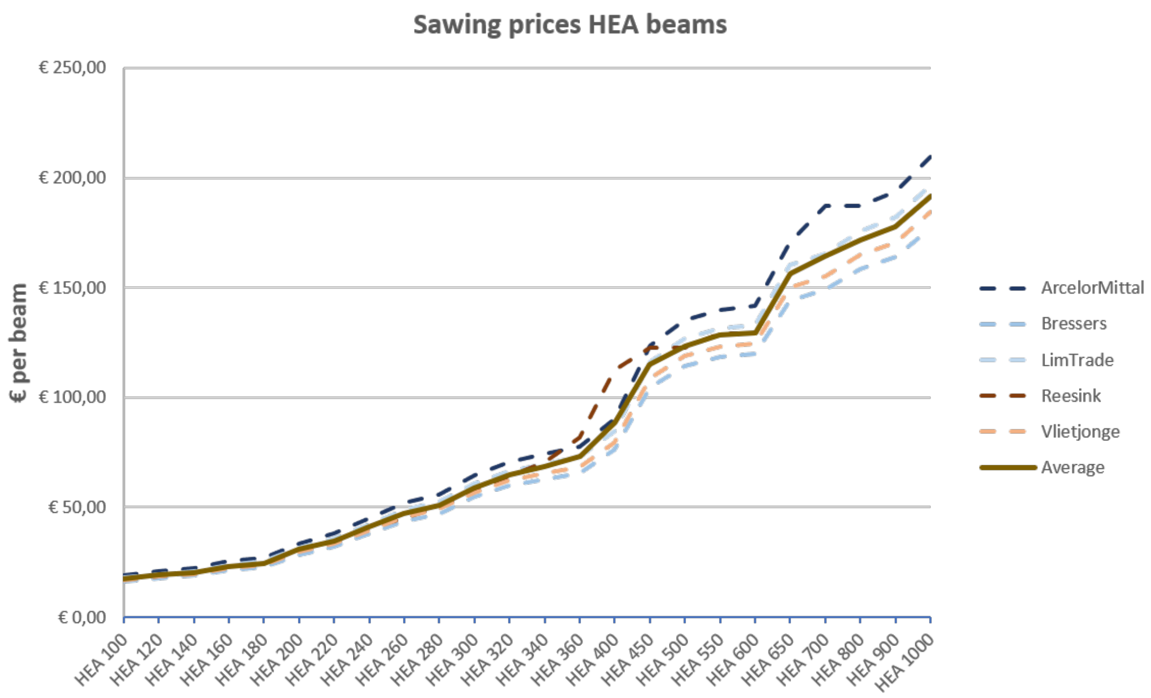


Figure C.2: Sawing prices HEA beams [6] [7] [8] [9] [10]

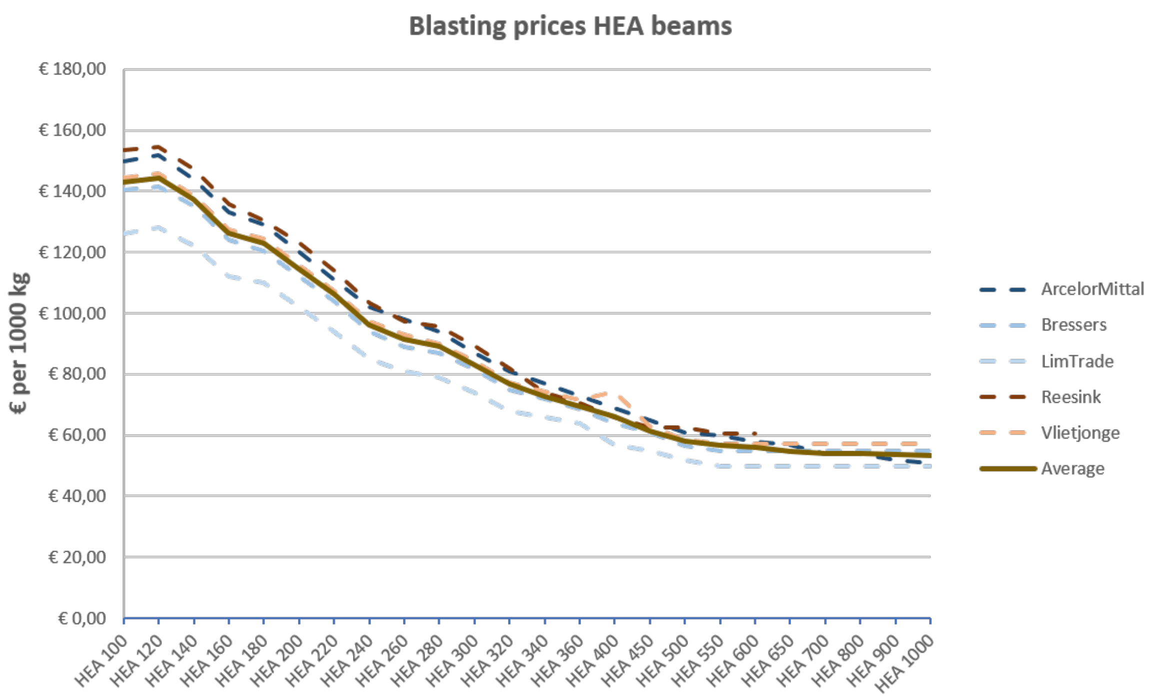


Figure C.3: Blasting prices HEA beams [6] [7] [8] [9] [10]

In the table below a list is given for the average costs of different HEA profiles that are cut and one meter long.

Profile	Material costs [€/m]	Sawing costs [€/beam]	Blasting costs [€/m]	Total costs [€]
HEA 100	€ 24.24	€ 17.48	€ 2.44	€ 44.17
HEA 120	€ 28.78	€ 19.15	€ 2.93	€ 50.86
HEA 140	€ 35.56	€ 20.34	€ 3.46	€ 59.36
HEA 160	€ 43.74	€ 23.24	€ 3.92	€ 70.90
HEA 180	€ 51.22	€ 24.64	€ 4.45	€ 80.31
HEA 200	€ 61.98	€ 30.73	€ 4.95	€ 97.66
HEA 220	€ 73.89	€ 34.57	€ 5.46	€ 113.93
HEA 240	€ 89.81	€ 41.17	€ 5.92	€ 136.91
HEA 260	€ 100.96	€ 47.06	€ 6.37	€ 154.39
HEA 280	€ 113.16	€ 50.90	€ 6.94	€ 171.00
HEA 300	€ 130.73	€ 58.99	€ 7.49	€ 197.21
HEA 320	€ 144.53	€ 64.70	€ 7.63	€ 216.86
HEA 340	€ 159.09	€ 68.59	€ 7.78	€ 235.46
HEA 360	€ 169.29	€ 73.33	€ 7.94	€ 250.56
HEA 400	€ 190.64	€ 88.48	€ 8.42	€ 287.54
HEA 450	€ 218.07	€ 115.08	€ 8.74	€ 341.90
HEA 500	€ 241.46	€ 123.57	€ 9.18	€ 374.21
HEA 550	€ 262.43	€ 128.51	€ 9.57	€ 400.51
HEA 600	€ 277.22	€ 129.58	€ 10.19	€ 416.99
HEA 650	€ 293.55	€ 156.21	€ 10.61	€ 460.38
HEA 700	€ 315.07	€ 164.22	€ 11.24	€ 490.53
HEA 800	€ 345.99	€ 171.53	€ 12.34	€ 529.85
HEA 900	€ 389.33	€ 177.58	€ 13.76	€ 580.67
HEA 1000	€ 420.10	€ 191.85	€ 14.77	€ 626.72

Table C.1: Average costs per meter HEA beam according to figures C.1, C.2, C.3

C.2. HEB BEAMS

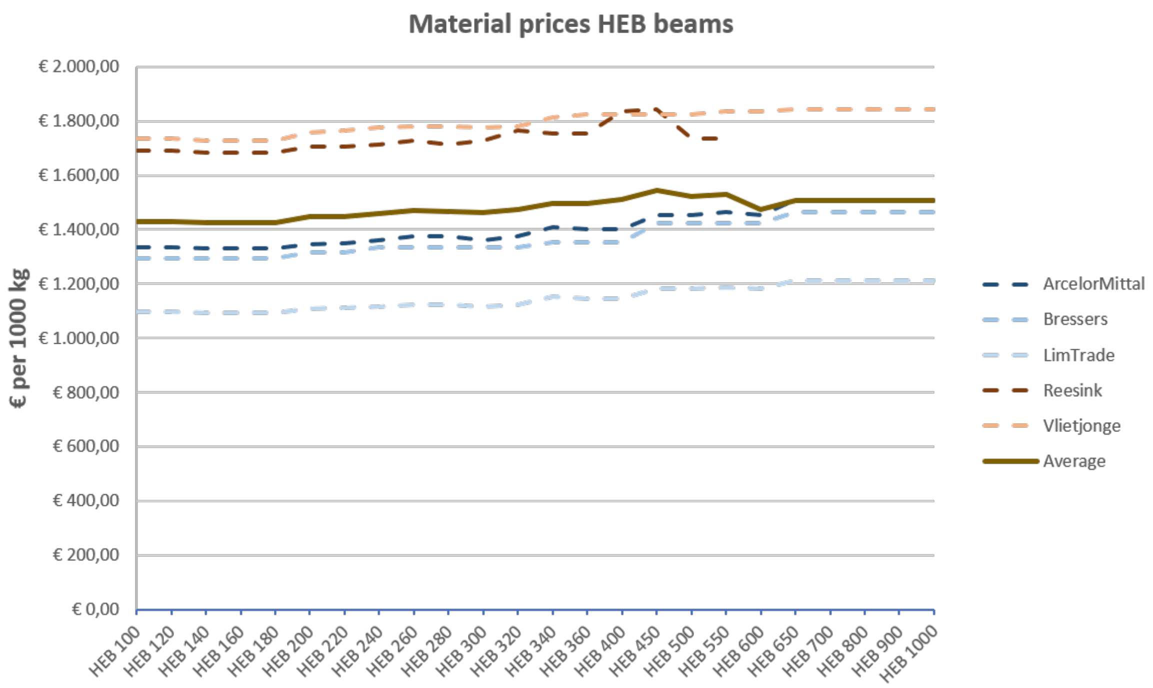


Figure C.4: Material prices HEB beams [6] [7] [8] [9] [11]

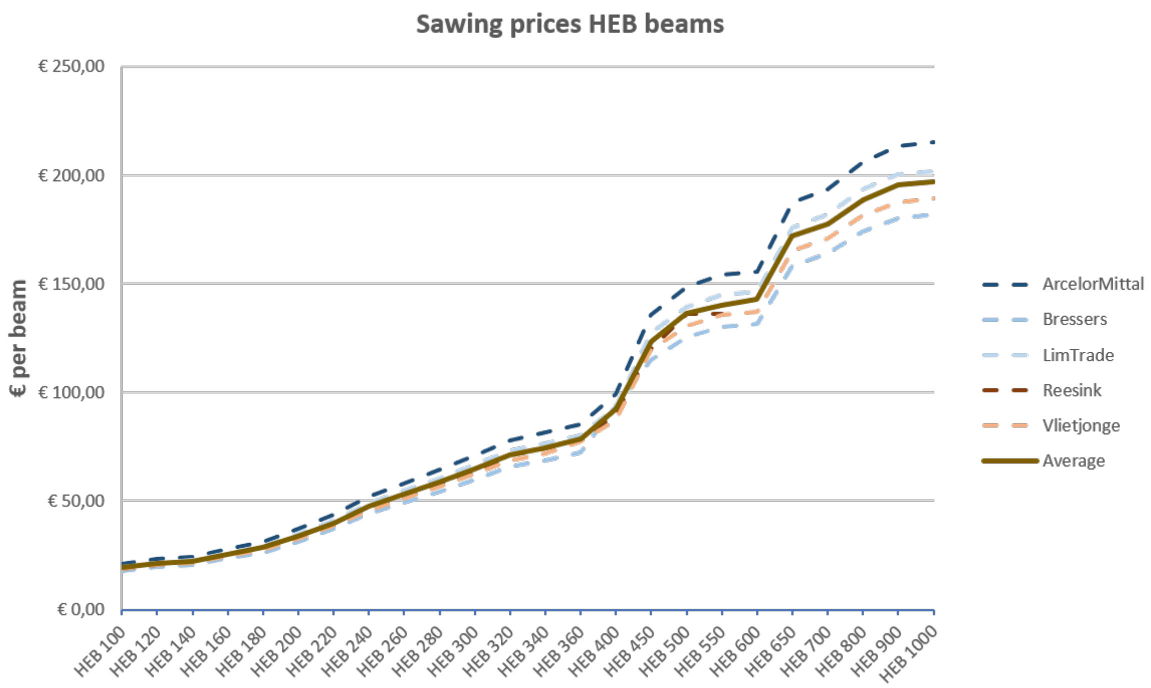


Figure C.5: Sawing prices HEB beams [6] [7] [8] [9] [11]

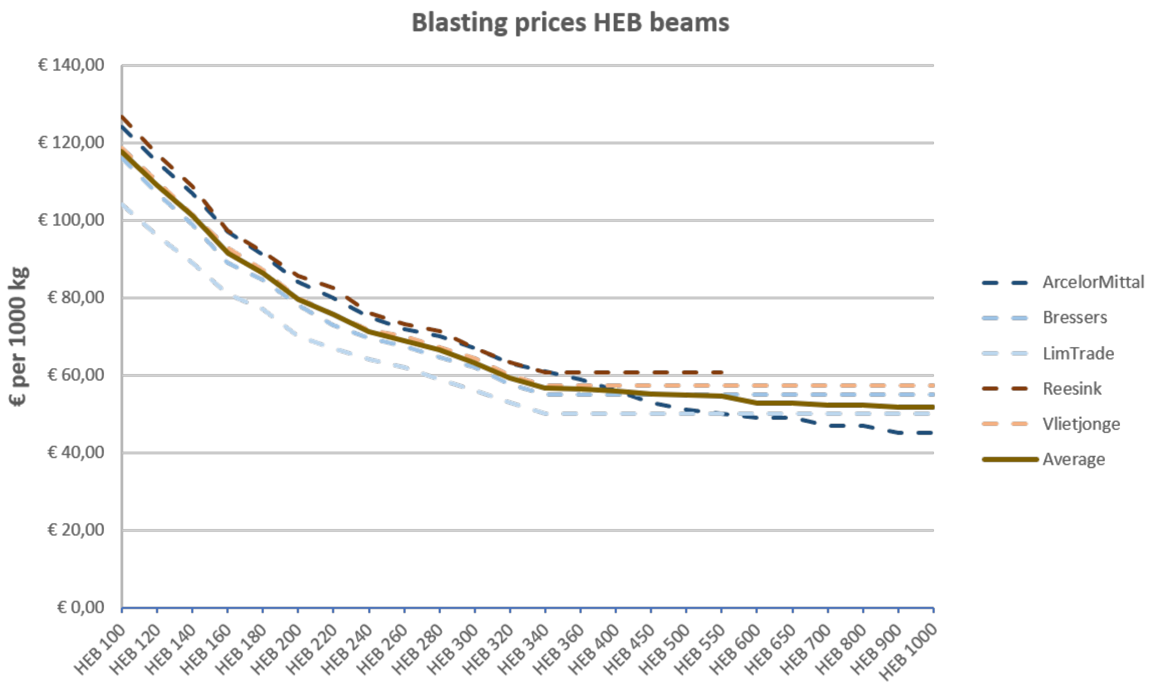


Figure C.6: Blasting prices HEB beams [6] [7] [8] [9] [11]

In the table below a list is given for the average costs of different HEB profiles that are cut and one meter long.

Profile	Material costs [€/m]	Sawing costs [€/beam]	Blasting costs [€/m]	Total costs [€]
HEB 100	€ 29.75	€ 19.28	€ 2.45	€ 51.48
HEB 120	€ 39.05	€ 21.14	€ 2.98	€ 63.17
HEB 140	€ 49.09	€ 22.33	€ 3.48	€ 74.89
HEB 160	€ 62.07	€ 25.54	€ 3.98	€ 91.59
HEB 180	€ 74.49	€ 28.45	€ 4.51	€ 107.45
HEB 200	€ 90.44	€ 33.80	€ 4.97	€ 129.21
HEB 220	€ 105.65	€ 39.95	€ 5.51	€ 151.11
HEB 240	€ 123.82	€ 47.42	€ 6.04	€ 177.28
HEB 260	€ 139.22	€ 53.07	€ 6.54	€ 198.83
HEB 280	€ 153.89	€ 58.74	€ 6.97	€ 219.60
HEB 300	€ 174.56	€ 64.81	€ 7.55	€ 246.92
HEB 320	€ 191.09	€ 71.20	€ 7.69	€ 269.98
HEB 340	€ 204.54	€ 74.58	€ 7.76	€ 286.88
HEB 360	€ 216.68	€ 78.63	€ 8.16	€ 303.48
HEB 400	€ 238.96	€ 92.57	€ 8.81	€ 340.35
HEB 450	€ 269.50	€ 123.55	€ 9.62	€ 402.67
HEB 500	€ 290.51	€ 136.19	€ 10.44	€ 437.15
HEB 550	€ 310.08	€ 140.33	€ 11.07	€ 461.48
HEB 600	€ 318.53	€ 142.68	€ 11.41	€ 472.62
HEB 650	€ 345.78	€ 171.81	€ 12.11	€ 529.70
HEB 700	€ 370.36	€ 177.58	€ 12.84	€ 560.79
HEB 800	€ 402.64	€ 188.68	€ 13.96	€ 605.28
HEB 900	€ 447.27	€ 195.38	€ 15.36	€ 658.01
HEB 1000	€ 482.56	€ 197.11	€ 16.58	€ 696.24

Table C.2: Average costs per meter HEB beam according to figures C.4, C.5, C.6

C.3. IPE BEAMS

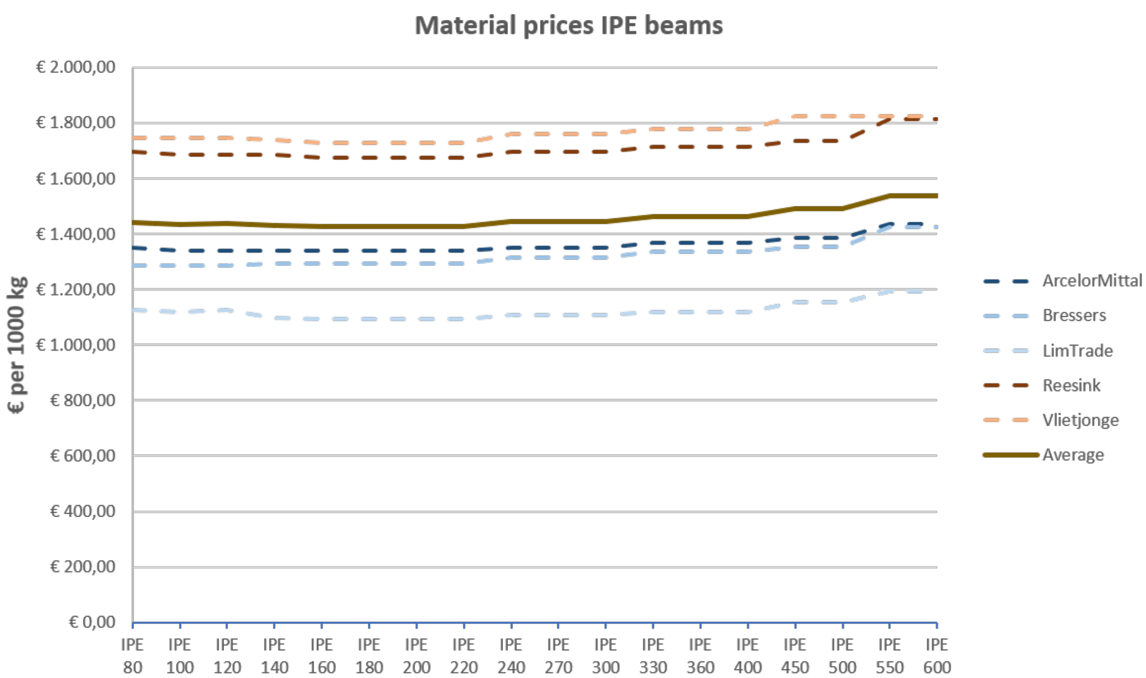


Figure C.7: Material prices IPE beams [6] [7] [8] [9] [12]

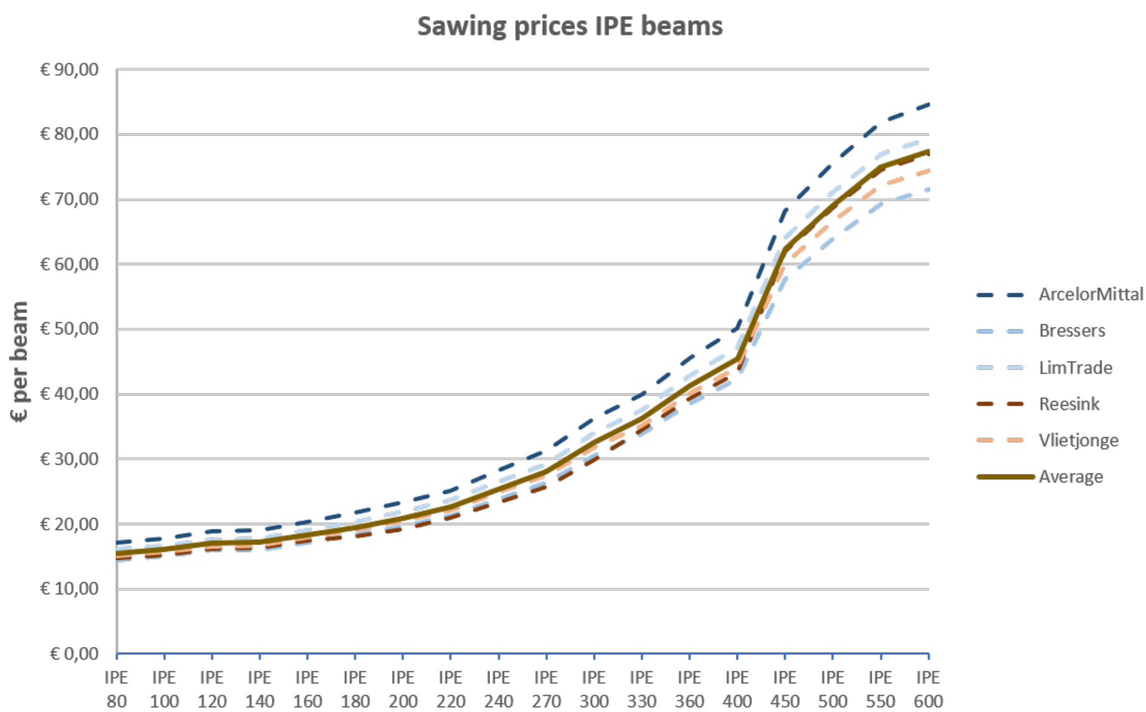


Figure C.8: Sawing prices IPE beams [6] [7] [8] [9] [12]

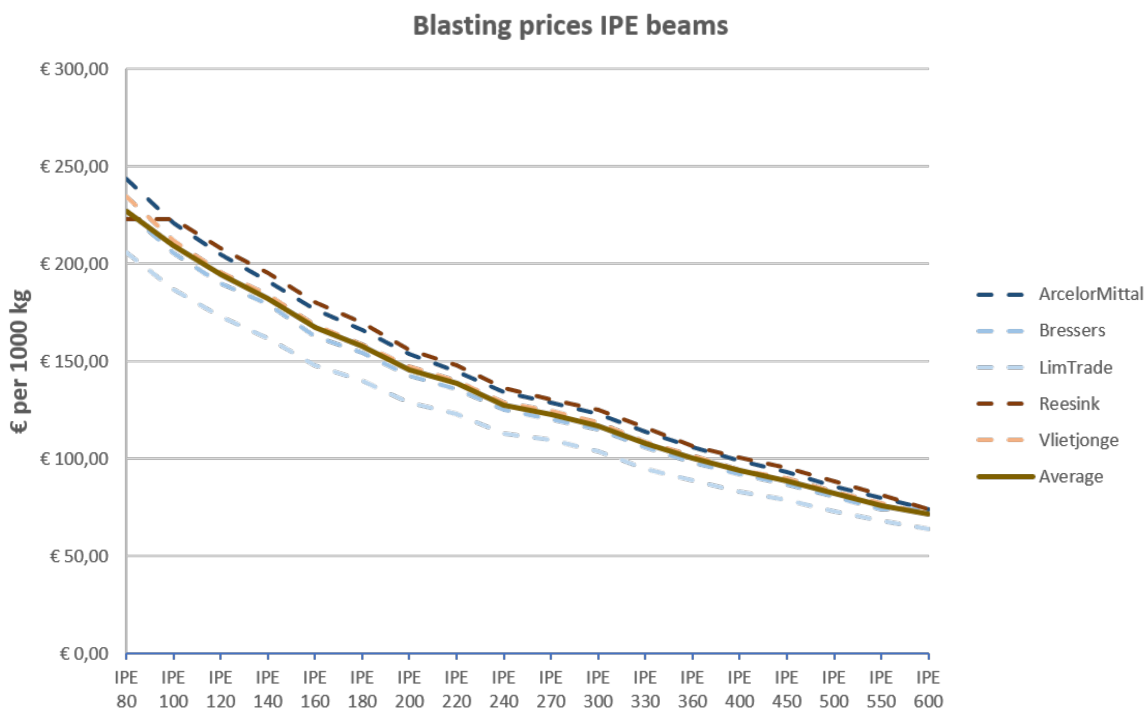


Figure C.9: Blasting prices IPE beams [6] [7] [8] [9] [12]

In the table below a list is given for the average costs of different IPE profiles that are cut and one meter long.

Profile	Material costs [€/m]	Sawing costs [€/beam]	Blasting costs [€/m]	Total costs [€]
IPE 80	€ 8.93	€ 15.52	€ 1.41	€ 25.85
IPE 100	€ 11.91	€ 16.06	€ 1.74	€ 29.71
IPE 120	€ 15.23	€ 17.04	€ 2.06	€ 34.33
IPE 140	€ 18.90	€ 17.17	€ 2.41	€ 38.48
IPE 160	€ 22.97	€ 18.38	€ 2.70	€ 44.05
IPE 180	€ 27.40	€ 19.52	€ 3.03	€ 49.94
IPE 200	€ 32.68	€ 20.93	€ 3.34	€ 56.95
IPE 220	€ 38.10	€ 22.60	€ 3.70	€ 64.40
IPE 240	€ 45.25	€ 25.37	€ 3.99	€ 74.62
IPE 270	€ 53.21	€ 28.05	€ 4.52	€ 85.78
IPE 300	€ 62.31	€ 32.52	€ 5.05	€ 99.88
IPE 330	€ 73.34	€ 36.23	€ 5.41	€ 114.97
IPE 360	€ 85.19	€ 41.24	€ 5.83	€ 132.26
IPE 400	€ 98.95	€ 45.41	€ 6.34	€ 150.71
IPE 450	€ 117.94	€ 62.37	€ 7.04	€ 187.34
IPE 500	€ 137.92	€ 69.13	€ 7.60	€ 214.64
IPE 550	€ 166.32	€ 74.99	€ 8.23	€ 249.54
IPE 600	€ 191.40	€ 77.38	€ 8.91	€ 277.69

Table C.3: Average costs per meter IPE beam according to figures C.7, C.8, C.9

C.4. UPE BEAMS

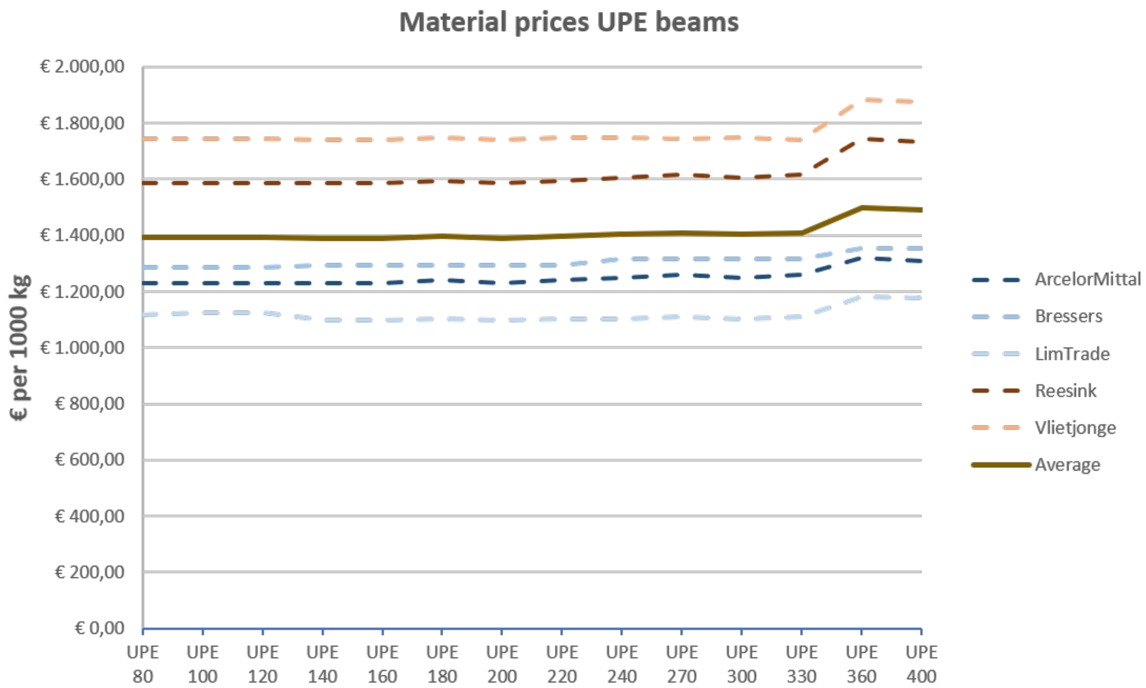


Figure C.10: Material prices UPE beams [6] [7] [8] [9] [13]

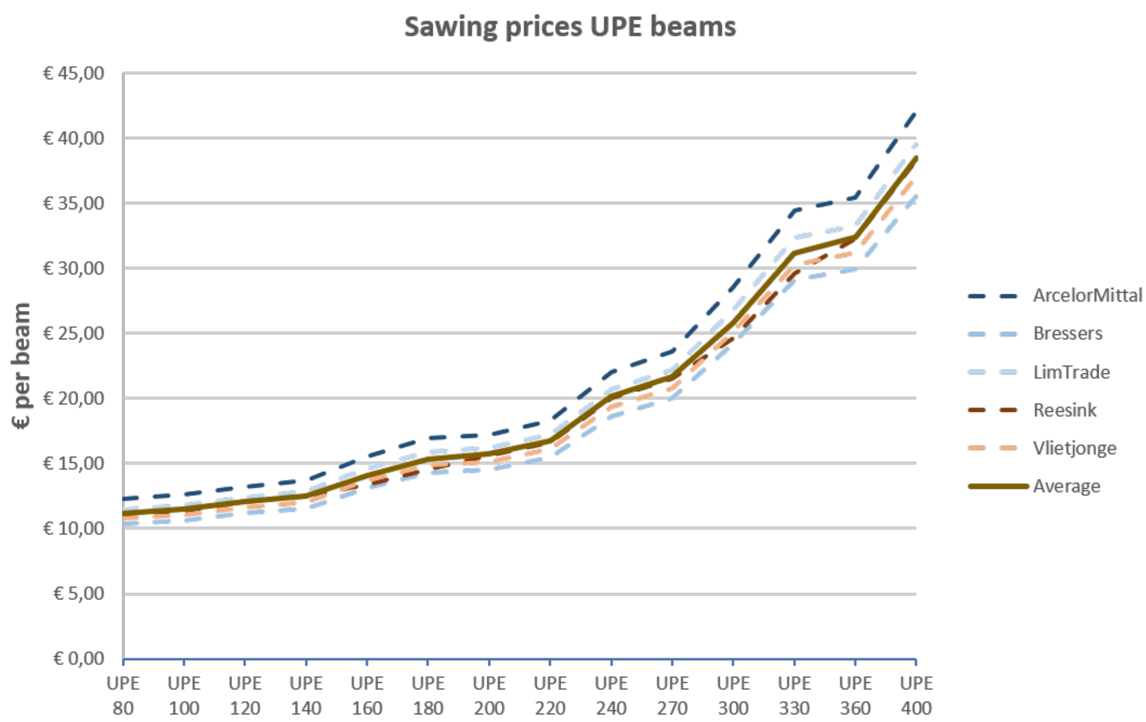


Figure C.11: Sawing prices UPE beams [6] [7] [8] [9] [13]

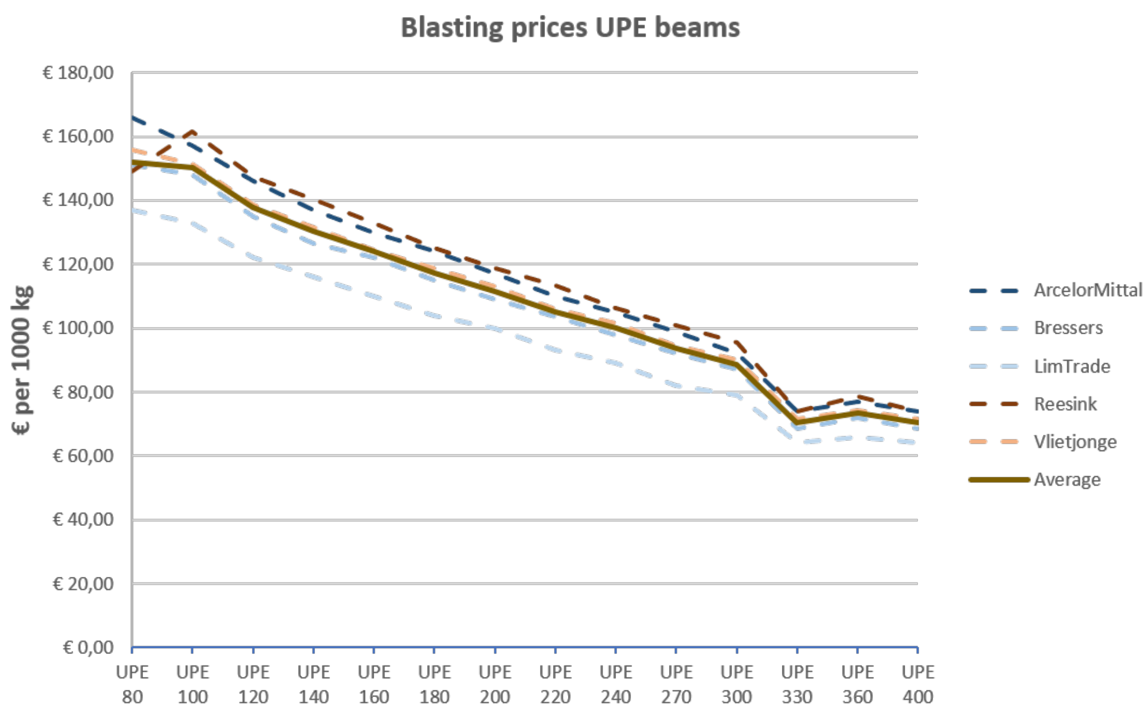


Figure C.12: Blasting prices UPE beams [6] [7] [8] [9] [13]

In the table below a list is given for the average costs of different UPE profiles that are cut and one meter long.

Profile	Material costs [€/m]	Sawing costs [€/beam]	Blasting costs [€/m]	Total costs [€]
UPE 80	€ 12.39	€ 11.20	€ 1.35	€ 24.94
UPE 100	€ 15.06	€ 11.50	€ 1.62	€ 28.18
UPE 120	€ 19.10	€ 12.09	€ 1.89	€ 33.08
UPE 140	€ 22.79	€ 12.53	€ 2.14	€ 37.46
UPE 160	€ 26.68	€ 14.06	€ 2.38	€ 43.12
UPE 180	€ 31.43	€ 15.32	€ 2.64	€ 49.39
UPE 200	€ 35.86	€ 15.74	€ 2.88	€ 54.47
UPE 220	€ 41.90	€ 16.72	€ 3.15	€ 61.78
UPE 240	€ 47.62	€ 20.17	€ 3.39	€ 71.18
UPE 270	€ 54.52	€ 21.63	€ 3.62	€ 79.78
UPE 300	€ 66.17	€ 25.84	€ 4.18	€ 96.18
UPE 330	€ 85.45	€ 31.15	€ 4.27	€ 120.88
UPE 360	€ 92.55	€ 32.42	€ 4.55	€ 129.52
UPE 400	€ 109.13	€ 38.47	€ 5.15	€ 152.75

Table C.4: Average costs per meter UPE beam according to figures C.10, C.11, C.12

D

PRICE COMPARISON CONNECTION PARTS

D.1. PLATES

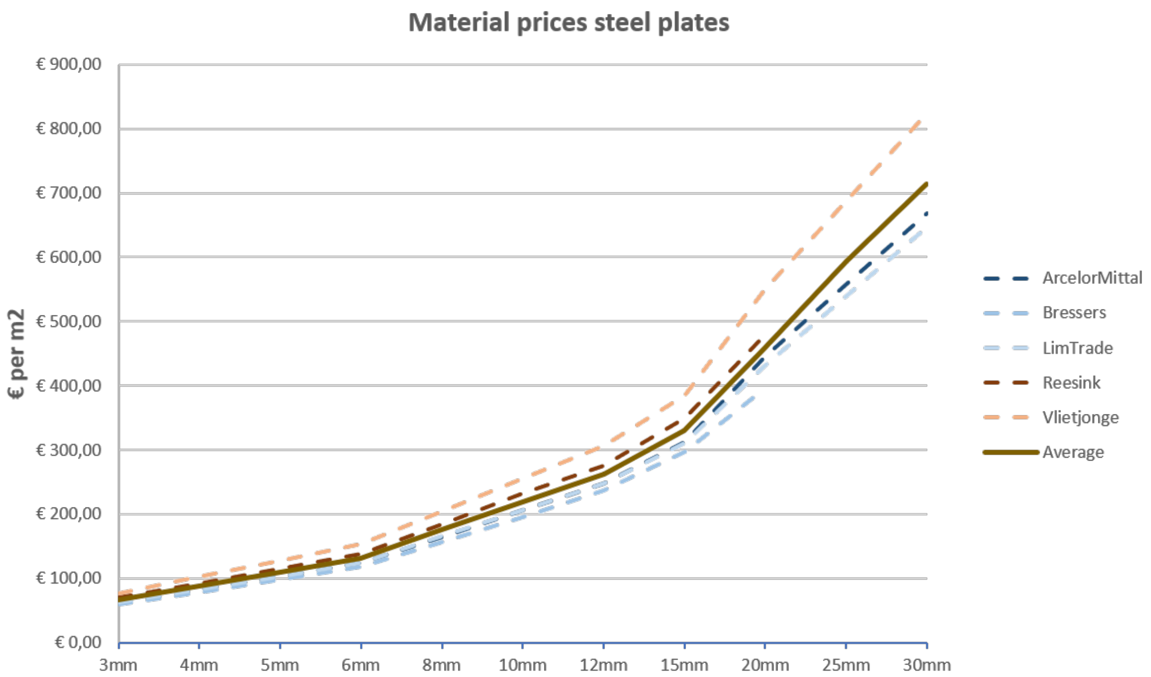


Figure D.1: Material prices steel plates [6] [7] [9] [14] [15]

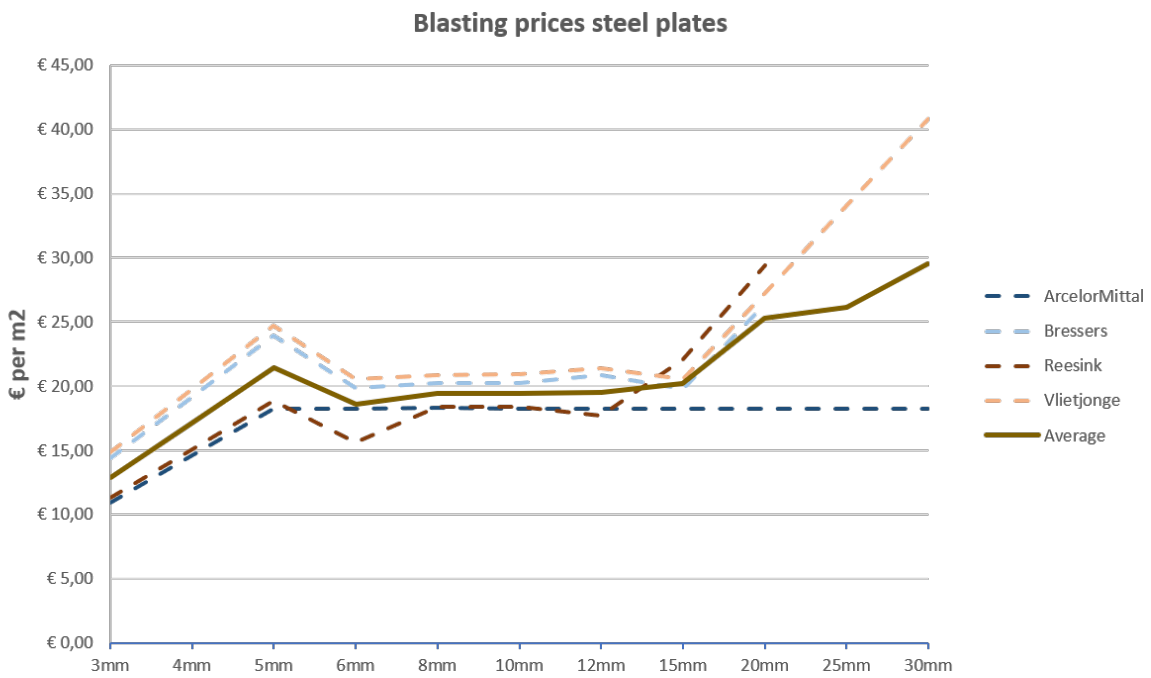


Figure D.2: Blasting prices steel plates [6] [7] [9] [14]

Figure D.3: Sawing prices steel plates [15]

In the table below a list is given for the average costs of different plates thicknesses that are cut and one squared meter.

Thickness [mm]	Material costs [€/m2]	Sawing costs [€/plate]	Blasting costs [€/m2]	Total costs [€/m2]
3	€ 65.89	€ 4.50	€ 12.86	€ 83.25
4	€ 87.82	€ 4.50	€ 17.14	€ 109.46
5	€ 109.63	€ 4.50	€ 21.43	€ 135.56
6	€ 131.72	€ 4.50	€ 18.56	€ 154.79
8	€ 175.41	€ 4.50	€ 19.45	€ 199.35
10	€ 218.77	€ 4.50	€ 19.46	€ 242.74
12	€ 262.85	€ 15.00	€ 19.54	€ 297.39
15	€ 330.40	€ 15.00	€ 20.18	€ 365.58
20	€ 460.31	€ 15.00	€ 25.33	€ 500.64
25	€ 594.45	€ 20.00	€ 26.14	€ 640.59
30	€ 713.34	€ 25.00	€ 29.54	€ 767.89

Table D.1: Average costs per squared meter plate D.1, D.2, D.3

D.2. BOLTS

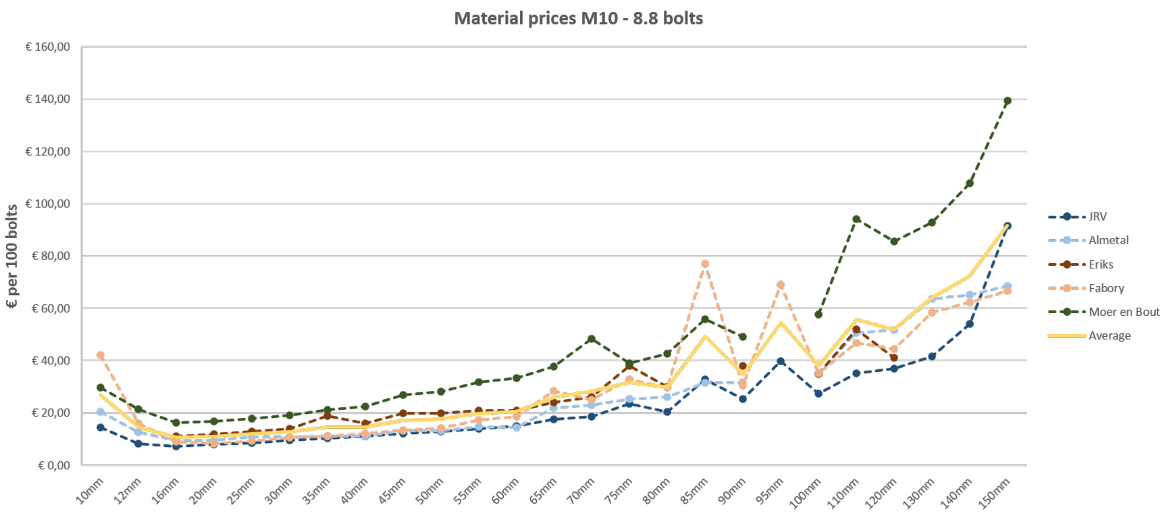


Figure D.4: Material prices 8.8 steel M10 bolts [16] [17] [18] [19] [20]

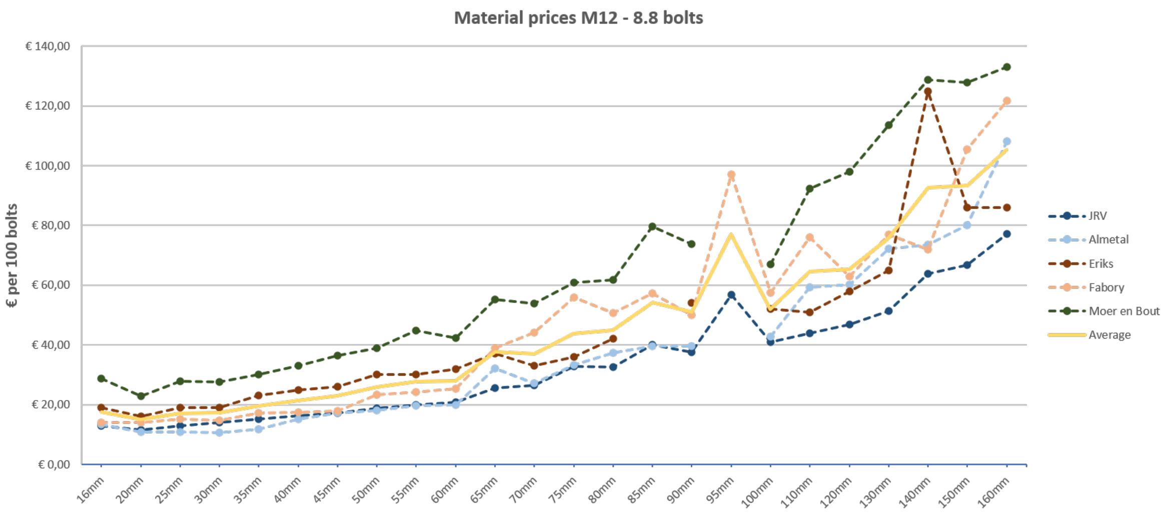


Figure D.5: Material prices 8.8 steel M12 bolts [16] [17] [18] [19] [20]

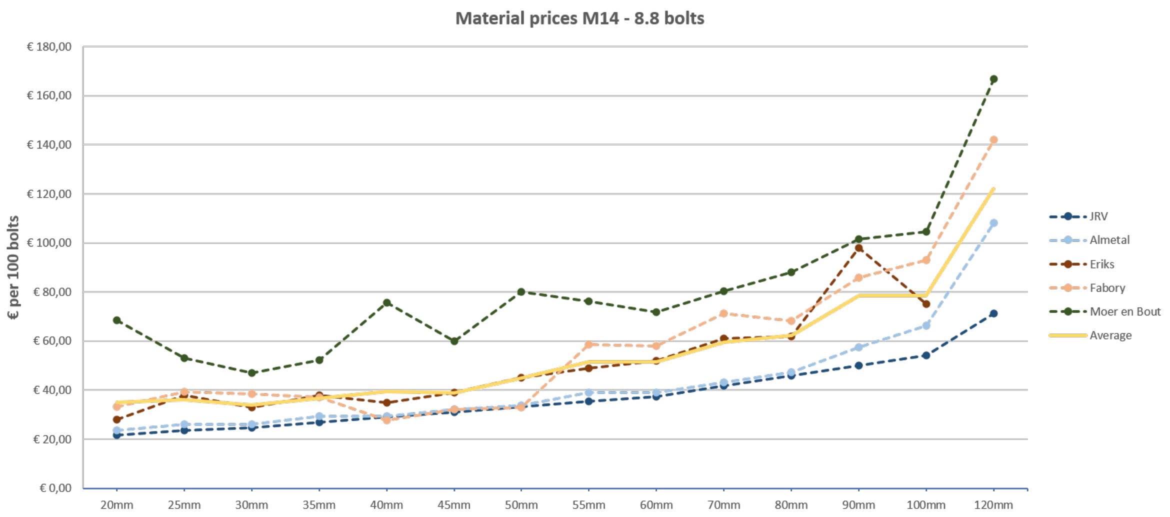


Figure D.6: Material prices 8.8 steel M14 bolts [16] [17] [18] [19] [20]

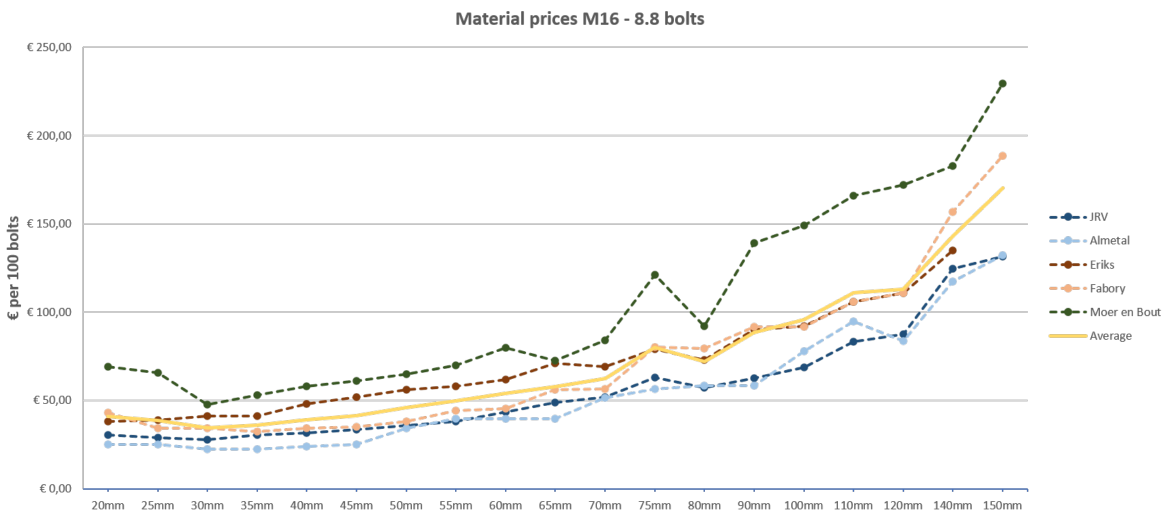


Figure D.7: Material prices 8.8 steel M16 bolts [16] [17] [18] [19] [20]

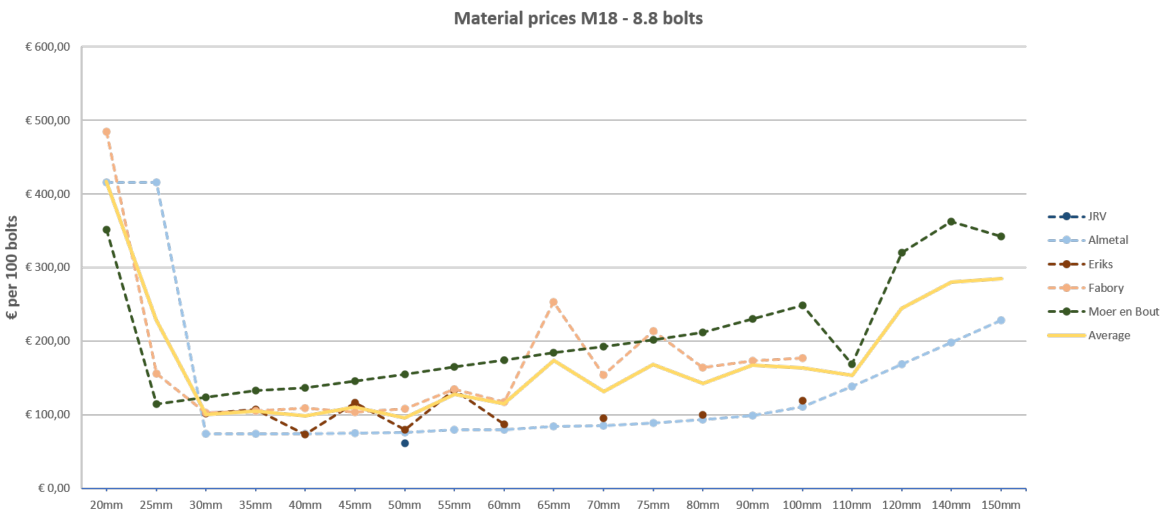


Figure D.8: Material prices 8.8 steel M18 bolts [16] [17] [18] [19] [20]

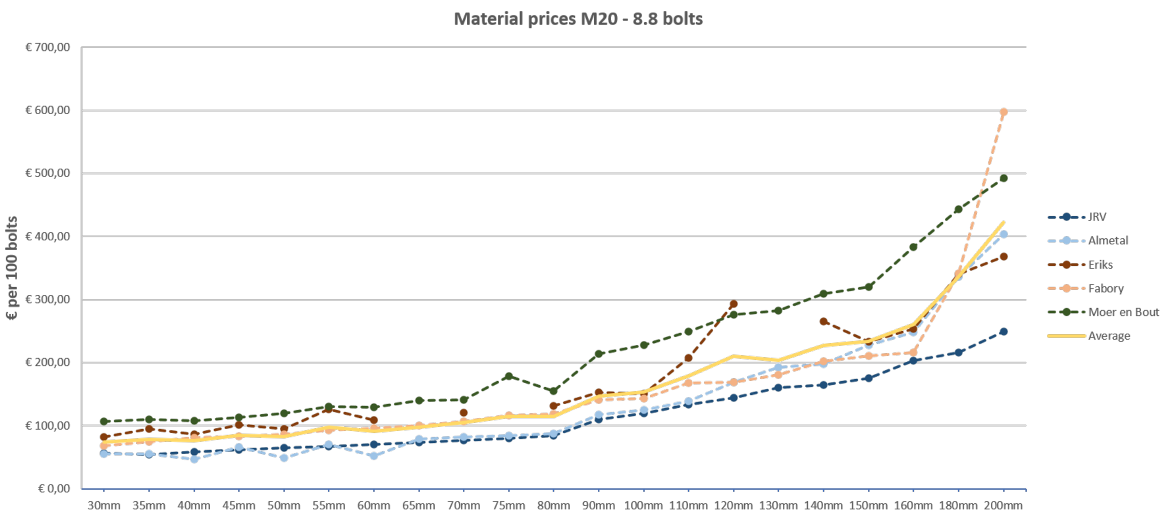


Figure D.9: Material prices 8.8 steel M20 bolts [16] [17] [18] [19] [20]

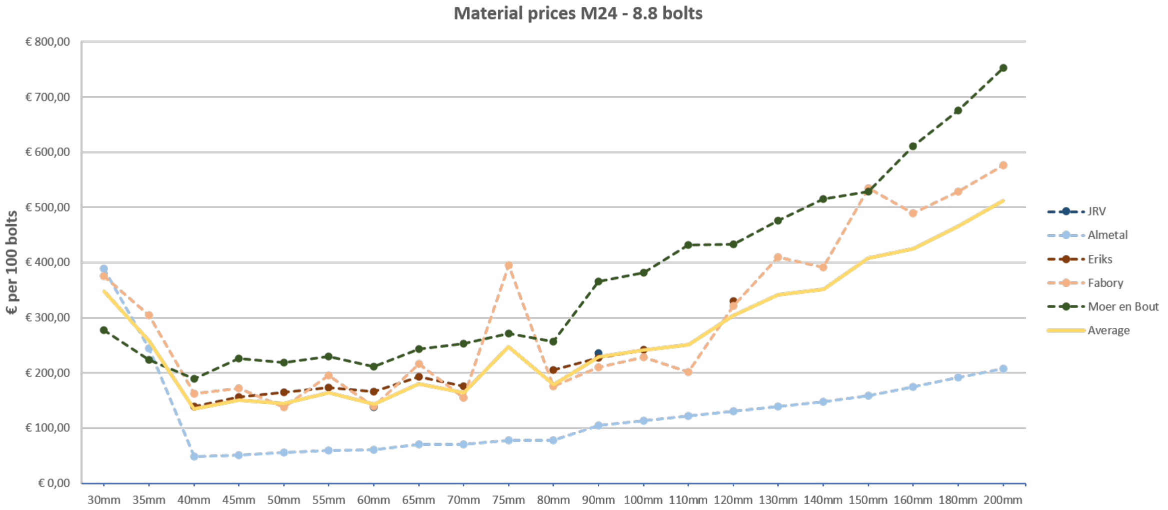


Figure D.10: Material prices 8.8 steel M24 bolts [16] [17] [18] [19] [20]

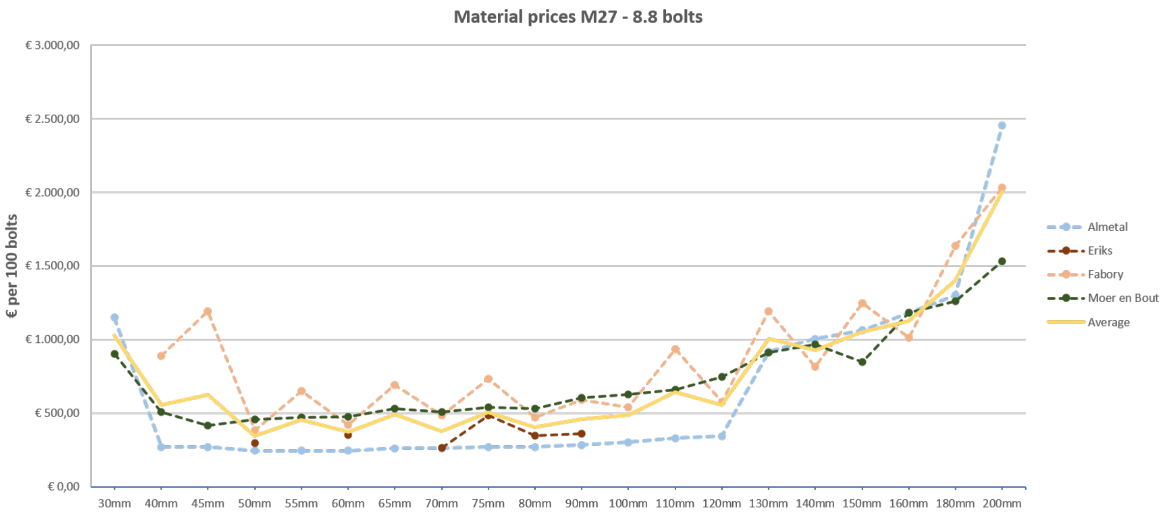


Figure D.11: Material prices 8.8 steel M27 bolts [17] [18] [19] [20]

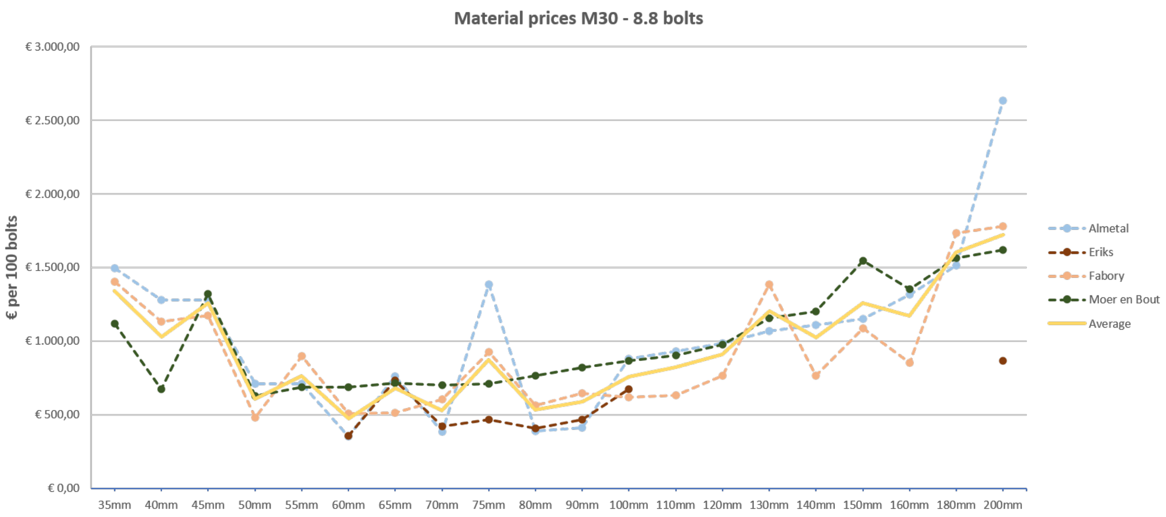


Figure D.12: Material prices 8.8 steel M30 bolts [17] [18] [19] [20]

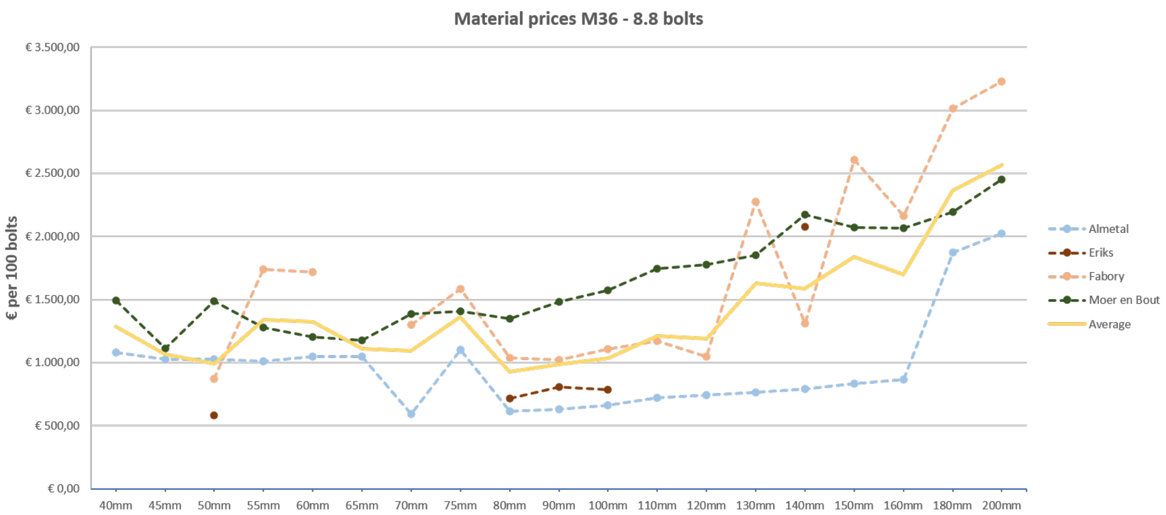


Figure D.13: Material prices 8.8 steel M36 bolts [17] [18] [19] [20]

In the table below a list is given for the average costs of different bolts with a steel strength of 8.8.

