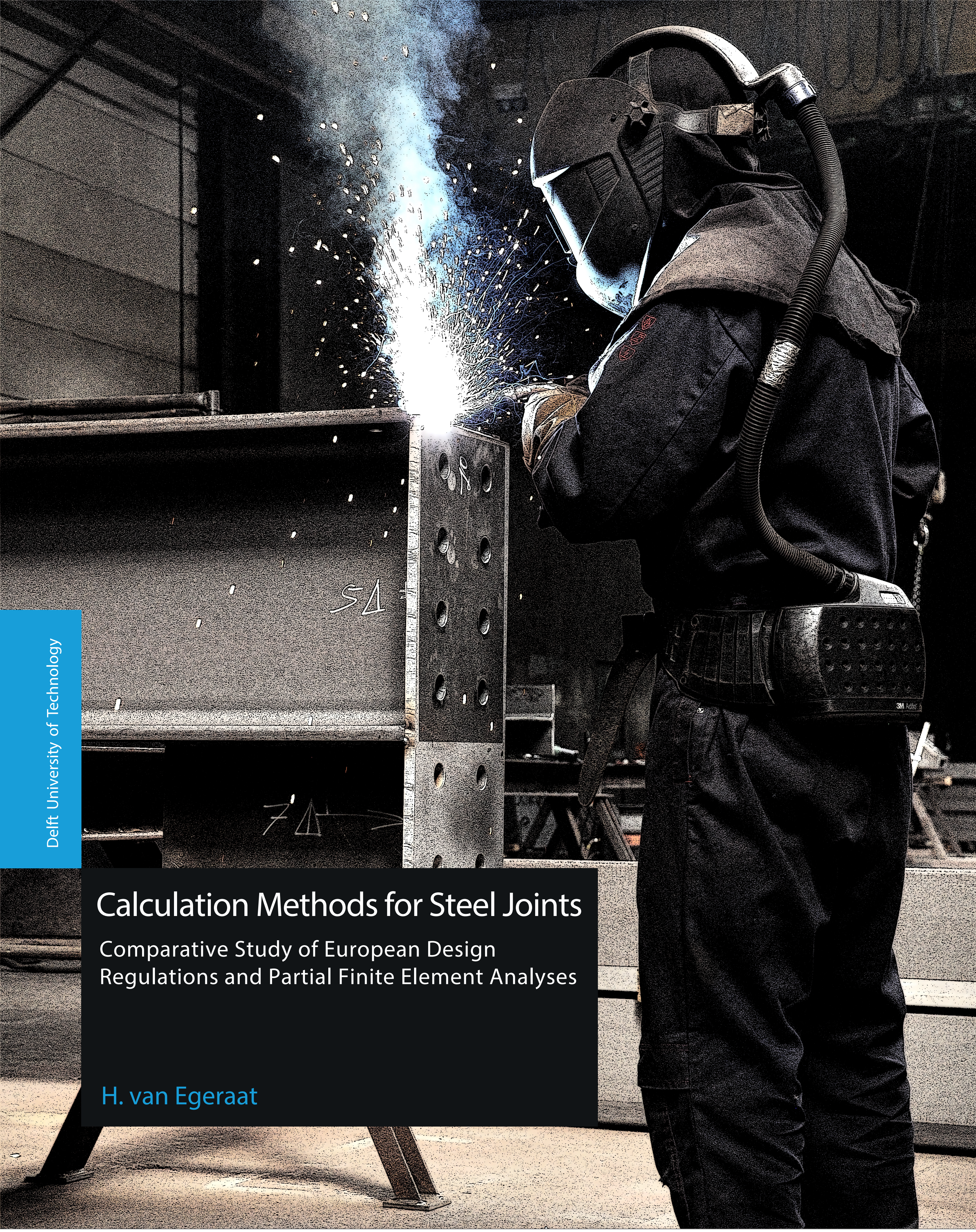


Delft University of Technology

Calculation Methods for Steel Joints

Comparative Study of European Design Regulations and Partial Finite Element Analyses

H. van Egeraat

TU Delft Delft University of Technology

Challenge the future

Calculation Methods for Steel Joints

Comparative study of European Design Regulations and partial Finite Element Analyses

Author:	H. van Egeraat	
Date:	June 2017	
Student number:	1510746	
Thesis committee:	Prof. Dr. M.V. Veljkovic	Delft University of Technology
	Ir. S. Pasterkamp	Delft University of Technology
	Dr. Ir. M.A.N. Hendriks	Delft University of Technology
	Ir. P.A. de Vries	Delft University of Technology
	Ir. L.J.M. Houben	Delft University of Technology
	Ing. T. van Lammeren	Hollandia Structures B.V.

Illustration on cover: *H. van Egeraat*, 5$^{\text{th}}$ April 2017;
Welder preparing a steel joint for ****** Project* (**client**) at Hollandia Structures B.V.

Calculation Methods for Steel Joints

Comparative study of European Design Regulations and partial Finite Element Analyses

THESIS submitted to the Faculty of Civil Engineering & Geosciences for the degree of MASTER OF SCIENCE in CIVIL ENGINEERING at DELFT UNIVERSITY OF TECHNOLOGY department STRUCTURAL ENGINEERING.

Canditate

Name: H. van Egeraat
Date of Birth: 1^{th} of October 1989
Student number: 1510746
E-mail: *************@gmail.com
Phone: +31 6 38******
Adress: Balthasar van der Polweg ***
2628ZH Delft
The Netherlands

Educational Institution

Delft University of Technology
Faculty of Civil Engineering & Geosciences
Department Structural Engineering
Stevinweg 1
2628CN Delft
The Netherlands

Company

Hollandia Structures B.V.
Markweg Zuid 1C
4794 SN
Heijningen

*"There are known knowns, things we know that we know;
and there are known unknowns, things that we know we don't know. But there are
also unknown unknowns, things we do not know we don't know."*

- Donald Rumsfeld, United States Secretary of Defense
Department of Defense news briefing on February the 12$^{\text{th}}$, 2002

Preface

This thesis brings an end to my academic time as a student at the Technical University Delft. Unlike most other students, I started my undergraduate studies with a practical bachelor education. This provided me with prior work experience at a construction site and at an engineering company. I concluded this first degree with a project for a company active in the chemical industry. My task was to conduct the initial design and the engineering of the main structural framework of a plant, in cooperation with an architectural and structural consultant.

The initial experience during my internship at an engineering firm instilled a growing interest in construction engineering. My structural drawings were transferred to a real structure at “Shell Moerdijk” (SNC Moerdijk) by the steel fabricator in cooperation with a concrete contractor. Because of this interest, I decided to extend my academic education with a Masters in Structural Engineering at the Technical University Delft.

An important task of a fabricator is the design, calculation and fabrication of the steel joints in the structure. I was surprised by the large amount of prescriptions for realisation of steel joints. In addition, I noticed that, after receiving the final design from the engineering contractor, a lot of detailed engineering and structural planning is still needed for the realisation of a structural framework on location.

Traditionally the detailed engineering of steel joints is calculated on the basis of calculation rules prescribed in the Eurocode. Because of the large amount of prescriptions, spreadsheets are developed and special purpose software is available to calculate the resistance of different types of steel joints, where some software developers are using the Finite Element Method to determine the stresses and strains in different components of the joints. A comparative study has been done between the traditional calculation method and this new method developed by one of these software developers. This report outlines and discusses the results of this research.

Hugo van Egeraat
May 2017

Acknowledgement

This research was supported by Hollandia Structures B.V. under the directions of ing. T. Van Lammeren. Hollandia Structures gave me the opportunity to conduct my research project, a core component of the Masters in Structural Engineering at the Technical University Delft. Importantly, I benefitted greatly from Hollandia's in-house expertise. I would like thank my direct colleagues from the Engineering Department for their support, and the engineers from the "IV-group" for answering my questions and their help with the FEM modelling process. At the Technical University, I would like to thank the thesis committee for their time and guidance during the graduation period. In particular, I am indebted to my principal supervisors, Ir. P. De Vries and Prof. Dr. M.V. Veljkovic, for their guidance, support and patience during the research process. Finally, I would like to thank my parents for stimulating me to conduct this Masters study as well as for their unrelenting financial support over the last years.

Contents

Abstract

Traditionally, steel joints are calculated by the calculation rules described in the Eurocode 3: NEN-EN 1993-1-8. Effective lengths are important parameters to determine the different resistances of the components in the steel joint. Finite Element Analyses (FEA) are becoming increasingly important in engineering, including in construction industry. Specialised software is developed to determine the stresses and corresponding strains in the plate elements of the joints by the Finite Element Method (FEM).

This thesis reports on a comparative study of the traditional calculation method and a method which is using partial FEA for determining the resistance of joints. The approach, assumptions and principles used for these calculations are explained in this report. It will be investigated whether the same components of a joint are decisive for each method and if there are differences in joint resistances. If so, the magnitude of the difference will be determined as well. This is done for different types of simple shear joints (SSJ) and different moment resisting joints (MRJ). For the last group, two joints configurations (Flush End Plate Joint and Extended End Plate Joint) are calculated manually, partially modelled with the FEM, and compared with the results of executed experiments.

The report first discusses and explains commonly used steel joints in structural steel projects. It presents the results of a review of the extant literature on European calculation prescriptions which are relevant for steel construction industry and specific for the calculation of steel joints (Eurocode 3 series and the Green Books). Another part of this research is to investigate what information is available about theory of steel joints and what principles are used for manual calculation. Reports of executed experiments are analysed and these are used in this report for validating the manual calculations and the partial FEA models.

The first element of the research project was orientating market research for software developers. The starting point was a visit to the "Staalbouwdag 2016 on the 11^{th} of October 2016 at "Kromhouthal aan het IJ" in Amsterdam. Staff at the stands of the different software developers at this event were interviewed. Several software programmes were downloaded with an educational license and tested on functionality. It was decided to continue the research with the software programme IDEA StatiCa Connection, a programme especially developed for the calculation of steel joints with the newly developed calculation approach: Component Based Finite Element Method (CBFEM). This method combines the use of FEM and the calculation rules from the Eurocode 3.

The first part of the comparative involved an examination of three types of simple shear joints: 1) Fin Plate Joint; 2) Short End Plate Joint; 3) Double Angle Cleat Joint. Higher resistance values are obtained by the partial FEA models. For manual calculation, as for FEA modelling it is very important what assumptions are made before calculation. In particular the location of the rotation centre drastically influences the outcome of the calculation.

After this, five moment resisting joints are evaluated. First, two bolted extended end plate joints of which one is symmetrically loaded, and the other a-symmetrically loaded. Also an a-symmetrically loaded welded moment resisting joint is evaluated. Higher bending moment resistances can be obtained by the partial FEA models which are a-symmetrical loaded. In manual calculation rules reductions factors must be applied which significantly reduces the resistances of several components of the joints. In contrast to a symmetrical load case, also the shearing of the column should be taken into account. The FEA models show straining in the column web of the joint due to shear deformation. In the symmetrically loaded joint stress concentration in the tension- and compression zone can be noticed. The total bending moment resistance in the FEA is similar to the outcome of the manual calculation.

Two types of moment resisting joints (Flush End Plate Joint and Extended End Plate Joint) were tested during an experimental programme. Configuration of these joints, depicted in the technical drawings in appendix D, are calculated manually and modelled. First, manual calculations are executed, with the experimental values (EV) obtained from tensile test. No partial factors are used for these calculations. FEA models are made on the basis of the experimental values, using bi-linear stress-strain relation to mimic the material behaviour. Second, the same manual calculations are executed, and same models are made, but now based on the design values described by the Eurocode 3, with the use of partial safety factors according to the Dutch National Annex.

For the calculation methods based on the experimental values holds that for both joints the manual calculation is most conservative. Results obtained by the partial FEA slightly surpass the resistance values from the experiments, but they remain in the same order of magnitude. For the calculation methods based on the design values of the EC3 with partial safety factors, a trend can be noticed. The manual calculation is most conservative, followed by the resistance of the FEA. The outputs of both methods stay under the resistance values obtained by experiments.

In general, it can be concluded that FEA models give higher resistance values than manual calculations based on the Eurocode 3. Exceptions to this are those cases where bolts are the governing components in the joints and where the joint is symmetrically loaded. For the flush end plate and the extended end plate moment resisting joint it holds that, using the prescription from the Eurocode 3, both the manual calculation and the FEA model remain under the resistance values which are shown by experiments.

Exact resistance values of the described manual calculations, FEA models and experiments, including graphical representations, are discussed in the conclusion. All executed manual calculation are included in Appendix A and complete elaborations are included on the USB Flash Drive. MATLAB scripts are presented in Appendix B. The FEA models are presented in Appendix C and on the enclosed USB Flash Drive.

This comparative study has been executed for specific types of simple shear joints and moment resisting joints. Future research may focus on examining other types of commonly used joints in structural projects, for example column/beam splices, plated joints and/or three dimensional joints.

Chapter 1

Introduction

These days, in 2017, steel (along with reinforced concrete) is still the most important construction material for buildings. A substantial number of parties are involved in steel construction projects to deliver the structural framework to the client. One of these parties is the steel fabricator whoses task is to connect the structural members of a steel framework by reliable joints. For this reason, the fabricator is responsible for the design, calculation and fabrication of these joints.

1.1 Hollandia Structures B.V.

Hollandia Structures B.V. (from hereon Hollandia) is a steel fabricator specialised in steel structures for non-residential buildings and the (petro)chemical sector. Standardized structural elements from the steel fabricator are transported to the factory. The client, commonly with the help of an engineering contractor, delivers the engineering package for construction. The task of Hollandia is to translate the engineering package into a complete steel construction on site. Hollandia does this in cooperation with Everest Montage, the party responsible for mounting the structure.

1.2 Information Technology in Structural Engineering

A large number of software companies develop applications for the Building Engineering and Structural Engineering sector to support architects, engineers, work preparators and logistics personnel. The available software packages help calculators with developing competitive proposals for tenders and to control the cost of man-hours, materials and other factors during construction projects.

The software applications are used in different niches of the construction industry. The sheer number of new software packages being released suggests that it is still a lucrative business. Apart from bringing out new features, an increasingly important issue concerns compatibility to other software packages.

For structural applications, software is developed to support engineers in the calculation of structural frameworks and connections. Finite Element Analysis (FEA) software is becoming increasingly popular in the engineering industry. Where before standards were used, (In the Netherlands, formerly the TGB standards and now the Eurocode with a specific national annex) now these calculation rules make place for sophisticated (combined) finite element analysis software. If calculations following the standards are clearly presented with references the used articles in the code , those calculations are auditable. The question is how to assess these new calculations with FEA software.

1.3 Motivation for the Research

IT has drastically changed the way of working in the steel (fabrication) industry. Tekla is used nowadays to model complete steel structures from where work drawings can be made for fabrication. The calculation of steel connections are based on the Eurocode and automated with spreadsheets. Alternatively, special purpose software is used.

There is a trend of architects coming up with increasingly special and prestigious designs. Their design software portfolio is developing too, which provides them the opportunity to create specially shaped designs. As a consequence for the structural designer, it is an increasing challenge to design an appropriate structural system, which can result in special non-standard steel frameworks with a diversity of special joints.

Finite element modelling (FEM) is more frequently used method to predict the behaviour of materials and to calculate the stresses and strains in materials. Traditional steel joints are calculated manually, with the use of design regulations. In most European states the Eurocodes are used. Applied loads are determined from the equilibrium conditions and resistances of the components are determined on the basis of the articles in the Eurocodes. Nowadays software is developed with (partial) finite element modelling which incorporates assumptions and procedures that are different from traditional methods. This thesis compares the calculation methods and evaluates the final results.

In a highly competitive market, for example the steel fabrication industry, it is important to be efficient. Necessary is to keep the way of working up to date. New release of software packages can ease the workload and bring more possibilities to evaluate design parts. Important is to check whether these programmes give reliable results. Further, do these programmes give new possibilities? Are there any limitations? Main focus on this thesis is to determine whether the software gives reliable results by comparing them with European design regulations and experimental values. Thesis has been written for the management of Hollandia. This thesis will help them to asses the added value of (a) new software release(s) and can be used as a basis for the decision to eventually purchase the software.

1.4 Research Question

In this thesis different types of simple shear joints and moment resisting joints are calculated according the Eurocode and modelled in a partial finite element analysis. The main research question is:

"For simple shear joints (SSJ) and moment resisting joints (MRJ), what are differences in calculation procedures between European design regulations and partial finite element analyses."

This main question is divided into three sub-questions:

Subquestion 1: *"What are relevant components and failure modes of SSJ's and MRJ's that need to be considered"*

Subquestion 2: *"What are the differences in approach by calculating traditionally according the Eurocode 3 and partial FEM modelling and what is the quantitative difference in resistance values for both methods?"*

Subquestion 3: *"Which components of those steel joints are governing, are the same components governing for each method?"*

1.5 Approach of the Comparative Study

The first chapters deal with different types of joints, theory of joints, design regulations and results of experiments on steel joints.

After, from chapter 6, in total eight different types of joints will be assessed with different configurations, three simple shear Joints (SSJ) :

CA1: SSJ, Fin Plate
CA1-V: SSJ, Fin Plate Variant
CA2: SSJ, Short End Plate
CA3: SSJ, Double Angle Cleats

Five moment resisting joints (MRJ) are assessed with different load situations, two of which are tested in a test programme of the University of Coimbra.

CA4: MRJ, Extended Endplate Joint symmetrical loading
CA5: MRJ, Extended Endplate Joint unsymmetrical loading
CA6: MRJ, Welded Joint
CA11: MRJ, Flush End Plate Joint
CA12: MRJ, Extended End Plate Joint

The above enumerated calculations of steel joints are:

1. Calculated manually, according to the Eurocode 3 (EC3)
2. Programmed and graphically presented in MATLAB.
3. Modelled with FEA software.

Finally parametric study has been executed for different types of simple shear joints and moment resisting joints. Totally three parametric studies have been executed. The parameters of these joints are changed and results of the partial FEA models and Eurocode calculations are compared to each other.

1. PS1: Fin Plate Joint
2. PS2: Short End Plate Joint
3. PS3: Moment Resisting Joint

Elaboration of the manual calculations, MATLAB script files and reports of the FEA models can be found in: Annex A, Manual Calculation; Annex B, MATLAB Scripts; Annex C, FEA Models. Technical drawings of the Flush End Plate Joint (CA11) and Extended End Plate Joint (CA12), together with the calculated component resistances, can be found in Annex D, Technical Drawings. Also the joint variants of the parametric study are attached to this Annex.

1.6 Reading Guide

This section outlines the structure of the report and the topics addressed in each chapter.

Chapter 2, *Joints in General*: This introductory chapter explains the different methods of connecting steel members. In addition, the most commonly fabricated joints will be mentioned. Based on internal documentation, it determines the share of the costs of engineering, preparation and fabrication of joints in steel construction projects.

Chapter 3, *Design Regulations*: This Chapter reports the first part of the literature research. It explores the relevant prescriptions for the the design of structural joints.

Chapter 4, *Theory and Experiments*: This Chapter reports on the second part of the literature research. It focuses on the theory of joints, notably: The difference in modelling and behaviour of rigid joints and nominally pinned joints. In addition, two experimental reports of tests on moment resisting steel joints will be analysed.

Chapter 5, *Software and Calculation Procedures*: The first part of this chapter describes the results of the market research. It is decided to conduct this study with the software programme IDEA StatiCa Connection. The principles and the calculation procedure of this program are explained. A comparison is made between the manual component method and the method, used by the software by using partly the FEM.

Chapter 6, *Simple Shear Joints*: This chapter is the start of the comparative study. It begins with an explanation of the general procedure for calculating a simple shear joint. Following this, three types op simple shear joints are evaluated: 1) Fin Plate Joint; 2) Simple Shear Joint; 3) Double Angle Cleat Joint.

Chapter 7, *Moment Resisting Joints*: This chapter reports on the second part of the comparative study. The general procedure for calculating the moment resisting joint is explained. Five manual calculations are executed on different type of joints with different load cases. In addition, five FEA models are made. Finally, two calculations are compared with the results of experiments.

Chapter 8, *Parametric Study*: In this chapter parameters are changed for the fin plate joint and the short end plate joint. First a standard model is made, which is loaded till the first failure mechanism occurred. The component which was failing will be strengthened and will be loaded again till failure. The standard configuration and strengthened configuration will also be calculated manually using the developed MATLAB scripts files.

Chapter 9, *Conclusion*: This final chapter summarizes and discussing the results from the comparative study. The results of the manual calculation, FEA models and experiments will be compared.

Appendices: The appendices include a presentation of the before mentioned manual calculations along with MATLAB scripts, FEA models and Technical Drawings. Some of these are attached as a hardcopy to this report. Other elements can be accessed via the enclosed USB flash drive.

Chapter 2

Joints in General

In most cases the engineering company is responsible for structural design of the structure. The engineering contractor has worked out a structural framework to support the building which is worked out by the architect. The contractor has determined the applied loads, designed the structural beams and checked them. The design is publicly tendered or directly awarded to a construction party for construction.

2.1 Methods of Connecting

The steel contractor is responsible for the realisation of the structure as well for the design, calculation and fabrication of the connections. The engineering contractor determines the classification of the connection- rigid, nominally pinned or something in between. In general the following possibilities are available to connect steel members:

- Bolting,
- Welding,
- Gluing,
- Seaming,
- Riveting,
- Larssen Connection.

Bolting and welding, or a combination of both, are commonly used methods to connect structural members in the steel fabrication industry. There is also a possibility of gluing steel, but this is not done often because of the unpredictability over time. Besides there are no norms available to calculate the strength behaviour during the design life of the structure. Seaming is a metalworking process that is frequently used for connecting steel cladding. It is also used in the food processing industry. Larssen connection can be found in hydraulic engineering and geo-technical engineering. Sheet piles are equipped with a Larssen Connection. This connection is able to transfer horizontal tensile and compression forces.

An old type of connecting which is no longer used is riveting. Riveting is a mechanical method of connecting steel members/plates. A rivet has a cylindrical shaft with a head and a tail. During installation the rivets are heated up and brought into the rivet holes. The tail of the rivet is smashed, creating another head . Cooling of the rivet causes shrinkage. Due to shrinkage of the rivet a tensile force in the rivet occurs exerting compression forces between the plates, creating a seamless (water)tight connection . Because of the hammering of the rivet the tolerance space will be filled up by plastic deformation of the bolt. For this reason, riveting was not only used in construction, but also applied in shipbuilding. Riveting was a frequently used method to connect structural elements, but it was labour-intensive. With the advent of electric welding around 1930, rivetting became outdated and is now only used for renovation and architectural purposes. [26]

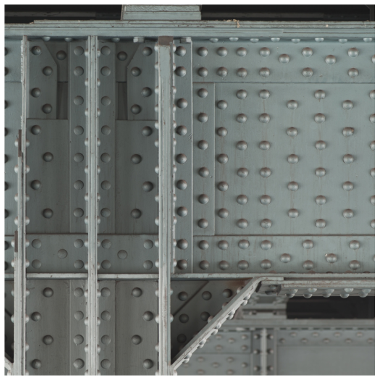


Figure 2.1: Riveted connection

For this reason the Eurocode still offers design regulations for rivets. These are comparable with the design checks for bolts. Most checks are similar, but for shear resistance and tension resistance differing prescriptions exist. Check for punching resistance is not needed. Equations are taken from NEN-EN 1993-1-8, Table 3.4.

Shear resistance for rivets:

$$F_{v,Rd} = \frac{0.6 * f_{ur} * A_0}{\gamma_{M2}} \tag{2.1}$$

Tension resistance for rivets:

$$F_{t,Rd} = \frac{0.6 * f_{ur} * A_0}{\gamma_{M2}} \tag{2.2}$$

2.2 Bolted Connections

Bolted and welded connections are commonly used in the steel industry. In the Eurocode 3, design of steel structures, a special norm, NEN-EN 1993-1-8 is dedicated to the design and calculation of steel connections.

The idea of a bolted connection is simple: two members with are parallel surface are placed on each other. Aligned holes with a small tolerance are made in the steel. Tolerances must be the same and not to large, all bolts in the joint must be able to contribute to the total resistance of the joint. Preferably large bolts are used with relative thin plates to activate all bolts by deformation. Brittle fracture as decisive failure mode of a joint is highly undesirable. A steel pin is put in the holes and is secured by a nut. Sometimes a washer is added for the spreading of the compression force, protection of the material during fastening or to prevent loosening of the bolt due to vibrations in the structure.

Bolts are available in different classes which all have a different nominal and design value [15]. The following values are included in the Eurocode.

Table 2.1: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt Class	**4.6**	**4.8**	**5.6**	**5.8**	**6.8**	**8.8**	**10.9**
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

Bolted joints are preferred over welded joints from economical point of view of the steel contractor. Most economical way to connect is to prefabricate and prepare the joint in the fabrication shop. Structural members are transported to site where the erection starts. Bolted connections have the benefit that they can be mounted on site and are more able to deal with (small) measurement deviations than welded connections. In populous areas pre-assembly is preferred because lack of working space, limited work time or the undesired presence of many lifting movements.

2.3 Welded Connections

Welding is a metallurgic process which connects steel members by heat input or under pressure [20]. Multiple welding techniques are available. Arc welding is the most used technique. The mother material is heated until liquefied and welding consumable is added during the process. After the heat input the weld solidifies and will obtain a strength larger than the mother material for normal steel grades. Double fillet welds are mostly executed, but can also be executed singular or as intermittent welds.

At Hollandia gas metal arc welding (GMAW) is used for fabrication. Metal inert gas / metal active gas (MIG/MAG) arc welding is a method of welding with a melting electrode under protection of an inert gas (e.g. Argon) or an active gas (carbon dioxide).

Benefits of MAG-welding are:

- No slag arises on the weld
- Limited heat input
- High production rate
- Continuous welding
- Good protection of the weld

Welds can be executed in different shapes depending on the pre fabrication of the members.

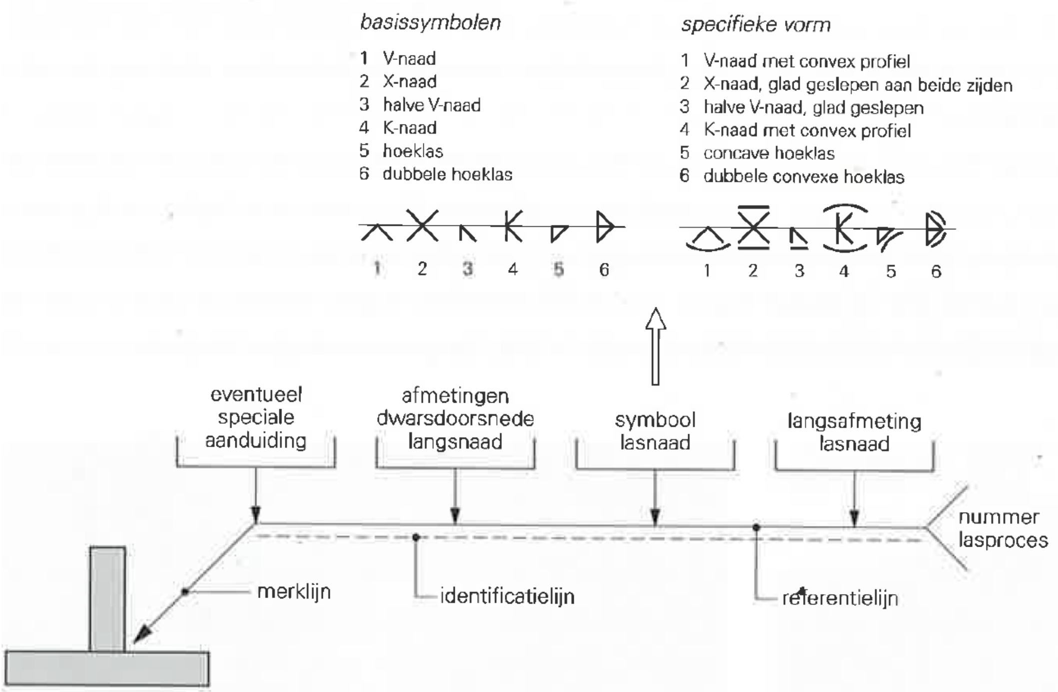


Figure 2.2: Symbolic representation of weld types on technical drawings [21]

Welding is a relatively expensive and labour-intensive process. Welds should have a sufficient throat thickness, but should not be oversized for economical reasons. Depending on the throat thickness, the amount of weld runs can be determined, figure 2.2. For large welds it is favourable to apply pre preparation of the weld plates, and to apply V-welds, K-welds or X-welds. From a throat thickness of $a = 12$ it is more economical to use pre preparation. If welding is required it should be executed inside the fabrication shop as much as possible. Conditions outside can hinder the welding process (for example rain, wind and frost). The accessibility is lower and ability for inspection is less.

2.4 Types of Steel Joint

Different types of commonly used steel joints are explained in this subchapter. Later on this report the main focus will lie on simple shear connections and moment resisting connections in a beam to column configuration.

2.4.1 Column Splice

The column splice together with the beam splice is the simplest joint. A splice, if applied to a column, is a lengthening of a column with butt welds or can be bolted with plates. Column splices are used to lengthen the column, because of maximum standard production dimensions, maximum transport dimensions or because it is not possible to deliver large columns to the construction site. Column splices can also be used to connect different types of columns to each other. The difference in dimensions can be filled with shim plates.

Splices must be able to provide sufficient strength and continuity of stiffness. Two types of column splices are classified depending on the way they transfer the load:

- Bearing type
- Non bearing type

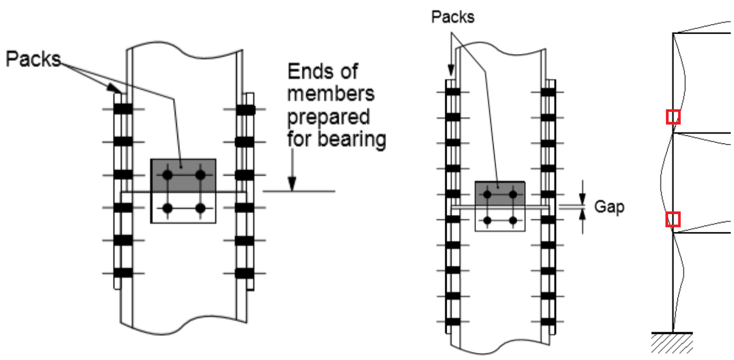


Figure 2.3: Bearing splice (left) and non bearing splice (middle), favourable position to apply a column splice [23]

Bearing is a type of joint which transfers the load directly through bearing. No gap is present between the individual. The NEN-EN 1993-1-8, art. 6.2.7.1(12) states that at least 25% of the load should be transferred by the plates and bolts. [23] Non bearing type splice is transferring all the loads through plates and bolts. A gap is present between the the structural members.
Best place to design a column splice is near the floors for mounting. This location is also preferred because it is away from buckling sensitive zone. To apply a bolted splice holes are drilled into the flanges and web, which causes a weakness in the cross-section.

2.4.2 Beam Splice

The beam splice is comparable with the column splice, difference is the lengthening in horizontal directions instead of the vertical. A beam splice must resist the co-existing design moment, axial force and shear in the beam by the web cover plates and the flange cover plates. [3]

To obtain a rigid joint, pre-tensioned bolts should be applied (Category B,C of table 3.2 of NEN-EN 1993-1-8). The strength of the pre tensioned bolts is depended on the friction coefficient which is related to the type of surface described in the NEN-EN 1993-1-8, Table 18 [13].

$$F_{s,Rd} = \frac{k_s * n * \mu}{\gamma_{M3}} * F_{p,c} \tag{2.3}$$

$$F_{p,c} = 0.7 * f_{ub} * A_s \tag{2.4}$$

For an economic design a beam splice should be designed in the vicinity of a support a the position with minimized internal forces. In a design of a bolted splice the plates on the flanges are designed to transfer the bending moment. The plate of the web is meant to transfer the shear force.

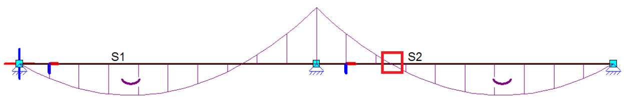


Figure 2.4: Most economical point to apply a beam splice

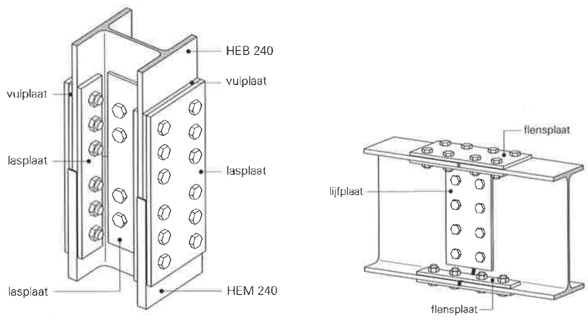


Figure 2.5: Example of a bolted column splice and a beam splice. [21]

2.4.3 Column-Beam Joint

The column-beam connection is a commonly used connection. Column-beam joints can be modelled in different ways: as a moment resisting joint, a shear joint or something in between.

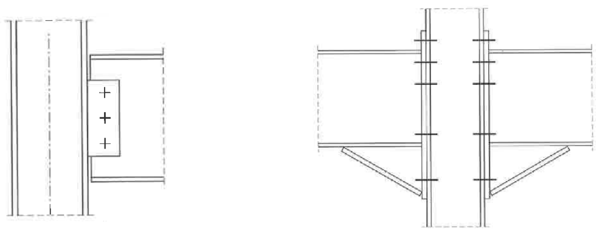


Figure 2.6: Example of a shear joint (left); example of a moment resisting joint. [21]

2.4.4 Beam-Beam Joint

Beam-beam joints or beam-girder connections are commonly used joints. This connection type is used to connect the primary/main beams to the secondary/supporting beams. These types are commonly designed as simple shear joints.

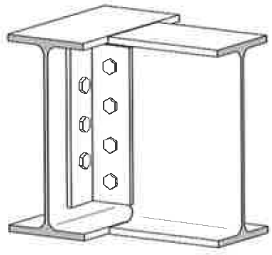


Figure 2.7: Example of a beam to beam joint. [21]

2.4.5 Base Plate Joint

A baseplate joint is a steel-concrete (composite) joint. It can be executed as hinged or moment resisting (Figure 2.9) [4]. The baseplate is designed to spread the concentrated stresses from the structural steel section trough the base plate (welding and/or contact bearing) finally to the concrete foundation. Stress reduction is needed because steel has a higher resisting stress value than concrete. The base plate must resist an axial force, shear force and bending moment, if executed as a moment resisting joint. If it is expected that the column will be subjected to large shear forces, shear keys will be attached the the joint configuration.

Depending on the load combinations the connection must be tested on tensile resistance, compressive resistance or a combination. Which also results in different leverage arms for determination of the moment resistance $M_{J,Rd}$.

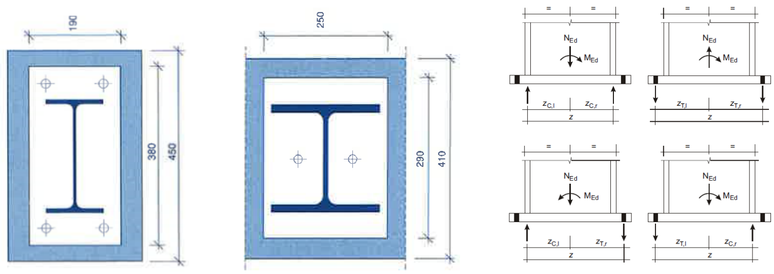


Figure 2.8: Example of baseplate joint executed as a moment resisting joint (left), executed as a hinged joint (middle), combination of loadings on a baseplate (right) [4]

2.4.6 Bracing Joint

Bracing joints are applied to guarantee the structural stability. The function of these diagonals are to guide the horizontal forces to the foundation. In many cases angle cleats are used as bracing. The braces are preferably positioned under an angle of about 45°. They are needed to resist the forces from wind loading and initial tilt. They may also resist tensile (and compression forces), depending if the bracing is executed singularly or double.

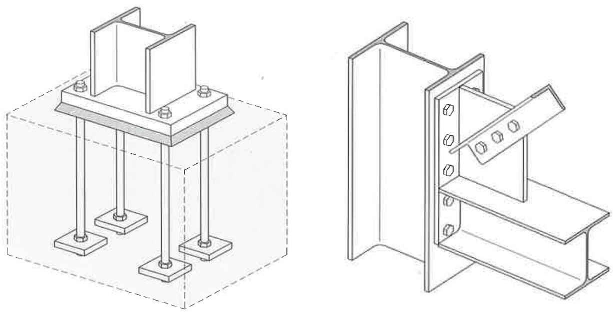


Figure 2.9: Example of a base plate joint; example of a bracing joint. [21]

2.5 Cost of joints in steel construction projects

This thesis focuses on the methods and calculation procedures for the design of joints. The cost of fabrication of these connections is a considerable part of the total construction costs of a steel structures. Globally main cost of steel structure projects are: Materials, fabrication, engineering, conservation, mounting, transportation and rental of equipment, see figure 2.10.

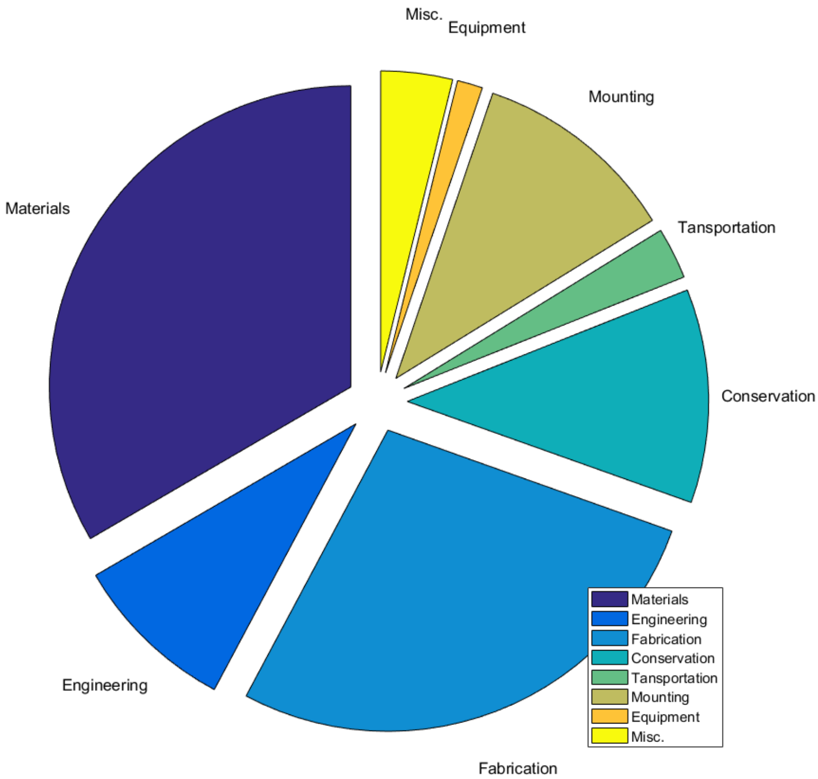


Figure 2.10: Overview total cost of steel construction project. [9]

Cost for fabrication of steel joints is not only depending on the cost of material and fabrication hours. Also time spent by engineers on the design, calculation and making detailed drawings for fabrication contribute to this expense. In additions, the connections, in particular the welds need to be checked systematically. All welds need to be checked visual and some parts by Magnetical Particle Inspection (MPI) and/or Ultrasonic Testing (UT). Those types of inspection are examples of non-destructive testing (NDT). The percentage of the welds that need by checked by NDT is decided before the start of the project.

Smart designs of steel joints can speed up the construction process in erection phase. Therefore, improved know how about design, calculation and fabrication may result in more economical design of joints and may reduce the overall realisation costs. Graphs presented in this section are based on an estimation of a regular project with predominantly simple shear joints. The complexity of the design strongly influences the overall expenses on joints.

