Analysis of simple bolted connections with closed hollow section columns

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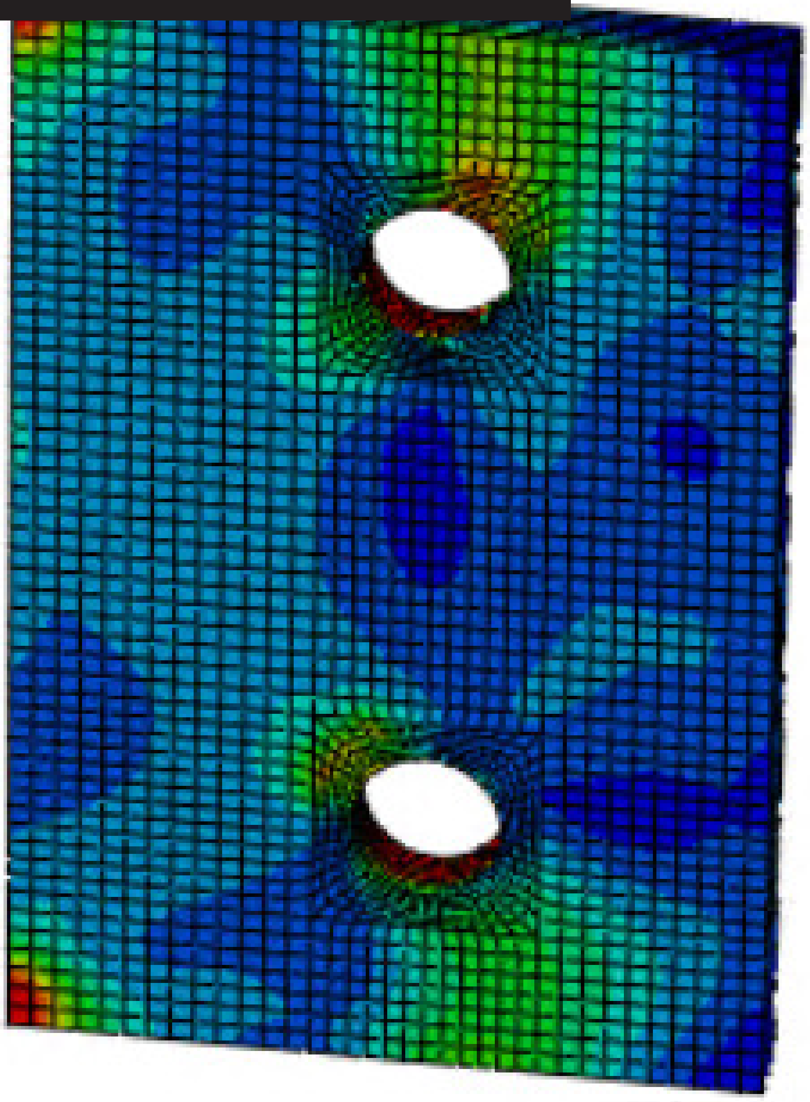

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Challenge the future

Analysis of simple bolted connections with closed hollow section columns

by

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Preface

This thesis was written as part of the bachelors degree Civil Engineering at the TU Delft.

All I knew before I started this work was that I wanted to do 'something' with Finite Elements Methods, as I had never done so before and I found this to be a significant deficiency in my personal toolkit as an engineer-to-be. Furthermore, my knowledge of bolted connections was quasi nill. This thesis has thought me a lot about both those things, for which I am very grateful. Therefore I would like to take this opportunity to express my gratitude towards my supervisors for their guidance.

I would like to end this preface with a quote of something prof. dr. M. Veljkovic said to me in our first meeting, and which I throughout this work have found out to be very true.

Greenland is in fact not green, and similarly, simple connections are not so simple.

H.J. Hendrikse
Delft, October 2021

Abstract

Eurocode 3 EN1993-1-8:2005 provides design rules for bolted connections in steel structures. However, for a class of connections called 'normally pinned' or 'simple' connections, designed to transfer only shear forces and normal forces without developing significant bending moments, no specific rules exist. EN1993-1-8:2021 does provide some guidelines for design of such joints, however this is limited to connections between profiles with open cross sections. When designing a normally pinned connection using a fin plate between a closed section SHS column and an I beam, the existing design rules do not cover the flexibility of the face of the column. However, in such a joint significant rotation due to this flexibility may occur.

Numerical analysis has revealed that for certain joint geometries this flexing can become governing for the total bearing capacity of the joint. In such cases, hand calculations alone do not suffice to predict the behaviour of the joint under load. Using an Abaqus model, a parametric study is conducted to reveal how the position of the point of zero bending moment is related to the geometry of the SHS column for a set joint design using a fin plate. In doing so, guidelines on the significance of column face flexibility are provided and the groundwork is laid for a study to find formulas to describe this flexibility accurately.

Contents

1

Introduction

Integral to building steel structures are the joints connecting the different members of a structure. Various types of connections exist. In this thesis an in depth analysis is made of one particular type of joint, in which a hollow section column and an open section beam are connected using a fin plate.
Various challenges arise when designing such a connection, mainly due to the flexing of the column face under load.The Eurocode 3 EN 1993-1-8:2005 provides little guidance to designers in regards to this problem.
In this work a comprehensive analysis of the connection is done using both analytical and numerical methods. This is done for varying dimensions and joint geometries so that relations between various connection characteristics and the behaviour under load can be studied. Furthermore, a comparison is made between the joint resistances predicted using hand calculations according to the Eurocode, those obtained using Finite Element Analysis using both IDEA StatiCa as well as Abaqus.

1.1. Problem statement

When designing a simple connection using a fin plate to connect an open section (I-beam) column to an open section beam the Eurocode provides comprehensive design rules and calculation methods. For such a connection, which is designed to only transfer shear forces and normal forces and no bending moment, the resistance can easily be determined using simple (hand) calculations. Central to these calculations lies the fact that the center of rotation - which is the point in which zero bending moment occurs - of such a connection can assumed to lie in the center of the bolt group. This is because the fin plate lies almost exactly in line with the web of the column, resulting in great stiffness to rotation of the fin plate itself. (Stark, 2012) [1]

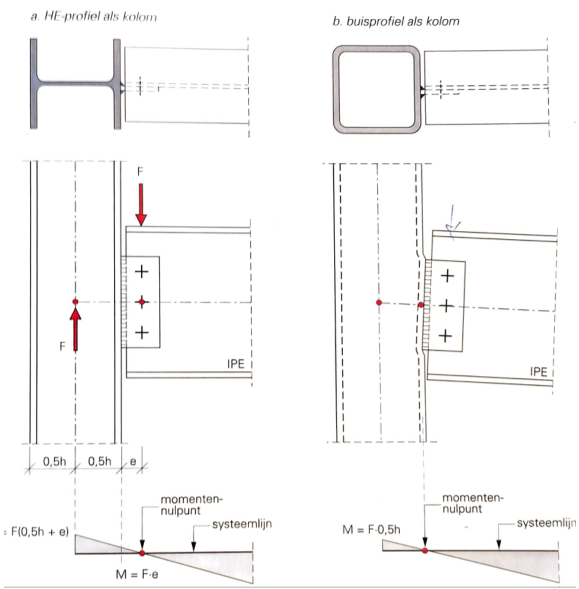


Figure 1.1: Location of the center of rotation in a connecting using an I-beam column and using an closed hollow section column (Stark, 2012)[1]

However when the column consists of a closed section and the fin plate is welded to the face of the column, the resulting connection is less stiff. Because the column face will flex under load as shown in 1.1, and thus the assumption of the center of rotation lying in the center of the

bolt group is no longer valid. In this case, the
center of rotation is assumed to lie in the face of the column. This is however only an approximation of the real behaviour of such a connection. In reality, the zero bending moment point lies somewhere in between these two extremes. No precise method for determining the center of rotation exists. Finding this point and providing analytical relations to do so is the central theme of this thesis. The research question for this work thus becomes

> *How is the point of zero bending moment changing position in a simple connection using a fin plate depending on a stiffness of the flange of an SHS column*

Furthermore the goal of this thesis is to find relations between various joint parameters and the location of the point of zero bending moment. Understanding where this point lies is important because knowledge of this is required to accurately predict the bending moment that will be transferred to the column due to the eccentricity of shear force. Also this thesis aims to provide an in depth analysis at how such a connection is to be calculated according to Eurocode 3, and explain what is and what is not provided by the code.

1.2. Methodology

In order to find a solution to this problem, first a joint design is presented. In this design the dimensions for the column, beam, fin plate and bolts are assumed based on industry standards. This first design will serve as a framework to demonstrate the simple (hand) calculations provided by the Eurocode. In these analytical calculations the flexibility of the column face is not considered. A Python program is created to allow for rapid iterations of various joint parameters later on.

In the second phase, a numerical analysis of the designed joint using Finite Element Analysis is done. The software used for this is IDEA StatiCa. IDEA StatiCa allows for analysis of the behaviour under load and results in obtaining the maximum joint resistance. The predicted maximum joint resistance and failure mode will than be compared to that obtained using hand calculations.