Length [mm]	M10	M12	M14	M16	M18	M20	M24	M27	M30	M36
10	€ 0.27	-	-	-	-	-	-	-	-	-
12	€ 0.15	-	-	-	-	-	-	-	-	-
16	€ 0.11	€ 0.18	-	-	-	-	-	-	-	-
20	€ 0.11	€ 0.15	€ 0.35	€ 0.41	€ 4.17	-	-	-	-	-
25	€ 0.12	€ 0.17	€ 0.36	€ 0.39	€ 2.29	-	-	-	-	-
30	€ 0.13	€ 0.17	€ 0.34	€ 0.35	€ 1.00	€ 0.74	€ 3.48	€ 10.27	-	-
35	€ 0.15	€ 0.19	€ 0.37	€ 0.36	€ 1.05	€ 0.78	€ 2.57	-	€ 13.40	-
40	€ 0.15	€ 0.21	€ 0.39	€ 0.39	€ 0.98	€ 0.76	€ 1.35	€ 5.57	€ 10.27	€ 12.85
45	€ 0.17	€ 0.23	€ 0.39	€ 0.41	€ 1.10	€ 0.85	€ 1.51	€ 6.27	€ 12.57	€ 10.68
50	€ 0.18	€ 0.26	€ 0.45	€ 0.46	€ 0.96	€ 0.83	€ 1.44	€ 3.46	€ 6.05	€ 9.91
55	€ 0.20	€ 0.28	€ 0.52	€ 0.50	€ 1.28	€ 0.97	€ 1.64	€ 4.56	€ 7.65	€ 13.43
60	€ 0.21	€ 0.28	€ 0.52	€ 0.54	€ 1.15	€ 0.91	€ 1.43	€ 3.74	€ 4.75	€ 13.23
65	€ 0.26	€ 0.38	-	€ 0.58	€ 1.74	€ 0.98	€ 1.81	€ 4.94	€ 6.81	€ 11.11
70	€ 0.28	€ 0.37	€ 0.60	€ 0.63	€ 1.32	€ 1.06	€ 1.64	€ 3.79	€ 5.28	€ 10.93
75	€ 0.32	€ 0.44	-	€ 0.80	€ 1.68	€ 1.15	€ 2.48	€ 5.07	€ 8.72	€ 13.63
80	€ 0.30	€ 0.45	€ 0.62	€ 0.72	€ 1.42	€ 1.15	€ 1.79	€ 4.04	€ 5.32	€ 9.28
85	€ 0.49	€ 0.54	-	-	-	-	-	-	-	-
90	€ 0.35	€ 0.51	€ 0.79	€ 0.88	€ 1.68	€ 1.47	€ 2.29	€ 4.60	€ 5.86	€ 9.85
95	€ 0.54	€ 0.77	-	-	-	-	-	-	-	-
100	€ 0.38	€ 0.52	€ 0.79	€ 0.96	€ 1.64	€ 1.53	€ 2.42	€ 4.89	€ 7.59	€ 10.32
110	€ 0.56	€ 0.65	-	€ 1.11	€ 1.54	€ 1.79	€ 2.52	€ 6.42	€ 8.23	€ 12.12
120	€ 0.52	€ 0.65	€ 1.22	€ 1.13	€ 2.45	€ 2.10	€ 3.04	€ 5.56	€ 9.09	€ 11.89
130	€ 0.64	€ 0.76	-	-	-	€ 2.04	€ 3.42	€ 10.08	€ 12.04	€ 16.31
140	€ 0.72	€ 0.93	-	€ 1.43	€ 2.80	€ 2.28	€ 3.51	€ 9.29	€ 10.26	€ 15.87
150	€ 0.92	€ 0.93	-	€ 1.70	€ 2.85	€ 2.33	€ 4.08	€ 10.53	€ 12.61	€ 18.37
160	-	€ 1.05	-	-	-	€ 2.61	€ 4.25	€ 11.25	€ 11.74	€ 16.99
180	-	-	-	-	-	€ 3.35	€ 4.65	€ 14.01	€ 16.04	€ 23.62
200	-	-	-	-	-	€ 4.22	€ 5.12	€ 20.06	€ 17.25	€ 25.68

Table D.2: Average costs per bolt from figures: D.4, D.5, D.6, D.7, D.8, D.9, D.10, D.11, D.12, D.13

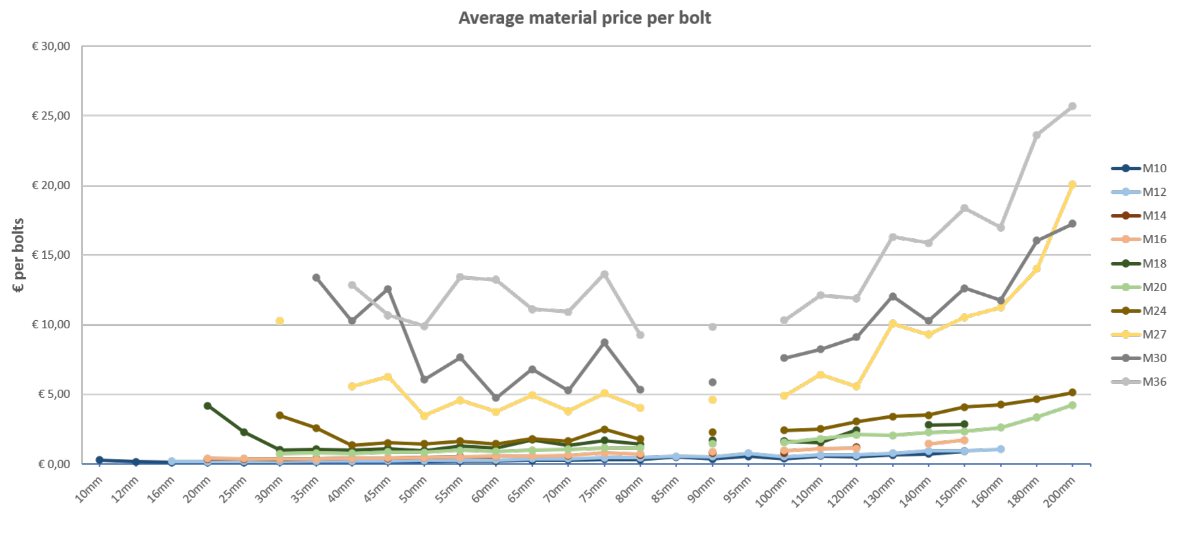


Figure D.14: Average material prices 8.8 steel bolts from table D.2

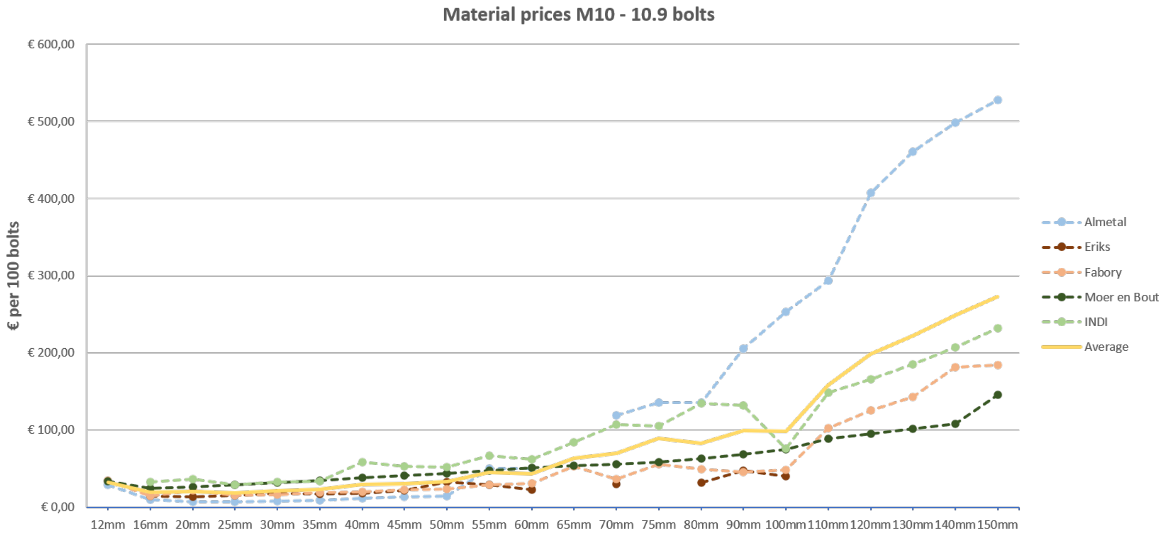


Figure D.15: Material prices 10.9 steel M10 bolts [21] [22] [23] [24] [25]

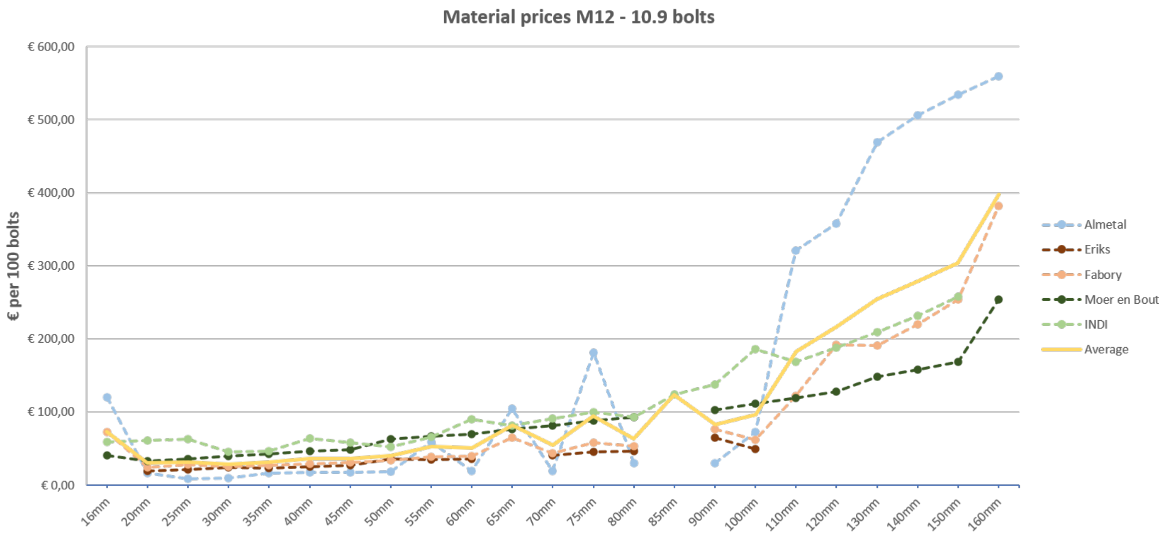


Figure D.16: Material prices 10.9 steel M12 bolts [21] [22] [23] [24] [25]

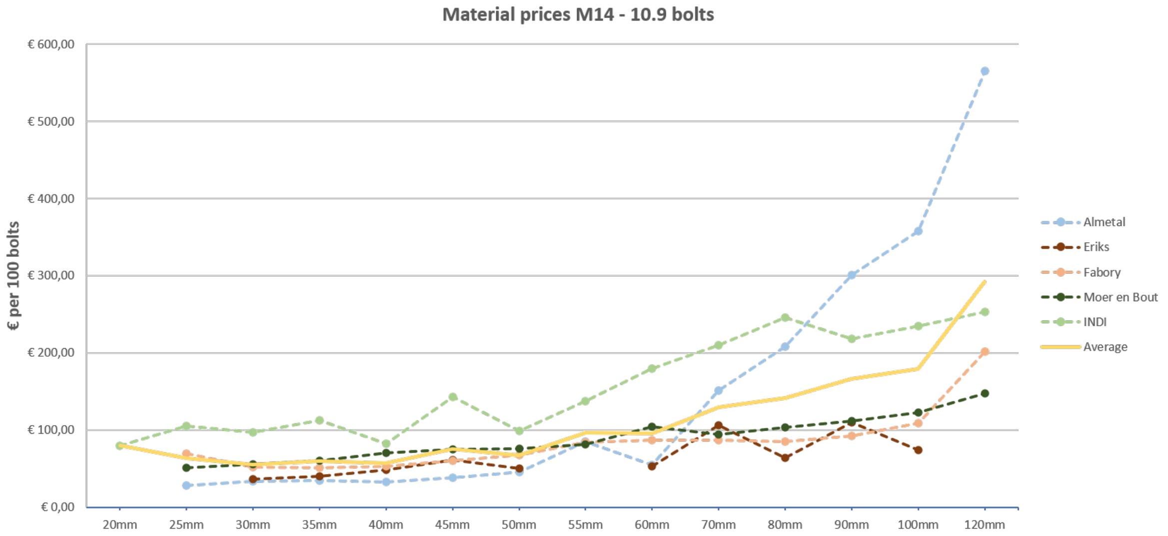


Figure D.17: Material prices 10.9 steel M14 bolts [21] [22] [23] [24] [25]

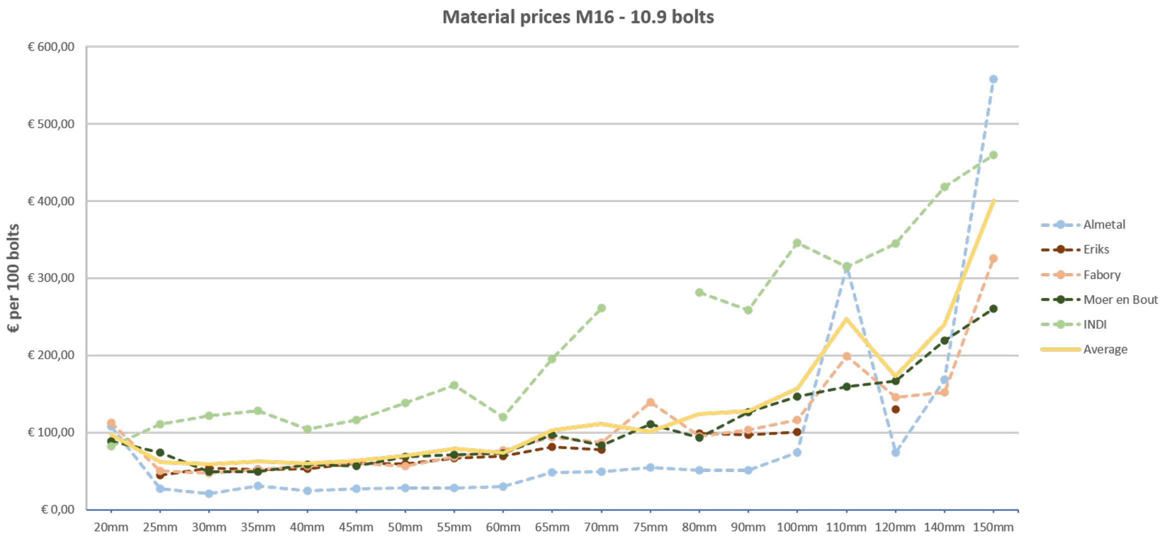


Figure D.18: Material prices 10.9 steel M16 bolts [21] [22] [23] [24] [25]

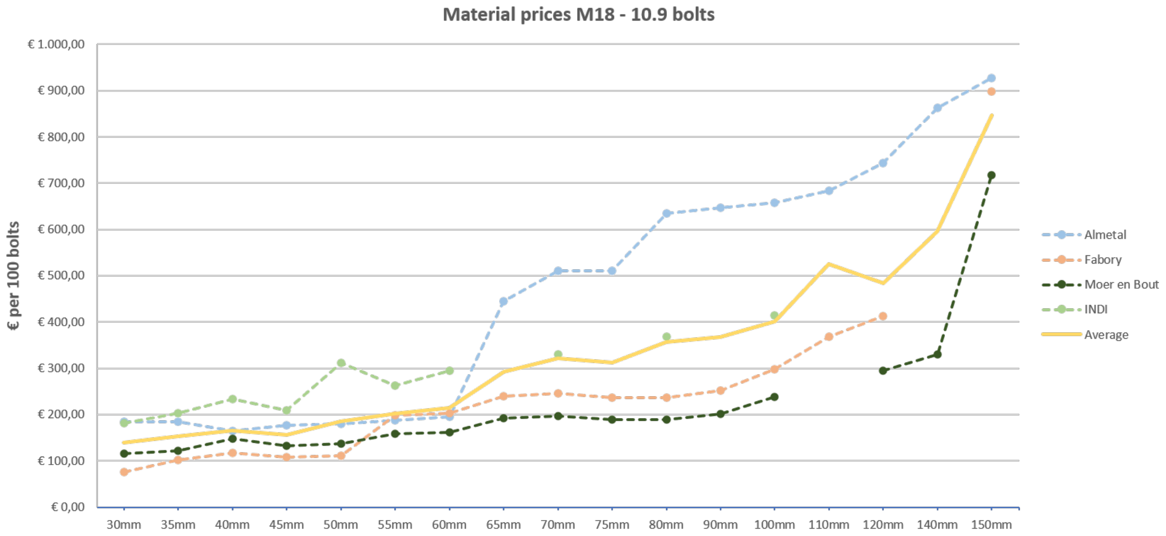


Figure D.19: Material prices 10.9 steel M18 bolts [21] [23] [24] [25]

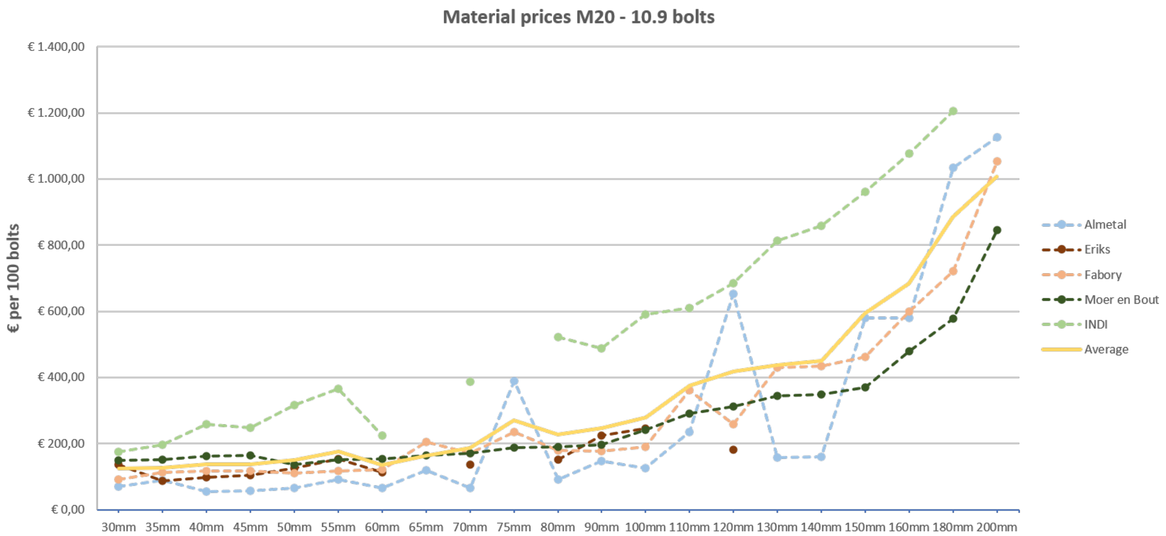


Figure D.20: Material prices 10.9 steel M20 bolts [21] [22] [23] [24] [25]

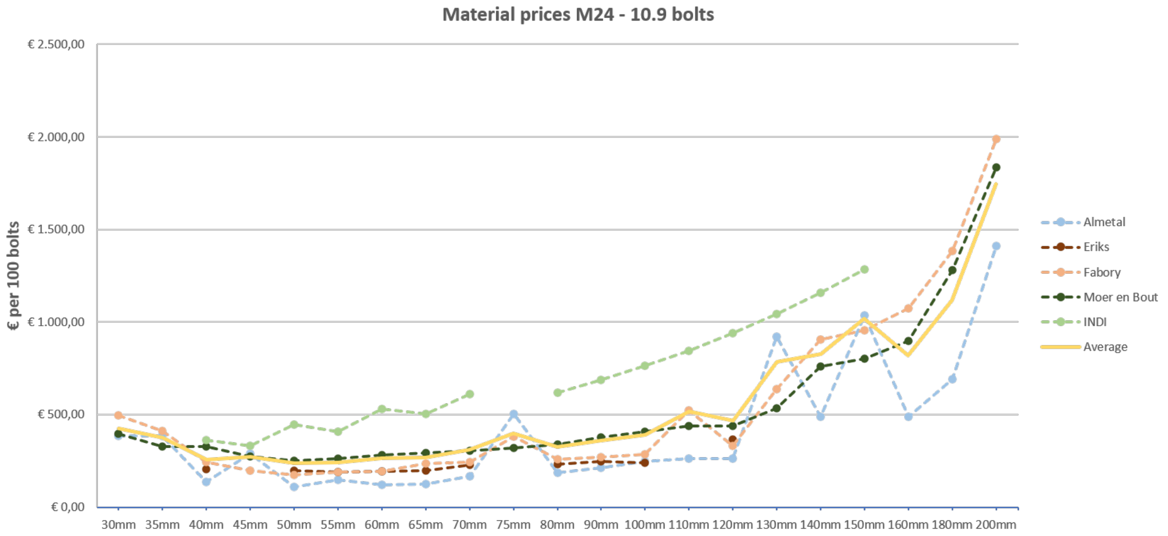


Figure D.21: Material prices 10.9 steel M24 bolts [21] [22] [23] [24] [25]

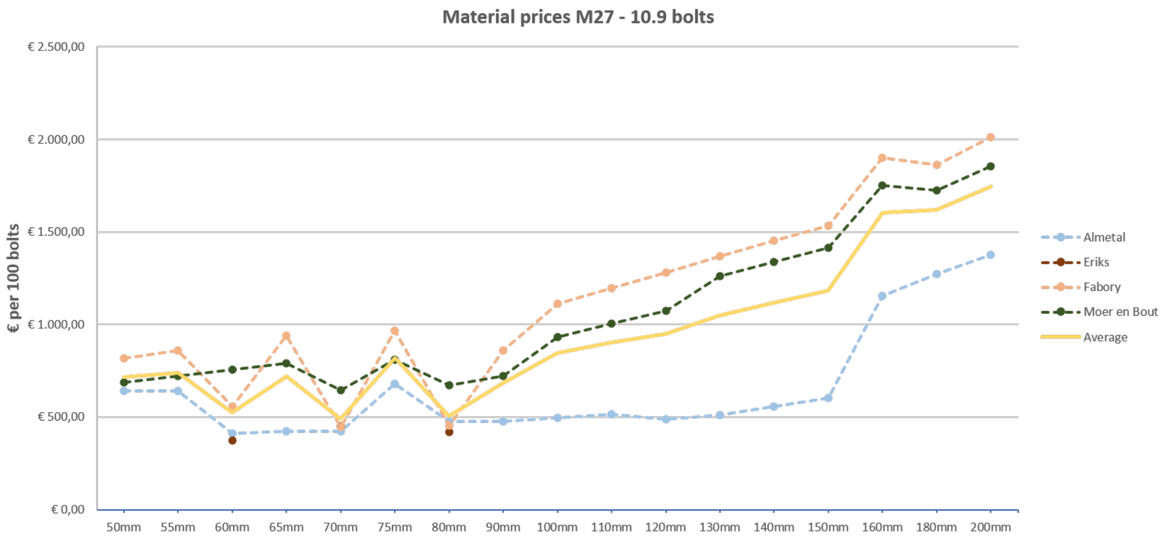


Figure D.22: Material prices 10.9 steel M27 bolts [21] [22] [23] [24]

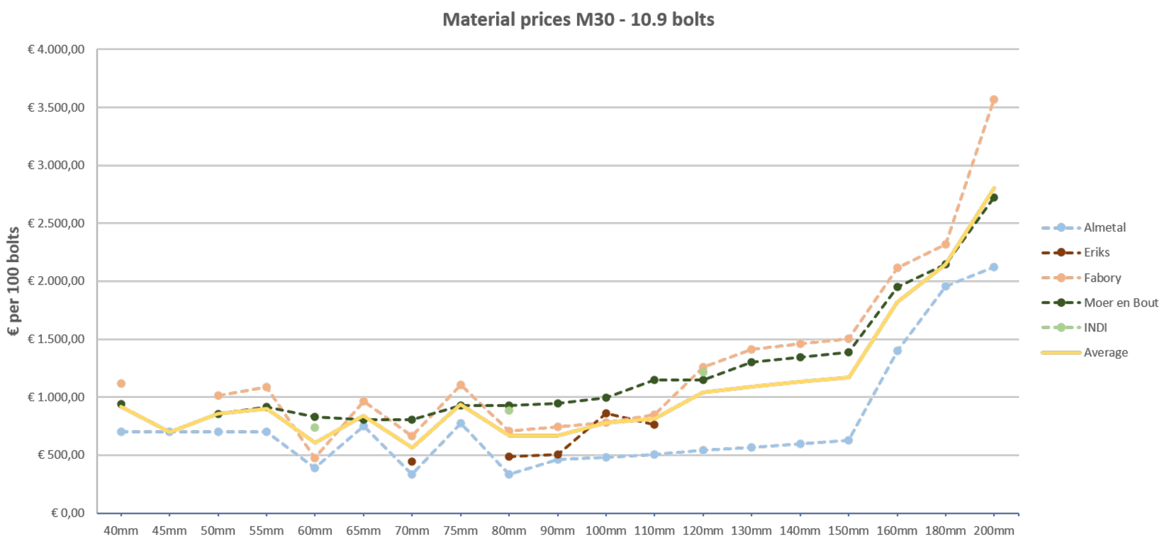


Figure D.23: Material prices 10.9 steel M30 bolts [21] [22] [23] [24] [25]

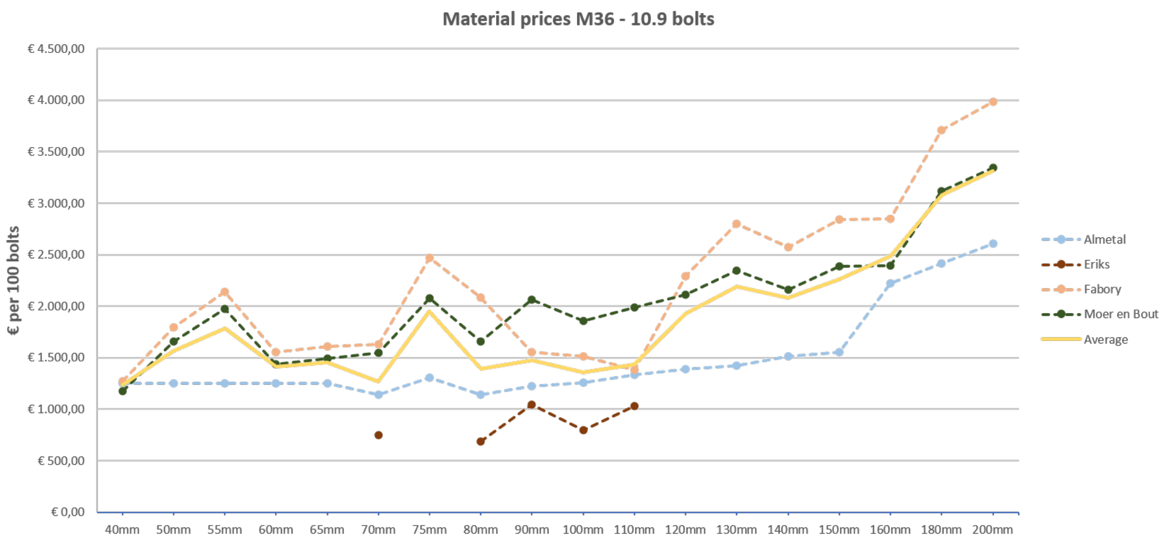


Figure D.24: Material prices 10.9 steel M36 bolts [21] [22] [23] [24] [25]

In the table below a list is given for the average costs of different bolts with a steel strength of 10.9.

Length [mm]	M10	M12	M14	M16	M18	M20	M24	M27	M30	M36
12	€ 0.33	-	-	-	-	-	-	-	-	-
16	€ 0.19	€ 0.73	-	-	-	-	-	-	-	-
20	€ 0.21	€ 0.31	€ 0.80	€ 0.98	-	-	-	-	-	-
25	€ 0.19	€ 0.31	€ 0.63	€ 0.62	-	-	-	-	-	-
30	€ 0.21	€ 0.29	€ 0.55	€ 0.59	€ 1.39	€ 1.24	€ 4.27	-	-	-
35	€ 0.23	€ 0.31	€ 0.60	€ 0.62	€ 1.53	€ 1.27	€ 3.75	-	-	-
40	€ 0.29	€ 0.36	€ 0.57	€ 0.59	€ 1.66	€ 1.38	€ 2.55	-	€ 9.18	€ 12.32
45	€ 0.30	€ 0.36	€ 0.75	€ 0.64	€ 1.57	€ 1.38	€ 2.73	-	€ 6.99	-
50	€ 0.34	€ 0.41	€ 0.68	€ 0.70	€ 1.85	€ 1.51	€ 2.36	€ 7.15	€ 8.56	€ 15.67
55	€ 0.45	€ 0.53	€ 0.97	€ 0.79	€ 2.02	€ 1.75	€ 2.40	€ 7.40	€ 9.00	€ 17.88
60	€ 0.43	€ 0.51	€ 0.96	€ 0.74	€ 2.14	€ 1.35	€ 2.63	€ 5.25	€ 6.07	€ 14.13
65	€ 0.64	€ 0.82	-	€ 1.03	€ 2.93	€ 1.62	€ 2.70	€ 7.18	€ 8.40	€ 14.51
70	€ 0.70	€ 0.55	€ 1.29	€ 1.12	€ 3.21	€ 1.86	€ 3.11	€ 4.91	€ 5.62	€ 12.67
75	€ 0.89	€ 0.95	-	€ 1.01	€ 3.12	€ 2.70	€ 4.00	€ 8.19	€ 9.36	€ 19.50
80	€ 0.83	€ 0.63	€ 1.41	€ 1.24	€ 3.57	€ 2.27	€ 3.26	€ 5.05	€ 6.68	€ 13.94
85	-	€ 1.24	-	-	-	-	-	-	-	-
90	€ 1.00	€ 0.82	€ 1.67	€ 1.27	€ 3.67	€ 2.47	€ 3.60	€ 6.87	€ 6.66	€ 14.73
100	€ 0.98	€ 0.96	€ 1.80	€ 1.57	€ 4.02	€ 2.78	€ 3.89	€ 8.46	€ 7.81	€ 13.56
110	€ 1.58	€ 1.83	-	€ 2.47	€ 5.26	€ 3.74	€ 5.17	€ 9.05	€ 8.18	€ 14.34
120	€ 1.98	€ 2.16	€ 2.92	€ 1.72	€ 4.84	€ 4.18	€ 4.68	€ 9.48	€ 10.39	€ 19.30
130	€ 2.23	€ 2.54	-	-	-	€ 4.37	€ 7.84	€ 10.47	€ 10.94	€ 21.91
140	€ 2.49	€ 2.79	-	€ 2.40	€ 5.96	€ 4.50	€ 8.27	€ 11.17	€ 11.34	€ 20.80
150	€ 2.72	€ 3.04	-	€ 4.01	€ 8.47	€ 5.93	€ 10.19	€ 11.83	€ 11.72	€ 22.62
160	-	€ 3.99	-	-	-	€ 6.84	€ 8.20	€ 16.02	€ 18.20	€ 24.87
180	-	-	-	-	-	€ 8.85	€ 11.19	€ 16.20	€ 21.42	€ 30.80
200	-	-	-	-	-	€ 10.08	€ 17.44	€ 17.47	€ 28.04	€ 33.14

Table D.3: Average costs per bolt from figures: D.15, D.16, D.17, D.18, D.19, D.20, D.21, D.22, D.23, D.24

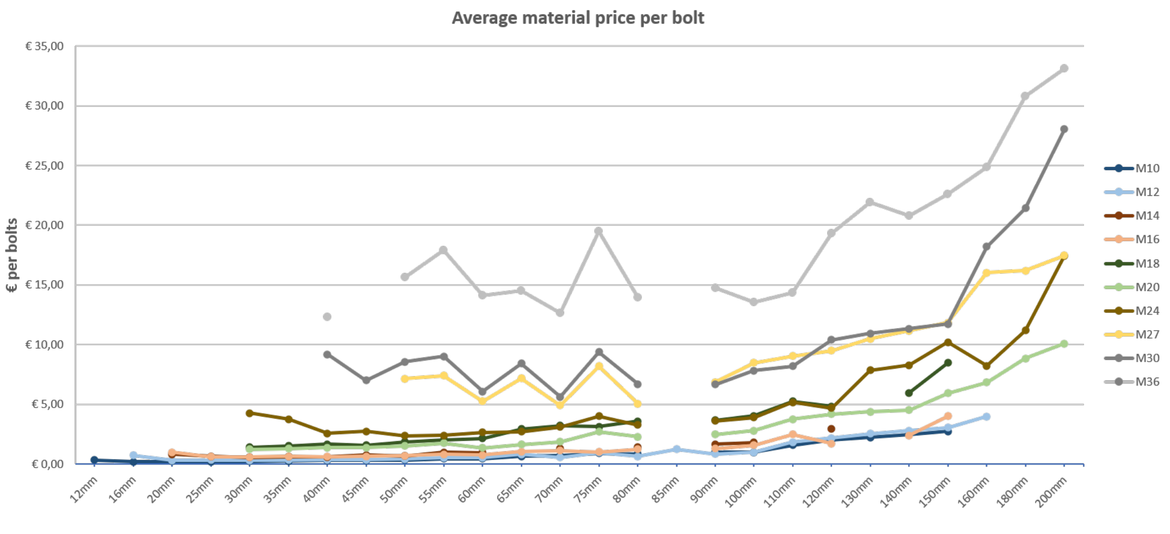


Figure D.25: Average material prices 10.9 steel bolts from table D.3

D.3. NUTS

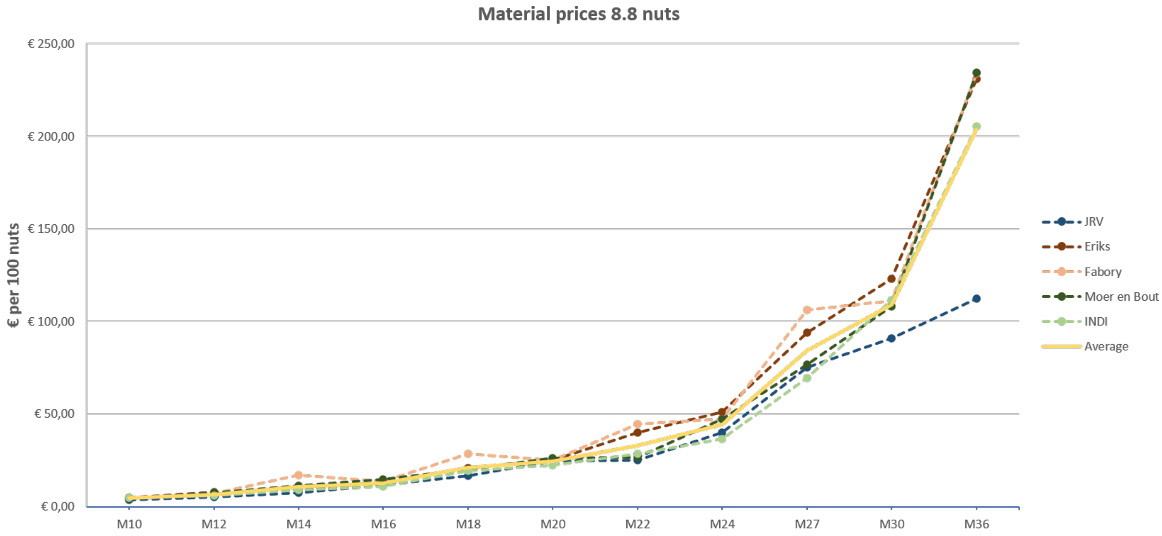


Figure D.26: Material prices 8.8 steel nuts [26] [27] [28] [29] [30]

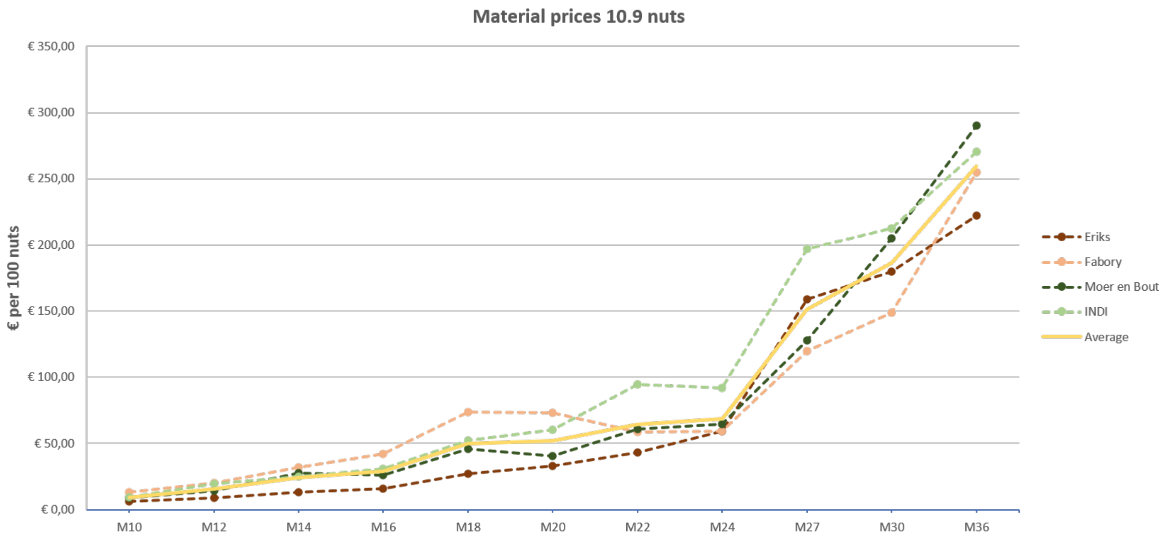


Figure D.27: Material prices 10.9 steel nuts [31] [32] [33] [34]

In the table below a list is given for the average costs of different nuts with a steel strength of 8.8 and 10.9.

Bolt type	M10	M12	M14	M16	M18	M20	M24	M27	M30	M36
8.8	€ 0.04	€ 0.07	€ 0.11	€ 0.13	€ 0.21	€ 0.25	€ 0.44	€ 0.84	€ 1.09	€ 2.03
10.9	€ 0.09	€ 0.16	€ 0.24	€ 0.29	€ 0.50	€ 0.52	€ 0.69	€ 1.51	€ 1.87	€ 2.59

Table D.4: Average costs per nut from figures: D.26, D.27

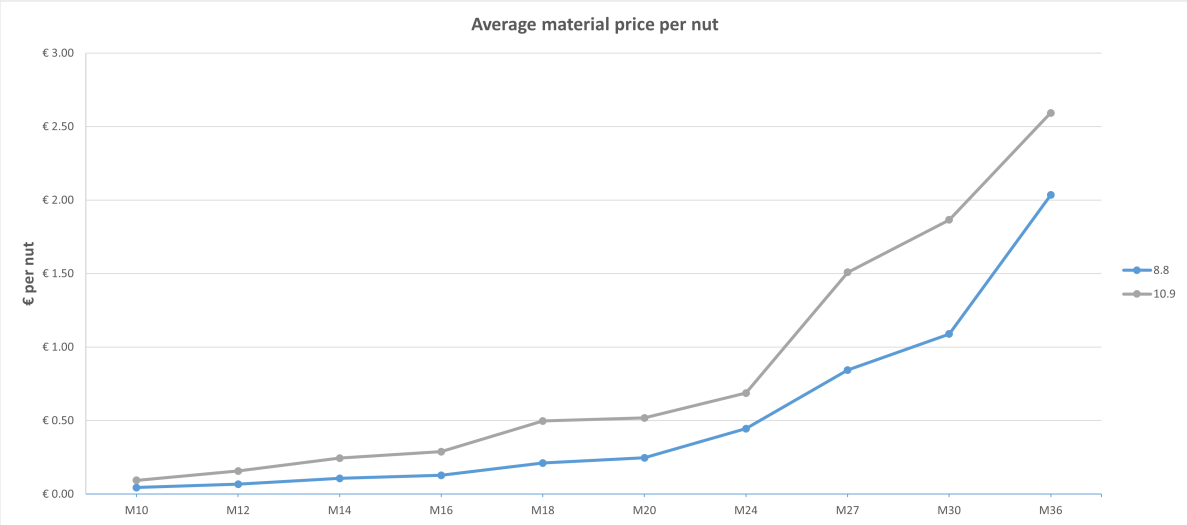


Figure D.28: Average material prices per nut from table D.4

D.4. WASHERS

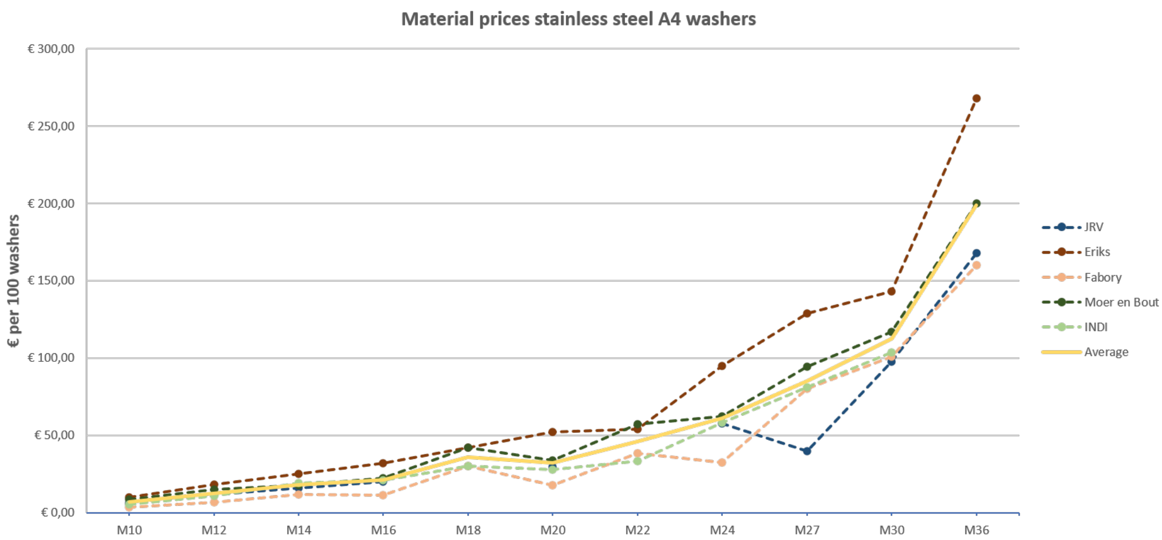


Figure D.29: Material prices stainless steel washers [35] [36] [37] [38] [39]

In the table below a list is given for the average costs of different washers.

M10	M12	M14	M16	M18	M20	M24	M27	M30	M36
€ 0.07	€ 0.12	€ 0.18	€ 0.21	€ 0.36	€ 0.32	€ 0.61	€ 0.85	€ 1.12	€ 1.99

Table D.5: Average costs per washer from figures: D.26

E

INTERMEDIATE STEPS OPTIMIZATION

In this appendix one loop is described including all the intermediate results.

The structure that is checked can be found in figure E.1. This is the same structure that is used for the cost optimization check, that can be found in section 5.2. The height of the structure is 5 m, the width is 10 m and the length is 30 m. The number of columns per frame are 2, the structure has 9 frames and 7 purlins. The beam column connection is set to the medium semi-rigid connection and the column base connection is set to a rigid connection. The loads used in this calculation are the same as used in the case study, see section 5.2.1.

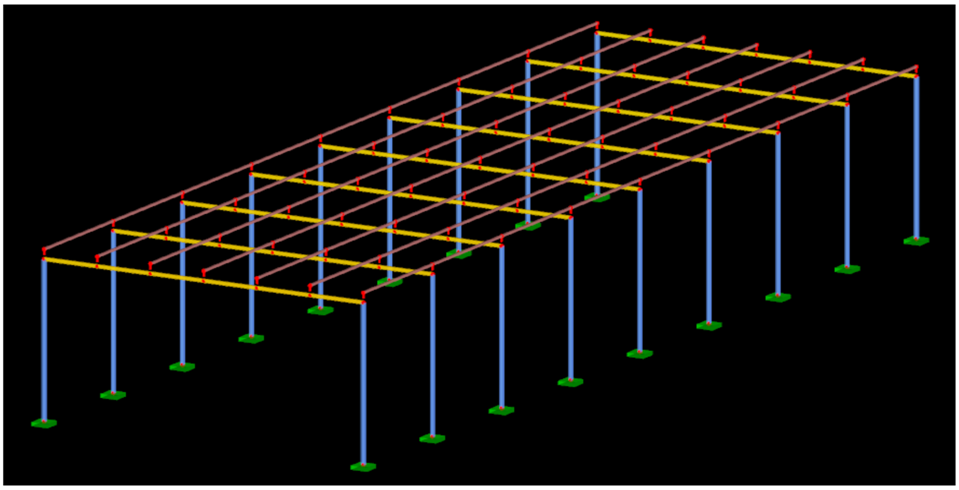


Figure E.1: Structure topology start

These parameters create the parametric model. This is send to RFEM and this structure is then optimized to costs, for the given connection parameter. This starts with the structural loop where the profiles are increased until all the beams have a unity check below 1.0. All intermediate steps for the profile sizes including the unity checks and the costs can be found in table E.1.

As can be seen in the table E.1 each step increases the costs. This means that as soon as all the unity checks are below the allowable unity check it is known that the cheapest possible structure is given. The final result can be found in figure E.2.

	Column	Beam	Purlin	UC-column	UC-beam	UC-purlin	Frame cost factor
1	HEA100	IPE80	UPE80	3,66	12,5	33,56	6,75
2	HEA120	IPE100	UPE80	2,29	5,81	15,77	7,52
3	HEB100	IPE120	UPE80	2,43	3,56	9,69	7,90
4	HEA140	IPE140	UPE80	1,51	2	5,63	8,89
5	HEA140	IPE160	UPE80	1,46	1,37	3,94	9,30
6	HEA140	IPE180	UPE80	1,42	1,01	2,97	9,73
7	HEA140	IPE200	UPE80	1,38	0,78	2,32	10,25
8	HEA140	IPE220	UPE80	1,35	0,62	1,9	10,78
9	HEB120	IPE220	UPE80	1,39	0,64	1,96	11,06
10	HEB120	IPE240	UPE80	1,39	0,52	1,67	11,75
11	HEA160	IPE240	UPE80	0,95	0,44	1,49	12,28
12	HEB140	IPE240	UPE80	0,88	0,45	1,52	12,71
13	HEB140	IPE270	UPE80	0,88	0,35	1,28	13,50
14	HEA180	IPE270	UPE80	0,71	0,31	1,19	13,80
15	HEB160	IPE270	UPE80	0,61	0,31	1,19	14,74
16	HEB160	IPE300	UPE80	0,6	0,25	1,03	15,65
17	HEA200	IPE300	UPE80	0,53	0,22	0,98	15,78

Table E.1: Intermediate steps frame optimization

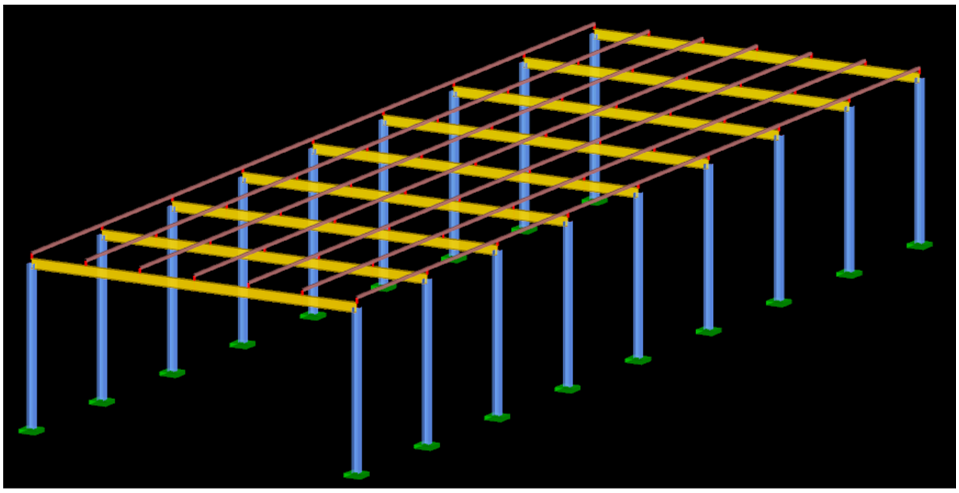


Figure E.2: Final structure topology

When the structural optimization is finished, the connection is optimized. For this connection optimization there is a list created from the connection database with all the connections that have a stiffness between 21535 kNm/rad and 26321 kNm/rad. For each connection in this list the costs of the connection is calculated and the list is sorted from lowest to highest cost. When this list is created, each connection from the list is checked in IdeaStatiCa and the component method with the governing forces taken from RFEM.

The different connections that are checked can be found in figures E.3 to E.7. The design variables for these connections, the unity checks and the actual stiffness are found in table E.2.