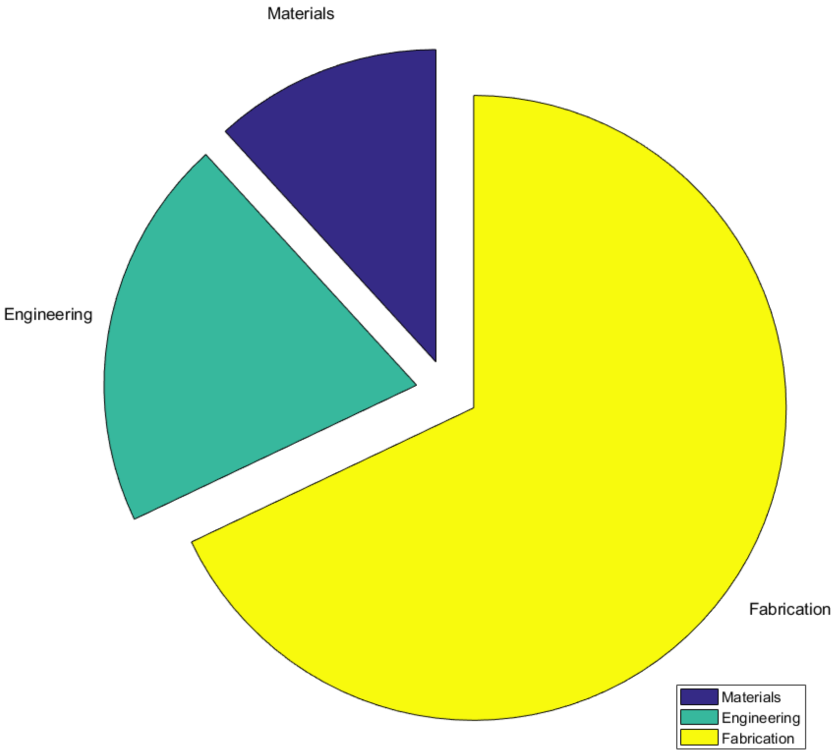


Figure 2.11: Global overview of the different expenses for realisation of joints. [9]

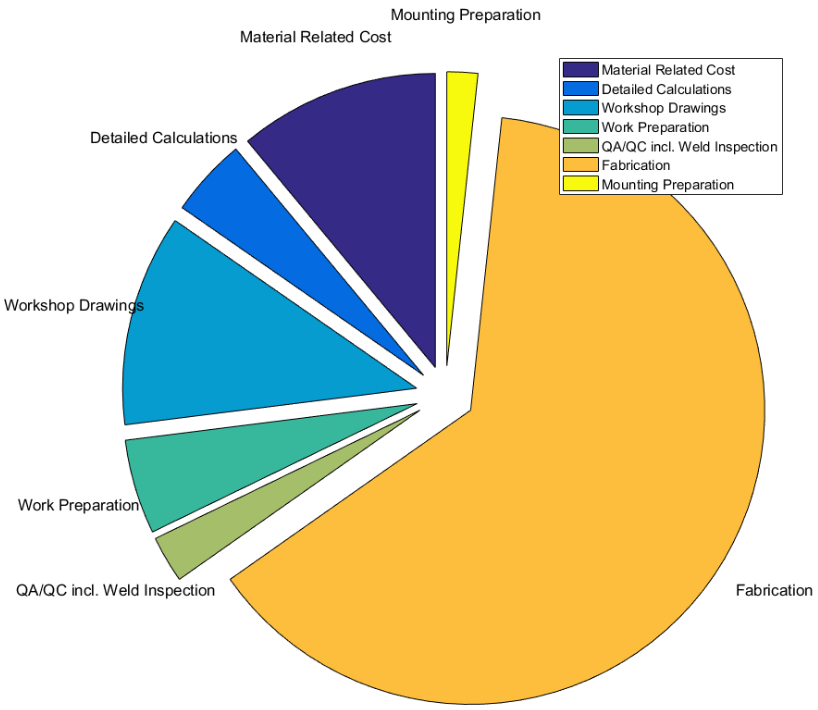


Figure 2.12: Specific overview of the different expenses for realisation of joints. [9]

Chapter 3

Design Regulations

3.1 Standardization

Standardisation is the process of implementing and developing technical standards. Not only for the construction industry norms are developed, but for many other market segments. In essential norms are established between stakeholders operating in the same market segment. Standards are not legally binding, but legislation or contracts may refer to certain norms and standards. Purpose of standardisation is to secure the quality and safety of services, processes and products. [12]

Norms have been developed for engineers proving that their structure is safe and satisfies the requirements. In the Netherlands the "Nederlands Normalisatie-Instituut" (NNI) is responsible for setting norms and is responsible for the publication of, not only Dutch standards, but also some international standards. The NNI is responsible for the administration and publication of the following standards:

- NEN - norms (NEderlandse Norm)
- EN - norms (European Norm)
- ISO - norms (Norm of the International Organisation for Standardisation)
- IEC - norms (Norm of the International Electrical Commission)

The CIDECT (International Committee for Research and Technical Support for Hollow Section Structures) is originally responsible for the development and publication of design recommendation for hollow section joints. Their findings are included in the Eurocode 3 and can be found in NEN-EN 1993-1-8, chapter 7. [10]

3.1.1 Eurocode

Before the Eurocode different NEN norms from the TGB series (NEN6700 up to and including NEN6790) were available and used for construction projects. With the development of the "Bouwbesluit 2012" the Dutch NEN-norms will be replaced by the Eurocode. On the 1st of April the "Bouwbesluit" was entered in force the following Eurocodes became mandatory for the engineering and construction of structures:

Eurocode 0:	Basis of Structural Design
Eurocode 1:	Actions of structures
Eurocode 2:	Design of concrete structures
Eurocode 3:	Design of steel structures
Eurocode 4:	Design of composite steel and concrete structures
Eurocode 5:	Design of timber structures
Eurocode 6:	Design of masonry structures
Eurocode 7:	Geotechnical design
Eurocode 8:	Design of structures for earthquake resistance
Eurocode 9:	Design of aluminium structures

Basis of the Eurocodes is the Eurocode 0. In the Eurocode 0 : NEN-EN 1990, Basis of structural design.[14] In this code basic principles are explained for the structural design. For example: Basic requirements, uses of limit states, combination of loads on structure and usage of partial safety factors.

For designing steel structures, the Eurocode 3 is of interest. The Eurocode 3, which is coded as NEN-EN 1993, consist of multiple parts. The NEN-EN 1993-1 series contains general rules for designing steel structures. For the design of steel-connections the **NEN-EN 1993-1-8: Design of joints** is developed. The NEN-EN 1993-2 up to and including 1993-6 specifically deal with the design of structures such as towers, chimneys, silos, tanks and crane supports. The NEN 6772 and NEN 2062, the norms which previously were used for designing steel connections, are to be replaced by the NEN-EN 1993-1-8. But, in some cases, it can occur that the Eurocode still refers to the NEN norms.

All members states of the European Union are forced to withdraw their national codes and implement the Eurocode. Main reasons why the EU decided to implement the universal construction regulations for all EU states are [11]:

- The removal of technical barriers between the EU member states.
- Eurocodes form the basis of a uniformly representation of the strength properties of building products with a CE marking.
- The Eurocodes are designated in European Tender Guides as the basis of structural calculations for extensive projects.

Members states are still able to determine their own national safety level. Every EU state has the possibility to determine their own Nationally Determined Parameters

(NDP's) and may do external determinations which are all described in the national annex (NA) which is added after every national version of the code.

Although the use of the Eurocode is only mandatory to use for EU states. Some non EU states, Singapore and Vietnam, use the Eurocode as standard on voluntary basis. More states with developing industries are interested in to use the Eurocode as well.

Although at the moment every country has the possibility to deviate from to code with their NDP's and external national determinations, the aim is develop one code applicable for projects for every EU country.

3.1.2 Eurocode 3 and other steel norms

The NEN-EN 1993-1 series is the current norm series used in the design and fabrication in the steel industry. The Eurocode 3 contains the following parts:

NEN-EN 1993-1-1	Rules for buildings.
NEN-EN 1993-1-2	Structural fire design.
NEN-EN 1993-1-3	Supplementary rules for cold-formed members and sheeting.
NEN-EN 1993-1-4	Supplementary rules for stainless steels.
NEN-EN 1993-1-5	Plated structural elements
NEN-EN 1993-1-6	Strength and stability of shell structures.
NEN-EN 1993-1-7	Strength and stability of planar plated structures subjected. to out of plane loading.
NEN-EN 1993-1-8	Design of joints
NEN-EN 1993-1-9	Fatigue
NEN-EN 1993-1-10	Material toughness and trough-thickness properties.
NEN-EN 1993-1-11	Design of structures with tension components
NEN-EN 1993-1-12	High strength steels.

The above mentioned norms are intended for the design of structures. These norms operate in conjunction with the EN-1990. The EN-1990 contains three parts, dealing with regulations for assembly, fabrications and erection of structural elements for steel structures as well as for aluminium structures. [16]

EN 1090-1:	Requirements for conformity assessment for structural components.
EN 1090-2:	Technical requirements for the execution of steel structures.
EN 1090-3:	Technical requirements for the execution of aluminium structures.

Besides these executional norms, the Eurocode 3 refers to the NEN-EN 1990: Eurocode - Basis of structural design and Eurocode 1: NEN-EN 1991 Actions on structures for applying the forces. In the case of a composite connection such as a base plate, the Eurocode 3 is related with Eurocode 2: NEN-EN 1992.

Relevant standards to determine metallurgic properties, classify metallurgic materials and norms with requirements of (structural) steel products are described in:

- EN 10002: Tensile testing of metallic materials. Method of test at ambient temperature.
- EN 10020: Definition and classification of grades of steel.
- EN EN 10025, part-1-5: Technical Delivery Conditions for structural steels.

These standards are used in the research *Behaviour of flush end-plate beam-to-column joints under bending and axial force*, by Luís Simões da Silva [5], this research will be analysed later in this document.

3.1.3 The Green Books

Noteworthy standard are included in "The Green Books". The Green Books were developed by The Steel Construction Institute (SCI) in conjunction with the British Constructional Steelwork Association (BCSA). The SCI/BCSA connection group, which was developed in 1987, published two books about simple connections and moment connections:

- Simple joints to Eurocode 3. [7]
- Moment-resisting joints to Eurocode 3. [3]

The purpose of these publications was to bring together academics, consultants and steelwork contractors on authoritative design of steel connections.

The checks prescribed in the green books are according to the rules written in the Eurocode. Using the green books as a design guide, account must be taken that the used safety factors are based on the national annex of the British Standard.

3.2 Calculation Methods and Procedures

Norms and standards describe the different procedures for the calculation of connections. Depending on assumptions, different procedures for calculations may be used. The general procedure of the structural calculation of (steel)construction projects is depicted in figure 3.

The engineering contractor starts the design of the structural frame work on basis of the "Basis of Design" (BOD) and the architectural drawings from the architect. The structural framework will be translated to a calculation model where the loads are applied in accordance with the prescriptions written in the NEN-EN 1990. These calculation models which are used should be worked out in accordance to NEN-EN 1993-1-1, chapter 5. In this chapter the requirements are stated for an appropriate model.

Calculation models provide a force distribution of the structural system. These models present moments, shear forces and axial forces. These internal forces are used by the contractor to design and calculate the connections.

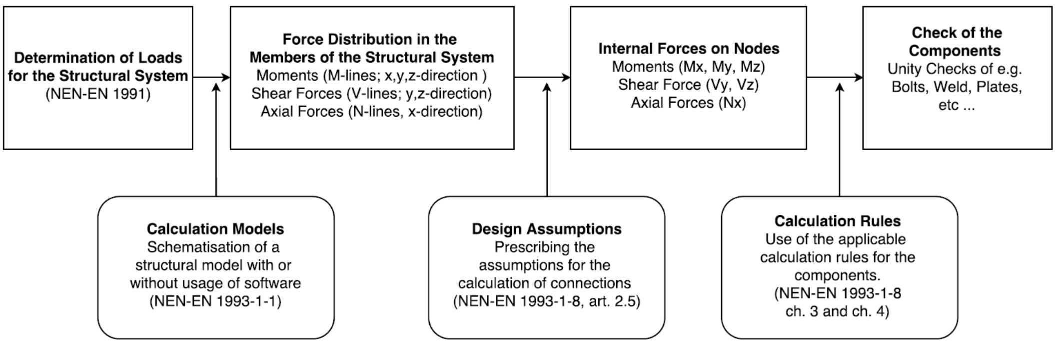


Figure 3.1: Flow Chart of the calculation procedure according the Eurocode

The calculation of the connections should be executed with the correct design assumptions which are included in NEN-EN 1993-1-8, art. 2.5.

On the basis of realistic assumptions, the calculation rules can be used for the checking of different connection components which are included in NEN-EN 1993-1-8, chapter 3 and chapter 4.

Chapter 4

Theory and Experiments

4.1 Theory of Steel Joints

In structural frameworks, the intersection points between the structural members are schematized as hinged connections or as full rigid connections. In hinged connections is assumed that the bending moment is zero and there is the possibility to have some rotation between the centrelines of the members. Rigid connections are able to take up a bending moment, but there is no rotation between the centrelines of the structural members.

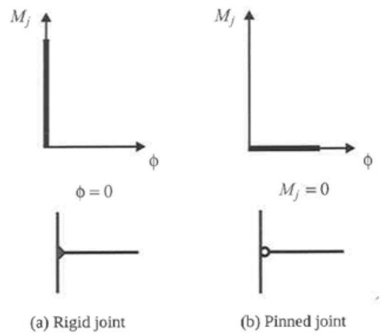


Figure 4.1: Idealised behaviour of a rigid joint and a pinned joint. [10]

The above described situations are idealised situations. In reality the behaviour of a connection is in between. There is always some moment uptake and always some small rotation in the connection. A connection behaves as a rotational spring, with a certain rotational stiffness. The stiffness of a connection behaves non-linear and decreases after loading the connection. Depending on the behaviour of the connection, the connection can be classified as: nominally pinned, semi-continuous and continuous according the Eurocode.

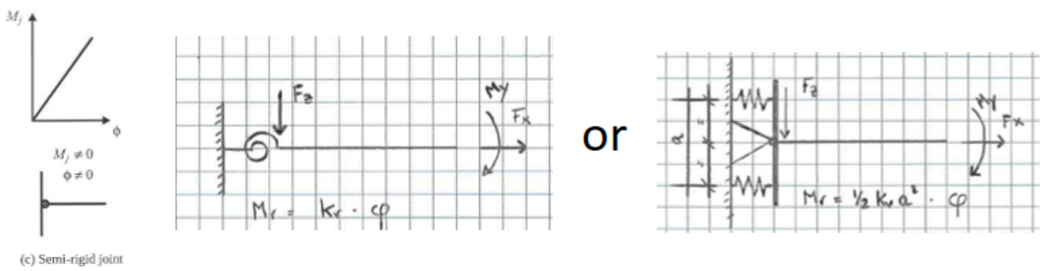


Figure 4.2: Spring model of a connection. [10]

The rotational behaviour of a connection is non-linear. Under a load the connections will rotate, a bending moment in the connections will be generated depending on the rotational capacity and ductility. Connections classified as continuous are generating a considerable bending moment after a small rotation between the centrelines. Connections classified as nominally pinned will also generate a small bending moment after a rotation.

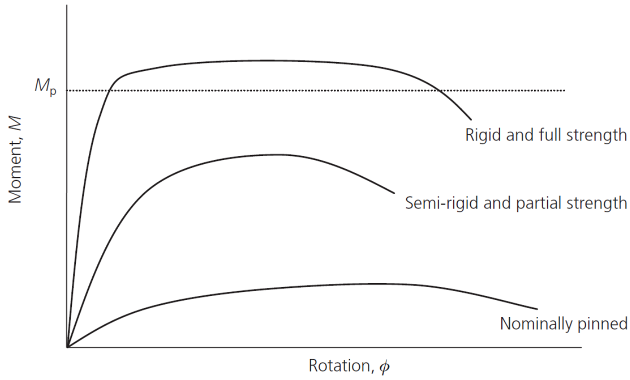


Figure 4.3: Moment-Phi representation of three different classes of connections. [8]

Just as the material behaviour of steel, the Eurocode also simplifies the behaviour of connections with a tri-linear representation. From the first linear line the initial stiffness can be derived. After the connection is loaded at 2/3 of its capacity the stiffness will be reduced by a factor etha, which is depending on the type of joint.

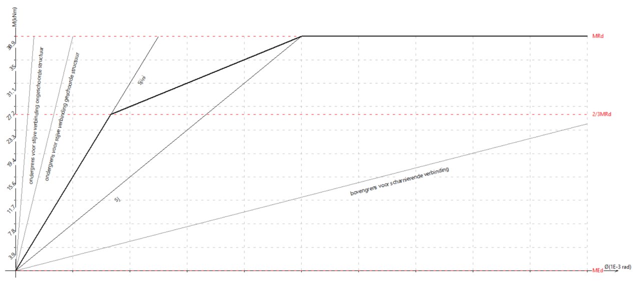


Figure 4.4: Moment-Phi representation of steel connections according the Eurocode

4.2 Experiments on Steel Joints

To understand the behaviour of connections under loading, literature research has been done into experimental behaviour of steel connections. The experimental results may be useful for the joints which may be partially modelled with FEM. In this chapter a description can be found of two experiments which are part from the same research program of the University of Coimbra, located in Portugal.

4.2.1 Analysed Report: Behaviour of flush end-plate beam-to-column joints under bending and axial force

First report with extended information given about the joint configurations and test result was:

- Behaviour of flush end-plate beam-to-column joints under bending and axial force, L.S da Silva et al, 2004. [5]

This research is part of a program where two moment resisting connection were tested under bending and axial force. In this program two connections are executed:

1. **Flush end plate moment resistant connection.**
2. Extended end plate moment resistant connection.

In this report only the experiment of the flush end plate connection will be explained. Purpose of the experiment is to get insight of the behaviour of the moment resisting connection of a flush end plate under a bending moment and an axial force. In this report 9 specimens (FE1 till and up to FE9) are tested under a different load combination. One specimen (FE2) is used as verification of the applied axial force. During this verification process the central load cells , individual load cells on all tension cables and strain gauges are calibrated. Specimens are taken of the structural steel to derive the real strength properties, yield strength, tensile strength and E-modulus, of the structural material. On basis of these specimen calculation are done without partial safety coefficients.

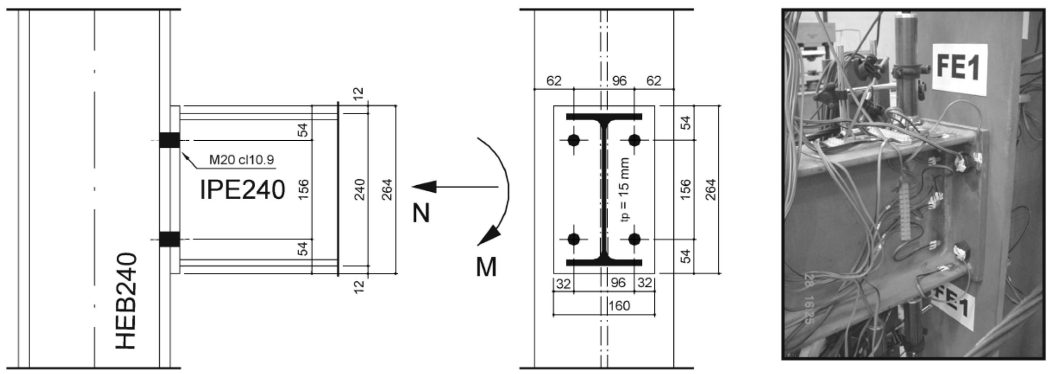


Figure 4.5: Configuration of the flush end plate moment resistant connection. [5]

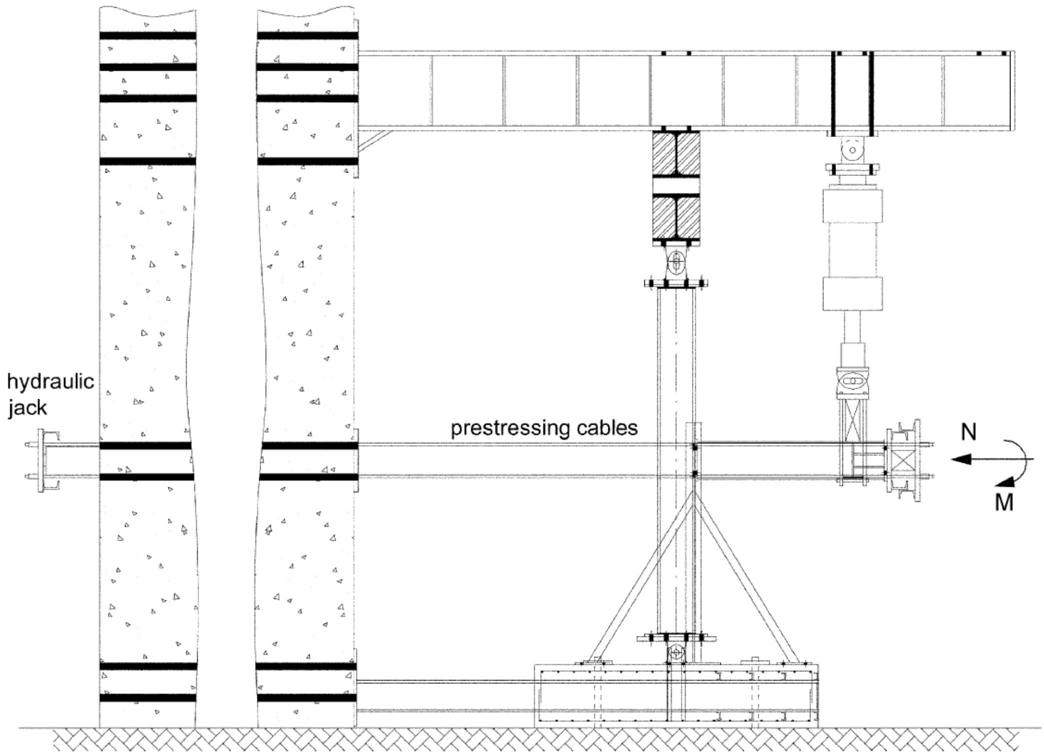


Figure 4.6: Test set-up of the experiments (Flush end plate, extended end plate). [5]

First one benchmark experiment is executed (FE1). This joint is only subjected to bending moments, no axial force is applied on this connection. Remarking notation in the report is that the initial stiffness calculated according the Eurocode for this moment resisting connection differs significantly with obtained experimental results.

After the first experiment other load combinations are applied to the test specimen (FE3 till and up to FE9). Results of these separated experiments are depicted in figure 4.7.

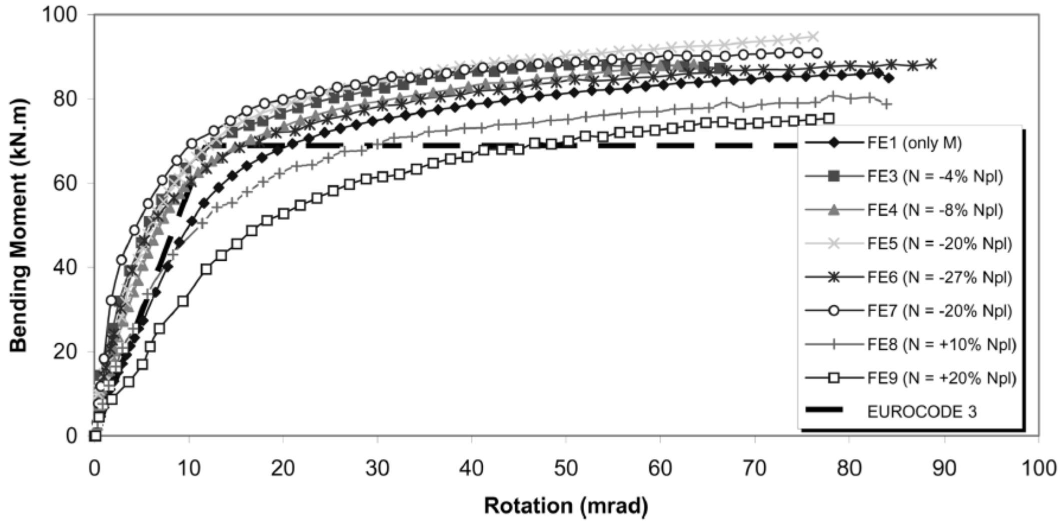


Figure 4.7: Results of the experiments of the flush endplate connections. [5]

4.2.2 Analysed report: Experimental evaluation of extended end plate beam-to-column joints subjected to bending and axial force

Second report with extended information given about the joint configurations and test result was:

- Experimental evaluation of extended endplate beam-to-column joints subjected to bending and axial force; L.R.O. de Lima et al., 2004. [6]

This experiment was the part of the same research program as previously described. In this program the same the connection is similar tested but with another joint configuration.

1. Flush end plate moment resistant connection.
2. **Extended endplate moment resistant connection.**

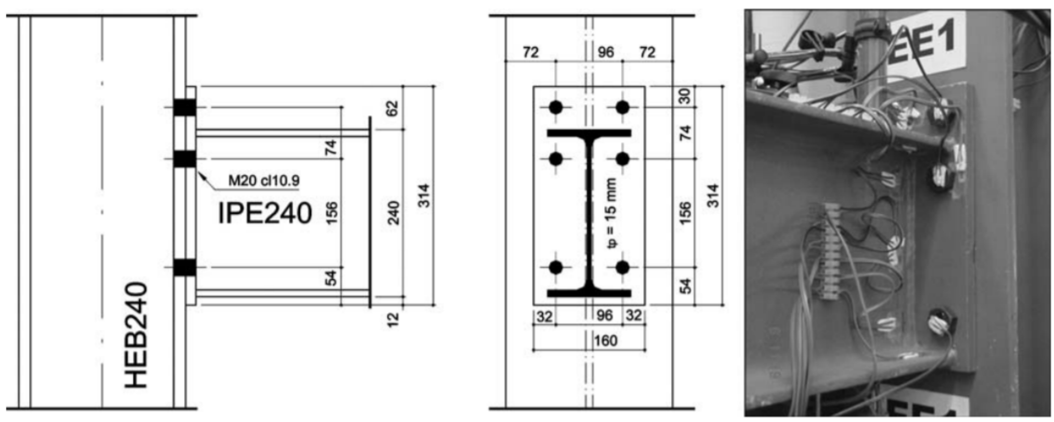


Figure 4.8: configuration of the flush end plate moment resistant connection. [6]

In this experiment seven test were executed, with a joint configuration depicted in fig. 4.8. Different load compositions are executed on the different specimen (EE1 - EE7). Results of this experiment are depicted in figure 4.9.

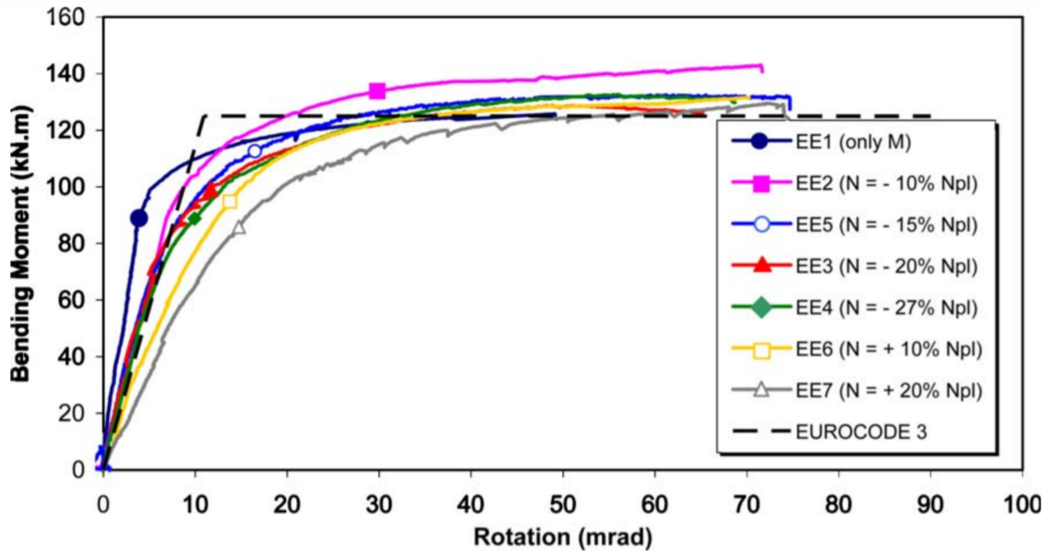


Figure 4.9: Results of the experiments of the extended end plate connections. [6]

Chapter 5

Software and Calculation Procedures

5.1 Software Availability

Information Technology (IT) has become an important factor in today's society. IT solutions give the opportunity to automate repetitive actions.

In structural engineering software is developed by different developers to ease calculation procedure. To get insight which calculation software is available for the sector, web research is done, inquiry is done at the engineers at the engineering department and a visit was done to the "Staalbouwdag 2016" on the 11th october 2016 at "Kromhouthal aan het IJ in Amsterdam".

Table 5.1: Overview of the found available software for the structural engineering with possibilities to design and analyse steel joints

Software Developer	**Software Name**	**Location**
Nemetschek	SCIA Engineer	München, Germany
Technosoft	Verbindingen	Deventer, The Netherlands
Buildsoft	Power Connect	Merelbeke, Belgium
IDEA RS	IDEA Connection	Brno, Czech Republic
Dlubal	RSTAB Connections	Germany
COP	COP	Germany

Software listed in table 5.1 is *special purpose software*, which is developed for structural engineering purposes. These group of software names are developed for calculating connections or have extended options to do so.

Beside these special purposed software you have *general purpose software* which can be used for modelling of materials. ANSYS, ABACUS and DIANA FEA are examples of general purpose software which have the possibility to calculate stresses and strains in materials by finite element modelling.

One software package listed in table is of special interest *IDEA StatiCa Connection*. Most conventional special purpose software packages for calculating connections are using the calculation rules described in the Eurocode. IDEA StatiCa Connection combines these prescribed calculation rules with finite element modelling. This new approach is called the (CB)FEM method. [27]

Beside software there are possibilities to downloaded pre programmed spreadsheets for the calculation of connections and other structural elements, for example Qec Excel-Rekenbladen [18], which was suggested during the first interim meeting on the 11th of January 2017.

5.2 Component Method

Traditional software applications (Technosoft, Steel Connect, COP) are using the component method with the calculation rules described in the Eurocode. Many software developers give the options to use other standards to calculate the connections for example AISC standards. (American Institute of Steel Construction). Calculations of these software are based on the Eurocode 3, resistance values are determined according Eurocode 3: NEN-EN 1993-1-8 for steel connections (NEN-EN 1992-1-1 is also used for base plate joints).

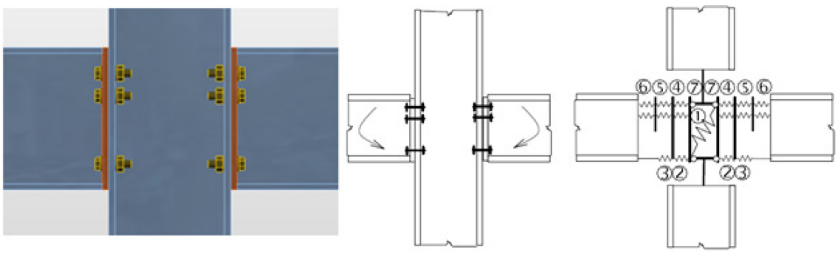


Figure 5.1: Component Method; Components of a steel connections; bolts, plates, flanges and webs are modelled as springs [22]

Using the component method the following steps are taken to determine the different resistance strength of the components and the total resistance strength of the connection.

- Step 1: Subdivision of the connection on basis of components, with possible failure modes.
- Step 2: Determination of the resistance strength of the components against failure.
- Step 3: Determining the governing failure component by executing unity checks, which is normative for the total resistance of the joint.

In general the following failure modes of these components for steel connections need to be checked:

- Column web panel in shear
- Column web in transverse compression
- Column web in transverse tension
- Column flange in bending
- End plate in bending
- Flange cleat in bending
- Beam or column flange and web in compression
- Beam web in tension
- Bolts in shear
- Bolts in bearing (on flange)
- Welds

5.3 Component Based Finite Element Method

A newly developed calculation method is the Component Based Finite Element Method (CBFEM). This method combines two methods of calculations. The most important assumptions of this method are:

- FEM calculations are used to calculate the stresses and strains in the web, flanges, fin plates and end plates.
- Resistance values are of the connecting components, e.g. bolts, welds and anchors are still calculated according standards

The CBFEM method is based on the idea, that the most of verified and very useful parts of the component methods should be kept. The weak point of component modelling, its generality when analysing stresses of individual components, should be replaced by finite element method.[22]

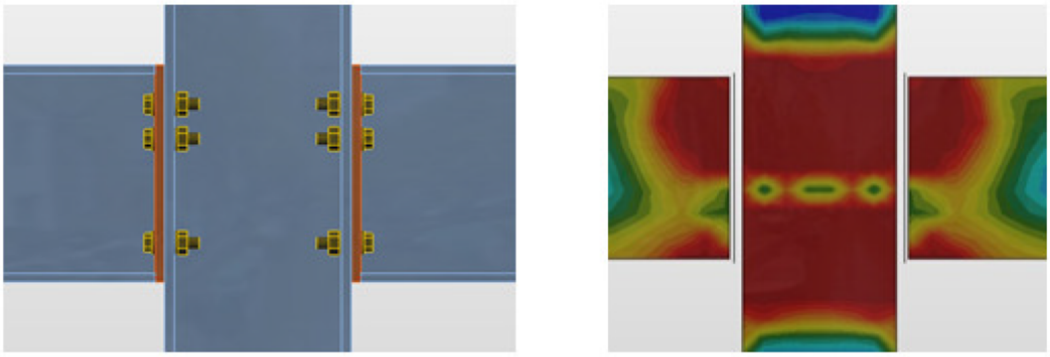


Figure 5.2: FEM Method; Plates, webs and flanges are modelled with a mesh for calculation of strain, stresses and displacement. [22]

Welds
For the calculation of the welds the directional methods will be still be applied. Weld check can be done elastically and plastically. Two options are available to determine the stress in the welds: Maximal stress, average stress and interpolation of the stress.

Bolts
Bolts are also determined according the Eurocode. Usual checks are executed with the table presented in NEN-EN 1993-1-8 table(3.4): Shear resistance $F_{v,Rd}$, bearing resistance $F_{b,Rd}$, tension resistance $F_{t,Rd}$ but also punching shear resistance $B_{p,Rd}$.

For the modelling of the connection it is important to take the elongation of the bolt into account. For the determination of the bolts elongation of the modelled with a bi-linear model designed depicted in fig 5.3

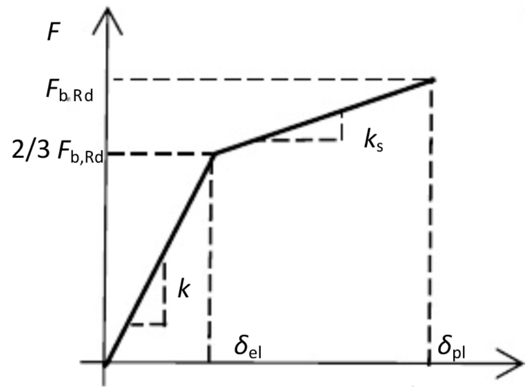


Figure 5.3: Bolt model which is used to determine the deformation of the bolt under loading. [22]

Interaction of shear force and tensile force is also calculated accordance EC3. For a connection where the bolts are loaded by a force in the transverse direction (shear force) and the longitudinal direction (tensile force), the following equation is applied.

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 * F_{t,Rd}} \leq 1.0 \tag{5.1}$$

In hand calculations often specific bolts are reserved for taking up the tension force and other bolts the shear force. In a partial FEM model of a connection no distinction is made in tension bolts and shear bolts. Loads applied on the bolts are derived from stress concentrations from the mesh.

Concrete

Idea Connection offers the possibility to design composite steel-concrete connections. Applied formulas for the checks of the checks are described in the Eurocode 2: NEN-EN 1992-1-1, which dedicated for designing and calculating concrete structures. [2]. For determining the resistance of anchors in concrete European Techical Approval Guidelines (ETAG-0001) are used.

Plates, flanges and webs

In contrast to the conventional method the CBFEM do not use the prescribed components in the Eurocode. It uses finite element modelling for determining the stresses and strains in the plated elements. Stresses are determined with the Von Mises yield criterion:

$$\sigma_{VM} = \sqrt{\frac{1}{2}[(\sigma_{xx} - \sigma_{yy})^2 + (\sigma_{yy} - \sigma_{zz})^2 + (\sigma_{zz} - \sigma_{xx})^2] + 3 * (\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2)} \tag{5.2}$$

Because the mesh is modelled 2-dimensional the Von Mises formula of the plate the is reduced to:

$$\sigma_{VM} = \sqrt{\frac{1}{2}(\sigma_{xx})^2 + (\sigma_{yy})^2 + (\sigma_{yy}) * (\sigma_{xx}) + 3 * (\tau_{xy})^2} \tag{5.3}$$

Yielding is allowed in the calculation model. After reaching the yielding point straining is allowed till a specific limit value. Although in the Eurocode NEN-EN 1993-1-1 art. 3.2.1. is stated that elongation of steel can be reached till a minimum of 15% for plated elements a strain limit of 5% in FEM calculations is used, which is prescribed in the NEN-EN 1993-1-5, NA C.8, for the prevention of plate instability by plate buckling. [17]

5.4 Differences between the Manual Component Method and CBFEM

Both methods are using the Eurocode 3 as basis for the calculation. Main difference is that the stresses in the plated elements in the joints (e.g. flanges, webs, end plates and fin plates) are determined with FEM. Resistances of bolts and welds are in both methods determined according the calculation rules described in the Eurocode 3. An overview can be found in the table below.

Table 5.2: Overview of the difference in approach between the Manual Component Method and Component Based FEM

Elements		Manual Component Method	Component Based FEM
Columns	Flanges	Shear resistance Tearing resistance Bearing resistance	Limit strain
	Web	Shear resistance Tearing resistance Bearing resistance	Limit strain
Beams	Flanges	Shear resistance Tearing resistance Bearing resistance	Limit strain
	Web	Shear resistance Tearing resistance Bearing resistance	Limit strain
Plates		Shear resistance Tearing resistance Bearing resistance Bending resistance in combination with shear	Limit strain
Bolts		Shear resistance Bearing resistance Tension resistance Comb. shear and tension resistance	Shear resistance Tension resistance Punching shear Bearing resistance Comb. shear and tension resistance
Welds		Directional method Full strength assumption	Directional method

Chapter 6

Simple Shear Joints

Simple shear joints are steel joints which are designed to transfer the load predominately by shear force. Although simple shear joints can take up some bending moment they are not designed to. In this chapter three types of shear joints are discussed in this chapter: fin plate joint, short end plate joint and the double angle cleat joint.

6.1 General Procedure Simple Shear Joints

Shear connections are modelled as a hinge in the structural framework. Fin plate connections derive their rotation capacity from (1) the bolt deformation in shear, (2) hole distortions in bearing in the fin plate and/or the beam web, and (3) out of plane bending of the plate, [19]. It is important to ensure that the flange of the beam does not rotate against the column or girder.

$$end\ clearance > \frac{h_{beam}}{50}, \quad with\ a\ minimum\ of\ 10\ mm. \tag{6.1}$$

To apply plastic calculation it is important to have sufficient deformation capacity in the joint. This can be obtained by designing a relative thin fin plate. To obtain bearing as a governing check the thickness of the fin plate must meet the following requirement:

$$t_p \leq \frac{\alpha_v * A}{k_1 * \alpha_b * d} \frac{f_{ub}}{f_u} \tag{6.2}$$

Simple shear joints, figure 6.1, are commonly executed as:

- a fin plate joint
- a short end plate joint
- double angle cleat joint

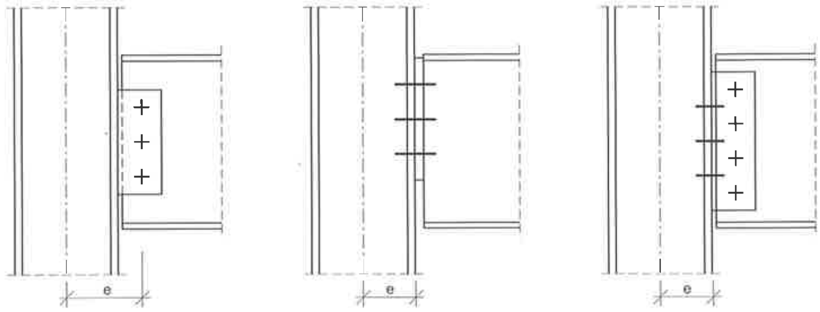


Figure 6.1: Shear connections: Fin plate connection, a short end plate connection and a connection with cleats

The shear resistance of the different elements is an important aspect for calculating the connection. The fact that the sections are weakened by bolt holes must be taken into account.

Shear Resistance: 1) shear resistance of the of the beam web (art. 6.2.1.1); Shear Resistance can be calculated with shear area of the column.

$$V_{pl,Rd} = \frac{A_{v,net} * \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} \tag{6.3}$$

2) Shear resistance fin plate/cleat (art. 6.2.1.1);

$$V_{pl,Rd} = \frac{A_{v,net} * \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} \tag{6.4}$$

3) Shear resistance of the bolts (art. 3.6)

$$F_{v,Rd} = \frac{\alpha_v * f_{ub} * A_s}{\gamma_{M2}} \tag{6.5}$$

Tearing Resistance: 1) Block tearing of the of the bolt holes in the beam (art. 3.10.2);

$$V_{eff,2,Rd} = \frac{0.5 * f_u * A_{nt}}{\gamma_{M2}} + \frac{\frac{f_y * A_{nv}}{\sqrt{3}}}{\gamma_{M0}} \tag{6.6}$$

2) Block tearing in the (fin) plate (art. 3.10.2).

$$V_{eff,1,Rd} = \frac{0.5 * f_u * A_{nt}}{\gamma_{M2}} + \frac{\frac{f_y * A_{nv}}{\sqrt{3}}}{\gamma_{M0}} \tag{6.7}$$

Bearing Resistance 1) Bearing resistance of the beam web (art. 3.6 Table 3.4).

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_u * d * t_w}{\gamma_{M2}} \tag{6.8}$$

2) Bearing resistance of the fin plate/cleat (art. 3.6 Table 3.4).

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_u * d * t_p}{\gamma_{M2}} \tag{6.9}$$

Buckling Resistance Buckling of the fin plate / cleat.
First check is the determine if a buckling check is needed.

$$\frac{m}{t_p} \leq 9 * \epsilon, \quad \frac{m}{t_p} = 9 * \sqrt{\frac{235}{f_y}} \tag{6.10}$$

This equation is similar to the equation to categorize profiles to cross-section class one. For cross-section class accounts, no buckling control is required. This equation can be derived [20].

$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{1}{12} * h_p * {t_p}^3}{h_p t_p}} \tag{6.11}$$

$$\lambda = \frac{L_{cr}}{i} = \frac{0.6 * c}{i}, \quad \lambda_1 = \pi * \sqrt{\frac{E}{f_y}} \quad \overline{\lambda} = \frac{\lambda}{\lambda_1} \quad \epsilon = \sqrt{\frac{E}{f_y}} \tag{6.12}$$

Buckling effects may be neglected if $\overline{\lambda} \leq 0.2$ according the buckling curve depicted in NEN-EN 1993-1-1, art. 6.3.1.2, figure 6.4. This may be done for every curve, see figure 6.1.