To gain a full understanding of the behaviour of the joint and in order to find the position of the zero bending moment point a more detailed Finite Element Analysis is then carried out using Abaqus. First a model of the fin plate connection is created and validated based on previous experimental work done by Sebastian Navarro. After the model in Abaqus has been validated, it is used to run simulations of the connection design previously studied using Eurocode design rules and IDEA StatiCa. The results of the Abaqus model are then compared to the results obtained using IDEA StatiCa and the hand calculations. Based on the resultant forces in the bolts and in the welds of the fin plate the location of the zero moment point can then be determined. Next a set of parameters for design of the connection, such as dimensions and thickness of the column, is chosen based on industry standards. The Abaqus model is then run using these varying parameters and the center of rotation is determined for each of the iterations. From this, formulas describing the location of the center of rotation are derived.

1.3. Document structure

In 2 existing literature on the design and calculation methods of steel joints is reviewed to provide the required background for this thesis. Different components of steel joints are described, and the classification of joints according to EN 1993-1-8 is explained.

In 3 the hand calculations for a fin plate connection are performed based on Eurocode 3 in order to find the maximum bearing capacity. This is done on the basis of different failure modes of the joint that may occur, which are all described. Also a new way to calculate the bearing capacity of bolt holes which is present in EN1993-1-8:2021 is compared to those present in EN1993-1-8:2005.

In 4 numerical models using both IDEA StatiCa and Abaqus are presented. Both software packages are described. The Abaqus model is validated against experimental work carried out by Sebastian Navarro in 2017.

In 5 the results of the numerical analysis are presented. Finally, in 6 a comparison between the results of the hand calculations and the different numerical analyses is made. Based on the results from the Abaqus analysis, conclusions are drawn about the location of the zero bending moment point, and recommendations for further work are made.

2
Literature review

The aim of this literature review is to provide the necessary background for understanding the work done in this thesis. It gives insight in some fundamental concepts used during the analysis performed on the joints. Essential to this is an understanding of the different types of bolted joints and their classification. The basis on which this classification is done is found in EN1993-1-8:2005.

2.1. Joint classification

In this section the two types of steel joint classification as presented in Eurocode EN1993-1-8:2005 are explained. Accurate classification of joints is important, because it is essential to the framework in which a structure is designed. The Eurocode provides guidelines on how a joint is to be modelled based on its classification. And thus, accurate prediction of the distribution of forces in a complex steel structure is directly dependant on accurate classification of joints. (Jaspart, 2016) [2]

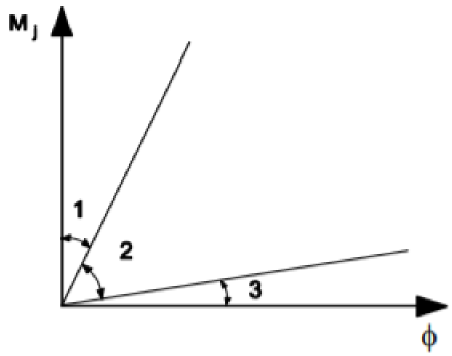
The Eurocode provides two main ways of classifying joints. It can be done either by their stiffness or by their resistance.

2.1.1. Joint classification by stiffness

Classification by stiffness is relevant when elastic behaviour is assumed in the structure. Regarding the rotational stiffness EN1993-1-8 provides the following classes:

- Rigid joints
- Semi-rigid joints
- Normally pinned joints

This classification is based solely on the properties of the beam. In figure 2.1 the design rules for joint classification are shown.(EN1993-1-8:2005) [3]



Zone 1: rigid, if $S_{j,ini} \geq k_b EI_b / L_b$

where:

$k_b = 8$ for frames where the bracing system reduces the horizontal displacement by at least 80 %

$k_b = 25$ for other frames, provided that in every storey $K_b/K_c \geq 0{,}1$ *)

Zone 2: semi-rigid

All joints in zone 2 should be classified as semi-rigid. Joints in zones 1 or 3 may optionally also be treated as semi-rigid.

Zone 3: nominally pinned, if $S_{j,ini} \leq 0{,}5\, EI_b / L_b$

*) For frames where $K_b/K_c < 0{,}1$ the joints should be classified as semi-rigid.

Key:

- K_b is the mean value of I_b/L_b for all the beams at the top of that storey;
- K_c is the mean value of I_c/L_c for all the columns in that storey;
- I_b is the second moment of area of a beam;
- I_c is the second moment of area of a column;
- L_b is the span of a beam (centre-to-centre of columns);
- L_c is the storey height of a column.

Figure 2.1: Joint classification by stiffness according to EN1993-1-8:2005

Rigid joints

The joints that fall in this category ensure full rotational stiffness. This mean full continuity of load transfer between elements is provided. Rigid joints may be assumed to be perfectly stiff when modelling the structure.

Normally pinned joints

A normally pinned or simple joint is able to transfer the loads between members without developing significant bending moments. The joint should also allow the rotations that arise when the members are under load.

Semi-rigid joints

Any joint that does not fall in the previous two categories is classified as semi-rigid.

2.1.2. Joint classification by strength

Classification based on strength of the joint is relevant when plastic deformation is assumed in the structure. (Jaspart, 2016)

Full strength joints

A joint is considered full strength when its resistance is greater than that of the elements which it connects. Furthermore, the code provides some limitations which the joint must meet which are presented in 2.2. When a joint is full strength, a plastic hinge does not develop in the joint itself but in the members attached to it.

Normally pinned joints

Like with classification by stiffness, a joint is considered Normally pinned if it is able to transfer loads between membems without developing significant bending moments. Furthermore, a maximum bending moment resistance of 25% of that of a full strength joint is defined in the Eurocode.

Partial strength joints

Partial strength joints are those that do not fall in the two categories mentioned above.

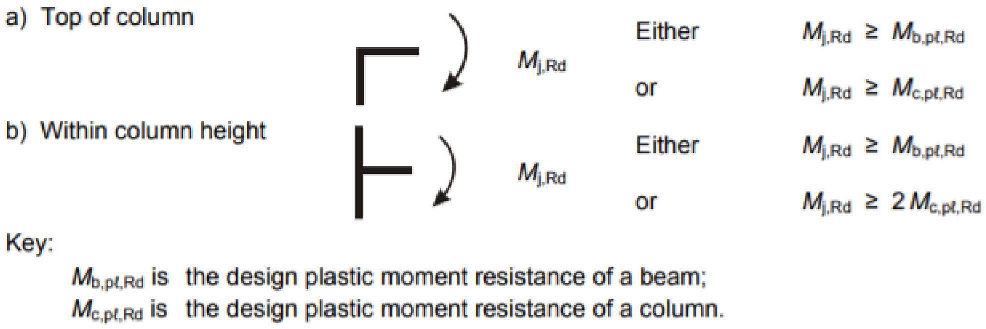


Figure 2.2: Joint classification by strength according to EN1993-1-8:2005

Rigid or semi-rigid joints do not fall in the scope of this thesis. However normally pinned joints, also known as simple joints are discussed in more detail in the next section.

2.2. Simple connections

In this section simple connections are discussed in more detail. Simple joints are assumed not to transfer any bending moment, they transfer only shear and normal forces. However, bending moments can develop in the column due to the eccentricity of the center of rotation of the joint as shown in 1.1.(Stark, 2012)[1]

2.2.1. Types of simple connections

Three types of simple connections can be identified:

- Header plate connection
- Web cleat connection
- Fin plate connection

(Jaspart, 2016)

Header plate connection

A header plate connection consists of a plate which is welded to the beam web and bolted to the column as shown in 2.3.

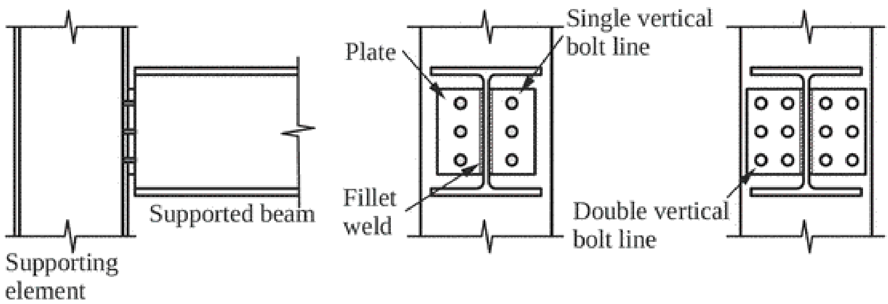


Figure 2.3: Pinned connection using a header plate

Web cleat connection

Web cleat connections consists of two web cleats which are bolted to both the beam web and the supporting column as shown in (Jaspart, 2016)

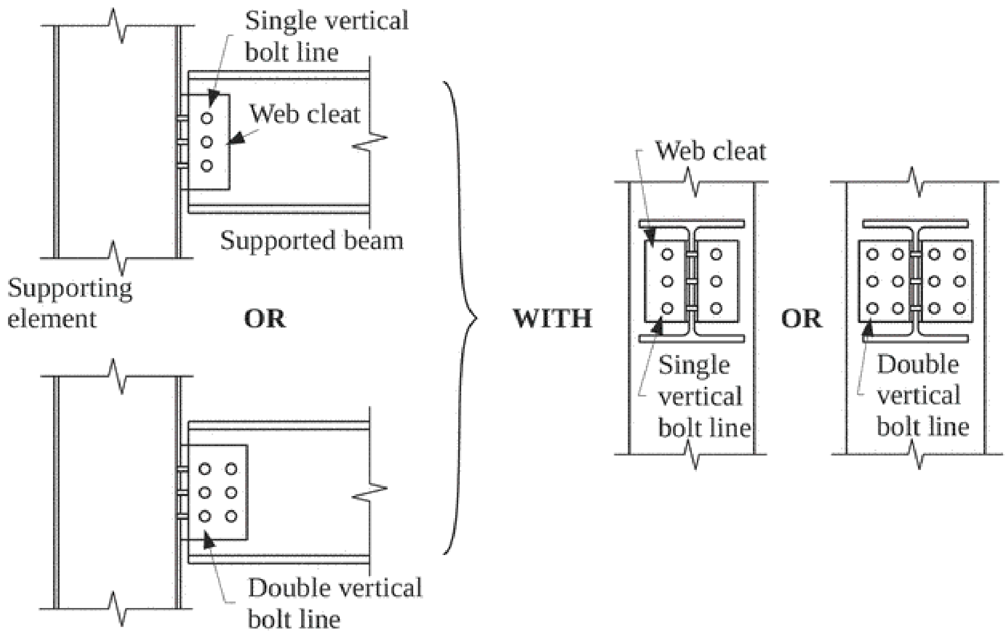


Figure 2.4: Pinned connection using web cleats

Fin plate connection

Fin plate connections adapt a fin plate which is welded to the column and bolted to the web of the beam as shown in 2.5. As fin plate connections are central to this thesis their design will be discussed in more detail in 2.2.2.