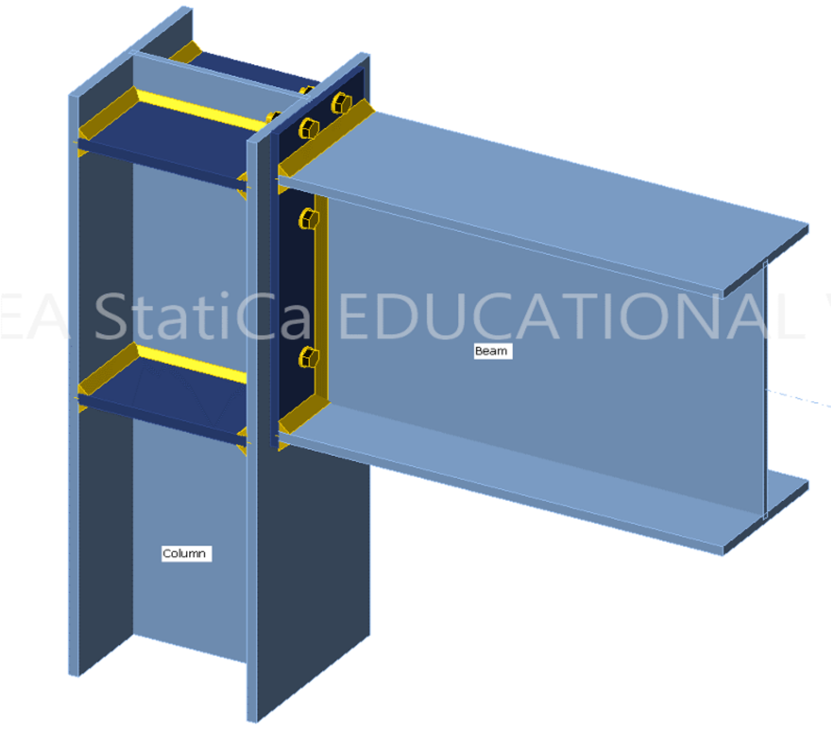


Figure E.3: Connection 1

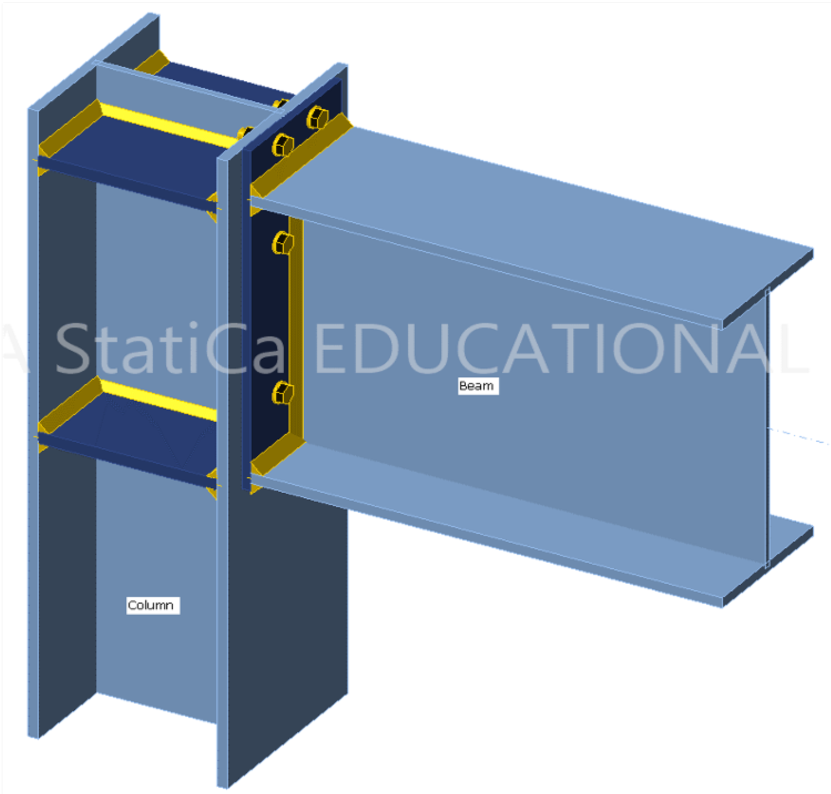


Figure E.4: Connection 2

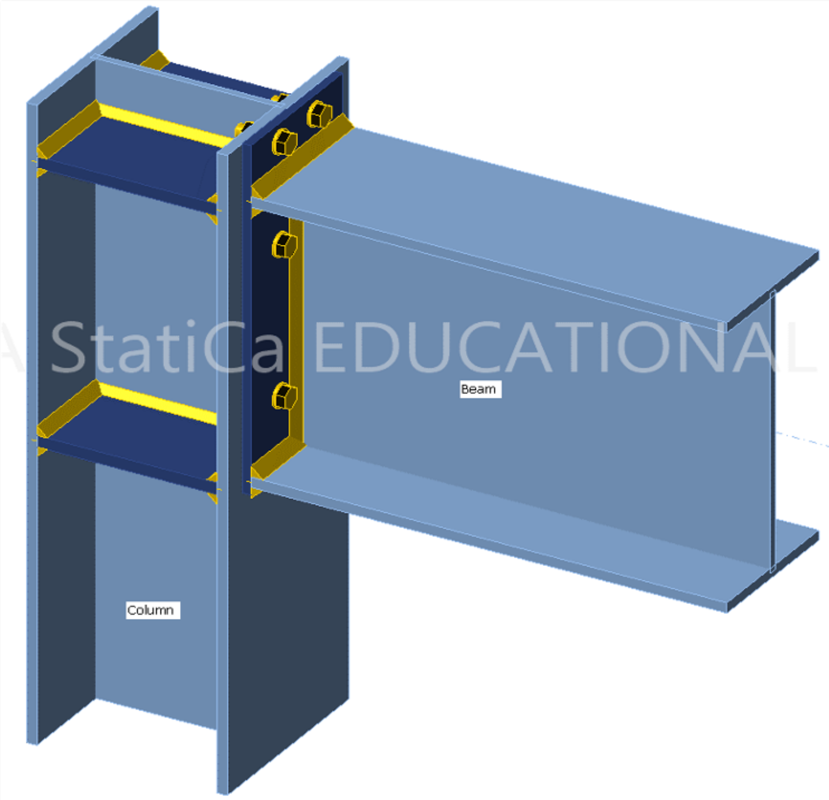


Figure E.5: Connection 3

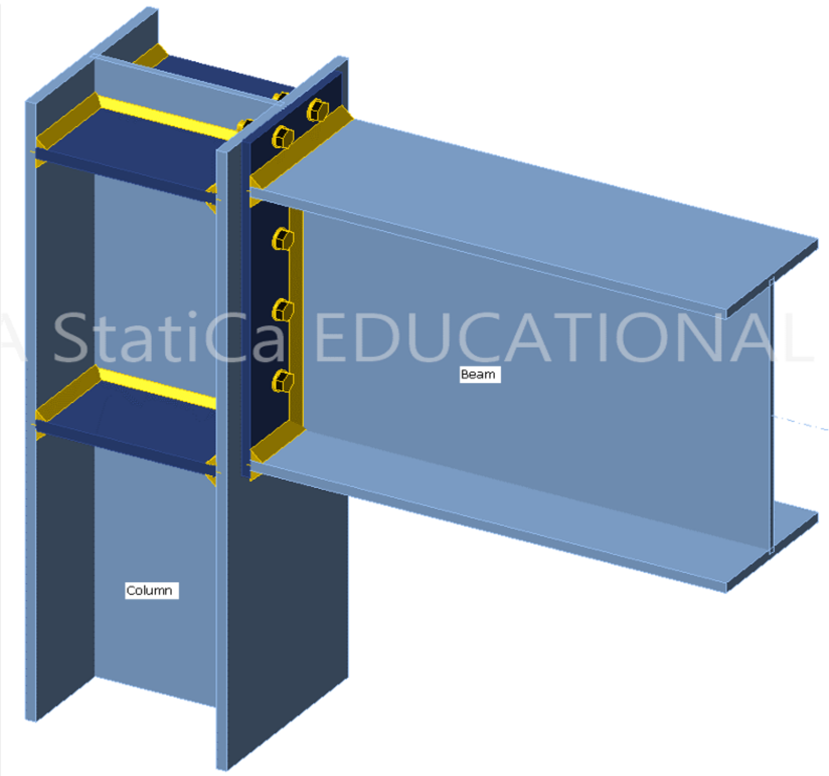


Figure E.6: Connection 4

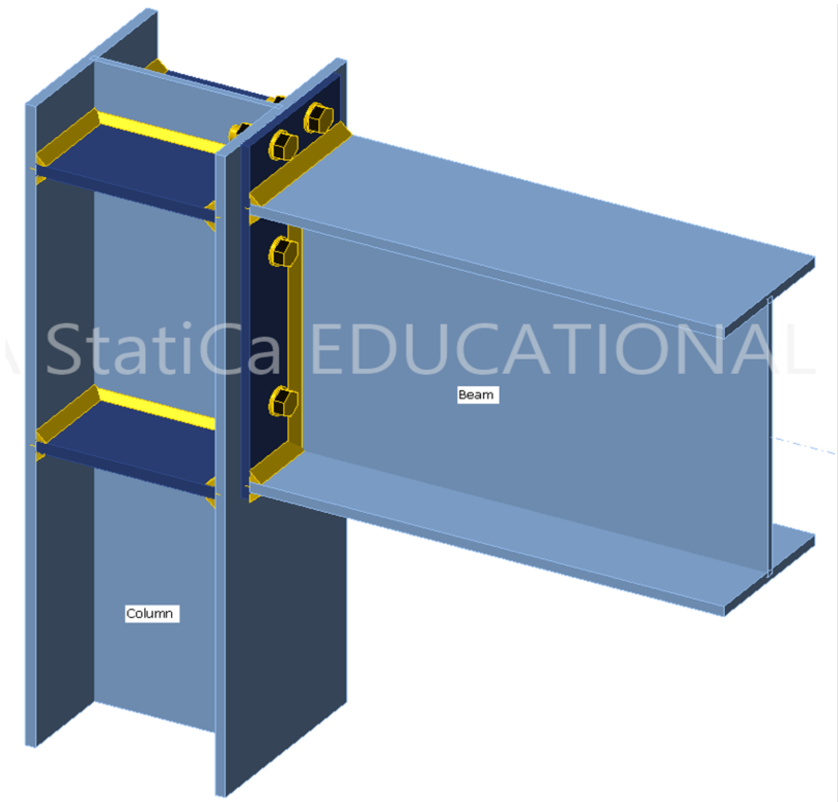


Figure E.7: Connection 5

	End plate thickness [mm]	Number of bolts	Stiffener thickness [mm]	Bolt	Strain %	Bolts UC	Welds UC	Stiffness [kNm/rad]
1	8	2	12	M12:8.8	0.0	0.64	0.53	33766
2	8	2	12	M12:10.9	0.0	0.51	0.53	33766
3	8	2	12	M14:8.8	0.0	0.48	0.52	29687
4	8	3	12	M12:8.8	0.0	0.62	0.53	32214
5	8	2	12	M16:8.8	0.0	0.37	0.65	26301

Table E.2: Intermediate steps connection optimization

The final cost factor of the connection is 0.20 per connection. The final cost factor of this optimization is 23.03. In table E.2 it can be seen that the stiffness goes down when the bolt size goes up this is not as expected when looked at section H.12. The reason for this drop in stiffness is due to an increasing m2 of the top bolt which increases the λ_2 which reduces the alpha in the calculation of the effective lengths and thus reduces the effective length. This reduction in effective length reduces the stiffness.

F

BENCHMARK CHECK EUROCODE SA

In this appendix the programmed Eurocode is checked with the build-in check from RFEM. This is done by checking all the results from RFEM with the results from the programmed code in the optimization process. In the case of significant differences between the two programs an explanation is given about these differences.

Below a figure of the checked structure can be found including the beam numbers used in the tables and graphs.

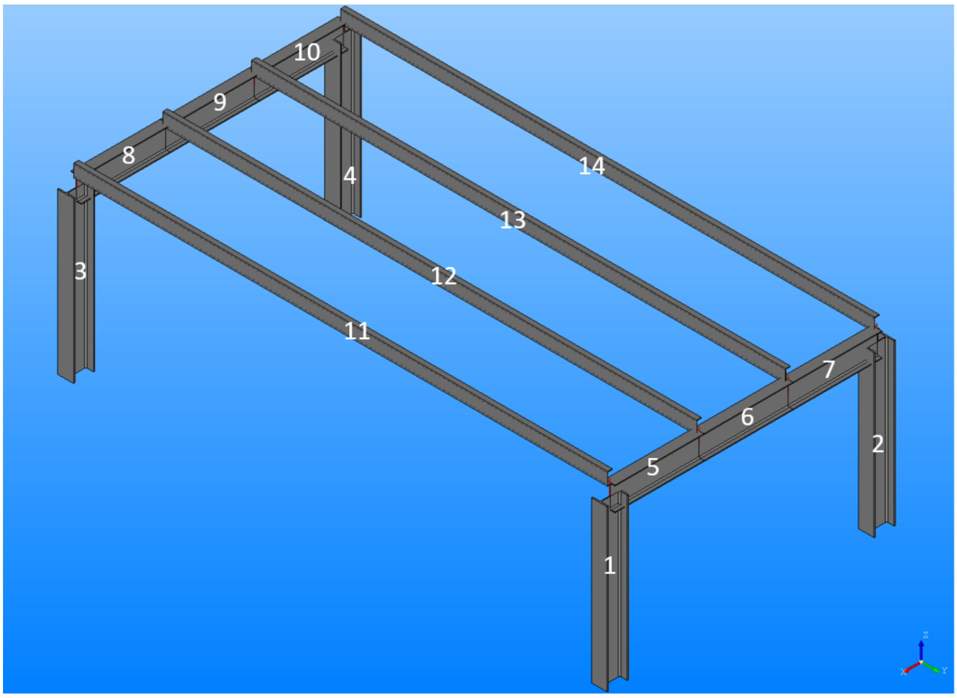


Figure F.1: Checked structure

The loads on the structure are the loads mentioned in section 3.2.3. The beam column connection and the column base connections are fully rigid in this check.

F.1. CROSS-SECTION CLASS

In this section a comparison is made for the cross section class calculation from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. This comparison can be found in tables F.1 and F.2. In these tables it can be found that there are no significant differences between the two programs. Cross section class 2, 3 and 4 are also checked but are not in this benchmark check.

RFEM		1	2	3	4	5	6	7	8	9	10	11	12	13	14
c_f	[mm]	116.3	116.3	116.3	116.3	69.3	69.3	69.3	69.3	69.3	69.3	68	68	68	68
t_f	[mm]	24	24	24	24	14.6	14.6	14.6	14.6	14.6	14.6	12.5	12.5	12.5	12.5
ε_f	[-]	0.814	0.814	0.814	0.814	0.814	0.814	0.814	0.814	0.814	0.814	1	1	1	1
$class_f$	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1
c_w	[mm]	298	298	298	298	378.8	378.8	378.8	378.8	378.8	378.8	185	185	185	185
t_w	[mm]	13.5	13.5	13.5	13.5	9.4	9.4	9.4	9.4	9.4	9.4	7	7	7	7
ε_w	[-]	0.814	0.814	0.814	0.814	0.814	0.814	0.814	0.814	0.814	0.814	1	1	1	1
$class_w$	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$class_{total}$	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Table F.1: RFEM EN checker, cross section class

Python		1	2	3	4	5	6	7	8	9	10	11	12	13	14
c_f	[mm]	116.25	116.25	116.25	116.25	69.3	69.3	69.3	69.3	69.3	69.3	68	68	68	68
t_f	[mm]	24	24	24	24	14.6	14.6	14.6	14.6	14.6	14.6	12.5	12.5	12.5	12.5
ε_f	[-]	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	1	1	1	1
$class_f$	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1
c_w	[mm]	298	298	298	298	378.8	378.8	378.8	378.8	378.8	3788	185	185	185	185
t_w	[mm]	13.5	13.5	13.5	13.5	9.4	9.4	9.4	9.4	9.4	9.4	7	7	7	7
ε_w	[-]	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	0.81	1	1	1	1
$class_w$	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$class_{total}$	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Table F.2: Implementation of EN checker in Python, cross section class

F.2. COMPRESSION

In this section a comparison is made for the compression check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. This comparison can be found in tables F.3 and F.4 and in figure F.2. In these tables and this figure there can be seen that there are no significant differences between the two programs. This is also checked for a structure with higher unity checks but they are not in this benchmark check.

RFEM		1	2	3	4	11	13	14
$N_{c,Ed}$	[kN]	15.98	18.46	15.98	18.46	2.99	12.04	2.02
A	[cm2]	197.80	197.80	197.80	197.80	38.50	38.50	38.50
f_y	[MPa]	345	345	345	345	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$N_{c,Rd}$	[kN]	6894.10	6894.10	6894.10	6894.10	904.75	904.75	904.75
UC	[-]	0.00	0.00	0.00	0.00	0.00	0.01	0.01

Table F.3: RFEM EN checker, compression check

Python		1	2	3	4	11	13	14
$N_{c,Ed}$	[kN]	15.98	18.46	15.98	18.46	2.99	12.04	2.02
A	[cm2]	197.78	197.78	197.78	197.78	38.50	38.50	38.50
f_y	[MPa]	345	345	345	345	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$N_{c,Rd}$	[kN]	6823.98	6823.98	6823.98	6823.98	904.75	904.75	904.75
UC	[-]	0.00	0.00	0.00	0.00	0.00	0.01	0.01

Table F.4: Implementation of EN checker in Python, compression check

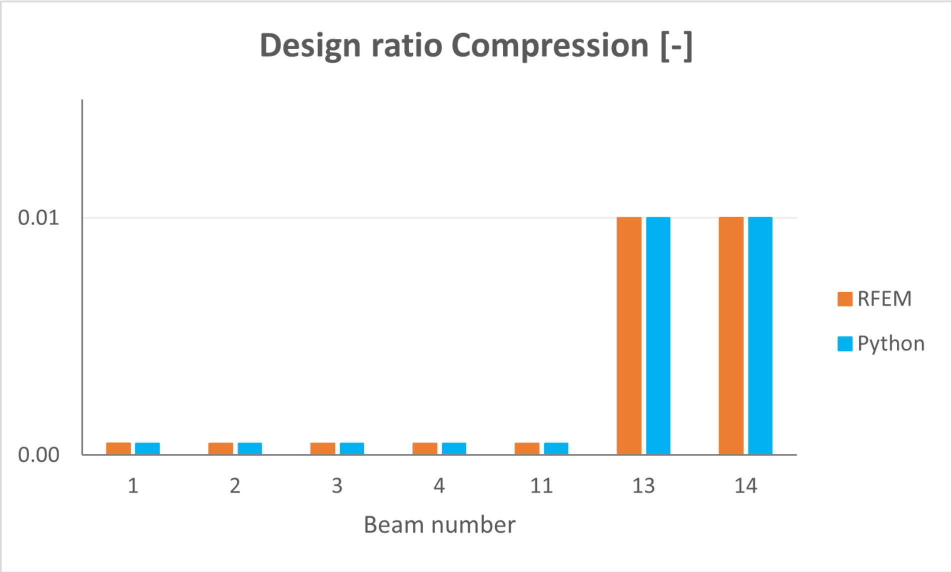


Figure F.2: Design ratios compression

F.3. TENSION

In this section a comparison is made for the tension check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. This comparison can be found in tables F.5 and F.6 and in figure F.3. In these tables and this figure there can be seen that there are no differences between the two programs. This is also checked for a structure with higher unity checks but they are not in this benchmark check.

RFEM		11	12	14
$N_{t,Ed}$	[kN]	2.03	7.74	2.68
A	[cm2]	38.5	38.5	38.5
f_y	[MPa]	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00
$N_{t,Rd}$	[kN]	904.75	904.75	904.75
UC	[-]	0.00	0.01	0.00

Table F.5: RFEM EN checker, tension check

Python		11	12	14
$N_{t,Ed}$	[kN]	2.03	7.74	2.68
A	[cm2]	38.5	38.5	38.5
f_y	[MPa]	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00
$N_{t,Rd}$	[kN]	904.75	904.75	904.75
UC	[-]	0.00	0.01	0.00

Table F.6: Implementation of EN checker in Python, tension check

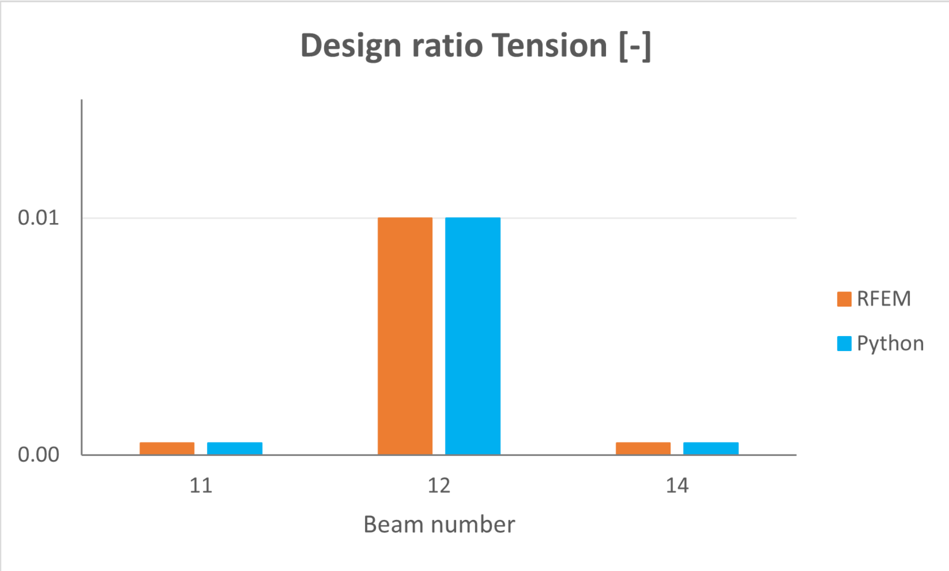


Figure F.3: Design ratios tension

F.4. SHEAR

In this section a comparison is made for the shear check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. This is done for shear in both directions. The comparison for the shear in the z-axis can be found in tables F.7 and F.8 and in figure F.4. The comparison for the shear in the y-axis can be found in tables F.9 and F.10 and in figure F.5. In these tables and figures there are no significant differences found between the two programs. This is also checked for a structure with higher unity checks but they are not in this benchmark check.

RFEM		1	2	3	4	5	6	7	8	9	10	11	12	13	14
$V_{z,Ed}$	[kN]	17.35	3.43	17.35	3.43	7.45	3.36	10.64	7.45	3.36	10.64	4.57	4.38	11.25	4.57
$A_{V,z}$	[cm2]	70.00	70.00	70.00	70.00	50.84	50.84	50.84	50.84	50.84	50.84	18.75	18.75	18.75	18.75
f_y	[MPa]	355	355	355	355	355	355	355	355	355	355	235	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$V_{pl,z,Rd}$	[kN]	1434.72	1434.72	1434.72	1434.72	1042.10	1042.10	1042.10	1042.10	1042.10	1042.10	254.39	254.39	254.39	254.39
UC	[-]	0.01	0.00	0.01	0.00	0.01	0.00	0.01	0.01	0.00	0.01	0.02	0.02	0.04	0.02

Table F.7: RFEM EN checker, shear check z-axis

Python		1	2	3	4	5	6	7	8	9	10	11	12	13	14
$V_{z,Ed}$	[kN]	17.35	3.43	17.35	3.43	7.45	3.36	10.64	7.45	3.36	10.64	4.57	4.38	11.25	4.57
$A_{V,z}$	[cm2]	69.98	69.98	69.98	69.98	50.84	50.84	50.84	50.84	50.84	50.84	18.75	18.75	18.75	18.75
f_y	[MPa]	355	355	355	355	355	355	355	355	355	355	235	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$V_{pl,z,Rd}$	[kN]	1434.30	1434.30	1434.30	1434.30	1042.10	1042.10	1042.10	1042.10	1042.10	1042.10	254.39	254.39	254.39	254.39
UC	[-]	0.01	0.00	0.01	0.00	0.01	0.00	0.01	0.01	0.00	0.01	0.02	0.02	0.04	0.02

Table F.8: Implementation of EN checker in Python, shear check z-axis

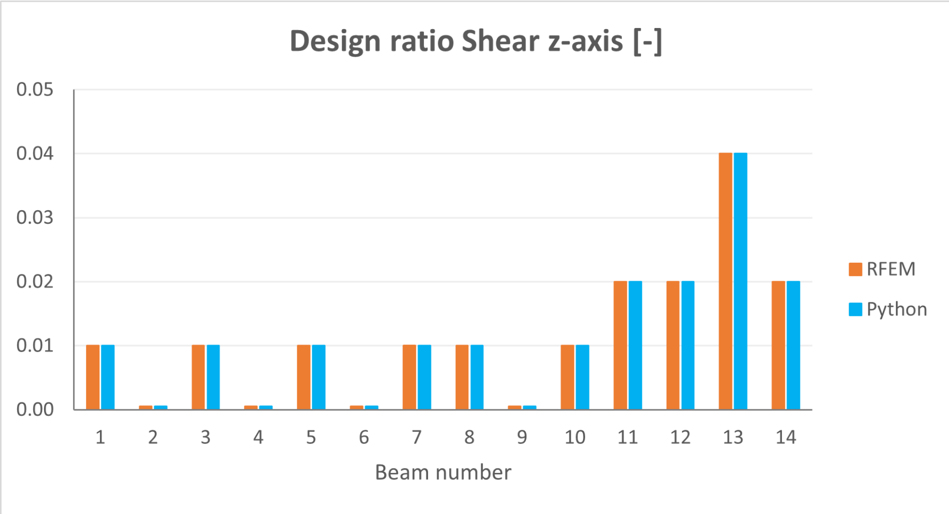


Figure F.4: Design ratios shear z-axis

RFEM		3	4	6	7	9	10
$V_{y,Ed}$	[kN]	11.80	11.80	6.53	5.51	6.53	5.51
$A_{V,y}$	[cm2]	149.47	149.47	58.34	58.34	58.34	58.34
f_y	[MPa]	345	345	355	355	355	355
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00
$V_{pl,y,Rd}$	[kN]	2977.18	2977.18	1195.68	1195.68	1195.68	1195.68
UC	[-]	0.00	0.00	0.01	0.01	0.01	0.01

Table F.9: RFEM EN checker, shear check y-axis

Python		3	4	6	7	9	10
$V_{y,Ed}$	[kN]	11.80	11.80	6.53	5.51	6.53	5.51
$A_{V,y}$	[cm2]	145.22	145.22	58.30	58.30	58.30	58.30
f_y	[MPa]	345	355	355	355	355	355
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00
$V_{pl,y,Rd}$	[kN]	2892.71	2892.71	1161.76	1161.76	1161.76	1161.76
UC	[-]	0.00	0.00	0.01	0.01	0.01	0.01

Table F.10: Implementation of EN checker in Python, shear check y-axis

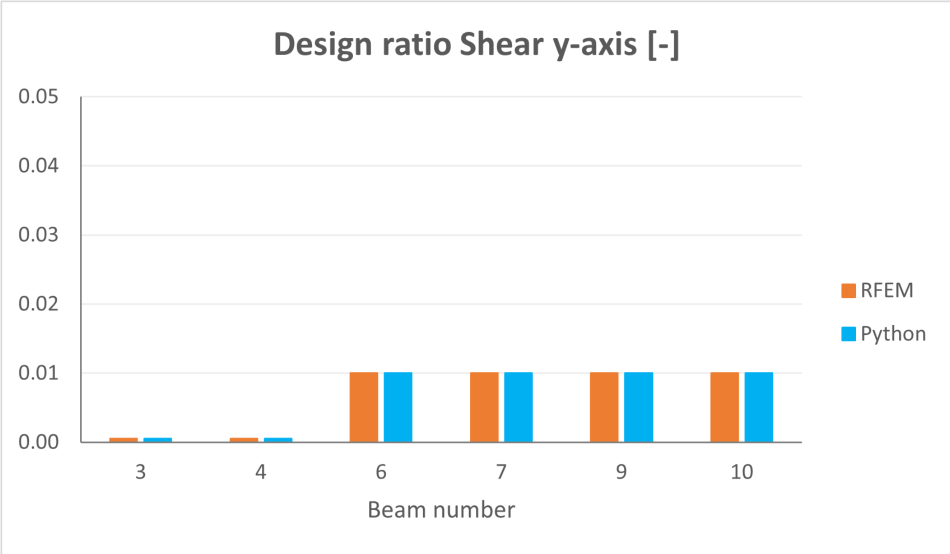


Figure F.5: Design ratios shear y-axis

F.5. BENDING AND SHEAR

In this section a comparison is made for the bending and bending + shear check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. This is done for bending and shear in both directions. The comparison for bending around the z-axis can be found in tables F.11 and F.12 and in figure F.6. The comparison for bending around the y-axis can be found in tables F.13 and F.14 and in figure F.7. The differences between the tables and figures are due to the used forces in the structure. In the RFEM code there is looked at the combination of the forces in a cross section of the beam and in the python code the biggest forces that are working in the beam are combined. This leads to a conservative check in the python code.

RFEM		1	2	3	4	5	6	7	8	9	10	11	14
$M_{z,Ed}$	[kNm]	4.28	6.54	12.40	12.40	1.12	1.09	4.03	1.12	1.09	4.03	0.10	0.23
$W_{pl,z}$	[cm3]	1104	1104	1104	1104	276	276	276	276	276	276	91	91
f_y	[MPa]	345	345	345	345	355	355	355	355	355	355	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,z,Rd}$	[kNm]	381.12	381.12	381.12	381.12	98.12	98.12	98.12	98.12	98.12	98.12	21.35	21.35
$V_{y,Ed}$	[kN]	1.73	3.36	11.80	11.80	0.75	0.00	5.46	0.75	0.00	5.46	0.00	0.00
$A_{V,y}$	[cm2]	149.47	149.47	149.47	149.47	58.34	58.34	58.34	58.34	58.34	58.34	23.45	23.45
$V_{pl,y,Rd}$	[kN]	2977.18	2977.18	2977.18	2977.18	1195.68	1195.68	1195.68	1195.68	1195.68	1195.68	318.16	318.16
$M_{c,z,Rd}$	[kNm]	381.12	381.12	381.12	381.12	98.12	98.12	98.12	98.12	98.12	98.12	21.35	21.35
UC	[-]	0.01	0.02	0.03	0.03	0.01	0.01	0.04	0.01	0.01	0.04	0.00	0.01

Table F.11: RFEM EN checker, bending and shear check z-axis

Python		1	2	3	4	5	6	7	8	9	10	11	14
$M_{z,Ed}$	[kNm]	6.53	6.54	12.40	12.40	2.13	8.54	8.58	2.13	8.54	8.58	0.12	0.30
$W_{pl,z}$	[cm3]	1096	1096	1096	1096	273	273	273	273	273	273	93	93
f_y	[MPa]	345	345	345	345	355	355	355	355	355	355	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,z,Rd}$	[kNm]	378.16	378.16	378.16	378.16	96.85	96.85	96.85	96.85	96.85	96.85	21.82	21.82
$V_{y,Ed}$	[kN]	3.36	3.36	11.80	11.80	1.36	6.53	5.51	1.36	6.53	5.51	0.00	0.01
$A_{V,y}$	[cm2]	149.00	149.00	149.00	149.00	58.30	58.30	58.30	58.30	58.30	58.30	23.50	23.50
$V_{pl,y,Rd}$	[kN]	2892.71	2892.71	2892.71	2892.71	1161.76	1161.76	1161.76	1161.76	1161.76	1161.76	309.13	309.13
$M_{c,z,Rd}$	[kNm]	378.16	378.16	378.16	378.16	96.85	96.85	96.85	96.85	96.85	96.85	21.82	21.82
UC	[-]	0.02	0.02	0.03	0.03	0.02	0.09	0.09	0.02	0.09	0.09	0.01	0.01

Table F.12: Implementation of EN checker in Python, bending and shear check z-axis

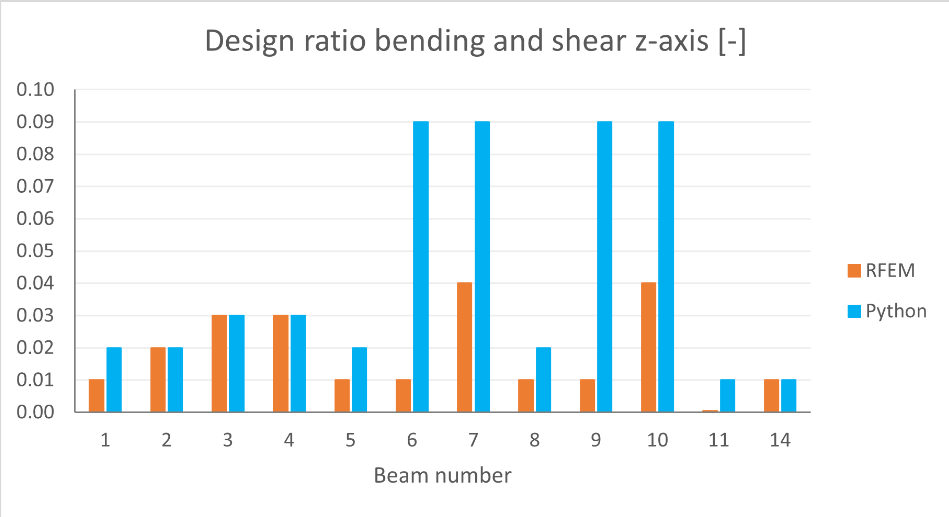


Figure F.6: Design ratios bending and shear z-axis

RFEM		2	4	5	7	8	10	11	12	13	14
$M_{y,Ed}$	[kNm]	4.64	4.64	7.20	4.63	7.20	4.63	7.88	10.41	10.41	7.88
$W_{pl,y}$	[cm3]	3232.00	3232.00	1702.00	1702.00	1702.00	1702.00	319.61	319.61	319.61	319.61
f_y	[MPa]	345	345	355	355	355	355	235	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	1119.22	1119.22	604.21	604.21	604.21	604.21	75.11	75.11	75.11	75.11
$V_{z,Ed}$	[kN]	2.78	2.78	7.45	5.62	7.45	5.62	4.57	0.00	0.00	4.57
$A_{V,z}$	[cm2]	70.00	70.00	50.84	50.84	50.84	50.84	18.75	18.75	18.75	18.75
$V_{pl,z,Rd}$	[kN]	1434.72	1434.72	1042.10	1042.10	1042.10	1042.10	254.39	254.39	254.39	254.39
$M_{c,y,Rd}$	[kNm]	1119.22	1119.22	604.21	604.21	604.21	604.21	75.11	75.11	75.11	75.11
UC	[-]	0.00	0.00	0.01	0.01	0.01	0.01	0.10	0.14	0.14	0.10

Table F.13: RFEM EN checker, bending and shear check y-axis

Python		2	4	5	7	8	10	11	12	13	14
$M_{y,Ed}$	[kNm]	8.14	8.14	7.20	8.48	7.20	8.48	7.88	10.41	24.12	7.88
$W_{pl,y}$	[cm3]	3125.00	3125.00	1623.92	1623.92	1623.92	1623.92	336.83	336.83	336.83	336.83
f_y	[MPa]	345	345	355	355	355	355	235	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	1078.34	1078.34	579.49	579.49	579.49	579.49	79.16	79.16	79.16	79.16
$V_{z,Ed}$	[kN]	3.43	3.43	7.45	10.64	7.45	10.64	4.57	4.38	11.25	4.57
$A_{V,z}$	[cm2]	69.98	69.98	50.84	50.84	50.84	50.84	18.75	18.75	18.75	18.75
$V_{pl,z,Rd}$	[kN]	1434.31	1434.31	1042.10	1042.10	1042.10	1042.10	254.39	254.39	254.39	254.39
$M_{c,y,Rd}$	[kNm]	1078.34	1078.34	579.49	579.49	579.49	579.49	79.16	79.16	79.16	79.16
UC	[-]	0.01	0.01	0.01	0.01	0.01	0.01	0.10	0.13	0.30	0.10

Table F.14: Implementation of EN checker in Python, bending and shear check y-axis

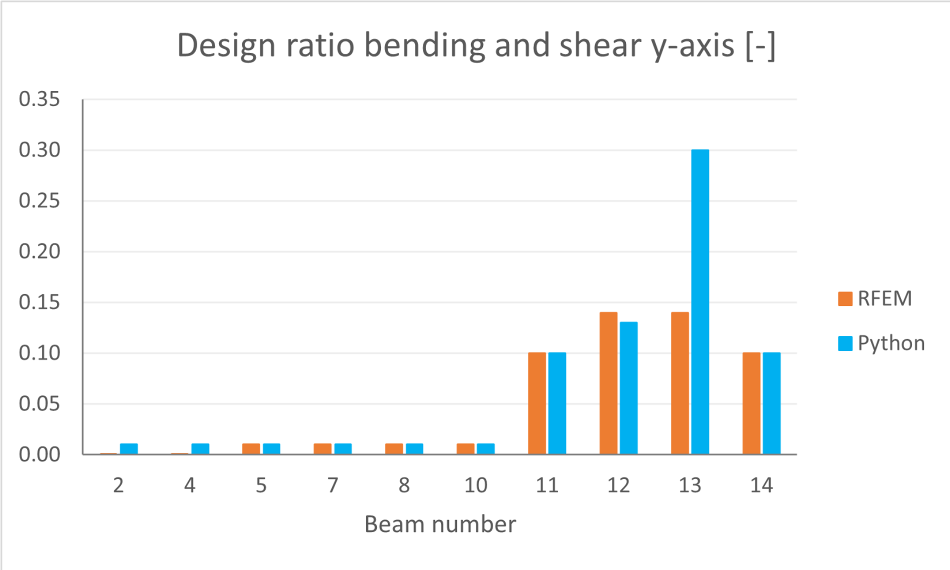


Figure F.7: Design ratios bending and shear y-axis

F.6. BENDING, SHEAR AND AXIAL

In this section a comparison is made for the bending, shear and axial check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. This is done for bending and shear in both directions. The comparison for bending around the y-axis can be found in tables F.15 and F.16 and in figure F.8. The comparison for bending around the z-axis can be found in tables F.17 and F.18 and in figure F.9. The differences between the tables and figures are due to the used forces in the structure. In the RFEM code there is looked at the combination of the forces in a cross section of the beam and in the python code the biggest forces that are working in the beam are combined. This leads to a conservative check in the python code.

RFEM		11	12	13	14
$M_{y,Ed}$	[kNm]	7.01	8.98	18.9	7.01
$W_{pl,y}$	[cm3]	319.61	319.61	320	319.61
f_y	[MPa]	235	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	75.11	75.11	75.1	75.11
$V_{z,Ed}$	[kN]	4.39	0	0	4.39
$A_{V,z}$	[cm2]	18.75	18.75	18.8	18.75
$V_{pl,z,Rd}$	[kN]	254.39	254.39	254	254.39
$M_{c,y,Rd}$	[kNm]	75.11	75.11	75.1	75.11
N_{Ed}	[kN]	2.03	7.73	9.02	2.03
A	[cm2]	38.5	38.5	38.5	38.5
$N_{pl,Rd}$	[kN]	904.75	904.75	905	904.75
UC	[-]	0.10	0.13	0.26	0.10

Table F.15: RFEM EN checker, bending, shear and axial check y-axis

Python		11	12	13	14
$M_{y,Ed}$	[kNm]	7.88	10.41	24.12	7.88
$W_{pl,y}$	[cm3]	336.83	336.83	336.83	336.83
f_y	[MPa]	235	235	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	79.16	79.16	79.16	79.16
$V_{z,Ed}$	[kN]	4.57	4.38	11.25	4.57
$A_{V,z}$	[cm2]	18.75	18.75	18.75	18.75
$V_{pl,z,Rd}$	[kN]	254.39	254.39	254.39	254.39
$M_{c,y,Rd}$	[kNm]	79.16	79.16	79.16	79.16
N_{Ed}	[kN]	2.99	7.74	12.04	2.68
A	[cm2]	38.5	38.5	38.5	38.5
$N_{pl,Rd}$	[kN]	904.75	904.75	904.75	904.75
UC	[-]	0.10	0.13	0.31	0.10

Table F.16: Implementation of EN checker in Python, bending, shear and axial check y-axis

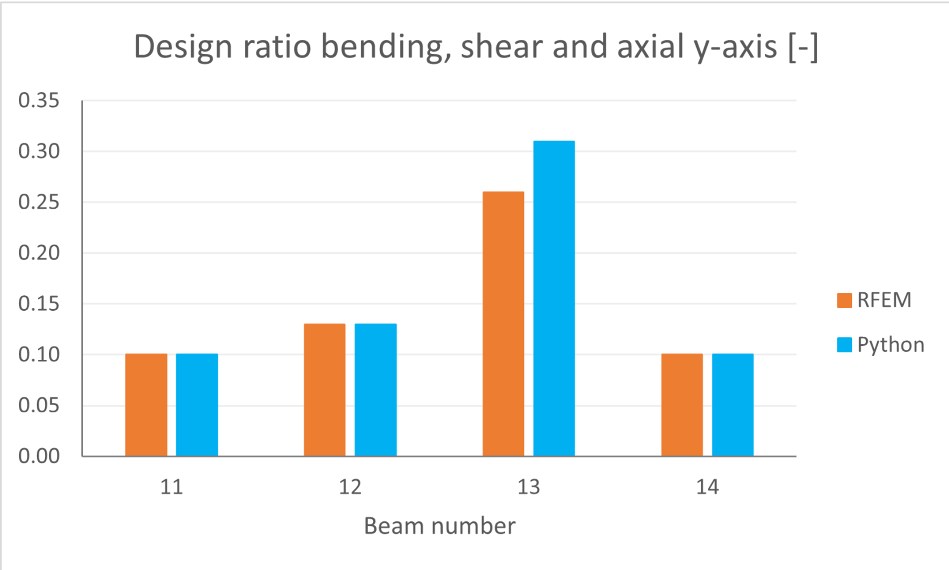


Figure F.8: Design ratios bending, shear and axial y-axis

RFEM		1	2	3	4	11	14
$M_{z,Ed}$	[kNm]	3.53	3.53	3.53	3.53	0.12	0.29
$W_{pl,z}$	[cm3]	1104	1104	1104	1104	90.86	90.86
f_y	[MPa]	345	345	345	345	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,z,Rd}$	[kNm]	381.12	381.12	381.1	381.12	21.35	21.35
$V_{y,Ed}$	[kN]	3.35	3.35	3.35	3.35	0	0
$A_{V,y}$	[cm2]	149.47	149.47	149.5	149.47	23.45	23.45
$V_{pl,y,Rd}$	[kN]	2977.18	2977.18	2977	2977.18	318.16	318.16
$M_{cr,z,Rd}$	[kNm]	381.12	381.12	381.1	381.12	21.35	21.35
N_{Ed}	[kN]	15.98	15.98	15.98	15.98	2.03	2.68
A	[cm2]	197.8	197.8	197.8	197.8	38.5	38.5
$N_{pl,Rd}$	[kN]	6894.1	6894.1	6894	6894.1	904.75	904.75
UC	[-]	0.01	0.01	0.01	0.01	0.01	0.02

Table F.17: RFEM EN checker, bending, shear and axial check z-axis

Python		1	2	3	4	11	14
$M_{z,Ed}$	[kNm]	6.53	6.54	12.40	12.40	0.12	0.30
$W_{pl,z}$	[cm3]	1096	1096	1096	1096	92.85	92.85
f_y	[MPa]	345	345	345	345	235	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,z,Rd}$	[kNm]	378.16	378.16	378.16	378.16	21.82	21.82
$V_{y,Ed}$	[kN]	3.36	3.36	11.8	11.8	0	0.01
$A_{V,y}$	[cm2]	149	149	149	149	23.5	23.5
$V_{pl,y,Rd}$	[kN]	2892.71	2892.71	2892.71	2892.71	309.13	309.13
$M_{cr,z,Rd}$	[kNm]	378.16	378.16	378.16	378.16	21.82	21.82
N_{Ed}	[kN]	15.98	18.46	15.98	18.46	2.99	2.68
A	[cm2]	197.78	197.78	197.78	197.78	38.50	38.50
$N_{pl,Rd}$	[kN]	6823.4	6823.4	6823.4	6823.4	904.75	904.75
UC	[-]	0.02	0.02	0.03	0.03	0.01	0.01

Table F.18: Implementation of EN checker in Python, bending, shear and axial check z-axis

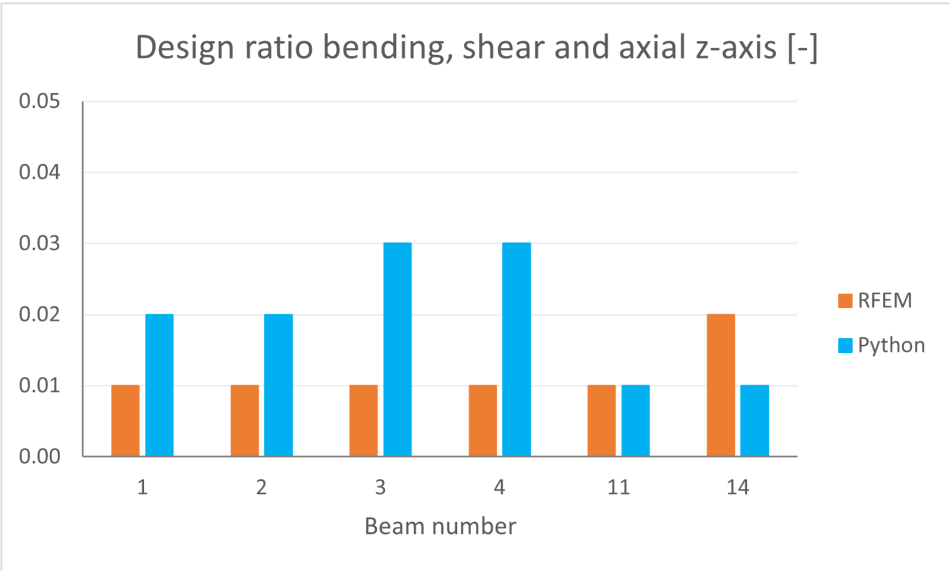


Figure F.9: Design ratios bending, shear and axial z-axis

F.7. BI-AXIAL BENDING AND SHEAR

In this section a comparison is made for the bi-axial bending and shear check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. The comparison for this check can be found in tables F.19 and F.20 and in figures F.10, F.11 and F.12. In these figures the design ratios are compared for the bending around the y-axis, bending around the z-axis and bi-axial bending. The differences between the tables and figures are due to the used forces in the structure. In the RFEM code there is looked at the combination of the forces in a cross section of the beam and in the python code the biggest forces that are working in the beam are combined. This leads to a conservative check in the python code.

RFEM		1	2	3	4	5	6	7	8	9	10	14
$M_{y,Ed}$	[kNm]	5.66	5.37	2.78	2.78	4.02	8.48	8.48	4.02	8.48	8.48	3.88
$f_{y,f}$	[MPa]	345	345	345	345	355	355	355	355	355	355	235
$f_{y,w}$	[MPa]	355	355	355	355	355	355	355	355	355	355	235
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	1119.22	1119	1119	1119.22	604.21	604.2	604.2	604.21	604.2	604.21	75.11
$V_{z,Ed}$	[kN]	2.78	2.4	1.24	1.24	6.02	1.98	9.27	6.02	1.98	9.27	2.45
$A_{V,z}$	[cm2]	70	70	70	70	50.84	50.84	50.84	50.84	50.84	50.84	18.75
$V_{pl,z,Rd}$	[kN]	1434.72	1435	1435	1434.72	1042.1	1042	1042	1042.1	1042	1042.1	254.39
$M_{z,Ed}$	[kNm]	6.53	6.51	7.89	7.89	2.13	8.54	8.58	2.13	8.54	8.58	0.23
$W_{pl,z}$	[cm3]	1104	1104	1104	1104	276.4	276.4	276.4	276.4	276.4	276.4	90.86
$M_{pl,z,Rd}$	[kNm]	381.12	381.1	381	381.12	98.12	98.12	98.12	98.12	98.12	98.12	21.35
$V_{y,Ed}$	[kN]	3.36	3.35	1.73	1.73	1.21	6.52	5.41	1.21	6.52	5.41	0
$A_{V,y}$	[cm2]	149.47	149.5	149	149.47	58.34	58.34	58.34	58.34	58.34	58.34	23.45
$V_{pl,y,Rd}$	[kN]	2977.18	2977	2977	2977.18	1195.68	1196	1196	1195.68	1196	1195.68	318.16
α	[-]	2	2	2	2	2	2	2	2	2	2	1
β	[-]	1	1	1	1	1	1	1	1	1	1	1
UC_y	[-]	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05
UC_z	[-]	0.02	0.02	0.02	0.02	0.02	0.09	0.09	0.02	0.09	0.09	0.01
UC_{bi}	[-]	0.02	0.02	0.02	0.02	0.02	0.09	0.09	0.02	0.09	0.09	0.06

Table F.19: RFEM EN checker, bi-axial bending and shear check

Python		1	2	3	4	5	6	7	8	9	10	14
$M_{y,Ed}$	[kNm]	16.20	8.14	16.20	8.14	7.20	8.48	8.48	7.20	8.48	8.48	7.88
$f_{y,f}$	[MPa]	345.00	345.00	345.00	345.00	355.00	355.00	355.00	355.00	355.00	355.00	235.00
$f_{y,w}$	[MPa]	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00	355.00	235.00
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	1096.04	1096.04	1096.04	1096.04	576.49	576.49	576.49	576.49	576.49	576.49	79.16
$V_{z,Ed}$	[kN]	17.35	3.43	17.35	3.43	7.45	3.36	10.64	7.45	3.36	10.64	4.57
$A_{V,z}$	[cm2]	69.98	69.98	69.98	69.98	50.84	50.84	50.84	50.84	50.84	50.84	18.75
$V_{pl,z,Rd}$	[kN]	1434.31	1434.31	1434.31	1434.31	1042.10	1042.10	1042.10	1042.10	1042.10	1042.10	254.39
$M_{z,Ed}$	[kNm]	6.53	6.54	6.54	6.54	2.13	8.54	8.58	2.13	8.54	8.58	0.30
$W_{pl,z}$	[cm3]	1096.04	1096.04	1096.04	1096.04	272.83	272.83	272.83	272.83	272.83	272.83	92.85
$M_{pl,z,Rd}$	[kNm]	378.16	378.16	378.16	378.16	96.85	96.85	96.85	96.85	96.85	96.85	21.82
$V_{y,Ed}$	[kN]	3.36	3.36	11.80	11.80	1.36	6.53	5.51	1.36	6.53	5.51	0.01
$A_{V,y}$	[cm2]	149.00	149.00	149.00	149.00	58.30	58.30	58.30	58.30	58.30	58.30	23.50
$V_{pl,y,Rd}$	[kN]	2892.71	2892.71	2892.71	2892.71	1161.76	1161.76	1161.76	1161.76	1161.76	1161.76	309.13
α	[-]	2	2	2	2	2	2	2	2	2	2	2
β	[-]	1	1	1	1	1	1	1	1	1	1	1
UC_y	[-]	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.10
UC_z	[-]	0.02	0.02	0.03	0.03	0.02	0.09	0.09	0.02	0.09	0.09	0.01
UC_{bi}	[-]	0.02	0.02	0.03	0.03	0.02	0.09	0.09	0.02	0.09	0.09	0.10

Table F.20: Implementation of EN checker in Python, bi-axial bending and shear check

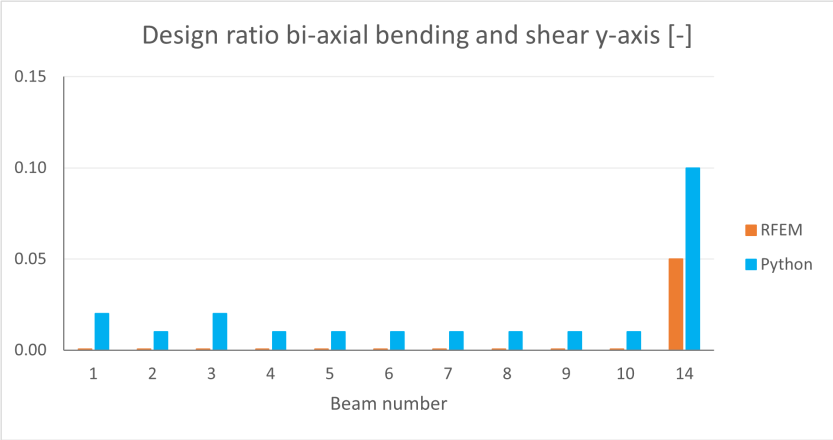


Figure F.10: Design ratios bi-axial bending and shear y-axis

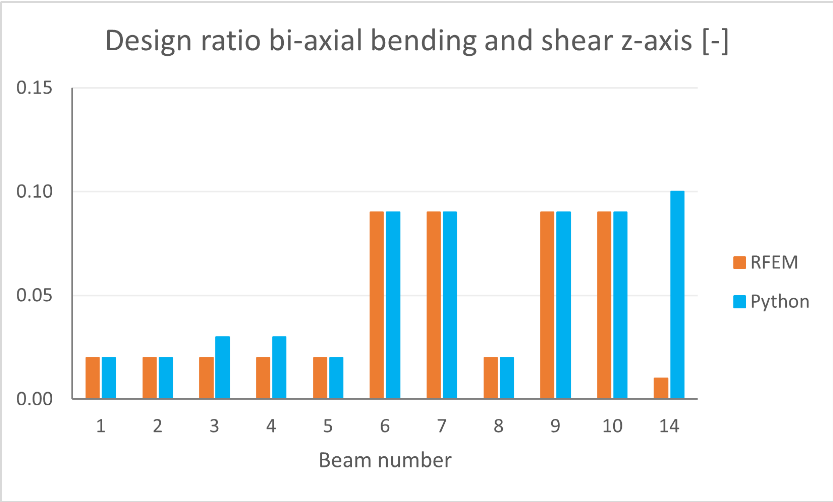


Figure F.11: Design ratios bi-axial bending and shear z-axis

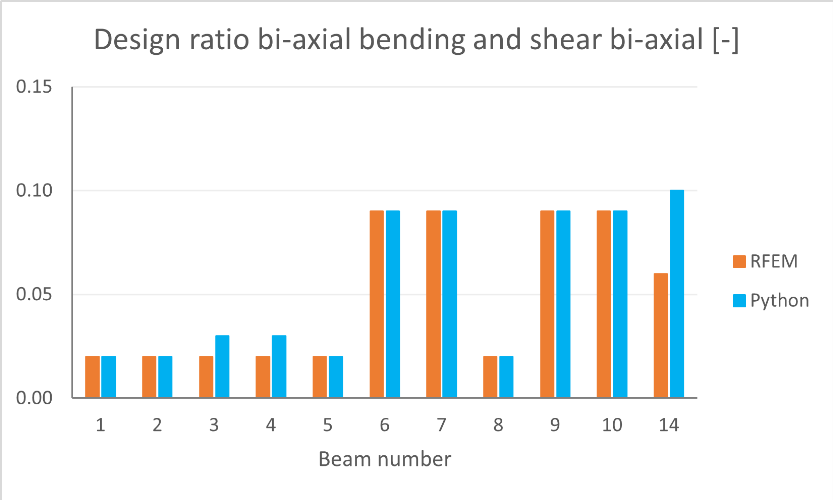


Figure F.12: Design ratios bi-axial bending and shear bi-axial

F.8. BI-AXIAL BENDING, SHEAR AND AXIAL

In this section a comparison is made for the bi-axial bending, shear and axial check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. The comparison for this check can be found in tables F.21 and F.22 and in figures F.13, F.14 and F.15. In these figures the design ratios are compared for the bending around the y-axis, bending around the z-axis and bi-axial bending. The differences between the tables and figures are due to the used forces in the structure. In the RFEM code there is looked at the combination of the forces in a cross section of the beam and in the python code the biggest forces that are working in the beam are combined. This leads to a conservative check in the python code.