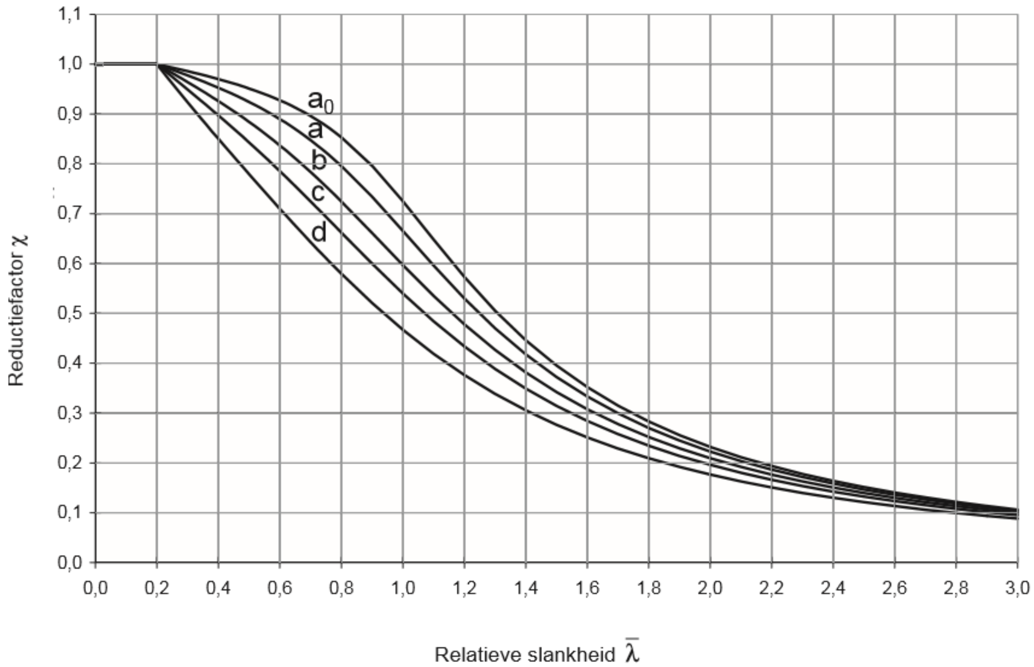


Figure 6.2: Buckling curve according NEN-EN 1993-1-8

Combining the above mentioned formulas and the buckling curve. No check for buckling resistance is required if the following equation is satisfied:

$$\frac{c}{t_p} \leq 9 * \epsilon \tag{6.13}$$

6.2 Fin Plate Joint (FPJ)

A fin plate joint is one of the simplest shear joint and is popular because it is one of the quickest connection to erect. [7]. The fin plate joint consist out of the following components that need to checked.

- Supported beam
- Fin plate
- Bolts, beam to fin plate
- Welds, fin plate to column
- Supporting column

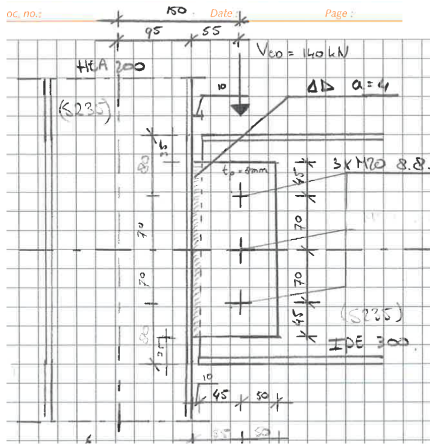


Figure 6.3: Joint Configuration

6.2.1 Joint configuration and assumptions FPJ

The following assumptions are done for manual calculation and modelling:

- Joint configuration as depicted in figure 6.3.
- Simple shear joint, load transfer by shear and axial force. Moment caused by excentricity is taken into account.
- Rotation centre of the joint is assumed in the middle bolt of the bolt pattern in the fin plate - beam web connection.

6.2.2 Manual Calculation FPJ

Manual calculation is executed according to the Eurocode 3. The following resistances are obtained for this joint. To determine total resistance of the joint, the following component resistances are calculated:

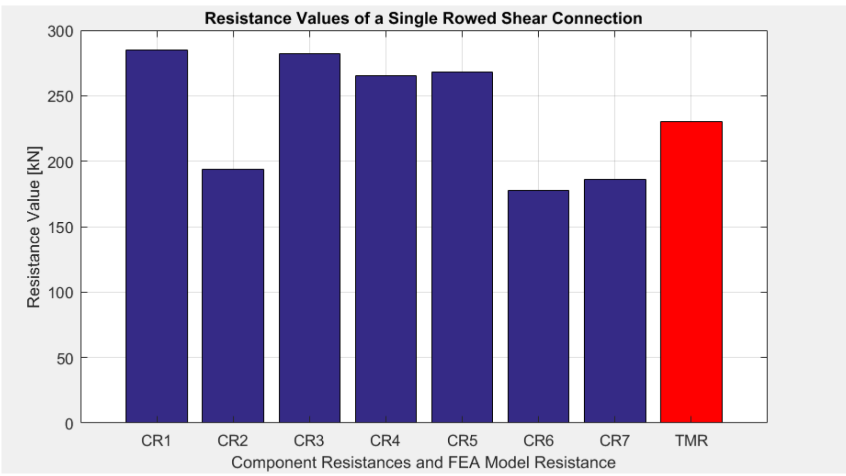


Figure 6.4: Component resistances Fin Plate Joint

Abbreviations used in the above mentioned figure are explained hereunder.

- Shear resistance of the beam web (CR1)
- Tearing resistance of the beam web (CR2)
- Shear resistance of the bolts (CR3)
- Bearing resistance of the fin plate (CR4)
- Bearing resistance of the beam web (CR5)
- Shear resistance of the fin plate (CR6)
- Tearing resistance of the fin plate (CR7)
- Total model resistance FEA (TMR)

6.2.3 Partial FEA model FPJ

A model has been made for the fin plate joint, the load is applied in the centre of the bolt pattern in the fin plate - beam web connection, which results in an eccentricity in the fin plate.

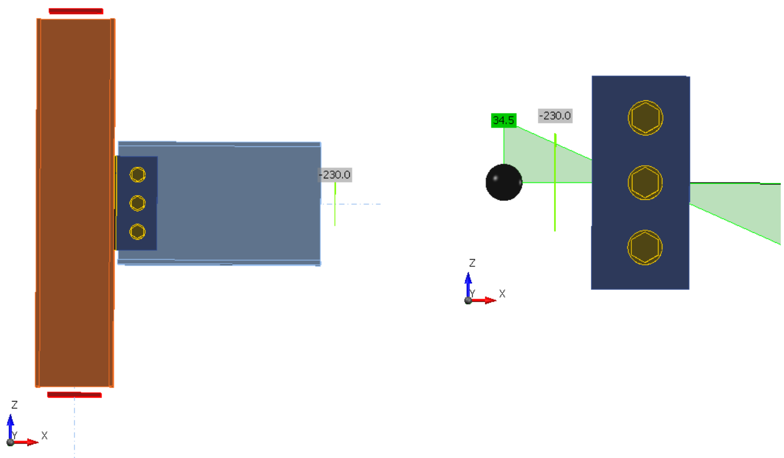


Figure 6.5: Model and loading on fin plate joint

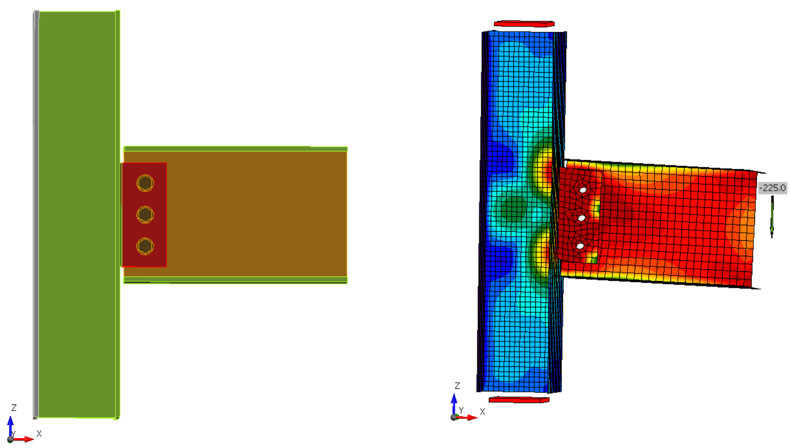


Figure 6.6: Decisive element and stress plot of the fin plate joint

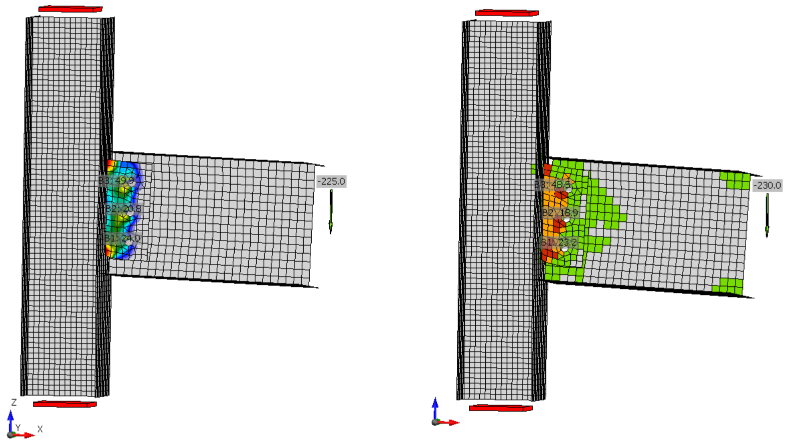


Figure 6.7: Strain development in the fin plate joint

6.2.4 Comparison Manual Calculation and partial FEA model

For both the manual calculation and the FEA model it holds that the fin plate is the governing component of the joint. Shearing and tearing resistance are relevant failure mechanisms of this joint. In the FEA model the first failure mechanism what will occur is exceedance of the limit strain in the fin plate. The following resistance values are obtained in this calculation:

- Resistance of the manual calculation $V_{Rd,EC3} = 178kN$
- Resistance of the partial FEA model $V_{Rd,model1} = 220kN$

Figure 6.8: Comparison resistances values fin plate, EC3 and FEA model

6.3 Short End Plate Joint (SEPJ)

Another type of a simple shear joint is the short end plate joint. This type of joint can tolerate a moderate offset in beam to column joints and is commonly used in connections with a skewed beam [7]

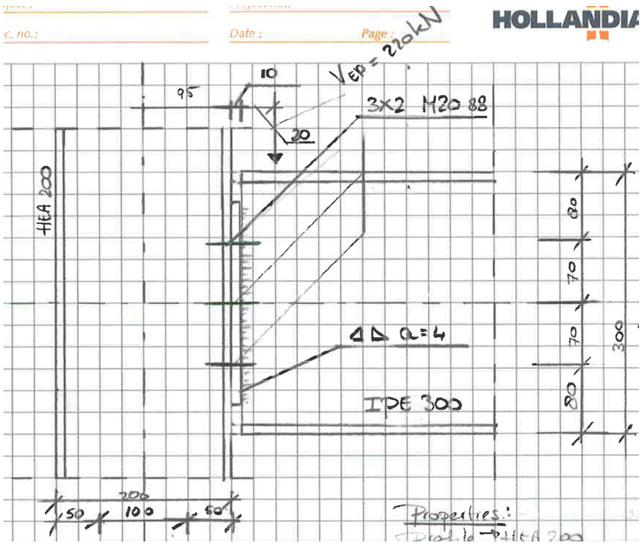


Figure 6.9: Configuration Joint

The short end plate joint consists out of the following components:

- Supported beam
- Short end plate
- Bolts, end plate to column
- Welds, beam web to end plate
- Supporting column

6.3.1 Joint configuration and assumptions SEPJ

The following assumptions are made for the manual calculation and the model:

- Joint configuration as depicted in figure 6.9.
- Plastic calculation, (shear) force is evenly distributed over the bolts.
- Rotation centre is assumed at the short end plate.
- All welds are full strength

6.3.2 Manual Calcualtion SEPJ

Manual calculation is executed according to the Eurocode 3. The following resistance values are obtained for this joint, figure 6.10.

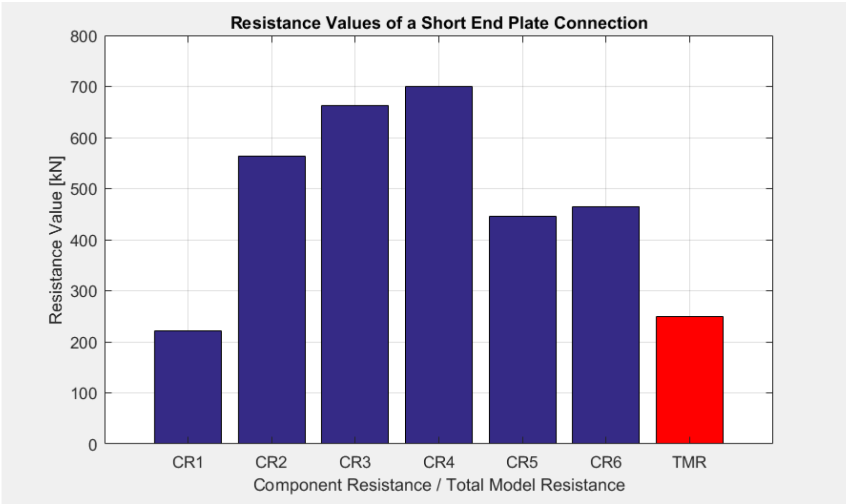


Figure 6.10: Component resistances in the short end plate joint

Abbreviations used in the above mentioned figure are explained hereunder.

- Shear resistance of the beam web (CR1)
- Shear resistance of the bolts (CR2)
- Bearing resistance of the end plate (CR3)
- Bearing resistance in the column flange (CR4)
- Shear resistance of the end plate (CR5)
- Tearing resistance of the short end plate (CR6)
- Total model resistance (TMR)

6.3.3 Partial FEA model SEPJ

A model has been made for the short end plate joint, the load is applied at the location of the end plate. In the FEA model straining in the beam web is the decisive failure mechanism.

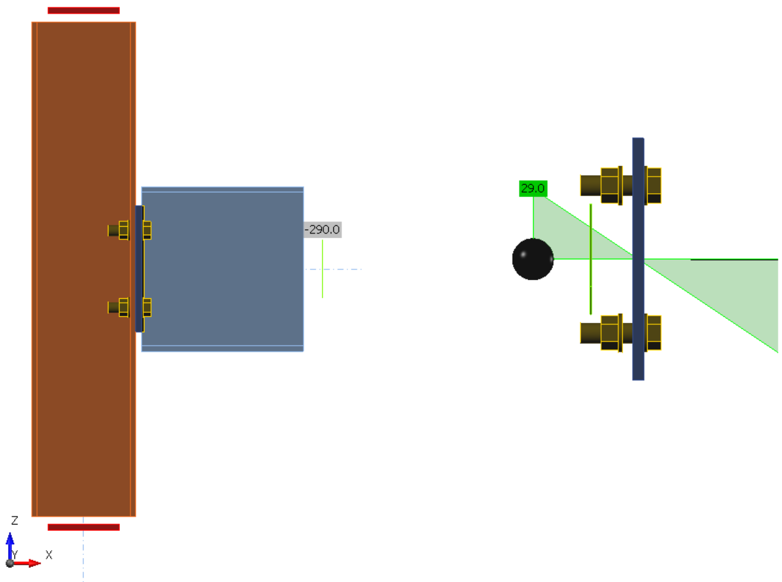


Figure 6.11: Model and loading of the short end plate joint

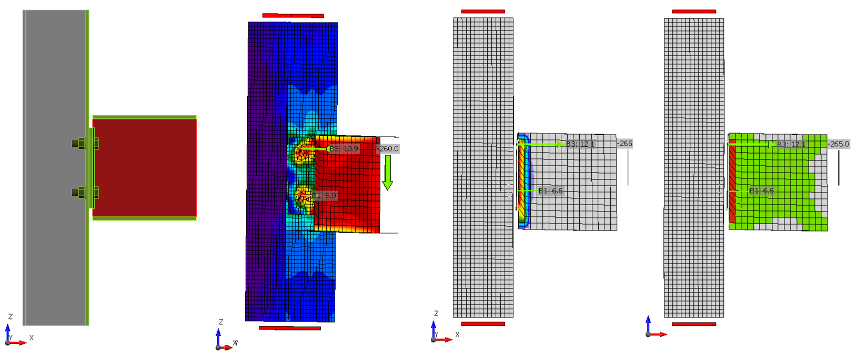


Figure 6.12: Governing component, Stress distribution, strain development in short end plate joint

6.3.4 Comparison Manual Calculation and partial FEA model SEPJ

For both the manual calculation and the FEA model the beam web is decisive. In the manual calculation shearing of the beam web is governing. In the FEA model exceedance of the 5% limit strain is the first mechanism that occurs. Graphic representation can be seen in figure 6.13.

- Resistance of the manual calculation $V_{Rd,EC3} = 222kN$
- Resistance of the partial FEA model $V_{Rd,model1} = 250kN$

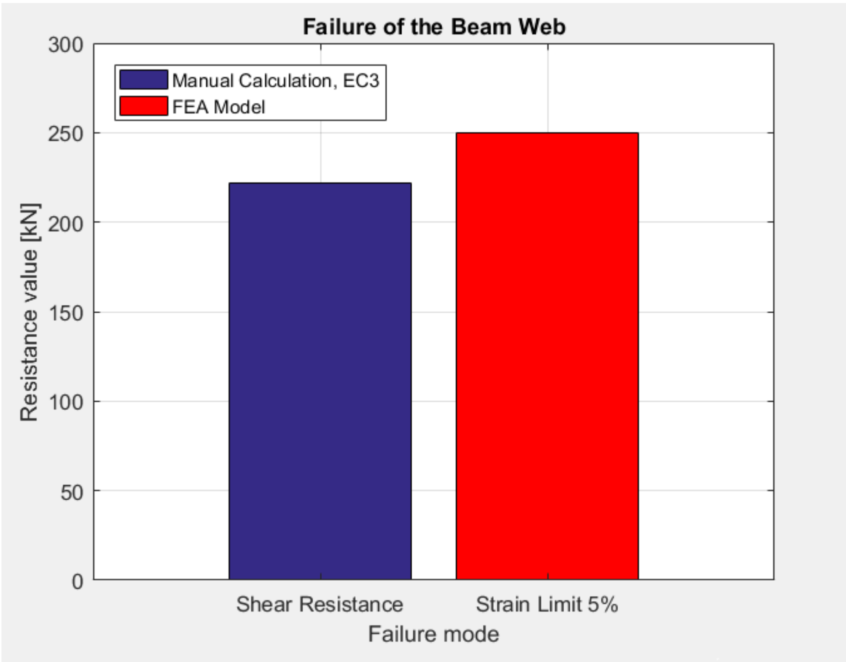


Figure 6.13: Comparison resistance beam web, EC3 and FEA

6.4 Double Angle Cleat Joint (DACJ)

Another type of shear connection is double angle cleat connection. Two bolts patterns on every leg are made to connect the column and the beam by bolting. In this configuration the bolt pattern in the column flange and beam web are differing, see figure 6.14.

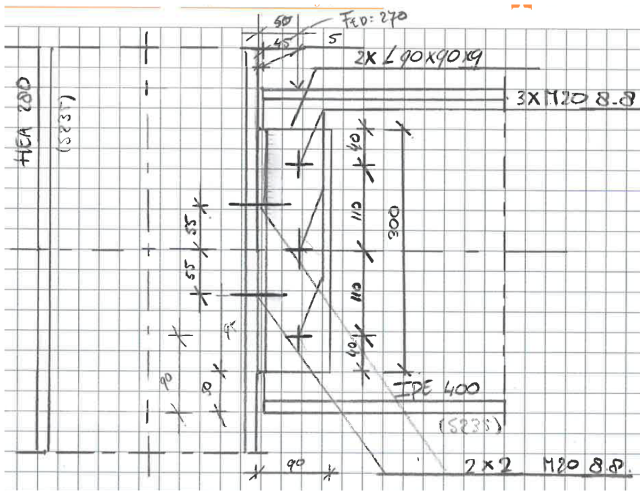


Figure 6.14: Joint configuration

The double angle cleat connections consist out of the following components:

- Supported beam
- Angle cleats, 2x
- Bolts, beam to cleat
- Bolts, cleat to column
- Supporting column

For this type of connection there are two possibilities to assume the rotation centre: 1) At the contact point between the column flange and 2) At the bolt pattern in the connection point at the beam and the cleats. Beside there are two possibilities to determine the resistance of bearing: 1) Elliptical Method [1] and 2) Method according to the Eurocode3, NEN-EN 1993-1-8, art. 3.6.1, table(3.4), note 3.

The elliptical method combines the bearing capacity in the multiple directions:

$$\sqrt{[\frac{F_{v,Ed,Hor}}{F_{b,Rd,Hor}}]^2 + [\frac{F_{v,Ed,Ver}}{F_{b,Rd,Ver}}]^2} \leq 1.0 \tag{6.14}$$

The Eurocode state that the bearing capacity of a bolt - plate connection may be considered separately.

$$\frac{F_{v,Ed,Hor}}{F_{b,Rd,Hor}} \leq 1.0 \tag{6.15}$$

$$\frac{F_{v,Ed,Ver}}{F_{b,Rd,Ver}} \leq 1.0 \tag{6.16}$$

6.4.1 Joint configuration and assumptions DACJ

Joint configuration is depicted in figure 6.14. For the manual calculation and the FEA model the following assumptions are made:

- Plastic calculation, for the manual calculation bolt forces in vertical direction are distributed equally over the bolts. Horizontal forces on bolts are based on equilibrium.
- Rotation centre of the manual calculation and first FEA model is assumed at the contact point of the beam flange and double angle cleats, see figure 6.13.
- Rotation centre of the second FEA model is assumed at the middle of the bolt pattern in the beam web - angle cleats connection.
- Welds are assumed full strength

6.4.2 Manual Calculation DACJ, rotation centre at contact point column flange - cleats

In the manual calculation the rotation centre is assumed at the contact point of column flange and double angle cleats. The different components are considered. In the following graph only the resistance of the governing bolts are presented.

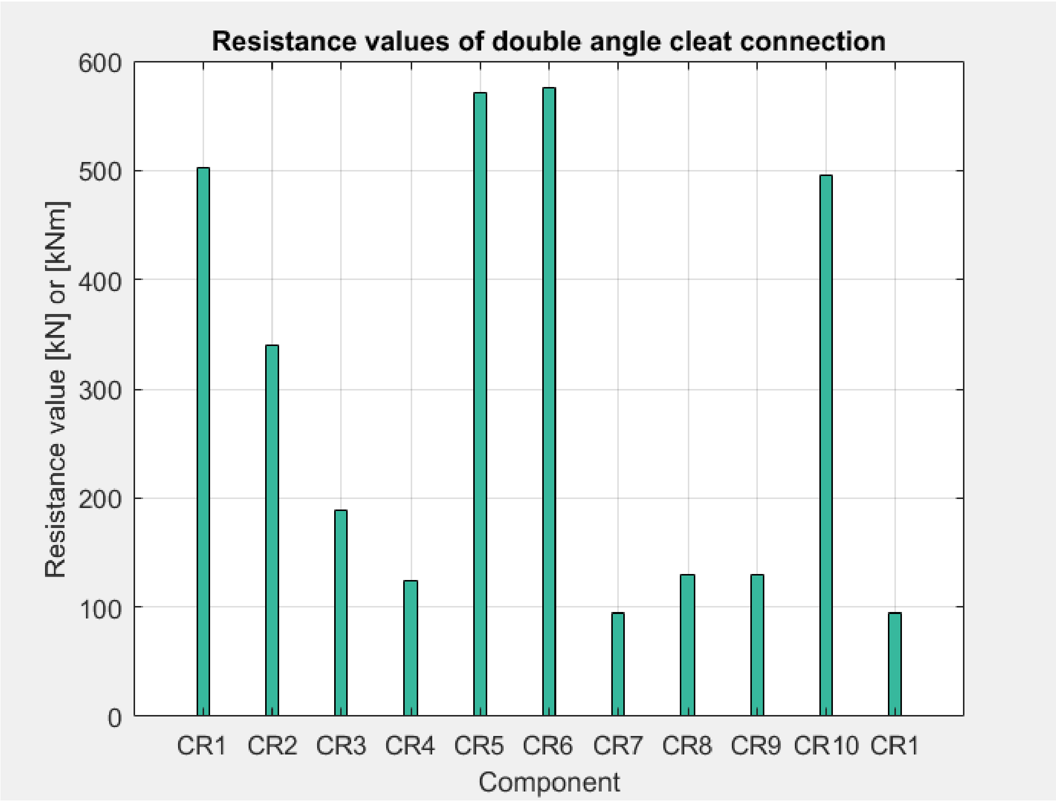


Figure 6.15: Component resistances for the double angle cleat joint

Abbreviations used in the above mentioned figure are explained hereunder.

- Shear resistance of the beam web (CR1)
- Tearing resistance of the beam web (CR2)
- Shear resistances of the bolt pattern in the beam (CR3)
- Bearing resistances of the bolt pattern in the beam (CR4)
- Shear resistance of the cleat (beam side) (CR5)
- Tearing resistance of the cleat (beam side) (CR6)
- Shear resistance of the cleat (column side) (CR7)
- Shear resistance of the bolt pattern in column (CR8)
- Bearing resistance of the bolt pattern in column (CR9)
- Tearing resistance of the cleat (CR10)
- Moment resistance of the cleat (CR11)

6.4.3 FEA model one DACJ, rotation centre at second bolt pattern column flange - angle cleat connection

A model has been made for the double angle cleat shear joint, the load is applied near the contact point between column flange and the angle cleats. The upper bolt in the beam flange is the weakest component, with a small increase of the load also beam web fails.

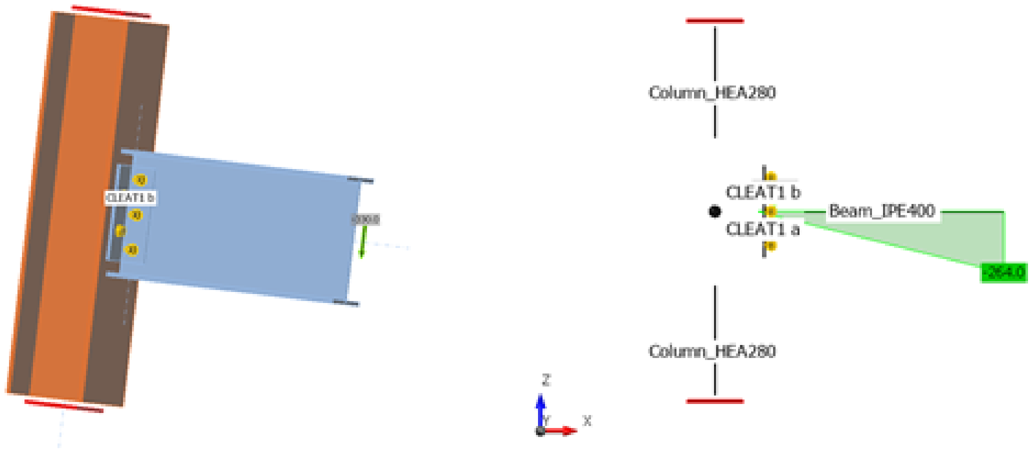


Figure 6.16: Model and load applied on the double angle cleat joint

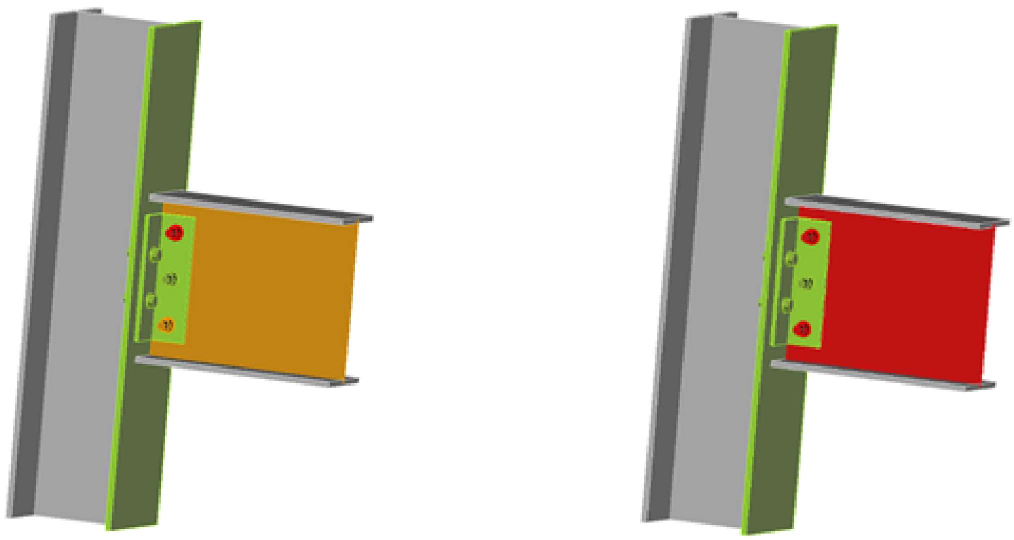


Figure 6.17: Governing components in the model of the double angle cleat joints

The maximum shear force which can be applied before failure is $V_{Ed} = 320kN$. Stress distribution is depicted in figure 6.18.

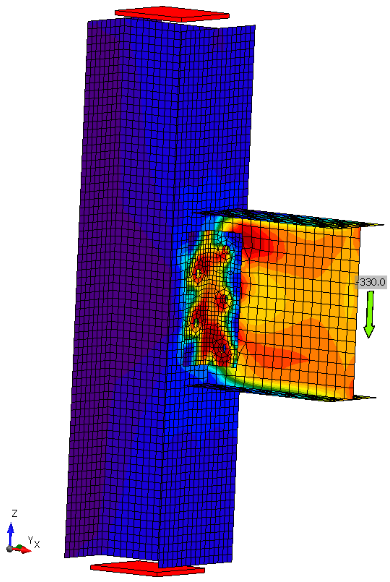


Figure 6.18: Stress distribution in model

6.4.4 Comparison Manual Calcualtion and FEA model

The manual calculation and the FEA model give similar results. For both methods the connection of the beam web to the angle cleats is governing. Although the bearing resistance $F_{b,Rd}$ is calculated identically the FEA model give a higher utilization value because in the model the outer bolts take up more forces, while in hand calculation forces on bolts are devided equally and based on equilibrium.

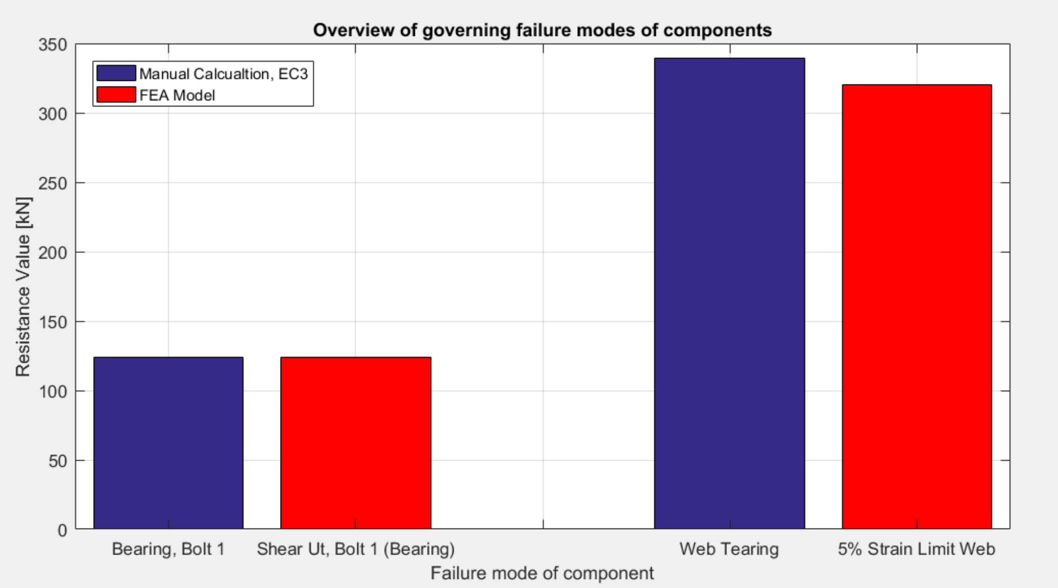


Figure 6.19: Comparison resistances manual calculation and FEA model.

Because not all bolts take up the same forces, unity checks are executed to determine the utilisation of the resistance. The following graph represent the unity-checks using the statements described in NEN-EN 1993-1-8, table 3.4, note 3. This method is less conservative than the elliptical method.

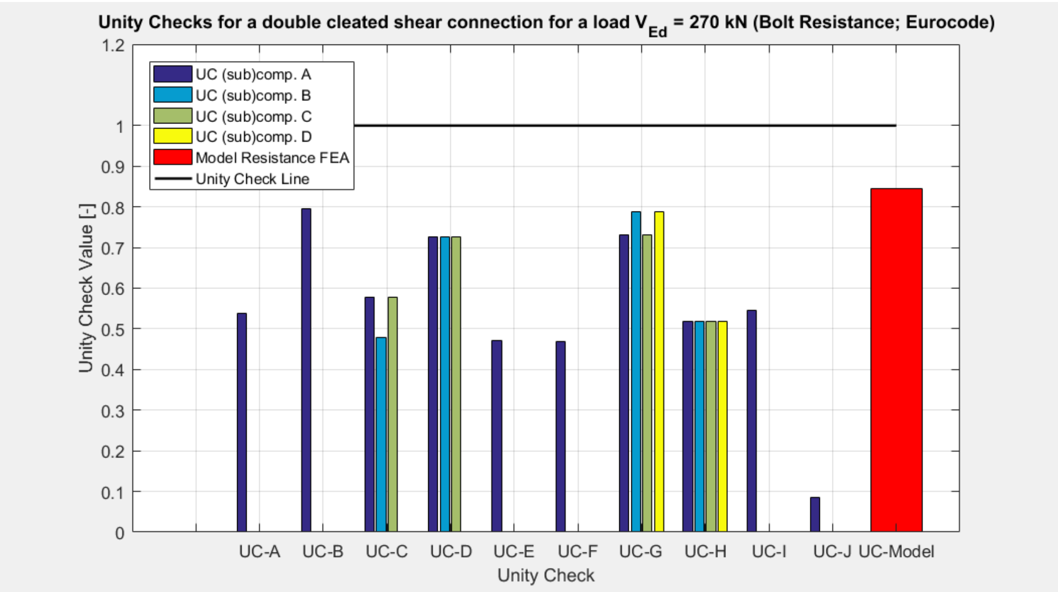


Figure 6.20: Comparison utilization of the component resistances

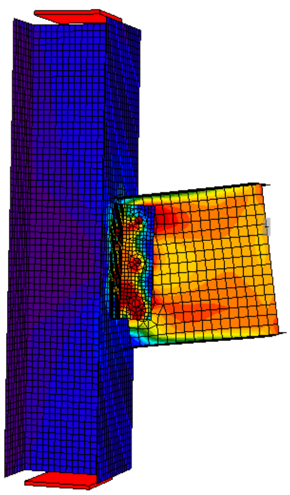
Explaination of the unity checks which are executed:

- Unity check shear resistance of the beam web. (UC-A)
- Unity check tearing resistance of the beam web. (UC-B)
- Unity checks of shear resistance of the 1) upper bolt, 2) middle bolt and 3) lower bolt. (UC-C; 1-3)
- Unity checks of the bearing resistance of the 1) upper bolt, 2) middle bolt and 3) lower bolt. (UC-D; 1-4)
- Unity check of the shear resistance of the angle cleats, beam side. (UC-E)
- Unity check of the shear tearing resistance of the angel cleats, beam side (UC-F)
- Unity checks of the shear resistance of bolt pattern in column flange (UC-G; 11-44)
- Unity checks of the bearing resistance of the bolt pattern in column flange (UC-H; 11-44)
- Unity checks of the tearing resistance of the bolt pattern in column flange (UC-I; 11-44)
- Unity check of the tearing resistance of the angle cleats, column side. (UC-I)
- Unity check of the moment resistance of the angle cleat at the radius. (UC-J)
- Unity check of the total FEA model $UC_{Model} = \frac{V_{Ed}}{V_{Rd,model}}$ (UC-model)

Evaluating the governing components in the manual calculation and by sequentially increasing the load on the FEA model till failure, the following resistance values are obtained:

- Resistance of the manual calculation $V_{Rd,EC3} = 340kN$
- Resistance of the partial FEA model $V_{Rd,model1} = 320kN$

Although the model and the manual calculation give similar results the question is if the model behaves realistic and if the assumption of the rotation centre is correct. Figure 6.21 shows the deformation in the mode (exaggerated). For this reason another model made with the rotation centre on the bolt pattern of the angle cleat - beam web connection.



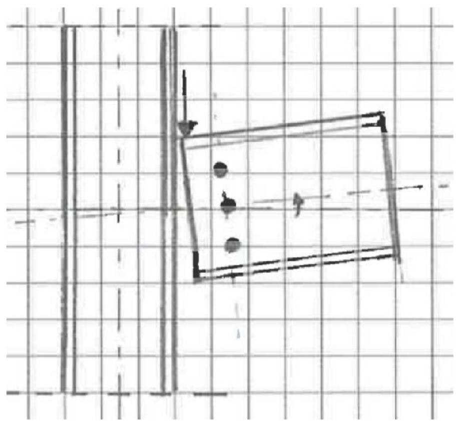


Figure 6.21: Deformation in the FEA model

6.4.5 FEA model two DACJ, rotation centre at angle cleat - beam web connection

A similar model has been made, this time only with another rotation centrum. Deformation can be seen in figure 6.22. The load applied on the model causes an eccentricity at the column flange - angle cleat connection. The first component which is failing is the upperbolt, in which the force is exceeding the combined shear and tension resistance.

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 * F_{t,Rd}} \leq 1.0 \tag{6.17}$$

For this FEA model the following resistance is obtained: $V_{Rd,model2} = 320kN$

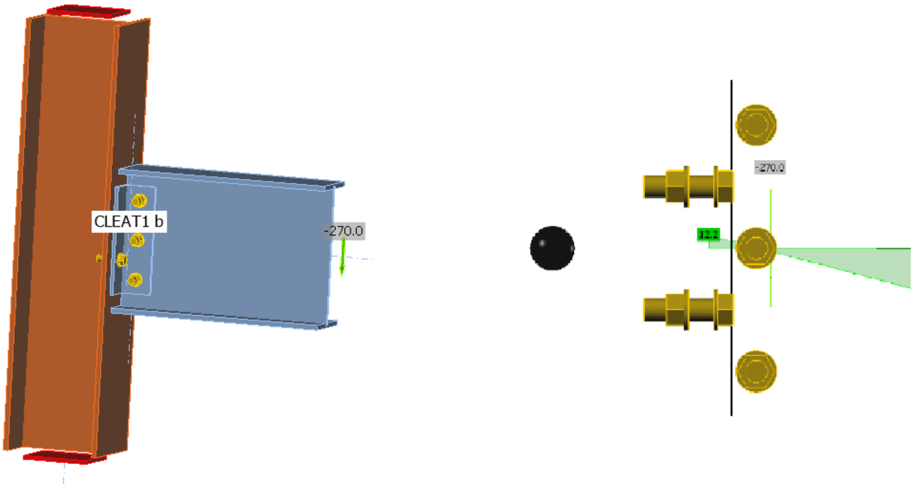


Figure 6.22: Model and load on bolt pattern (beam web - angle cleat)

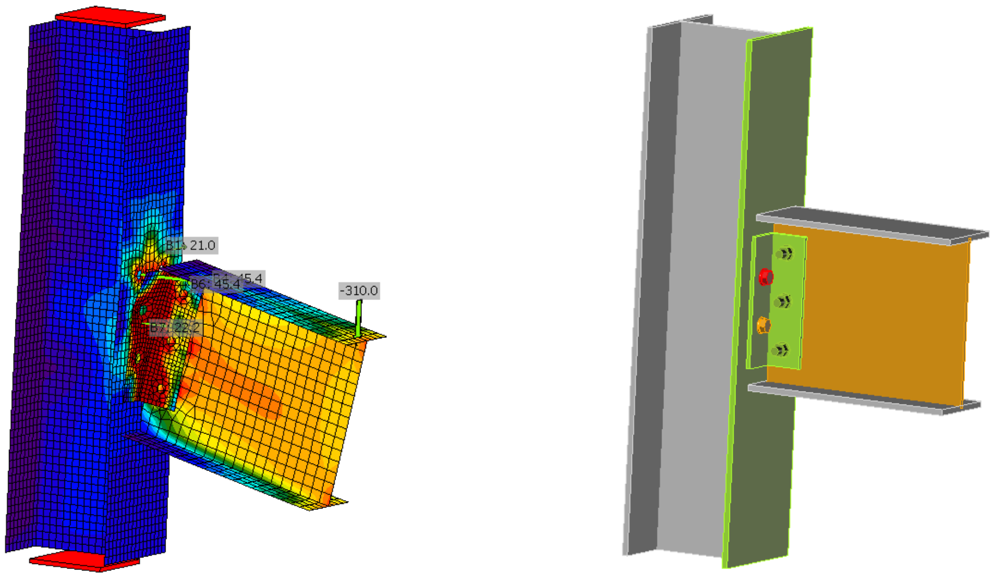


Figure 6.23: Stress development double angle cleat joint with deformation, failure component in the joint

6.5 Fin Plate Joint Variant (FPJV)

A variant is made of the fin plate joint. In this variant the end plate is connected to a square hollow section (SHS). Specifcally a configuration is choses where the SHS is decisive in the FEA model, to evaluate the behaviour of the column. A SHS 200x200x8 is chosen, other configurations can been seen in figure 6.24. According to book, *Verbinden* [20] the following assumptions can be done for determining the excentricity in the joint, figure 6.25. Two methods are available for determine the resistance of the SHS, namely 1) Eurocode 3 and 2) Cidect Recommendation. The latter method is described in the book *Hollow Sections in Structural Applications* i.a. written by J. Wardenier.[28]. For both methods no specific formula is given for determining the bending moment resistance of the column. A publication of TATA Steel, *Design of Welded Joints* [24] [25], does give design formulas to determine the bending moment resistance for their tubular steel products (Celsius®355 and Hybox®355) based on the aforementioned design prescriptions.