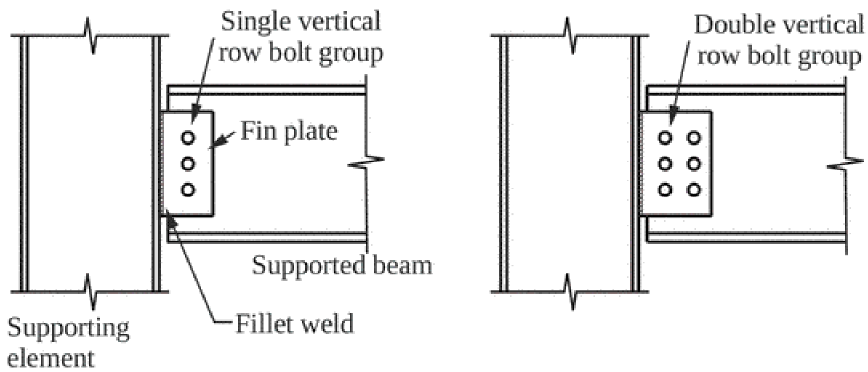


Figure 2.5: Pinned connection using a fin plate

2.2.2. Design of simple connections

EN:1993-1-8:2005 does not provide any specific design rules for simple connections. However, various formulae are given to calculate the resistances of different parts of the joint against different failure modes. The resistance of the joint simply follows the lowest of those resistances. In EN:1993-1-8:2021 however, a guide for the design of simple connections is present, following the same procedure.(Jaspart, 2016)

Jaspart and Weynand describe two characteristics to which all simple connections must adhere. Simple connections should:

- Possess sufficient rotational capacity
- Posses sufficient ductility

The first requirement ensures that the joint can deform sufficiently without developing significant bending moments. Thus the joint can be accurately modelled as a hinge.

The second requirement is linked to the failure mode of the joint. It is there to ensure that a brittle failure does not occur. From this follows that a design in which a weld failure or a shear failure of the bolts is governing is not acceptable. The different failure modes of a fin plate connection and whether the are ductile or not is shown in table 2.1. The different failure modes will be discussed in more detail in chapter 3.(Jaspart, 2016)

Failure mode	**Type**
Bolt shear	Brittle
Bearing of in plate	Ductile
Gross section failure of fin plate	Ductile
Net section failure of fin plate	Ductile
Block failure of fin plate	Ductile
Bearing of in beam web	Ductile
Gross section failure of beam web	Ductile
Net section failure of beam web	Ductile
Block failure of beam web	Ductile
Buckling of fin plate	Brittle
Failure of weld	Brittle

Table 2.1: Different failure modes of a simple connection using a fin plate

Weld failure

In order to avoid brittle behaviour of the joint, the weld rupture strength should always be higher than the yield strength of the connected parts. When this criterion is met the strength of the weld is greater than the strength of the fin plate. A weld that meets this criterion is considered 'full strength'. For fin plate connection design Jaspart and Weynand provide the following minimum throat thickness for welds in order to ensure that they are full strength based on clause 4.5.3.2. of EN1993-1-8.(Jaspart, 2016)

Steel grade	S235	S275	S355	S420 M	S420 N	S460 M	S460 N
Full strength double fillet welds	$a \geq 0.46t$	$a \geq 0.48t$	$a \geq 0.55t$	$a \geq 0.71t$	$a \geq 0.68t$	$a \geq 0.74t$	$a \geq 0.70t$

Figure 2.6: Minimum weld thickness to ensure full strength welds

3

Hand calculations according to Eurocode

In this chapter hand calculations for the maximum bearing capacity of the simple fin plate connection between a column and a beam based on the EN199-1-8:2005 are demonstrated. For this, an arbitrary joint design is chosen. The details of the joint are described in 3.1. Next, the different failure modes are described and their respective calculations are done in 3.2. The bearing capacity of the joint then follows:

$$V_{Rd,joint} = min(V_{Rd,1}, V_{Rd,2} ... V_{Rd,11}) \tag{3.1}$$

An alternative way of calculating the bearing capacity of the bolt holes than that described in EN1993-1-8:2005 has recently been proposed. This method is part of EN1993-1-8:2021. A comparison between the two methods is made in this chapter.(Stark, 2014) [4][1]

3.1. Joint design

The simple joint is designed using a fin plate and two bolts with geometry as shown in figure 3.1. In table 3.1 the dimensions and parameters of the joint are described.

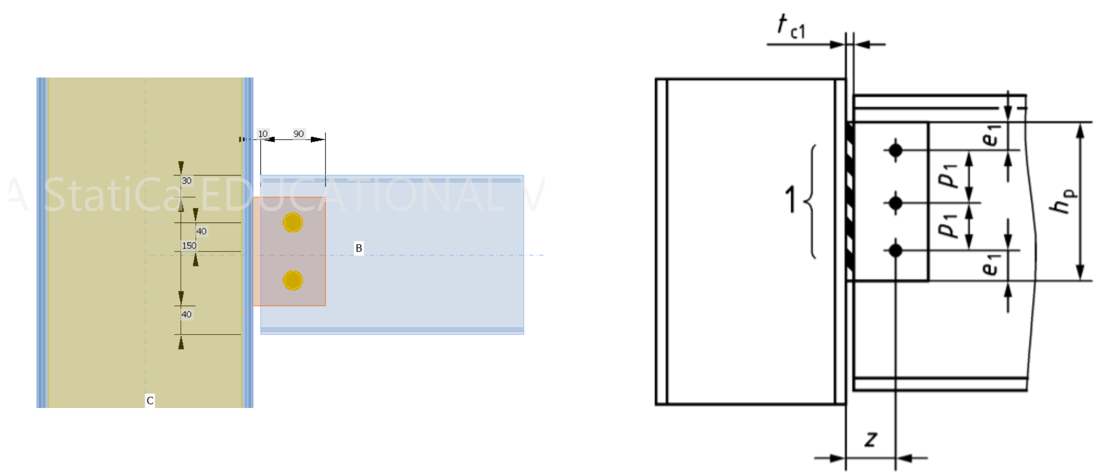


Figure 3.1: Joint design and description of the joint parameters

Column profile	SHS300/300x6.3
Column material	S355
Beam profile	IPE220
Beam material	S355
Bolts	M20x8.8
No. bolts	2
Fin plate thickness	10mm
Fin plate material	S355
Welds	a4

Table 3.1: Properties of the joint

3.2. Bearing resistances according to EN1993-1-5:2005

In this section the different failure modes for the joint are illustrated, and their respective bearing capacities are calculated based on Eurocode 3.

3.2.1. Shear resistance of bolts

The shear resistance of the bolts follows

$$F_{V,rd} = \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M_2}} \tag{3.2}$$

Because the effective area through the threaded part of the bolt A is equal to $245mm^2$ it follows that:

$$F_{V,rd} = \frac{0.6 \cdot 800 \cdot 245}{1.25} = 94,1kN \tag{3.3}$$

for one bolt. The shear resistance of all bolts follows from:

$$V_{Rd,1} = \frac{n \cdot F_{V,rd}}{\sqrt{1 + \left(\frac{6e}{(n+1)p_1}\right)^2}} \tag{3.4}$$

with n the number of bolts, p_1 the distance between two bolts and e the eccentricity of the joint. Thus for this joint we find:

$$V_{Rd,1} = \frac{2 \cdot 94.6}{\sqrt{1 + \left(\frac{6 \cdot 55}{(2+1)50}\right)^2}} = 110.7kN \tag{3.5}$$