RFEM		1	2	3	4	14
$M_{y,Ed}$	[kNm]	3.08	4.72	3.08	4.72	4.26
$W_{pl,y}$	[cm3]	3232.00	3232.00	3232.00	3232.00	319.61
$f_{y,w}$	[MPa]	355.00	355.00	355.00	355.00	235.00
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	1119.22	1119.22	1119.22	1119.22	75.11
$V_{z,Ed}$	[kN]	3.43	3.42	3.43	3.42	2.69
$A_{v,z}$	[cm2]	70.00	70.00	70.00	70.00	18.75
$V_{pl,z,Rd}$	[kN]	1434.72	1434.72	1434.72	1434.72	254.39
ρ	[-]	0.00	0.00	0.00	0.00	0.00
N_{Ed}	[kN]	15.27	15.04	15.27	15.04	2.68
A	[cm2]	197.80	197.80	197.80	197.80	38.50
$N_{pl,Rd}$	[kN]	6894.10	6894.10	6894.10	6894.10	904.75
a_1	[-]	0.27	0.27	0.27	0.27	0.42
a_2	[-]	0.27	0.27	0.27	0.27	0.42
$N_{vz,Rd}$	[kN]	6894.10	6894.10	6894.10	6894.10	904.75
$M_{y,V,Rd}$	[kNm]	1119.22	1119.22	1119.22	1119.22	75.11
$M_{y,N,V,Rd}$	[kNm]	1119.22	1119.22	1119.22	1119.22	75.11
$M_{z,Ed}$	[kNm]	1.86	2.57	1.86	2.57	0.30
$W_{pl,z}$	[cm3]	1104.00	1104.00	1104.00	1104.00	90.86
$f_{y,f}$	[MPa]	345.00	345.00	345.00	345.00	235.00
$W_{pl,z,Rd}$	[cm3]	381.12	381.12	381.12	381.12	90.86
$V_{y,Rd}$	[kN]	1.81	2.83	1.81	2.83	0.01
$A_{V,y}$	[cm2]	149.47	149.47	149.47	149.47	23.45
$V_{pl,y,Rd}$	[kN]	2977.18	2977.18	2977.18	2977.18	318.16
$M_{pl,z,V,Rd}$	[kNm]	381.12	381.12	381.12	381.12	21.35
$N_{vy,Rd}$	[kN]	6894.10	6894.10	6894.10	6894.10	904.75
$M_{z,V,Rd}$	[kNm]	381.12	381.12	381.12	381.12	21.35
$M_{z,N,V,Rd}$	[kNm]	328.77	328.77	328.77	328.77	21.07
α_1	[-]	2	2	2	2	1
α_2	[-]	2	2	2	2	1
β_1	[-]	1	1	1	1	1
β_2	[-]	1	1	1	1	1
UC_y	[-]	0.00	0.00	0.00	0.00	0.06
UC_z	[-]	0.00	0.00	0.00	0.00	0.01
UC_{bi}	[-]	0.00	0.00	0.00	0.00	0.07

Table F.21: RFEM EN checker, bi-axial bending, shear and axial check

Python		1	2	3	4	14
$M_{y,Ed}$	[kNm]	16.20	8.14	16.20	8.14	7.88
$W_{pl,y}$	[cm3]	3125.00	3125.00	3125.00	3125.00	336.83
$f_{y,w}$	[MPa]	355.00	355.00	355.00	355.00	235.00
γ_{M0}	[-]	1.00	1.00	1.00	1.00	1.00
$M_{pl,y,Rd}$	[kNm]	1078.34	1078.34	1078.34	1078.34	79.16
$V_{z,Ed}$	[kN]	17.35	3.43	17.35	3.43	4.57
$A_{V,z}$	[cm2]	69.98	69.98	69.98	69.98	18.75
$V_{pl,z,Rd}$	[kN]	1434.30	1434.30	1434.30	1434.30	254.39
ρ	[-]	0.00	0.00	0.00	0.00	0.00
N_{Ed}	[kN]	15.98	18.46	15.98	18.46	2.68
A	[cm2]	197.78	197.78	197.78	197.78	38.50
$N_{pl,Rd}$	[kN]	6823.98	6823.98	6823.98	6823.98	904.75
a_1	[-]	0.27	0.27	0.27	0.27	0.42
a_2	[-]	0.27	0.27	0.27	0.27	0.42
$N_{vz,Rd}$	[kN]	6823.98	6823.98	6823.98	6823.98	904.75
$M_{y,V,Rd}$	[kNm]	1078.34	1078.34	1078.34	1078.34	79.16
$M_{y,N,V,Rd}$	[kNm]	1078.34	1078.34	1078.34	1078.34	79.15
$M_{z,Ed}$	[kNm]	6.53	6.54	12.40	12.40	0.30
$W_{pl,z}$	[cm3]	1096.00	1096.00	1096.00	1096.00	92.85
$f_{y,f}$	[MPa]	345.00	345.00	345.00	345.00	235.00
$W_{pl,z,Rd}$	[cm3]	378.16	378.16	378.16	378.16	92.85
$V_{y,Ed}$	[kN]	3.36	3.36	11.80	11.80	0.01
$A_{V,y}$	[cm2]	149.00	149.00	149.00	149.00	23.45
$V_{pl,y,Rd}$	[kN]	2892.71	2892.71	2892.71	2892.71	309.13
$M_{pl,z,V,Rd}$	[kNm]	378.16	378.16	378.16	378.16	21.82
$N_{vy,Rd}$	[kN]	6823.98	6823.98	6823.98	6823.98	904.75
$M_{z,V,Rd}$	[kNm]	378.16	378.16	378.16	378.16	21.82
$M_{z,N,V,Rd}$	[kNm]	378.16	378.16	378.16	378.16	12.35
α_1	[-]	2	2	2	2	2
α_2	[-]	2	2	2	2	2
β_1	[-]	1	1	1	1	1
β_2	[-]	1	1	1	1	1
UC_y	[-]	0.02	0.01	0.02	0.01	0.10
UC_z	[-]	0.02	0.02	0.03	0.03	0.01
UC_{bi}	[-]	0.02	0.02	0.03	0.03	0.10

Table F.22: Implementation of EN checker in Python, bi-axial bending, shear and axial check

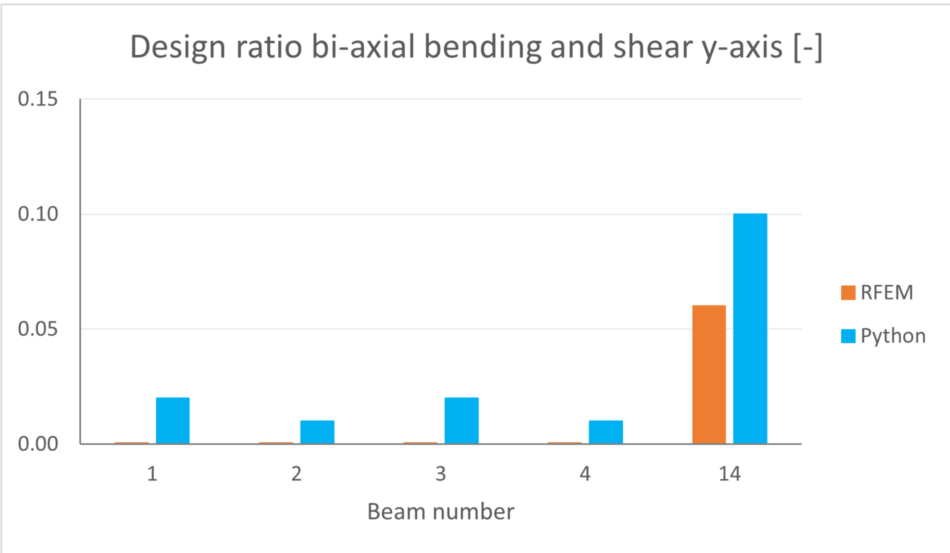


Figure F.13: Design ratios bi-axial bending, shear and axial y-axis

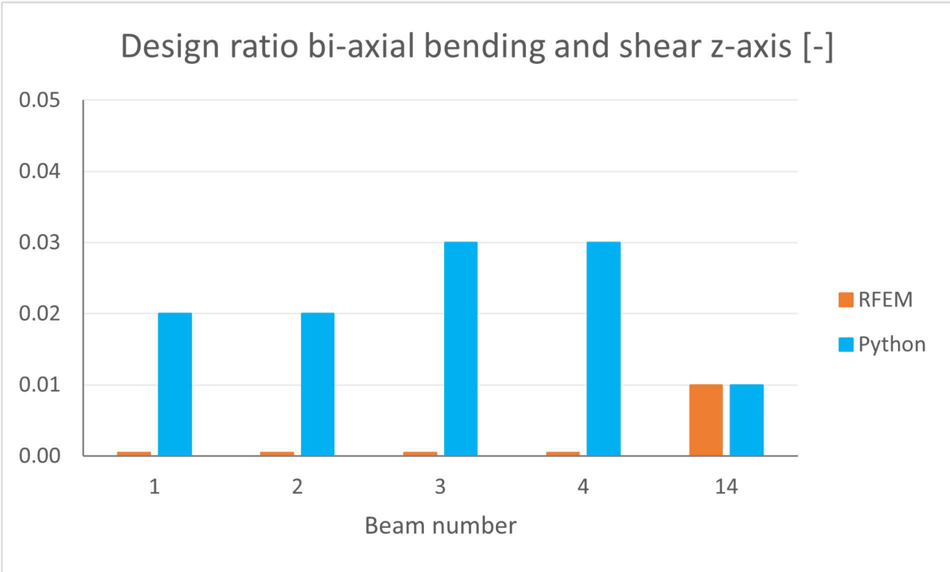


Figure F.14: Design ratios bi-axial bending, shear and axial z-axis

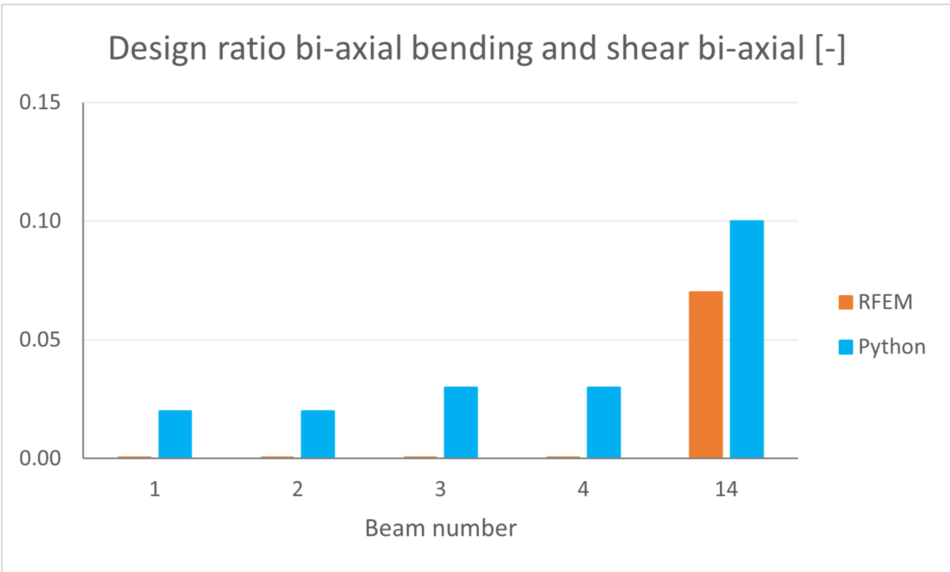


Figure F.15: Design ratios bi-axial bending, shear and axial bi-axial

F.9. LATERAL TORSIONAL BUCKLING

In this section a comparison is made for the lateral torsional buckling check from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. The comparison for this check can be found in tables F.23 and F.24 and in figure F.16. The differences between the tables and the figure are due to different coefficients used to calculate the critical bending moment. In the python code simplifications are used for these coefficients which lead to conservative values for the lateral torsional buckling check.

RFEM		1	3	11	12	13	14
α_{LT}	[-]	0.34	0.3 4	0.76	0.76	0.76	0.76
E	[MPa]	210000	210000	210000	210000	210000	210000
G	[MPa]	80769.2	80769	80769.2	80769.2	80769.2	80769.2
L	[m]	3	3	10	10	10	10
I_z	[cm4]	10820	10820	310.9	310.9	310.9	310.9
I_w	[cm6]	3817000	4E+06	26420	26420	26420	26420
I_t	[cm4]	355.7	355.7	15.14	15.14	15.14	15.14
M_{cr}	[kNm]	11110.6	11111	49.68	28.61	28.51	49.68
W_y	[cm3]	3244.12	3244.1	319.61	319.61	319.61	319.61
f_y	[MPa]	345	345	235	235	235	235
λ_{LT}	[-]	0.317	0.317	-	1.62	1.623	1.623
$\lambda_{LT,0}$	[-]	0.4	0.4	0.4	0.4	0.4	0.4
β	[-]	0.75	0.75	-	-	-	-
ϕ_{LT}	[-]	0.524	0.524	-	2.353	2.358	2.358
χ_{LT}	[-]	1	1	-	0.246	0.246	0.246
k_c	[-]	0.632	0.632	-	-	-	-
f	[-]	0.902	0.902	-	-	-	-
γ_{M1}	[-]	1	1	-	1	1	1
$M_{b,Rd}$	[kNm]	1119.22	1119.2	-	18.51	18.46	18.46
$M_{y,Ed}$	[kNm]	11.6	11.6	7.88	10.41	18.9	7.88
UC	[-]	0.01	0.01	0.159	0.56	1.02	0.159

Table F.23: RFEM EN checker, lateral torsional buckling check

Python		1	3	11	12	13	14
α_{LT}	[-]	0.21	0.21	0.76	0.76	0.76	0.76
E	[MPa]	210000	210000	210000	210000	210000	210000
G	[MPa]	81000	81000	81000	81000	81000	81000
L	[m]	3	3	10	10	10	10
I_z	[cm4]	10819	10819	310.9	310.9	310.9	310.9
I_w	[cm6]	3817152	3817152	27762	27762	27762	27762
I_t	[cm4]	355.70	355.70	15.14	15.14	15.14	15.14
M_{cr}	[kNm]	14454.71	14454.71	29.55	29.55	29.55	29.55
W_y	[cm3]	3125.4	3125.4	336.831	336.831	336.831	336.831
f_y	[MPa]	345	345	235	235	235	235
λ_{LT}	[-]	0.27	0.27	1.63	1.63	1.63	1.63
$\lambda_{LT,0}$	[-]	-	-	-	-	-	-
β	[-]	-	-	-	-	-	-
ϕ_{LT}	[-]	0.480	0.480	2.385	2.385	2.385	2.385
χ_{LT}	[-]	0.984	0.984	0.243	0.243	0.243	0.243
k_c	[-]	-	-	-	-	-	-
f	[-]	-	-	-	-	-	-
γ_{M1}	[-]	1.00	1.00	1.00	1.00	1.00	1.00
$M_{b,Rd}$	[kNm]	1060.76	1060.76	19.21	19.21	19.21	19.21
$M_{y,Ed}$	[kNm]	16.20	16.20	7.88	10.41	24.12	7.88
UC	[-]	0.02	0.02	0.41	0.54	1.26	0.41

Table F.24: Implementation of EN checker in Python, lateral torsional buckling check

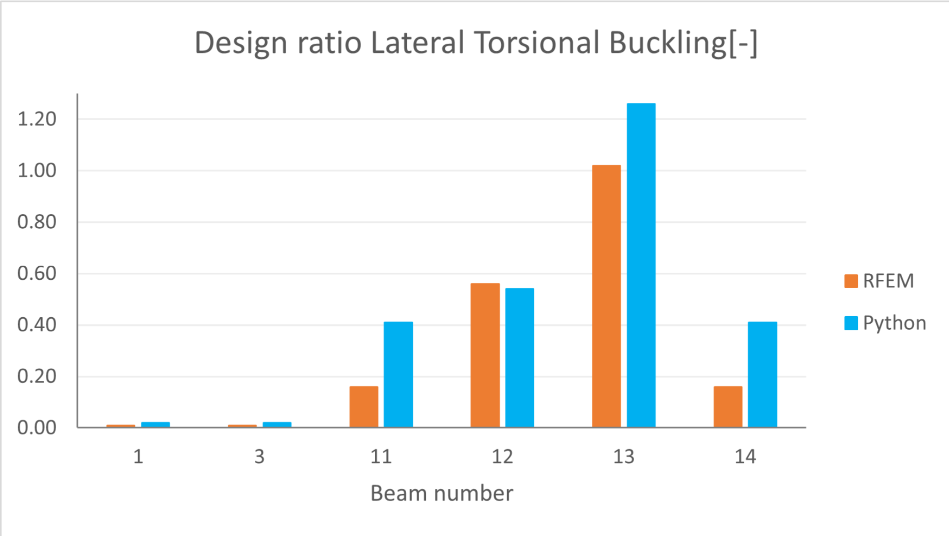


Figure F.16: Design ratios lateral torsional buckling

F.10. BI-AXIAL BENDING BUCKLING

In this section a comparison is made for the stability check for bi-axial bending from NEN-EN 1993-1-1 [80], between the build in RFEM code and the created python code to check the created code. The comparison for this check can be found in tables F.25 and F.26 and in figures F.17 and F.18. The differences between the tables and the figure are due to different coefficients used to calculate the interaction factors. In the python code simplifications are used for these coefficients which lead to slightly less conservative values for the bi-axial bending stability check.

RFEM		1	3	5	6	7	8	9	10
α_{LT}	[-]	0.34	0.34	0.49	0.49	0.49	0.49	0.49	0.49
E	[MPa]	210000	210000	210000	210000	210000	210000	210000	210000
G	[MPa]	80769	80769	80769	80769	80769	80769	80769	80769
L	[m]	3.00	3.00	1.67	1.67	1.67	1.67	1.67	1.67
I_w	[cm6]	3817000	3817000	791000	791000	791000	791000	791000	791000
I_t	[cm4]	355.70	355.70	66.87	66.87	66.87	66.87	66.87	66.87
M_{cr}	[kNm]	10919.20	10919.20	7771.24	3600.41	7561.84	7771.24	3600.41	7561.84
W_y	[cm3]	3244.12	3244.12	1702.00	1702.00	1702.00	1702.00	1702.00	1702.00
λ_{LT}	[-]	0.32	0.32	0.28	0.41	0.28	0.28	0.41	0.28
$\lambda_{LT,0}$	[-]	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
β	[-]	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
ϕ_{LT}	[-]	0.53	0.53	0.50	0.57	0.50	0.50	0.57	0.50
χ_{LT}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
k_c	[-]	0.63	0.63	0.66	0.85	0.61	0.66	0.85	0.61
f	[-]	0.90	0.90	0.92	0.95	0.91	0.92	0.95	0.91
C_{my}	[-]	0.40	0.40	0.40	0.82	0.40	0.40	0.82	0.40
C_{mz}	[-]	0.40	0.40	0.63	0.49	0.58	0.63	0.49	0.58
C_{mLT}	[-]	0.40	0.40	0.40	0.82	0.40	0.40	0.82	0.40
k_{yy}	[-]	0.40	0.40	0.40	0.82	0.40	0.40	0.82	0.40
k_{yz}	[-]	0.24	0.24	0.38	0.30	0.35	0.38	0.30	0.35
k_{zy}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
k_{zz}	[-]	0.40	0.40	0.63	0.49	0.58	0.63	0.49	0.58
$M_{y,Ed}$	[kNm]	16.20	16.20	7.20	8.48	8.48	7.20	8.48	8.48
$M_{y,Rk}$	[kNm]	1119.22	1119.22	604.21	604.21	604.21	604.21	604.21	604.21
$M_{z,Ed}$	[kNm]	6.53	6.53	2.13	8.54	8.58	2.13	8.54	8.58
$M_{z,Rk}$	[kNm]	381.12	381.12	98.12	98.12	98.12	98.12	98.12	98.12
UC_1	[-]	0.01	0.01	0.01	0.04	0.04	0.01	0.04	0.04
UC_2	[-]	0.02	0.02	0.03	0.06	0.06	0.03	0.06	0.06

Table F.25: RFEM EN checker, bi-axial bending buckling check

Python		1	3	5	6	7	8	9	10
α_{LT}	[-]	0.21	0.21	0.34	0.34	0.34	0.34	0.34	0.34
E	[MPa]	210000	210000	210000	210000	210000	210000	210000	210000
G	[MPa]	81000	81000	81000	81000	81000	81000	81000	81000
L	[m]	3	3	1.667	1.667	1.667	1.667	1.667	1.667
I_w	[cm6]	3817152	3817152	791005.1	791005.1	791005.1	791005.1	791005.1	791005.1
I_t	[cm4]	355.7	355.7	66.9	66.9	66.9	66.9	66.9	66.9
M_{cr}	[kNm]	14454.71	14454.71	8820.805	8820.805	8820.805	8820.805	8820.805	8820.805
W_y	[cm3]	3125.4	3125.4	1623.92	1623.92	1623.92	1623.92	1623.92	1623.92
λ_{LT}	[-]	0.273	0.273	0.256	0.256	0.256	0.256	0.256	0.256
$\lambda_{LT,0}$	[-]	-	-	-	-	-	-	-	-
β	[-]	-	-	-	-	-	-	-	-
Φ_{LT}	[-]	0.545	0.545	0.542	0.542	0.542	0.542	0.542	0.542
χ_{LT}	[-]	0.984	0.984	0.980	0.980	0.980	0.980	0.980	0.980
k_c	[-]	-	-	-	-	-	-	-	-
f	[-]	-	-	-	-	-	-	-	-
C_{my}	[-]	0.46	0.46	0.4	0.4	0.4	0.4	0.4	0.4
C_{mz}	[-]	0.46	0.46	0.4	0.4	0.4	0.4	0.4	0.4
C_{mLT}	[-]	0.46	0.46	0.4	0.4	0.4	0.4	0.4	0.4
k_{yy}	[-]	0.46	0.46	0.40	0.40	0.40	0.40	0.40	0.40
k_{yz}	[-]	0.28	0.28	0.24	0.24	0.24	0.24	0.24	0.24
k_{zy}	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
k_{zz}	[-]	0.46	0.46	0.40	0.40	0.40	0.40	0.40	0.40
$M_{y,Ed}$	[kNm]	16.20	16.20	7.20	8.48	8.48	7.20	8.48	8.48
$M_{y,Rk}$	[kNm]	1078.34	1078.34	576.49	576.49	576.49	576.49	576.49	576.49
$M_{z,Ed}$	[kNm]	6.53	12.40	2.13	8.54	8.58	2.13	8.54	8.58
$M_{z,Rk}$	[kNm]	378.16	378.16	96.85	96.85	96.85	96.85	96.85	96.85
UC_1	[-]	0.01	0.02	0.01	0.03	0.03	0.01	0.03	0.03
UC_2	[-]	0.03	0.03	0.02	0.05	0.05	0.02	0.05	0.05

Table F.26: Implementation of EN checker in Python, bi-axial bending buckling check

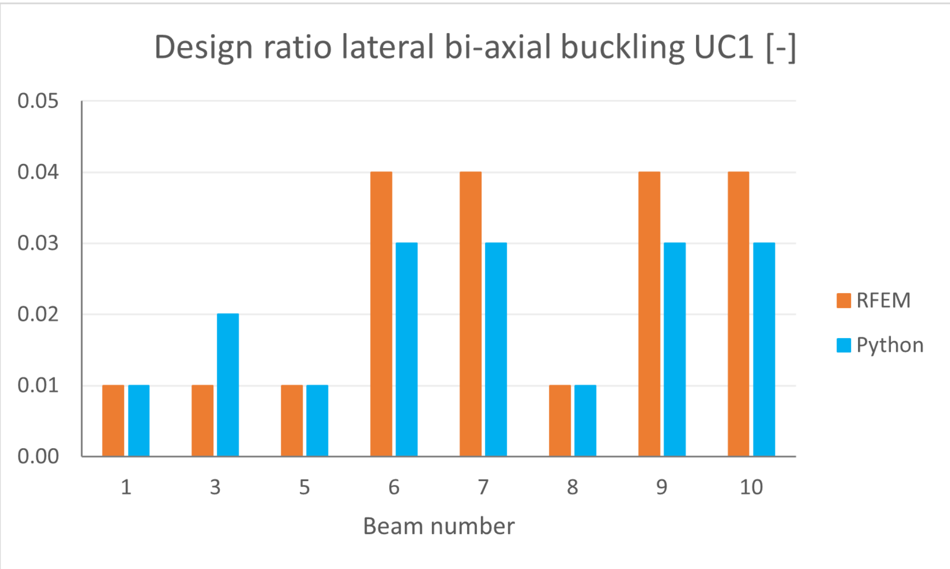


Figure F.17: Bi-axial bending buckling UC1

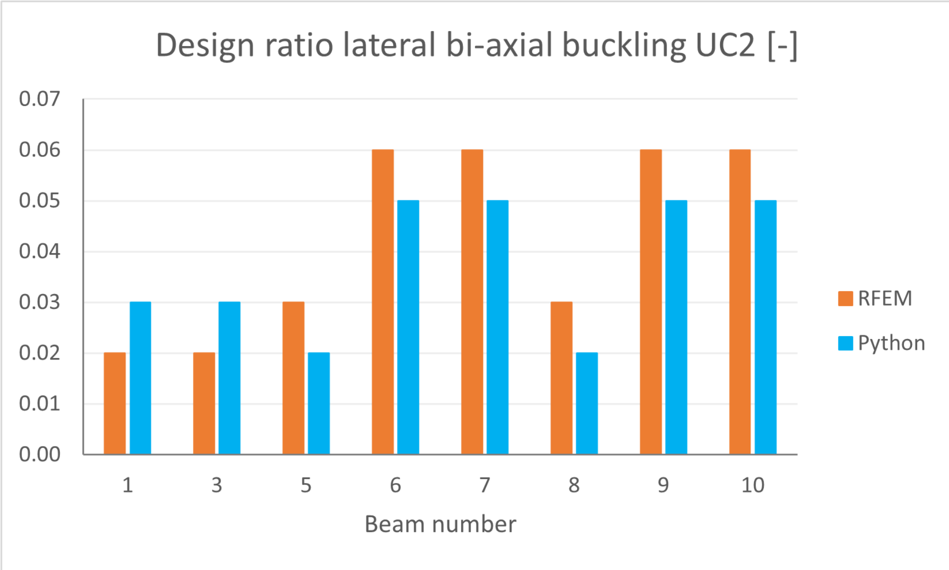


Figure F.18: Bi-axial bending buckling UC2

F.11. TOTAL DESIGN RATIOS

In this section a list is given with all the design ratios from the sections above for both RFEM and python, see tables F.27 and F.28. The maximum design ratios for all the beams can be found in figure F.19.

RFEM	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	9	10	11	12	13	14
Compression	0.00	0.00	0.00	0.00	-	-	-	-	-	-	0.00	-	0.01	0.01
Tension	-	-	-	-	-	-	-	-	-	-	0.00	0.01	-	0.00
Shear z-axis	0.01	0.00	0.01	0.00	0.01	0.00	0.01	0.01	0.00	0.01	0.02	0.02	0.04	0.02
Shear y-axis	-	-	0.00	0.00	-	0.01	0.01	-	0.01	0.01	-	-	-	-
B+S z-axis	-	0.00	-	0.00	0.01	-	0.01	0.01	-	0.01	0.10	0.14	0.14	0.10
B+S y-axis	0.01	0.02	0.03	0.03	0.01	0.01	0.04	0.01	0.01	0.04	0.00	-	-	0.01
B+S+A z-axis	-	-	-	-	-	-	-	-	-	-	0.10	0.13	0.26	0.10
B+S+A y-axis	0.01	0.01	0.01	0.01	-	-	-	-	-	-	0.01	-	-	0.02
Bi-axial B+S y-axis	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-	-	-	0.05
Bi-axial B+S z-axis	0.02	0.02	0.02	0.02	0.02	0.09	0.09	0.02	0.09	0.09	-	-	-	0.01
Bi-axial B+S bi-axial	0.02	0.02	0.02	0.02	0.02	0.09	0.09	0.02	0.09	0.09	-	-	-	0.06
Bi-axial B+S+A y-axis	0.00	0.00	0.00	0.00	-	-	-	-	-	-	-	-	-	0.06
Bi-axial B+S+A z-axis	0.00	0.00	0.00	0.00	-	-	-	-	-	-	-	-	-	0.01
Bi-axial B+S+A bi-axial	0.00	0.00	0.00	0.00	-	-	-	-	-	-	-	-	-	0.07
LTB	0.01	-	0.01	-	-	-	-	-	-	-	0.16	0.56	1.02	0.16
Bi-axial buckling UC1	0.01	-	0.01	-	0.01	0.04	0.04	0.01	0.04	0.04	-	-	-	-
Bi-axial buckling UC2	0.02	-	0.02	-	0.03	0.06	0.06	0.03	0.06	0.06	-	-	-	-
Max	**0.02**	**0.02**	**0.03**	**0.03**	**0.03**	**0.09**	**0.09**	**0.03**	**0.09**	**0.09**	**0.16**	**0.56**	**1.02**	**0.16**

Table F.27: RFEM EN checker, design ratios:

Python	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Compression	0.00	0.00	0.00	0.00	-	-	-	-	-	-	0.00	-	0.01	0.01
Tension	-	-	-	-	-	-	-	-	-	-	0.00	0.01	-	0.00
Shear z-axis	0.01	0.00	0.01	0.00	0.01	0.00	0.01	0.01	0.00	0.01	0.02	0.02	0.04	0.02
Shear y-axis	-	-	0.00	0.00	-	0.01	0.01	-	0.01	0.01	-	-	-	-
B+S z-axis	-	0.01	-	0.01	0.01	-	0.01	0.01	-	0.01	0.10	0.13	0.30	0.10
B+S y-axis	0.02	0.02	0.03	0.03	0.02	0.09	0.09	0.02	0.09	0.09	0.01	-	-	0.01
B+S+A z-axis	-	-	-	-	-	-	-	-	-	-	0.10	0.13	0.31	0.10
B+S+A y-axis	0.02	0.02	0.03	0.03	-	-	-	-	-	-	0.01	-	-	0.01
Bi-axial B+S y-axis	0.02	0.01	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	-	-	-	0.10
Bi-axial B+S z-axis	0.02	0.02	0.03	0.03	0.02	0.09	0.09	0.02	0.09	0.09	-	-	-	0.10
Bi-axial B+S bi-axial	0.02	0.02	0.03	0.03	0.02	0.09	0.09	0.02	0.09	0.09	-	-	-	0.10
Bi-axial B+S+A y-axis	0.02	0.01	0.02	0.01	-	-	-	-	-	-	-	-	-	0.10
Bi-axial B+S+A z-axis	0.02	0.02	0.03	0.03	-	-	-	-	-	-	-	-	-	0.01
Bi-axial B+S+A bi-axial	0.02	0.02	0.03	0.03	-	-	-	-	-	-	-	-	-	0.10
LTB	0.02	-	0.02	-	-	-	-	-	-	-	0.41	0.54	1.26	0.41
Bi-axial buckling UC1	0.01	-	0.02	-	0.01	0.03	0.03	0.01	0.03	0.03	-	-	-	-
Bi-axial buckling UC2	0.03	-	0.03	-	0.02	0.05	0.05	0.02	0.05	0.05	-	-	-	-
Max	**0.03**	**0.02**	**0.03**	**0.03**	**0.02**	**0.09**	**0.09**	**0.02**	**0.09**	**0.09**	**0.41**	**0.54**	**1.26**	**0.41**

Table F.28: Implementation of EN checker in Python, design ratios:

Where:

B	=	Bending
S	=	Shear
A	=	Axial
LTB	=	Lateral Torsional Buckling

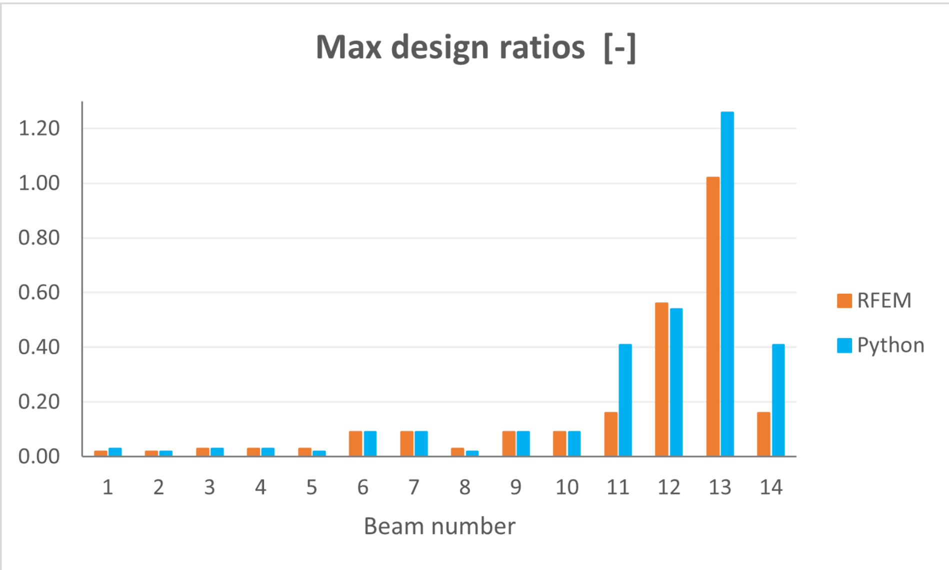


Figure F.19: Maximum design ratio per beam

G

Benchmark check Component Method

The structural analysis of the connection is checked with both the Component Method from the Eurocode and the software IDEA StatiCa. It is done in both because the current version of the software IDEA StatiCa cannot give the initial stiffness and the bending moment resistance of a connection through the API. This is why the Component Method is used to calculate the stiffness and the bending moment resistance of a connection. IDEA StatiCa is used to check the strength of the different connection elements more precisely.

The Component Method is programmed in python, the formulas used can be found in NEN-EN 1993-1-8 [41].

The programmed component method is verified by checking 6 examples. These examples are verified by comparing them in different programs. The comparison between three of these examples are done between IDEA StatiCa, Excel, Programmed code and an pre-calculated example from books or lectures. The other three examples are compared between IDEA StatiCa, Excel and the Programmed code. The examples including the results of the comparison can be found in the sub-sections below.

G.1. Case 1, Bolted 2-sided end-plate beam/column connection

This example is taken from chapter 1 of the book Knopen [100]. The design of this connection can be found in Figure G.1 and G.2.

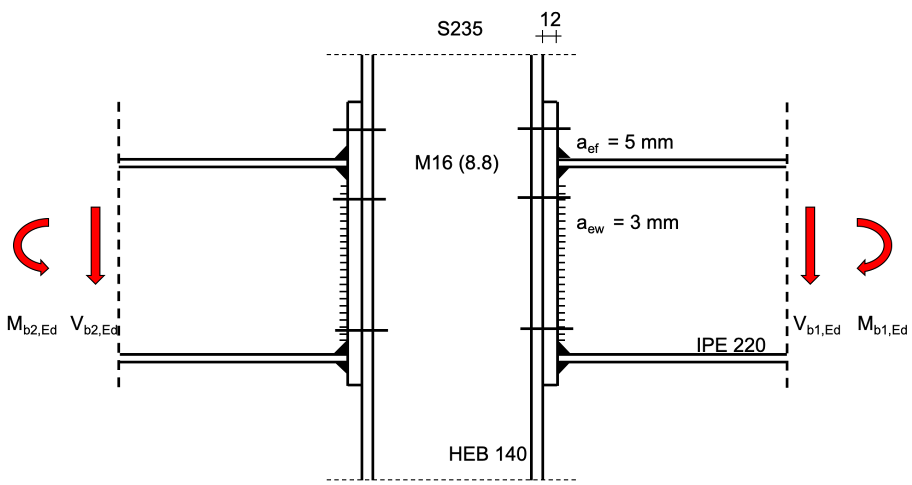


Figure G.1: Side view case 1

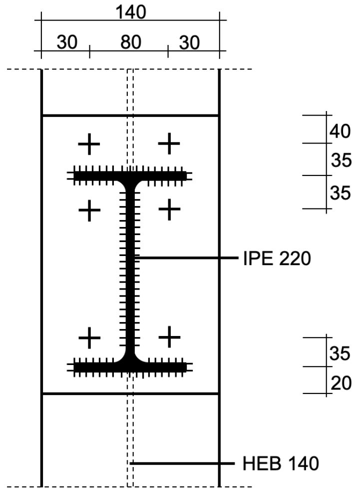


Figure G.2: Front view case 1

In table G.1 the results of the comparison can be found. The components considered for this connection are:

- CM1 (Column web panel in shear)
- CM2 (Column web in transverse compression)
- CM3 (Column web in transverse tension)
- CM4 (Column flange in bending)
- CM5 (End-plate in bending)
- CM7 (Beam or column flange and web in compression)
- CM8 (Beam web in tension)
- CM10 (Bolts in tension)

It can be seen that there are very small differences in the different components between the book, the programmed code of the component method and the component method in excel. These differences are due to rounding values in between. The IDEA StatiCa software cannot give the result of all the components, but only the bending moment resistance and the initial rotational stiffness of the connection. The big differences between IDEA StatiCa and the component method in bending moment resistance and stiffness can be found in the section G.7.

Components		Book	Code	Excel	IdeaStatica
CM1	$V_{wp,Rd}$ [kN]	160	160	160	-
	k1 [mm]	∞	∞	∞	-
CM2	$F_{c,Rd}$ [kN]	255	256	256	-
	k2 [mm]	7.05	7.07	7.07	-
CM3	$F_{t,Rd}$ [kN]	355	355	355	-
	k3 [mm]	4.56	4.56	4.56	-
CM4	$F_{c,Rd}$ [kN]	254	254	254	-
	k4 [mm]	8.53	8.62	8.62	-
CM5,out	$F_{t,Rd}$ [kN]	96	96	96	-
	k5out [mm]	7.22	7.22	7.22	-
CM5, in	$F_{t,Rd}$ [kN]	137	137	137	-
	k5in [mm]	7.92	7.96	7.96	-
CM7	$F_{c,Rd}$ [kN]	318	318	318	-
	k7 [mm]	∞	∞	∞	-
CM8	$F_{t,Rd}$ [kN]	269	269	269	-
	k8 [mm]	∞	∞	∞	-
CM10	$F_{t,Rd}$ [kN]	180	181	181	-
	k10 [mm]	7.08	7.08	7.08	-
$M_{j,Rd}$ [kNm]		47.7	47.7	47.7	60.8
Stiffness [MNm/rad]		21.5	21.6	21.6	11.4

Table G.1: Comparison of case 1

G.2. CASE 2, BOLTED 1-SIDED END-PLATE BEAM/COLUMN CONNECTION

This example is taken from chapter 3 of the book Knopen [100]. The design of this connection can be found in Figure G.3 and G.4.

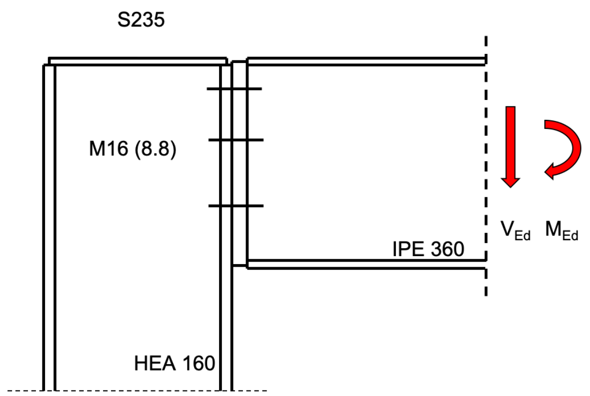


Figure G.3: Side view case 2

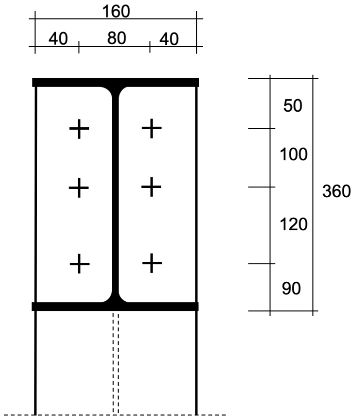


Figure G.4: Front view case 2

In table G.2 the results of the comparison can be found. The components considered for this connection are CM1, CM2, CM3, CM4, CM5, CM7 and CM10.

It can be seen that there are very small differences in the different components between the book, the programmed code of the component method and the component method in excel. These differences are due to rounding values in between. The IDEA StatiCa software cannot give the result of all the components, but only the bending moment resistance and the initial rotational stiffness of the connection. The big differences between IDEA StatiCa and the component method in bending moment resistance and stiffness can be found in the section G.7.

Components		Book	Code	Excel	IdeaStatica
CM1	**$V_{wp,Rd}$ [kN]**	161	161	161	-
	k1 [mm]	2.1	2.1	2.1	-
CM2	**$F_{c,Rd}$ [kN]**	170	169	169	-
	k2 [mm]	6.2	6.2	6.2	-
CM3	**$F_{t,Rd}$ [kN]**	167	166	166	-
	k3 [mm]	5.5	5.3	5.3	-
CM4	**$F_{c,Rd}$ [kN]**	114	114	114	-
	k4 [mm]	5.7	5.3	5.3	-
CM5	**$F_{t,Rd}$ [kN]**	144	144	144	-
	k5 [mm]	9.1	9.1	9.1	-
CM7	**$F_{c,Rd}$ [kN]**	690	690	690	-
	k7 [mm]	∞	∞	∞	-
CM10	**$F_{t,Rd}$ [kN]**	90	90	90	-
	k10 [mm]	7.5	7.7	7.7	-
$M_{j,Rd}$ [kNm]		45	45	45	60.5
Stiffness [MNm/rad]		13.5	13.6	13.6	11.7

Table G.2: Comparison of case 2

G.3. CASE 3, BOLTED 1-SIDED END-PLATE BEAM/COLUMN CONNECTION

This example is taken from an assignment of the TU-Delft [101]. The design of this connection can be found in Figure G.5 and G.6.

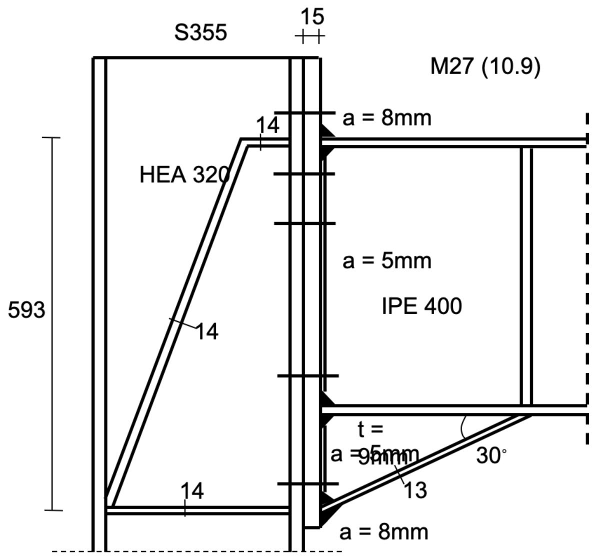


Figure G.5: Side view case 3

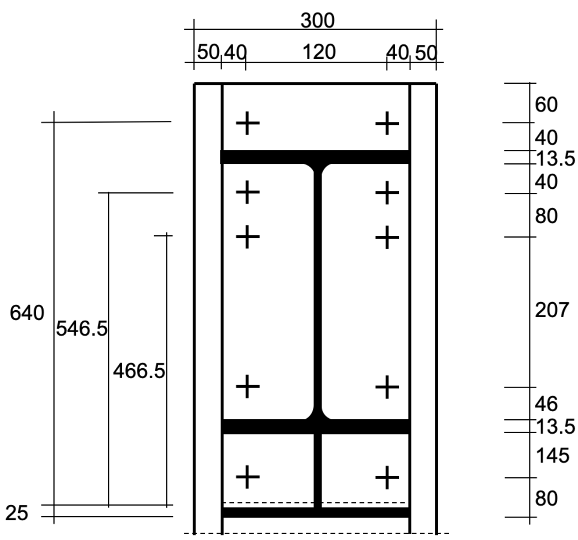


Figure G.6: Front view case 3

In table G.3 the results of the comparison can be found. The components considered for this connection are CM1, CM2, CM3, CM4 and CM5.

It can be seen that there are very small differences in the different components between the report, the programmed code of the component method and the component method in excel. These differences are due to rounding values in between. The IDEA StatiCa software cannot give the result of all the components, but only the bending moment resistance and the initial rotational stiffness of the connection. The big differences between IDEA StatiCa and the component method in bending moment resistance and stiffness can be found in the section G.7.

Components		Report	Code	Excel	IdeaStatica
CM1	**$V_{wp,Rd}$ [kN]**	798	796	796	-
	k1 [mm]	-	-	-	-
CM2	**$F_{c,Rd}$ [kN]**	557	557	557	-
	k2 [mm]	-	-	-	-
CM3	**$F_{t,Rd}$ [kN]**	588	588	588	-
	k3 [mm]	5.8	5.8	5.8	-
CM4 bolt-row 1	**$F_{c,Rd}$ [kN]**	521	521	521	-
	k4 [mm]	17.8	17.8	17.8	-
CM5 bolt-row 1	**$F_{t,Rd}$ [kN]**	258	258	258	-
	k5 [mm]	10.25	10.25	10.25	-
$M_{j,Rd}$ [kNm]		412.5	412	412	∞
Stiffness [MNm/rad]		516	516	516	490

Table G.3: Comparison of case 4

G.4. CASE 4, BOLTED 1-SIDED END-PLATE BEAM/COLUMN CONNECTION

The first three cases in the subsections above show that the excel and code are precise enough compared to the examples found in books and from the university. The other three examples are created to check the code with designs that are possible with the design restrictions mentioned in section 4.3. Here the differences between excel, programmed code and IDEA StatiCa is done. The design of case 4 can be found in figure G.7.

The design includes:

- Beam Profile: IPE 360
- Column Profile: HEB 240
- Steel class: S355
- Plate thickness: 20 mm
- Bolts: M16:8.8
- Weld throat size: 4 mm

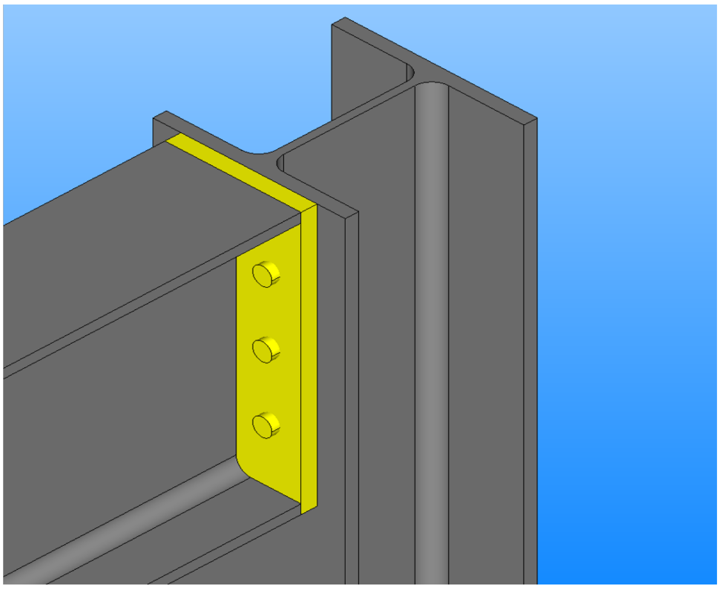


Figure G.7: ISO view case 4

In table G.4 the results of the comparison can be found. The components considered for this connection are CM1, CM2, CM3, CM4, CM5, CM7, CM8 and CM10. In the table only the values for the stiffness and bending moment resistance are shown.

The IDEA StatiCa software cannot give the result of all the components, but only the bending moment resistance and the initial rotational stiffness of the connection. The big differences between IDEA StatiCa and the component method in bending moment resistance and stiffness can be found in the section G.7.

Components	Excel	Code	IdeaStatica
$M_{j,Rd}$ [kNm]	79.1	79.1	78.4
Stiffness [MNm/rad]	19.4	19.4	12.8

Table G.4: Comparison of case 4

G.5. CASE 5, BOLTED 1-SIDED END-PLATE BEAM/COLUMN CONNECTION

The design of case 5 can be found in figure G.8. This is design 3 of section 4.3 without the extension at the bottom and including the stiffeners.

The design includes:

- Beam Profile: IPE 360
- Column Profile: HEB 240
- Steel class: S355
- Plate thickness: 12 mm
- Bolts: M16:8.8
- Stiffener thickness: 12 mm
- Weld throat size: 4 mm

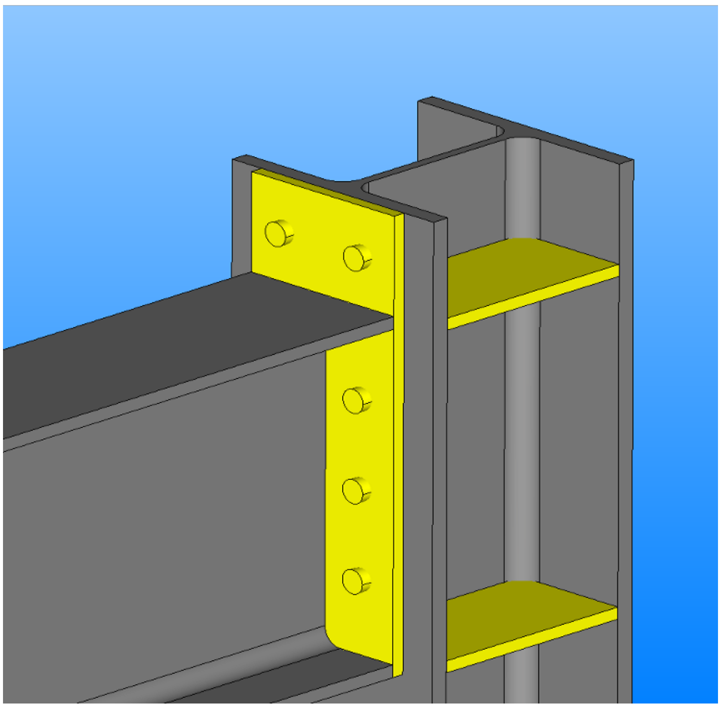


Figure G.8: ISO view case 5

In table G.5 the results of the comparison can be found. The components considered for this connection are CM1, CM2, CM3, CM4, CM5, CM7, CM8 and CM10. In the table only the values for the stiffness and bending moment resistance are shown.

The IDEA StatiCa software cannot give the result of all the components, but only the bending moment resistance and the initial rotational stiffness of the connection. The big differences between IDEA StatiCa and the component method in bending moment resistance and stiffness can be found in the section G.7.

Components	Excel	Code	IdeaStatica
$\mathbf{M_{j,Rd}}$ [**kNm**]	86	86	122
Stiffness [MNm/rad]	53.6	53.6	24.9

Table G.5: Comparison of case 5

G.6. CASE 6, BOLTED 1-SIDED END-PLATE BEAM/COLUMN CONNECTION

The design of case 6 can be found in figure G.9.

The design includes:

- Beam Profile: IPE 360
- Column Profile: HEB 240
- Steel class: S355
- Plate thickness: 16 mm
- Plate height: 252 mm
- Bolts: M16:8.8
- Weld throat size: 4 mm

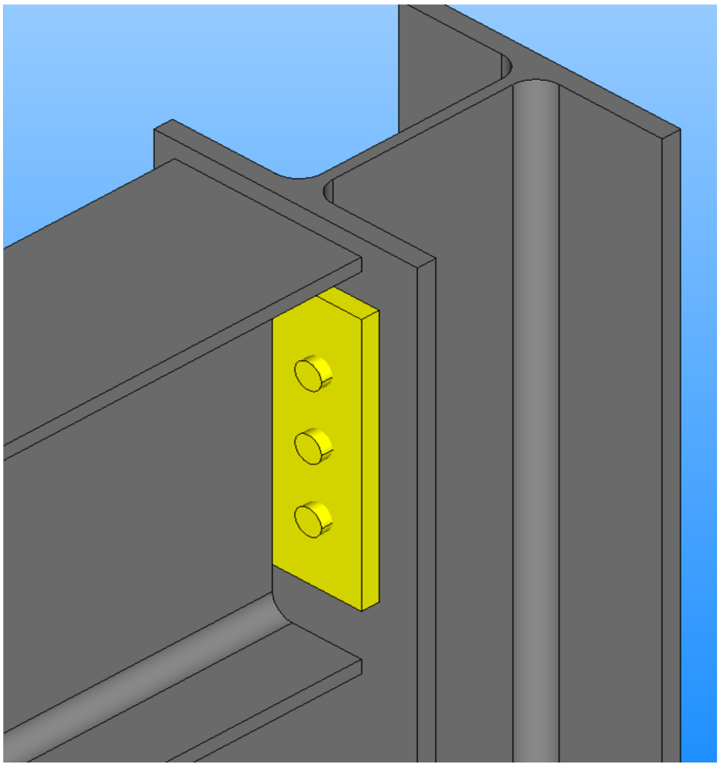


Figure G.9: ISO view case 6

In table G.6 the results of the comparison can be found. The components considered for this connection are CM1, CM2, CM3, CM4, CM5, CM7, CM8 and CM10. In the table only the values for the stiffness and bending moment resistance are shown.

The IDEA StatiCa software cannot give the result of all the components, but only the bending moment resistance and the initial rotational stiffness of the connection. The big differences between IDEA StatiCa and the component method in bending moment resistance and stiffness can be found in the section G.7.

Components	Excel	Code	IdeaStatica
$M_{j,Rd}$ [kNm]	36.7	36.7	24.1
Stiffness [MNm/rad]	8.98	8.98	5.2

Table G.6: Comparison of case 6

G.7. DISCUSSION

The difference between the initial rotational stiffness between the component method and IdeaStatica can be explained with figure G.10 from the book "Benchmark cases for advanced design of structural steel connections" [40]. In this figure it can be seen that the component method is a linear elastic model that goes much faster to the bending moment resistance value than CBFEM and FEM. This means that the component method is conservative and gives a higher initial rotational stiffness. This can be seen in the tables of the 6 cases above. CBFEM is a method that goes much more along the line of the FEM and experiments which means that it is more realistic, but gives therefore a lower initial rotational capacity. In the tables of the 6 cases it can also be seen that the component method always give a lower bending moment resistance which is also conservative compared to the FEM and CBFEM. This means that the calculations that are used in the optimization process of this research are conservative values.

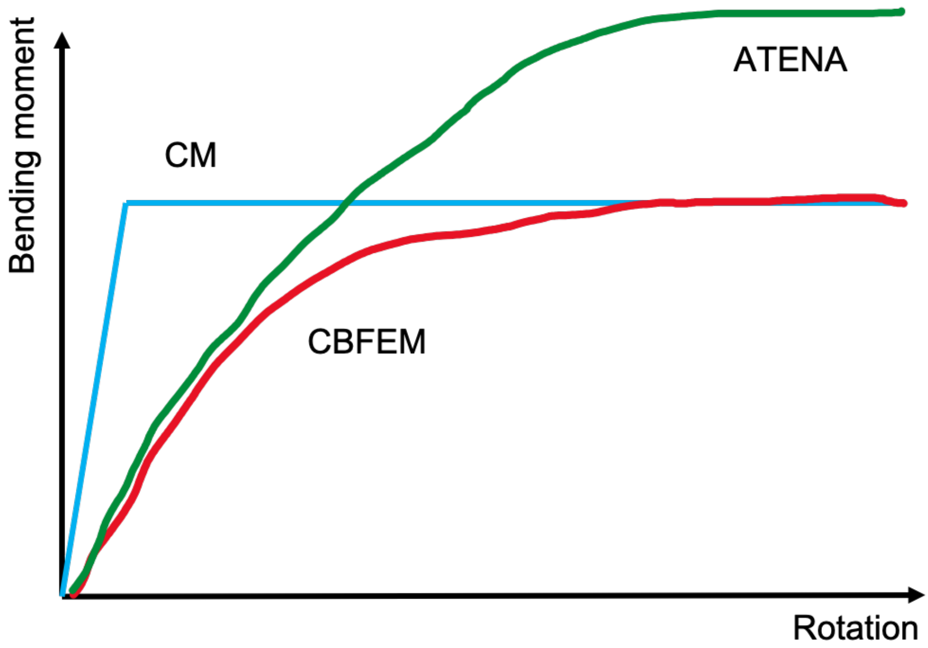


Figure G.10: Component Method VS IDEA StatiCa VS FEM [40]

H

INFLUENCE OF CONNECTION VARIABLES

To use connection design in the optimization process limitations need to be made on the variables considered in the design. The first decision made, is that only end-plate connections are considered in the design.This is done to limit the types of connections and because end-plate connections can act like a hinged, semi-rigid and fully-rigid connections.

For this end-plate connection there is looked at the influence on the stiffness and the bending moment resistance of the connection for the different variables in the study. This stiffness is calculated with the component method. The forces on the connections used in this study are a bending moment of 70 kNm and a shear force of 50 kN. The variables used in this study are:

- End plate length
- End plate length including only a top stiffener
- End plate length including top and bottom stiffener
- Haunch height/width ratio
- Haunch height
- Haunch height including only a top stiffener in the column
- Haunch height including a top and bottom stiffener in the column
- Haunch height including a top and bottom stiffener in the column and a beam stiffener
- Number of bolt rows
- Number of bolt rows including one bolt row above the beam
- Plate length above the beam
- Bolt size
- Bolt strength class
- End plate thickness

In the subsections below the different variables mentioned above are discussed in more detail.

H.1. END PLATE LENGTH

In this subsection the influence of the length of the end-plate is shown. The design used can be found in figure H.1. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360
- No stiffeners
- No haunch
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm platethickness

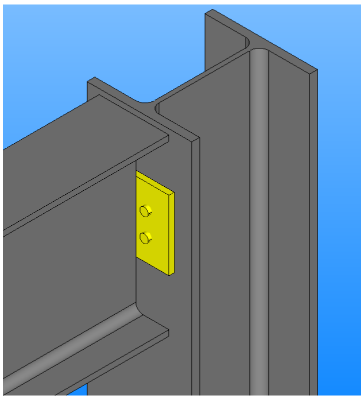


Figure H.1: End-plate length study design

The length of the end-plate varies from 200mm to 550mm with steps of 5mm until it reaches the beam height (500mm) then the steps become 10mm. The plate length is increased at the top and bottom until the length is the same as the beam height then the extension is only at the bottom.