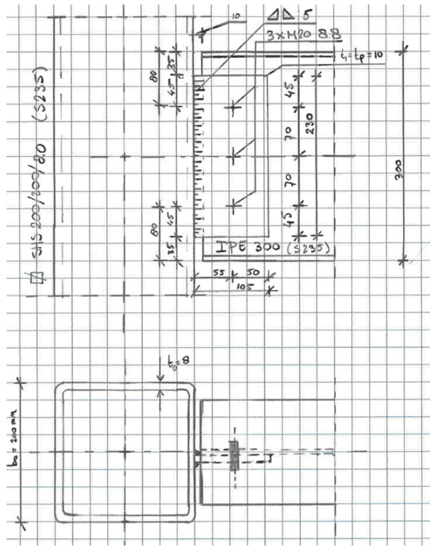


Figure 6.24: Joint Configuration

Eurocode 3, NEN-EN 1993-1-8, art. 7.5.2, table(7.13):

$$N_{1,Rd} = \frac{\frac{k_m * f_{y0} * t_0^2}{1 - \frac{t_1}{b_0}} * \left(\frac{2 * h_1}{b_0} + 4 * \sqrt{\left(1 - \frac{t_1}{b_0}\right)}\right)}{\gamma_{M5}} \tag{6.18}$$

CIDECT, *Design Recommendation.* Hollow Sections in Structural Applications, Chapter 9, table(9.4) [28]

$$N_{1,Rd} = 2 * f_{y0} * t_0^2 * [\eta + 2\sqrt{1 - \frac{t_1}{b_0}}] * Q_f \tag{6.19}$$

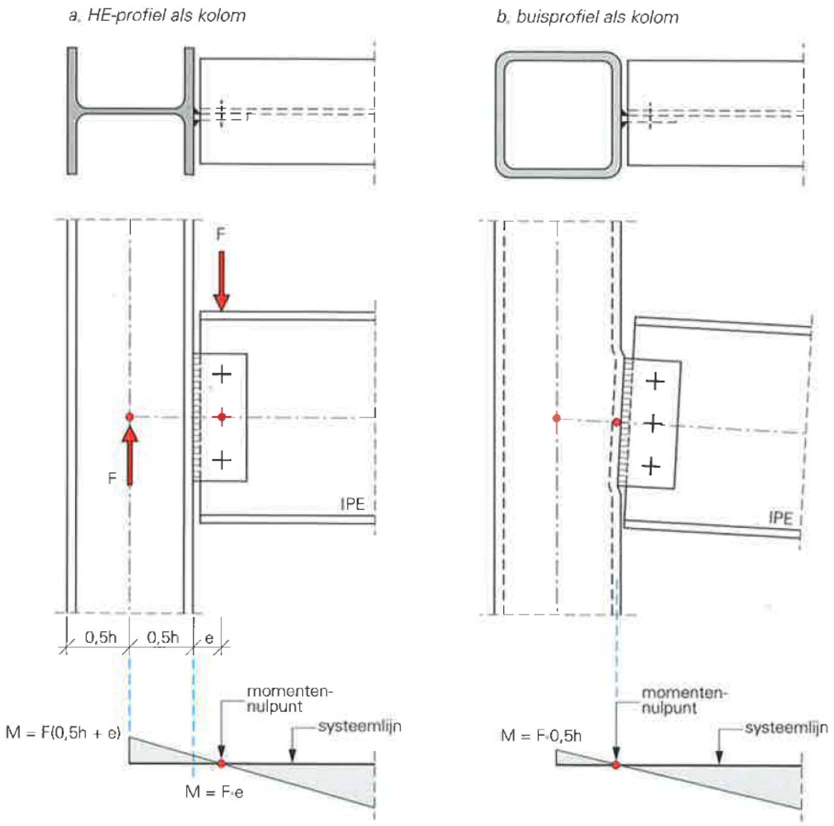


Figure 6.25: Rotation centre and internal bending moment in the joint [20]

In contrast to other comparisons first the partial FEA model is made, after that the manual calculation is done using the Eurocode 3 and CIDECT Recommendations.

6.5.1 Partial FEA model FPJV

First the model is set up according the figure depicted in figure 6.25. The bending moment is zero at the connection of the fin plate and face of the SHS. As can be seen in the illustration, the same phenomenon can be noticed as in the first model of the double angle cleat joint, which is unrealistic (figure 6.21).

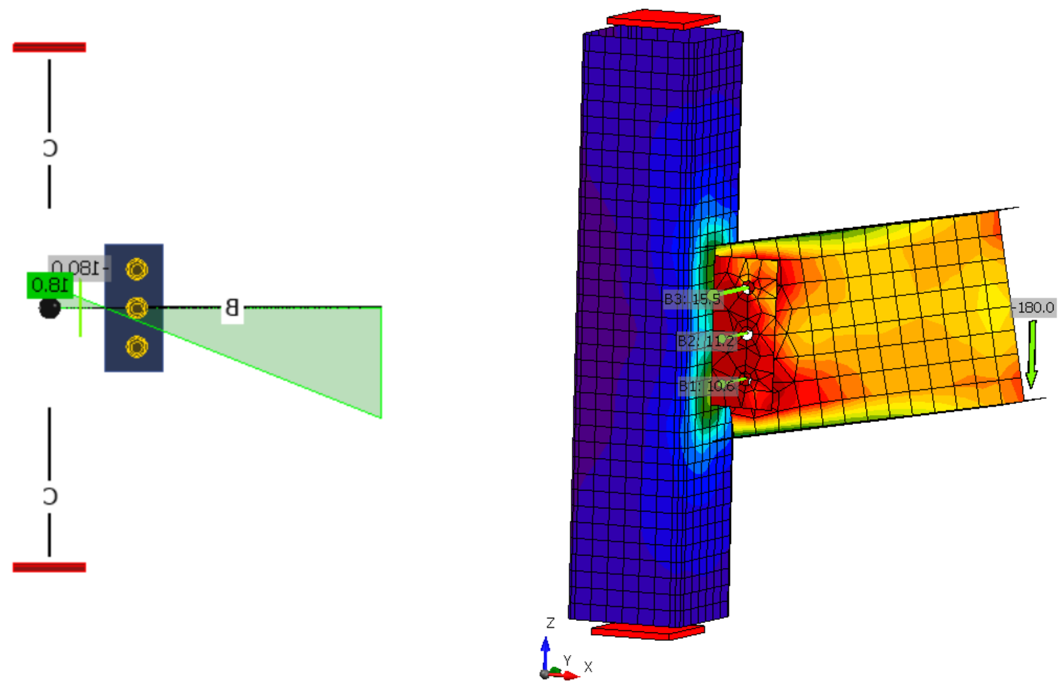


Figure 6.26: Model with zero point of the bending moment at the fin plate - SHS face connection

Previous model (CA3) has shown that by transferring load to the zero point of the bending moment in the joint to the bolt pattern in the fin plate, resulted in a more realistic behaviour of the joint. For this reason also in this case the shear load is applied on bolt row in the fin plate.

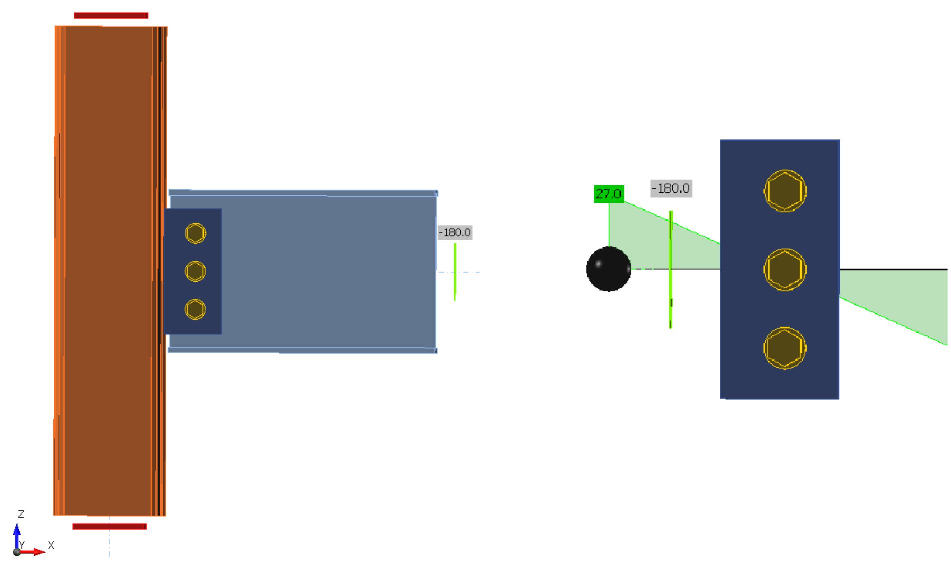


Figure 6.27: Model of the joint and how the load is applied on the joint.

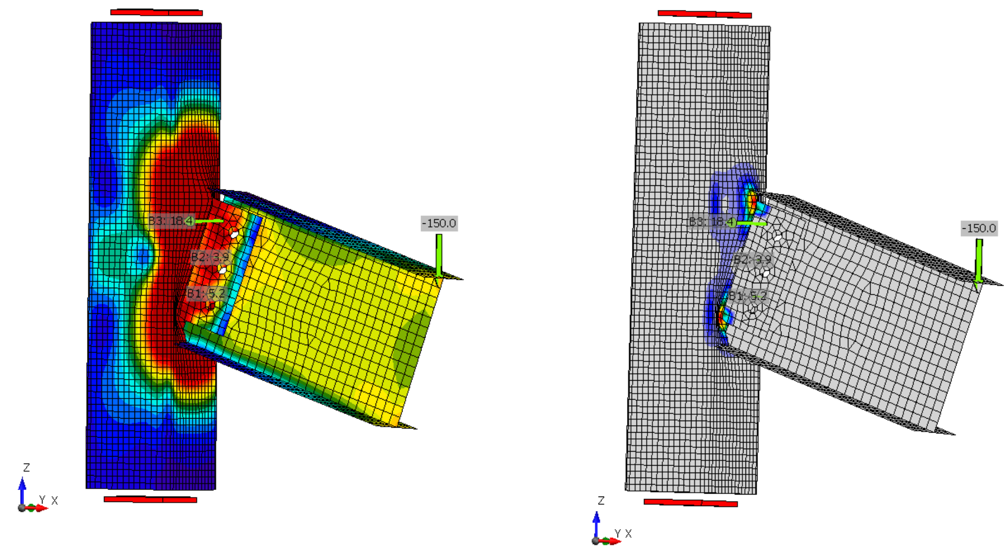


Figure 6.28: Stress distribution and peak strain in the joint.

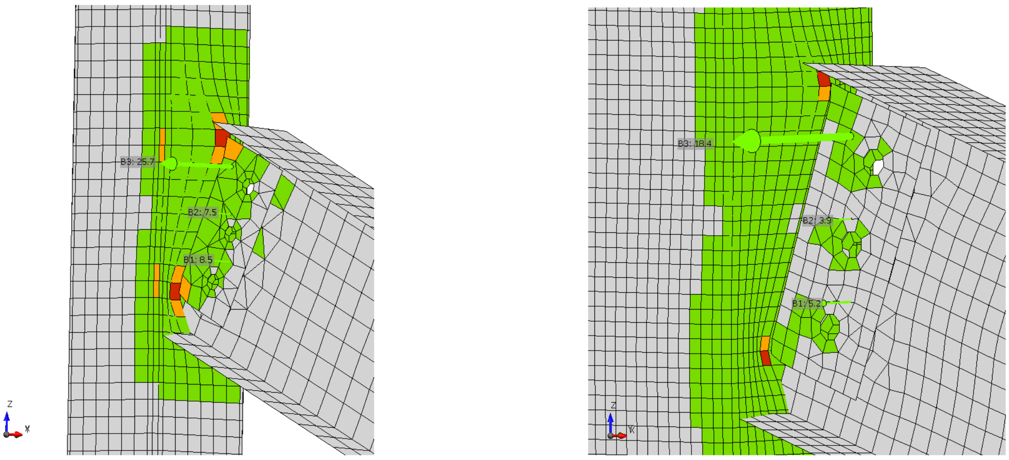


Figure 6.29: Decisive elements using standard mesh (left) V_{Ed} = 200 kN. Decisive elements using a refined mesh (right) at V_{Ed} = 160 kN

Using the standard settings of the software a model resistance can be obtained of V_{Rd} = 185 kN. Stress concentrations occur at the edges of the fin plate as expected. Also strains and deformation out of the plane of the column face can be noticed. After mesh refinement, resizing of the elements in smaller parts, the model resistance decreases significantly. Failure of some small resized elements occur at a lower applied force at the location of the stress concentration (see figure 6.29). The model resistance after refinement is V_{Rd} = 155 kN.

6.5.2 Manual Calculation FPJV (EC3 and CIDECT)

Following the prescriptions of the EC3, first the stresses on the chord face should be determined. These calculated stresses are of importance in further calculation, to determine the resistance of the SHS column. High stress in the column face, reduce the resistance of the SHS (k_m-factor, see figure 6.30). Iterations are needed to determine the final resistance of the SHS.

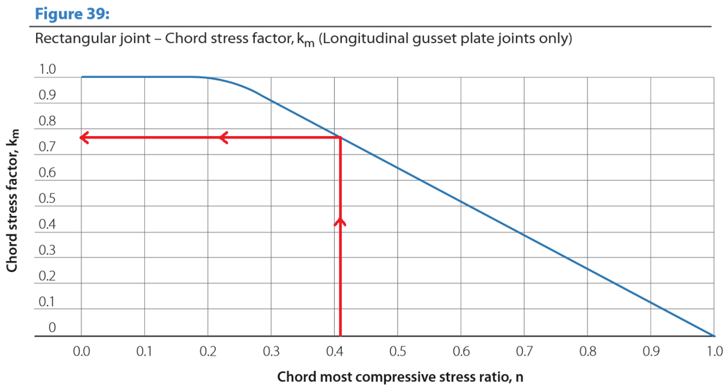


Figure 6.30: Relation stress ratio n and chord stress factor k_m. [24]

The CIDECT recommendation is also using the stress ratio factor n for determining the resistance. This factor can be found in the Q_f function.

$$N_{1,Rd} = 2 * f_{y0} * t_0^2 * [\eta + 2\sqrt{1 - \frac{t_1}{b_0}}] * Q_f, \quad where \quad Q_f = (1 - |n|)^{C_1} \tag{6.20}$$

Comparing both calculation methods the following resistance values are obtained for the chord face resistance of the SHS:

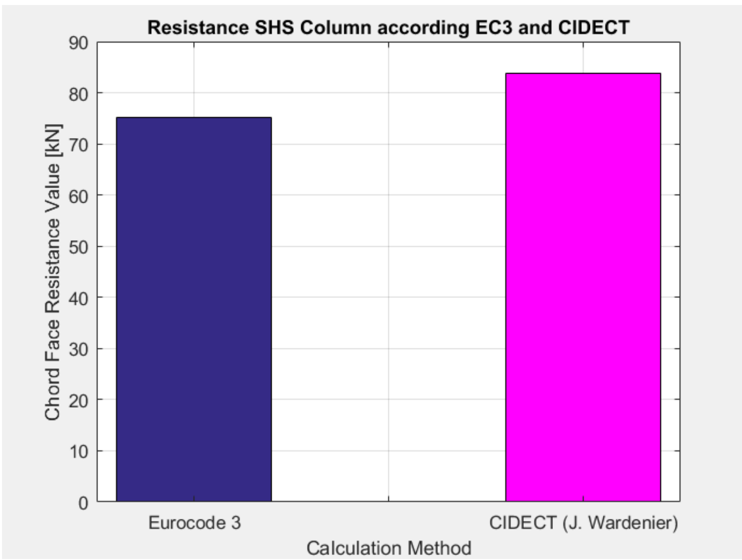


Figure 6.31: Comparison chord face resistance EC3 and CIDECT

Because the stress is of influence for the resistance for manual calculations, unity checks are executed. Not only the axial resistances need to be checked, also moment resistances caused by eccentricity. The Eurocode calculation gives a lower resistance than the CIDECT calculation. For the unity checks a design load is chosen of V_{Ed} = 155 kN. For this design value the unity check of the column resistance (EC3) approaches a value of 1.0.

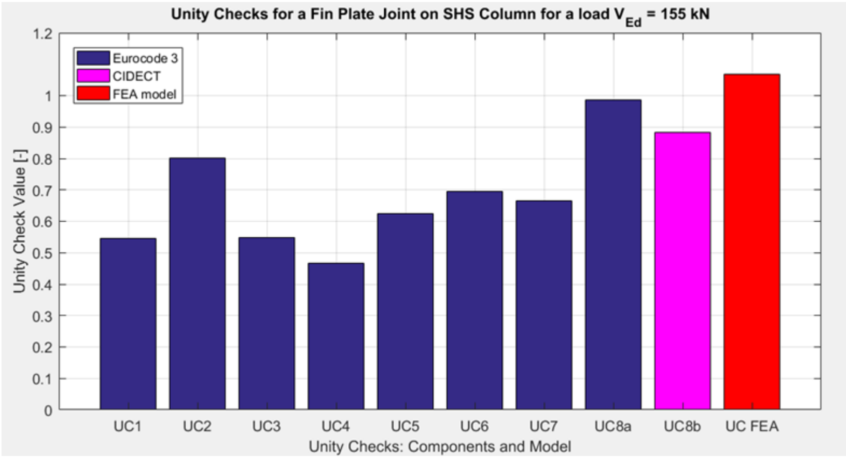


Figure 6.32: Unity checks components according EC3, CIDECT and FEA model

Explaination abbreviations in figure:

- Utilization shearing resistance of the beam web (UC1)
- Utilization tearing resistance of the beam web (UC2)
- Utilization shearing resistance of the of the bolts in fin plate (UC3)
- Utilization bearing resistance of the fin plate (UC4)
- Utilization bearing resistance of the beam web (UC5)
- Utilization shearing resistance of the fin plate (UC6)
- Utilization tearing resistance of the fin plate (UC7)
- Utilization chord face resistance, Eurocode 3 (UC8a)
- Utilization chord face resistance, CIDECT (UC8b)
- Utilization partial FEA model resistance (UC FEA)

6.5.3 Comparison Manual Calcualtions and FEA model

The calculations have shown that the square hollow section is in all cases the weakest component of the joint. For the EC3 calculation as for the CIDECT calculation the stress on the chord face of the SHS is of importance to determine the resistance. Settings of the mesh of the FEA model are of importance for the final results. Using the standard mesh configuration of the software, this will result in the highest resistance (of the three calculation methods), while a fine mesh configuration drastically decreases the model resistance below a value obtained by the EC3 and CIDECT calculation.

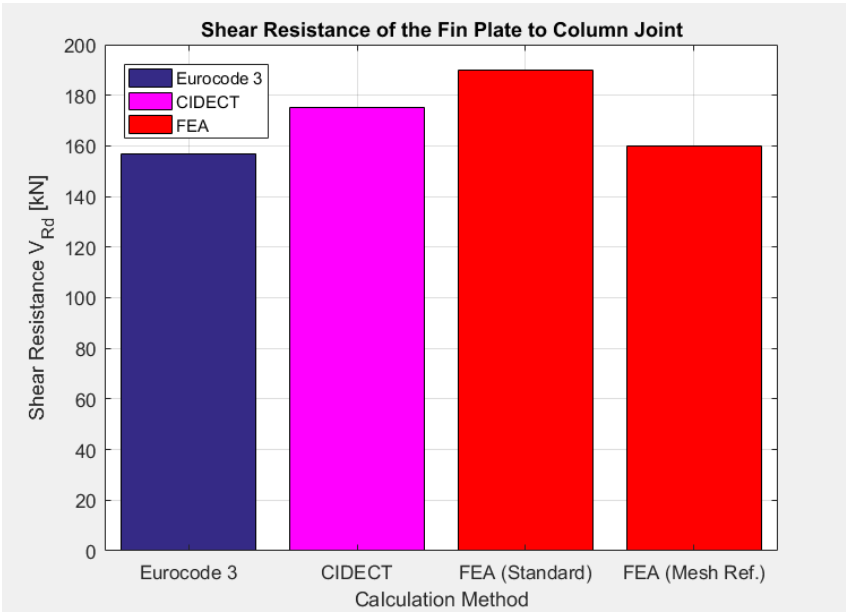


Figure 6.33: Overview joint resistances (EC3, CIDECT, FEA standard mesh, FEA refined mesh)

Chapter 7

Moment Resisting Joints

Moment Resisting Joints (MRJs) are steel joints which are designed to resist a bending moment (in combination with an occurring shear and/or normal force). Moment resisting joints are designed in structural frameworks where the overall stability of the structure in not provided by structural bracing.

7.1 General Procedure Calculation of a Moment Resisting Joint

A moment resisting joint with a bolted end plate is a frequently designed joint. The design of a beam/column connection can be subdivided into three zones.

1. tension zone
2. shear zone
3. tension zone

All three zones must satisfy component specific checks to obtain a proper connection. For a standard bolted moment connection with end plates the following checks need to be done.

Tension zone:
A. Column flange in bending (EC3: NEN-EN 1993-1-8; art. 6.2.6.4) For the check on column flange in bending the failure mechanisms are used of a T-stub. Which are 1. Yielding of the flanges, 2. Bolt failure under tension, 3. Bolt failure.

$$F_{T,1,Rd} = \frac{4M_{pl,Rd}}{m} \tag{7.1}$$

$$F_{T,2,Rd} = \frac{2 * M_{pl,Rd} + 2 * n * \sum F_{t,Rd}}{m + n} \tag{7.2}$$

$$F_{T,3,Rd} = \sum F_{t,Rd} \tag{7.3}$$

B. End-plate in bending (EC3: NEN-EN 1993-1-8; art. 6.2.6.5) The resistance value of the end plate is calculated with the following formula:

$$F_{t,wc,Rd} = \frac{\omega * b_{eff,c,wc} * t_{wc} * f_{y,wc}}{\gamma_{M0}} \tag{7.4}$$

C. Column web in transverse tension (EC3: NEN-EN 1993-1-8; art. 6.2.6.5)

$$F_{t,wb,Rd} = \frac{b_{eff,t,wb} * t_{wb} * f_{y,wb}}{\gamma_{M0}} \tag{7.5}$$

D. Beam web in tension (EC3: NEN-EN 1993-1-8; art. 6.2.6.8),

$$F_{t,wb,Rd} = \frac{b_{eff,t,wb} * t_{wb} * f_{y,wb}}{\gamma_{M0}} \tag{7.6}$$

E. Bolts in tension (EC3: NEN-EN 1993-1-8; art. 3.6 T3.4 & art. 6.2.6.4),

$$F_{t,Rd} = \frac{k_2 * f_{ub} * A_s}{\gamma_{M2}} \tag{7.7}$$

F. Welds (EC3: NEN-EN 1993-1-8; art. 4 and art. 6.2.3. Full strength welds can be applied or welds can be designed based on the derived design stresses. To design full strength welds (plastic calculation), with use of the directional method, the formulas in the table below can be used to determine the throat thickness [20]. Applying these rules no checks of the welds are needed because the stress in the connecting plate material is decisive.

	S235	**S275**	**S355**	Caculation Method
$\sigma_x = f_y$	a > 0.46t	a > 0.48t	a > 0.58t	plastic calculation

If there is no requirement that mother material should be decisive, the formula of the directional method can be applied using the design stresses.

$$\sqrt[2]{\sigma^2 + (\tau_\perp^2 + \tau_\parallel^2)} \leq \frac{f_y}{\beta_w * \gamma_{M2}} \tag{7.8}$$

Shear zone:

F. Column web panel in shear. (EC3: NEN-EN 1993-1-8; art. 6.2.6.1). The column web must be checked on shear resistance with the following formula.

$$W_{wp,Rd} = \frac{0.9 f_{y,wc} A_{vc}}{\sqrt{3}\gamma_{m0}} \tag{7.9}$$

Compression zone:

G. Column web in transverse tension (EC3: NEN-EN 1993-1-8; art. 6.2.6.2) Compression has the risk of buckling failure. There are several buckling failures that can occur and which must be checked.

- Local buckling, local buckling is a failure of the transition of from the flange to the web.
- Global buckling, global buckling is buckling of the web / plate.
- Sway, sway is lateral displacement of the flanges.

Those failure modes can be checked with the following formula:

$$F_{c,wc,Rd} = \frac{\omega * k_{w,c} * b_{eff,c,wc} * t_{wc} * f_{y,wc}}{\gamma_{M0}} \leq \frac{\omega * k_{w,c} * \rho * b_{eff,c,wc} * t_{wc} * f_{y,wc}}{\gamma_{M1}} \tag{7.10}$$

H. Beam web or beam flange and web in compression (art. 6.2.6.7). The beam can resist a maximum moment in the flanges, where from a maximum resistance force can be derived.

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h_b - t_{fb}} = \frac{\frac{W_{pl} * f_y}{\gamma_{m0}}}{h_b - t_{fb}} \tag{7.11}$$

7.2 Bolted Extended Endplate MRJ (BEEMRJ), symmetrical loaded

Moment resisting joints can be loaded symmetrical in case that two beams are connected to the column with identical load cases. In this situation no reduction of the resistance in the tension zone and compression zone is needed. The transformation factor beta (β) must be calculated if there is a difference in bending moment according NEN-EN 1993-1-8, art. 5.3(9).

$$\beta_1 = |1 - \frac{M_{j,b2,Ed}}{M_{j,b1,Ed}}| \leq 2 \tag{7.12}$$

$$\beta_2 = |1 - \frac{M_{j,b1,Ed}}{M_{j,b2,Ed}}| \leq 2 \tag{7.13}$$

if the difference in bending moment is small and the beta values stay lower than 0.5 ($\beta < 0.5$), no reduction is needed to determine the component resistance.

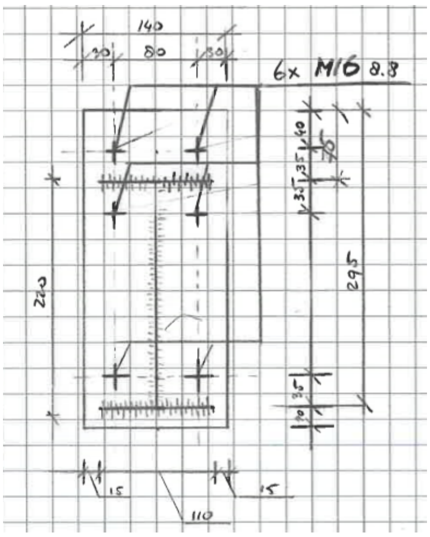


Figure 7.1: Joint Configuration

7.2.1 Joint configuration and assumptions, BEEMRJ symmetrical loaded

The following configuration of a bolted extended endplate moment resisting joint is calculated according to the Eurocode 3. The following assumption have been made for the manual calculation:

- Configuration of the joint as depicted in figure 7.1.
- Moment resisting joint, Upper two bolt rows take up tension force. Compression force is transferred from the lower beam flange through the end plate, to the column.
- Rotation centre is assumed in the contact point between the column flange and the end plate at the height of the middle of the beam flange.
- Plastic calculation, plastic development of the bolt pattern.
- Symmetrical loading, transformation parameter $\beta = 0$.
- Reduction factor $k_{wc} = 1.0$ is assumed.

7.2.2 Manual Calculation, BEEMRJ symmetrical

In the manual calculation the tension zone and compression zone are considered. The following resistance values are found for the tension zone.

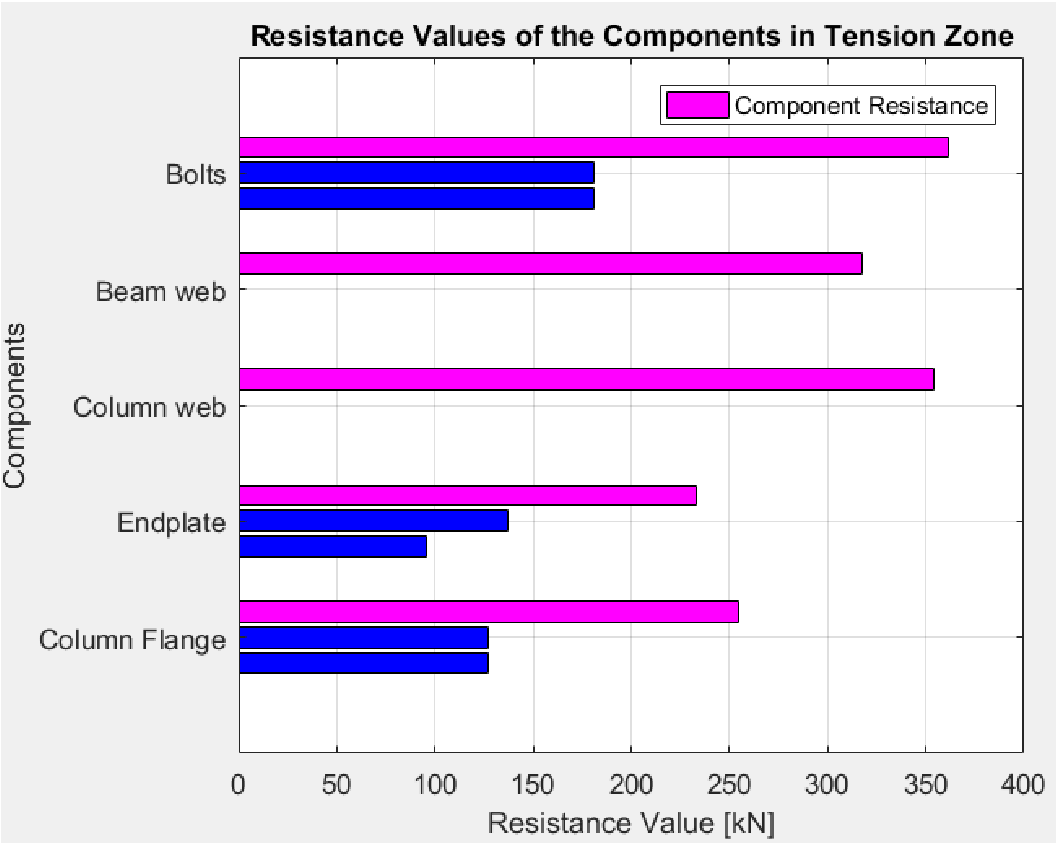


Figure 7.2: Results of the components in tension according EC3

In the tension zone the end plate is the governing component. Using the NEN-EN 1993-1-8 end plate in bending is the decisive failure mechanism. To calculate the plastic moment resistance the governing yield pattern (figure 7.3) must be determined separately for the bolt row above the beam profile and inside the profile. The plastic moment resistance derived from the governing yield pattern is used in the T-stub calculations to determine the decisive failure mechanism.

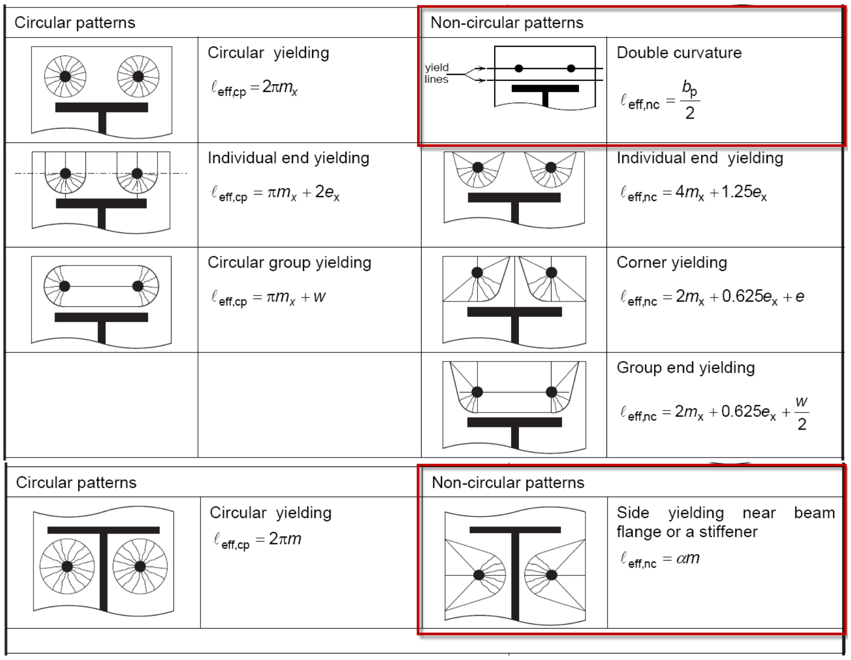


Figure 7.3: Governing yield patterns in end plate

For a symmetrical load case also the compression zone of the joint should be evaluated. Which can be seen in figure 7.4.

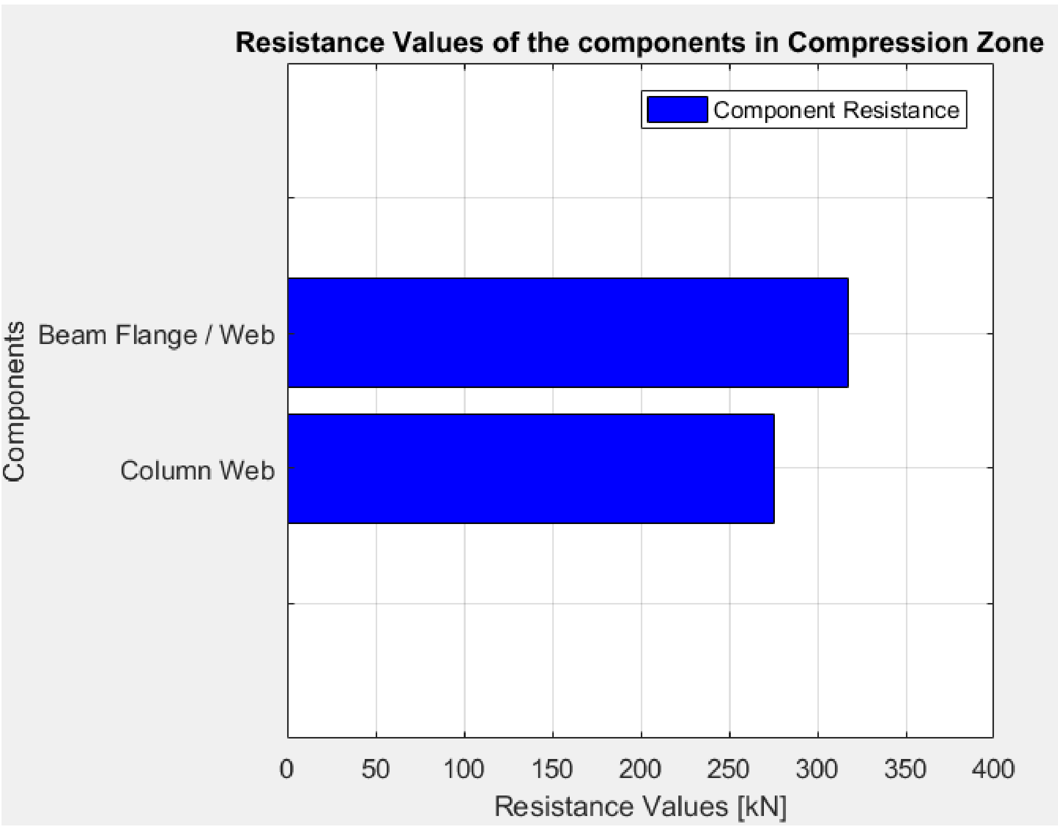


Figure 7.4: Results of the components in tension according EC3

Tot determine the moment resistance it is necessary to check which zone is governing. If the tension resistance is lower than compression, no reduction is needed. If the compression zone is governing bolt forces should be reduced to obtain horizontal equilibrium. In this case the tension zone is governing, moment resistance can be calculated by simply multiplying the tension resistance for each bolt row with the corresponding leverage arm.

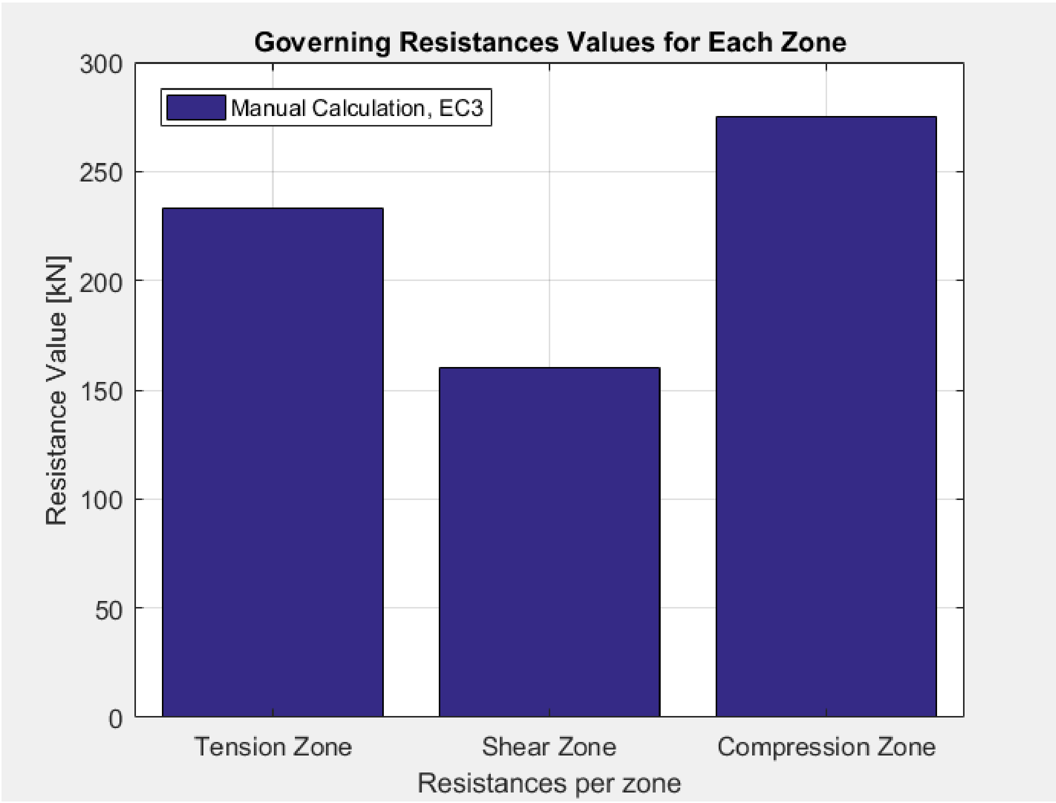


Figure 7.5: Overview of the different resistance values of each zone

7.2.3 Partial FEA Model, BEEMRJ symmetrical

A partial FEA model has been made with the same configurations and the same material properties, prescribed by the EC3. The determine which maximum resistance of the model the load on the partial FEA model is increased sequentially till the first limit values are reached. The load case before failure is defined as the model resistance $M_{Rd,model}$

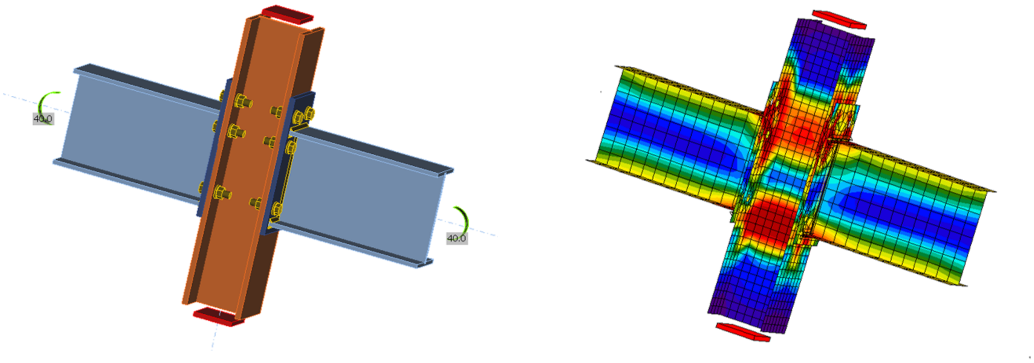


Figure 7.6: Layout Partial FEA Model, Stress distribution in FEA model at model resistance $M_{Rd,model}$.

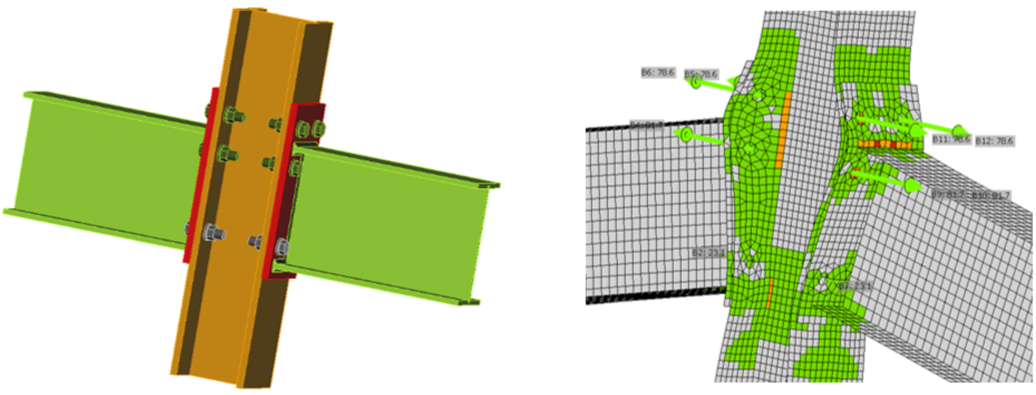


Figure 7.7: Governing component (red) after exceeding model resistance ($M_{Rd,model}$), Strain plot of the 2D-elements (shell elements).

In the model you can see the stress concentration in the tension zone and compression zone of the joint. The extended end plate is the governing component of the joint. The shell elements near the welds at top flange of the beam are exceeding the limit strain of 5% during the sequential load increase.

7.2.4 Comparison Manual Calculation and Partial FEA model

For the manual calculation as for the partial FEA model the end plate is the weakest component. The non-circular, double curvature, yield pattern is governing. In the FEA model at location of the yield line, strain developed can be observed, which finally will lead to failure of the model. For the manual calculation and the model the following moment resistance values were obtained:

- Resistance of the manual calculation: $M_{Rd,EC3} = 48$ kNm
- Resistance of the partial FEA model $M_{Rd,model} = 45$ kNm.