3.2.2. Bearing resistance of fin plate

For end bolts in the direction perpendicular to the load transfer the Eurocode gives:

$$k_1 = min\left(2.8\frac{e_2}{d_0} - 1.7; 1.4\frac{p_2}{d_0} - 1.7; 2.5\right) \tag{3.6}$$

and

$$\alpha_b = min\left(\frac{e_1}{3d_0}; 1.0\right) \tag{3.7}$$

with $d_0 = 22mm$, $e_1 = 55mm$, $p_2 = 80mm$ and $e_2 = 45mm$ we find:

$$k_1 = min\left(2.8\frac{55}{22} - 1.7; 1.4\frac{80}{22} - 1.7; 2.5\right) = 2.5 \tag{3.8}$$

and

$$\alpha_b = 0.83 \tag{3.9}$$

Next it follows that

$$F_{b,hor,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M_2}} \tag{3.10}$$

$$F_{b,hor,Rd} = \frac{2.5 \cdot 0.83 \cdot 490 \cdot 20 \cdot 10}{1.25} = 162.7kN \tag{3.11}$$

and

$$F_{b,ver,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M_2}} \tag{3.12}$$

$$F_{b,ver,Rd} = \frac{2.5 \cdot 0.83 \cdot 490 \cdot 20 \cdot 10}{1.25} = 162.7kN \tag{3.13}$$

The total resistance follows:

$$V_{Rd,2} = \frac{n}{\sqrt{\left(\frac{1+n\cdot\alpha}{F_{b,ver,Rd}}\right)^2 + \left(\frac{n\cdot\beta}{F_{b,hor,Rd}}\right)^2}} \tag{3.14}$$

Because there is only one bolt row $\alpha = 0$. $\beta = \frac{6\cdot z}{p_1\cdot n(n+1)} = \frac{6\cdot 55}{80\cdot 2(2+1)} = 0.68$ So we find

$$V_{Rd,2} = 98.8kN \tag{3.15}$$

3.2.3. Failure of the gross section of the fin plate

Shear failure in the gross section of the fin plate can be predicted using:

$$V_{Rd,3} = \frac{h_t \cdot t_p}{1.27} \cdot \frac{f_{yp}}{\sqrt{3} \cdot \gamma_{M_0}} \tag{3.16}$$

with $h_t = 150mm$ and $t_p = 10mm$ we find

$$V_{Rd,3} = \frac{150 \cdot 10}{1.27} \cdot \frac{355}{\sqrt{3} \cdot 1.1} = 219.2kN \tag{3.17}$$

3.2.4. Failure of the net section of the fin plate

Shear failure in the net section of the fin plate can be predicted using:

$$V_{Rd,4} = A_{v,net} \cdot \frac{f_{up}}{\sqrt{3} \cdot \gamma_{M_0}} \tag{3.18}$$

with $A_{v,net} = t_p(h_p - (n \cdot d_0)) = 10(150 - (2 \cdot 22)) = 1060mm^2$ we find

$$V_{Rd,4} = 1060 \cdot \frac{490}{\sqrt{3} \cdot 1.1} = 197.5kN \tag{3.19}$$

3.2.5. Block failure of the fin plate

In this failure mode, there is a partial shear and a partial tension failure fin the fin plate.

$$V_{Rd,5} = 0.5\frac{f_{up} \cdot A_{nt}}{\gamma_{M_2}} + \frac{1}{\sqrt{3}} \cdot f_{yp} \cdot \frac{A_{nv}}{\gamma_{M_0}} \tag{3.20}$$

Where the net area in tension $A_{nt} = t_p(e_2 - \frac{d_0}{2} = 10(45 - \frac{22}{2} = 340mm^2$ and the net area in shear $A_{nv} = t_p(h_p - e_1 - (n - 0.5)d_0 = 10(150 - 35 - (2 - 0.5)d_0 = 820mm^2$ resulting in:

$$V_{Rd,5} = 0.5\frac{490 \cdot 340}{1.25} + \frac{1}{\sqrt{3}} \cdot 355 \cdot \frac{820}{1.1} = 219.4kN \tag{3.21}$$

3.2.6. Bearing resistance of the beam web

$$k_1 = min\left(2.8\frac{e_2}{d_0} - 1.7; 1.4\frac{p_2}{d_0} - 1.7; 2.5\right) \tag{3.22}$$

and

$$\alpha_b = min\left(\frac{e_1}{3d_0}; 1.0\right) \tag{3.23}$$

with $d_0 = 22mm$, $e_1 = 55mm$, $p_2 = 80mm$ and $e_2 = 45mm$ we find:

$$k_1 = min\left(2.8\frac{55}{22} - 1.7; 1.4\frac{80}{22} - 1.7; 2.5\right) = 2.5 \tag{3.24}$$

and

$$\alpha_b = 0.83 \tag{3.25}$$

Next it follows that

$$F_{b,hor,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M_2}} \tag{3.26}$$

$$F_{b,hor,Rd} = \frac{2.5 \cdot 0.83 \cdot 490 \cdot 20 \cdot 10}{1.25} = 162.7kN \tag{3.27}$$

and

$$F_{b,ver,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{up} \cdot d \cdot t_p}{\gamma_{M_2}} \tag{3.28}$$

$$F_{b,ver,Rd} = \frac{2.5 \cdot 0.83 \cdot 490 \cdot 20 \cdot 10}{1.25} = 162.7kN \tag{3.29}$$

The total resistance follows:

$$V_{Rd,6} = \frac{n}{\sqrt{\left(\frac{1+n\cdot\alpha}{F_{b,ver,Rd}}\right)^2 + \left(\frac{n\cdot\beta}{F_{b,hor,Rd}}\right)^2}} \tag{3.30}$$

Because there is only one bolt row $\alpha = 0$. $\beta = \frac{6\cdot z}{p_1\cdot n(n+1)} = \frac{6\cdot 55}{80\cdot 2(2+1)} = 0.68$ So we find

$$V_{Rd,6} = 172.6kN \tag{3.31}$$

3.2.7. Failure of the gross section of the beam web

Shear failure in the gross section of the beam web can be predicted using:

$$V_{Rd,7} = A_{b,v} \cdot \frac{f_{ybw}}{\sqrt{3} \cdot \gamma_{M_0}} \tag{3.32}$$

with $A_{b,v} = h \cdot t_w = 220 \cdot 7.1 = 1562mm^2$ it is found that

$$V_{Rd,7} = 2130 \cdot \frac{355}{\sqrt{3} \cdot 1.1} = 318.0kN \tag{3.33}$$

3.2.8. Failure of the net section of the beam web

Shear failure of the net section of the beam web can be predicted using:

$$V_{Rd,8} = A_{b,v,net} \cdot \frac{f_{ubw}}{\sqrt{3} \cdot \gamma_{M_0}} \tag{3.34}$$

with $A_{b,v,net} = A_{b,v} - (n \cdot d_0 \cdot t_b w) = 1562 - (2 \cdot 22 \cdot 7.1) = 1250mm^2$ it is found that

$$V_{Rd,8} = 1250 \cdot \frac{355}{\sqrt{3} \cdot 1.1} = 232.9kN \tag{3.35}$$

3.2.9. Block failure of the beam web

In this failure mode, there is again a partial shear and partial tension failure.

$$V_{Rd,9} = 0.5\frac{f_{ubw} \cdot A_{nt}}{\gamma_{M_2}} + \frac{1}{\sqrt{3}} \cdot f_{ybw} \cdot \frac{A_{nv}}{\gamma_{M_0}} \tag{3.36}$$

Where the net area in tension $A_{nt} = 45 \cdot 7.1 = 320mm^2$ and the net area in shear $A_{nv} = 125 \cdot 7.1 = 888mm^2$ resulting in:

$$V_{Rd,9} = 0.5\frac{490 \cdot 320}{1.25} + \frac{1}{\sqrt{3}} \cdot 355 \cdot \frac{888}{1.1} = 228.1kN \tag{3.37}$$

3.2.10. Bending resistance of the fin plate

According to the Eurocode, if $h_p \geq 2.73z$ then this failure more is not relevant. For the joint considered here $h_p = 150$ and $2.73z = 150$, so this failure mode is not considered.

3.2.11. Buckling of the fin plate

For buckling resistance of the fin plate the Eurocode states that if

$$z_p \leq \frac{t_p}{0.15} \tag{3.38}$$

$$V_{b,p,Rd} = \frac{t_p h_p^2 f_{y,p}}{6z\gamma_{M_0}} \tag{3.39}$$

For the joint considered here $z_p = 55mm and \frac{t_p}{0.15} = \frac{10}{0.15} = 66.7mm$, thus 3.38 is satisfied. Thus the resulting buckling resistance equals:

$$V_{b,p,Rd} = \frac{10 \cdot 150^2 \cdot 355}{6 \cdot 55 \cdot 1.1} = 220.0kN \tag{3.40}$$

4

Numerical analysis

In this chapter the numerical analysis that was carried out on the simple bolted connections is described. Basic background information on Finite Element analysis is given and the different software packages used to analyse the simple connections are explained.

4.1. Finite Element Analysis

Finite Element Analsis (FEA) is a technique which can be used to accurately model physical effects suchs as the behaviour of materials under load. At the core of FEA lies the fact that large problems may be subdivided into smaller elements which can be analysed more easily. In regards to joint design, the complex differential equations that arise when calculating the behaviour under loads are solved numerically by subdividing the system into smaller parts which can then individually be solved algebraically. For engineering purposes, FEA allows accurate prediction of the behaviour of a joint under load. However, there are many variables and phenomena to be taken into account when setting up a model of a steel joint, such as geometric details, (nonlinear) material properties frictional interactions between different elements to be taken into account. Therefore, validation of a model is essential before conclusions can be drawn.
(Dassault Systemes Simulia Corp., 2012)

Two different software packages were used to model the model of the fin plate connections. Their respective properties and the model setup will be described in the next section.