In figure H.2 the initial rotational stiffness and bending moment resistance can be found for different end-plate lengths. It can be seen that the stiffness and bending moment increase in the same way. It can also be seen that adding a small extension below the beam increases the stiffness and bending moment resistance, this can be explained by component 2 (column web in transverse compression). It increases the effective width of the column web in compression, which increases the bending moment resistance and the stiffness. It can also be seen that this extension at the bottom of the beam only has influence for a small part. Increasing it any further has no influence on the stiffness or bending moment resistance, this is because there is a maximum effective width of the column web in compression.

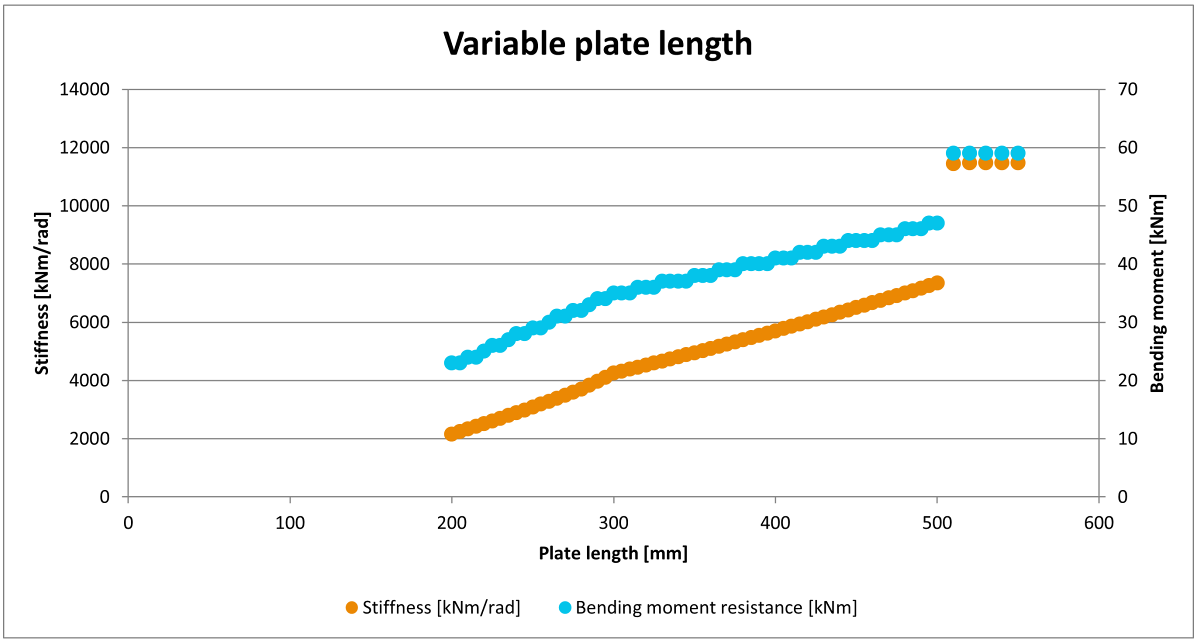


Figure H.2: Initial rotational stiffness and Bending moment resistance for increasing end-plate length

In figure H.3 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that increasing the length of the end plate increases the rotational stiffness.

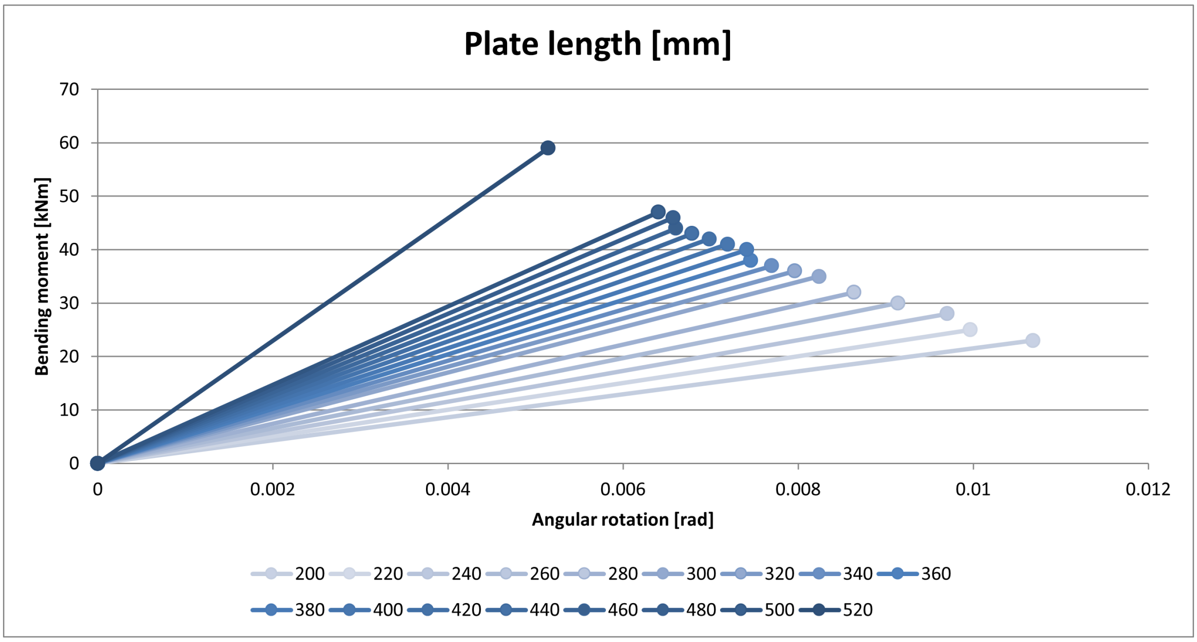


Figure H.3: Bending moment resistance and rotation for increasing end-plate length

H.2. END PLATE LENGTH INCLUDING TOP STIFFENER

In this subsection the influence of the length of the end-plate is shown. The design used is the same as in figure H.1 but only with top stiffeners. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360

- Top stiffener
- No haunch
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm platethickness

The length of the end-plate varies from 200mm to 550mm with steps of 5mm until it reaches the beam height (500mm) then the steps become 10mm. The plate length is increased at the top and bottom until the length is the same as the beam height then the extension is only at the bottom.

In figure H.4 the initial rotational stiffness and bending moment resistance can be found for different end-plate lengths. It can be seen that the stiffness and bending moment increase in the same way. It can also be seen that adding a small extension below the beam increases the stiffness and bending moment resistance, this can be explained by component 2 (column web in transverse compression). It increases the effective width of the column web in compression, which increases the bending moment resistance and the stiffness. It can also be seen that this extension at the bottom of the beam only has influence for a small part. Increasing it any further has no influence on the stiffness or bending moment resistance, this is because there is a maximum effective width of the column web in compression. When this graph is compared to the graph without stiffeners it can be seen that adding only a top stiffener has only an influence on the stiffness when the end-plate length is small as soon as the end-plate length gets bigger than 280mm the top stiffener has no influence on the stiffness or moment resistance of the connection. The stiffness is a bit lower in the case of only a top stiffener this is because of the fact that it decreases the effective lengths in component 4 (Column flange in transverse bending).

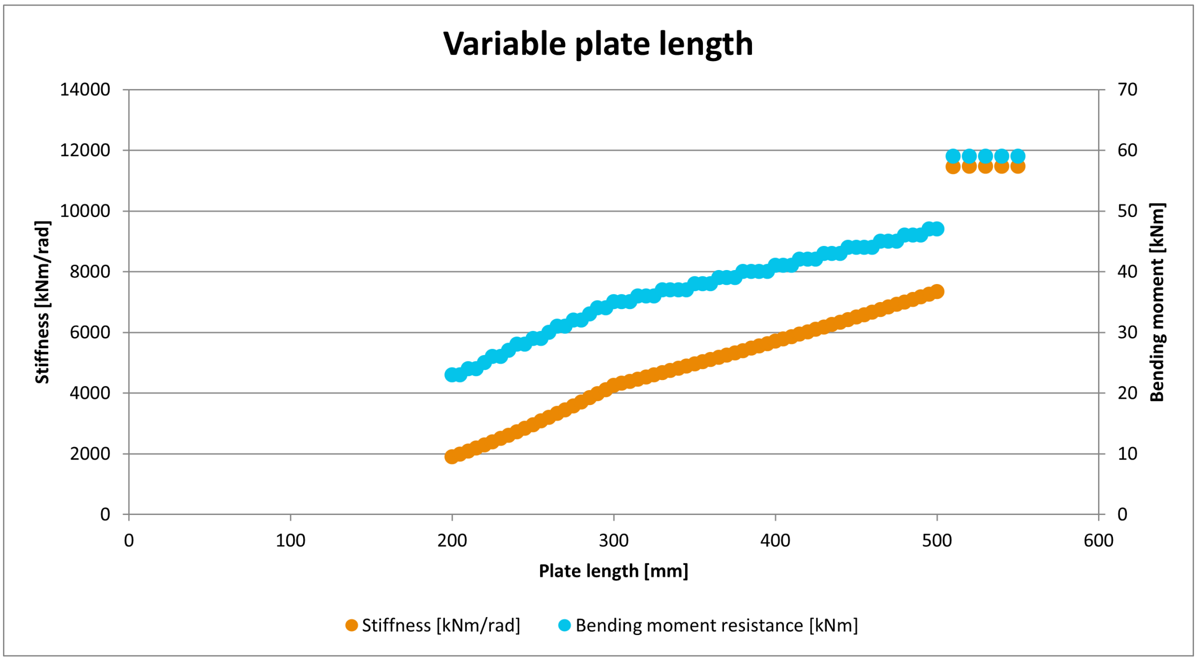


Figure H.4: Initial rotational stiffness and Bending moment resistance for increasing end-plate length

In figure H.5 the bending moment resistance is shown on the y-axis and the rotation of the connection on the y-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that increasing the length increases the rotational stiffness.

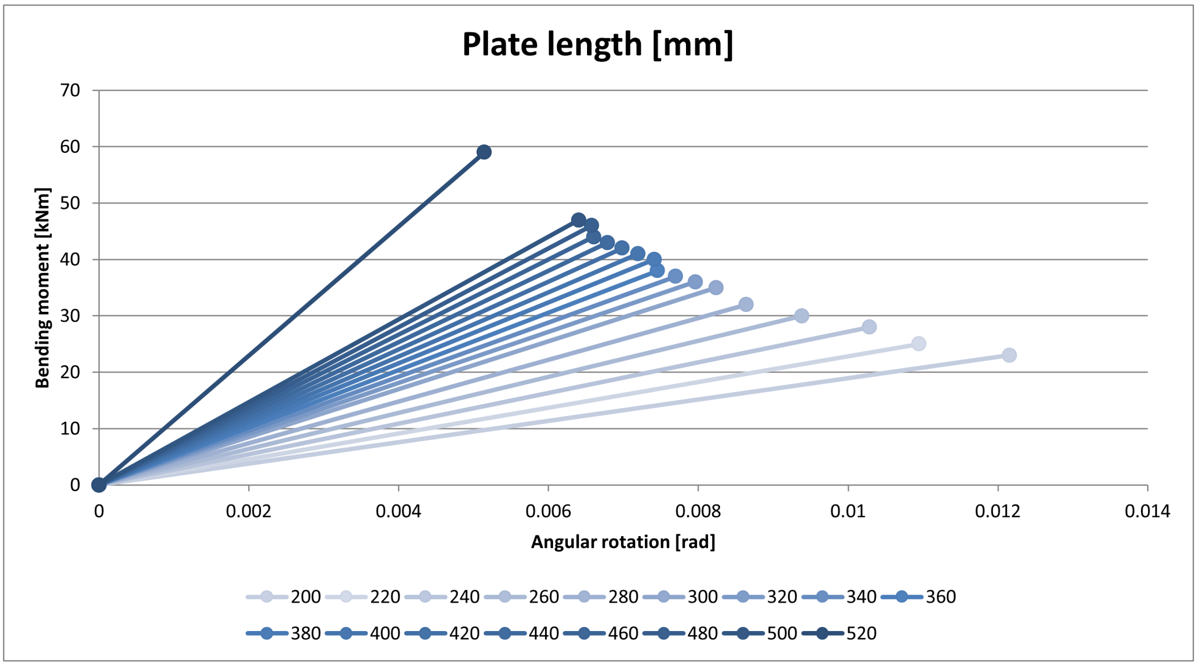


Figure H.5: Bending moment resistance and rotation for increasing end-plate length

H.3. END PLATE LENGTH INCLUDING TOP AND BOTTOM STIFFENER

In this subsection the influence of the length of the end-plate is shown. The design is the same as in figure H.1 but with both top and bottom stiffeners. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360
- Top and bottom stiffener
- No haunch
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm plate thickness

The length of the end-plate varies from 200mm to 550mm with steps of 5mm until it reaches the beam height (500mm) then the steps become 10mm. The plate length is increased at the top and bottom until the length is the same as the beam height then the extension is only at the bottom.

In figure H.6 the initial rotational stiffness and bending moment resistance can be found for different end-plate lengths. It can be seen that the stiffness and bending moment increase in the same way. It can also be seen that adding a small extension below the beam increases the stiffness and bending moment resistance, this can be explained by component 2 (column web in transverse compression). It increases the effective width of the column web in compression, which increases the bending moment resistance and the stiffness. It can also be seen that this extension at the bottom of the beam only has influence for a small part. Increasing it any further has no influence on the stiffness or bending moment resistance, this is because there is a maximum effective width of the column web in compression. When this graph is compared to the two cases above it can be seen that adding both a top and bottom stiffener has a positive influence on the stiffness of the connection. It has however no influence on the bending moment resistance of the connection.

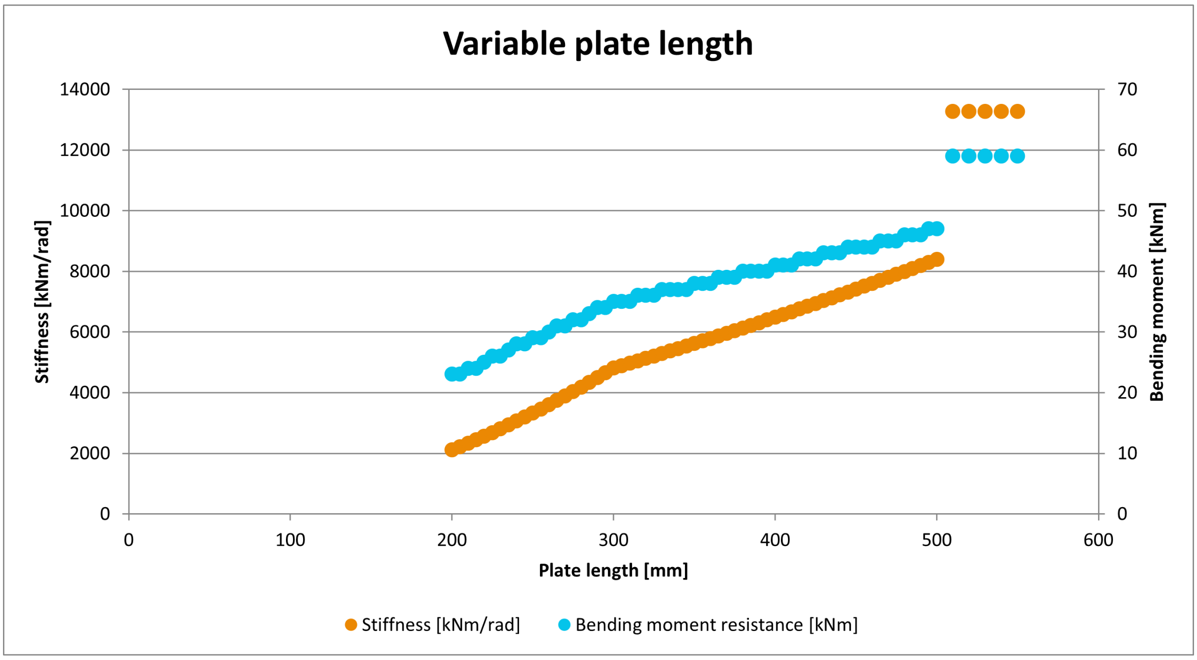


Figure H.6: Initial rotational stiffness and Bending moment resistance for increasing end-plate length

In figure H.7 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that increasing the length increases the rotational stiffness.

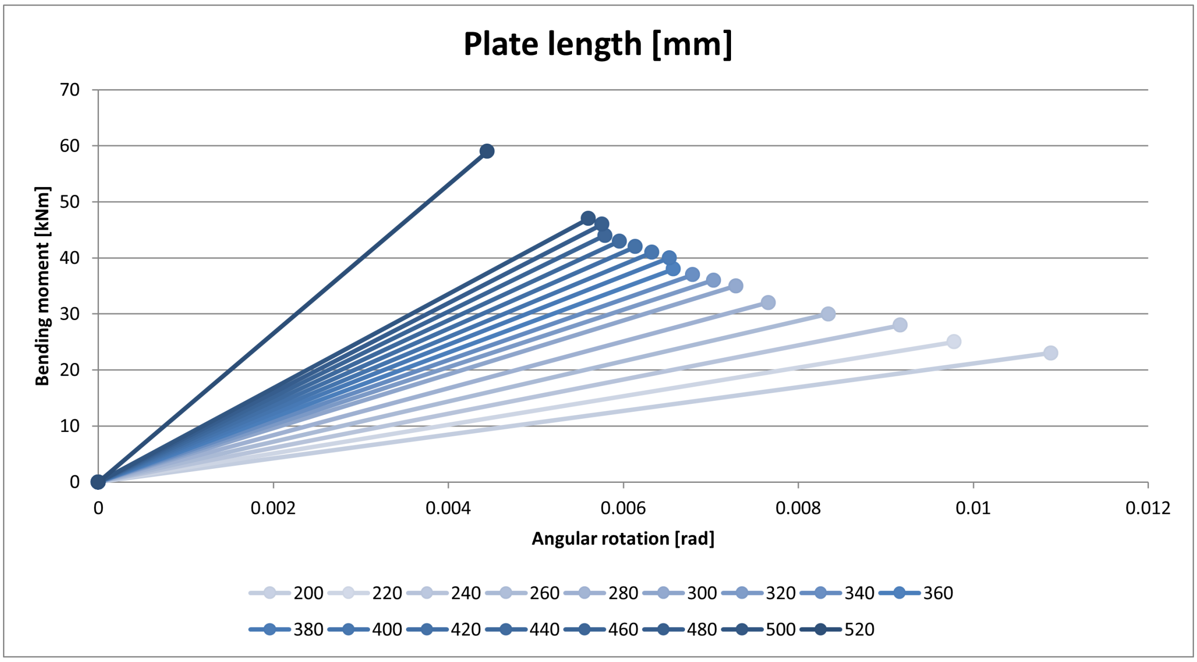


Figure H.7: Bending moment resistance and rotation for increasing end-plate length

H.4. HAUNCH RATIO

In this subsection the influence of the haunch length over height is shown. The design used can be found in figure H.8. The values in this design are:

- Beam profile: IPE 500

- Column profile: HEB 360
- No stiffener
- Haunch including haunch flange
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm plate thickness
- Plate length 830 mm
- Haunch height 300 mm

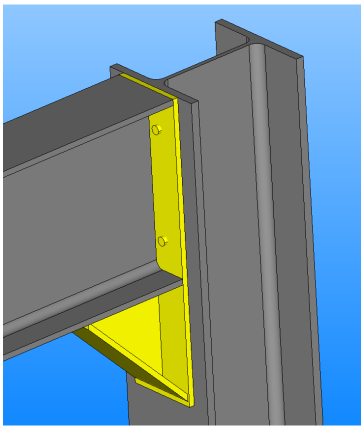


Figure H.8: Haunch study design

The length of the haunch varies from 150mm to 600mm. This length varies with the haunch length over height ratio. This ratio is varied from 0.5 to 2 with steps of 0.01. It only increases the length of the haunch and not the height.

In figure H.9 the initial rotational stiffness and bending moment resistance can be found for different haunch ratios. It can be seen that the stiffness and bending moment do not change if only the length of the haunch increases.

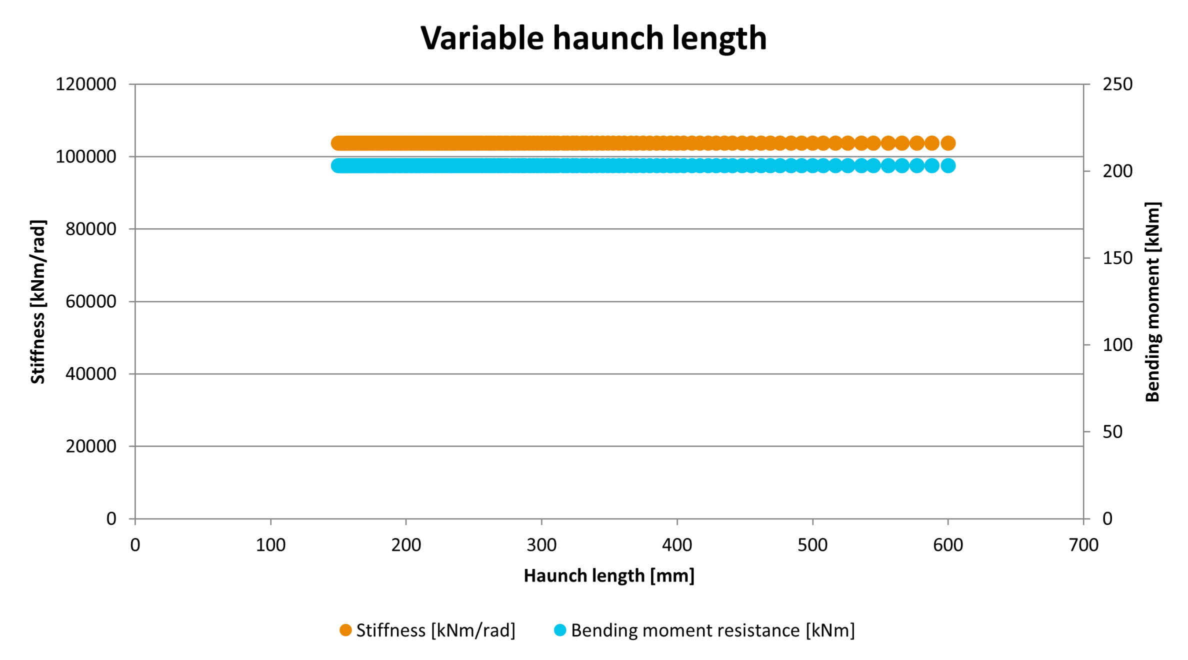


Figure H.9: Initial rotational stiffness and Bending moment resistance for increasing haunch length

In figure H.10 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that there are no changes for an increasing haunch length, because all points are the same.

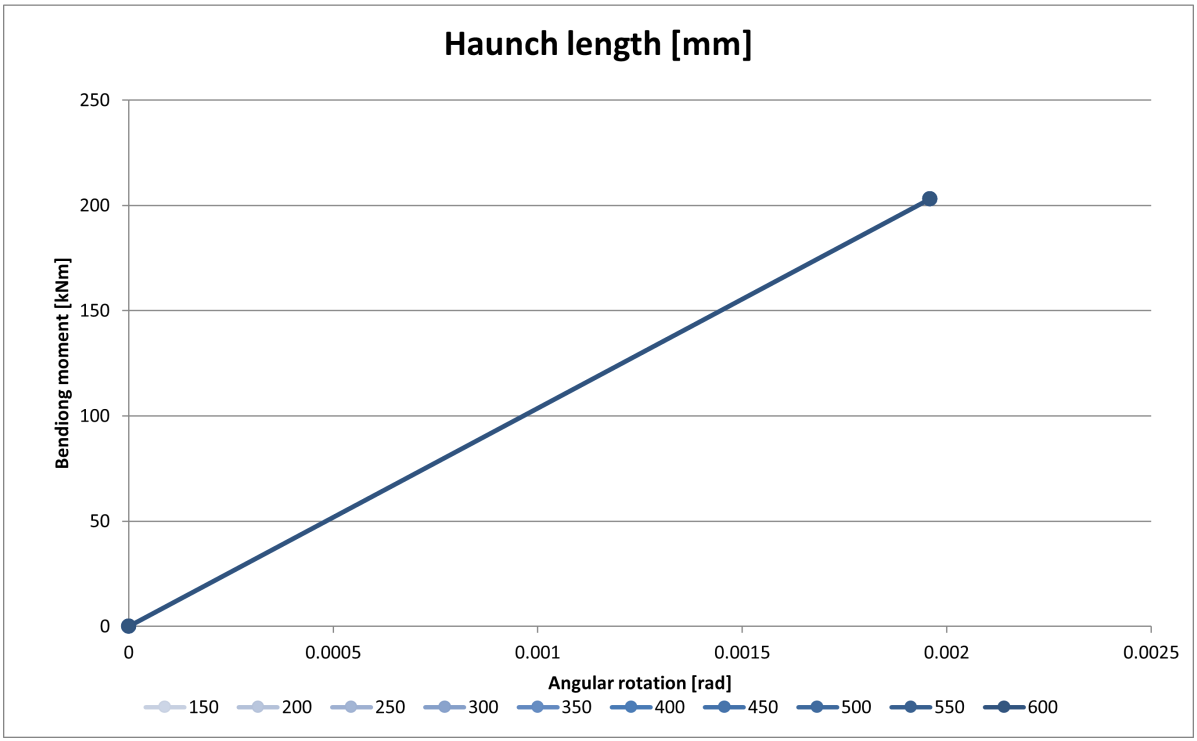


Figure H.10: Bending moment resistance and rotation for increasing haunch length

H.5. HAUNCH HEIGHT

In this subsection the influence of the haunch height is shown. The design used can be found in figure H.8. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360
- No stiffener
- Haunch including haunch flange
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm plate thickness
- Haunch ratio: 1

The height of the haunch varies from 0mm to 500mm with steps of 10mm. Because the ratio of the haunch height over length stays the same when the haunch height increases the haunch length also increases automatically. Besides this also the plate length increases with the haunch height. This plate extension length below the beam increases from 30mm to 530mm.

In figure H.11 the initial rotational stiffness and bending moment resistance can be found for different haunch heights. It can be seen that with an increasing haunch height the stiffness and bending moment resistance also increase. A big jump can be found due to the fact that with a certain haunch height the second bolt row also comes in the tension zone. Meaning that this also plays a part in the stiffness and resistance calculations. The haunch height where this jump appears depend on the loading on the connection and the beam height.

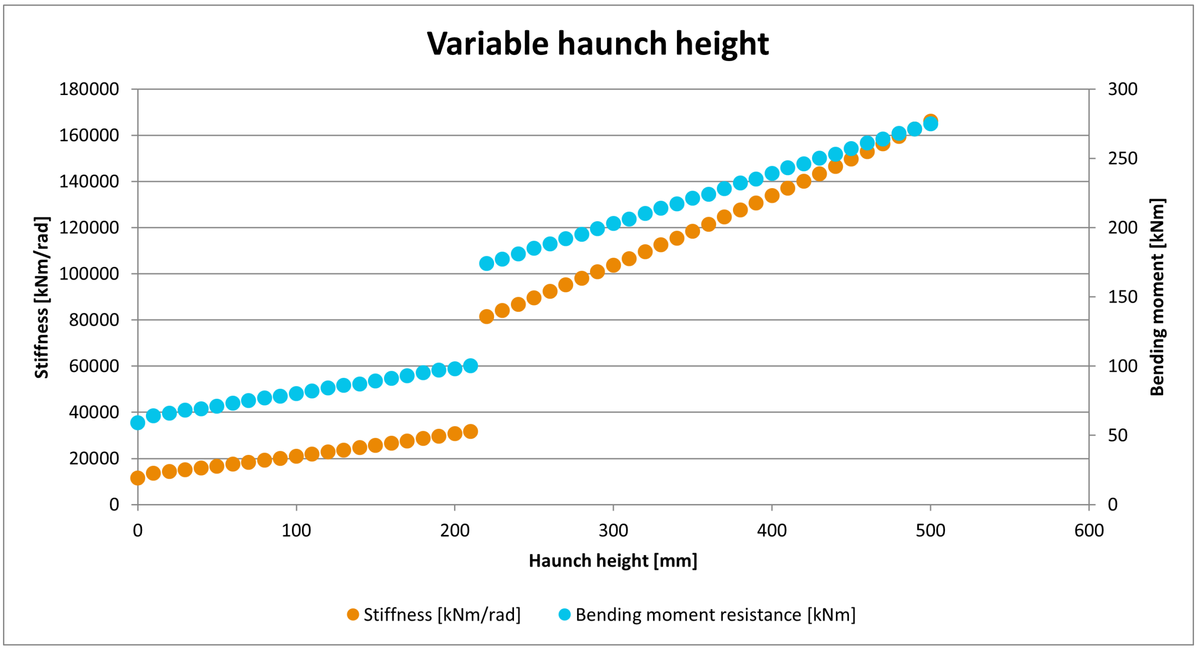


Figure H.11: Initial rotational stiffness and Bending moment resistance for increasing haunch height

In figure H.12 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. When the haunch height is increased the rotational stiffness also increases.

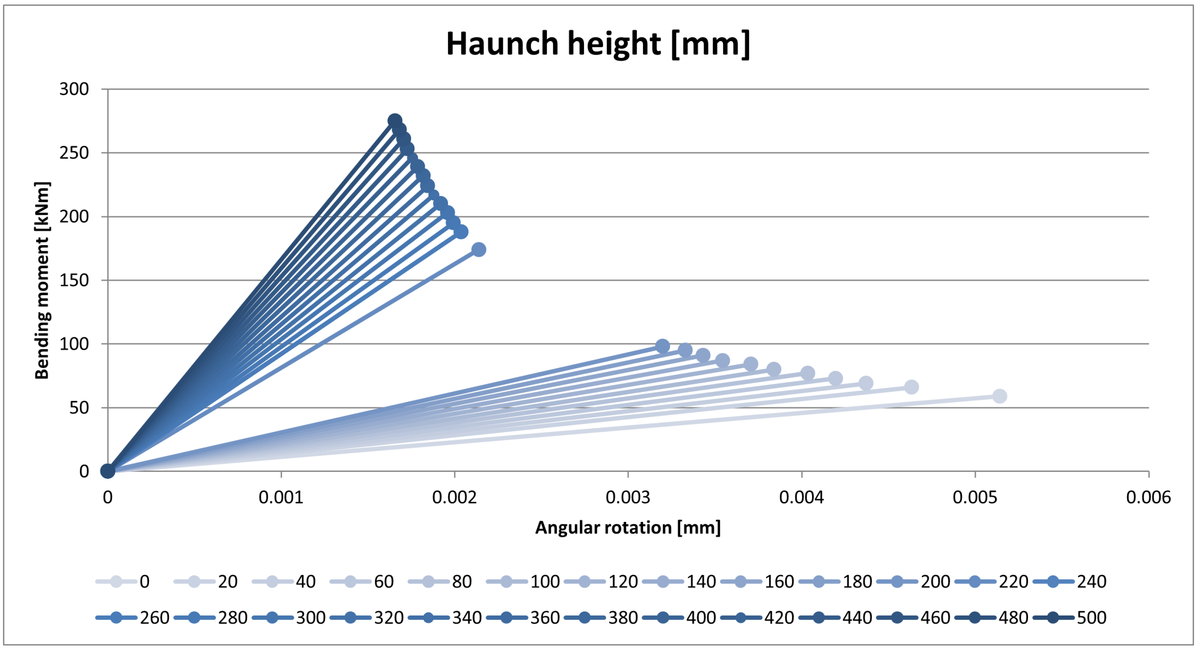


Figure H.12: Bending moment resistance and rotation for increasing haunch height

H.6. HAUNCH HEIGHT INCLUDING TOP STIFFENER IN COLUMN

In this subsection the influence of the haunch height is shown. The design used can be found in figure H.8. The only change is that it has now a top stiffener in the column. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360
- Top stiffener in column
- Haunch including haunch flange
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm plate thickness
- Haunch ratio: 1

The height of the haunch varies from 0mm to 500mm with steps of 10mm. Because the ratio of the haunch height over length stays the same when the haunch height increases the haunch length also increases automatically. Besides this also the plate length increases with the haunch height. This plate extension length below the beam increases from 30mm to 530mm.

In figure H.13 the initial rotational stiffness and bending moment resistance can be found for different haunch heights. It can be seen that with an increasing haunch height the stiffness and bending moment resistance also increase. A big jump can be found due to the fact that with a certain haunch height the second bolt row also comes in the tension zone. Meaning that this increases the stiffness and resistance calculations. The haunch height where this jump appears depend on the loading on the connection and the beam height. If this graph is compared with the graph without a top stiffener it can be seen that only a top stiffener has no influence on the stiffness or bending moment resistance of the connection.

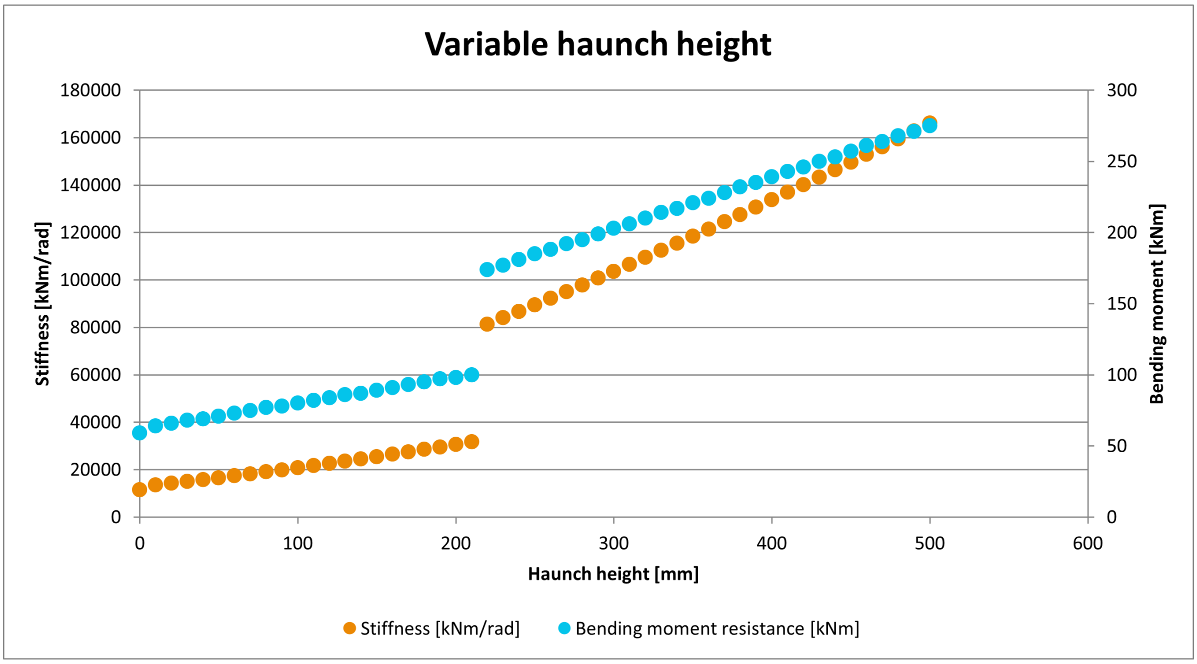


Figure H.13: Initial rotational stiffness and Bending moment resistance for increasing haunch height

In figure H.14 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines shows the initial rotational stiffness. Here it can also be seen that increasing the haunch height increases the initial rotational stiffness.

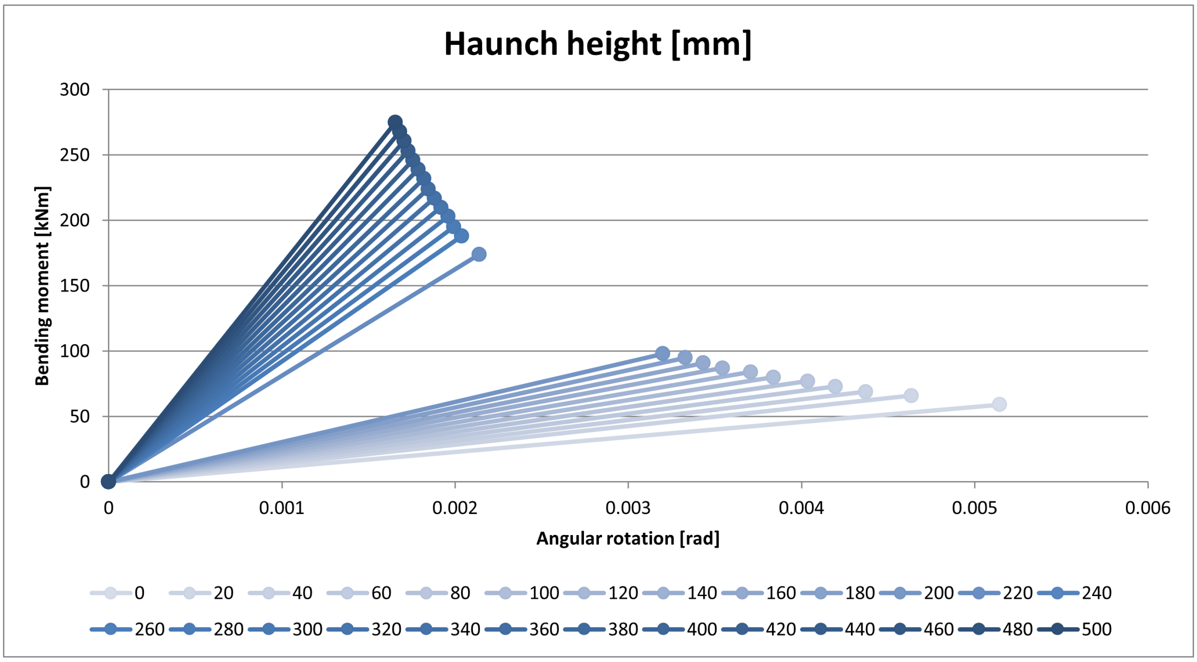


Figure H.14: Bending moment resistance and rotation for increasing haunch height

H.7. HAUNCH HEIGHT INCLUDING TOP AND BOTTOM STIFFENER IN COLUMN

In this subsection the influence of the haunch height is shown. The design used can be found in figure H.8. The only change is that it has now a top and bottom stiffener in the column. The values in this design are:

- Beam profile: IPE 500

- Column profile: HEB 360
- Top and bottom stiffener in column
- Haunch including haunch flange
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm plate thickness
- Haunch ratio: 1

The height of the haunch varies from 0mm to 500mm with steps of 10mm. Because the ratio of the haunch height over length stays the same when the haunch height increases the haunch length also increases automatically. Besides this also the plate length increases with the haunch height. This plate extension length below the beam increases from 30mm to 530mm.

In figure H.15 the initial rotational stiffness and bending moment resistance can be found for different haunch heights. It can be seen that with an increasing haunch height the stiffness and bending moment resistance also increase. A big jump can be found due to the fact that with a certain haunch height the second bolt row also comes in the tension zone. Meaning that this increases the stiffness and resistance calculations. The haunch height where this jump appears depend on the loading on the connection and the beam height. If this graph is compared with the graph with a top stiffener it can be seen that both a top and a bottom stiffener can have a big influence on the stiffness of the connection. Adding stiffeners has no influence on the bending moment resistance of the connection.

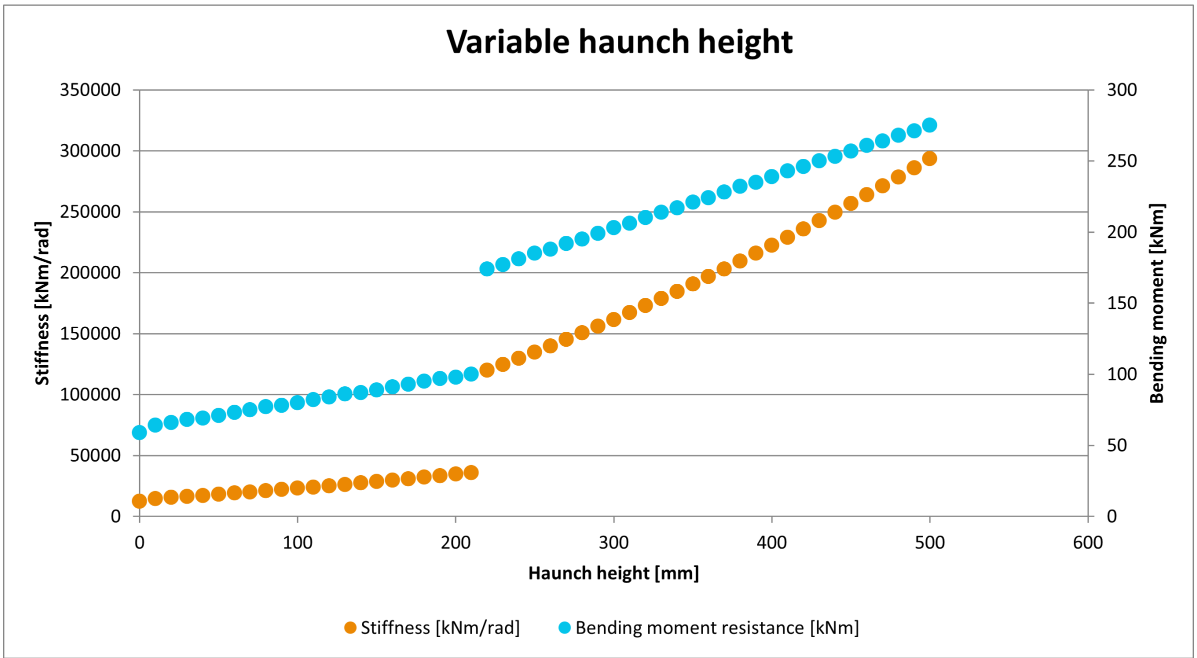


Figure H.15: Initial rotational stiffness and Bending moment resistance for increasing haunch height

In figure H.16 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that increasing the haunch height increases the initial rotational stiffness.

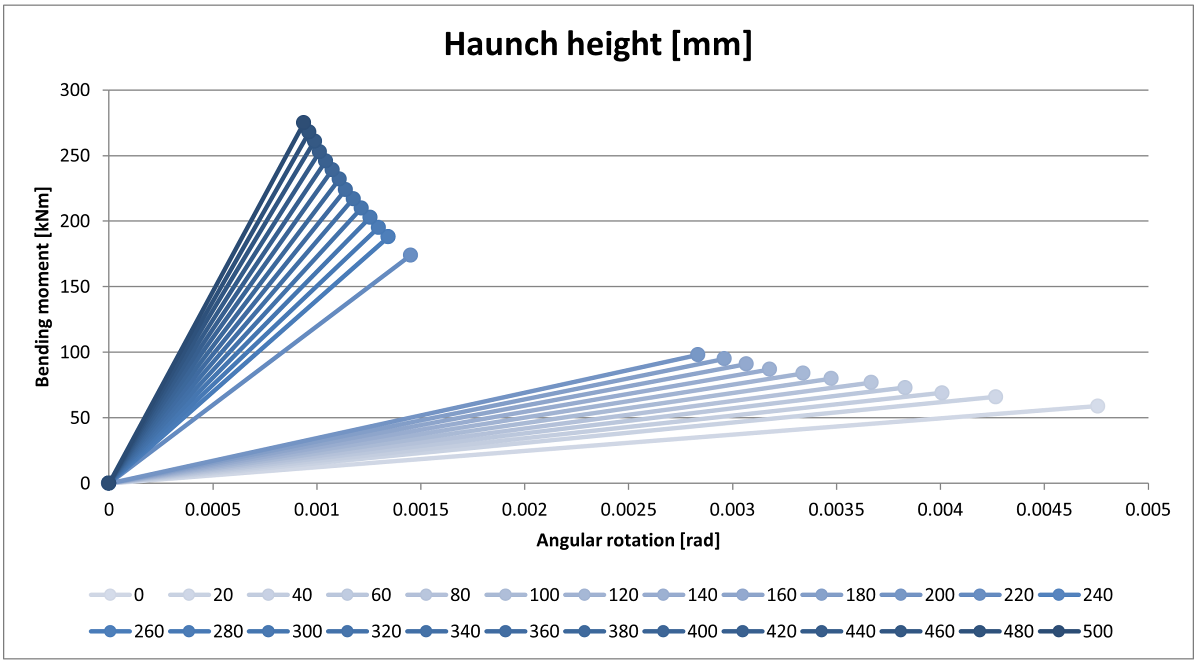


Figure H.16: Bending moment resistance and rotation for increasing haunch height

H.8. HAUNCH HEIGHT INCLUDING TOP AND BOTTOM STIFFENER IN COLUMN AND A STIFFENER IN THE BEAM

In this subsection the influence of the haunch height is shown. The design used can be found in figure H.8. The only change is that it has now stiffeners in the top and bottom of the column and stiffeners in the beam. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360
- Top and bottom stiffener in column and stiffener in beam
- Haunch including haunch flange
- M16:8.8 bolts
- 2 Bolt columns
- 2 Bolt rows
- 15 mm plate thickness
- Haunch ratio: 1

The height of the haunch varies from 0mm to 500mm with steps of 10mm. Because the ratio of the haunch height over length stays the same when the haunch height increases the haunch length also increases automatically. Besides this also the plate length increases with the haunch height. This plate extension length below the beam increases from 30mm to 530mm.

In figure H.17 the initial rotational stiffness and bending moment resistance can be found for different haunch heights. It can be seen that with an increasing haunch height the stiffness and bending moment resistance also increase. A big jump can be found due to the fact that with a certain haunch height the second bolt row also comes in the tension zone. Meaning that this increases the stiffness and resistance calculations. The haunch height where this jump appears depend on the loading on the connection and the beam height. If this graph is compared with the graph with both a top and bottom stiffener it can be seen that adding a stiffener in the beam has no influence on the stiffness or the bending moment resistance of the connection.

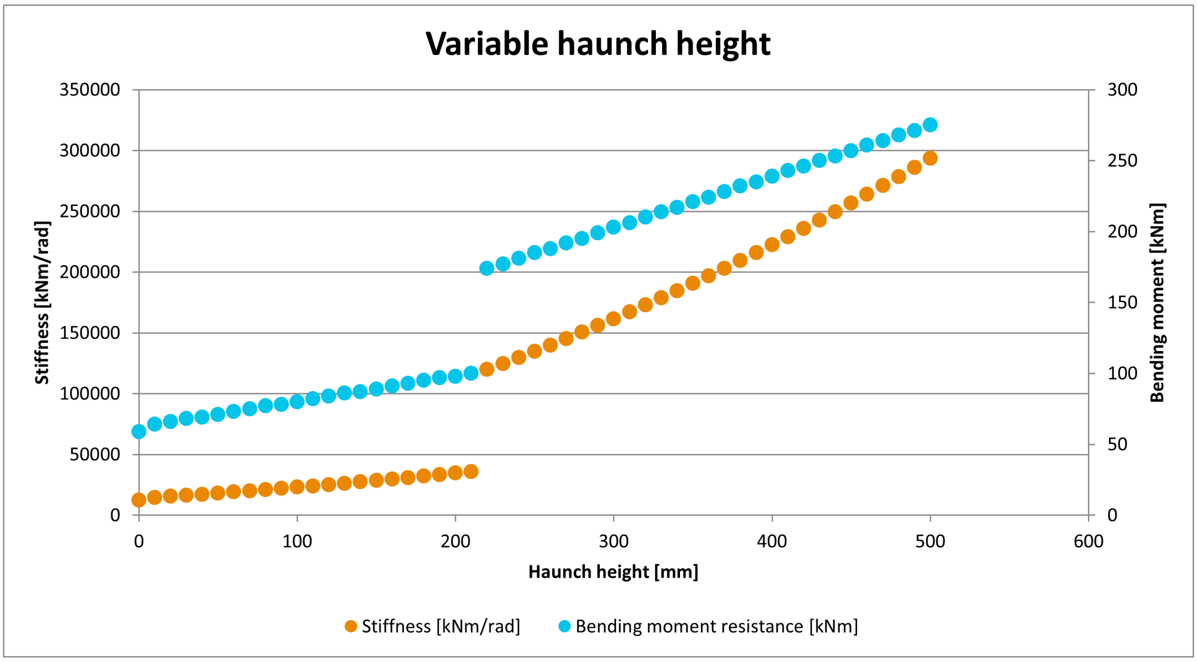


Figure H.17: Initial rotational stiffness and Bending moment resistance for increasing haunch height

In figure H.18 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can be seen that increasing the haunch height increases the initial rotational stiffness.

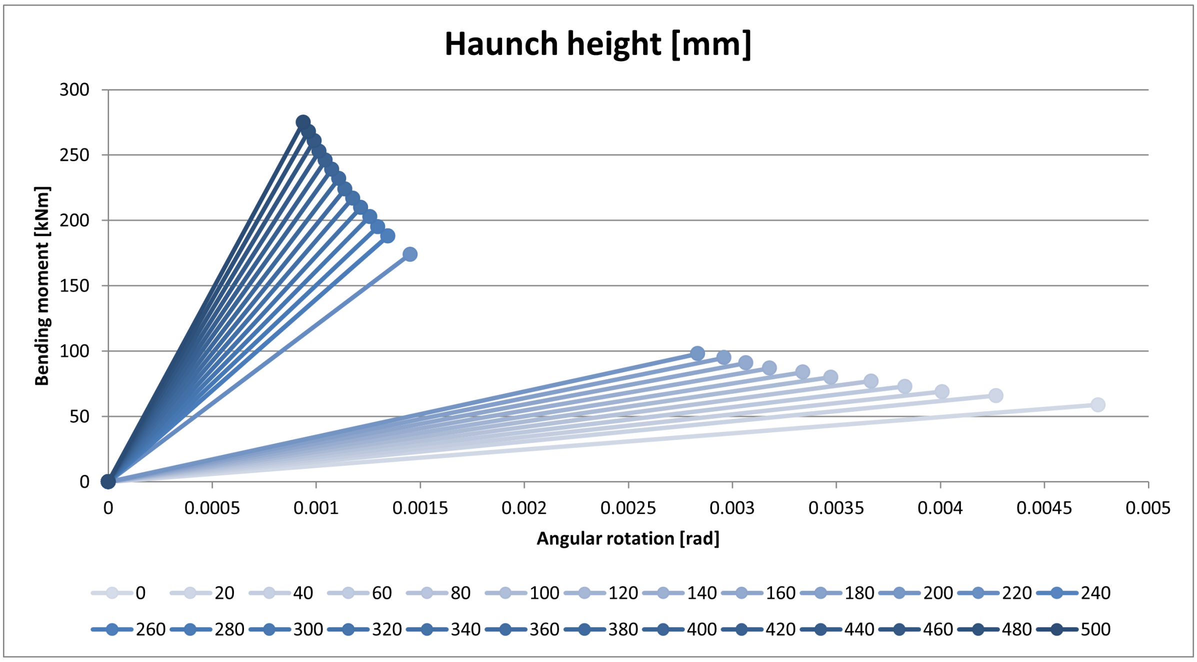


Figure H.18: Bending moment resistance and rotation for increasing haunch height

H.9. Number of bolt rows

In this subsection the influence of the number of bolt rows is shown. The design used can be found in figure H.19. The values in this design are:

- Beam profile: IPE 500

- Column profile: HEB 360
- No stiffener
- No Haunch
- M16:8.8 bolts
- 2 Bolt columns
- 15 mm plate thickness
- Plate length 530 mm

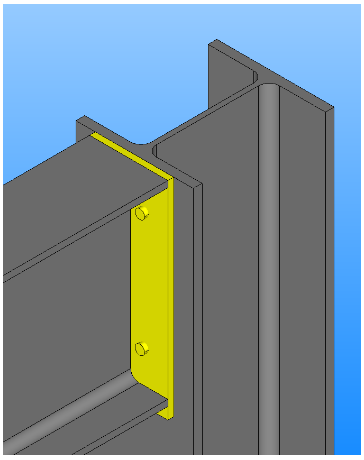


Figure H.19: Number of bolt-rows study design

The number of bolt-rows vary from 2 to 8 and are evenly distributed if possible.

In figure H.20 the initial rotational stiffness and bending moment resistance can be found for different number of bolt rows. It can be seen that the stiffness and bending moment increase if the number of bolt rows increase. and that the bending moment increases linearly and the stiffness increases non-linear. The stiffness increases most in the first steps and than increases much slower. This is because the distance to the compression zone decreases for each extra bolt-row which means that each bolt-row has less influence.

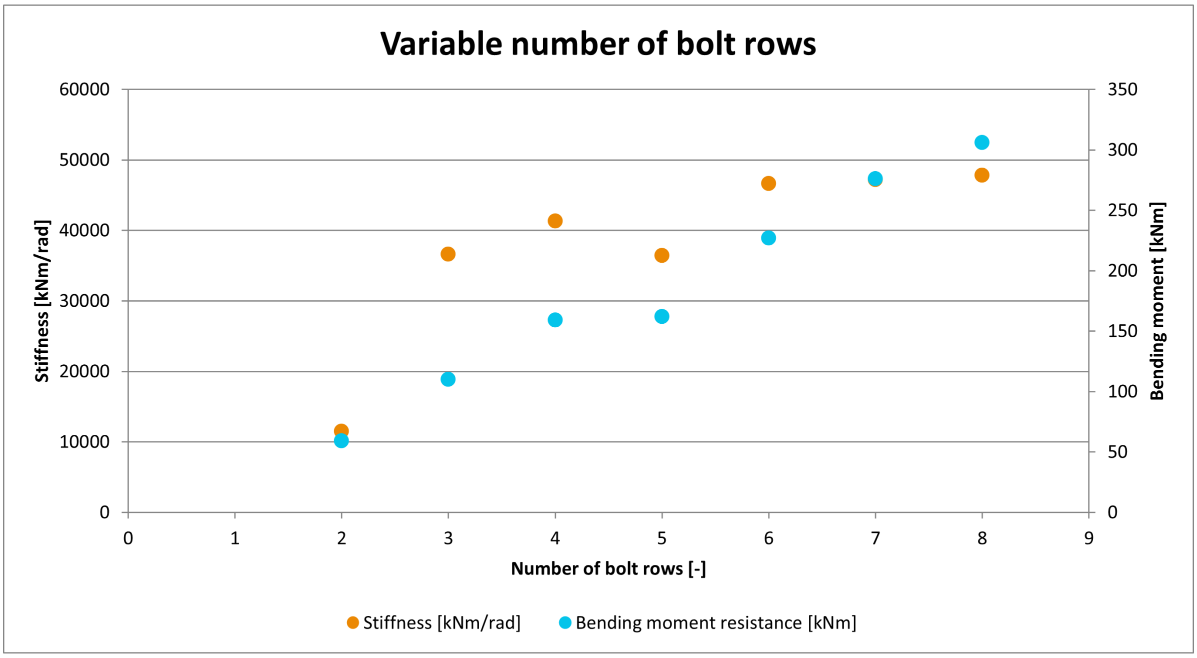


Figure H.20: Initial rotational stiffness and Bending moment resistance for increasing number of bolt-rows

In figure H.21 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that the stiffness increases less and less with each additional bolt-row.

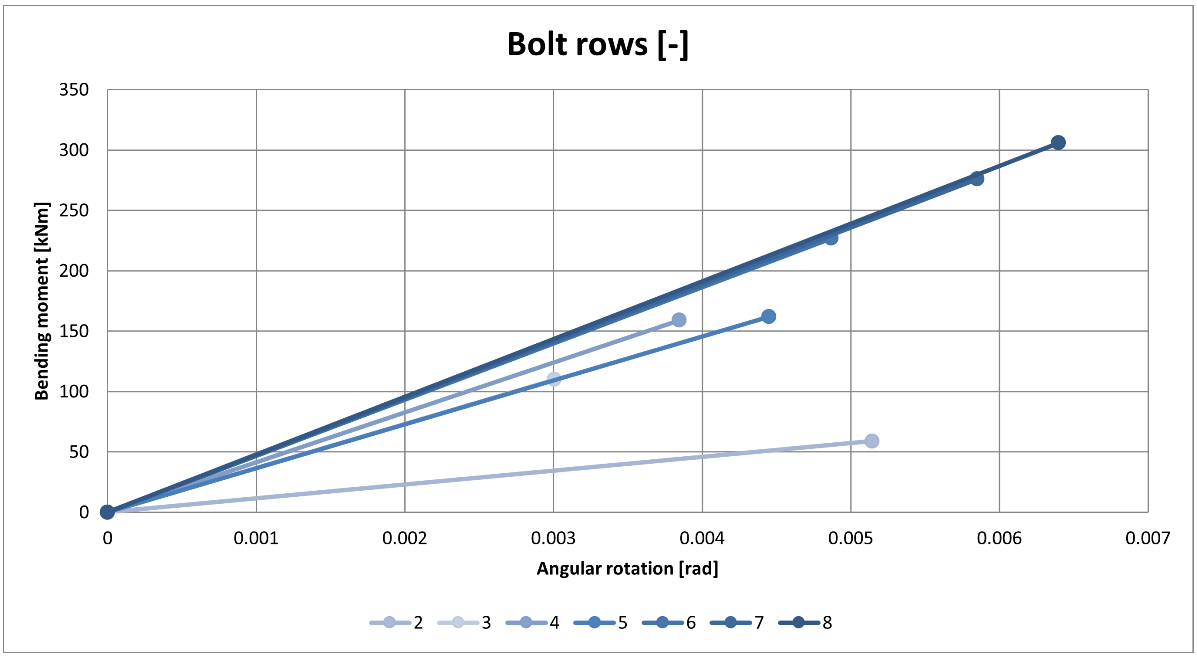


Figure H.21: Bending moment resistance and rotation for increasing number of bolt-rows

H.10. NUMBER OF BOLT ROWS INCLUDING ONE BOLT-ROW ABOVE THE BEAM

In this subsection the influence of the number of bolt rows is shown. The design used can be found in figure H.22. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360

- No stiffener
- No Haunch
- M16:8.8 bolts
- 2 Bolt columns
- 15 mm plate thickness
- Plate length 530 mm
- 1 bolt-row above beam

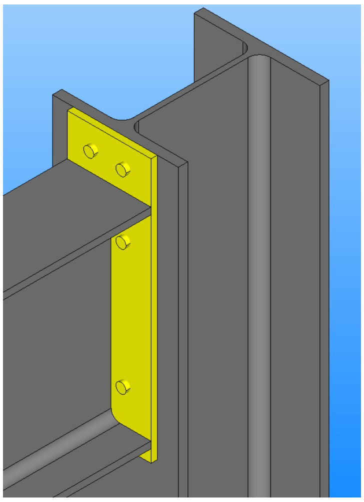


Figure H.22: Number of bolt-rows study design

The number of bolt-rows vary from 2 to 8 and are evenly distributed if possible.