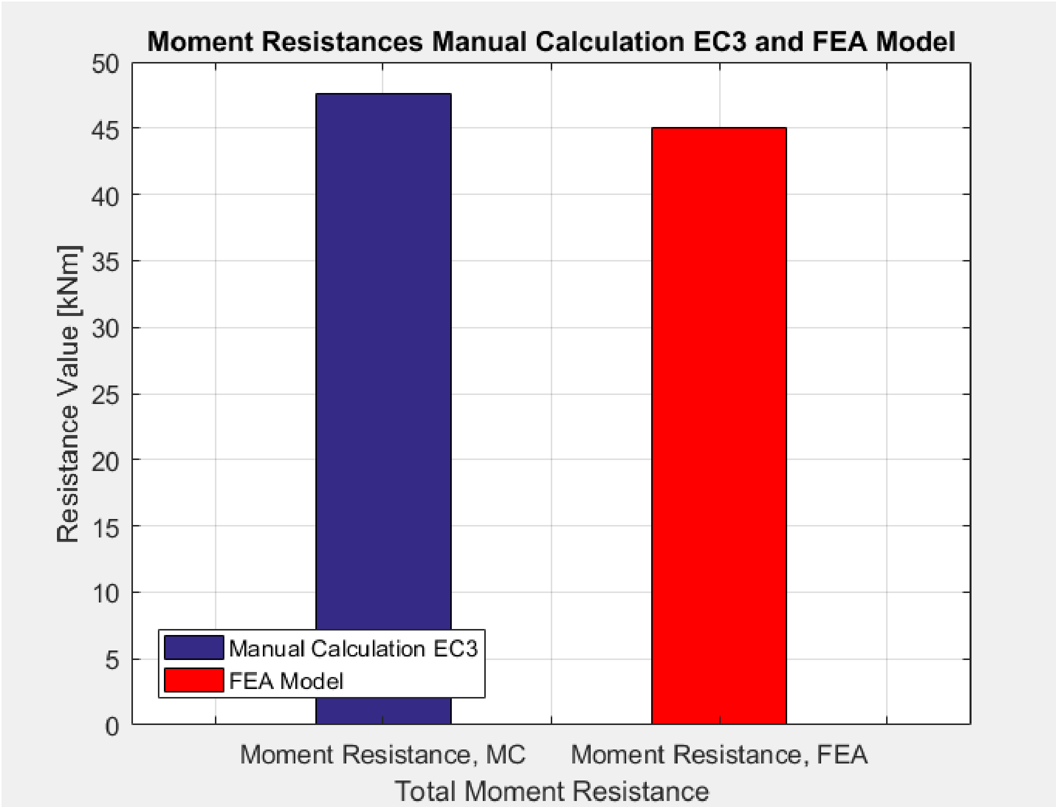


Figure 7.8: Comparison manual calculation EC3 and FEA model

7.3 Bolted Extended Endplate MRJ (BEEMRJ), asymmetrical loaded

A moment resisting joint in a situation where only one beam is connect to the column can be asymmetrical loaded. Also a MRJ where the beam is loaded by two different bending moments is asymmetrical loaded. If the difference in bending is large, $\beta > 0.5$, reduction is needed in tension en compression zone. Depending on the beta value (β), the reduction factor omega must be applied (ω_1, ω_2). If a MRJ is purely symmetrical loaded the shear zone is not governing. The joint must only resist the introduction of tensile and compression forces. In a asymmetrical load scenario the shear zone of the joint should be evaluated as well. The configuration of the MRJ which is manually calculated and modelled with FEA software is depicted in figure 7.9. The elaboration of this manual calculation and the MATLAB script can be found in the annex B.

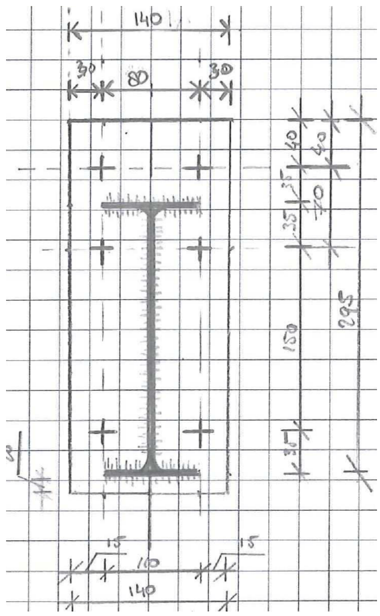


Figure 7.9: Joint Configuration

7.3.1 Joint configuration and assumptions, BEEMRJ asymmetrical

The following joint configuration is used for the manual calculation and partial FEA model. The following assumptions are made for the manual calculation:

- Configuration of the joint as depicted in figure 7.9.
- Moment resisting joint, upper two bolts rows take up tension force. Compression force is transferred from the lower beam flange through the end plate, to the column.
- Rotation centre is assumed in the contact point between the column flange and the end plate at the heigth of the middle of the beam flange.
- Plastic calculation, plastic development of the bolt pattern.
- Asymmetrical loading, transformation paramater $\beta = 1$.
- Reduction factor $k_{wc} = 1.0$ is assumed.

7.3.2 Manual Calculation BEEMRJ, asymmetrical

In the manual calculation the tension zone, shear zone and compression zone are considered. Because of the asymetrical loading the transformation factor beta (β) is equal to one. Consequence is that reduction factor omega (ω_1) must be calculated accoording NEN-EN 1993-1-8, art. 6.2.6.2 table 6.3. This reduction factor must be used to reduce resistance in tension and compression of the column web. Plate buckling of the column web is considered, no reduction is needed; the plate slenderness $\overline{\lambda_p}$ remains under the limit value; no reduction is needed for buckling. Because of the asymmetrical loading shearing should be taken into account. Column web in shear is the governing component in this joint configuration. Configuration of the joint is the same, so the governing yield pattern which are found in previous calculation are also the same.

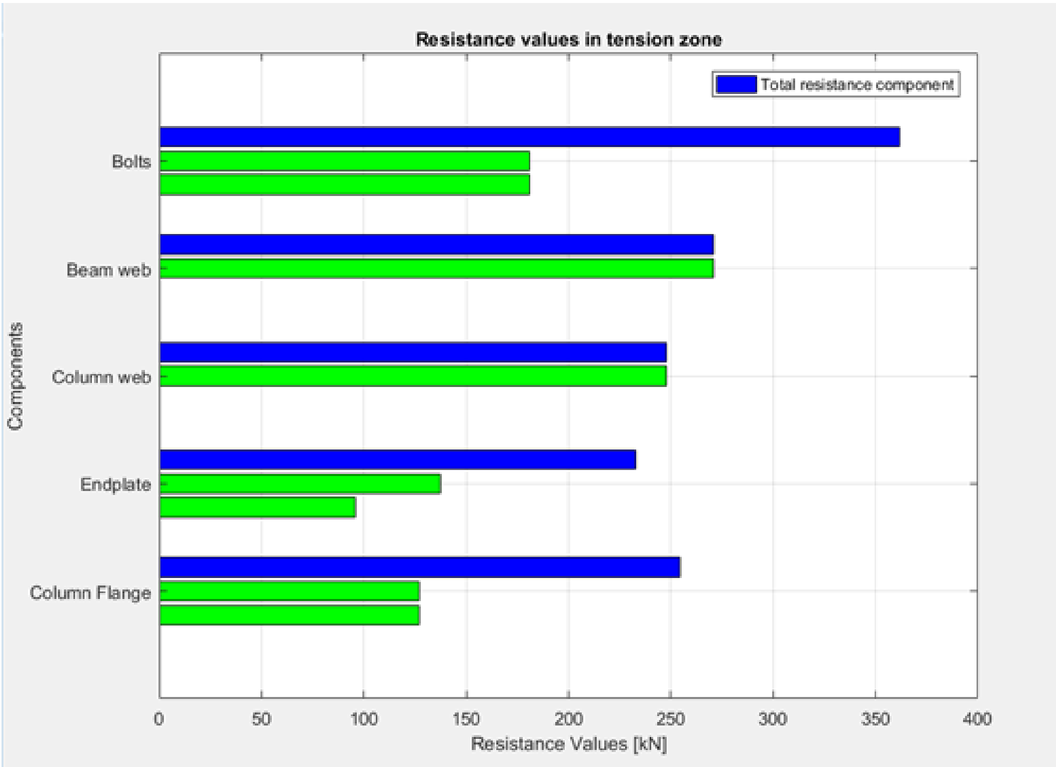


Figure 7.10: Results of the components in tension according EC3

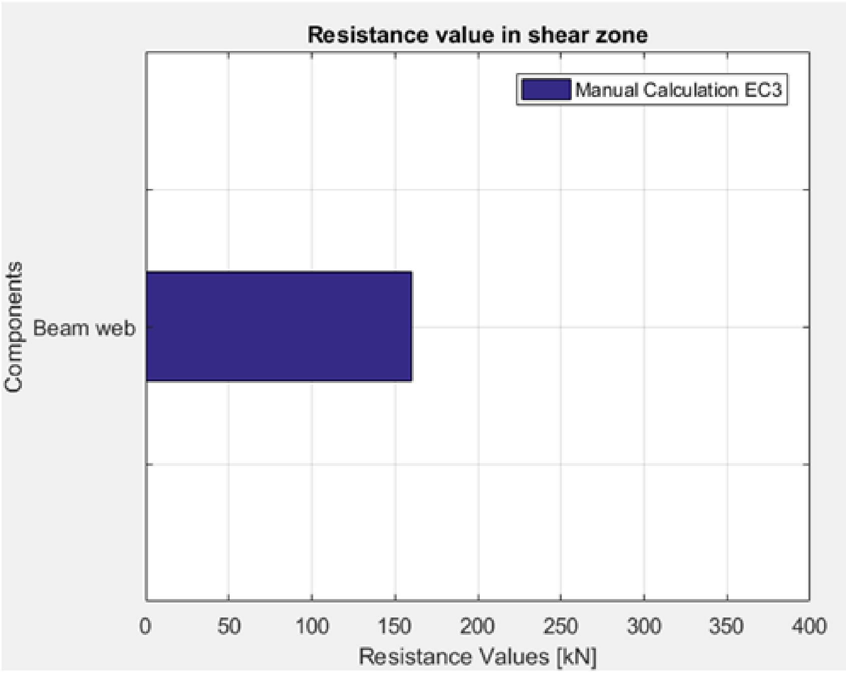


Figure 7.11: Result of the component in shear according EC3

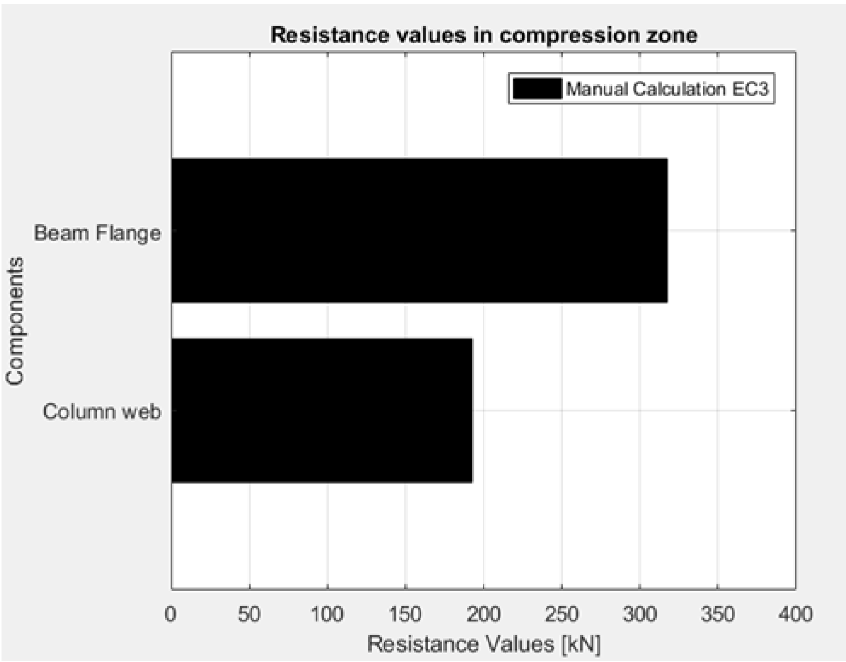


Figure 7.12: Results of the components in compression according EC3

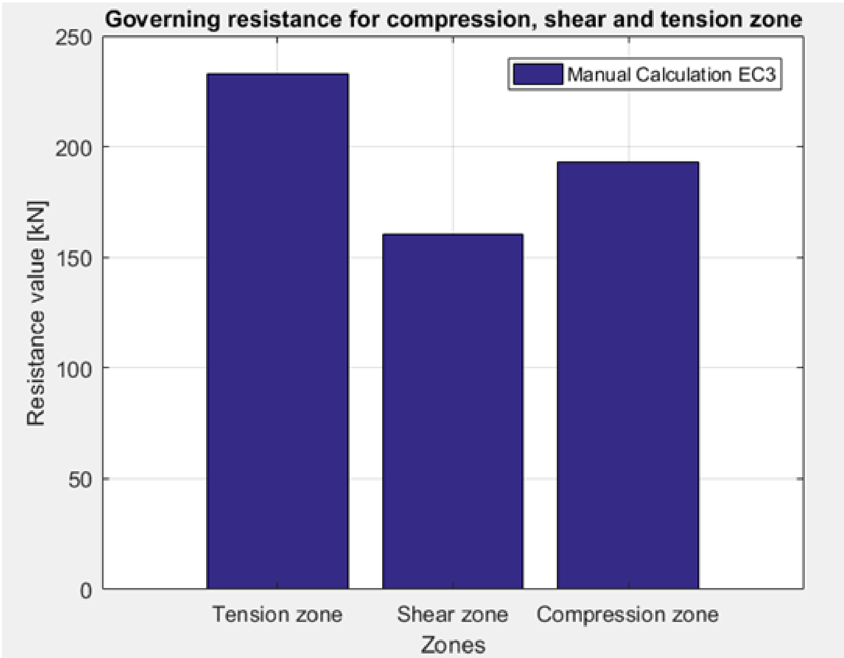


Figure 7.13: Results of the components in compression according EC3

7.3.3 Partial FEA Model BEEMRJ, asymmetrical

The bolted extended end plate moment resisting joint is modelled and is asymmetrical loaded. Because of the asymmetric loading different yield and strain contours occur. Where in a symmetrical load case stress and strain concentrations occur in the tension and compression zone, stress and strain concentrations now can be noticed in the shear panel of the column web. Although the moment resistance value is similar with previous calculation, failure occurs in the compression zone of the joint.

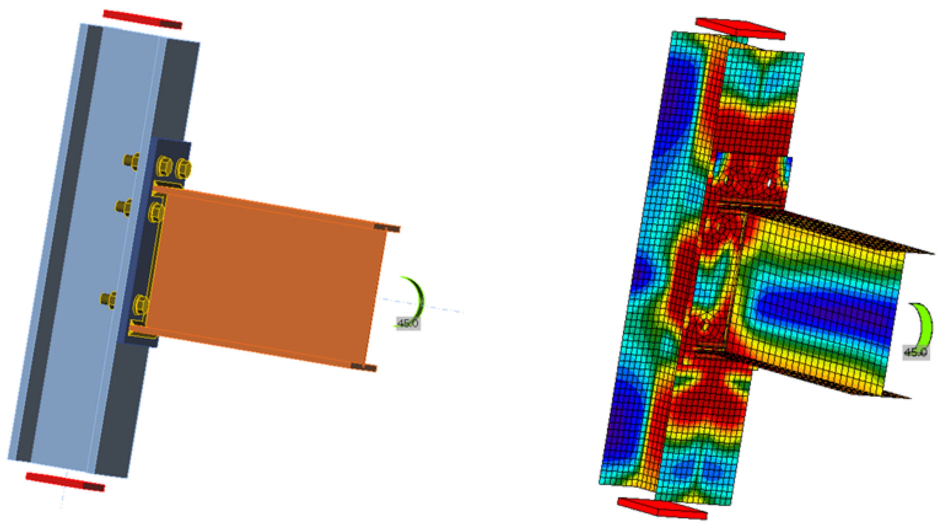


Figure 7.14: Results of the components in tension according EC3

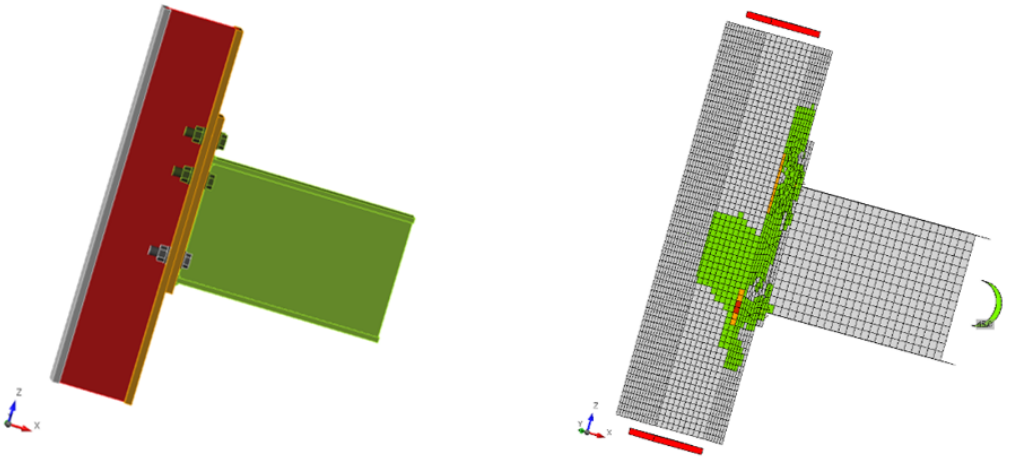


Figure 7.15: Results of the components in tension according EC3

7.3.4 Comparison Manual Calculation EC3 and FEA model

Compared to the symmetrical loaded joint, the asymmetric joint gives a similar moment resistance value. Only the failure mode is different which is shearing of the column. In the manual calculation the shear zone should be taken into account and is in this configuration governing. To obtain horizontal equilibrium the tension force must be reduced, which causes a reduction of the total moment resistance. For this joint configuration and load case the following moment resisting values were obtained:

- Resistance of the manual calculation: $M_{Rd,EC3} = 35$ kNm
- Resistance of the partial FEA model $M_{Rd,model} = 44$ kNm.

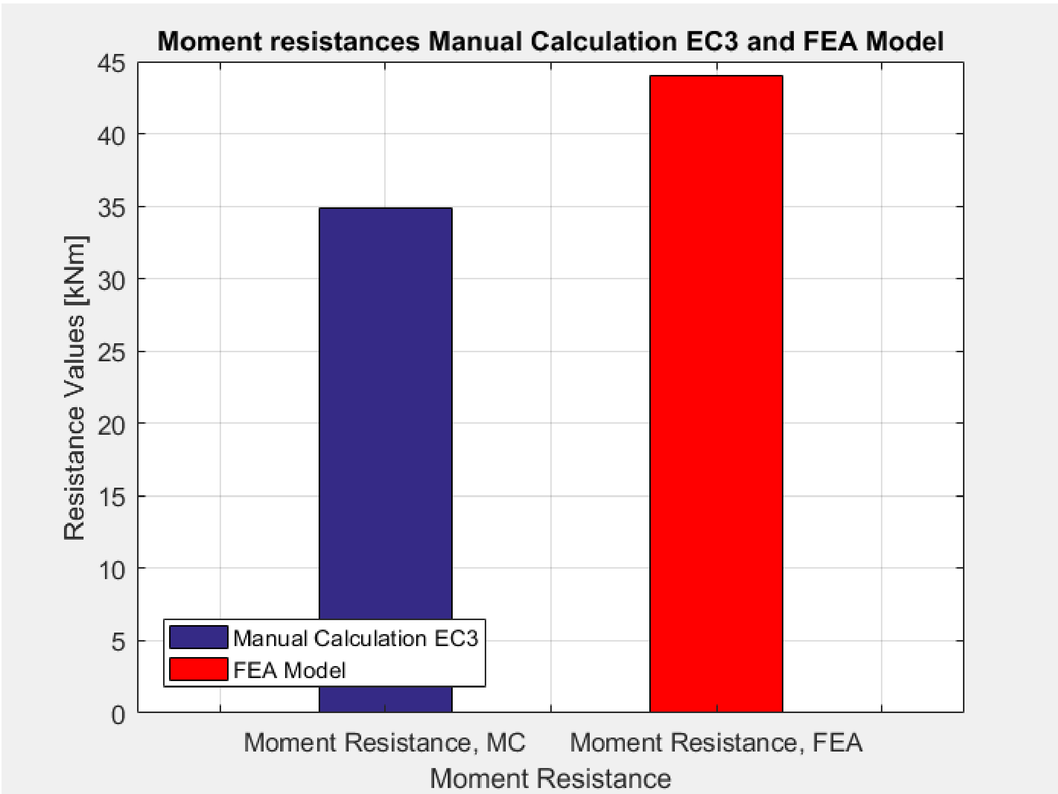


Figure 7.16: Comparison EC3 calculation and FEA model

7.4 Welded Moment Resisting Joint (WMRJ)

Bolted moment resisting joints are commonly used in structural projects. In general bolted joints are less expensive to fabricate. In some cases a welded moment connection is preferred over a bolted connection, in case of a small cantilever which must be attached to the column. Also in projects where many lifting movements is undesired, more structural elements are connected to each other by welding only.

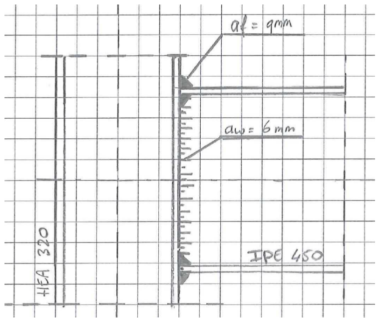


Figure 7.17: Joint Configuration

7.4.1 Manual Calculation WMRJ

In the manual calculation the tension zone, shear zone and compression zone is evaluated. Because this joint is asymmetrical loaded transformation factor and reduction factor omega (ω_1) should be applied. Further in this joint configuration also reduction for buckling effects (ρ) in the column web should be taken into account.

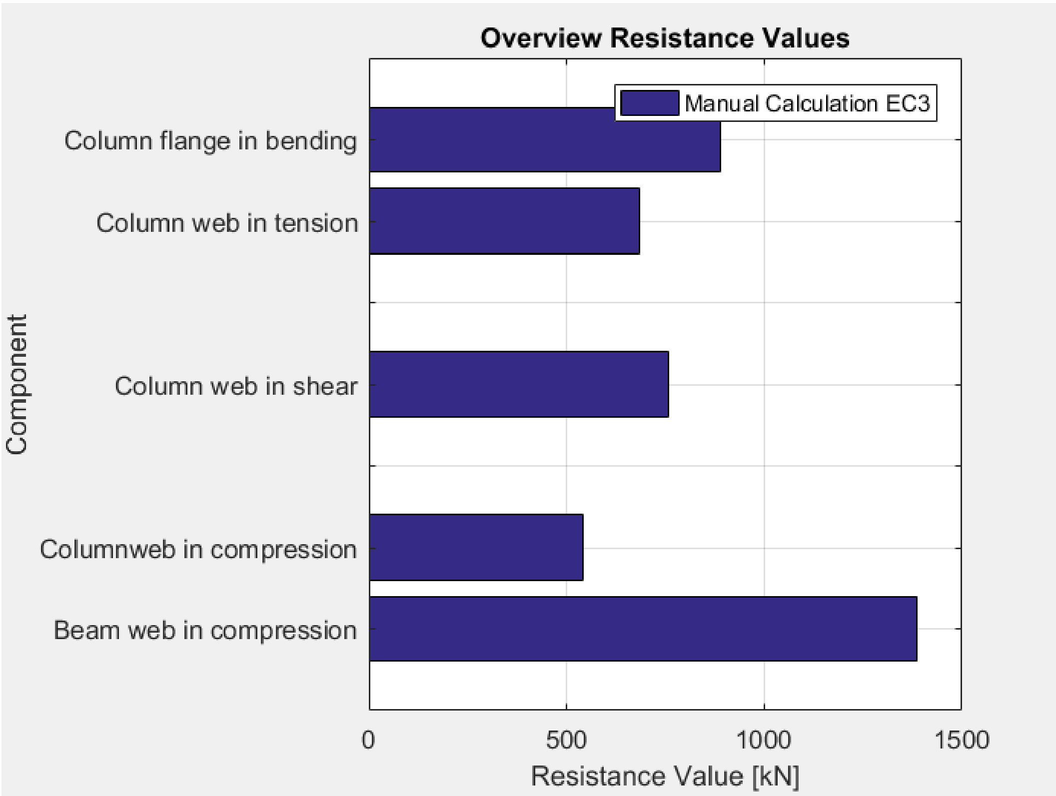


Figure 7.18: Overview resistance values of the joint components

7.4.2 Partial FEA model WMRJ

The welded moment resisting joint is modelled and asymmetrical loaded till the failure of the model occurs. The applied moment causes deformation in the tension zone and in the shear zone. A stress concentration and straining between in the column web is developing caused by shearing. At the beginning of the shear plane in the web the first elements are exceeding the 5 % limit value. Failure of the elements in tension zone and in compression zone occur simultaneously.

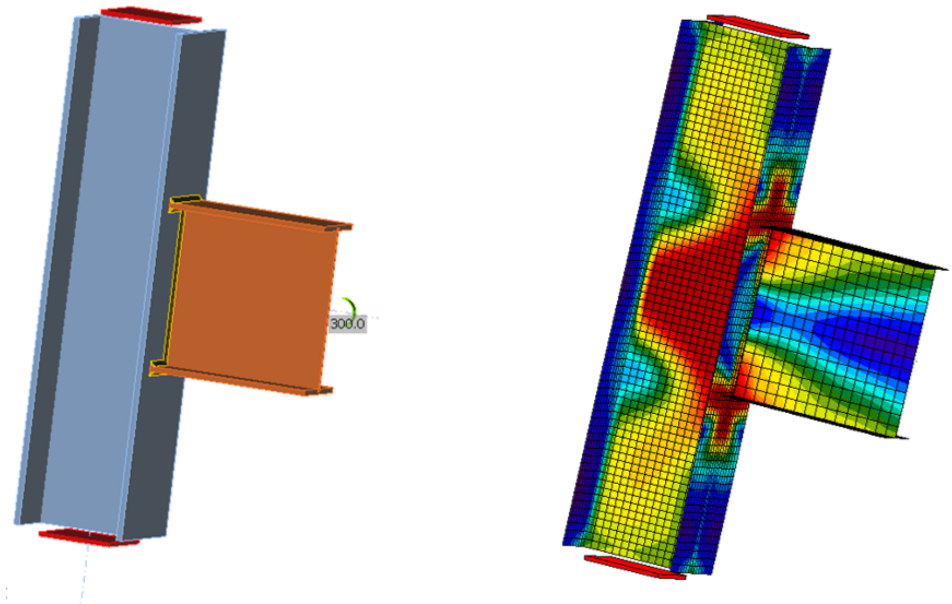


Figure 7.19: Results of the components in tension according EC3

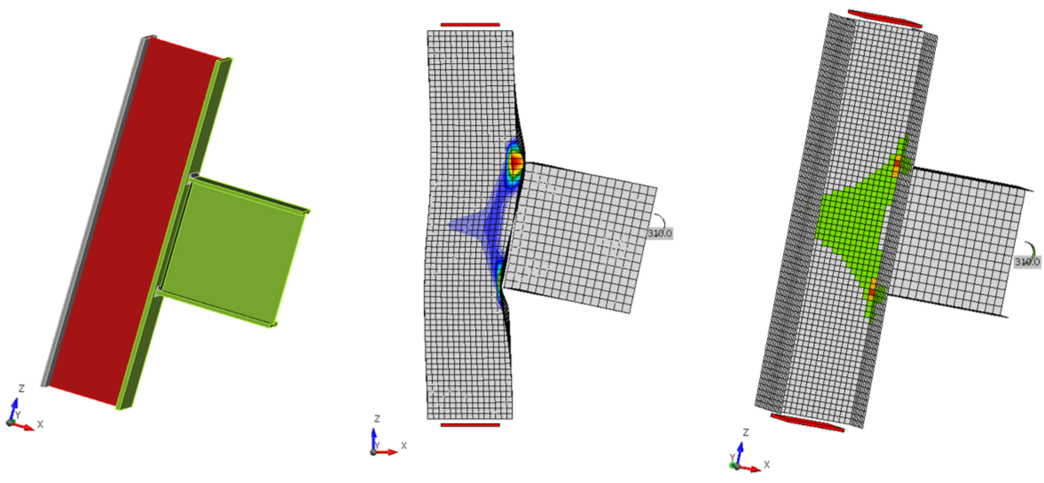


Figure 7.20: Results of the components in tension according EC3

7.4.3 Comparison Manual Calculation and partial FEA model

Comparing the manual calculation to the FEA model, manual calculation gives a lower resistance value than the FEA model. The compression zone is governing, the resistance is reduced by factor applied for asymmetrical loading, buckling sensitivity. In the FEA model buckling phenomena is excluded by the 5% limit strain. In the NEN-EN 1993-1-5 is stated that buckling cannot occur before this limit value.

For the welded moment resisting joint the following resisting values are obtained:

- Resistance of the manual calculation: $M_{Rd,EC3} = 235$ kNm
- Resistance of the partial FEA model $M_{Rd,model} = 300$ kNm.

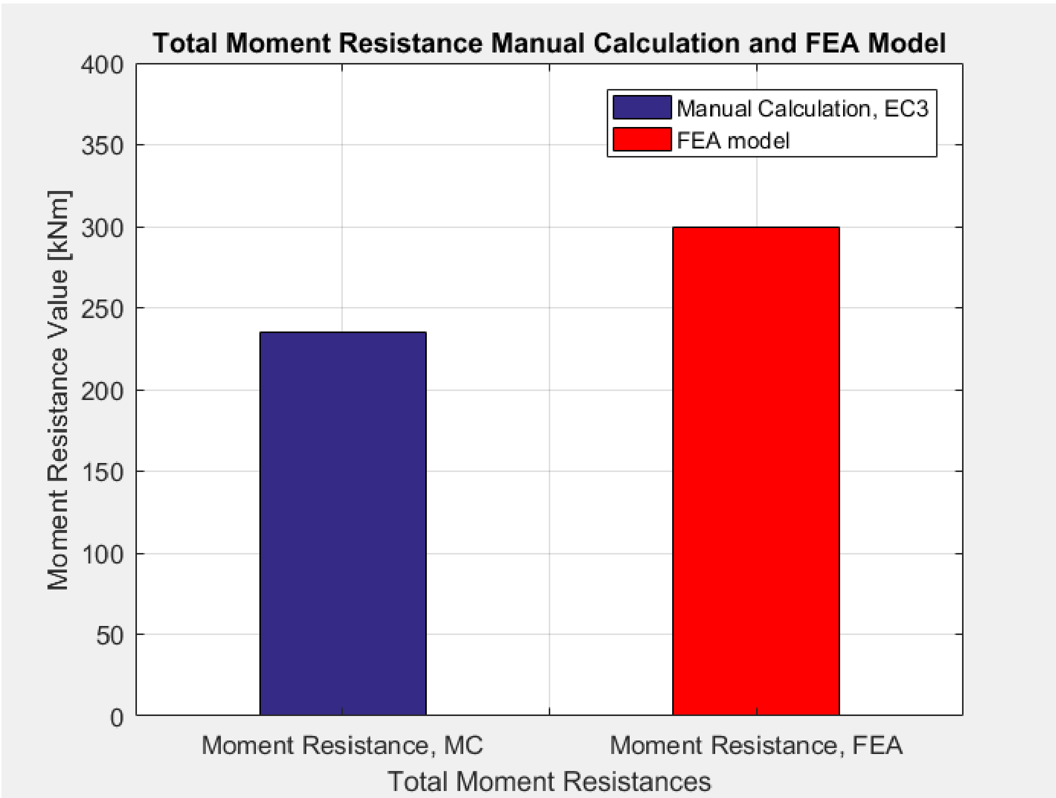


Figure 7.21: Moment resistances: Manual Calculation and FEA model

7.5 Validation Flush End Plate Moment Resisting Joint (FEMRJ)

Beside the comparison between manual calculation and FEA models also real experiments will be validated. This has been done for two type of joints: Flush end plate moment resisting joint and the extended endplate moment resisting joint.

7.5.1 Experiment and assumptions FEMRJ

Experiment has been executed according the experimental test set up depicted in figure 4.6. Because not all information about the experiment the following things are assumed such as:

- Full strength welds in the joints at the web and flanges, a = 6mm
- System length, $l_{sys} = 1.150m$, derived from a sketch of the test setup.
- Joint is loaded by a bending moment in combination with a shear force. The following mechanical scheme is used to determine the relation of shear force and bending moment, figure 7.22. The relation between moment ans shear force scheme follows from this scheme, which is $V_{Ed} = \frac{M_{Ed}}{l_{sys}}$
- Bolts are not mechanical tested predefined in contrast to the beam, endplate and column. In calculation EC3 values are used to determine the bolt resistance.

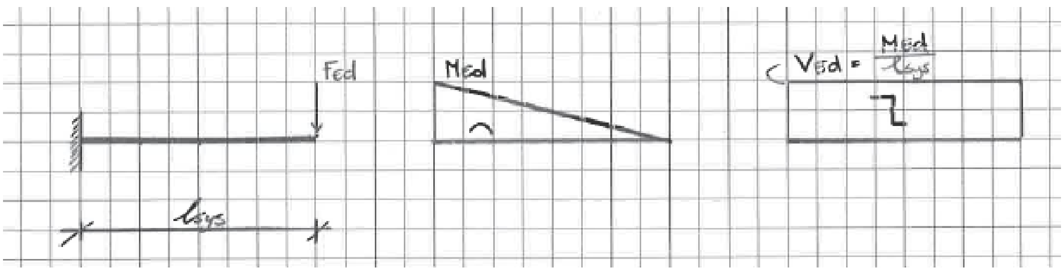


Figure 7.22: Mechanical scheme used to determine the shear load.

The joint configuration which is used in this experiment is depicted in figure 7.23. Specimen are tested the following material properties are obtained and presented in figure 7.24.

For the manual calculation and partial FEA model the following assumptions are made:

- Joint configuration as depicted in figure 7.23.
- Plastic calculation, using of the plastic moment resistance and plastic section modulus.
- Material properties for the manual calcualtion presented in figure 7.24.
- Upper bolt row inside the profile contribute to the tension resistance of the joint.
- Lower inside bolts contribute to the shear resistance of the joint.
- Compression force is transferred from the lower beam flange through the end plate, to the column.

- Rotation centre is assumed at the contact point between the column flange and (flush) end plate at the height of the lower beam flange.
- no safety factors are used for calculation.

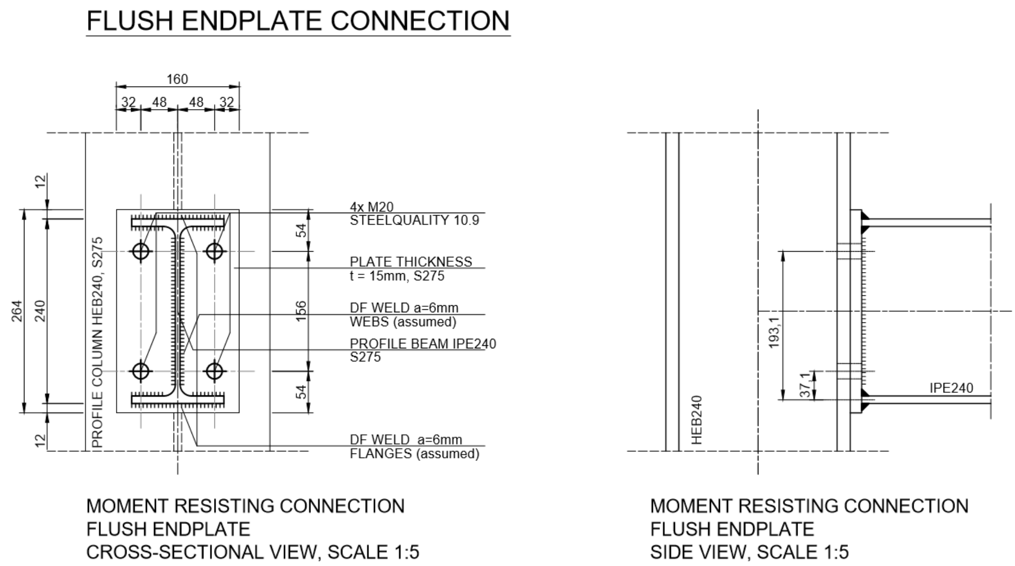


Figure 7.23: Joint configuration flush end plate joint

PROPERTIES JOINT ELEMENTS, FLUSH ENDPLATE						
IPE240	EXPERIMENTAL VALUES			DERIVED CALCUALTION VALUES		
YIELD STRENGTH	fy	351.7	N/mm2	fy	350	N/mm2
TENSILE STRENGTH	fu	451.3	N/mm2	fu	450	N/mm2
E-MODULUS	E	209468	N/mm2	E	210000	N/mm2
HEB240						
YIELD STRENGTH	fy	357.5	N/mm2	fy	360	N/mm2
TENSILE STRENGTH	fu	463.0	N/mm2	fu	460	N/mm2
E-MODULUS	E	213864	N/mm2	E	210000	N/mm2
END PLATE						
YIELD STRENGTH	fy	369.4	N/mm2	fy	370	N/mm2
TENSILE STRENGTH	fu	503.5	N/mm2	fu	500	N/mm2
E-MODULUS	E	200248	N/mm2	E	20000	N/mm2

Figure 7.24: Material properties elements flush end plate joint

7.5.2 Manual Calculation FEMRJ

The different components are executed according the calculation rules prescribed by the Eurocode. In contrast to other calculations not the predefined material properties are used, but the material properties which have been obtained by test on specimens. No material test were done on the bolts, for that reason predefined EC3 values are used in calculation. No partial safety factors are used in manual calculation and in the FEA model.

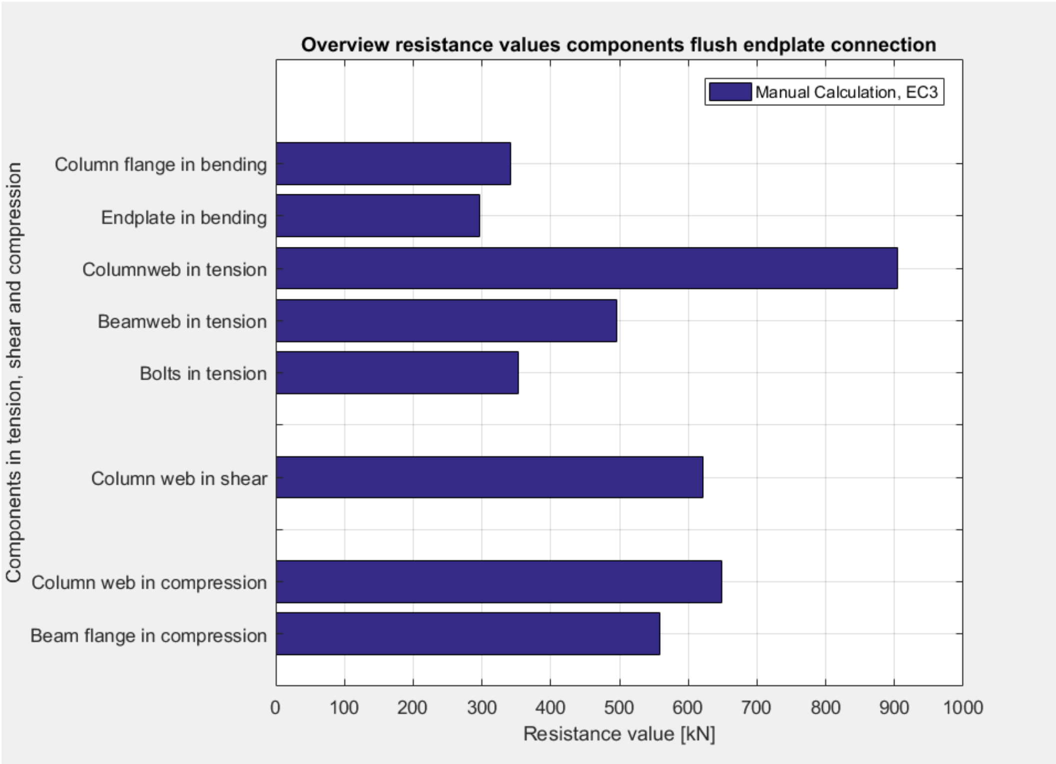


Figure 7.25: Overview resistance values components

The tension zone is governing, end plate in bending is the decisive failure mechanism in tension zone. According to the manual calculation the non-circular pattern is governing. Because the resistance in tension is governing the moment resistance can be directly calculated from the T-stub resistances multiplied with the leverage arm.

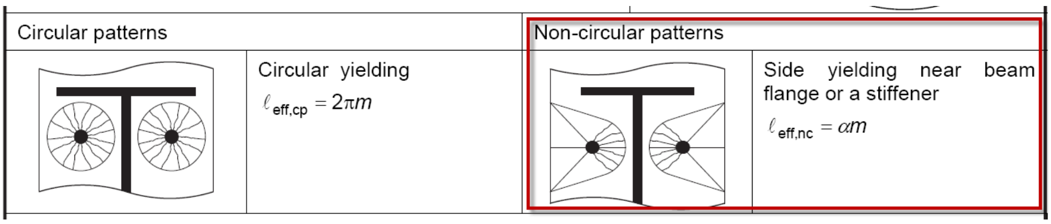


Figure 7.26: governing yield pattern end plate

7.5.3 Partial FEA model FEMRJ

A partial FEA model is made with the prescribed joint configuration. Because no information is given about the welds, full strength welds are assumed with at least the quality of the mother material. The experimental values are used to derive bi-linear stress-strain relations for the FEA calculation. For the column, end plate and beam different bi-linear stress-strain are used. According to NEN-EN 1993-1-8, art. 5.4.3(4) this model may be used for FEA modeling, but also more accurate stress-strain behaviour may be used. After sequentially load increase the strain increase was found developed in the tension zone of the connection. Strains in the beam flange, beam web and end plate were exceeding the 5 % limit strain.