4.2. IDEA StatiCa

IDEA StatiCa is an engineering tool designed specifically for the analysis of steel joints. It combines traditional finite element methods with the rules specified in the Eurocode through a technique called 'Component Based Finite Element Methods' or CBFEM. In this method, individual components of a joint are checked according to the rules specified by the Eurocode. However, in order to obtain accurate stress distributions, Finite Element Analysis is used to compute the stresses and deformations in the components themselves. Thus, IDEA StatiCa is a hybrid between hand calculations described by the Eurocode and pure Finite Element Analysis.

IDEA StatiCa contains a library of all industry standard steel members and fasteners. Thus, setting up a joint model in IDEA StatiCa can be done with very little effort. There are however some simplifications built in to the program which are important to note.

Geometrical simplifications

IDEA StatiCa makes use of several geometrical simplifications when running an analysis. For instance, all members are built up out of plate elements as shown in 4.1. I-beam radii are thus not taken into account.(IDEAStatiCa Corp, 2019) [5]

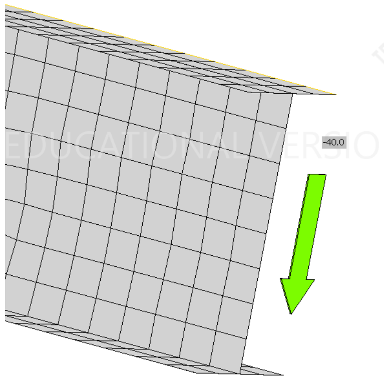


Figure 4.1: I-beam mesh in IDEA StatiCa

Material simplifications

Furthermore, IDEA StatiCa adapts a simplified material model for steel. It assumes ideal plastic behaviour under stress. Elastic behaviour is assumed untill the yield stress is reached, after which the material yields but stresses no longer increase. Any strain hardening effects under plastic deformation are thus ignored, which can lead to an underestimation of the joint resistance. The tool does however allow to setup a maximum plastic strain. Based on recommendations from Navarro navarrocite a limit strain of 5 % was chosen for all analysis in this thesis.

4.3. Abaqus

Whilst IDEA StatiCa allows for modelling of the joint behaviour with very little effort its options for the analysis of the results are limited. While the bearing capacity of a design can easily and quickly be found, it is not suited for the kind of in depth analysis needed to study the point of zero bending moment in a fin plate. For that, a model using Abaqus was created.

Abaqus is a FEA program that allows for simulation of a wide variety of physical phenomena. It is well suited for modelling of steel joints. Because there is a wide variety of parameters to set up, validation of a model is essential. Because lab experiments were not possible in the scope of this thesis, a model was set up and validated against experimental work done by Sebastian Navarro in 2017 on a similar simple fin plate connection between a hollow section column and a beam.(Dassault Systemes Simulia Corp., 2012)

4.3.1. Model setup

In this section different aspects of the setup of the Abaqus model are described.

Geometry

The joint geometry is modelled according to the nominal dimensions of the standard profiles, except for the bolts and the welds. The shank of the bolt, the bolt head and the nut are all modelled as cylindrical elements to limit mesh complexity and save computational. This way of simplifying the bolts by ignoring the threads still yields accurate results. (Kim, Yoon, Kang, 2006)[6]. For the weld geometry, a triangular cross section with the specified throat thickness was used as shown in 4.2

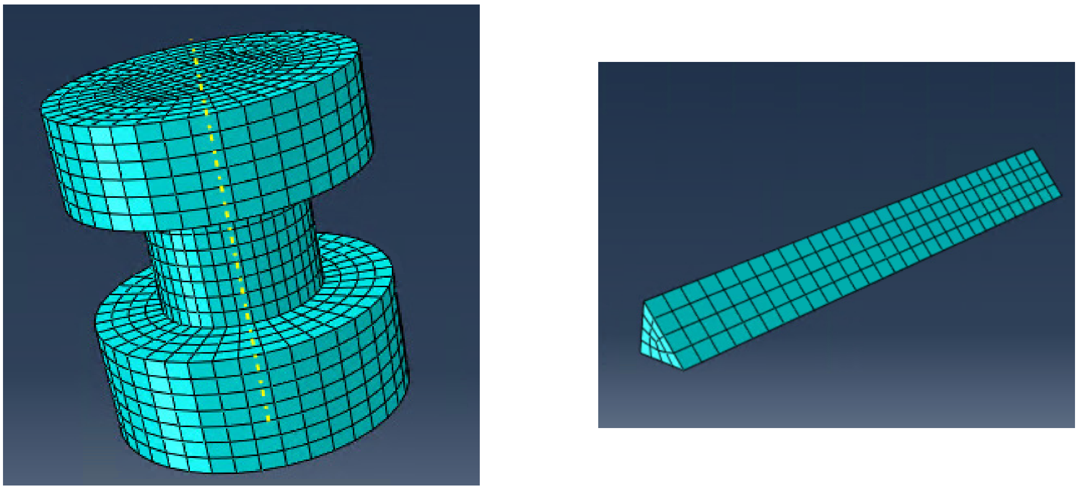


Figure 4.2: Simplified geometry of the bolts and the weld

Mesh
Hexagonal C3D8R elements with reduced integration were used for the mesh. A minimum of 4 elements was set through the thickness of all parts. At areas of interest where high stresses are expected, such as around the bolt holes a higher mesh density is applied as can be seen in 4.3

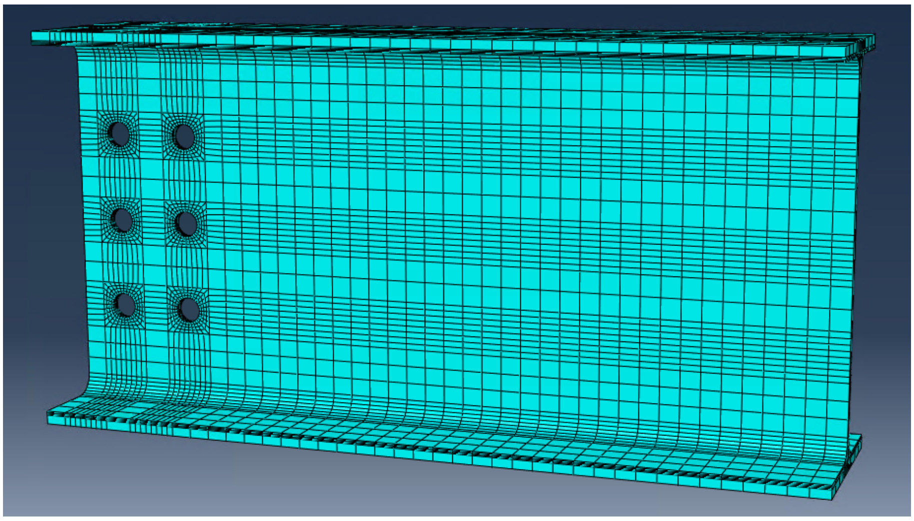


Figure 4.3: Mesh of the beam

Contact interactions
Frictional contact is specified between the bolts and the boltholes, between the faces of the bolt heads and nuts and the beam and fin plate, and between the fin plate and the beam. To save computational time, the interaction between the fin plate and the beam has only been specified at 20x20mm patches on the corners of the fin plate. Hard contact is specified for normal interaction, and a friction coefficient of 0.4 is specified for tangential movement.

Boundary conditions
In order to ensure onl shear force is applied to the connection, the set of boundary conditions was chosen as follows: The free end of the beam was fixed in place. The face of the column opposite to the joint is fixed to be symmetrical in the y-plane, allowing only for movement along the z-axis. The deformed joint and boundary conditions are shown in 4.4

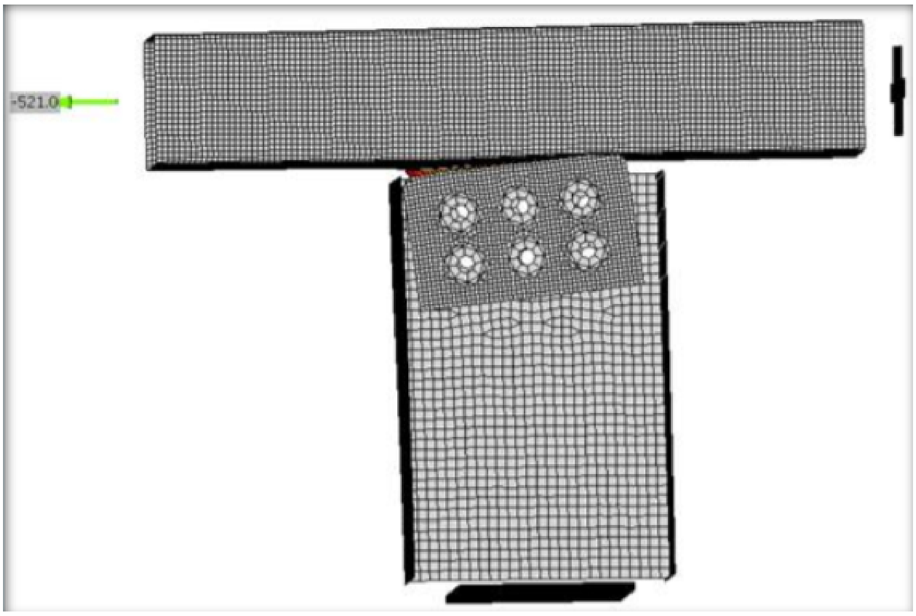


Figure 4.4: Boundary conditions of the model

Solver

Because of the high non-linearity of the model and large initial movement due to the clearance between the boltholes and the bolts the Dynamic/Explicit solver was used. This is done to ensure that convergence problems do not arise. With this solver, run time for the model is around 3 hours.