In figure H.23 the initial rotational stiffness and bending moment resistance can be found for different number of bolt rows. It can be seen that the stiffness and bending moment increase if the number of bolt rows increase. and that the bending moment increases almost linearly and the stiffness increases linear. The stiffness increases almost linear compared to the graph without top stiffener because this bolt-row is highest and thus has the biggest influence on the stiffness and the bending moment resistance.

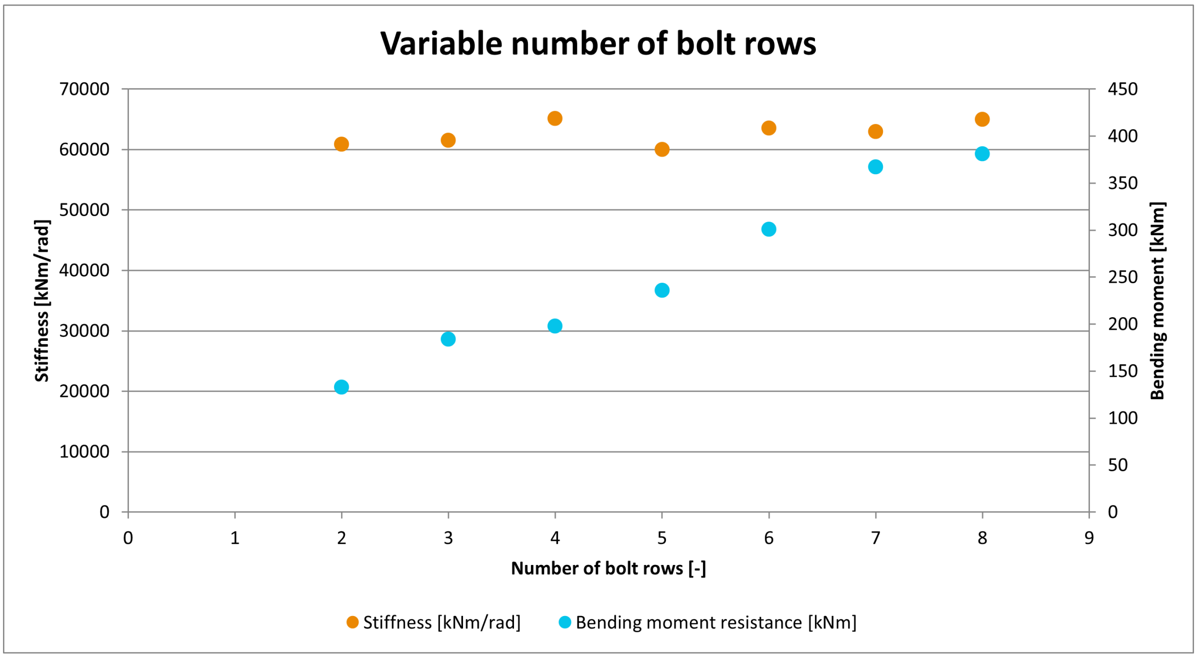


Figure H.23: Initial rotational stiffness and Bending moment resistance for increasing number of bolt-rows

In figure H.24 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that there are almost no changes for an increasing number of bolt-rows.

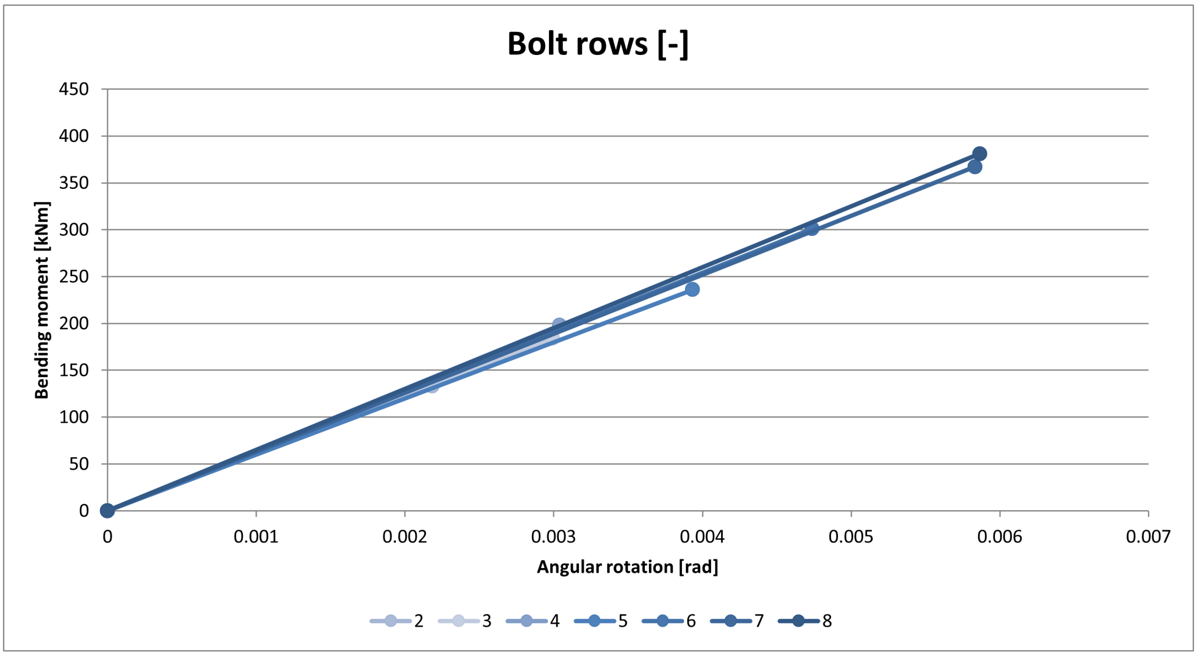


Figure H.24: Bending moment resistance and rotation for increasing number of bolt-rows

H.11. PLATE LENGTH ABOVE THE BEAM

In this subsection the influence of the plate length above the beam is shown. The design used can be found in figure H.25. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360

- No stiffener
- No Haunch
- M16:8.8 bolts
- 2 Bolt columns
- 3 Bolt rows
- 15 mm platethickness
- Plate length 530 mm
- 1 bolt-row above beam

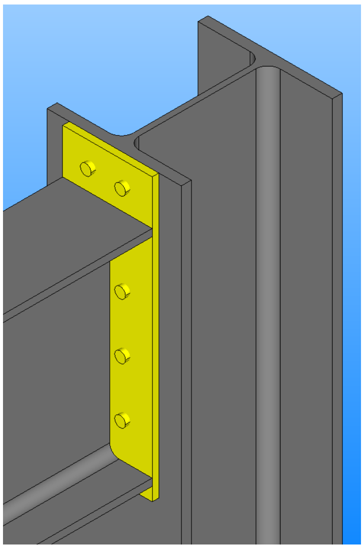


Figure H.25: Plate length above beam study design

The plate length above the beam varies from 50 mm to 300 mm with steps of 10 mm.

In figure H.26 the initial rotational stiffness and bending moment resistance can be found for different number of bolt rows. It can be seen that the stiffness and bending moment slightly increase at first but then decrease until it gets stable. This is because the further away the top bolt row moves the less influence do the other bolt rows have. This has more influence than the increasement of the arm of the top bolt-row. Which means that it decreases the stiffness and the moment resistance.

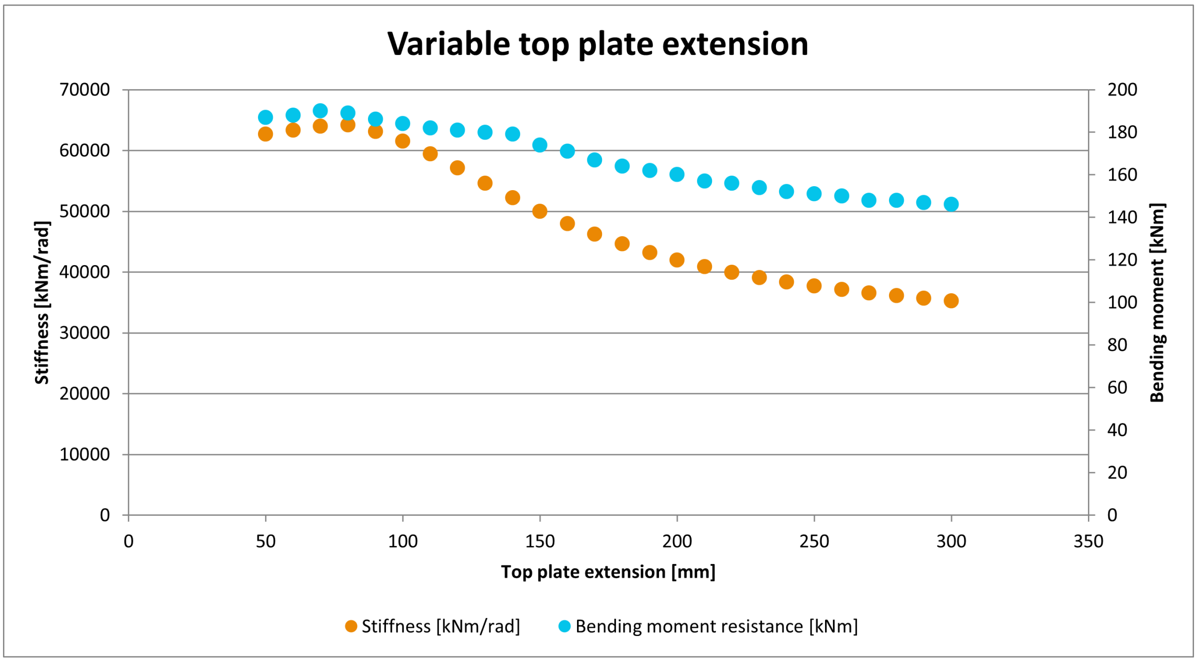


Figure H.26: Initial rotational stiffness and Bending moment resistance for increasing plate length above beam

In figure H.27 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that increasing the length decreases the stiffness.

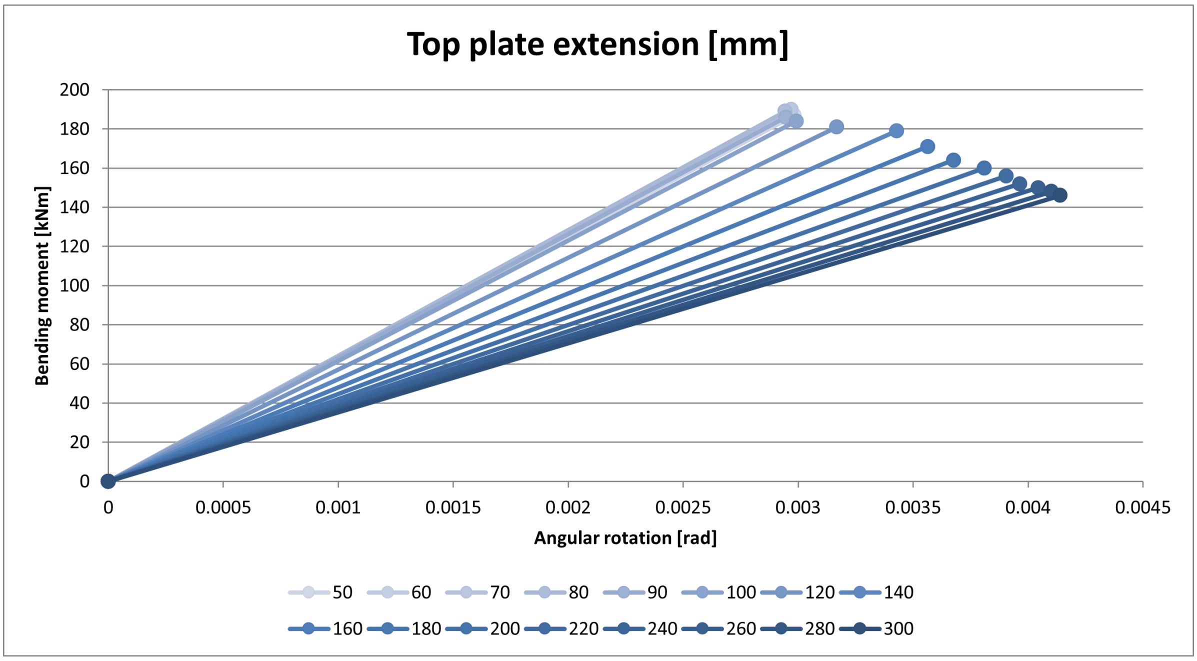


Figure H.27: Bending moment resistance and rotation for increasing plate length above beam

H.12. BOLT SIZE

In this subsection the influence of bolt size is shown. The design used can be found in figure H.28. The values in this design are:

- Beam profile: IPE 500

- Column profile: HEB 360
- No stiffener
- No Haunch
- 2 Bolt columns
- 3 Bolt rows
- 15 mm platethickness
- Plate length 530 mm

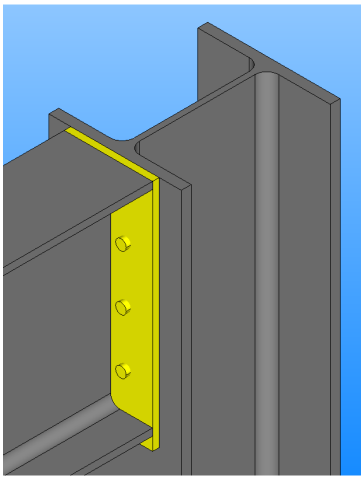


Figure H.28: Plate length above beam study design

The Bolt sizes used are M10, M12, M14, M16, M18, M20, M22, M24, M27 and M30.

In figure H.29 the initial rotational stiffness and bending moment resistance can be found for different bolt sizes. It can be seen that the stiffness and bending moment increase if the bolt size increases. This can be explained because the bolt size influence multiple components due to the influence on the effective yield lengths and the bolt tension, shear and bearing resistance.

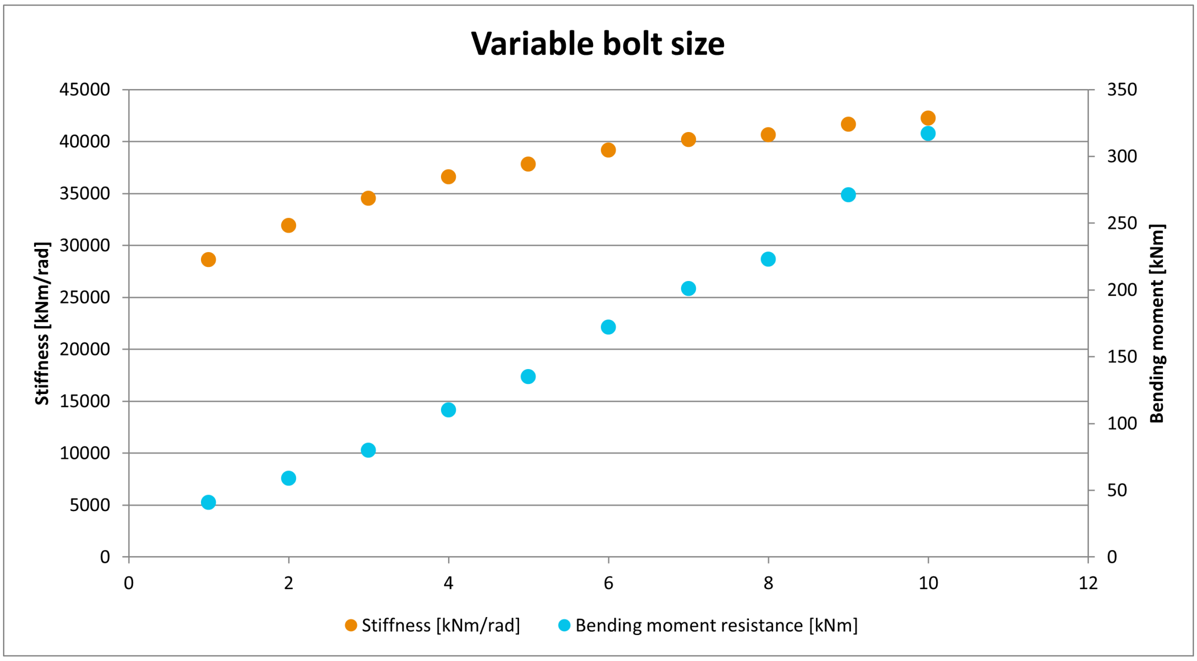


Figure H.29: Initial rotational stiffness and Bending moment resistance for increasing the bolt size

In figure H.30 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines shows the initial rotational stiffness. Here it can also be seen that increasing the bolt size increases the stiffness.

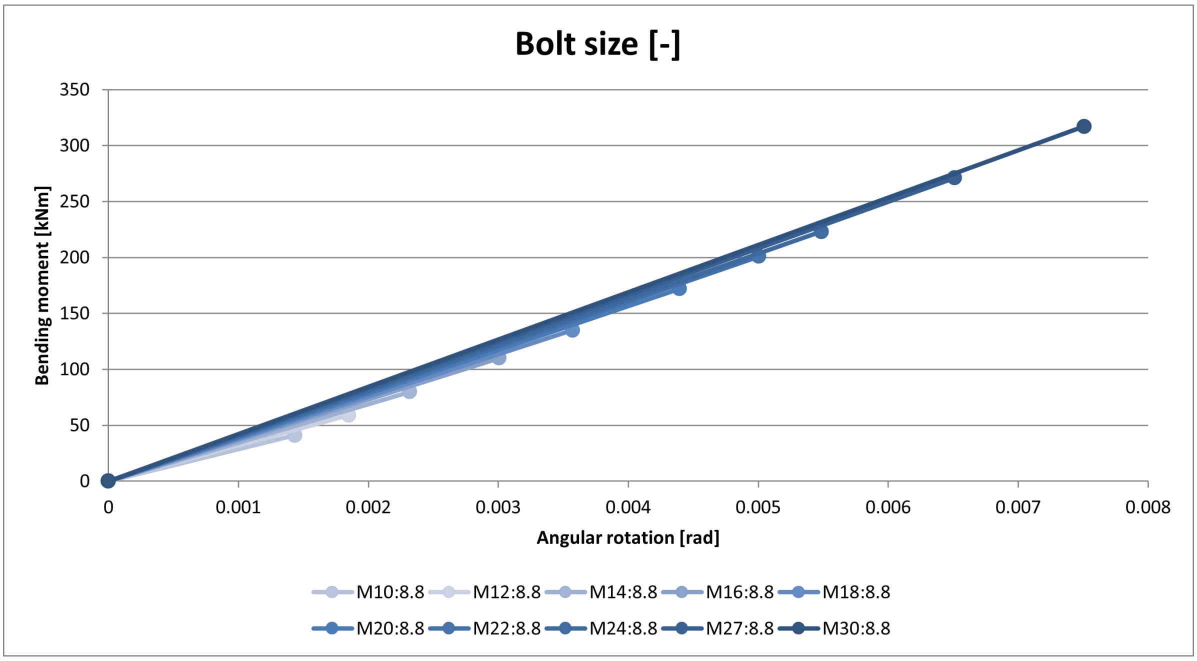


Figure H.30: Bending moment resistance and rotation for increasing bolt size

H.13. BOLT STRENGTH CLASS

In this subsection the influence of bolt strength is shown. The design used can be found in figure H.28. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360

- No stiffener
- No Haunch
- 2 Bolt columns
- 3 Bolt rows
- 15 mm platethickness
- Plate length 530 mm

The strength classes used are 4.6, 4.8, 5.6, 5.8, 6.8, 8.8 and 10.9

In figure H.31 the initial rotational stiffness and bending moment resistance can be found for different bolt strength classes. It can be seen that the stiffness stays the same and the bending moment increase if the bolt strength increases. This can be explained because the bolt strength influence multiple component due to the bolt tension and shear resistance.

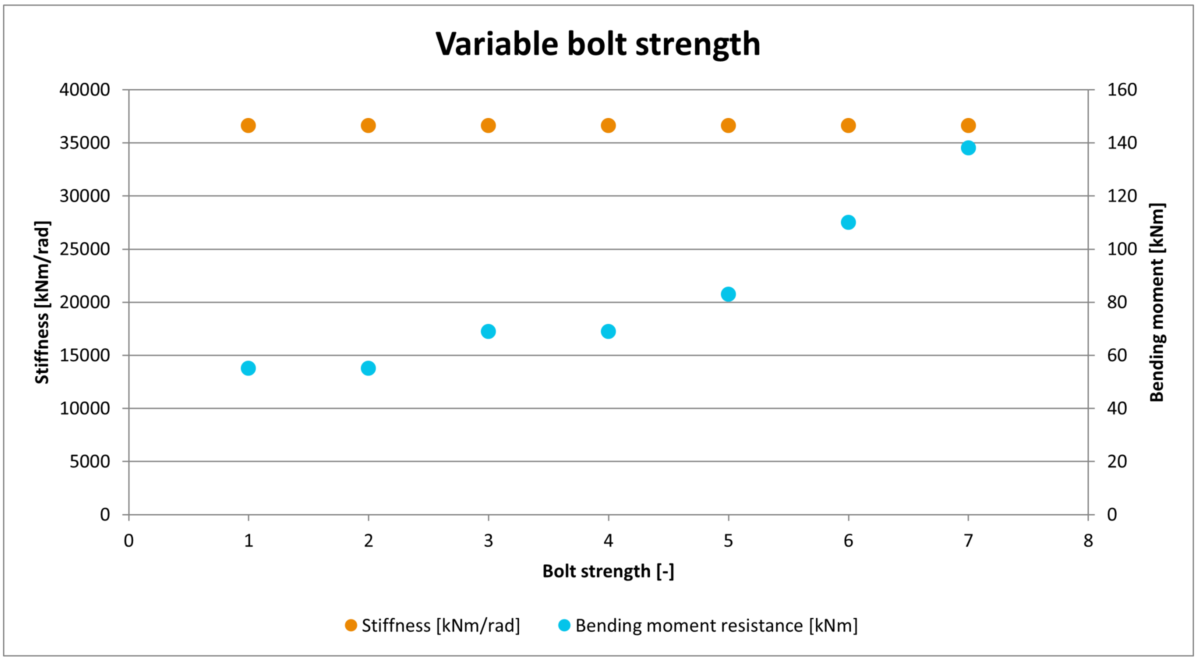


Figure H.31: Initial rotational stiffness and Bending moment resistance for increasing the bolt strenght

In figure H.32 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that increasing the bolt strength has no influence on the stiffness.

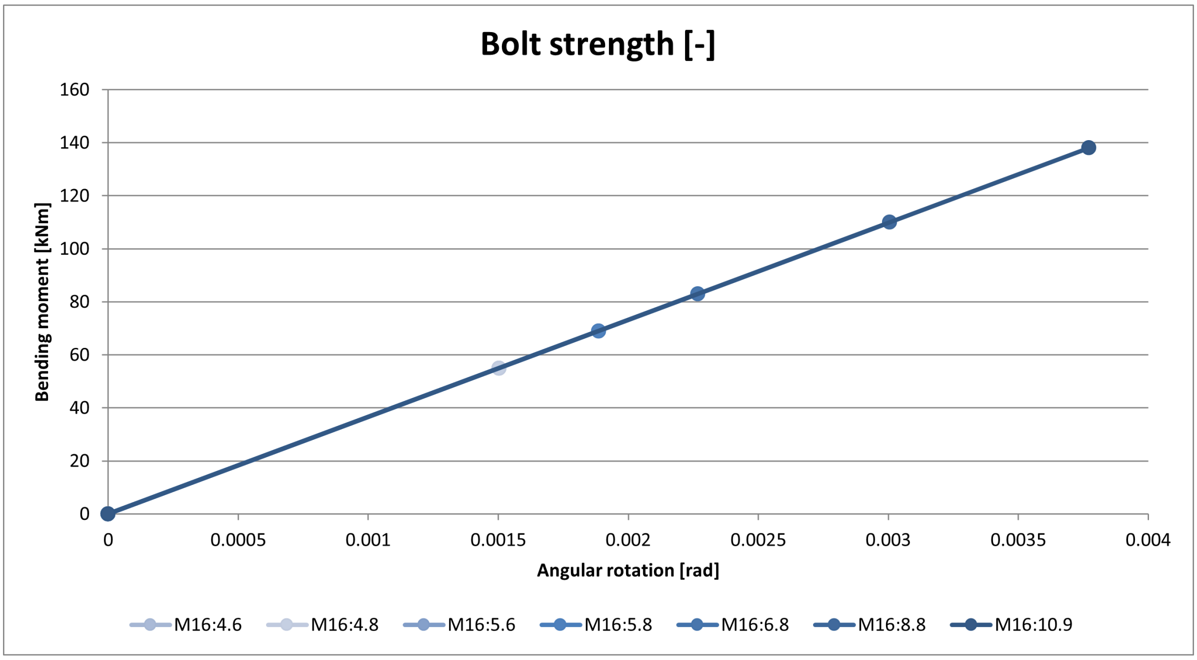


Figure H.32: Bending moment resistance and rotation for increasing bolt strength

H.14. PLATE THICKNESS

In this subsection the influence of plate thickness is shown. The design used can be found in figure H.28. The values in this design are:

- Beam profile: IPE 500
- Column profile: HEB 360
- No stiffener
- No Haunch
- 2 Bolt columns
- 3 Bolt rows
- M15:8.8 bolts
- Plate length 530 mm

The plate thicknesses used are 3, 4, 5, 6, 8, 10, 12, 15, 20, 25 and 30

In figure H.33 the initial rotational stiffness and bending moment resistance can be found for different plate thicknesses. It can be seen that the stiffness and the bending moment increase in the same way if the plate thickness increases. The plate thickness only has a limited influence because at a certain moment another component becomes weakest and influences the bending moment the most.

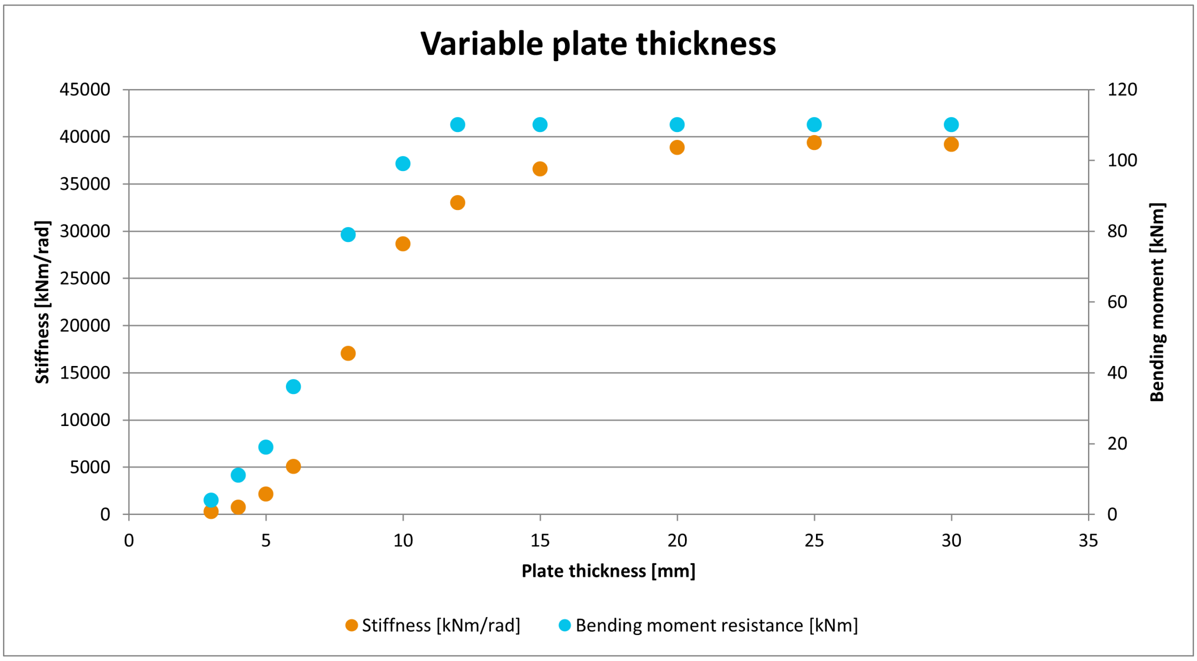


Figure H.33: Initial rotational stiffness and Bending moment resistance for increasing plate thickness

In figure H.34 the bending moment resistance is shown on the y-axis and the rotation of the connection on the x-axis. The angle of the lines with the x-axis shows the initial rotational stiffness. Here it can also be seen that increasing the plate thickness also increases the stiffness up till a certain point.

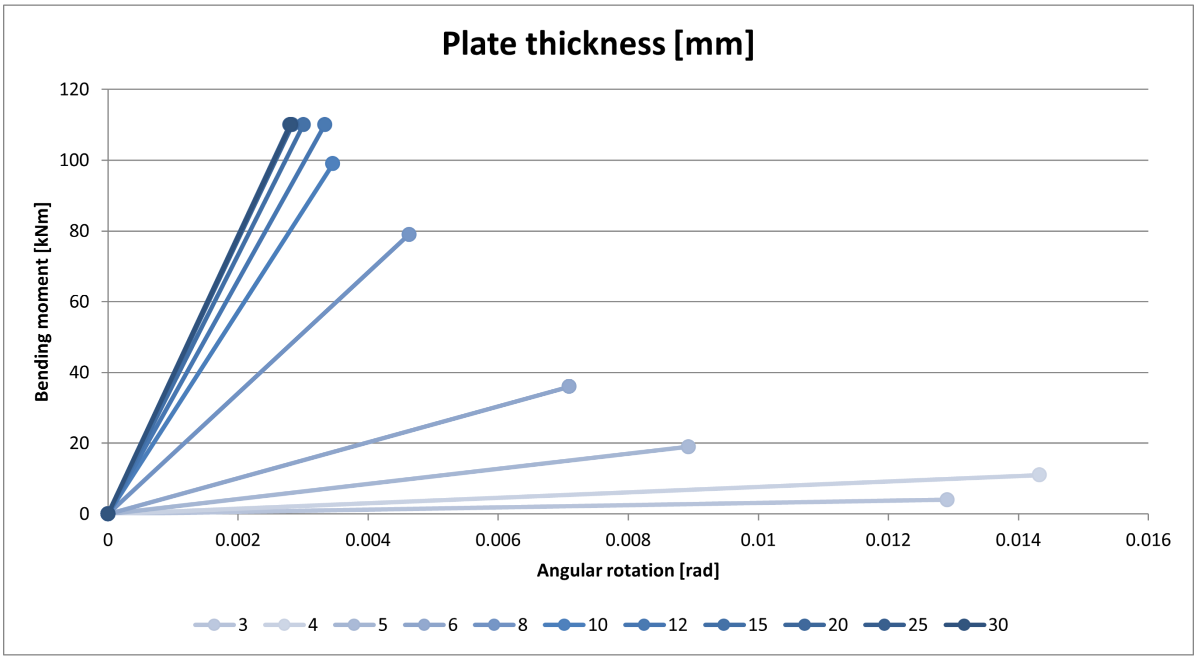


Figure H.34: Bending moment resistance and rotation for increasing plate thickness

I

Result graphs

In this appendix all the result graphs from the case study are presented. These results are divided into 4 categories:

- Fixed number of purlins graphs for hinged column-base connection
- Fixed beam-column connection graphs for hinged column-base connection
- Fixed number of purlins graphs for rigid column-base connection
- Fixed beam-column connection graphs for rigid column-base connection

I.1. Fixed number of purlins graphs for hinged column-base connection

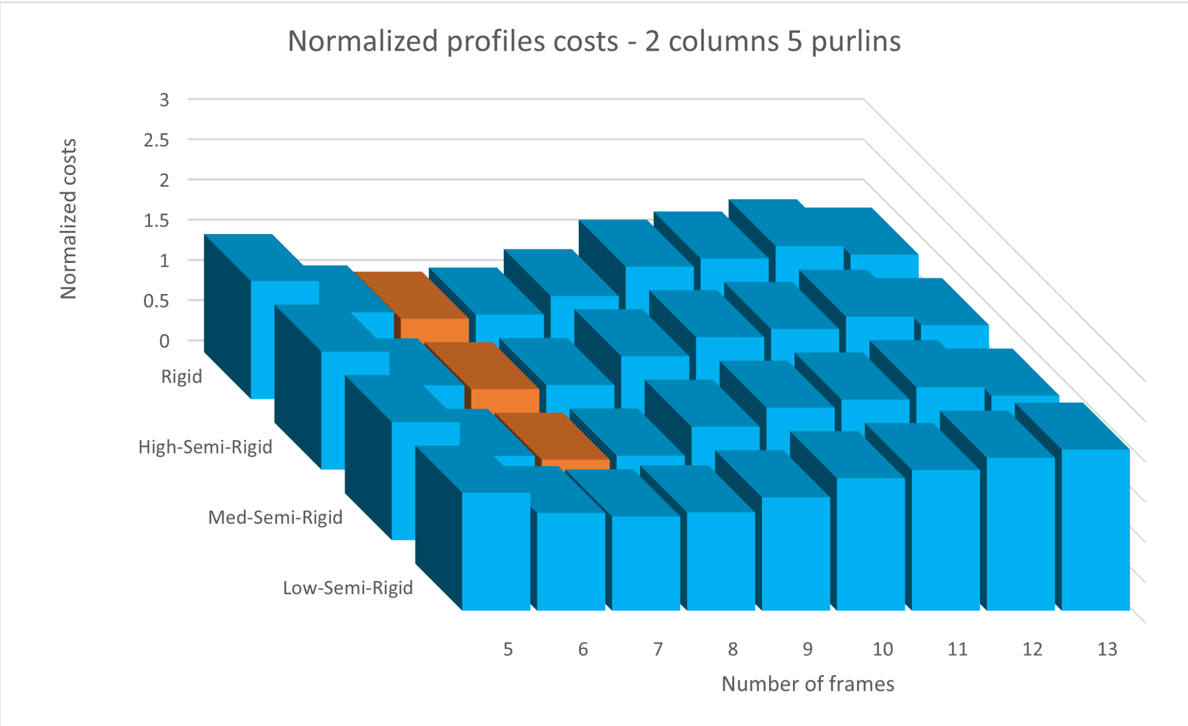


Figure I.1: Normalized profile costs for 5 purlins

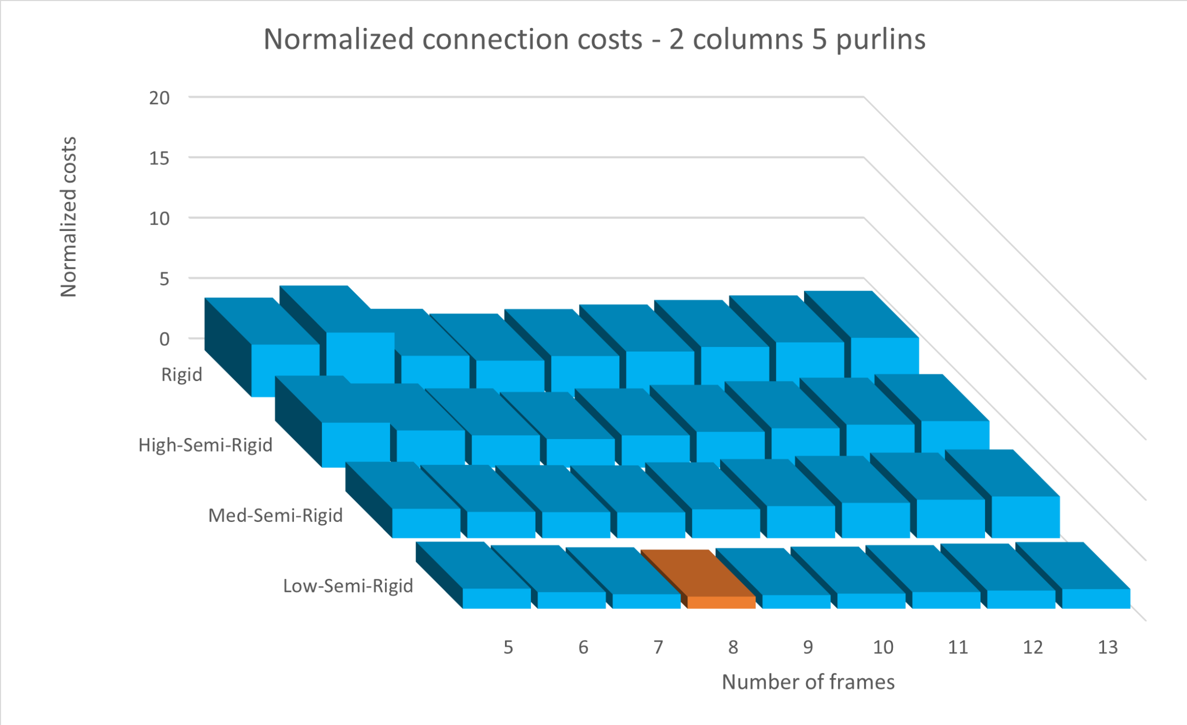


Figure I.2: Normalized connection costs for 5 purlins

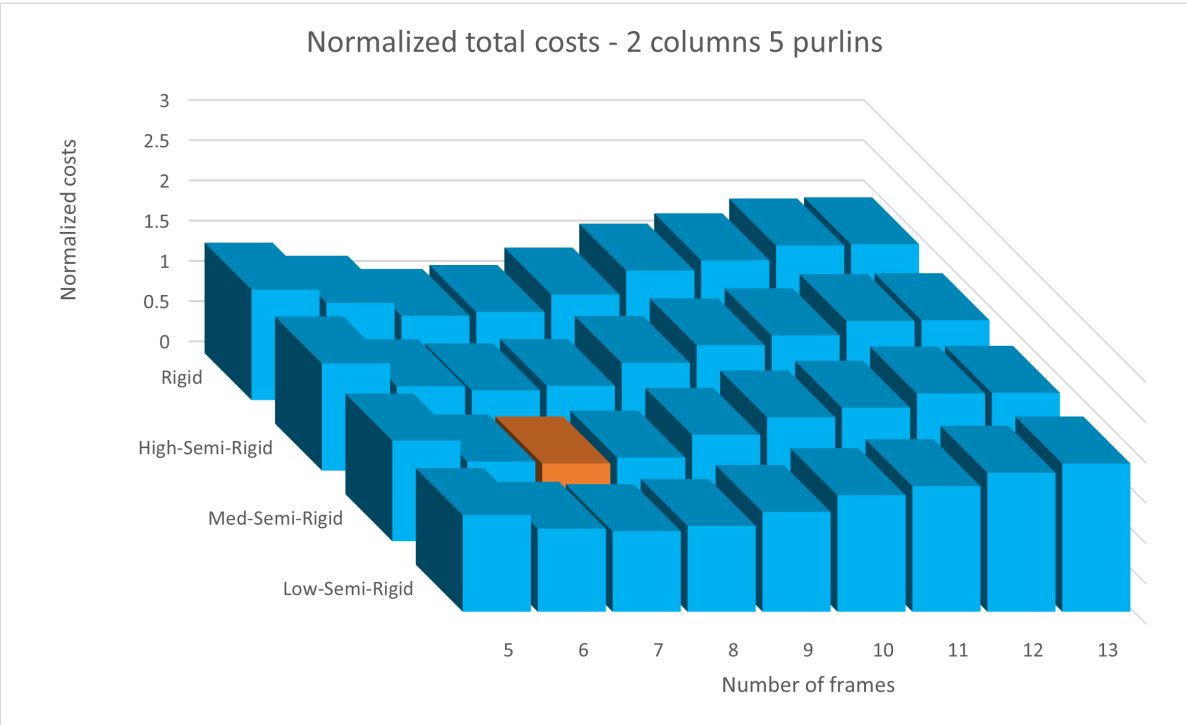


Figure I.3: Normalized total costs for 5 purlins

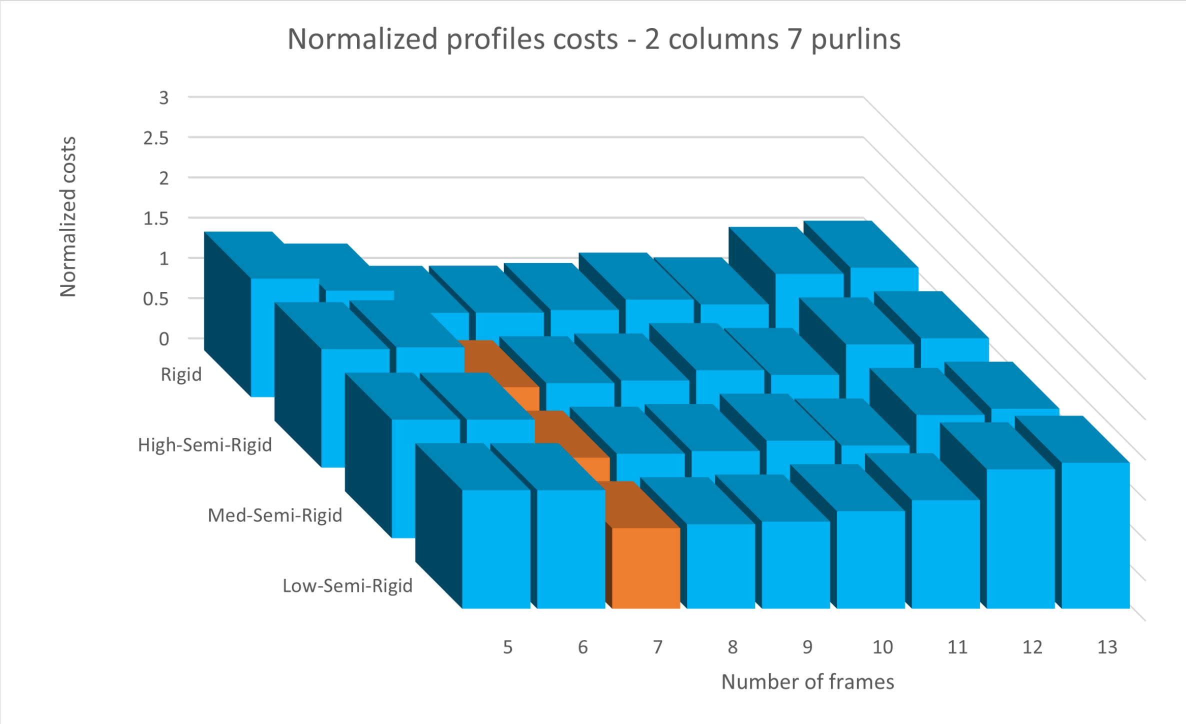


Figure I.4: Normalized profile costs for 7 purlins

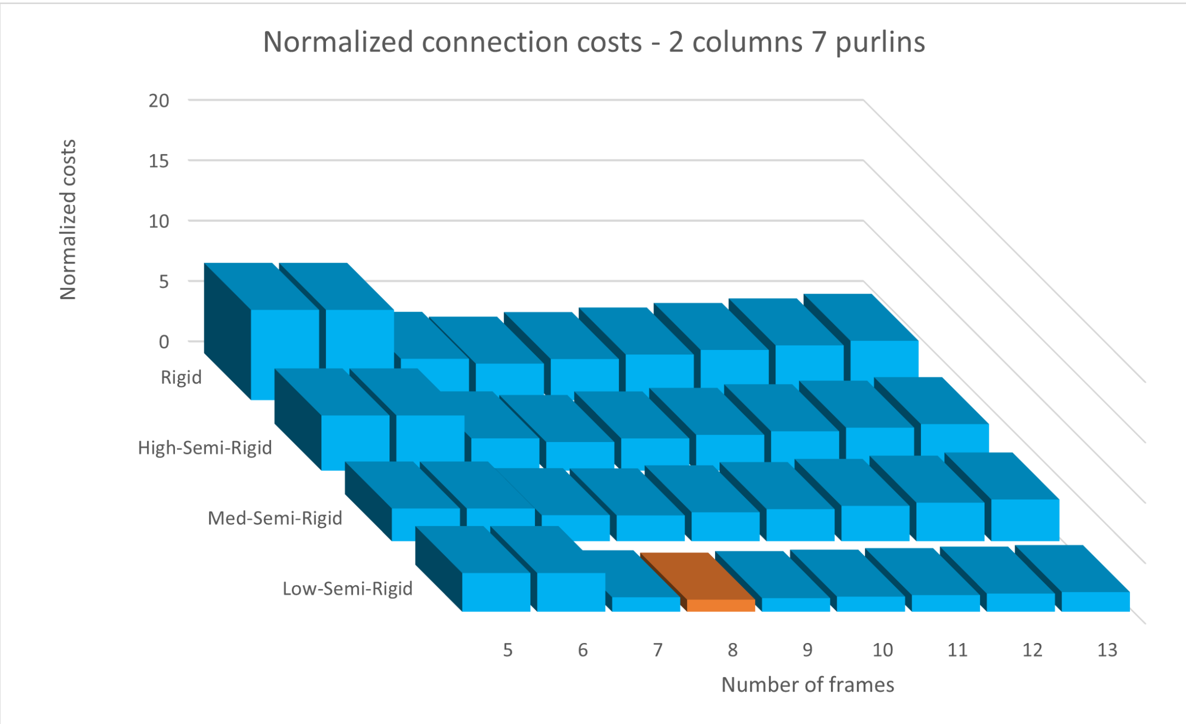


Figure I.5: Normalized connection costs for 7 purlins

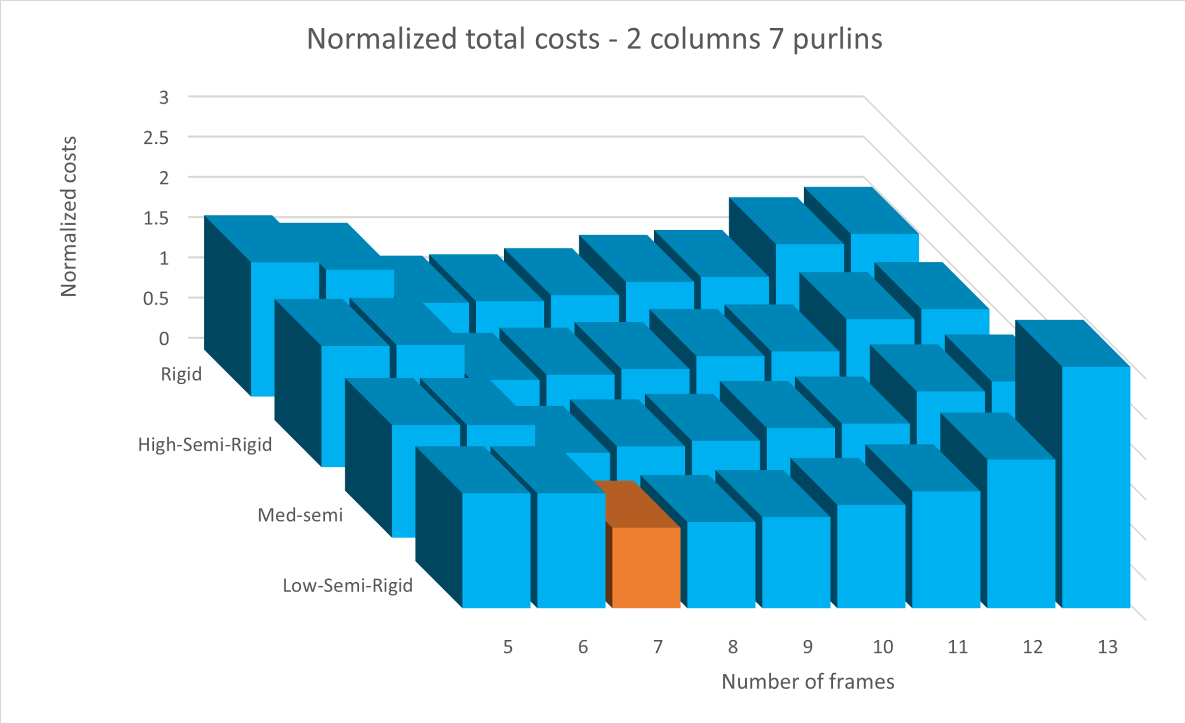


Figure I.6: Normalized total costs for 7 purlins

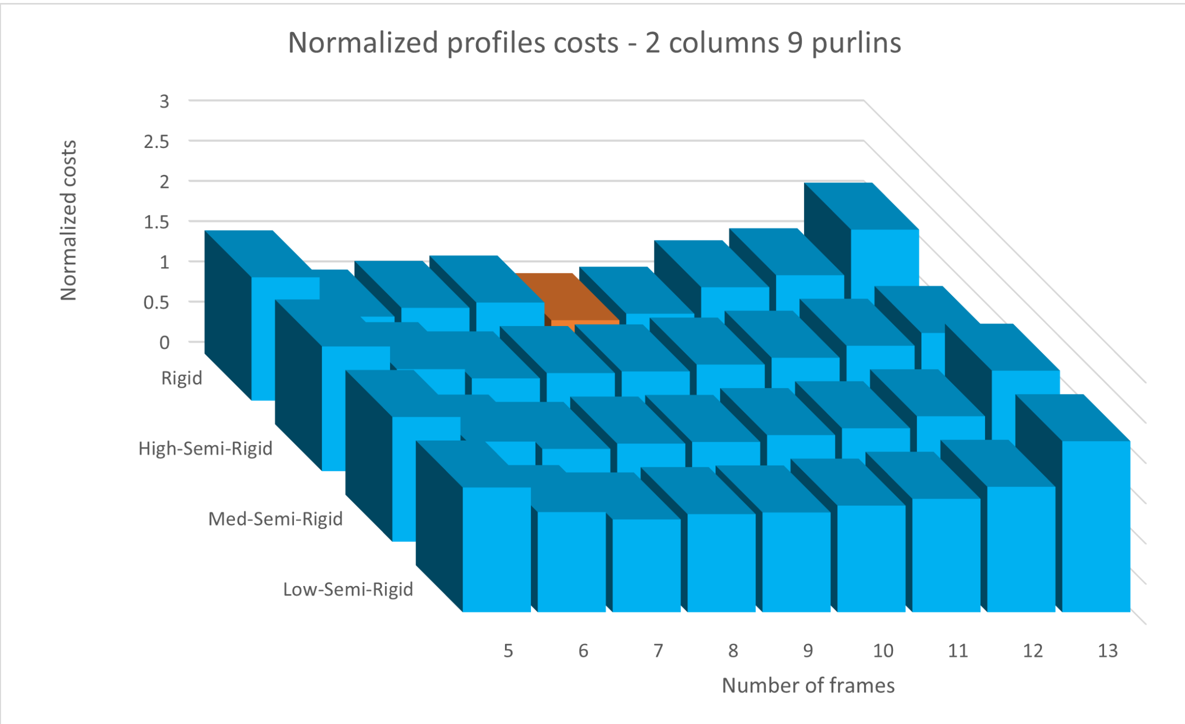


Figure I.7: Normalized profile costs for 9 purlins

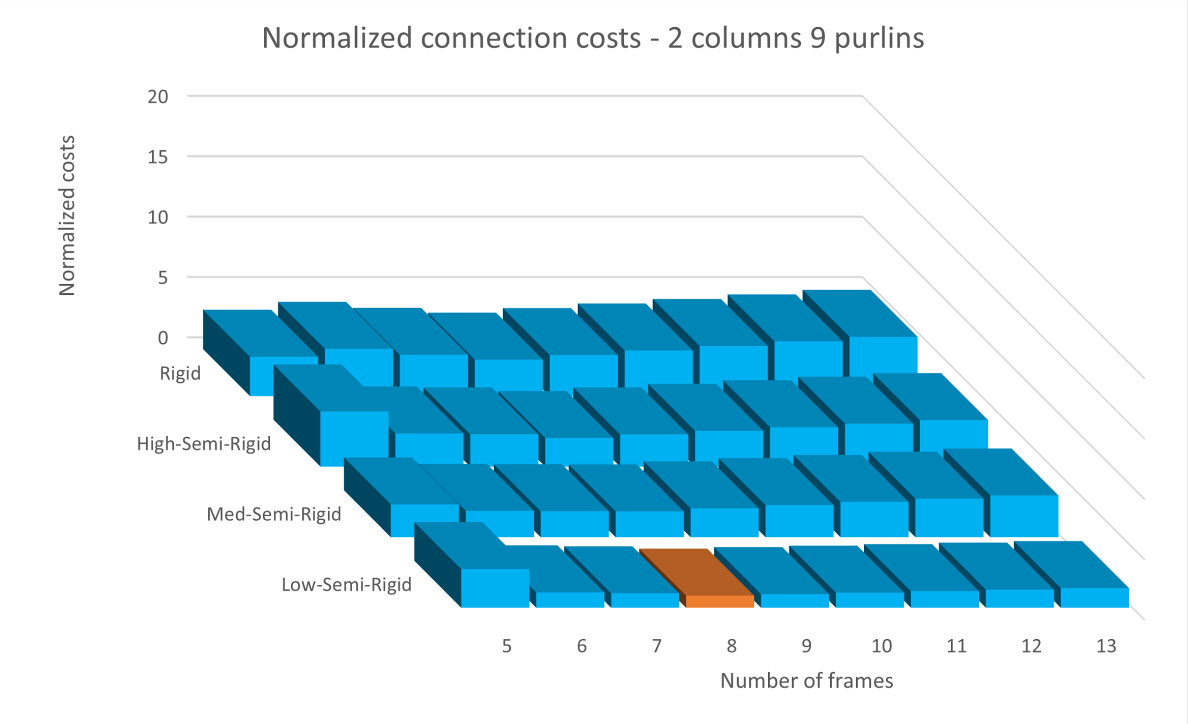


Figure I.8: Normalized connection costs for 9 purlins

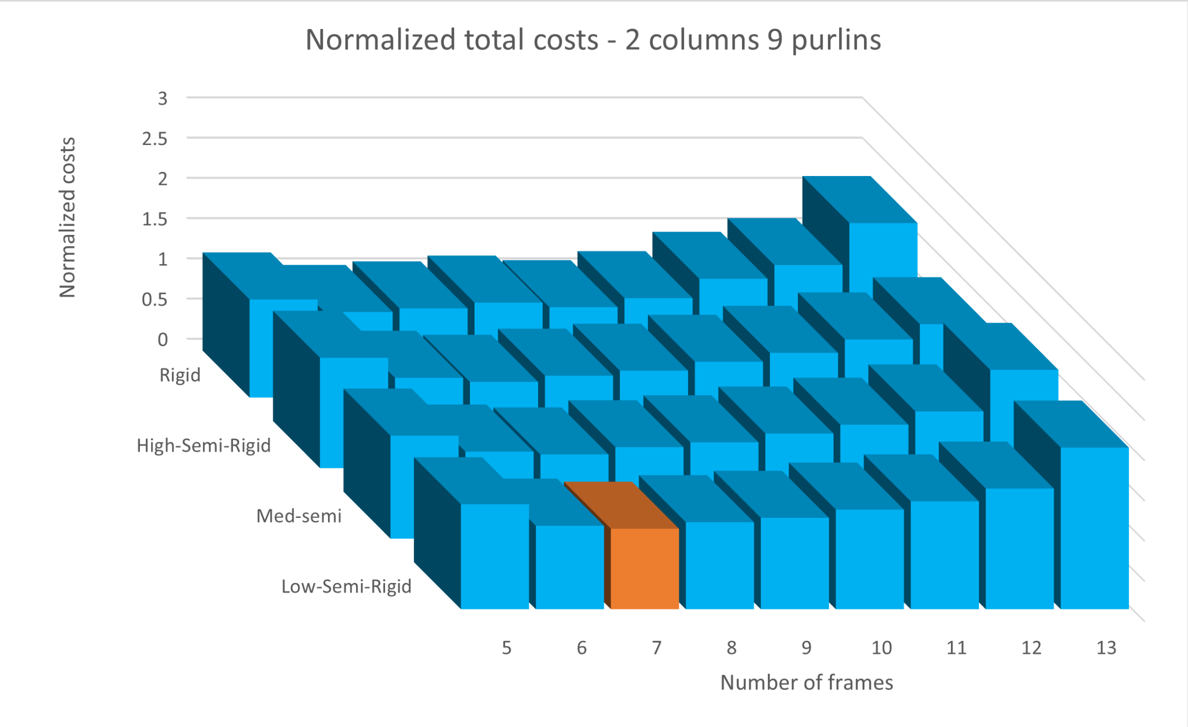


Figure I.9: Normalized total costs for 9 purlins

I.2. FIXED BEAM-COLUMN CONNECTION GRAPHS FOR HINGED COLUMN-BASE CONNECTION

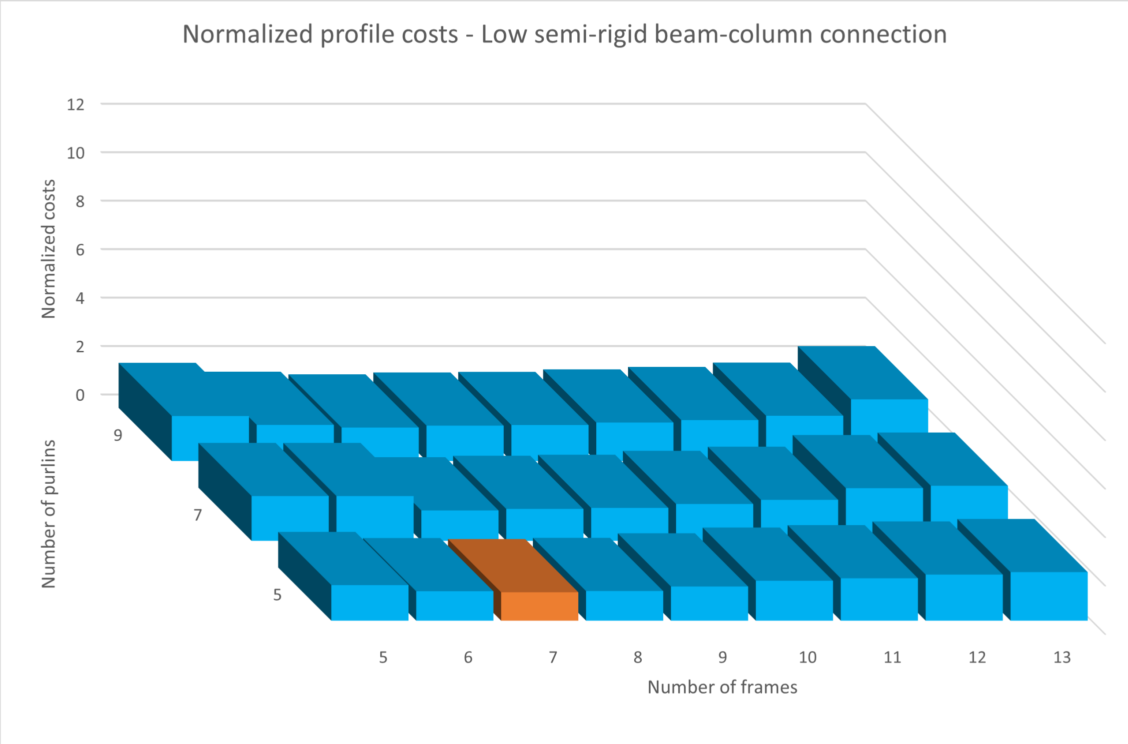


Figure I.10: Normalized profile costs for low semi-rigid beam-column connection

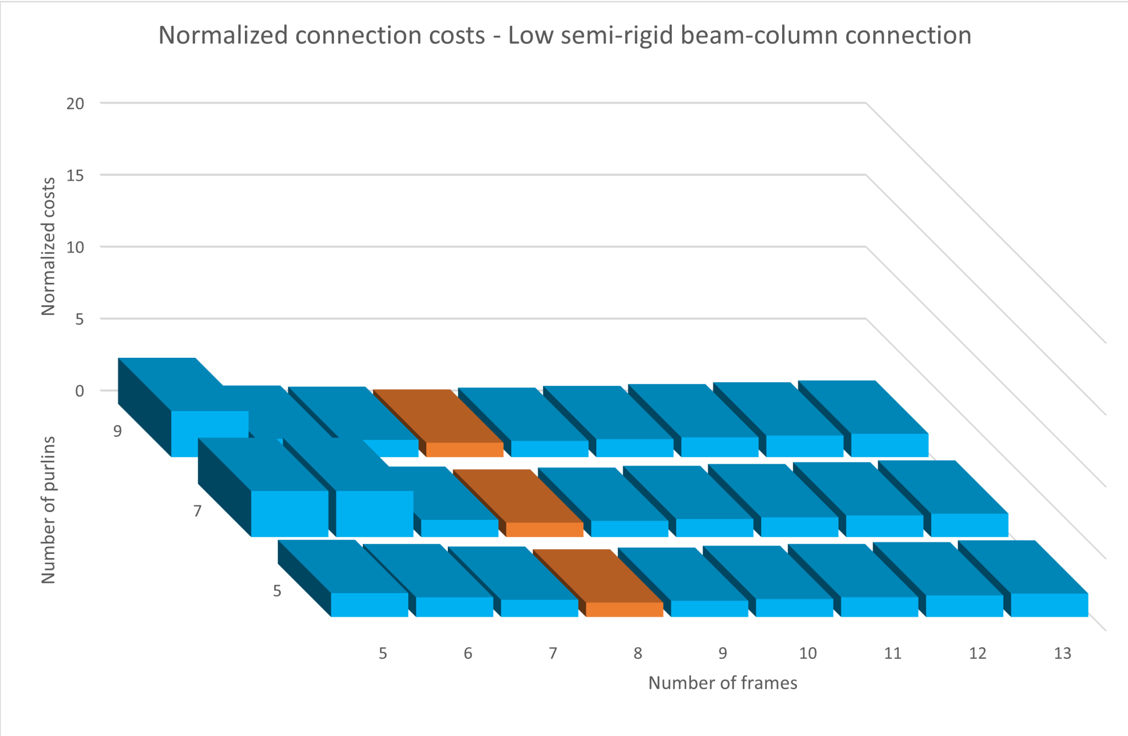


Figure I.11: Normalized connection costs for low semi-rigid beam-column connection