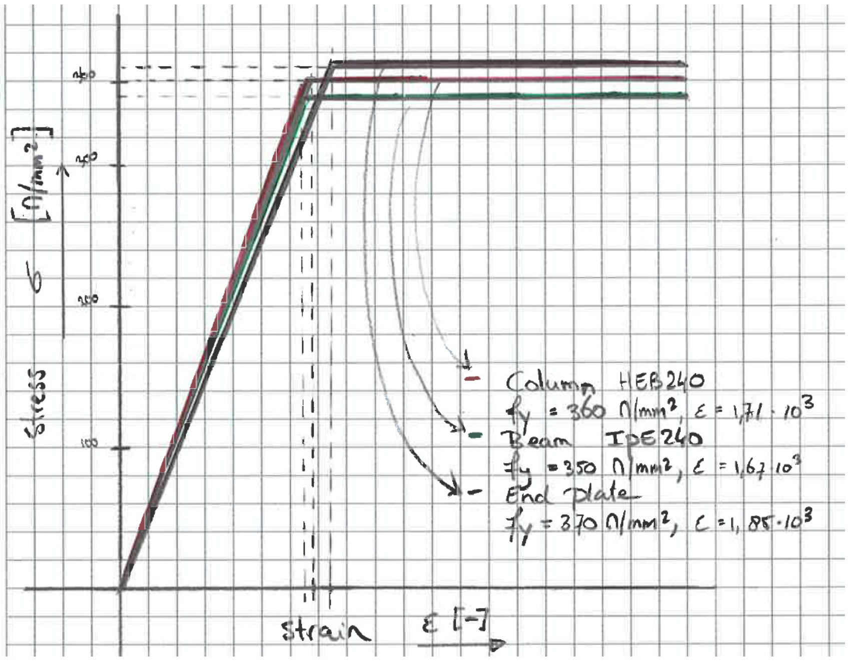


Figure 7.27: Bi-linear material behaviour of the different joint elements for the FEA

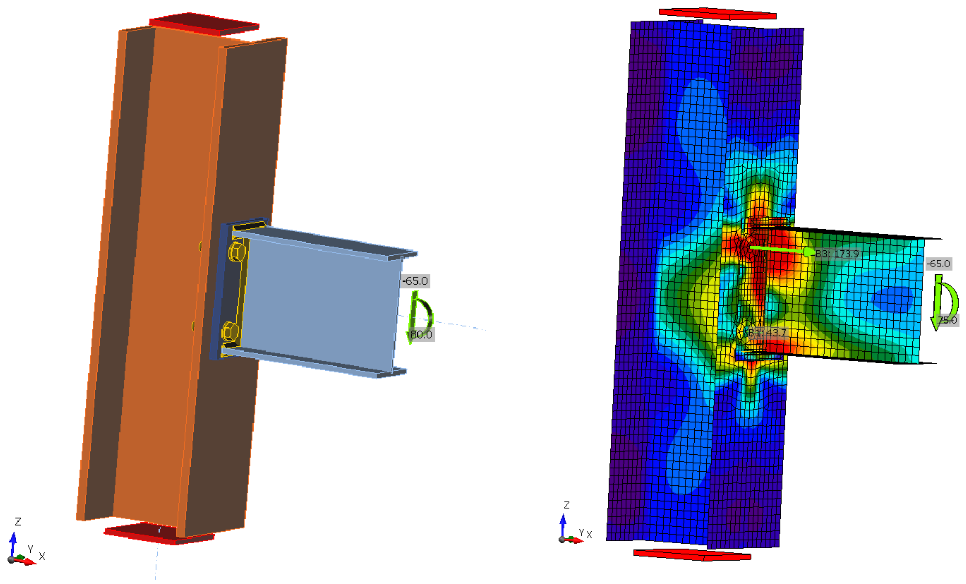


Figure 7.28: FEA model and stress distribution in the model

7.5.4 Comparison Manual Calculation and FEA model FEMRJ

The resistance values which are obtained by this comparison are:

- Manual Calculation: $M_{R,Ec3} = 65$ kNm
- FEA model: $M_{R,model} = 80$ kNm
- Experimental resistance: $M_{R,Exp} = 85$ kNm.

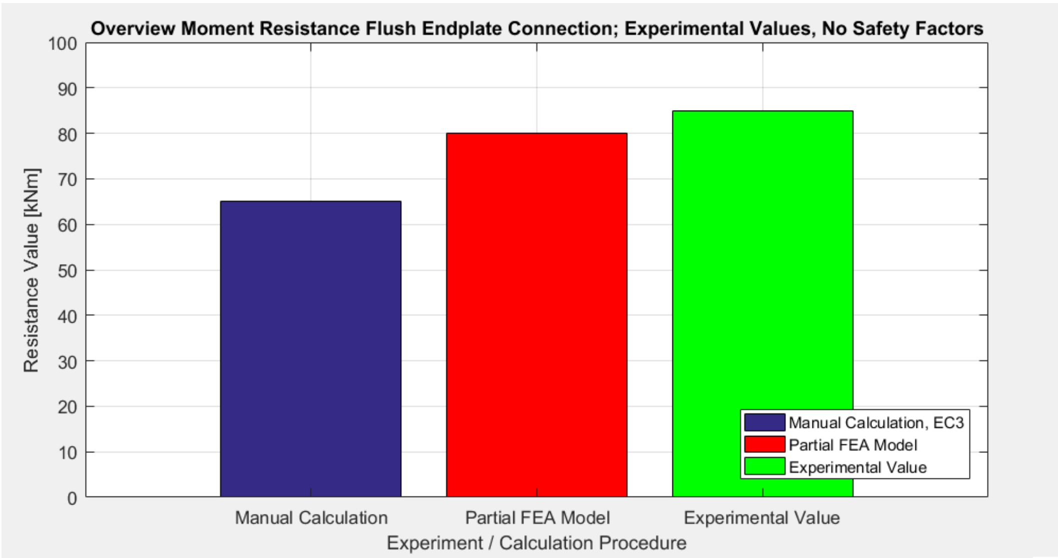


Figure 7.29: Comparison resistance values of manual calculation, FEA model and experiment

7.5.5 Calculation Flush End Plate Joint with EC3 values

Previous sub chapter has shown that the FEA model gives a higher moment resistance value than the in reality. In the manual calculation and the FEA models, the experimental values are used and no partial safety factors are applied. Interesting is to see which resistance values will be obtained when the predefined EC3 values will be used.

Table 7.1: Values for manual calculation

Column	yield strength	$f_{y,c}$	$275N/mm^2$
	tension strength	$f_{u,c}$	$430N/mm^2$
Beam	yield strength	$f_{y,b}$	$275N/mm^2$
	tension strength	$f_{u,b}$	$430N/mm^2$
End Plate	yield strength	$f_{y,ep}$	$275N/mm^2$
	tension strength	$f_{y,ep}$	$430N/mm^2$
Partial Factors	resistance cross-sections	γ_{M0}	1.0
	resistance on stability	γ_{M1}	1.0
	resistance cross-section in tension till rupture	γ_{M2}	1.25

Using the values depicted above, table 7.1. The following moment resistance values will be obtained. In this case the bending moment resistance as the FEA model are lower the the bending moment obtained in the experiment, see figure 7.30.

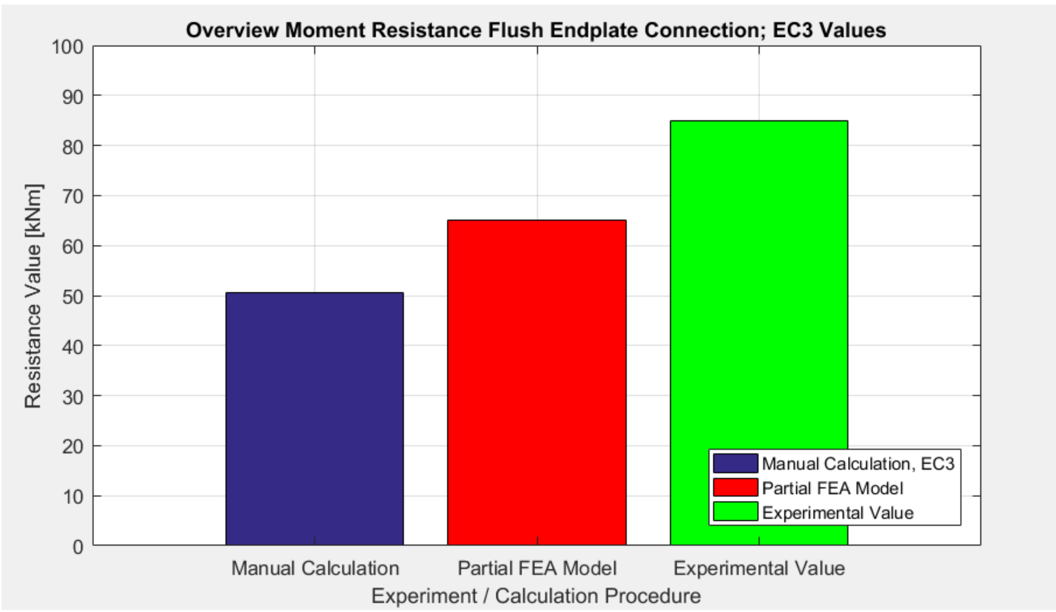


Figure 7.30: Comparison resistances values with EC3 yield- and tensile strength and partial safety factors according to the Dutch National Annex.

7.6 Validation Extended End Plate Moment Resisting Joint (EEPMRJ)

In the research programme also tests were done on an extended end plate moment resisting joint.

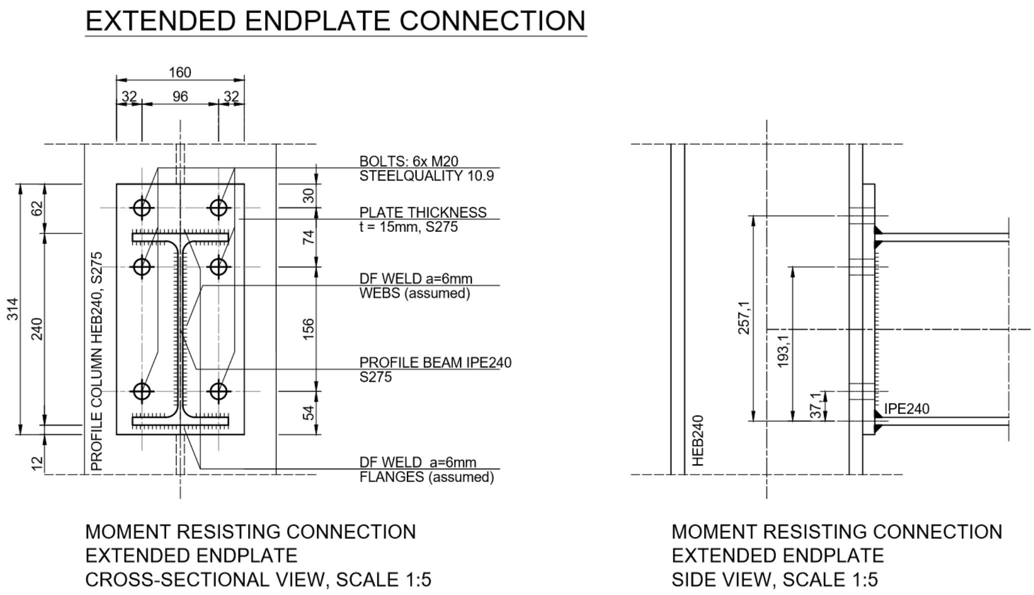


Figure 7.31: Configuration of the extended end plate joint

7.6.1 Experiment and assumptions

Same test set-up is used as the flush end plate joint. For the manual calculation and the partial FEA model the following assumptions are made:

- Joint configuration as depicted in figure 7.31.
- Plastic calculation, using plastic moment resistance and plastic section modulus
- Material properties for the manual calculation are presented in figure 7.32.
- The upper two bolt rows contribute to the tension resistance of the joint.
- Lower bolts contribute to the shear resistance of the bolts
- Compression force is transferred from the lower beam flange through the end plate to the column.

PROPERTIES JOINT ELEMENTS, EXTENDED ENDPLATE						
IPE240	EXPERIMENTAL VALUES			DERIVED CALCUALTION VALUES		
YIELD STRENGTH	fy	351.5	N/mm2	fy	350	N/mm2
TENSILE STRENGTH	fu	451.2	N/mm2	fu	450	N/mm2
E-MODULUS	E	209468	N/mm2	E	210000	N/mm2
HEB240						
YIELD STRENGTH	fy	357.5	N/mm2	fy	360	N/mm2
TENSILE STRENGTH	fu	463.0	N/mm2	fu	460	N/mm2
E-MODULUS	E	213864	N/mm2	E	210000	N/mm2
END PLATE						
YIELD STRENGTH	fy	369.4	N/mm2	fy	370	N/mm2
TENSILE STRENGTH	fu	503.5	N/mm2	fu	500	N/mm2
E-MODULUS	E	200248	N/mm2	E	200000	N/mm2

Figure 7.32: Material properties of the extended end plate joint

7.6.2 Manual Calculation

The same procedure has been executed for an extended end plate joint. The different components are executed according to the calculation rules prescribed by the Eurocode. Using the above mentioned assumptions together with the described material properties the following resistance values are obtained in tension.

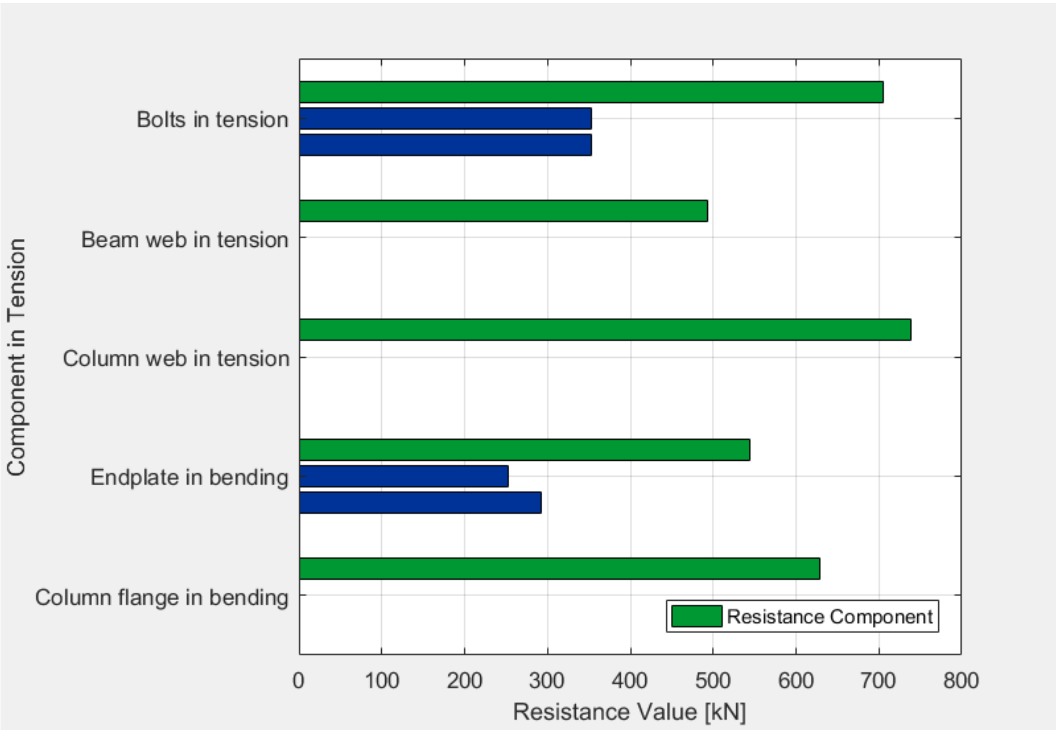


Figure 7.33: Resistance values of the components in tension zone

An overview of all resistance values of other components are depicted in figure 7.33. Beam web in tension is the weakest component, for this reason the resistance values of the end plate in bending are reduced to determine the moment resistance of the connection. Because the tension zone is the weakest component zone no further reduction is needed to obtain horizontal equilibrium.

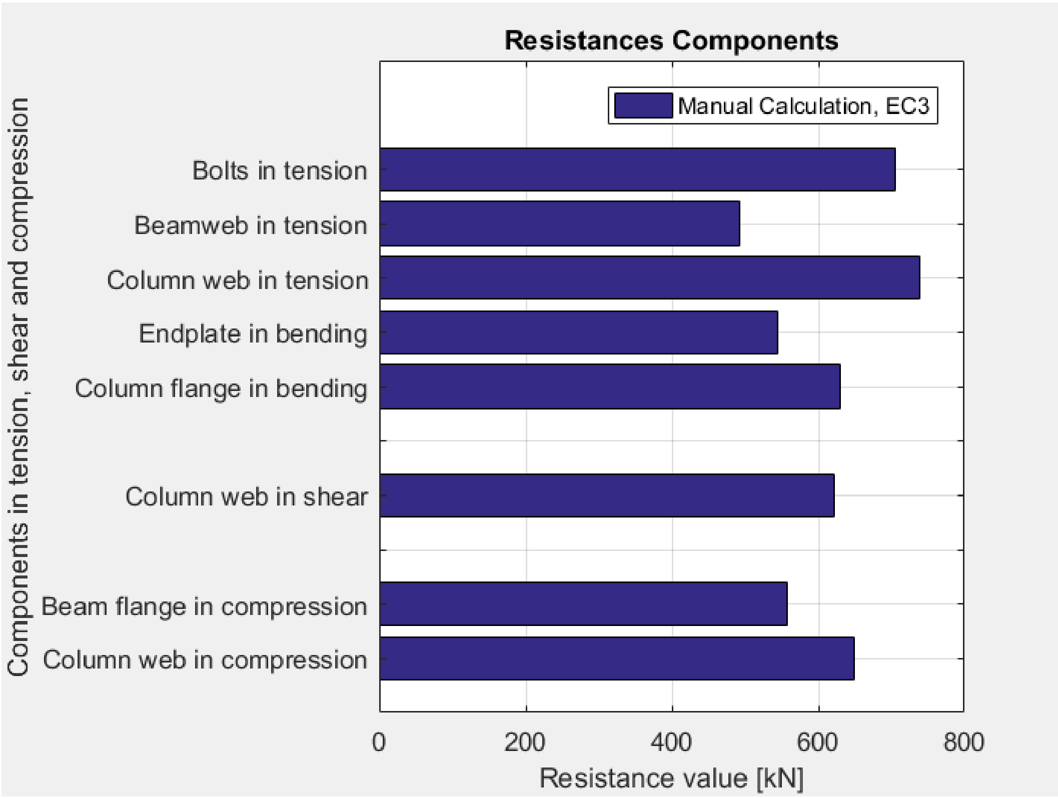


Figure 7.34: Overall resistances components in extended end plate joint

7.6.3 Partial FEA model

A partial FEA model is made with the prescribed joint configuration. Also in this experiment no information is provided of the weld size and strength. So also in this case full strength welds are assumed. The experimental values are used to derive bi-linear stress-strain relations for the FEA modeling. For the column, end plate and beam different bi-linear stress strain relation are used, which are same as the flush end plate model.

7.6.4 Comparison Manual Calculation, FEA model, Experiment

Manual calculated moment resistance, the partial FEA model and experimental values give similar results, figure 7.35. The FEA model give a higher results than is observed from experiments. The following end results are obtained:

- Moment resistance calculated manually: $M_{Rd,EC3} = 110$ kNm
- Moment resistance from FEA model: $M_{Rd,model} = 125$ kNm
- Maximum moment experiment: $M_{R,exp} = 125$ kNm

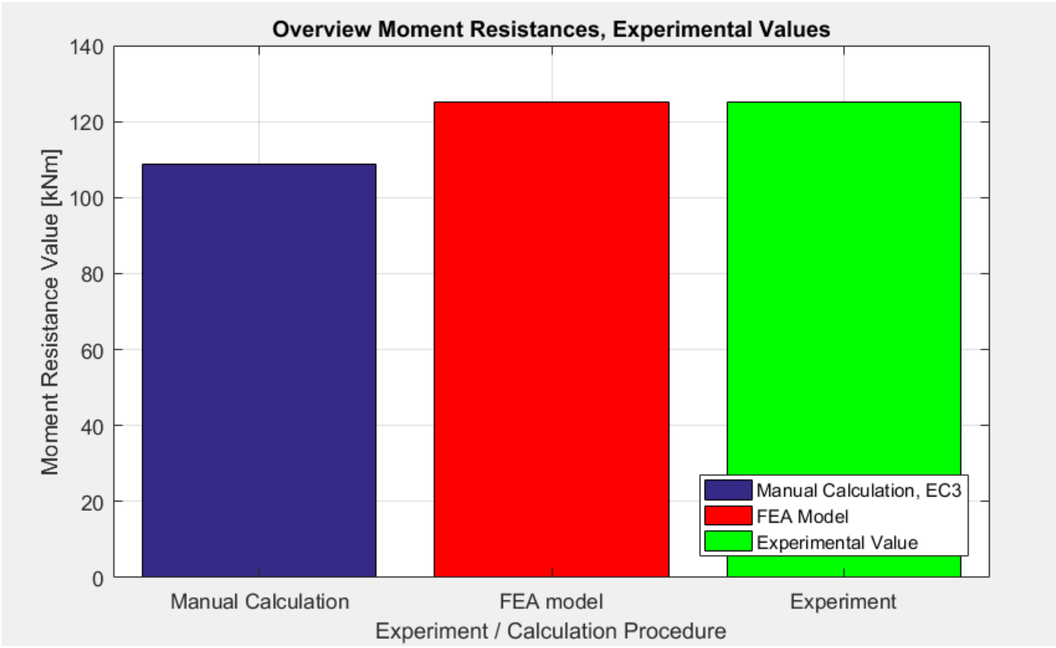


Figure 7.35: Comparison resistance values manual calculation, FEA model and experiment

The resistance value of the FEA models corresponds with the resistance value of the experiment. De resistance value according to the Eurocode 3 calculate is below previous values.

7.6.5 Calculation Extended End Plate Joint with EC3 values

For the following calculation and model not experimental values are used, but the design values presribed by the NEN-EN 1993-1-1, art. 3.2.3, table(3.1) for steelgrade S275.

Table 7.2: Values for manual calculation

Column	yield strength	$f_{y,c}$	$275N/mm^2$
	tension strength	$f_{u,c}$	$430N/mm^2$
Beam	yield strength	$f_{y,b}$	$275N/mm^2$
	tension strength	$f_{u,b}$	$430N/mm^2$
End Plate	yield strength	$f_{y,ep}$	$275N/mm^2$
	tension strength	$f_{y,ep}$	$430N/mm^2$
Partial Factors	resistance cross-sections	γ_{M0}	1.0
	resistance on stability	γ_{M1}	1.0
	resistance cross-section in tension till rupture	γ_{M2}	1.25

Linear behaviour in the FEA calculation is schematised bi-linear prescribed acco-ording the NEN-EN 1993-1-1, art. 5.4.3(4), figure (5.8).

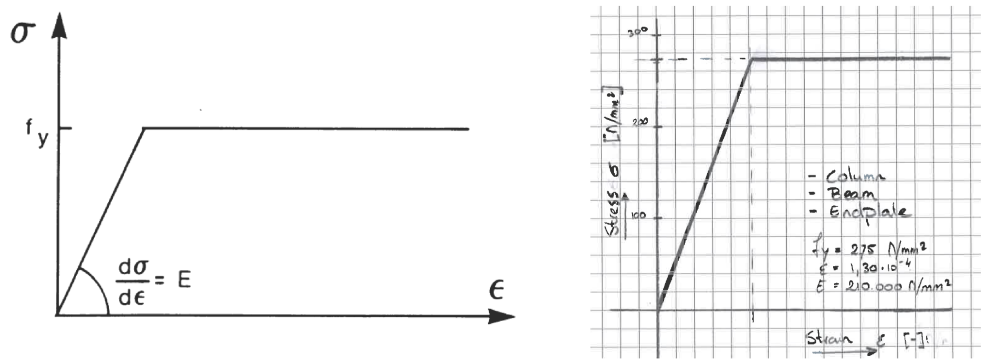


Figure 7.36: Bi-linear material behaviour prescribed by the Eurocode 3: NEN-EN 1993-1-1

In the manual calculation the following resistance values were obtained of the components. The following stress distribution was obtained at failure of the FEA model. Strains in the upper flange of the beam and end plate were exceeding the 5% limit strain.

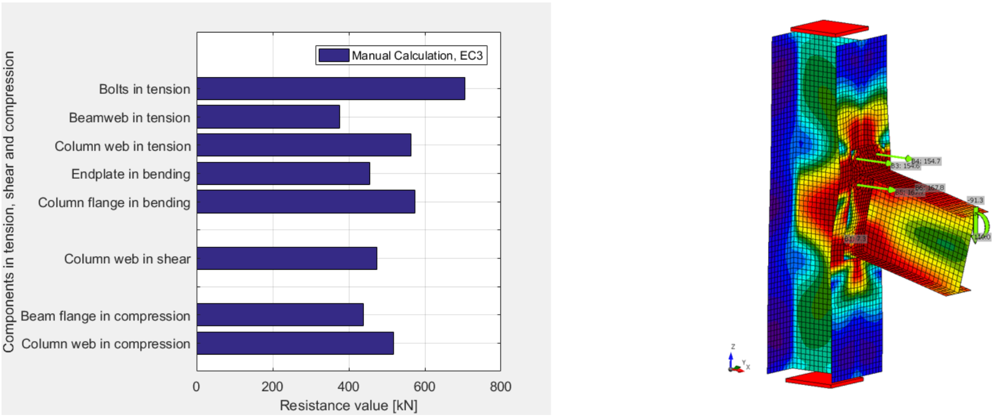


Figure 7.37: Bi-linear material behaviour used for the FEA models

Comparing the results of the individually calculation methods the following moment resistances were obtained compared to the executed experiment, figure 7.38:

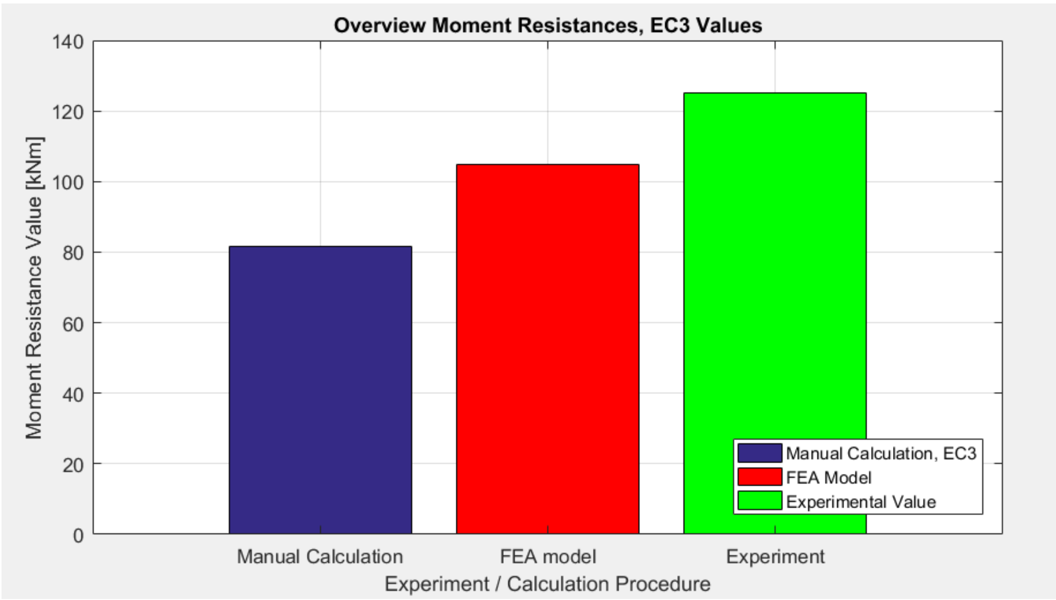


Figure 7.38: Moment resistances using the prescribed yield- and tensile strength values and partial safety factors according to the Eurocode 3.

Chapter 8

Parametric Study

A parametric study has been executed for different types of joints: Fin plate joint and simple shear joint. Different variants, with different geometry and properties have been developed and are modelled first in a partial FEA. Afterwards these variants are calculated manually with the previously developed MATLAB scripts. Drawings of these variants can be found in Annex D.

8.1 Parametric Study Fin Plate Joint

Different variants of fin plates are developed and modelled. Variant A is the standard configuration, the model resistance is determined by sequentially increasing the load. After exceeding the resistance of the model. The component of the joints that is failing will be strengthened.

Fin Plate Joint, Variant A

Variant A is the first and standard joint configuration of the fin plate joint. This standard configuration is depicted in figure 8.1.

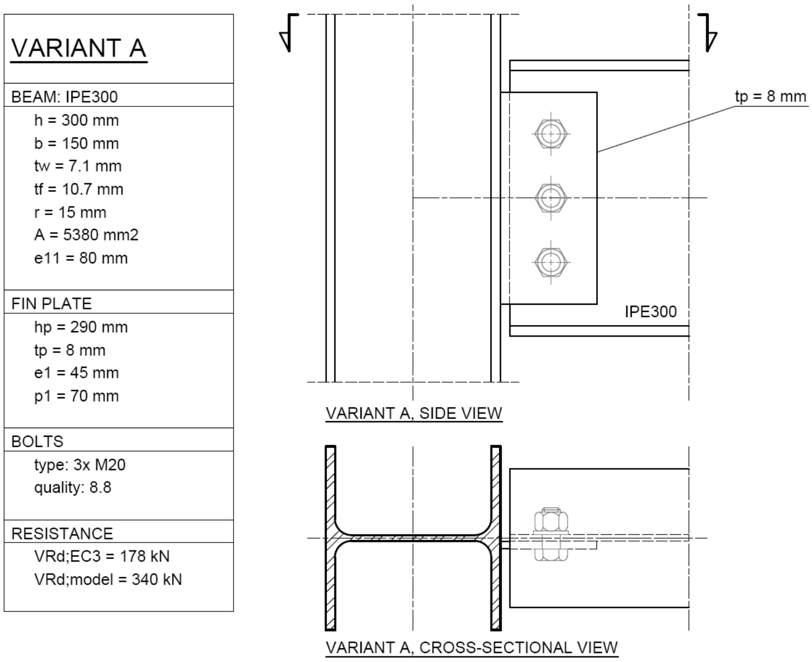


Figure 8.1: Fin Plate Joint Configuration, Variant A

The 5 % limit strain is exceeded in elements in the fin plate at a applied shear load of $V_{Ed} = 220kN$

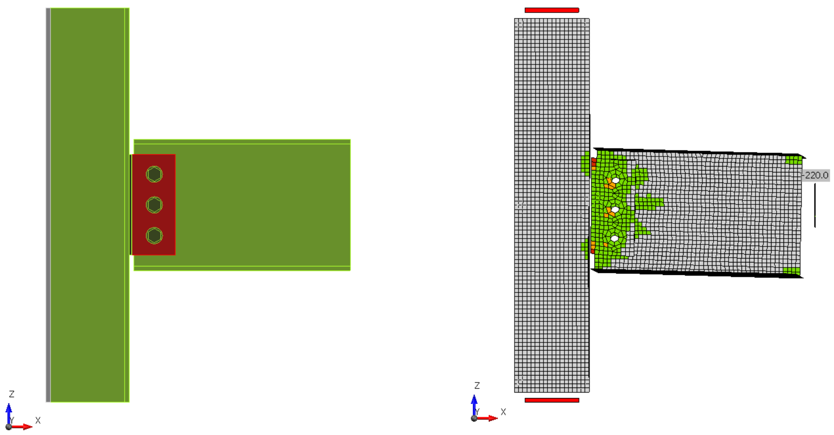


Figure 8.2: Fin Plate Joint FEA, Variant A

Using the above described configuration the following resistances are obtained by manual calculation, figure 8.3.

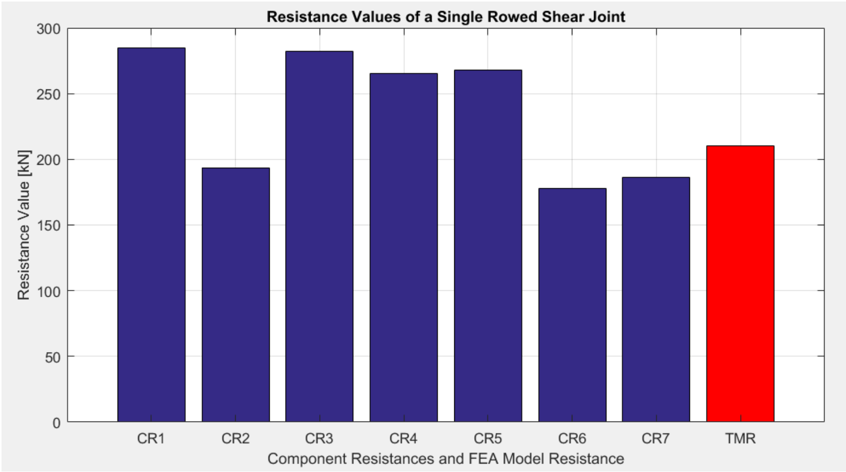


Figure 8.3: Fin Plate Joint Manual Calculation, Variant A

Abbreviations of the graph and upcoming graphs are depicted hereunder:

- Shear resistance of the beam web (CR1)
- Tearing resistance of the beam web (CR2)
- Shear resistance of the bolts (CR3)
- Bearing resistance of the fin plate (CR4)
- Bearing resistance of the beam web (CR5)
- Shear resistance of the fin plate (CR6)
- Tearing resistance of the fin plate (CR7)
- Total resistance of the FEA model (TMR)

Fin Plate Joint, Variant B

In variant B the fin plate is strengthened. Load on the model is further increased till next failure mechanism occurs in the model. Geometry of Variant B is depicted in figure 8.4.

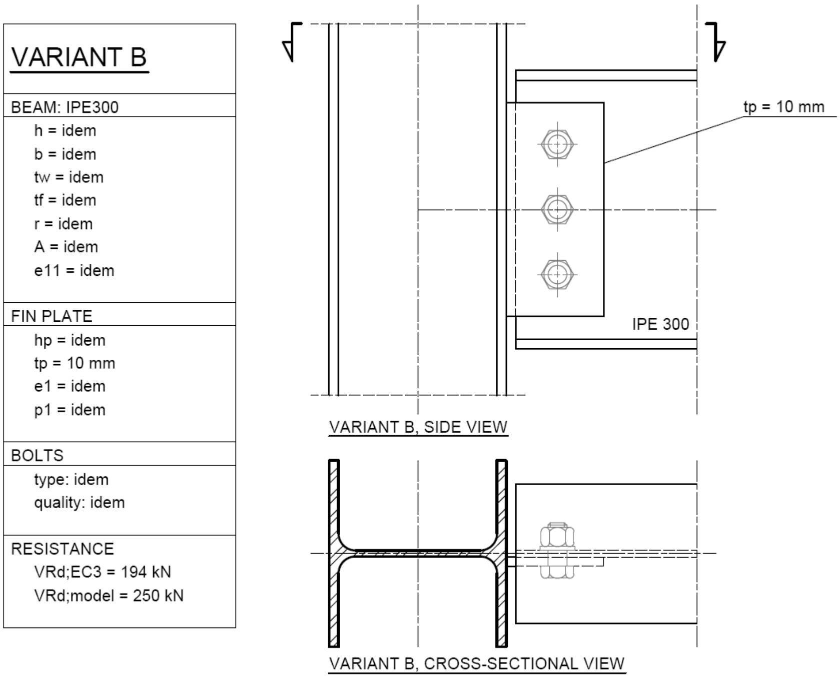


Figure 8.4: Fin Plate Joint Configuration, Variant B

This configuration is used in the partial FEA model. At a load of $V_{Ed} = 260kN$ the beam web of the connection will fail. Elements in the vicinity of the beam web will fail.

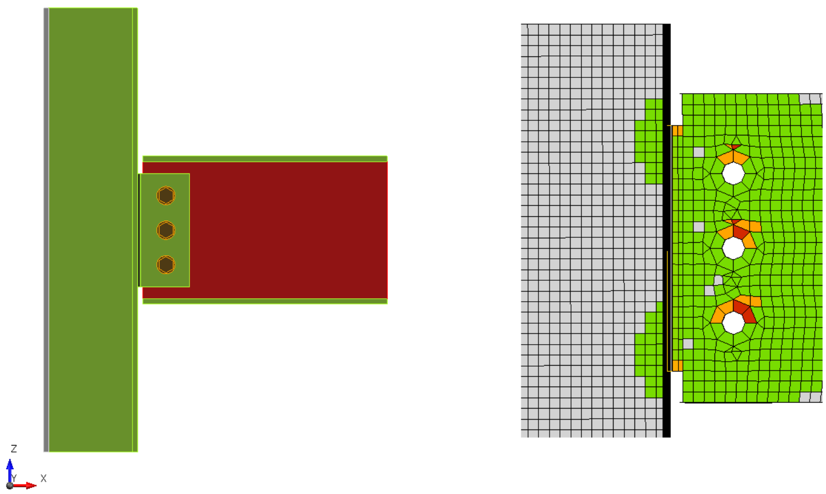


Figure 8.5: Fin Plate Joint FEA, Variant B

After, this configuration of the fin plate joint is calculated manually. Results can be found in figure 8.5.

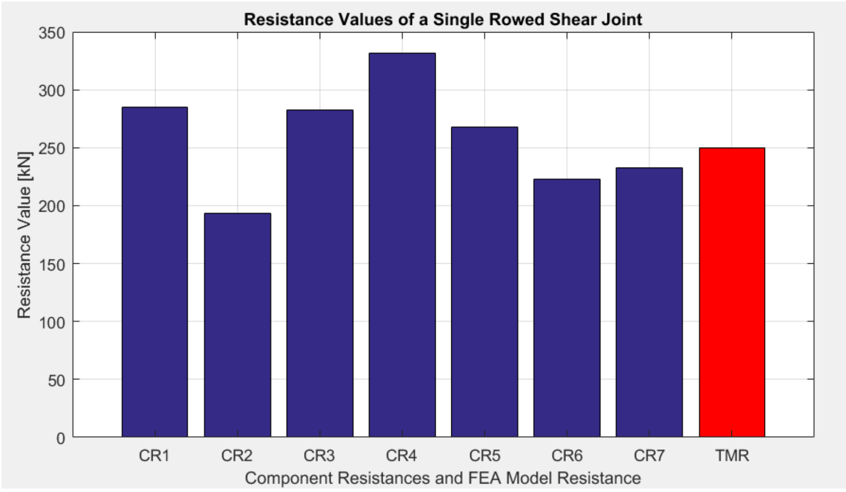


Figure 8.6: Fin Plate Joint Manual Calculation, Variant B

Abbreviations of the graph are explained hereunder:

- Shear resistance of the beam web (CR1)
- Tearing resistance of the beam web (CR2)
- Shear resistance of the bolts (CR3)
- Bearing resistance of the fin plate (CR4)
- Bearing resistance of the beam web (CR5)
- Shear resistance of the fin plate (CR6)
- Tearing resistance of the fin plate (CR7)
- Total resistance of the FEA model (TMR)

Fin Plate Joint, Variant C

In Variant C the beam web is strengthened by selecting a heavier steel profile.

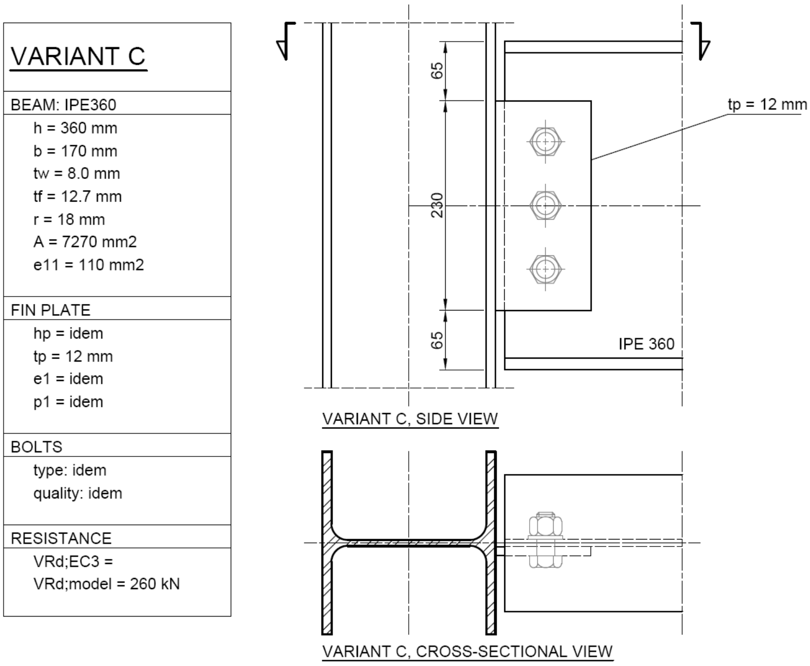


Figure 8.7: Fin Plate Joint Configuration, Variant C

This configuration is modelled and loaded till first failure mechanism occurred, which is failure of the bolts in the fin plate.

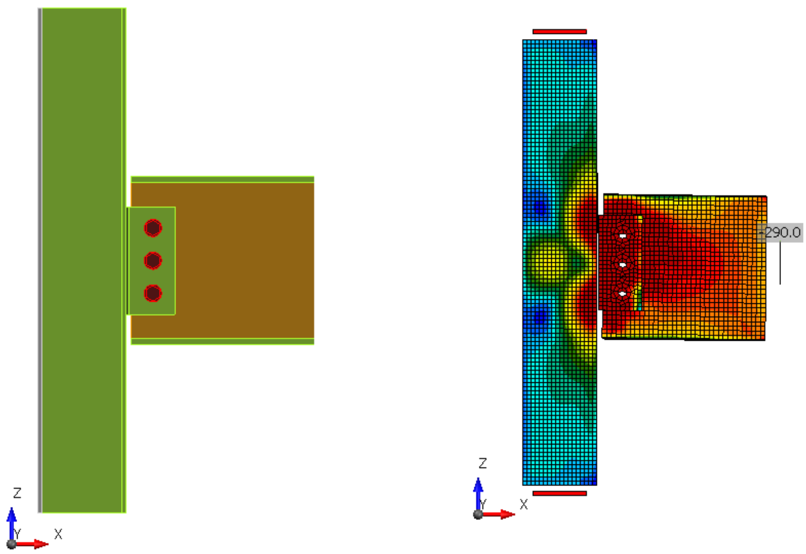


Figure 8.8: Fin Plate Joint FEA, Variant C

Same configuration is used for manual calculation. Results are presented in figure 8.9.