4.3.2. Model validation

In this section, the model is validated based on numerical analysis and experimental work carried out by Navarro in 2017. The joint design on which Navarro carried out experimental consists of a fin plate design using six bolts. The elements are described in table 4.1 and the geometry of the model is shown in 4.5. (Navarro, 2017)

Element	Profile/plate	Steel grade
Column	SHS200x8CR	S355
Beam	IPE400	S355
Fin plate	200x10	S355
Bolts	M24	10.9

Table 4.1: Joint elements

The resulting predicted joint resistance is compared to both the numerical analysis and experimental work carried out by Navarro in 2017. The governing failure mechanism was found to be a failure of the weld, although significant bearing of the boltholes is also observed. The results of the comparison are shown in table 4.2.

Description	Joint resistance (kN)
Numerical analysis (2017)	
Experimental results (2017)	
Numerical analysis	

Table 4.2: Joint resistance

Finding the point of zero bending moment in the fin plate

Abaqus allows for detailed analysis of the obtained stress distributions in all different parts of the joint. By using the section cut function, the stresses at a chosen cross section can be examined. Furthermore,

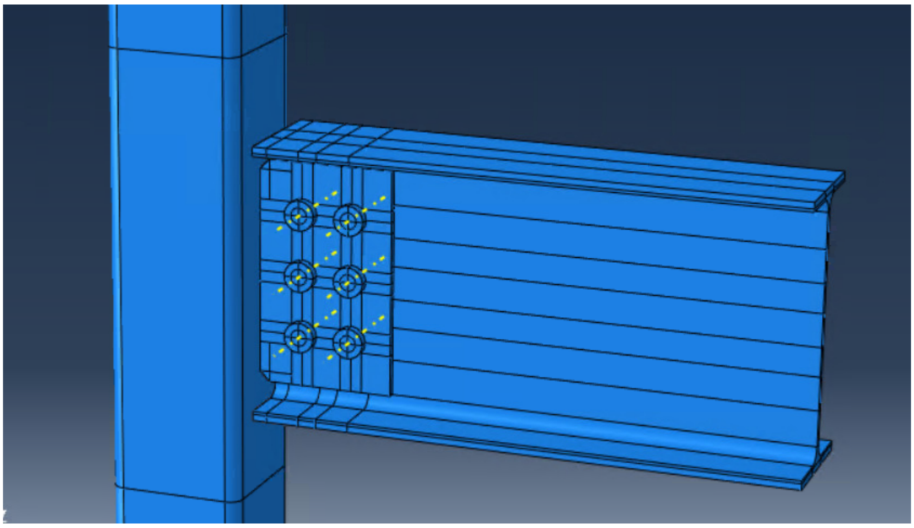


Figure 4.5: Geometry of the assembled joint

using integration of all the surface stresses of a chosen section cut, the resultant force and bending moment of a surface can be found. The components of the bending moment can also be displayed, resulting in the bending moment around one axis. This is shown in 4.7.

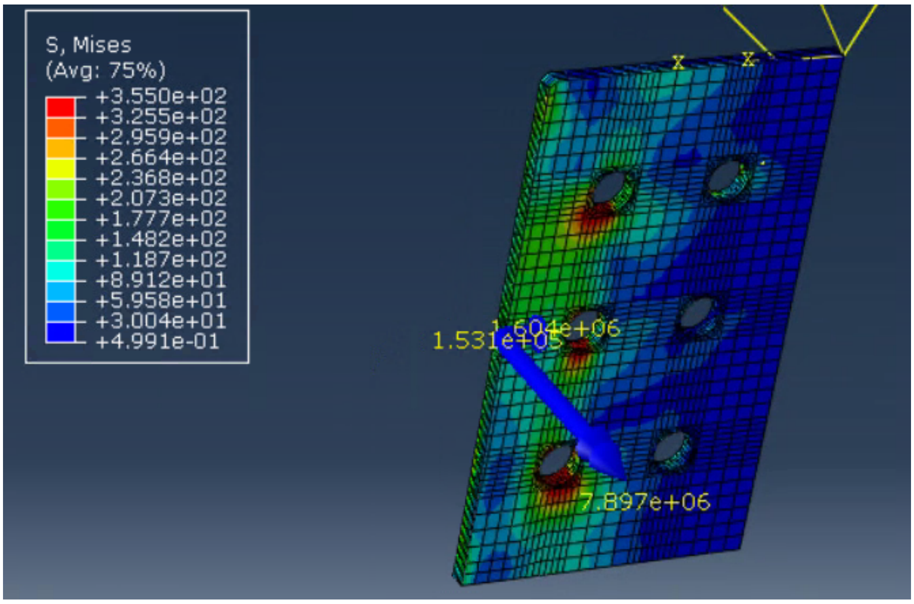


Figure 4.6: Section cut of the fin plate showing the resulting bending moments

Next, a local coordinate system is created with the XY-plane parallel to the top of the fin plate, and the X-axis along the width of the fin plate. By adjusting the surface cut such that the moment around the Y-axis is near zero, the location of the point of zero bending moment in the fin plate can now be found. In figure 4.7 this technique is demonstrated.

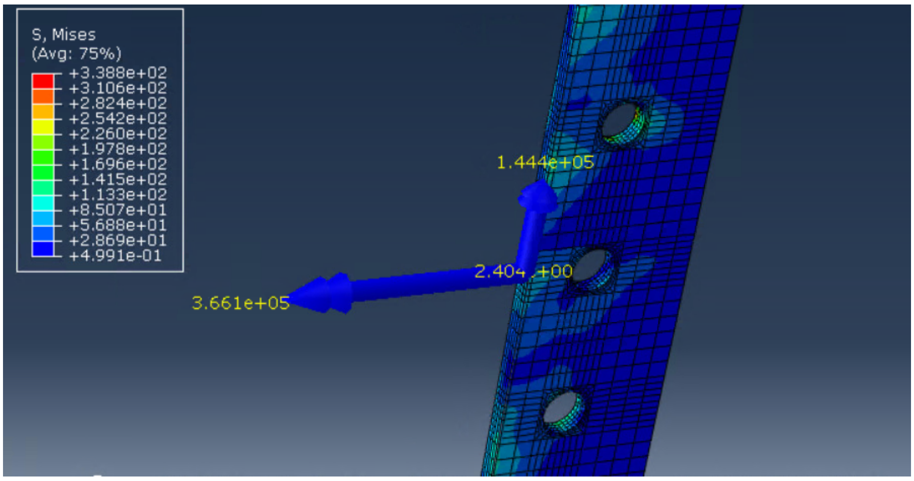


Figure 4.7: Section cut showing the location of the point of zero bending moment

Furthermore, by adjusting the step based on the displacement-force curve the point of zero bending moment can be found for both the elastic and the plastic deformations of the joint. In table 4.3 the results of this analysis is shown, and compared to EN1993-1-8, which predicts the point of zero bending moment to lie in the center of gravity of the bolt group.

Description	Distance from column face (mm)
EN1993-1-9	102.5
Elastic analysis	101.5
Plastic analysis	72.1

Table 4.3: Location of the point of zero bending moment

4.3.3. Parametric analyses of different joint geometries

In order the influence geometry has on the location of the point of zero bending moment, a set of joints was analysed in Abaqus using the same model. First, a design is chosen. Based on that design, a parametric study is carried out by altering certain parameters of the joint. For each iteration, the location of the point of zero bending moment is found using the process described in 4.3.2. For the sake of comparison, analysis of each joint design is also carried out using IDEA StatiCA. Furthermore, the results of the hand calculations according to EN1993-1-8:2005 as discussed in chapter 3 are computed. The results of this analysis are discussed in chapter 6.

Joint design

A joint design using a fin plate with two bolts was used. As discussed in chapter 3, the weld size is chosen such that the weld is full strength. For the column, fin plate and beam S355 is used. For the square tubing of the column, cold formed profile was chosen. The bolts are M24x8.8. Fitted bolts are used, thus $d_0 = d$. A gap of 10mm is kept between the beam and the column. The geometry of the fin plate is shown in 4.8

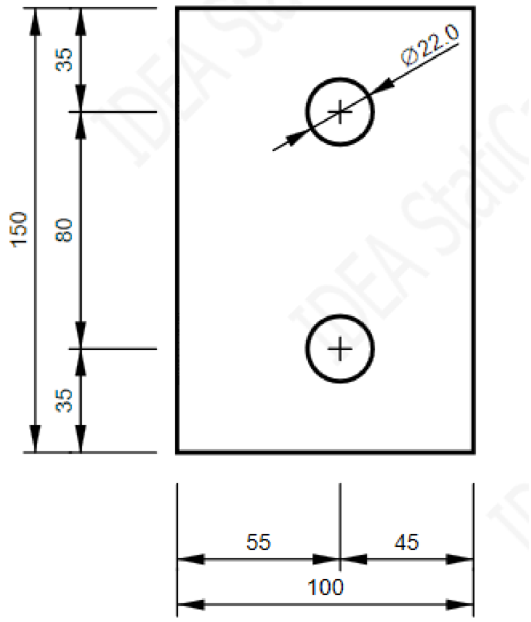


Figure 4.8: Fin plate geometry

Geometry set

For the parametric study, a set of profiles was chosen based on industry standards. Because the location of the point of zero bending moment is dependant on the stiffness of the column face, only the column profile was altered on each iteration. The set of beams that was chosen is shown in table 4.4.