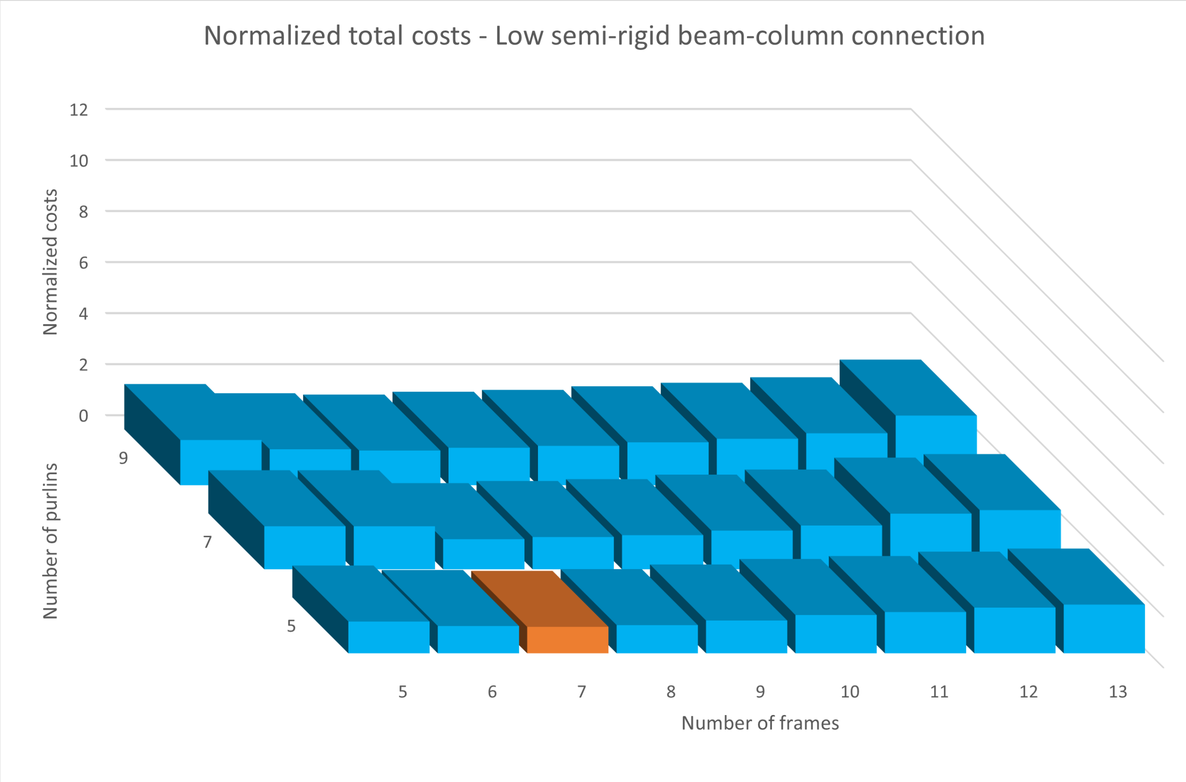


Figure I.12: Normalized total costs for low semi-rigid beam-column connection

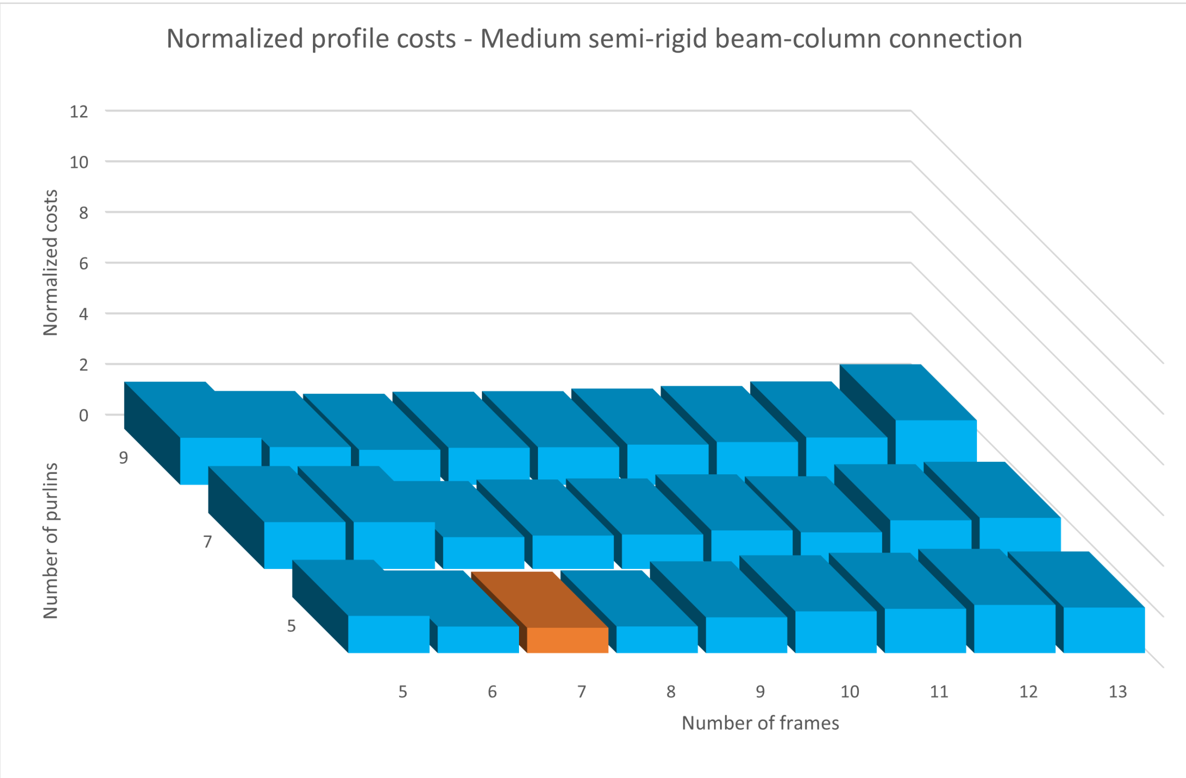


Figure I.13: Normalized profile costs for medium semi-rigid beam-column connection

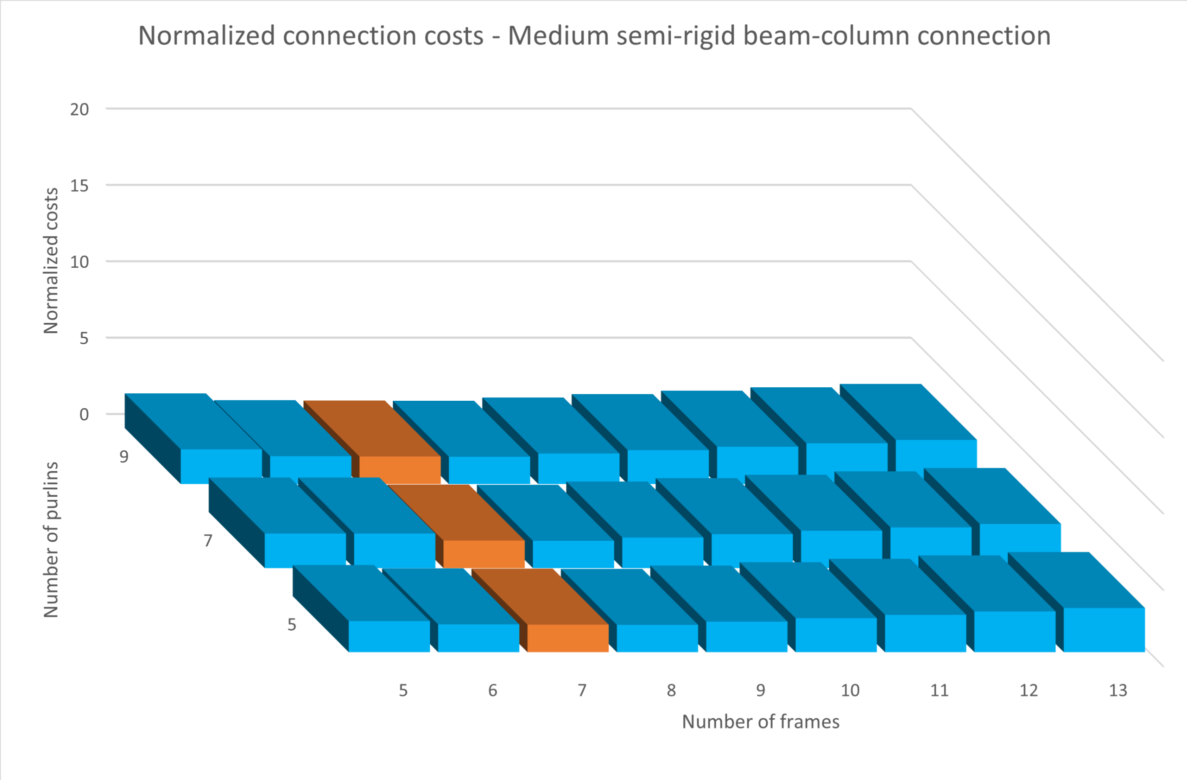


Figure I.14: Normalized connection costs for medium semi-rigid beam-column connection

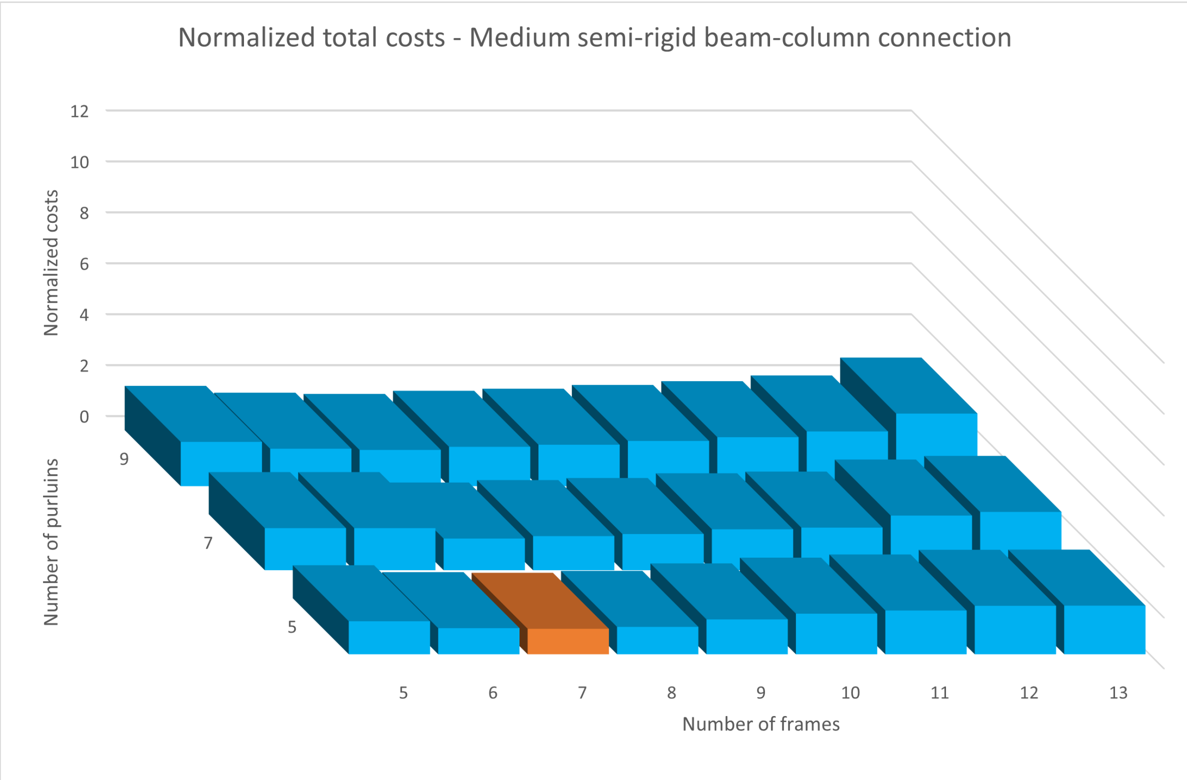


Figure I.15: Normalized total costs for medium semi-rigid beam-column connection

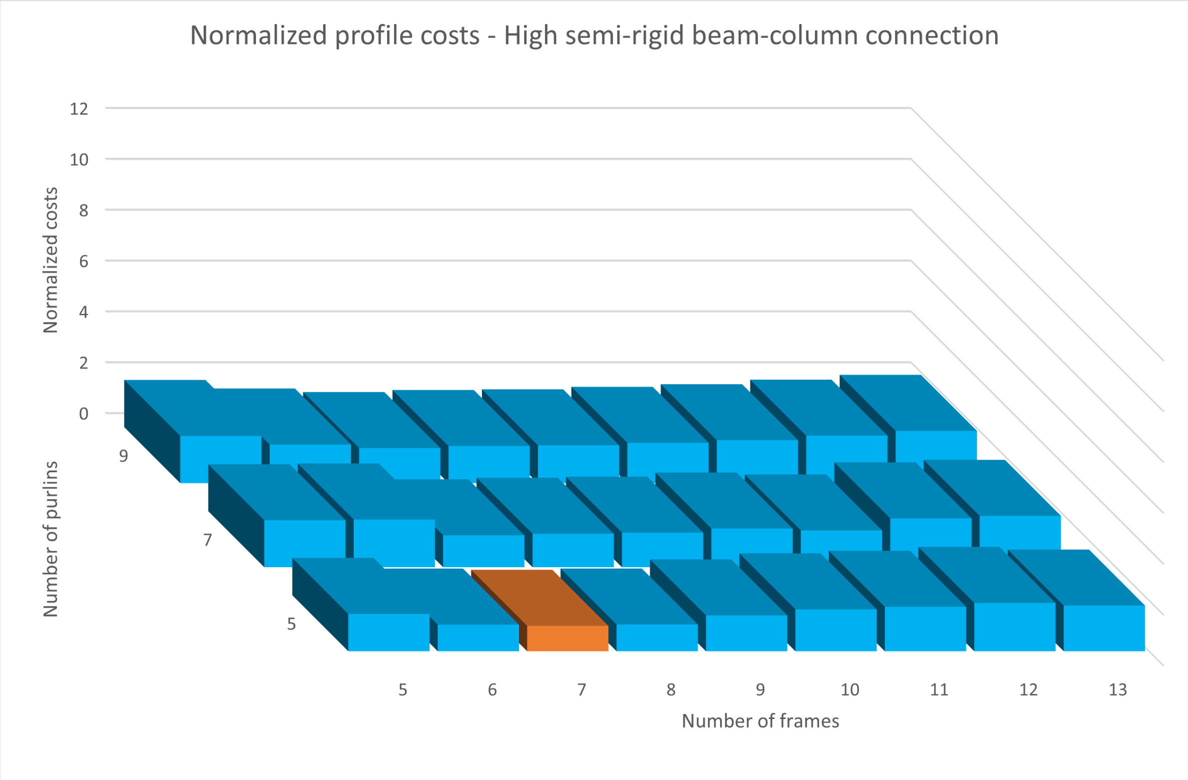


Figure I.16: Normalized profile costs for high semi-rigid beam-column connection

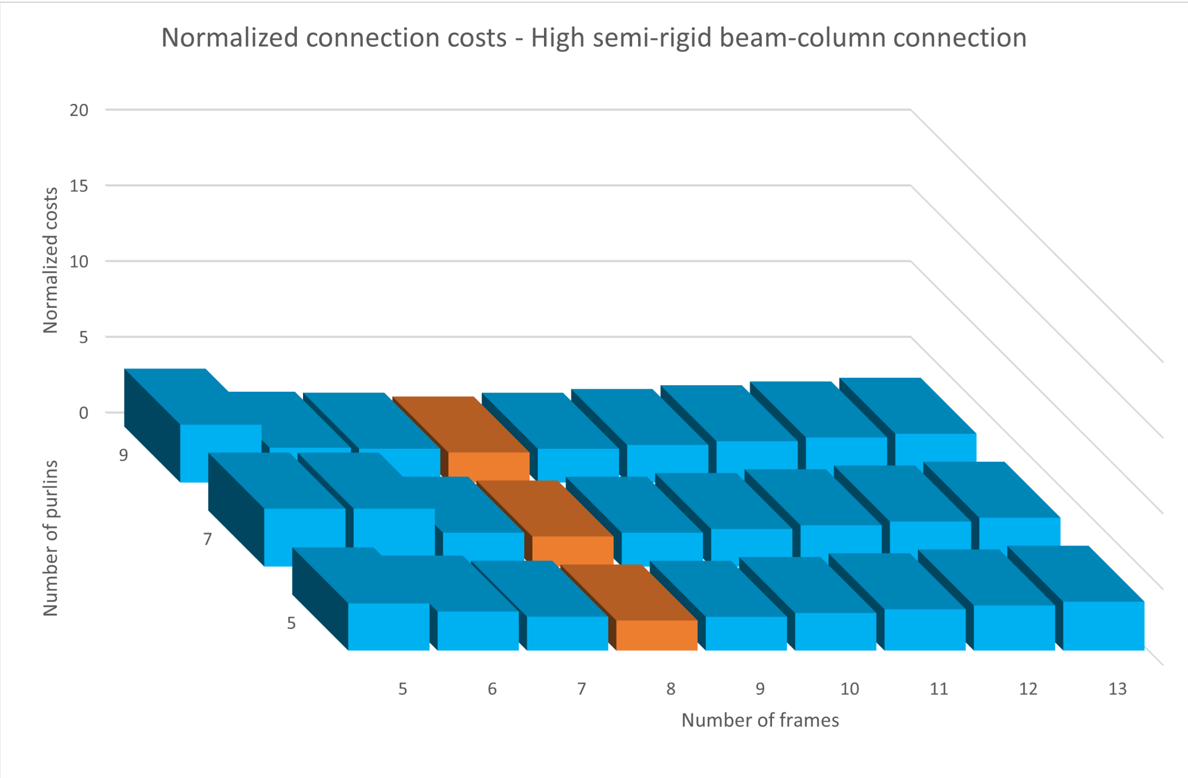


Figure I.17: Normalized connection costs for high semi-rigid beam-column connection