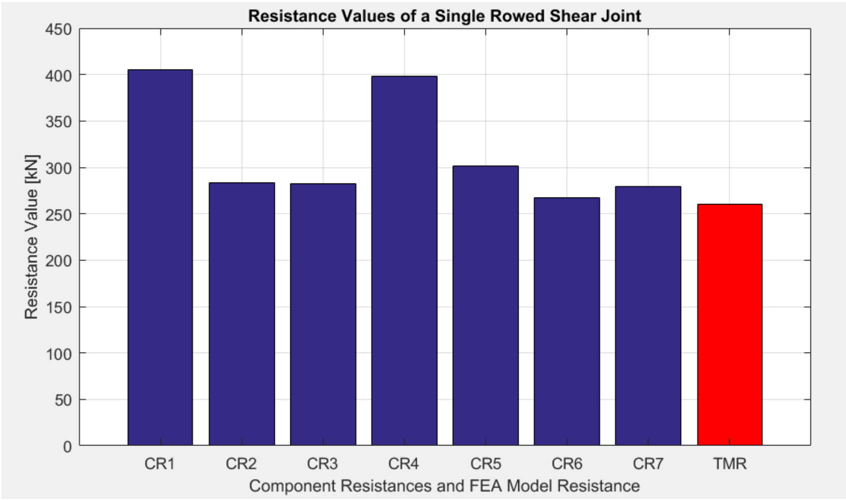


Figure 8.9: Fin Plate Joint Manual Calculation, Variant C

Abbreviation in graph are explained here:

- Shear resistance of the beam web (CR1)
- Tearing resistance of the beam web (CR2)
- Shear resistance of the bolts (CR3)
- Bearing resistance of the fin plate (CR4)
- Bearing resistance of the beam web (CR5)
- Shear resistance of the fin plate (CR6)
- Tearing resistance of the fin plate (CR7)
- Total resistance of the FEA model (TMR)

Fin Plate Joint, Variant D

In Variant D the resistances of the bolts are increased by selecting bigger sized bolts, further the fin plate is enlarged and the bolt spacing has been changed which is influencing the parameters of edge- and end distances.

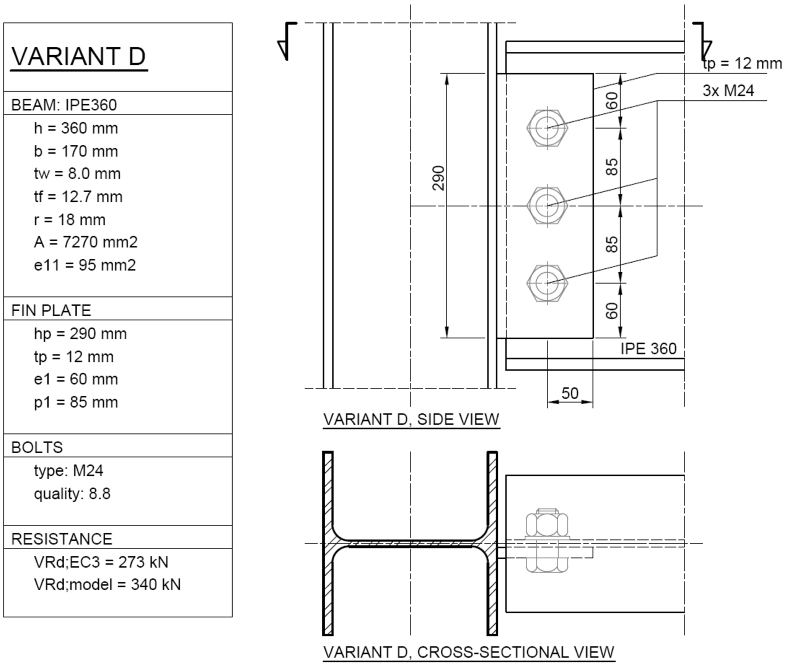


Figure 8.10: Fin Plate Joint Configuration, Variant D

This configuration is modelled and loaded till the next failure mechanism occurred, which is failure of the beam web by exceeding the 5 % limit value in the finite elements.

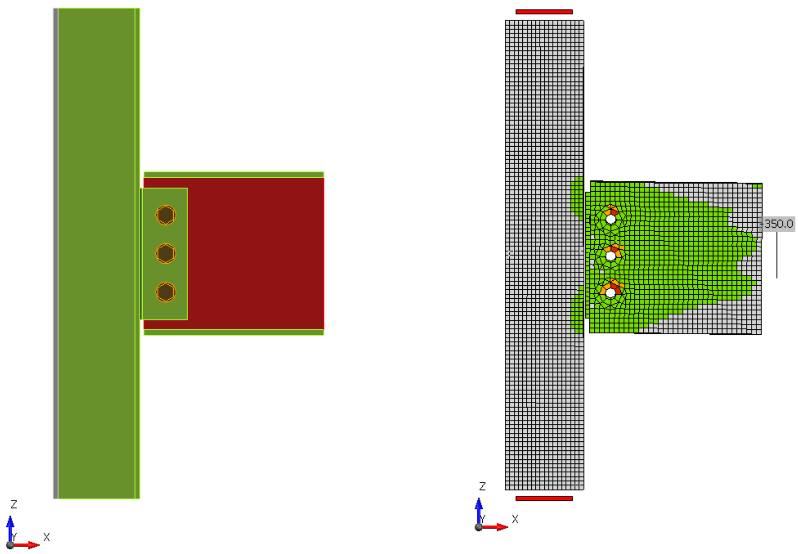


Figure 8.11: Fin Plate Joint FEA, Variant D

Same configuration is used for manual calculation. Results are presented in figure 8.12.

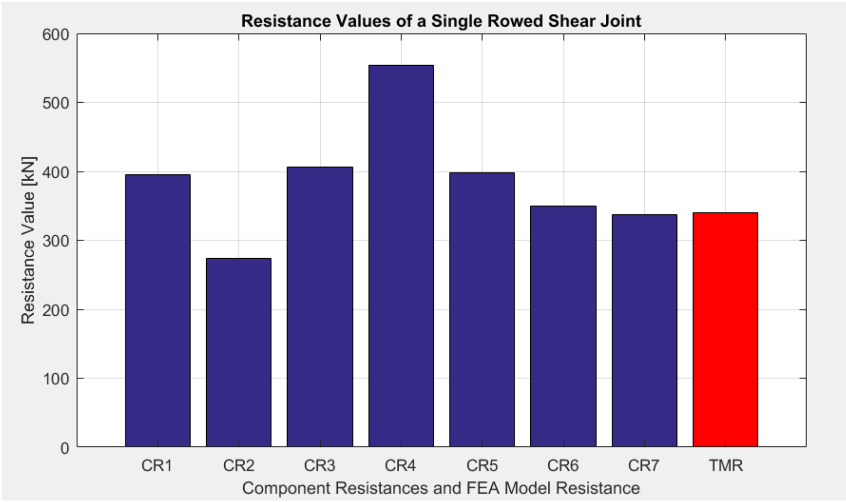


Figure 8.12: Fin Plate Joint Manual Calculation, Variant D

Abbreviation in graph are explained hereunder:

- Shear resistance of the beam web (CR1)
- Tearing resistance of the beam web (CR2)
- Shear resistance of the bolts (CR3)
- Bearing resistance of the fin plate (CR4)
- Bearing resistance of the beam web (CR5)
- Shear resistance of the fin plate (CR6)
- Tearing resistance of the fin plate (CR7)
- Total model resistance FEA(TMR)

Summary Results
Resistance values together with the decisive components of all the variants of the fin plate joint are presented in the table hereunder.

Table 8.1: Results parametric study, Fin Plate Joint

Fin Plate Joint	$V_{Rd,EC3}$	Component		$V_{Rd,mod}$	Component
Variant A	178 kN	Fin Plate		210 kN	Fin Plate
Variant B	194 kN	Beam Web		250 kN	Beam Web
Variant C	267 kN	Fin Plate		260 kN	Bolts
Variant D	273 kN	Beam Web		340 kN	Beam Web

8.2 Parametric Study Short End Plate Joint

Different variants of the short end plate joint are developed and modelled. Variant A is the standard configuration. The model resistance is determined by sequentially increasing the load. After exceeding the resistance of the model. The component of the joints that is failing will be strengthened.

Short End Plate Joint, Variant A
Variant A is the first and standard joint configuration of the short end plate joint. This standard configuration is depicted in figure 8.13.

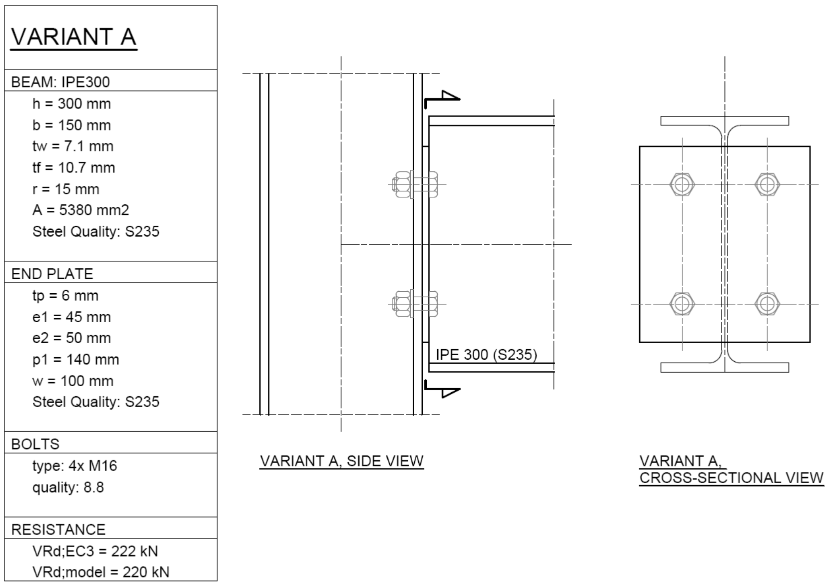


Figure 8.13: Short End Plate Joint Configuration, Variant A

This variant of a short end plate joint is modelled with the same assumptions as in one of the previous chapters.

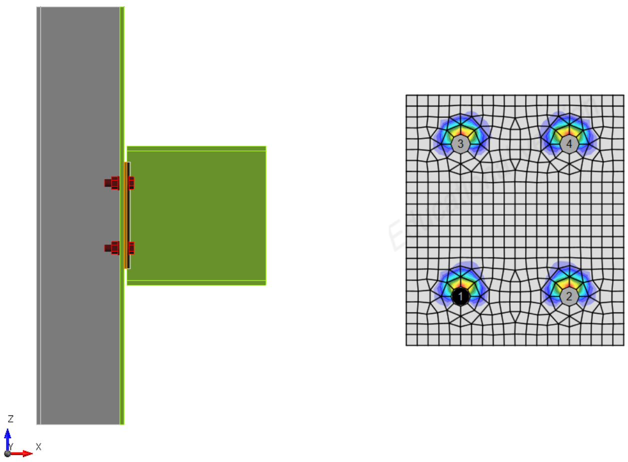


Figure 8.14: Partial FEA model, Variant A

Same configuration is calculated manually. Beam web is the weakest component according to manual calculation. The component that is decisive after the beam web are the bolts in the end plate, which is in correspondence with the FEA model.

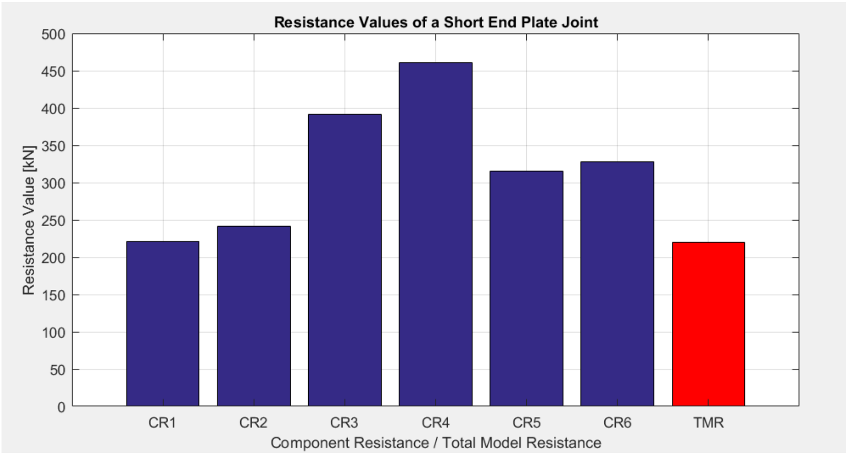


Figure 8.15: Results Manual Calculation, Variant A

Abbreviations in graph are explained hereunder:

- Shear resistance of the beam web (CR1)
- Shear resistance of the bolts (CR2)
- Bearing resistance of the end plate (CR3)
- Bearing resistance in the column flange (CR4)
- Shear resistance of the end plate (CR5)
- Tearing resistance of the end plate (CR6)
- Total model resistance (TMR)

Short End Plate Joint, Variant B

In the previous model the bolts were the weakest component in the joint. For that reason 2 extra bolts with the same size and bolt quality are added to the joint, figure 8.16.

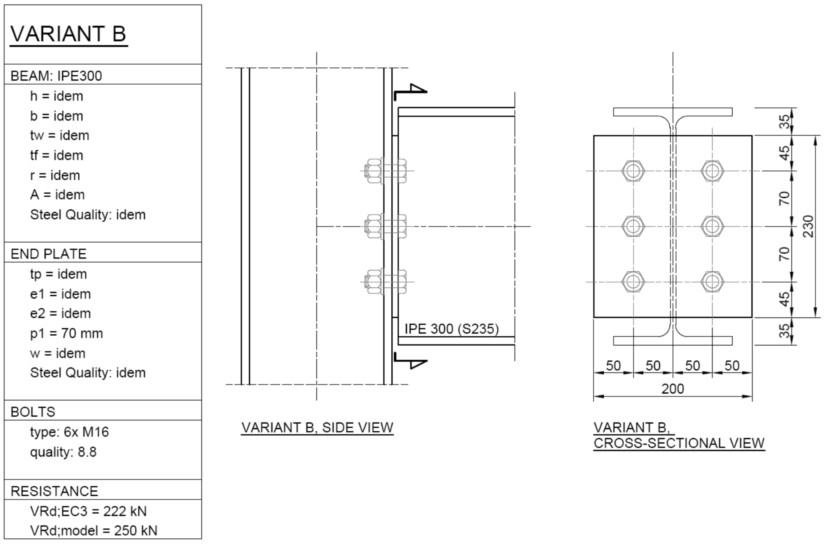


Figure 8.16: Short End Plate Joint Configuration, Variant B

Configuration of variant B is modelled.

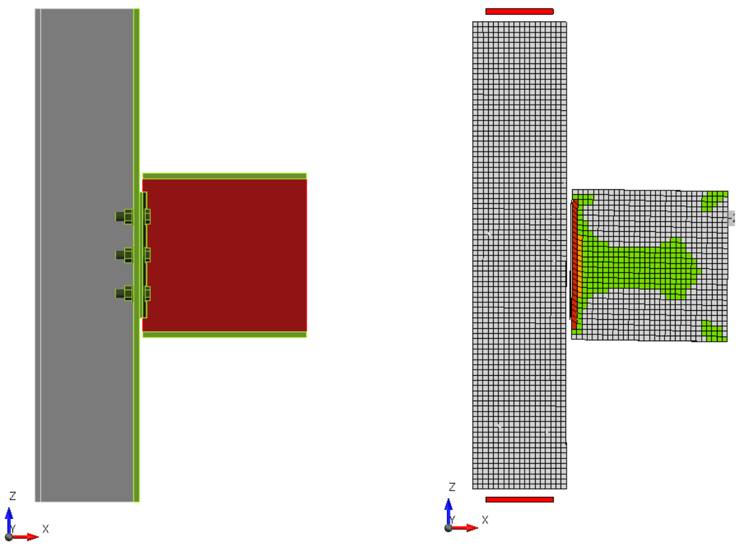


Figure 8.17: Partial FEA model, Variant B

In manual calculation also the beam web is governing, because this component has not been changed. For this configuration it holds that the beam web is the decisive component in the model and in the manual calculation.

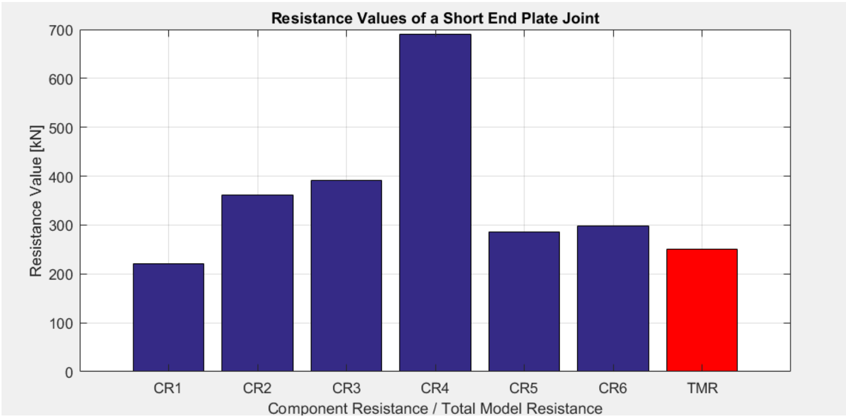


Figure 8.18: Results Manual Calculation, Variant B

Abbreviations in graph:

- Shear resistance of the beam web (CR1)
- Shear resistance of the bolts (CR2)
- Bearing resistance of the end plate (CR3)
- Bearing resistance in the column flange (CR4)
- Shear resistance of the end plate (CR5)
- Tearing resistance of the end plate (CR6)
- Total model resistance (TMR)

Short End Plate Joint, Variant C

In variant B the beam web was decisive in the partial FEA model and the manual calculation. For this reason beam web is chosen with a higher steel grade (S355). Also the steel grade of the end plate is upgraded (S355). Configuration is depicted in figure 8.19.

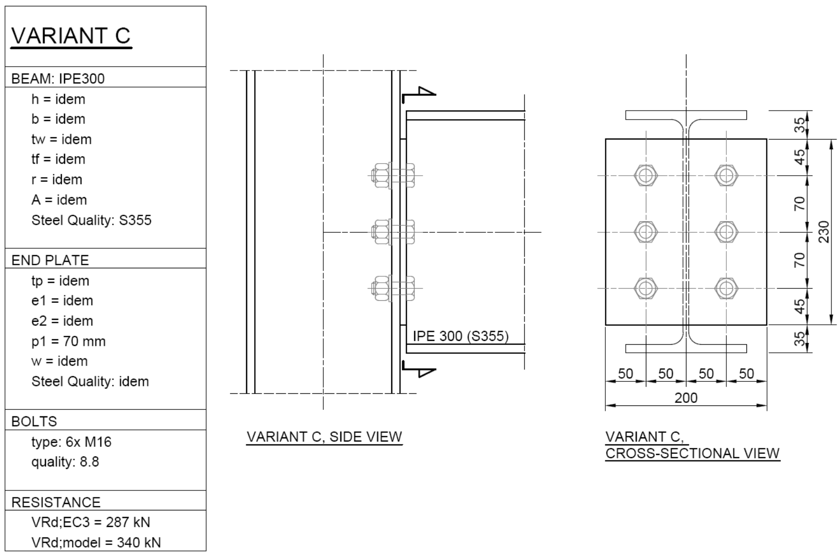


Figure 8.19: Short End Plate Joint Configuration, Variant C

Variant C is modelled. The bolts in the joint are again governing.

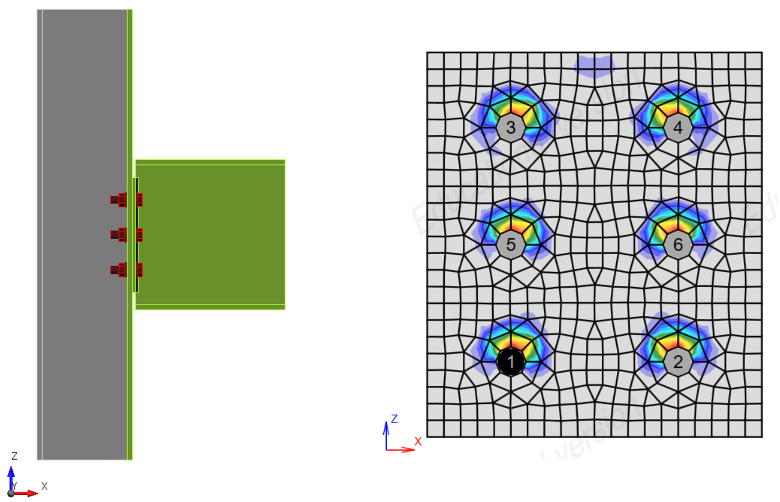


Figure 8.20: Partial FEA model, Variant C

Similar results follow from the manual calculation as for variant A. Beam web is decisive, the bolts are the second weakest component.

Figure 8.21: Hello, I'm Waldo

Abbreviations in graph:

- Shear resistance of the beam web (CR1)
- Shear resistance of the bolts (CR2)
- Bearing resistance of the end plate (CR3)
- Bearing resistance in the column flange (CR4)
- Shear resistance of the end plate (CR5)
- Tearing resistance of the end plate (CR6)
- Total model resistance (TMR)

Short End Plate Joint, Variant D
Bolts were decisive in previous model. For that reason the bolts are resized to M20, with an increase of the plate thickness. Configuration can be found in figure 8.22.

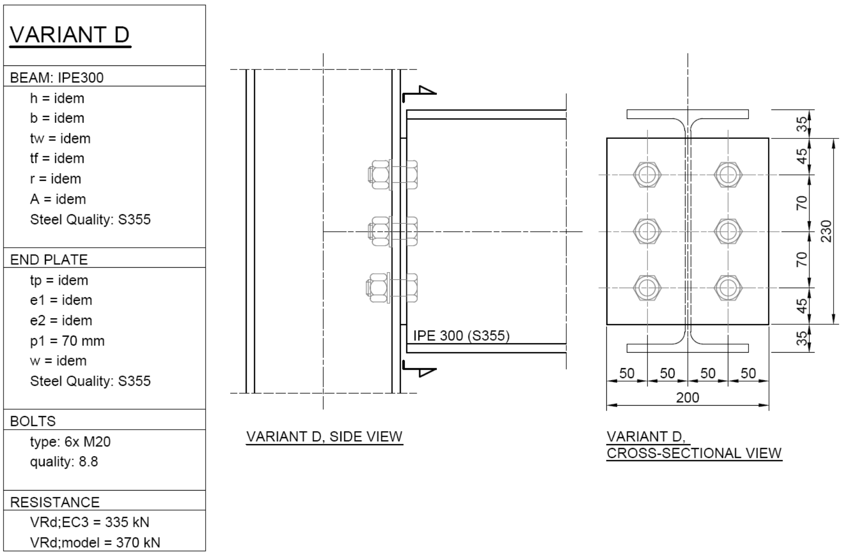


Figure 8.22: Short End Plate Joint Configuration, Variant D

A model has been made of this configuration.

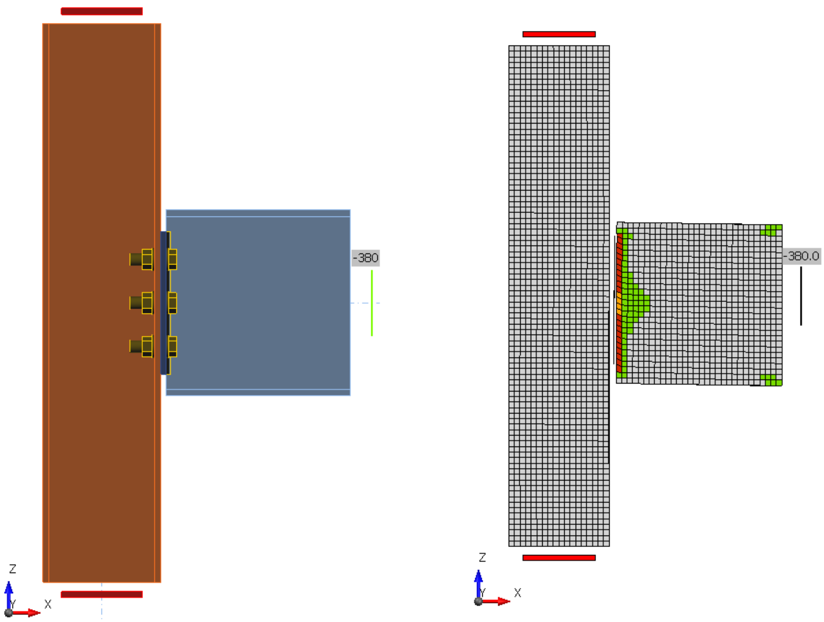


Figure 8.23: Partial FEA model, Variant D

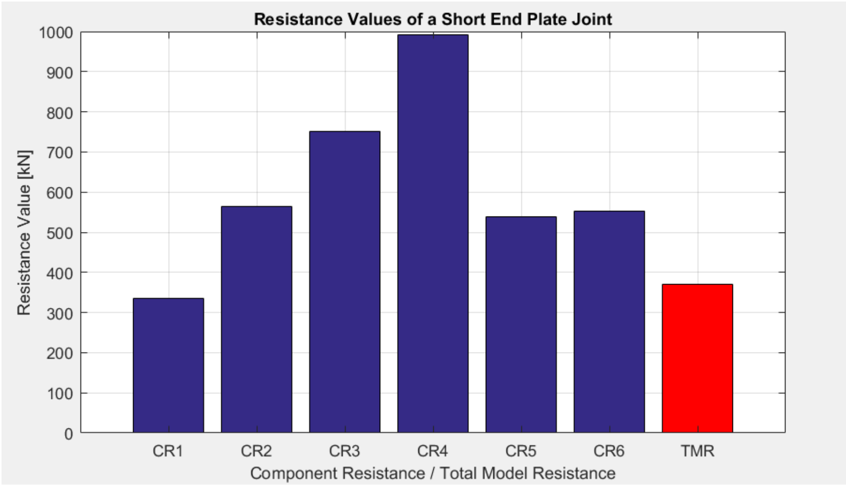


Figure 8.24: Results Manual Calculation, Variant D

Abbreviations in graph:

- Shear resistance of the beam web (CR1)
- Shear resistance of the bolts (CR2)
- Bearing resistance of the end plate (CR3)
- Bearing resistance in the column flange (CR4)
- Shear resistance of the end plate (CR5)
- Tearing resistance of the end plate (CR6)
- Total model resistance (TMR)

Summary Results
Resistance values together with the decisive components of all the variants of the short end plate are presented in the table hereunder.

Table 8.2: Results parametric study, short end plate joint

Short End Plate Joint	$V_{Rd,EC3}$	Component		$V_{Rd,mod}$	Component
Variant A	222 kN	Beam Web		220 kN	Bolts
Variant B	222 kN	Beam Web		250 kN	Beam Web
Variant C	287 kN	Beam Web		340 kN	Bolts
Variant D	335 kN	Beam Web		370 kN	Beam Web

8.3 Parametric Study Moment Resisting Joint

Same procedure has been executed for a moment resisting joint. Different configuration of a flush end plate joint and a moment resisting joint are designed. First these configurations are modelled, after they are calculated manually. The first failure mechanism according the partial FEA models are strengthened and then loaded till the next failure mechanism.

Configuration MRJ's , variant A-D
The configurations of the flush end plate joints are depicted in the figures: 8.25 - 8.28

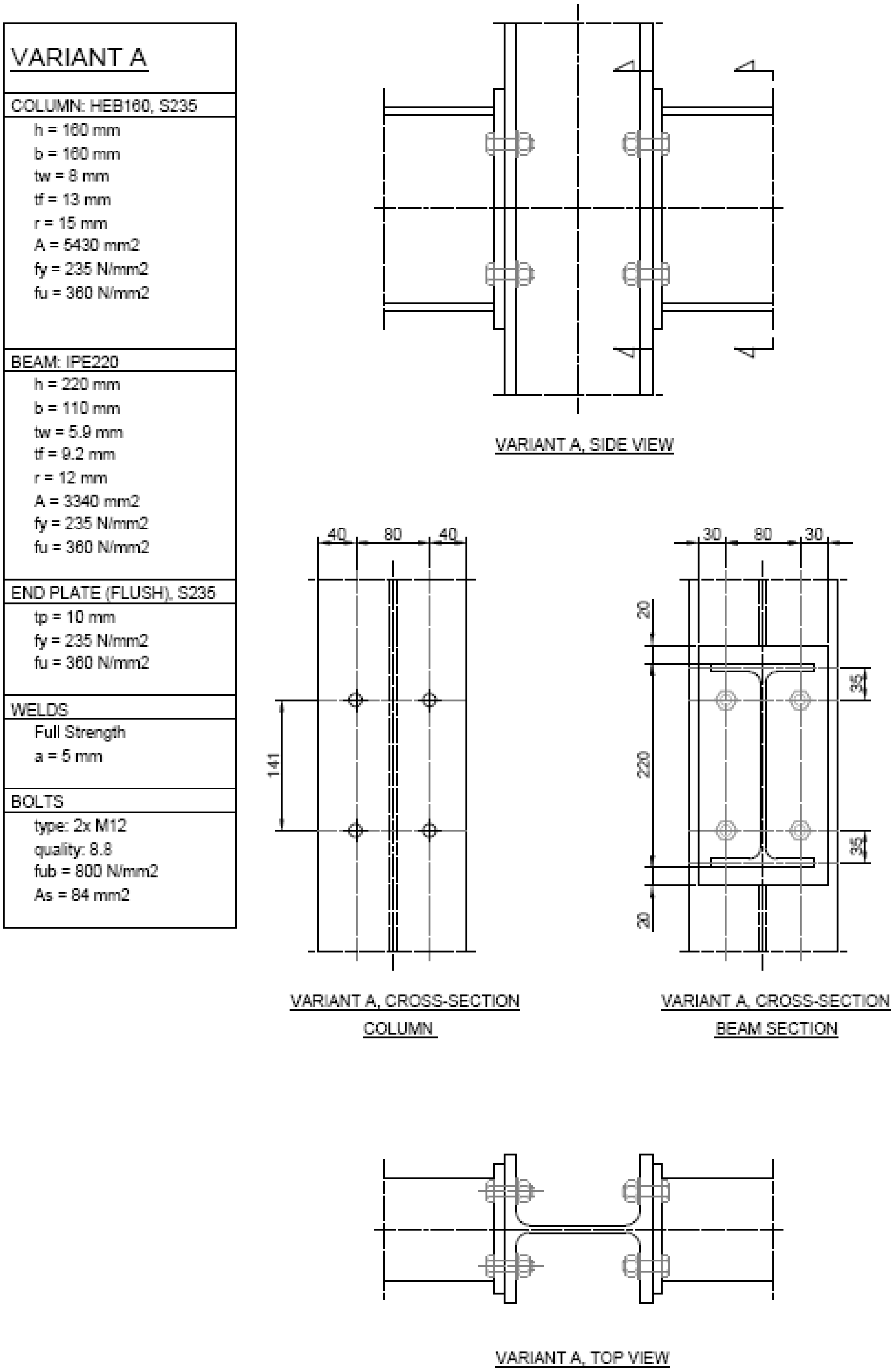


Figure 8.25: Flush End Plate MRJ, Variant A

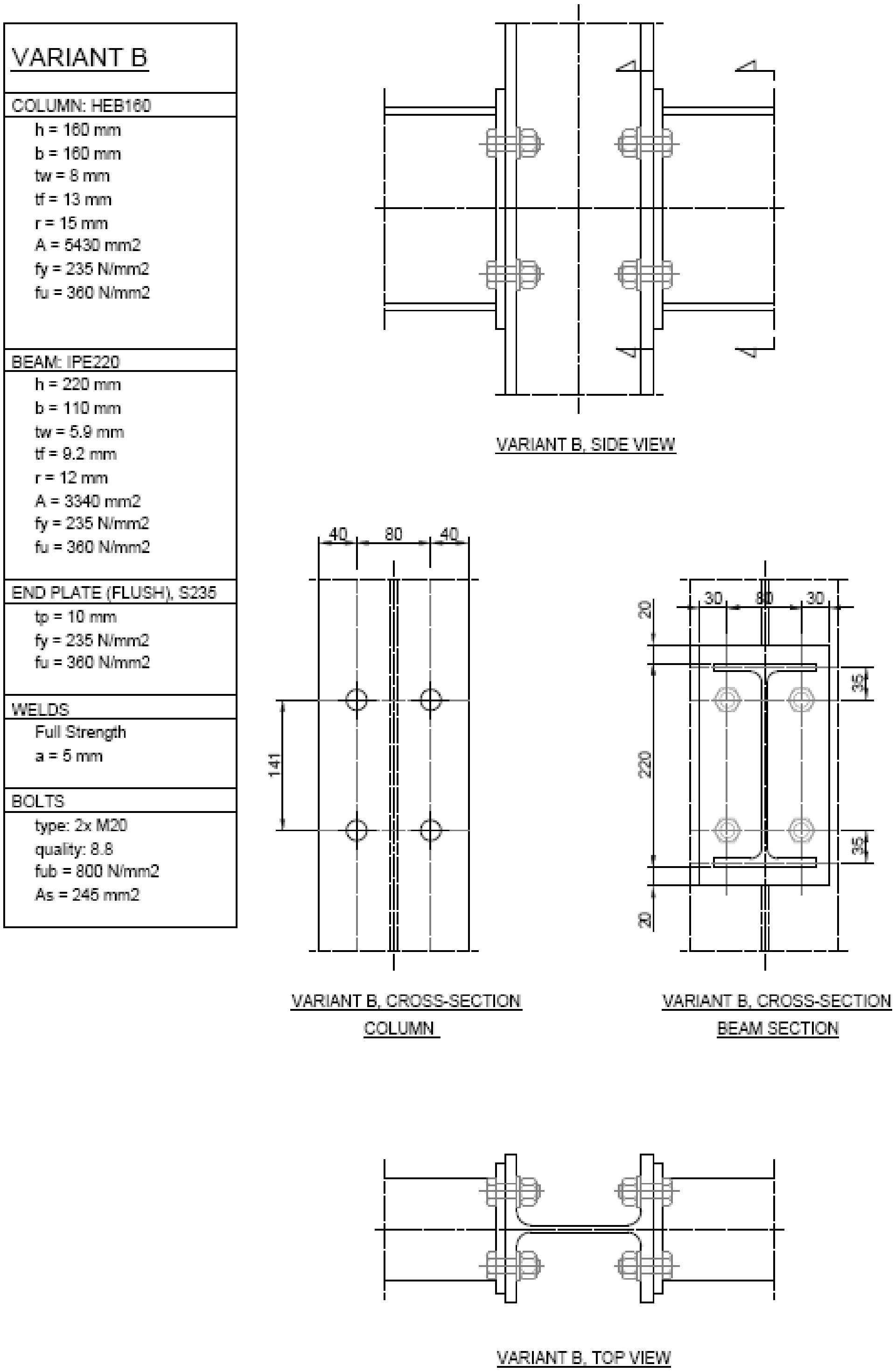


Figure 8.26: Flush End Plate MRJ, Variant B

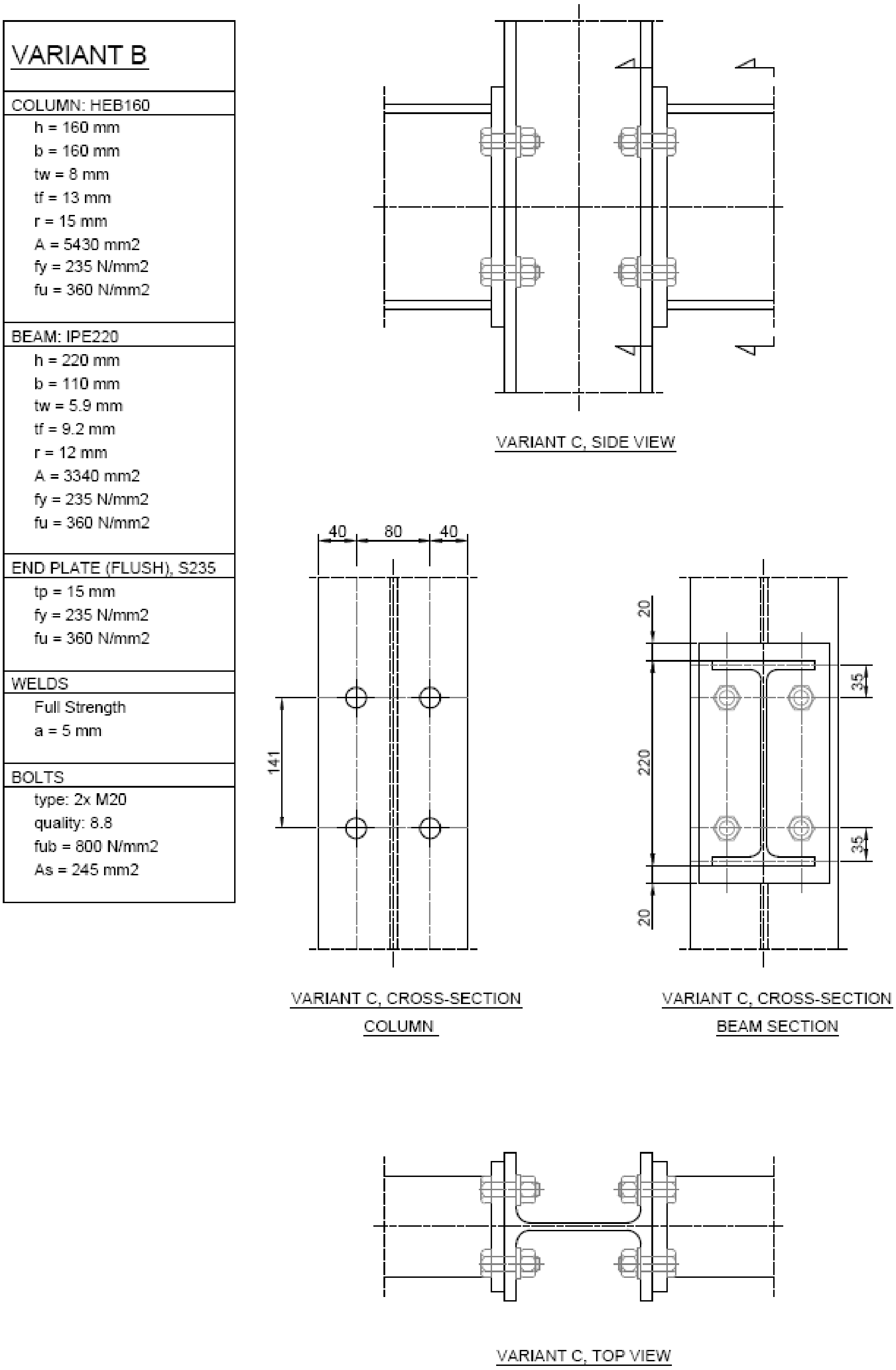


Figure 8.27: Flush End Plate MRJ, Variant C

VARIANT D

COLUMN: HEM160

- h = 180 mm
- b = 166 mm
- tw = 14 mm
- tf = 23 mm
- r = 15 mm
- A = 9710 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

BEAM: IPE220

- h = 220 mm
- b = 110 mm
- tw = 5.9 mm
- tf = 9.2 mm
- r = 12 mm
- A = 3340 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

END PLATE (FLUSH), S235

- tp = 15 mm
- fy = 235 N/mm2
- fu = 360 N/mm2

WELDS

- Full Strength
- a = 5 mm

BOLTS

- type: 2x M20
- quality: 8.8
- fub = 800 N/mm2
- As = 245 mm2

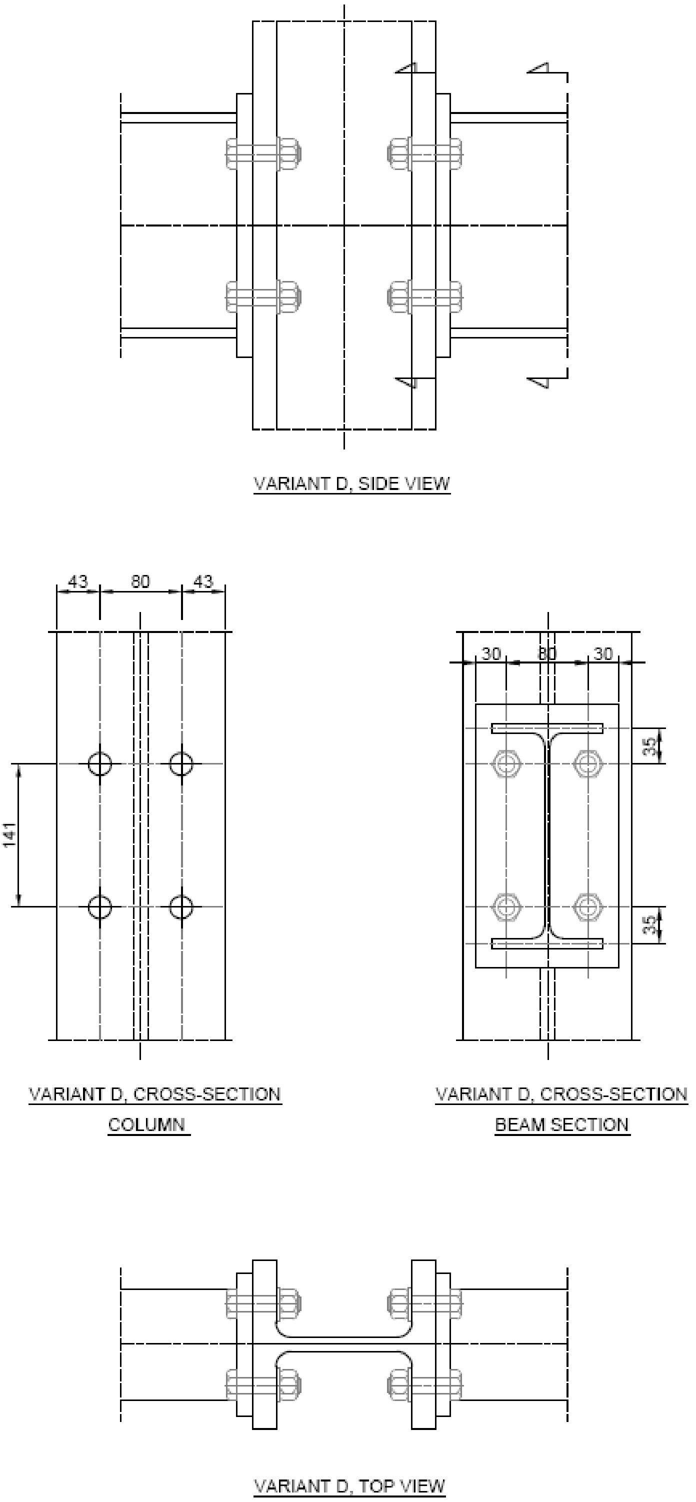


Figure 8.28: Flush End Plate MRJ, Variant D

Partial FEA models, variant A
The result of the first partial FEA model is depicted in figure 8.29. First failure mechanism of the model, failure of the bolts in tension zone, occurs at a applied load of $M_{Ed} = 25kNm$

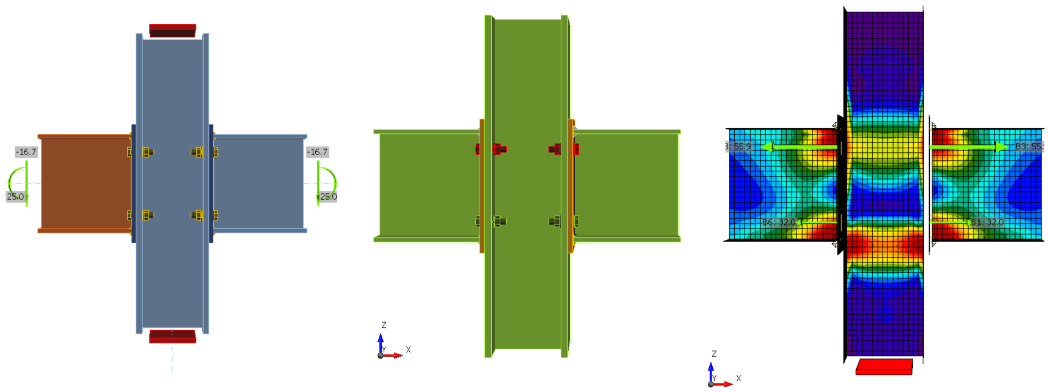


Figure 8.29: Partial FEA model Flush End Plate MRJ, Variant A

Partial FEA models, variant B
The joint is strengthened by resizing the the bolts. Again the model is sequentially loaded till failure. At a load of $M_{Ed} = 36kNm$ the joint fails by exceeding the 5 % limit strain in the end plate, see figure 8.30.