SHS200x5	SHS200x6	SHS200x10	SHS200x12.5
SHS250x6	SHS250x8	SHS250x10	SHS250x12.5

Table 4.4: Column profile set for parametric study

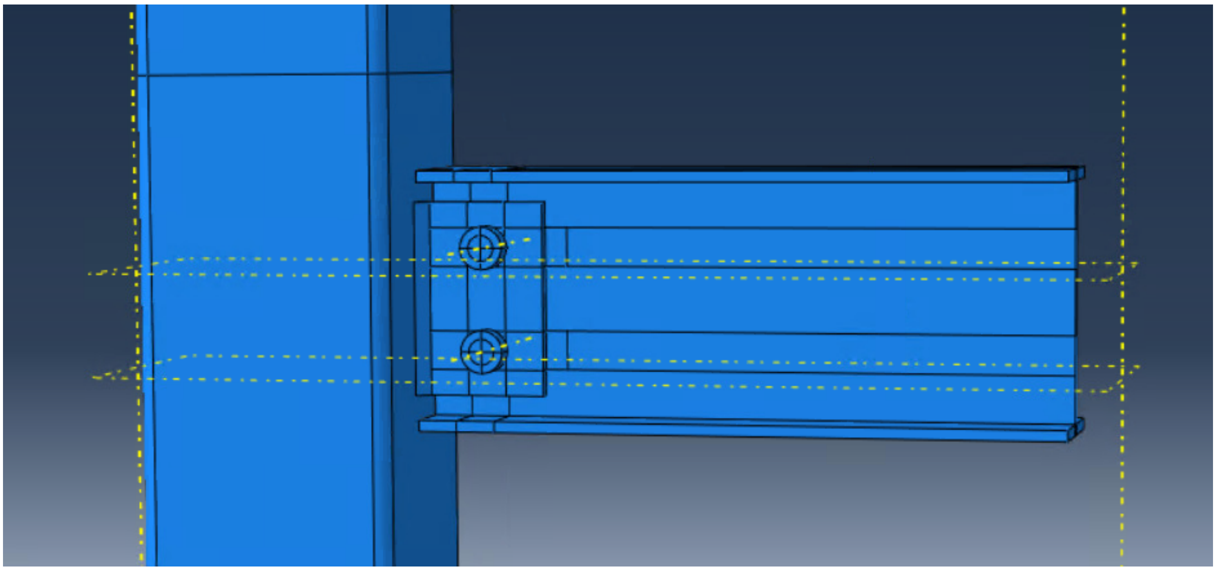


Figure 4.9: Abaqus model of the joint geometry

5
Results

In this chapter, the results of the numerical analysis are discussed. First, a comparison is made between the results obtained using Abaqus and those obtained using IDEA StatiCa. The predicted bearing resistance of the joints are given, and the failure mechanisms are discussed. Furthermore, the different bearing resistances to each failure mode resulting from calculations according to EN1993-1-8:2005 are presented. In 5.2 the location of the point of zero bending moment for each joint design is given.

5.1. Bearing capacity

In this section, the model results using Abaqus and IDEA StatiCa are discussed and compared. The bearing capacities of each joint found using both methods is given and compared to hand calculations.

5.1.1. Hand calculations

The hand calculations as discussed and demonstrated in 3 are again performed for this joint design. The bearing capacity found using hand calculations according to EN1993-1-8:2005 was found to be 135kN. In 5.1 the resistance of the joint against different failure modes is shown. For the bearing resistances, the new method of EN1993-1-8:2021 discussed in 3 was used. Bearing resistance of the bolt holes was found to be governing.

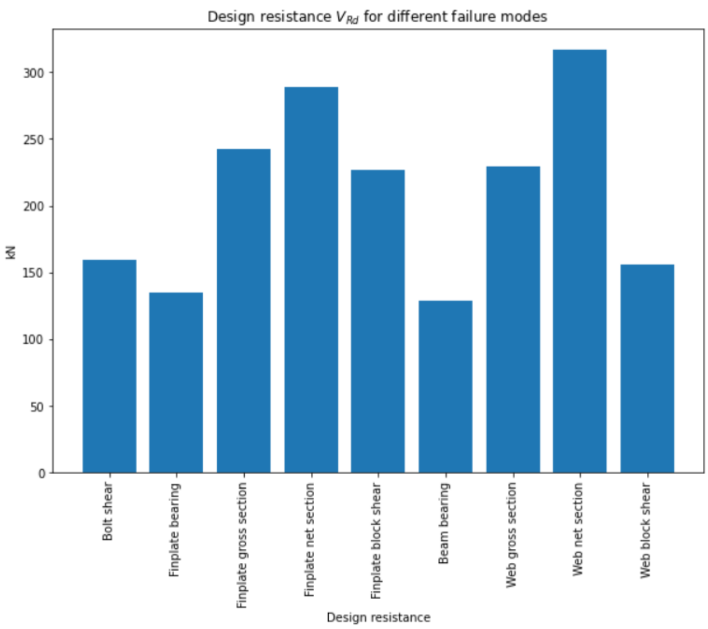


Figure 5.1: Resistance against different failure modes

IDEAStatiCa

Using the CBFEA of IDEAStatiCa, the flexing of the column face can be predicted. To demonstrate this, the results of the joint using SHS200x5 is shown in figure. The deformed shape shown in 5.2 shows high stresses and flexing at the column face. The plastic strain limit of 5% is reached in the face of the column. Thus in this case, the column face is governing for the resistance of the joint.

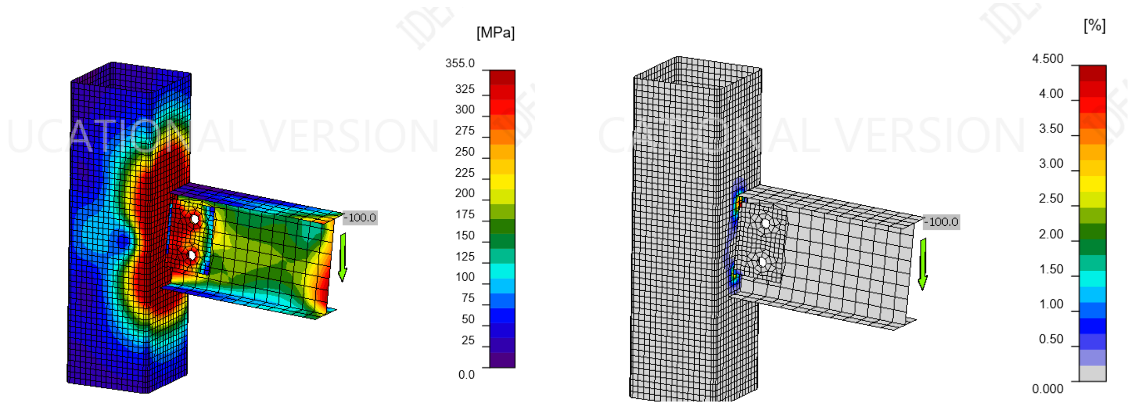


Figure 5.2: Stress and plastic strain of joint using SHS200x5 under load

Comparing these results to those of a design employing a column with higher stiffness such as SHS200x12.5 we find significantly lower stresses and no plastic strain in the column as shown in 5.3

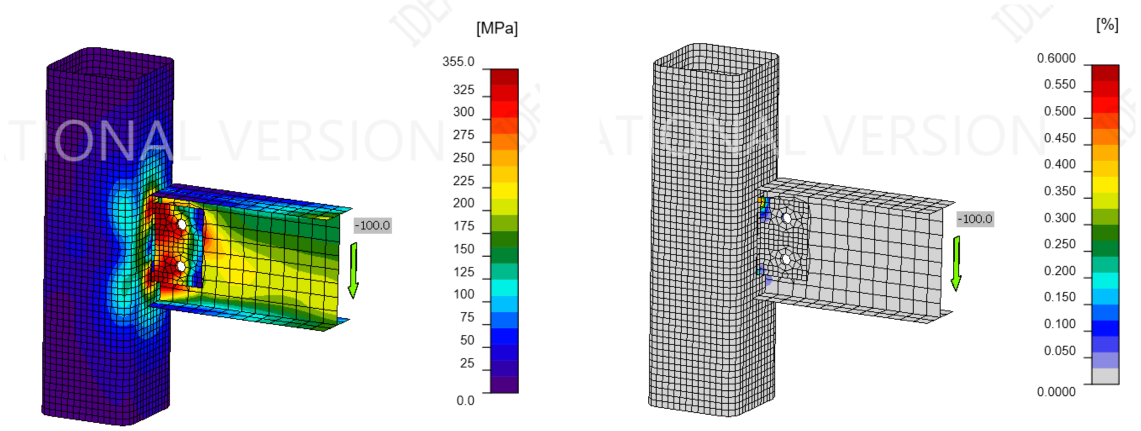


Figure 5.3: Stress and plastic strain of joint using SHS200x12.5 under load

Thus in this design flexibility of the column face is not governing.