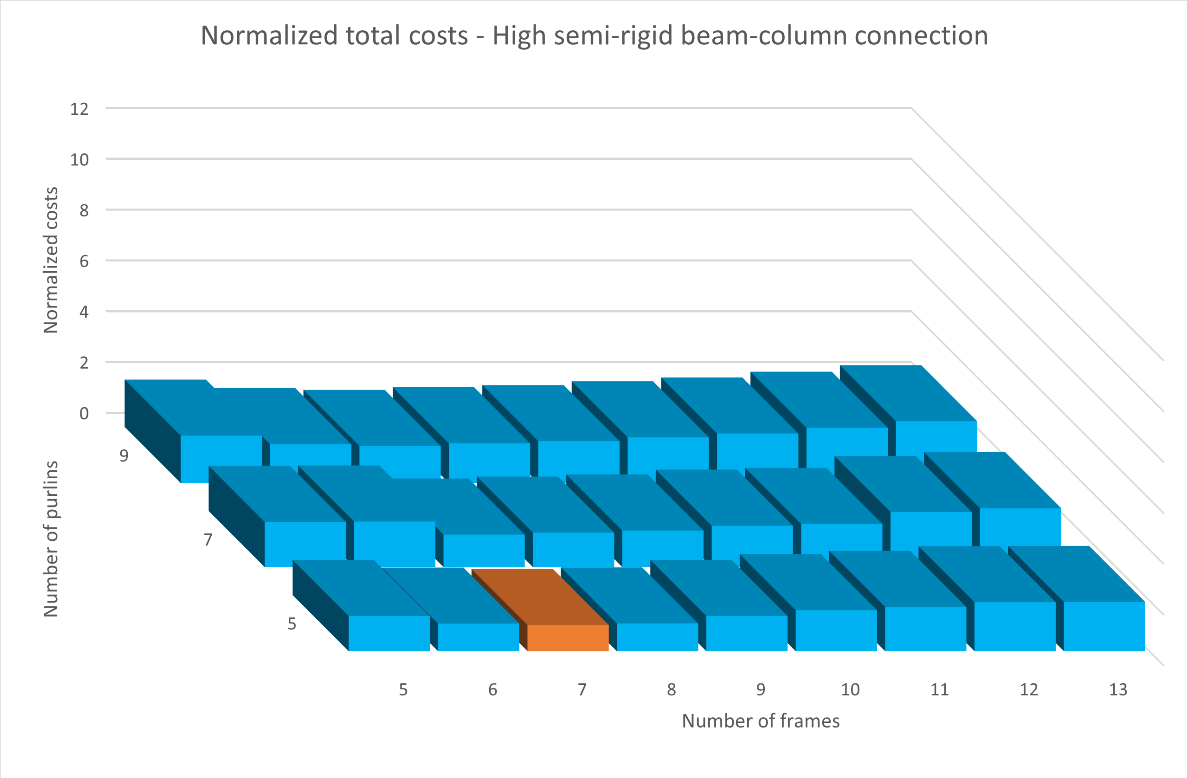


Figure I.18: Normalized total costs for high semi-rigid beam-column connection

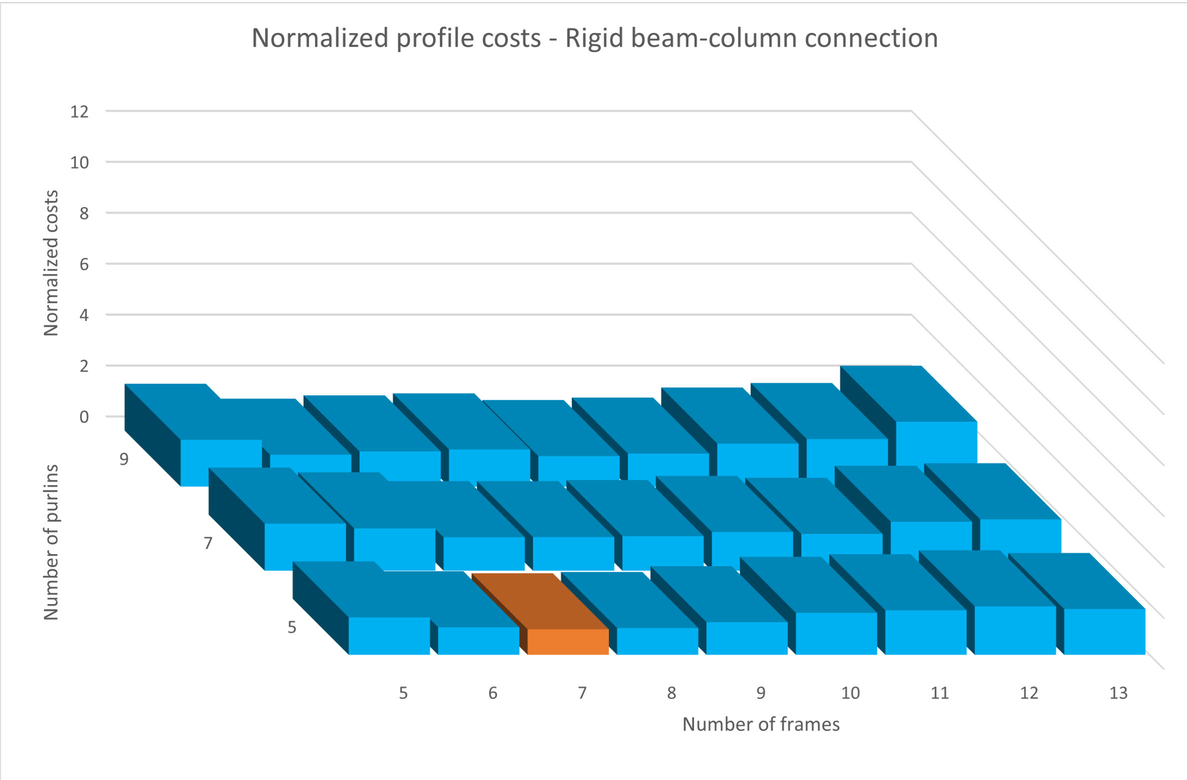


Figure I.19: Normalized profile costs for rigid beam-column connection

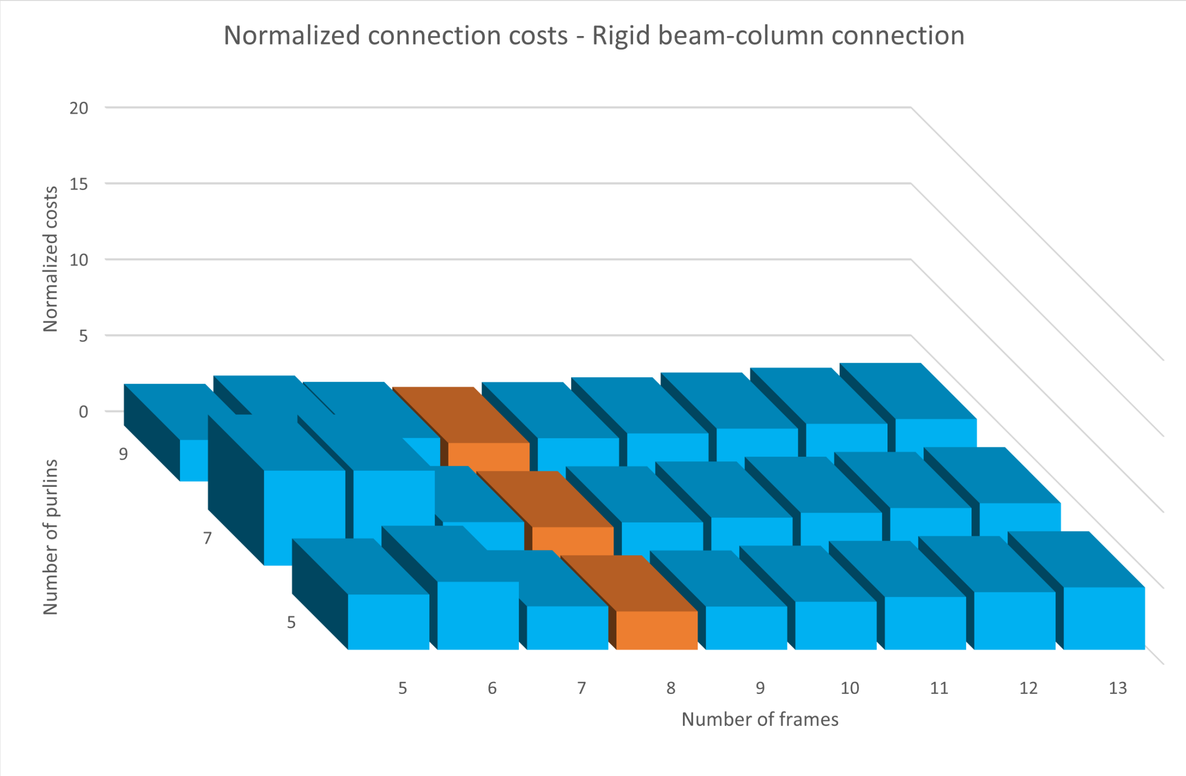


Figure I.20: Normalized connection costs for rigid beam-column connection

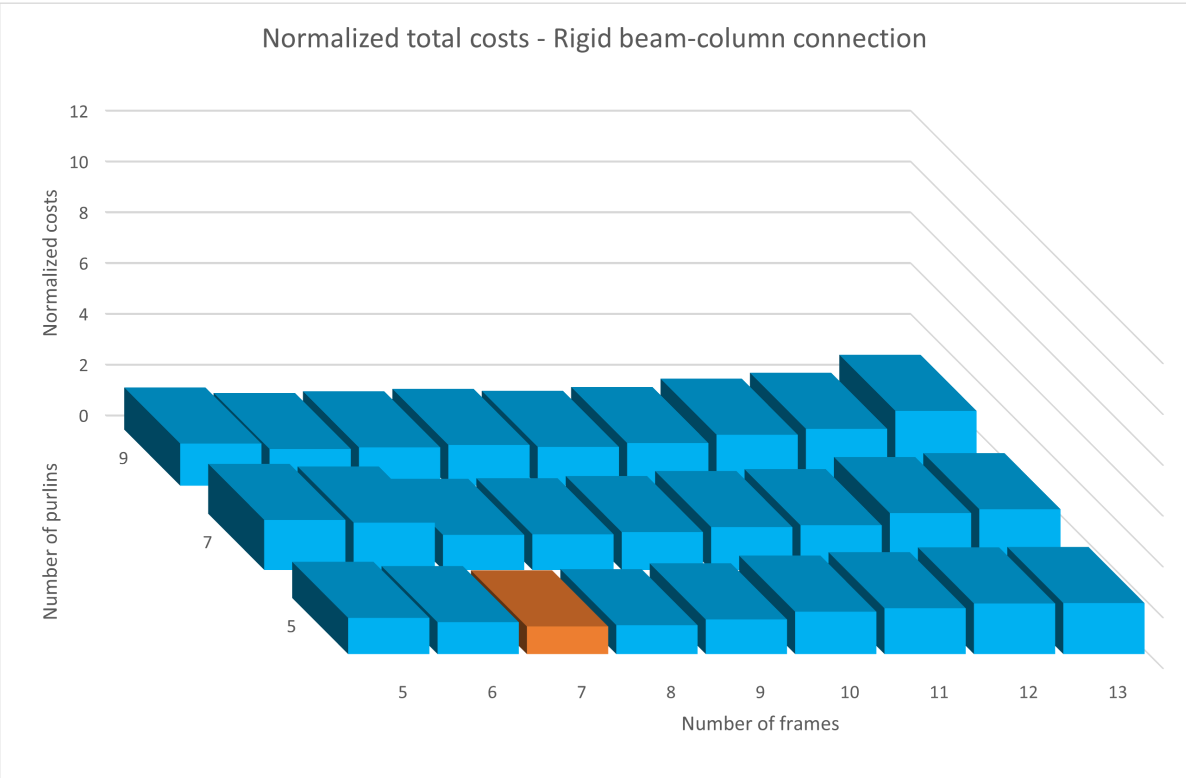


Figure I.21: Normalized total costs for rigid beam-column connection

I.3. FIXED NUMBER OF PURLINS GRAPHS FOR RIGID COLUMN-BASE CONNECTION

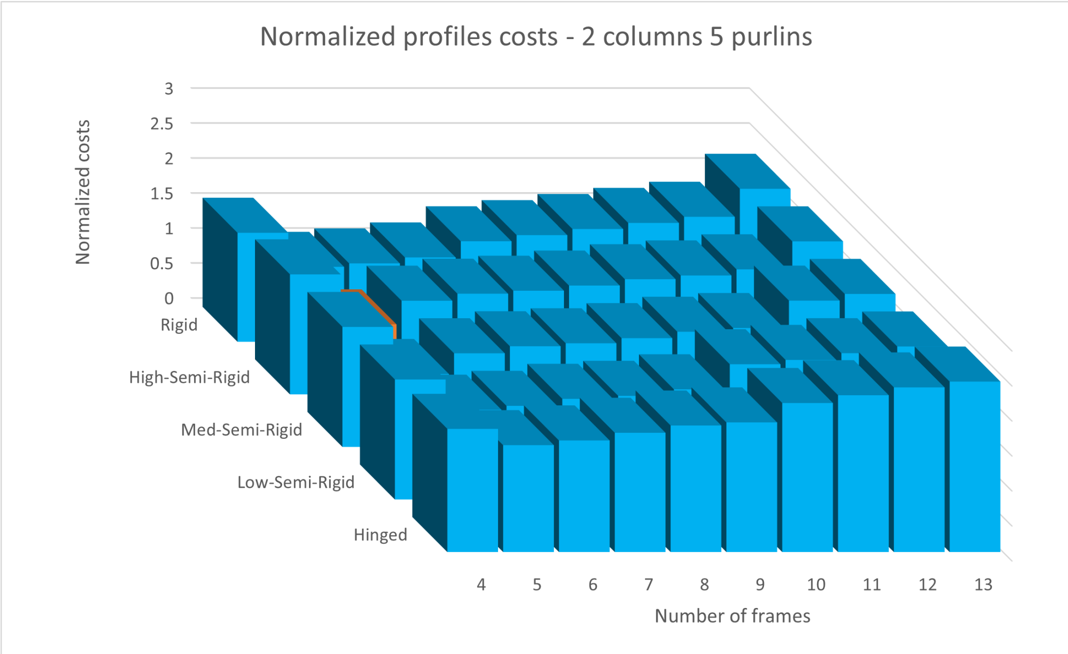


Figure I.22: Normalized profile costs for 5 purlins

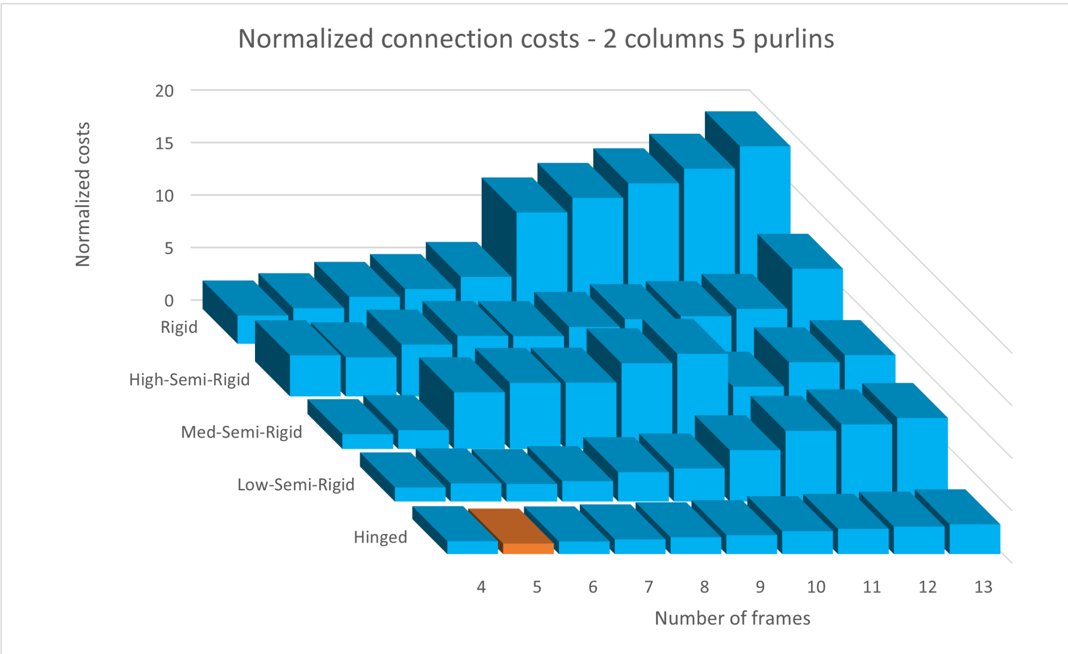


Figure I.23: Normalized connection costs for 5 purlins

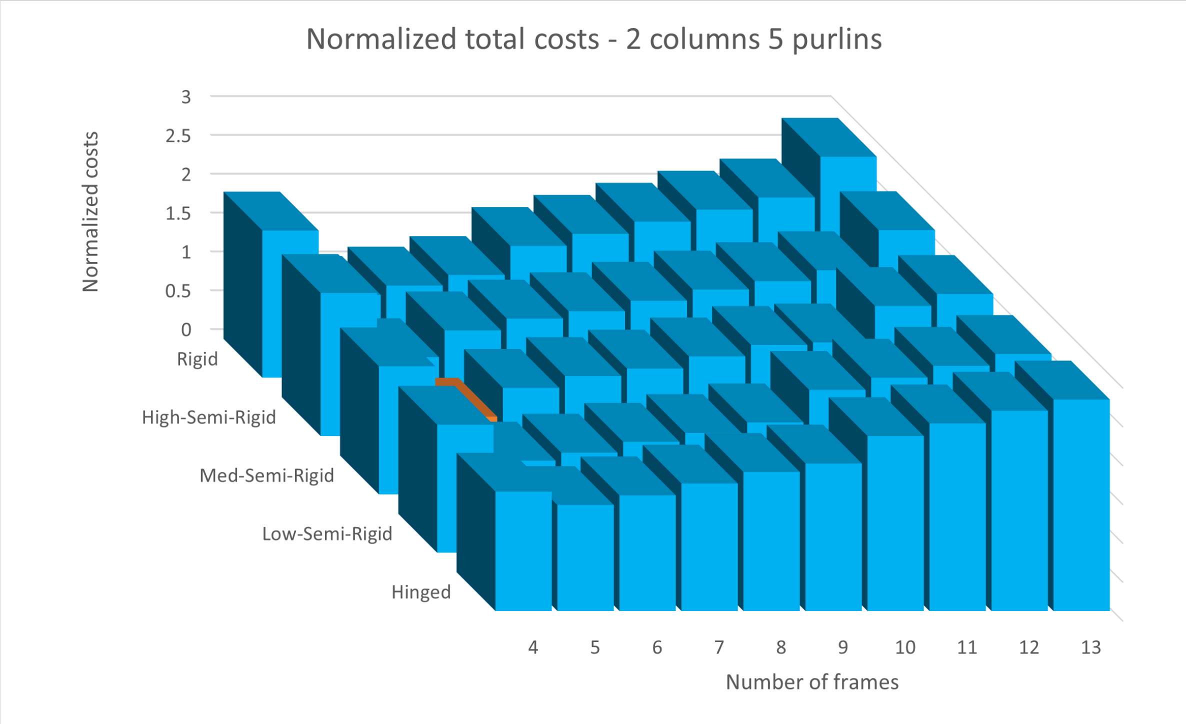


Figure I.24: Normalized total costs for 5 purlins

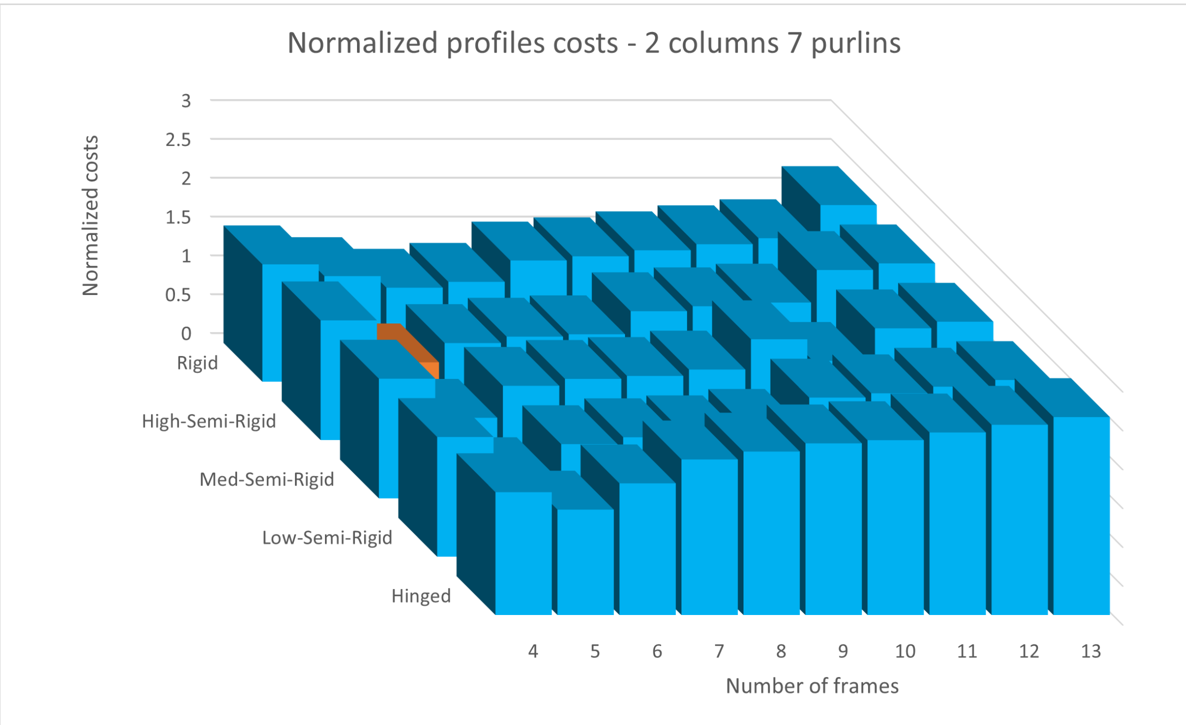


Figure I.25: Normalized profile costs for 7 purlins

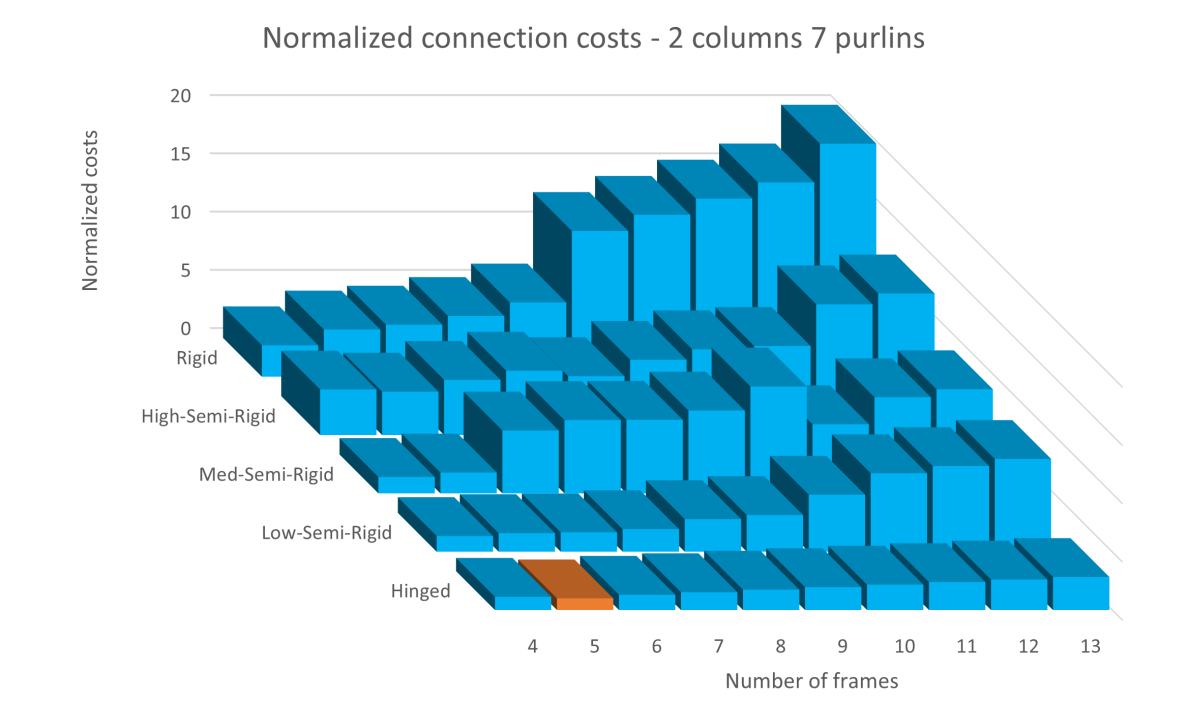


Figure I.26: Normalized connection costs for 7 purlins

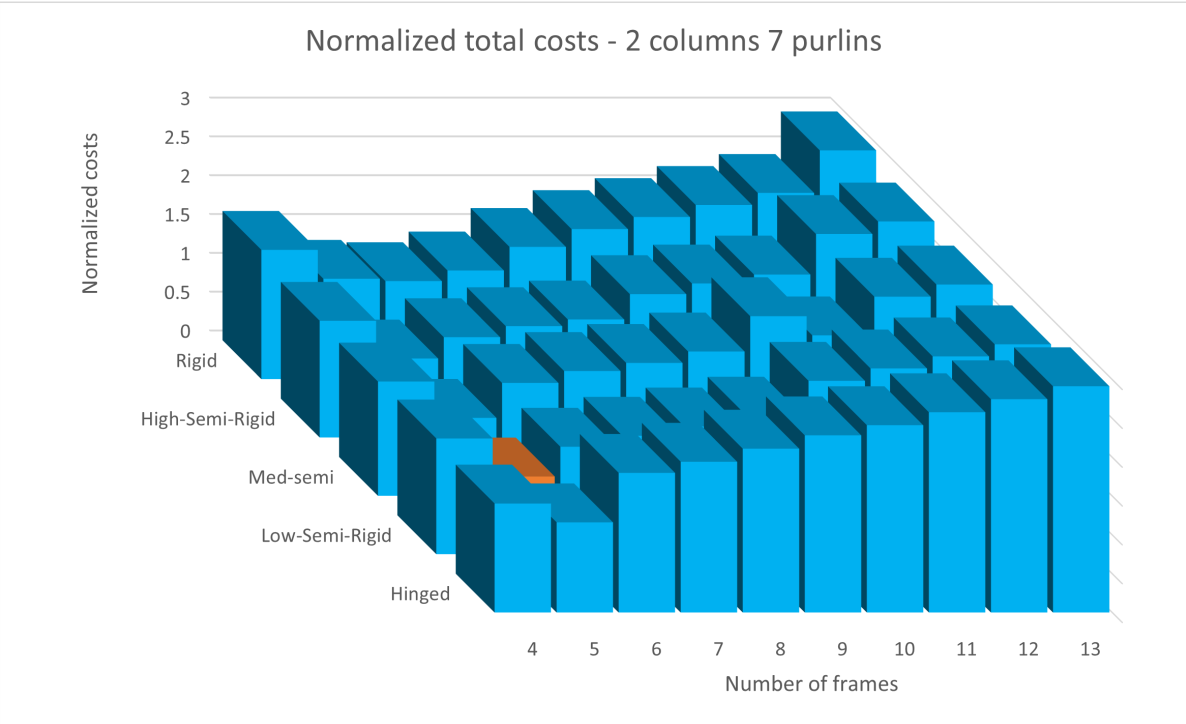


Figure I.27: Normalized total costs for 7 purlins

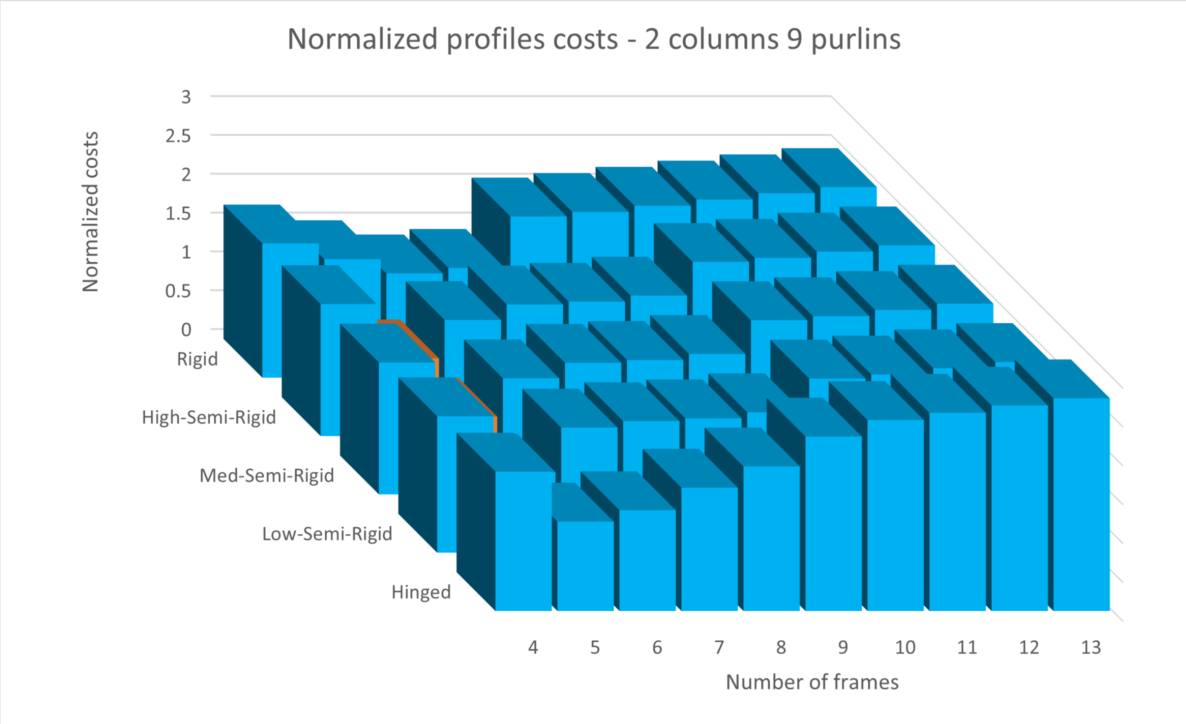


Figure I.28: Normalized profile costs for 9 purlins

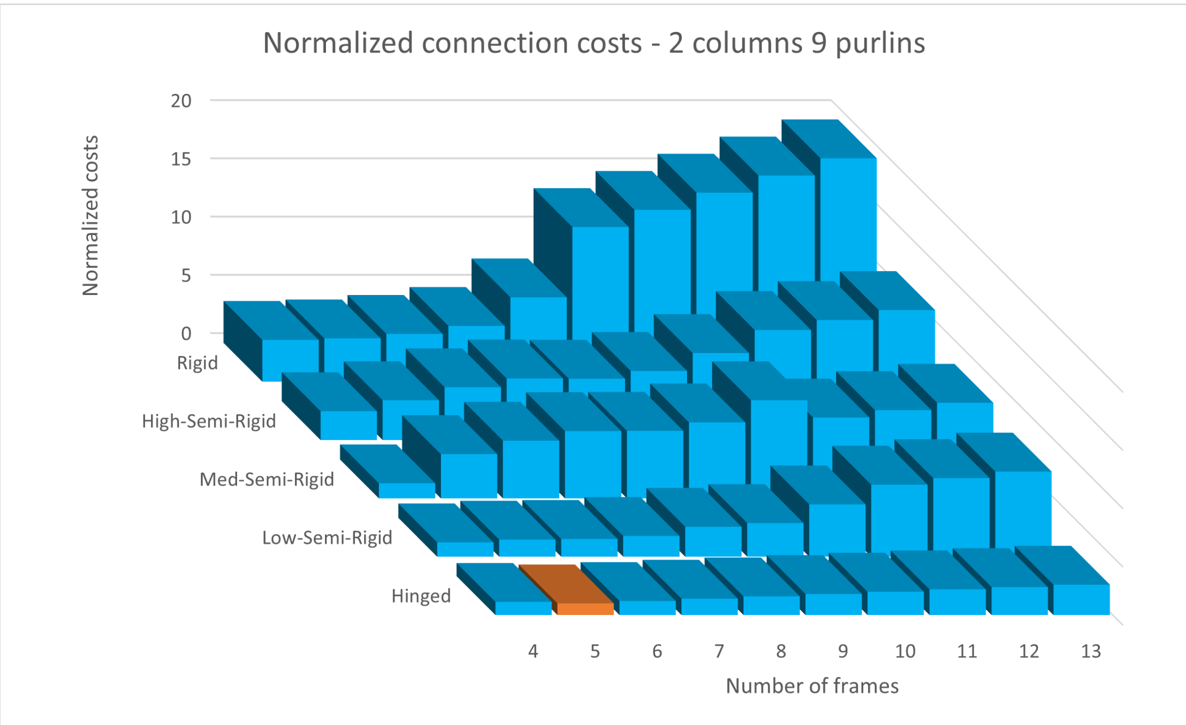


Figure I.29: Normalized connection costs for 9 purlins

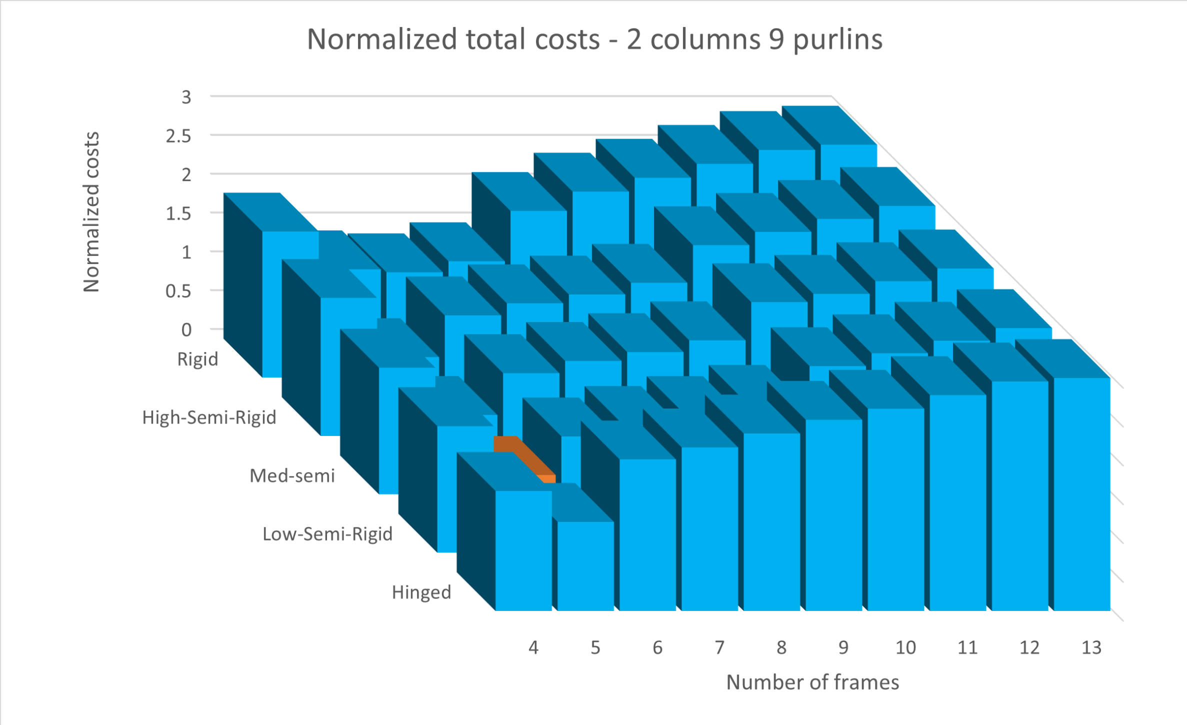


Figure I.30: Normalized total costs for 9 purlins

I.4. FIXED BEAM-COLUMN CONNECTION GRAPHS FOR RIGID COLUMN-BASE CONNECTION

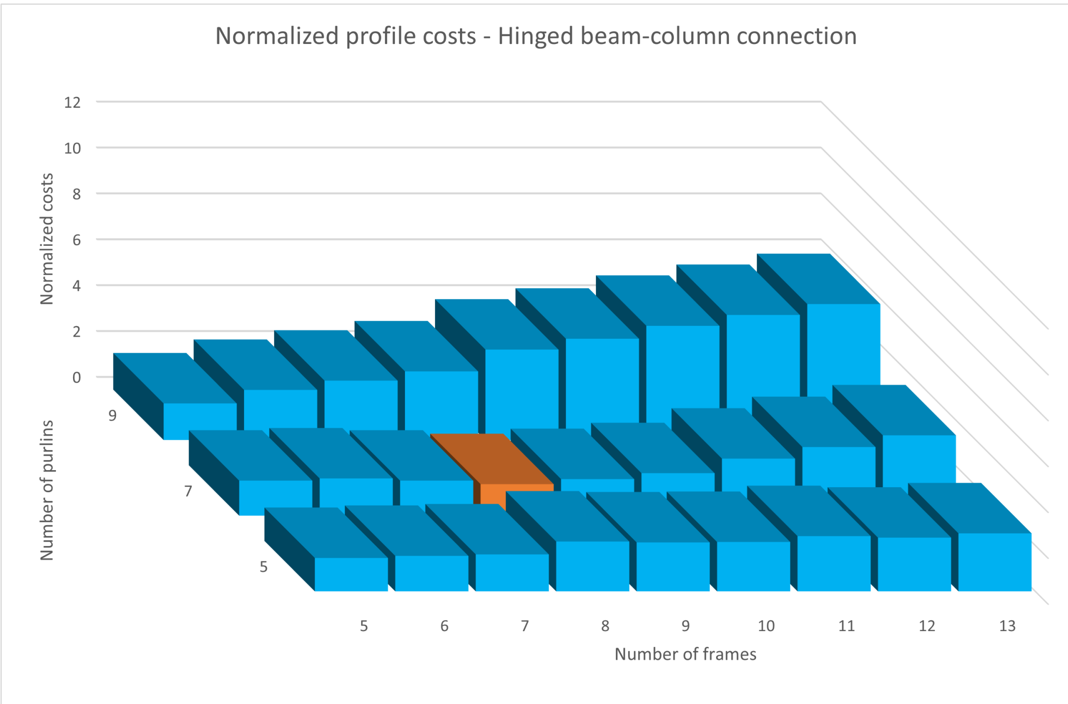


Figure I.31: Normalized profile costs for hinged beam-column connection

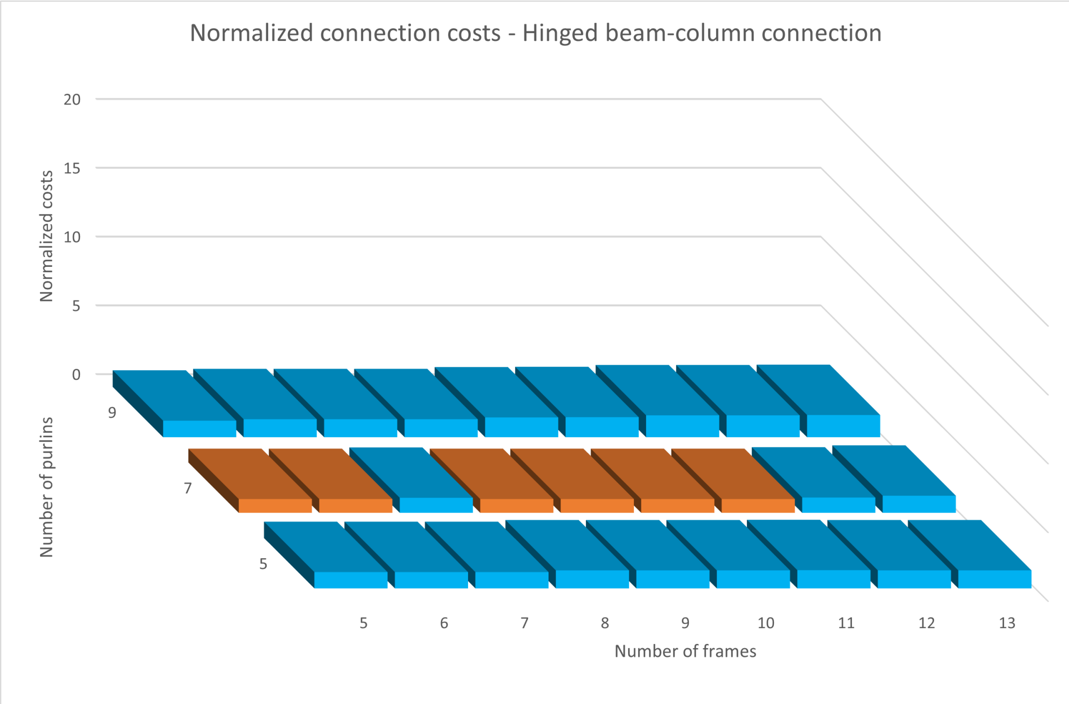


Figure I.32: Normalized connection costs for hinged beam-column connection

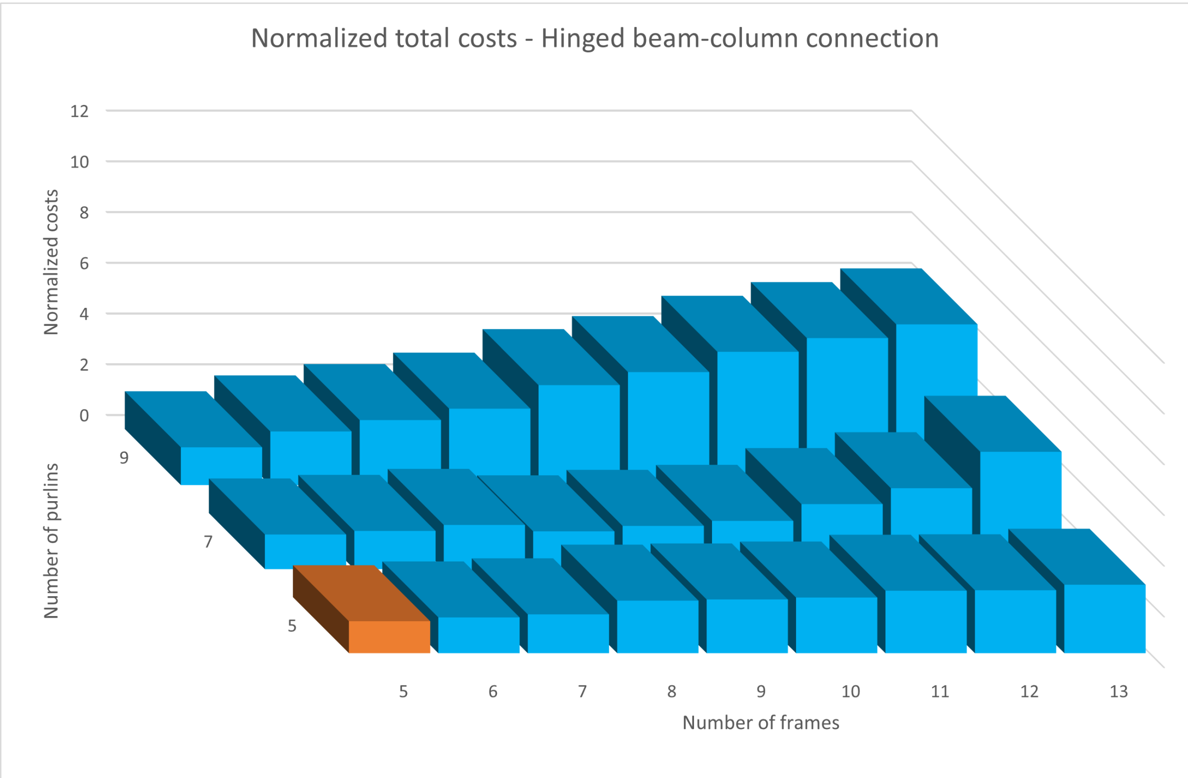


Figure I.33: Normalized total costs for hinged beam-column connection

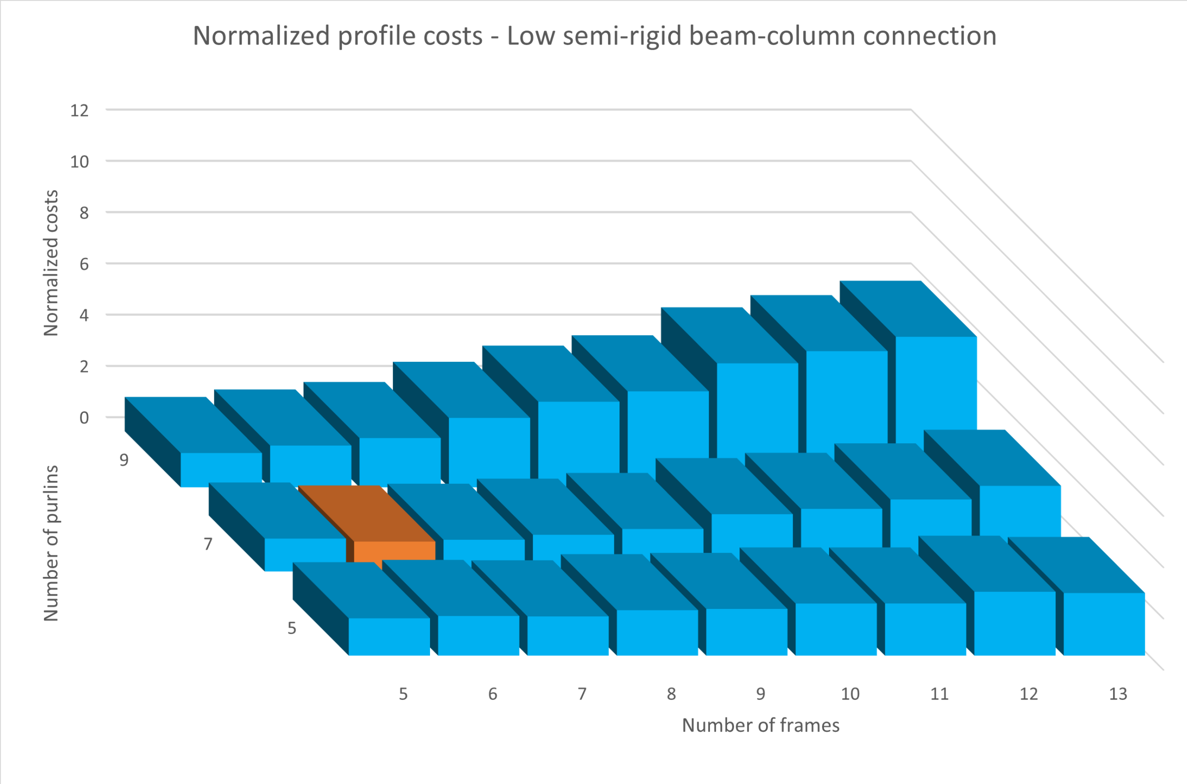


Figure I.34: Normalized profile costs for low semi-rigid beam-column connection

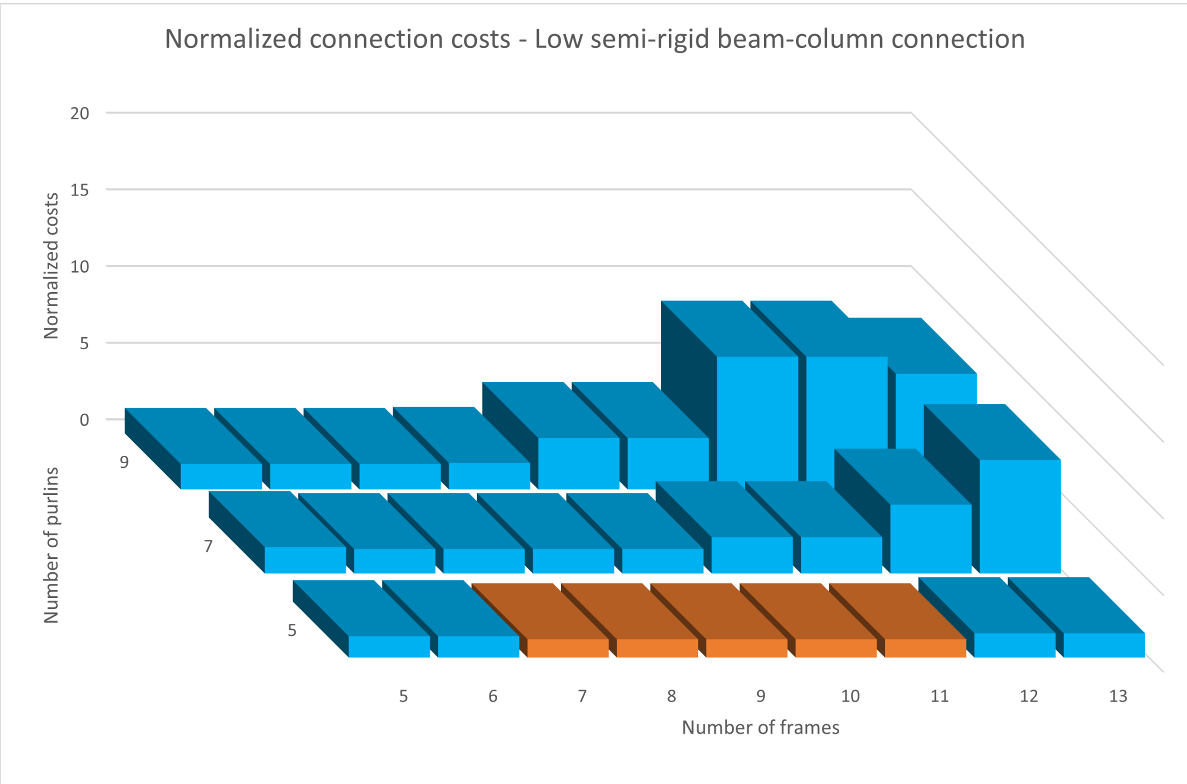


Figure I.35: Normalized connection costs for low semi-rigid beam-column connection

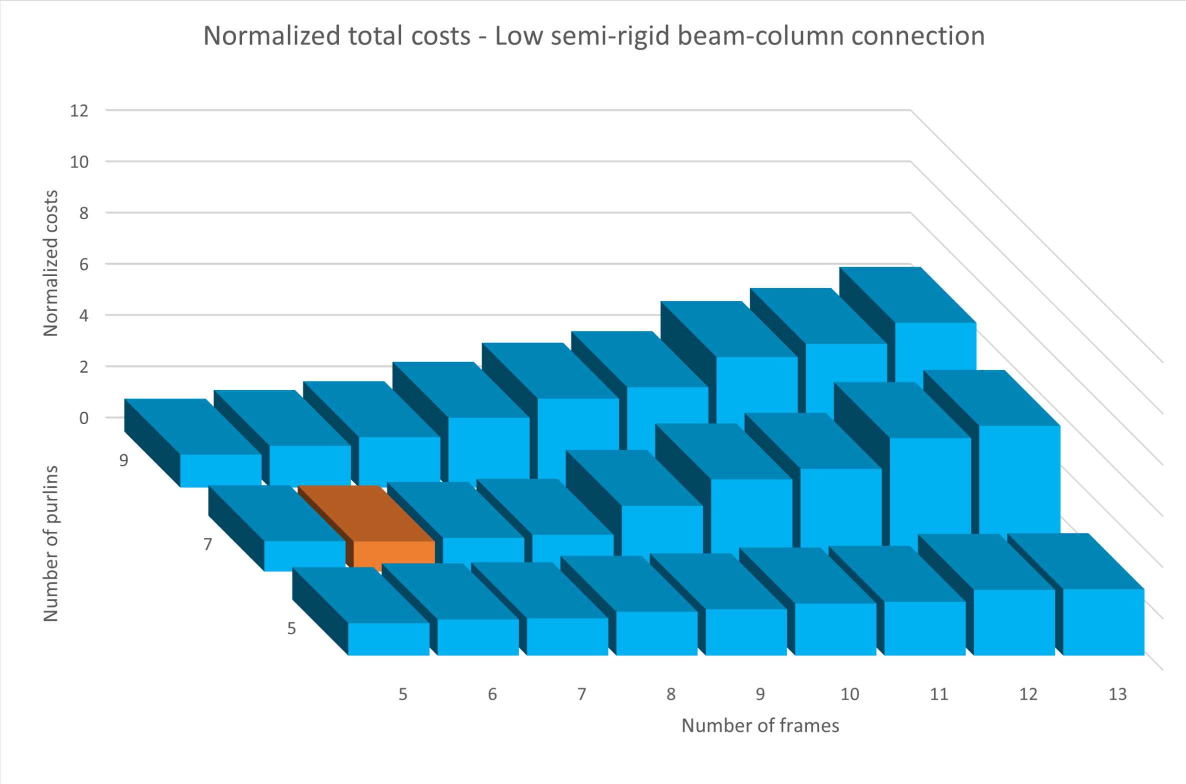


Figure I.36: Normalized total costs for low semi-rigid beam-column connection

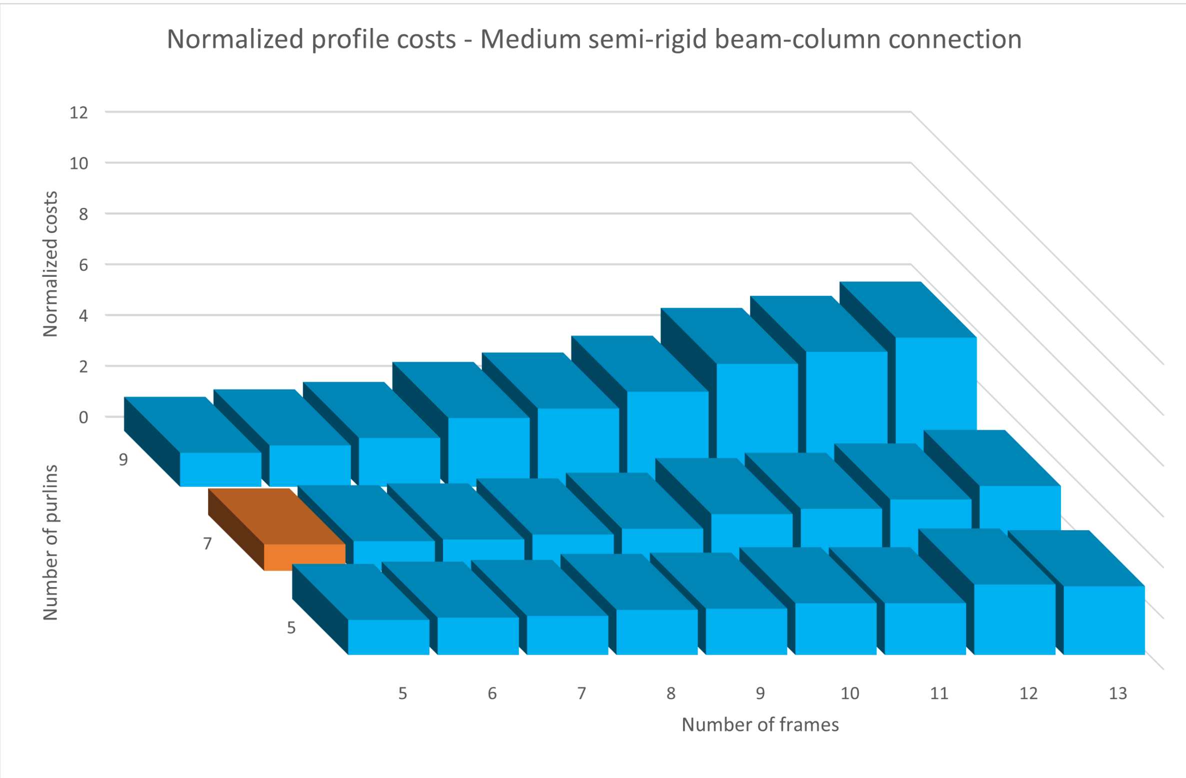


Figure I.37: Normalized profile costs for medium semi-rigid beam-column connection

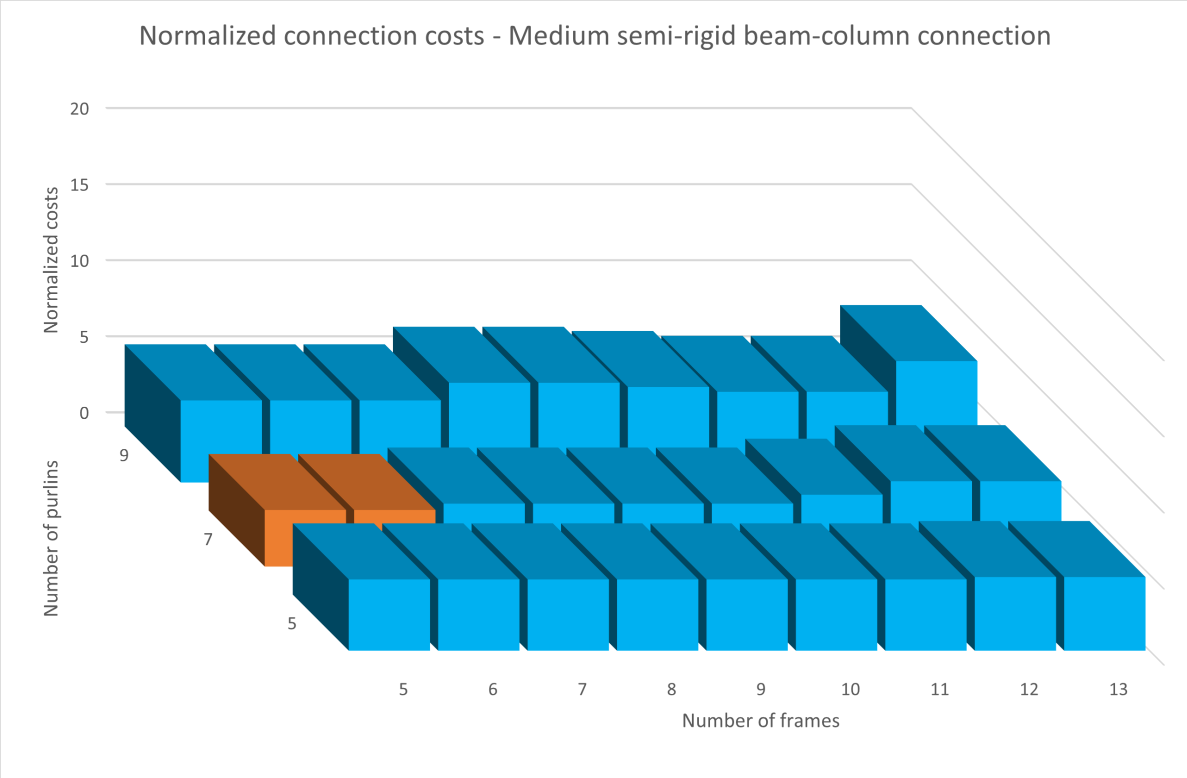


Figure I.38: Normalized connection costs for medium semi-rigid beam-column connection

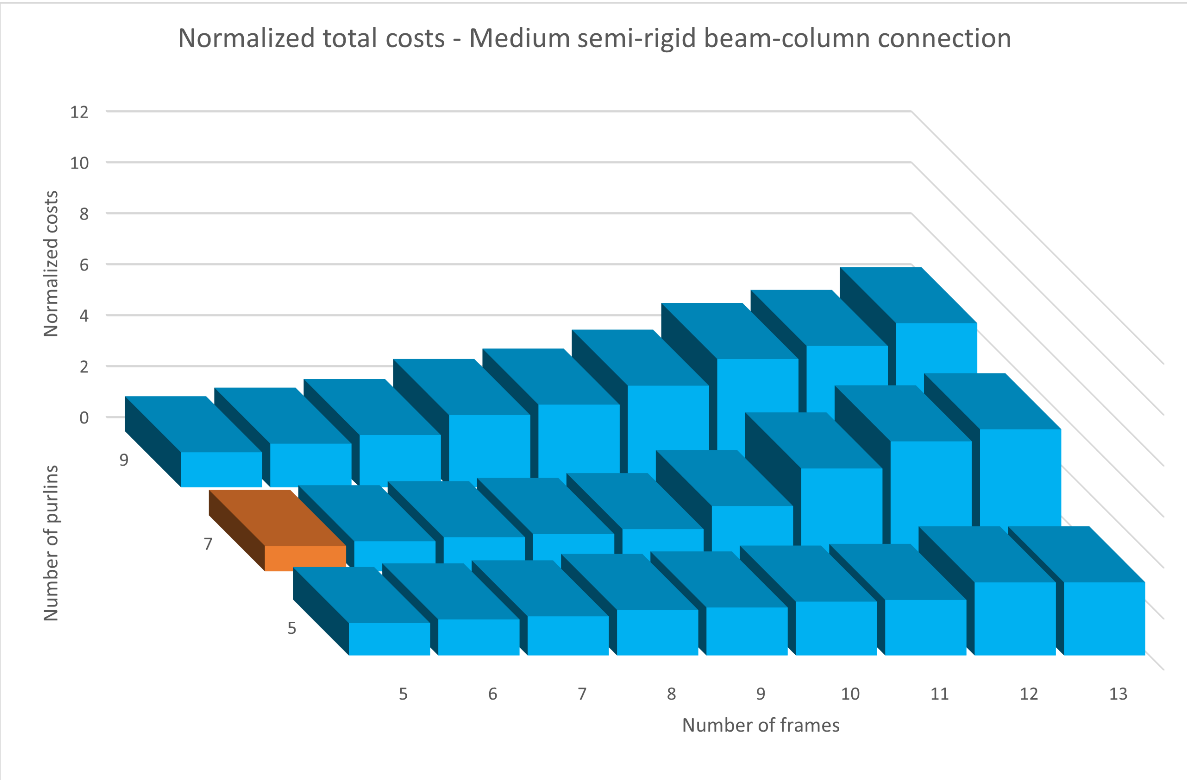


Figure I.39: Normalized total costs for medium semi-rigid beam-column connection

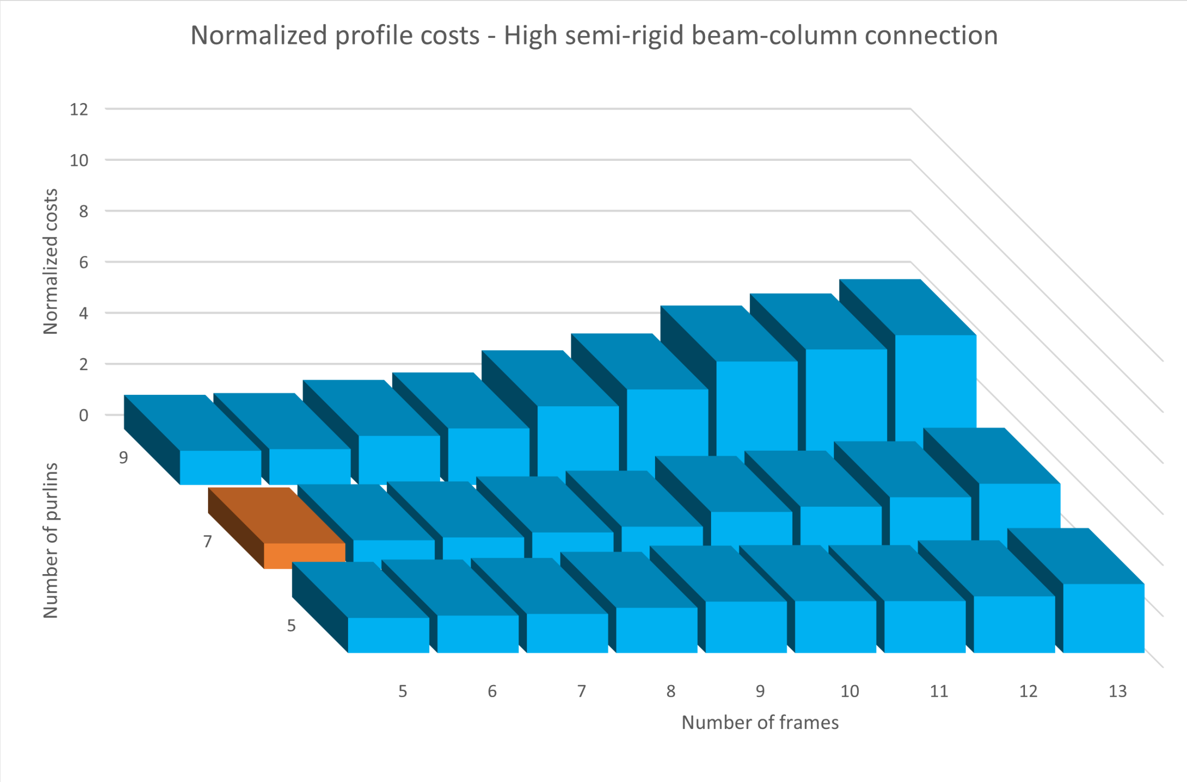


Figure I.40: Normalized profile costs for high semi-rigid beam-column connection

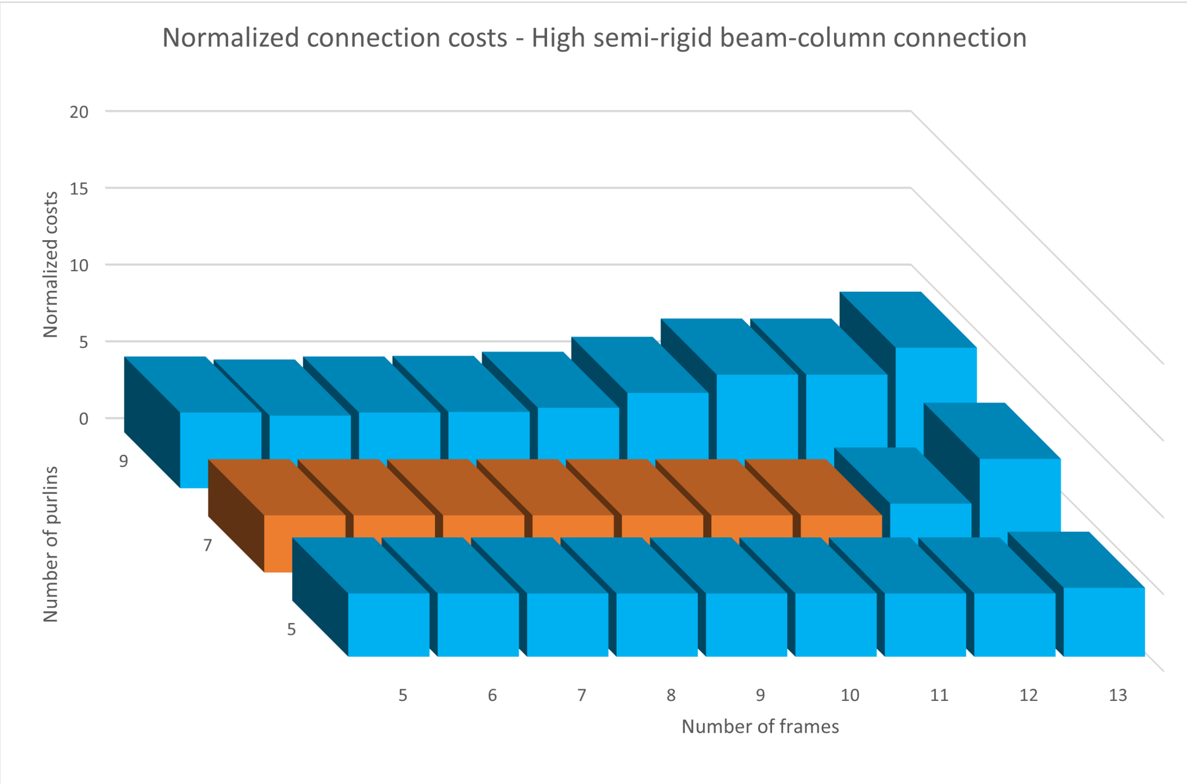


Figure I.41: Normalized connection costs for high semi-rigid beam-column connection

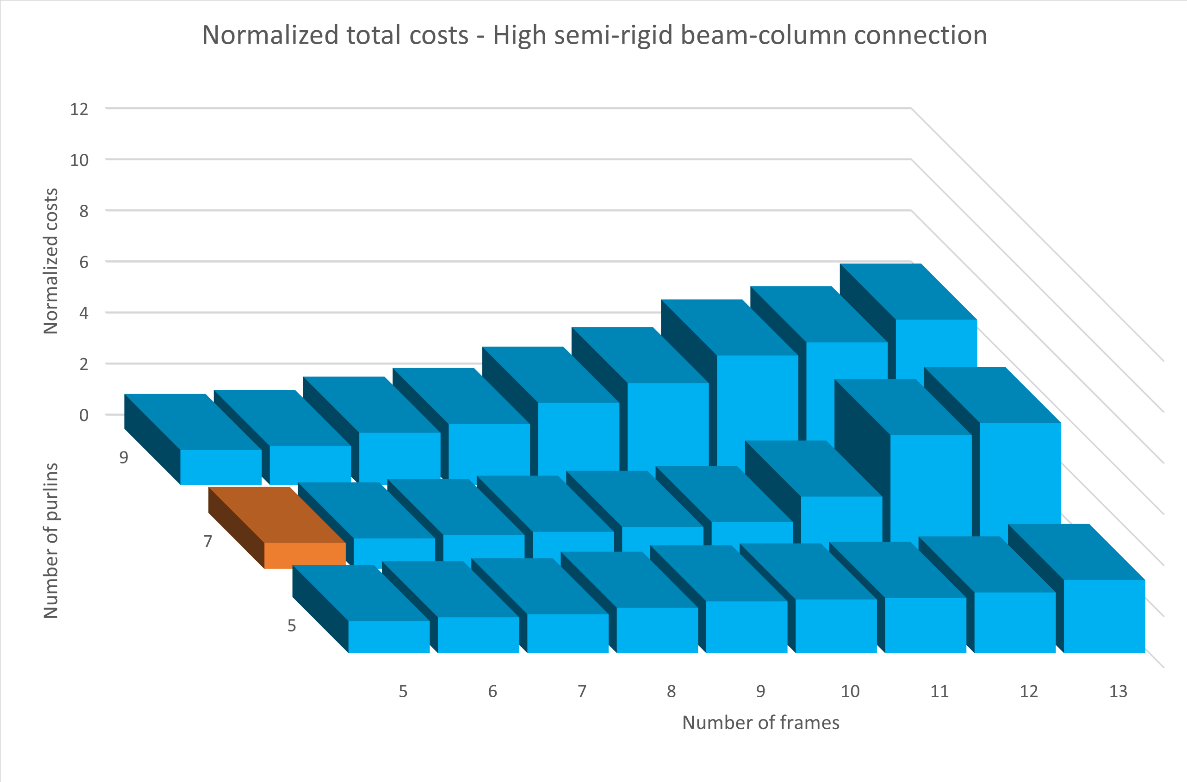


Figure I.42: Normalized total costs for high semi-rigid beam-column connection

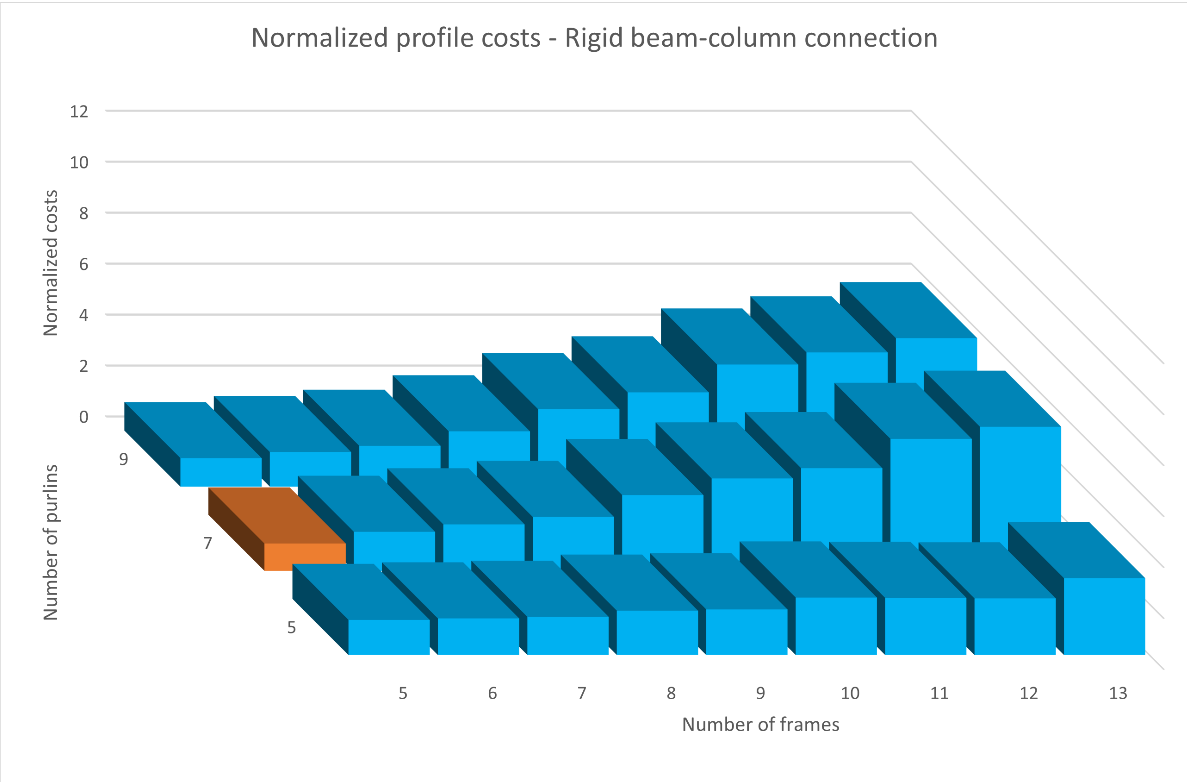


Figure I.43: Normalized profile costs for rigid beam-column connection

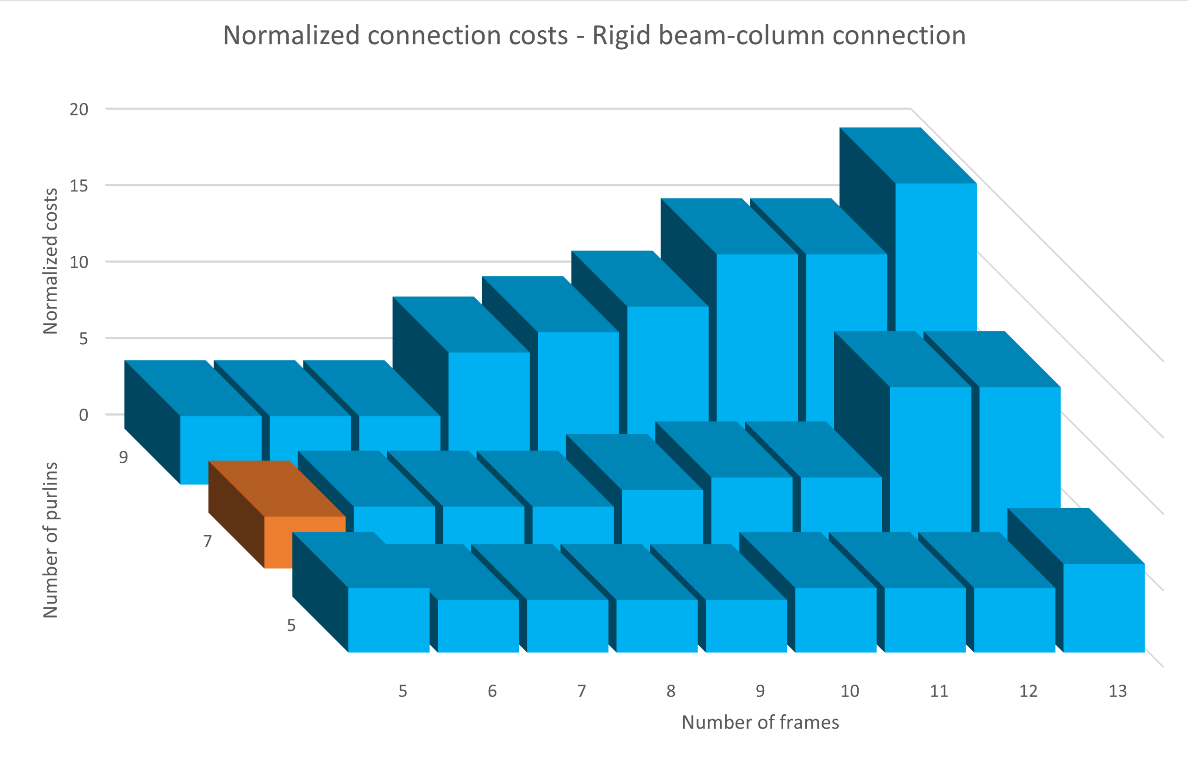


Figure I.44: Normalized connection costs for rigid beam-column connection

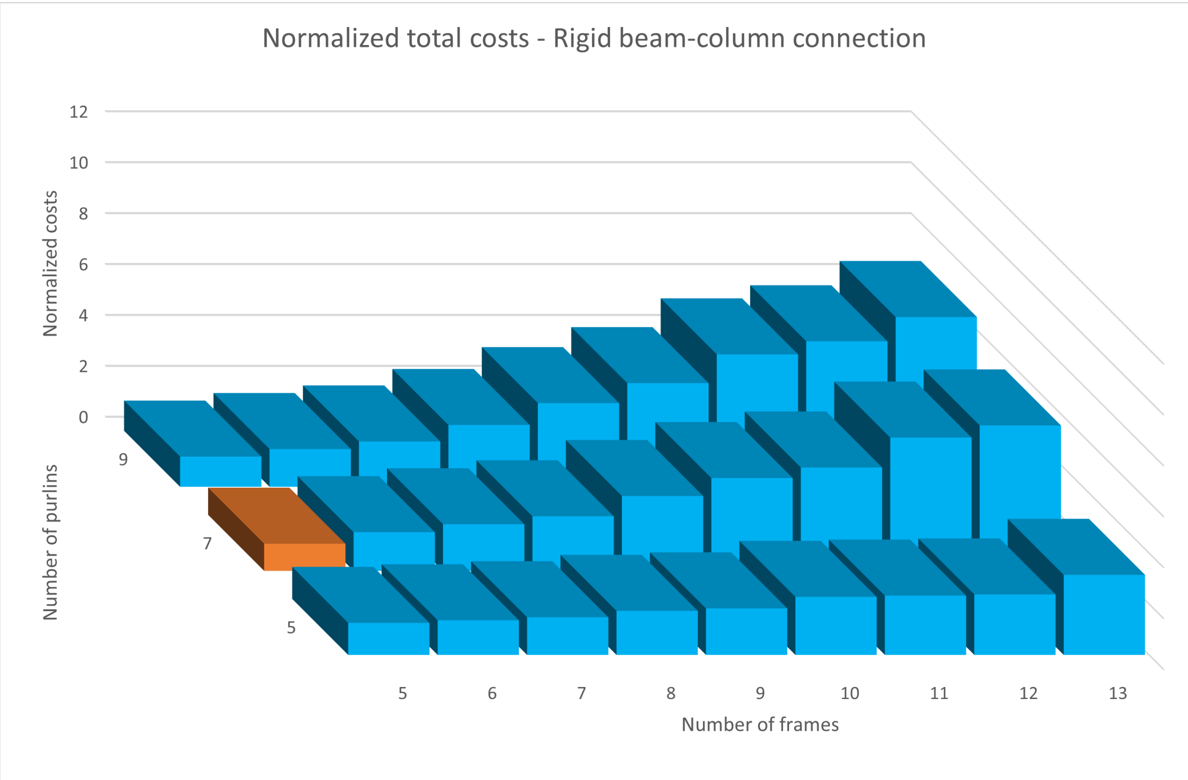


Figure I.45: Normalized total costs for rigid beam-column connection

Bibliography

[1] R. Oldham, *Cost planning through design stages,* Steel Insight **02** (2012).

[2] T. Wiliams, *System integration is a critical element in a project design,* Control Engineering (2013).

[3] cbfem, *About cbfem,* (2020).

[4] M. Steenhuis, K. Weynand, and A. Gresnigt, *Strategies for economic design of unbraced steel frames,* Bouwen met Staal (1998).

[5] International-Transport-Forum, *dimensions-road-transport-europe,* (2020).

[6] ArcelorMittal, *Prijscourant,* (2018).

[7] Bressers, *Prijslijst balken,* (2018).

[8] Reesink, *Prijslijst,* (2018).

[9] Vlietjonge, *Prijslijst,* (2018).

[10] LimTrade, *Hea-balken,* (2020).

[11] LimTrade, *Heb-balken,* (2020).

[12] LimTrade, *Ipe-balken,* (2020).

[13] LimTrade, *Upe-balken,* (2020).

[14] Bressers, *Prijslijst platen,* (2018).

[15] LimTrade, *Wgw-platen,* (2020).

[16] JRV, *Prijslijsten iso-4017-8.8,* (2020).

[17] Almetal, *Prijslijsten iso-4017-8.8,* (2020).

[18] Eriks, *Prijslijsten iso-4017-8.8,* (2020).

[19] Fabory, *Prijslijsten iso-4017-8.8,* (2020).

[20] M. en Bout, *Prijslijsten iso-4017-8.8,* (2020).

[21] Almetal, *Prijslijsten iso-4017-10.9,* (2020).

[22] Eriks, *Prijslijsten iso-4017-10.9,* (2020).

[23] Fabory, *Prijslijsten iso-4017-10.9,* (2020).

[24] M. en Bout, *Prijslijsten iso-4017-10.9,* (2020).

[25] INDI, *Prijslijsten iso-4017-10.9,* (2020).

[26] JRV, *Prijslijsten iso-4032-8.8,* (2020).

[27] Eriks, *Prijslijsten iso-4032-8.8,* (2020).

[28] Fabory, *Prijslijsten iso-4032-8.8,* (2020).

[29] M. en Bout, *Prijslijsten iso-4032-8.8,* (2020).

[30] INDI, *Prijslijsten iso-4032-8.8,* (2020).

[31] Eriks, *Prijslijsten iso-4032-10.9,* (2020).

[32] Fabory, *Prijslijsten iso-4032-10.9,* (2020).

[33] M. en Bout, *Prijslijsten iso-4032-10.9,* (2020).

[34] INDI, *Prijslijsten iso-4032-10.9,* (2020).

[35] JRV, *Prijslijsten iso-7089-a4,* (2020).

[36] Eriks, *Prijslijsten iso-7089-a4,* (2020).

[37] Fabory, *Prijslijsten iso-7089-a4,* (2020).

[38] M. en Bout, *Prijslijsten iso-7089-a4,* (2020).

[39] INDI, *Prijslijsten iso-7089-a4,* (2020).

[40] *Benchmark cases for advanced design of structural steel connections* (Frantisek Wald et al., 2016).

[41] *Nen-en 1993-1-8, design of steel structures - design of joints,* (2006).

[42] P. W. Christensen and A. Klarbring, *An introduction to structural optimization* (2008).

[43] *Civil engineering procedure* (Institution of Civil Engineering (ICE), 2015).

[44] H. de Ridder, *Design and construct in civil engineering,* (2009).

[45] I. Pečanac, *Structural parametric design for variant studies in the preliminary design phase,* (2019).

[46] R. van Mellaert, *Optimal design of steel structures according to the Eurocodes using mixed-integer linear programming methods*, Ph.D. thesis, KU Leuven (2017).

[47] D. E. Grierson and L. Xu, *Computer automated design of semirigid steel frameworks,* Journal of structural engineering **119** (1993).

[48] S. O. Degertekin and M. S. Hayalioglu, *Harmony search algorithm for minimum cost design of steel frames with semi-rigid connections and column bases,* Structural and Multidisciplinary optimization **42** (2010).

[49] M. S. Hayalioglu and S. O. Degertekin, *Minimum cost design of steel frames with semi-rigid connections and column bases via genetic optimization,* Computers and structures **60** (2005).

[50] J. van der Ploeg, *A knowledge-based framework for structural optimisation,* (2013).

[51] J. Haapio, T. Jokinen, M. Heinisuo, and M. Laasonen, *Feature-based cost and co2 equivalent optimizaiton of semi-rigid steel frames,* Design, fabrication and economy of metal structures (2013).

[52] K. Jármai, J. Farkas, and P. Uys, *Optimum design and cost calculation of a simple frame with welded or bolted corner joints,* IIW **24** (2003).

[53] E. Kameshki and M. Saka, *Optimum design of nonlinear steel frames with semi-rigid connections using a genetic algorithm,* Computers and Structures **79** (2001).

[54] R. Ajouz, *Optimising production costs of steel trusses,* (2018).

[55] G. J. P. S. Koen van Vliegen, Robin van der Have, *Mogelijkheden van digital engineering,* CEMENT **7** (2018).

[56] L. van der Linden, *Parametric engineering,* (2018).

[57] J. Coenders, *Definitie en geschiedenis parametrisch ontwerpen,* CEMENT **6** (2018).

[58] E. J. Reddy, C. Sridhar, and V. P. Rangadu, *Knowledge based engineering: Notionm, approaches and future trends,* American Journal of Intelligent systems **5**, 17 (2015).

[59] V. D. Holla, *Knowledge based engineering,* (2018).

[60] Viktor, *Automated engineering process analysis,* (2020).

[61] vericon, *Hoe werkt het,* (2020).

[62] Comsol, *The finite element method (fem),* (2020).

[63] R. Bijlaard and J. Brekelmans, *Joint design for economy and verification based on ec3,* Connections in steel structures **VI** (2008).

[64] ideastatica, *Why bother with connection stiffness,* (2021).

[65] A. van der Horst, *Construction technology of civil engineering projects* (2016).

[66] I. Jan, *Direct costs and indirect costs,* (2020).

[67] *Verbinden* (Bouwen met Staal, 2012).

[68] K. Watson, S. Dallas, and N. van der Kreek, *Costing of steelwork from feasibility through to completion,* Steel construction **30-02** (1996).

[69] P. Kraljic, *Puchasing must become supply management,* Harvard business review **9-10** (1983).

[70] Gottlieb, S. Christoffer, and K. Haugbølle, *The repetition effect in building and construction works,* (2010).

[71] *Salaris checker,* (2020).

[72] *Daily steel prices,* (2020).

[73] *Nen-en 1090-2,* (2018).

[74] *Four step process for execution class selection,* (2020).

[75] *Nen-en 1991-1-7+c1/nb, actions on structures - part 1-7: General actions - accidental actions,* (2011).

[76] *Stalen hallen* (Bouwen met staal, 2007).

[77] *Nen-en 1991-1-1, actions on structures - general actions - densities, self-weight, imposed loads for buildings,* (2011).

[78] *Hallen* (Bouwen met staal, 2019).

[79] A. Kling, *Mailcontact, crowners services,* (2020).

[80] *Nen-en 1993-1-1, design of steel structures, general rules and rules for buildings,* (2005).

[81] A. Mittal, P. Träger, and Corus, *Single-storey steel buldings, concept design,* (2008).

[82] *Inrichting laadkuil- en perron*, Van Wijk Nederland BV (2015).

[83] *Het inrichten van een laadperron*, Stertil Dock Products (2019).

[84] C. Jones, *Corrugated roof sheet sizes,* (2020).

[85] Huisman-Gemert, *Damwand dakplaten,* (2020).

[86] Bauder, *insulation products,* (2020).

[87] IDL-Group, *Upe u-channels,* (2020).

[88] IDL-Group, *Heb steel beams,* (2020).

[89] IDL-Group, *Hea steel beams,* (2020).

[90] IDL-Group, *Ipe steel beams,* (2020).

[91] *Nen-en 1991-1-4, general actions - wind actions,* (2005).

[92] *Nen-en 1991-1-3+c1 actions on structures - general actions - snow loads, national annex (2011),* (2011).

[93] L. M. C. Simões, *Optimization of frames with semi-rigid connections,* Computers and structures **60** (1996).

[94] A. A. van Douwen, *Connection design and economy (in dutch),* Bouwen met Staal **47** (1979).

[95] J. Scherrenberg, *Brug tussen hoofd- en detailontwerp,* Bouwen met Staal (2018).

[96] B. Flyvbjerg, M. S. Holm, and S. Buhl, *Underestimating costs in public works projects,* Journal of the american planning association **68** (2002).

[97] S. Kravanja and T. Zula, *Advances in engineering software,* Journal of the american planning association **68** (2010).

[98] J. Haapio, *Feature-Based costing method for skeletal steel structures based on the process approach*, Ph.D. thesis, Tampere University of Tehcnology (2012).

[99] *Nen-en 1991-1-4+a1+c2/nb, national annex to nen-en 1991-1-4,* (2006).

[100] *Knopen* (Bouwen met Staal, 2014).

[101] M. Nijgh, H. Xin, P. de Vries, R. Yan, and M. Veljkovic, *Cie4115 solution of home assignment 3, 2019/2020,* (2020).