Abaqus

Using the Abaqus model with an ideal plastic material model, a similar stress distribution is found. The plastic strain limit is reached in the column face, however some plastic deformations are also found in the fin plate and beam, suggesting some bearing is happening.

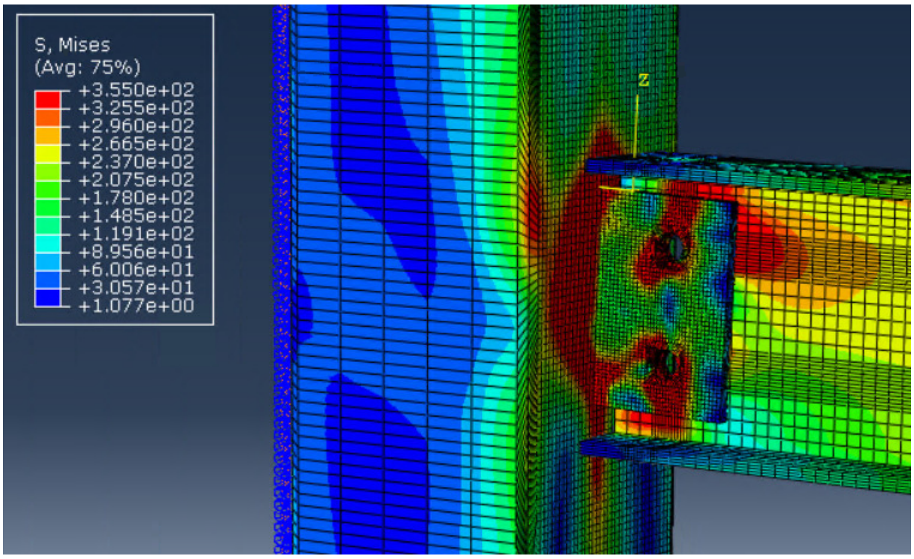


Figure 5.4: Stress distribution SHS200x5

Bearing capacity

In table 5.1 the predicted joint resistance using Abaqus and IDEA Statica is shown.

Column profile	IDEA StatiCa	Abaqus
SHS200x5	49kN	71kN
SHS200x6	88kN	122kN
SHS200x10	146kN	192kN
SHS200x12.5	146kN	194kN
SHS250x6	77kN	105kN
SHS250x8	109kN	178kN
SHS250x10	131kN	195kN
SHS250x12.5	147kN	199kN

Table 5.1: Comparison of bearing capacities found using Abaqus and IDEA Statica

5.2. Point of zero bending moment

In this section, the point of zero bending moment in the fin plate of the different joints is given. The method used is the same as described in 4.3.2. The method is also shown in figure 5.5, showing the near-zero point at 24.6mm from the column face. Because of numerical error, the bending moment found will never be exactly zero. The distances between the point of zero bending moment and the column face e_{zbm} are shown in table 5.2. In the third column the ratio between of the point zero bending moment to the total distance between the column face and the center of gravity of the bolt group $\frac{e_{zbm}}{e}$ is shown. For an infinetly stiff column, the ratio would be 1.0.

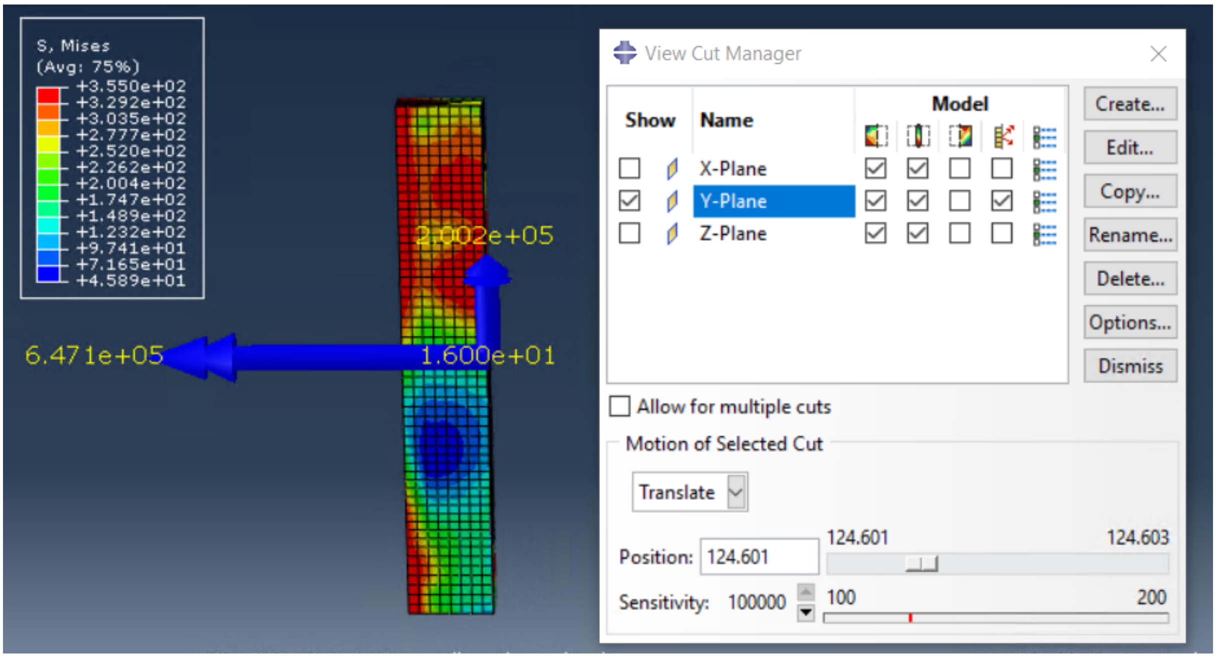


Figure 5.5: Point of zero bending moment using SHS200x5 column

Profile	Distance	Ratio
SHS200x5	25mm	0.45
SHS200x6	32mm	0.58
SHS200x10	46mm	0.83
SHS200x12.5	50mm	0.91
SHS250x6	26mm	0.47
SHS250x8	37mm	0.67
SHS250x10	49mm	0.89
SHS250x12.5	51mm	0.92

Table 5.2: Distance between the column face and the point of zero bending moment

6

Conclusions

In this chapter the results from the numerical and analytic calculations are discussed in detail. Several implications of the findings in the parametric study are discussed. Furthermore, a recommendation for further work is made.

6.0.1. Conclusions

Hand calculation provide a safe prediction for the bearing capacity of pinned connections using a fin plate. The degree of safety provided by the Eurocode is clearly demonstrated in the comparison between the bearing resistance found by hand calculations and those found using Abaqus.

When using closed section columns great care must be taken that the flexibility of the face of the column does not become governing. As demonstrated in chapter 5 for certain profiles, great overestimation of the bearing resistance of the joint can take place as the flexibility of the column face is not take into account.

For certain joint geometries, the flexibility of the face of the column is not a factor of concern in regards to the bearing capacity. In those cases, numerical analysis supports the bearing resistances predicted using hand calculations.

The new method for calculation the bearing capacity of bolt holes presented in EN1993-1-8:2021 yields results closer to those predicted using Finite Element Analysis.

When modelling a simple fin plate connection to a closed section hollow column, the assumption that the center of rotation is found in the face of the column as shown in figure 1.1is demonstrated to be inaccurate for the joint geometries chosen in chapter 4. Even with the most unfavourable column profile chosen, the point of zero bending moment was found at 45% of the distance between the column face and the center of gravity of the bolt group. Using the assumption shown in figure 1.1 will lead to underestimation of the moment transferred to the column, because the eccentricity of the true point of zero bending moment is not taken into account. To error on the side of caution, it would be safer to estimate the moment due to eccentricity the same way it is done for simple connections using open profiles following:

$$M = F(0.5h + e) \tag{6.1}$$

6.0.2. Recommendation for further work

Much work remains to be done on this topic. The process for determining the position of the point of zero bending moment laid out in this thesis yields interesting results. Further expansion of the parametric study however remains to be done. A limiting factor during the scope of this thesis was the shear computational effort required for the finite element analysis. Analysis of one joint requires roughly 3 hours using 8 processor cores requiring a license which has to be shared with the faculty. Given more modelling time, the parametric study could be expanded to include:

- A wider range of column thicknesses
- A wider range of column profiles
- Different joint geometries using more bolts and two boltrows
- The differences between cold and hot formed profiles. Hot formed profiles have smaller corner radii, so more flexibility of the column face is expected
- A different incremental increase of column thickness. By adhering not to only industry standards, but instead using a defined incremental step increase a more accurate (fitted) prediction of column face flexibility becomes possible
- An in depth analysis of the location of the point of zero bending moment during different phases of joint deformation (linear, plastic).

With such expansion of the parametric study formulas for the position of the point of zero bending moment could be derived. The location of the point of zero bending moment is dependant on the thickness and width of the column. The stiffness of the column face is dependant on the thickness and width of the column. However, because the radii of the column edges provide great stiffness to the face of the column, and almost all flexing occurs in the flat part of the column face it would seem that a formula using the column width minus two times the radius of the corners instead of the profile width itself could yield more accurate results.

Whilst several attempts were made to fit such a formulae to the results obtained within the scope of this thesis, an accurate function could not be obtained using the limited data points available.

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