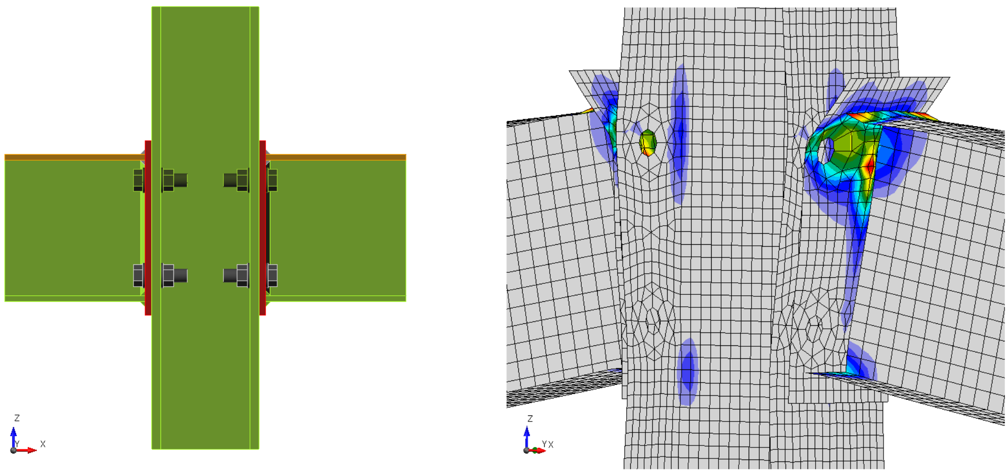


Figure 8.30: Partial FEA model Flush End Plate MRJ, Variant B

Partial FEA models, variant C & variant D

Same procedure has been executed. The joint is strengthened by applying a thicker end plate and loaded again till failure. The column flanges fail at a load of $M_{Ed} = 44kNm$, see figure 8.31. The column is strengthened by using a HEM profile, increasing the thickness of the flanges and web of the column. The next failure mechanism is failure of the beam at a load of $M_{Ed} = 50kNm$.

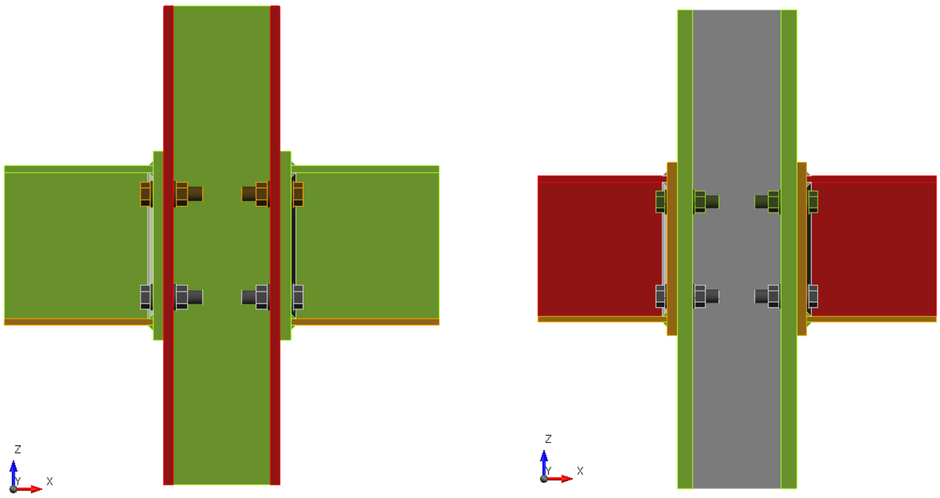


Figure 8.31: Partial FEA model Flush End Plate MRJ, Variant C & Variant D

Manual Calculation, Variant A-D

The configurations of variants A-D, are calculated manually. A new MATLAB script is developed based on previous scripts. Scripts are controlled manually, for example PS3. Manual calculations and MATLAB scripts can be found respectively Annex A and Annex B. Results of the manual calculations are shown in figure 8.32 and in figure 8.33.

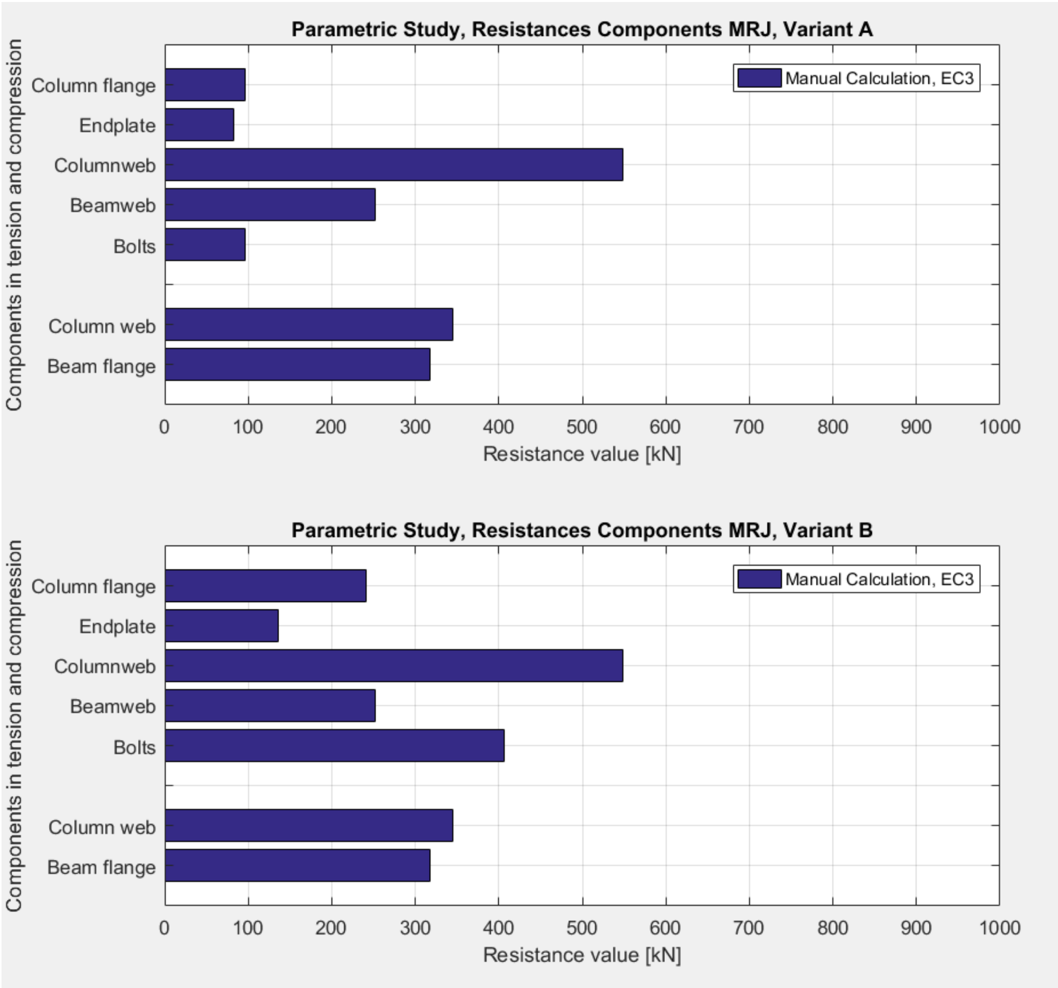


Figure 8.32: Results Manual Calculation, Variant A & Variant B

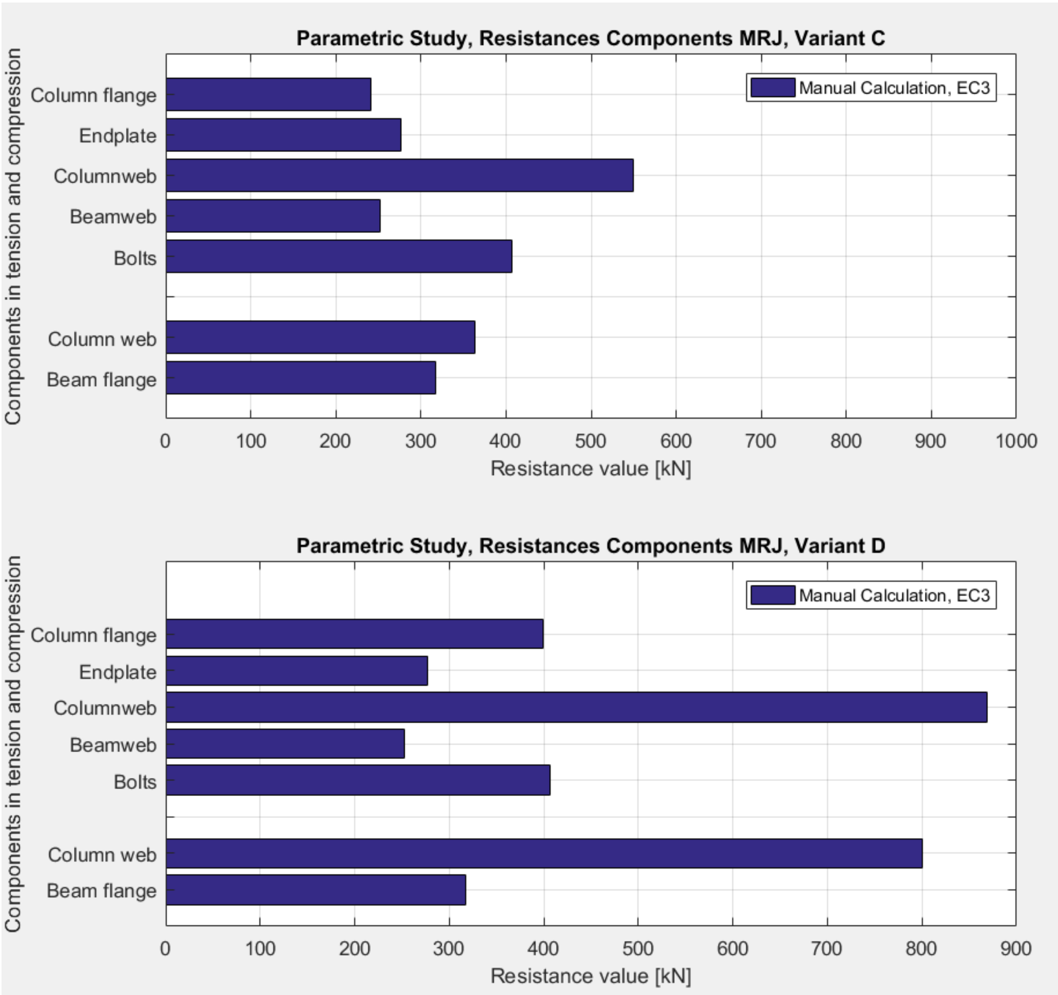


Figure 8.33: Results Manual Calculation, Variant C & Variant D

Configuration MRJ , variants E

Two extra configurations of MRJ's are designed. These configurations have an extended end plate and a double rowed bolt pattern. In variant E the column is unstiffened, see figure 8.34

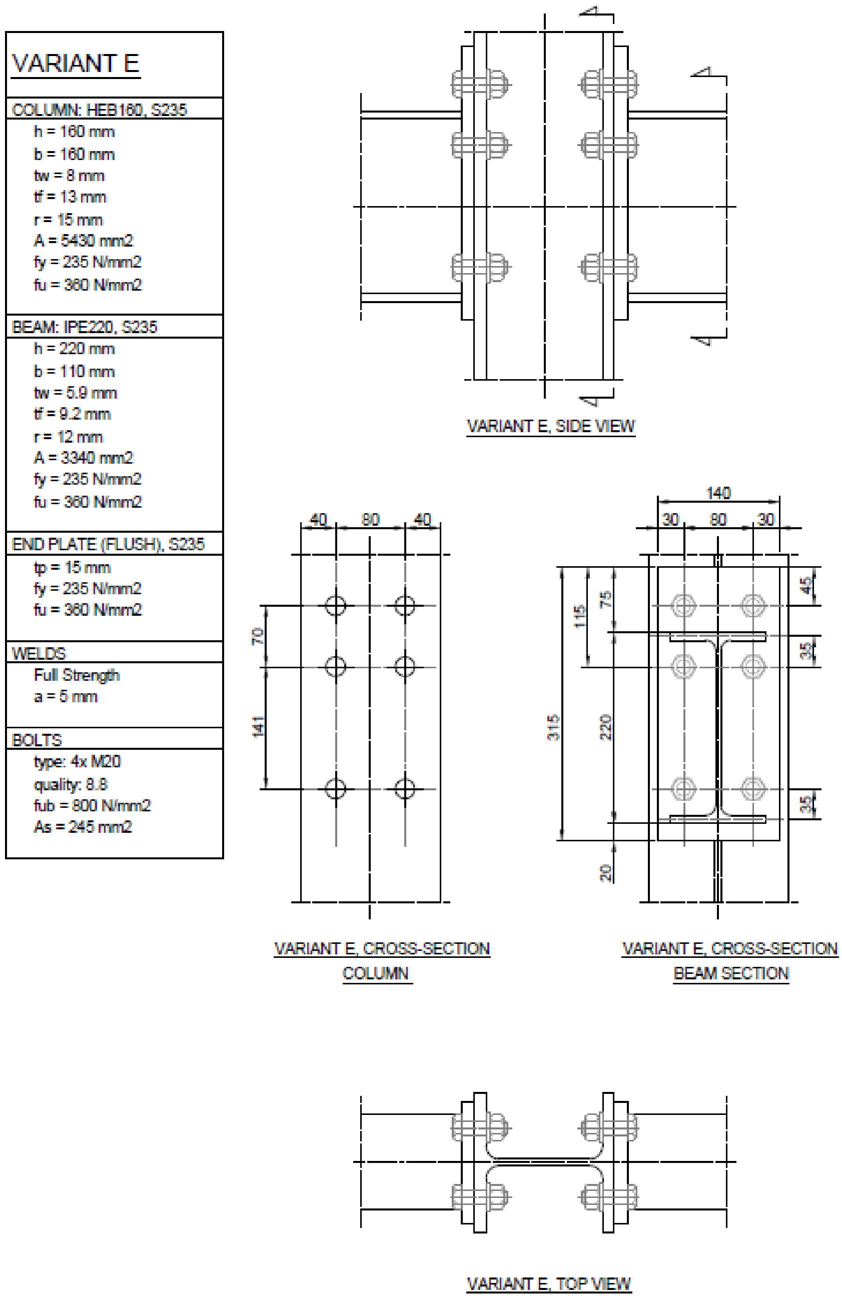


Figure 8.34: Flush End Plate MRJ, Variant E

FEA models MRJ's, variant E
Variant E is modelled. Results of this model are depicted in figure 8.35. In variant E, first failure mechanism which occurs is exceeding the 5 % limit strain in the compression zone of the MRJ.

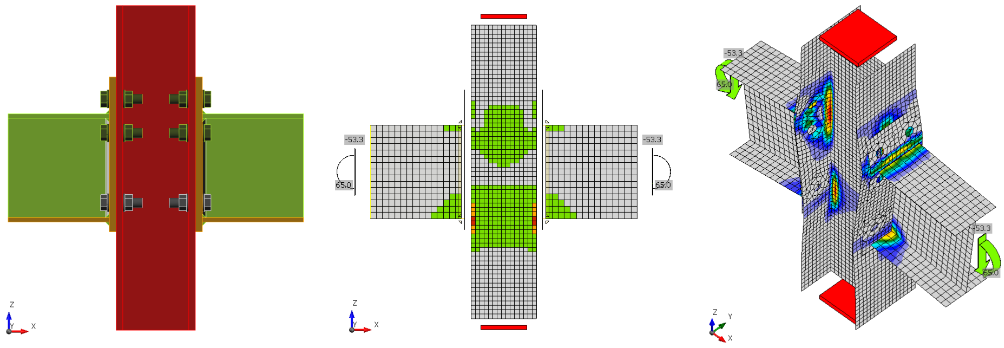


Figure 8.35: Partial FEA model Flush End Plate MRJ, Variant E

Manual Calculation, variant E
Results of the manual calculation are depicted in figure 8.32

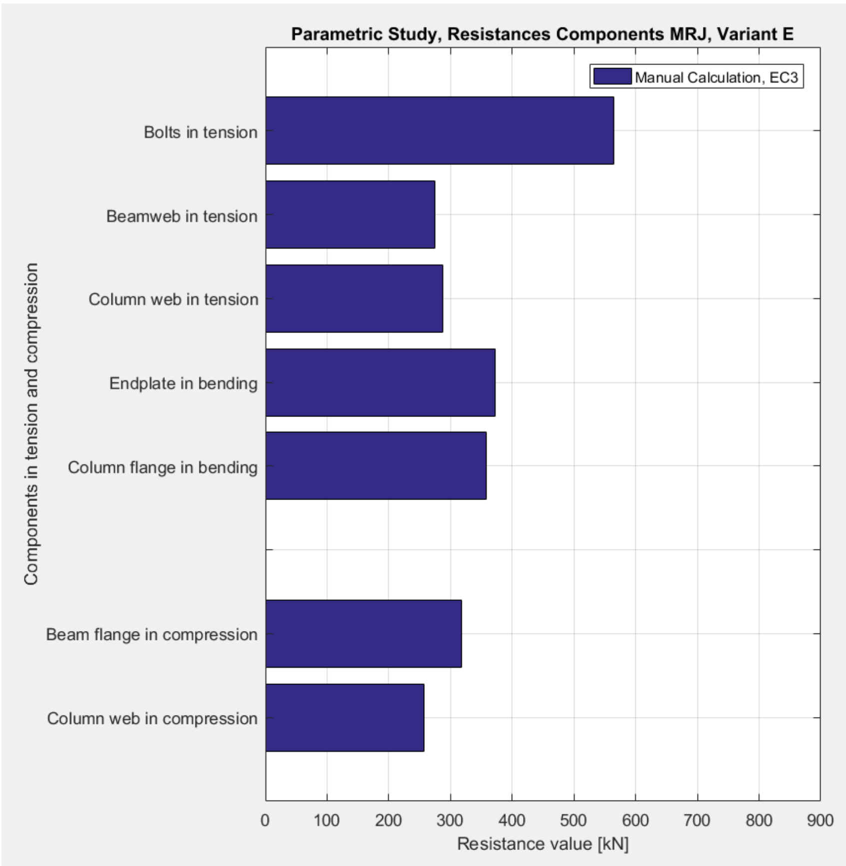


Figure 8.36: Results Manual Calculation, Variant E

Configuration MRJ , variant F

In the previous variant the column was the decisive component in joint according to the FEA model and manual calculation. In variant F the column is strengthened by designing two weld plates. Configuration of this joint is depicted in figure 8.37

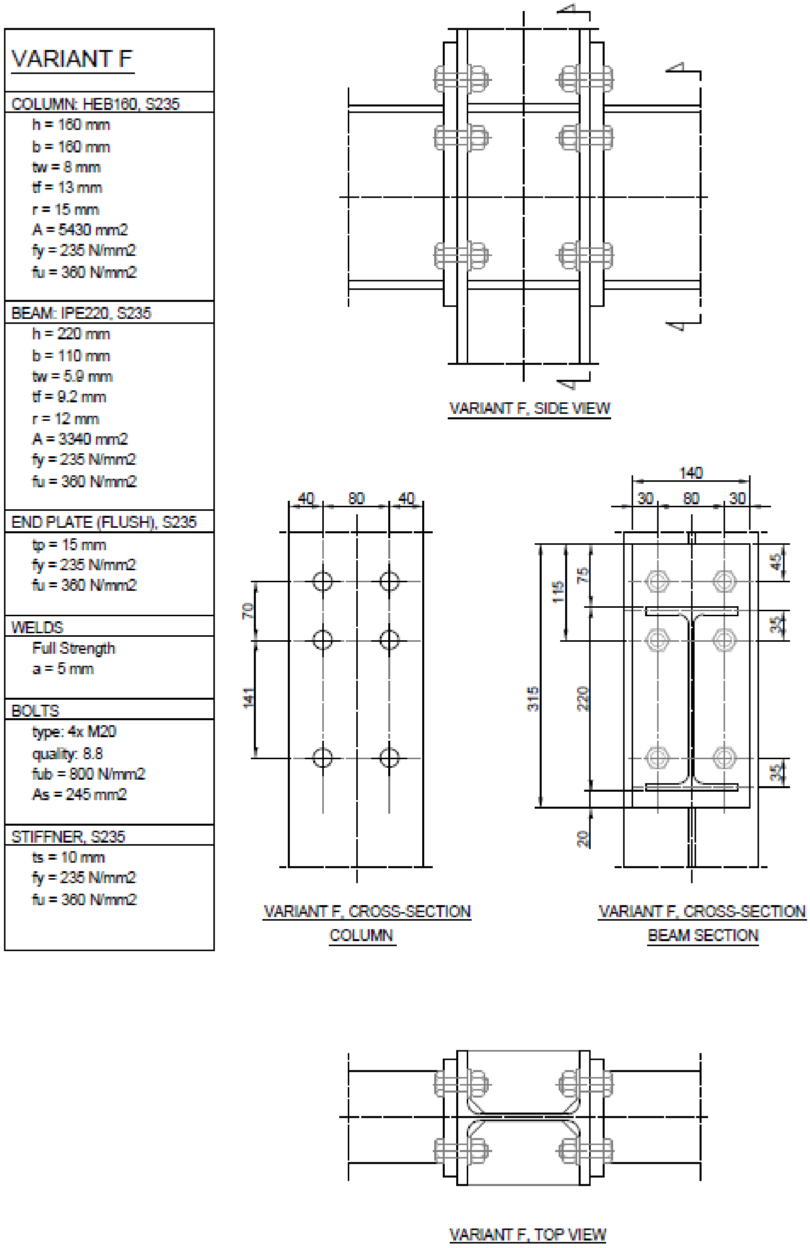


Figure 8.37: Flush End Plate MRJ, Variant F

FEA models MRJ, variant F

Variant F is modelled including the weld plates. Results of this model are depicted in figure 8.38. At a applied moment of $M_{Ed} = 74kNm$ the finite elements in the column flange are exceeding the 5 % limit strain.

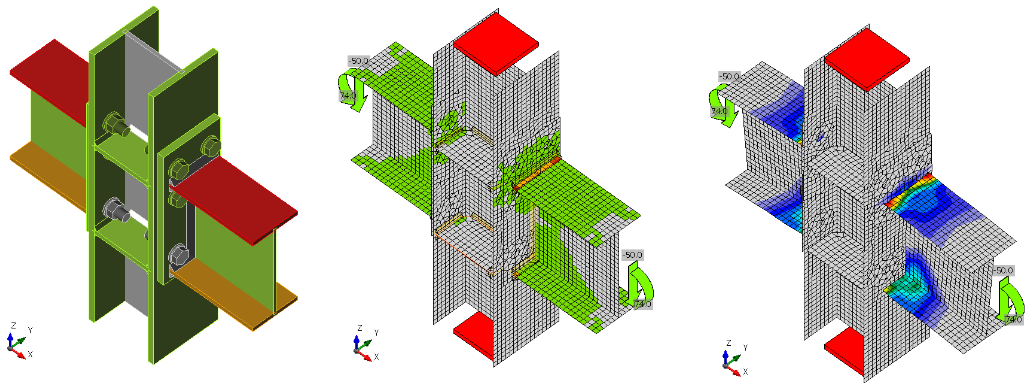


Figure 8.38: Partial FEA model Flush End Plate MRJ, Variant F

Manual Calculation, variant F

The joint is calculated manually. The column is strengthened with two weld plates. Column web in compression is not longer decisive, but the beam web in tension. The resistance of the stiffener is calculated according to calculation rules prescribed by the green book: Moment Resisting Joints To Eurocode 3 [3].

The following formula is used to determine the resistance of the stiffener. Also plate buckling of the stiffener should be checked.

$$N_{c,Rd} = \frac{A_{s,eff} * f_y}{\gamma_{M0}} \leq N_{b,Rd} = \frac{\chi * A_{s,eff} * f_y}{\gamma_{M1}} \tag{8.1}$$

where:

$$A_{s,eff} = (30 * \epsilon * t_w + t_s)t_w + 2 * b_{sg} * t_s \tag{8.2}$$

To check the occurrence of buckling of the stiffener, the non-dimensional slenderness is determined:

$$\overline{\lambda} = \frac{l}{i_s * \lambda_1}, \quad where \quad i_s = \sqrt{\frac{I_s}{A_{s,eff}}} \quad and \quad I_s = \frac{1}{12} * t_s * (2 * b_{sg} + t_{wc})^3 \tag{8.3}$$

If the non-dimensional slenderness remains lower than $\overline{\lambda} \leq 0.2$. No reduction is needed. Also the following check can be executed $\frac{b_{sg}}{t_s} < 14 * \epsilon$. This check is similar to classify cross-sections to class 3.

The results of the manual calculation is depicted in figure 8.39.

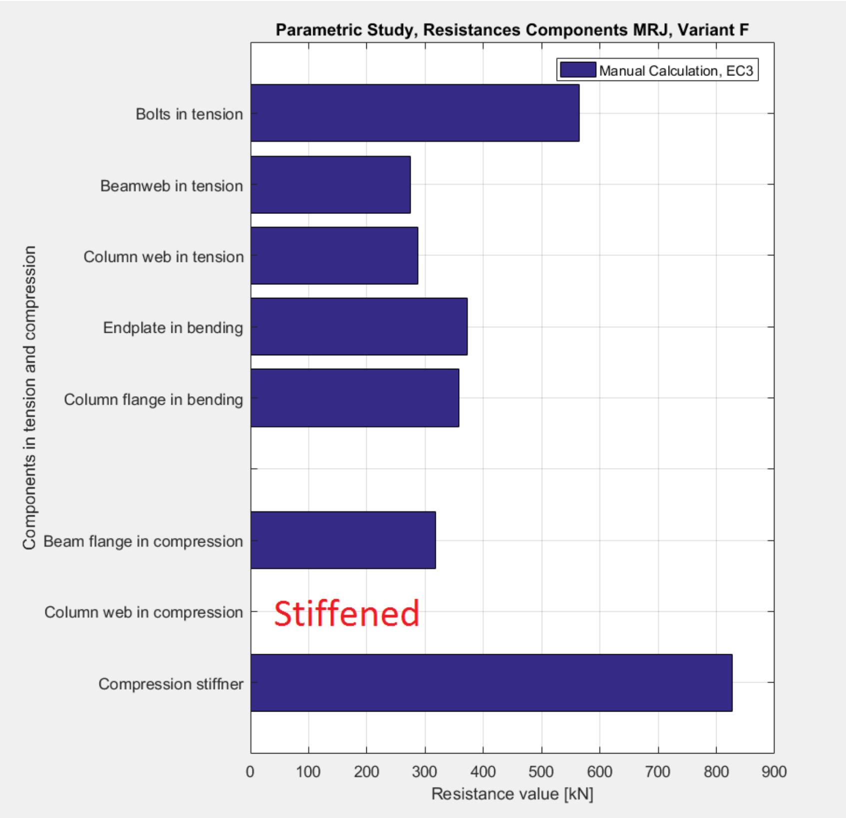


Figure 8.39: Results Manual Calculation, Variant F

Summary Results Parametric Study 3

Resistance values together with the decisive components of all the variants of moment resisting joints are presented in table 8.3.

Table 8.3: Results parametric study 3, Moment resisting joints

Moment Resisting Joint	$V_{Rd,EC3}$	Component	$V_{Rd,Model}$	Component
Flush End Plate				
Variant A	14 kNm	End Plate (T-stub FM 2)	20 kNm	Bolts
Variant B	24 kNm	End Plate (T-stub FM 1)	34 kNm	End Plate
Variant C	42 kNm	Column Flange	42 kNm	Column
Variant D	44 kNm	Beam Web	48 kNm	Beam
Extended End Plate				
Variant E	54 kNm	Column web	64 kNm	Column
Variant F	58 kNm	Beam Web	72 kNm	Beam

8.4 Evaluation failure mechanism partial FEA models

The behaviour of the partial FEA models are examined more in detail. Considered is whether the failure mechanisms can be recognised in those models as described by the Eurocode. First the strains are examined at a load where the joint still full fills the requirements. After, the joint is loaded till failure and beyond. Strain development the plated elements of the joint is examined.

8.4.1 Evaluation failure mechanisms variants parametric study one

PS1: Fin plate Joint, variant A; Strain development in the fin plate
In figure 8.40 the strain development in the fin plate can be seen. At a load of V_{Ed} = 220 kN, no critical strains occur. While increasing the load finite elements are exceeding the limit strain and the strains will be larger in those elements. Based on the initial location and the development of the straining, shear in the fin plate is a probable failure mechanism of this model.

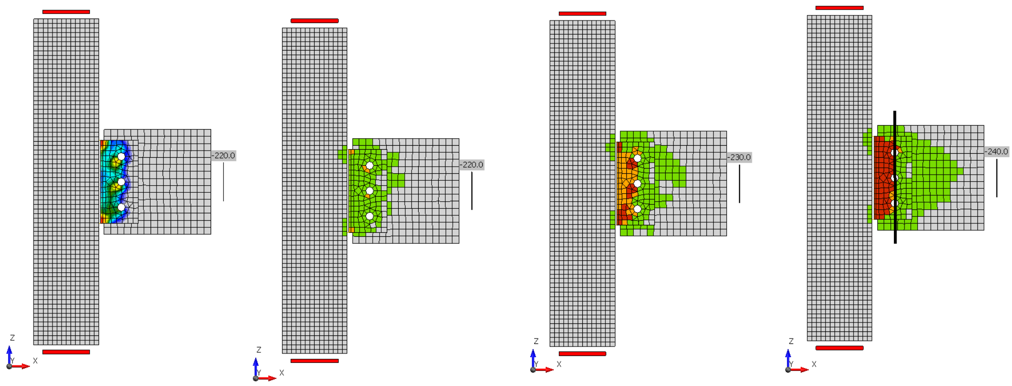


Figure 8.40: PS1, Variant A; V_{Ed} = 220 kN, V_{Ed} = 220 kN, V_{Ed} = 230 kN and V_{Ed} = 240 kN

PS1: Fin plate Joint, variant B; Strain development in the beam web
In figure 8.41 straining starts again at the location of the bolt holes. By further increase of the shear load the strain will be larger in those finite elements and also the surrounding elements are exceeding the limit strain. Based on the strain pattern shear failure or block tearing of the beam web is a probable failure mechanism of this model.

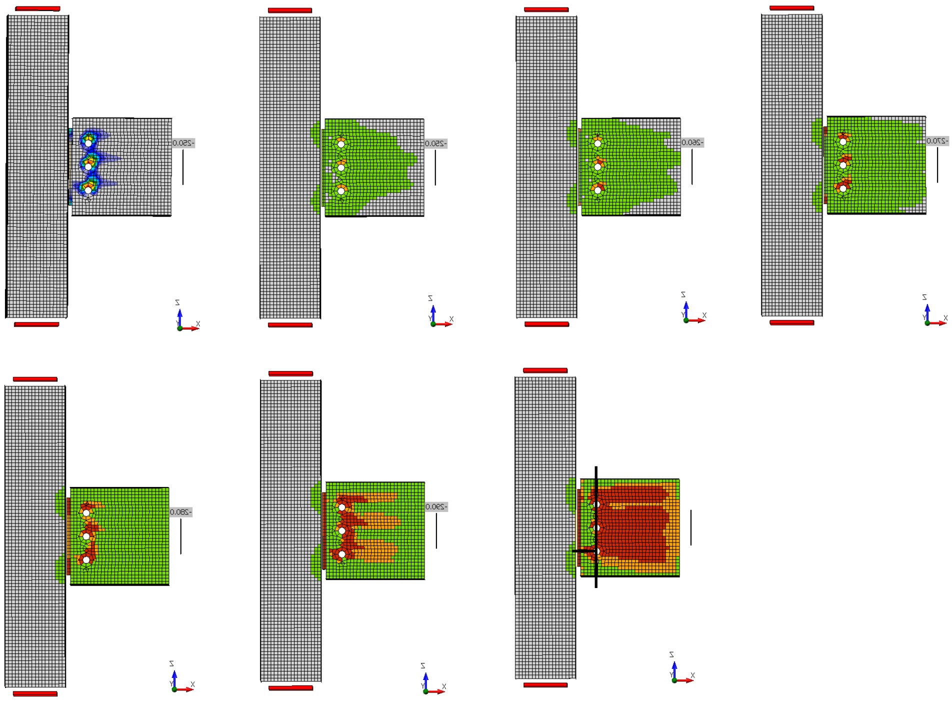


Figure 8.41: PS1, Variant B; $V_{Ed} = 250$ kN, $V_{Ed} = 250$ kN, $V_{Ed} = 260$ kN, $V_{Ed} = 270$ kN, $V_{Ed} = 280$ kN, $V_{Ed} = 290$ kN, $V_{Ed} = 300$ kN

PS1: Fin plate Joint, variant D; Strain development in the beam web

Strain development in figure 8.42 is similar to previous variant. Strains near the bolt holes are again exceeding the limit strain initially and will strain further after increasing the shear load. Based on the strain pattern block tearing is the probable failure mechanism.

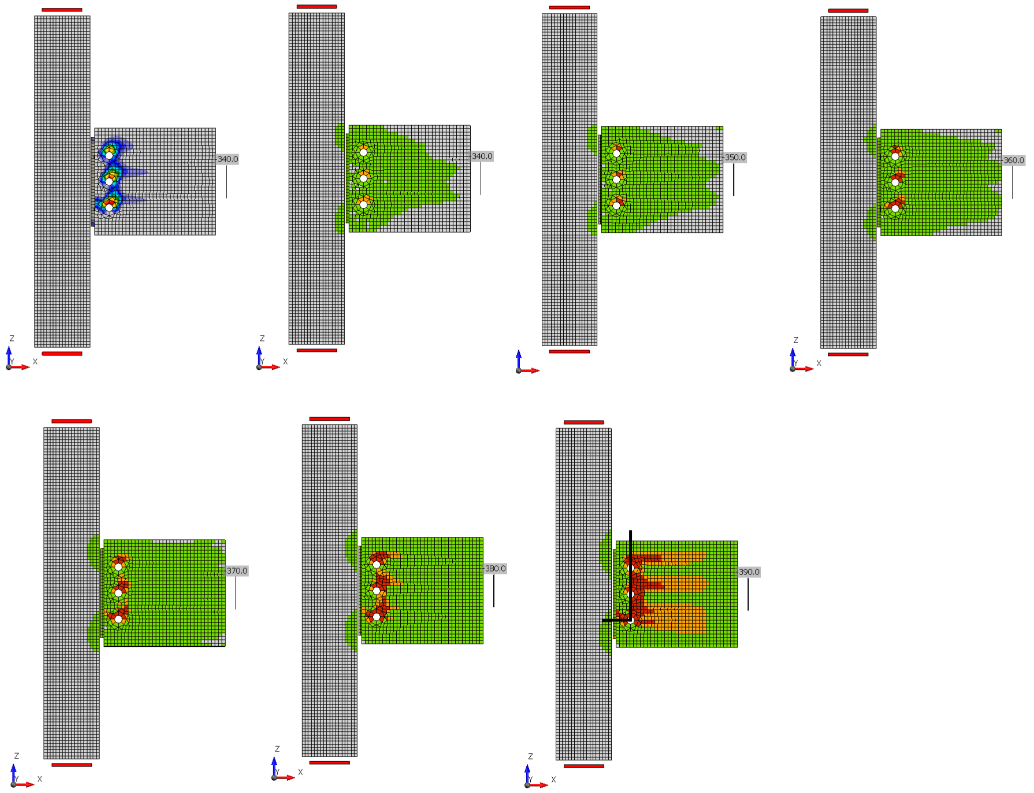


Figure 8.42: PS1, Variant D; V_{Ed} = 340 kN, V_{Ed} = 340 kN, V_{Ed} = 350 kN, V_{Ed} = 360 kN, V_{Ed} = 370 kN, V_{Ed} = 380 kN, V_{Ed} = 390 kN

8.4.2 Evaluation failure mechanisms variants parametric study three

PS3: Moment Resisting Joint (Flush), variant B; Strain development in the flush end plate and beam

In figure 8.43 can be seen that first elements are exceeding the limit strain in the tension zone. Elements of the beam flange and end plate are exceeding the limit strain. After load increase straining develops in the end plate and connected welded parts, the beam flange and the beam web, which are connected to the end plate by welds.

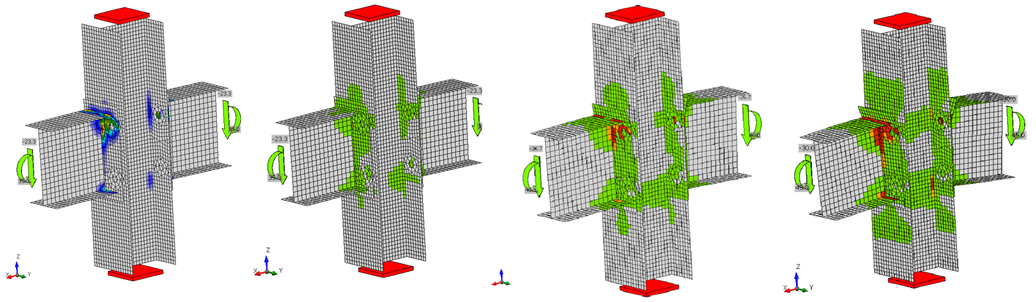


Figure 8.43: PS3, Variant B; M_{Ed} = 35 kNm, M_{Ed} = 35 kNm, M_{Ed} = 40 kNm, M_{Ed} = 45 kNm

PS3: Moment Resisting Joint (Flush), Variant C; Strain development in column

In figure 8.44 strain development can be noticed in the plated elements of the column. First elements are exceeding the limit strain in the flanges which are in the vicinity of the beam web in the tension zone of the joint. After load increase also elements in column in the compression zone of the joint are exceeding the limit strain.

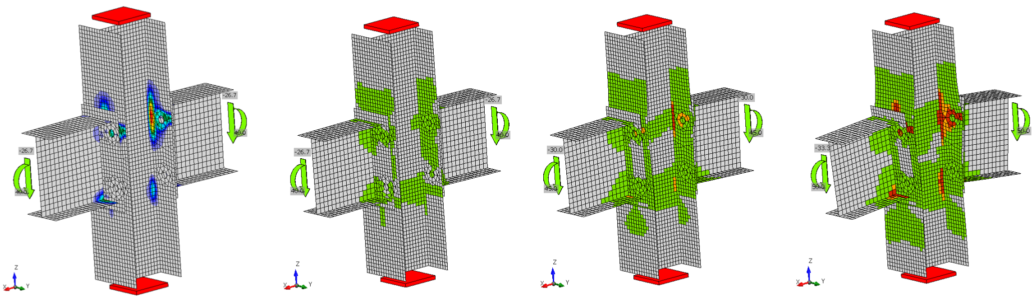


Figure 8.44: PS3, Variant C; M_{Ed} = 40 kNm, M_{Ed} = 40 kNm, M_{Ed} = 45 kNm, M_{Ed} = 50 kNm

PS3: Moment Resisting Joint (Flush), Variant D; Strain development in beam
In figure 8.45 strains are developing in the tension zone of the beam. First elements are exceeding the limit strain at a load of the $M_{Ed} = 45$ kNm in tension of the beam. After load increase also elements in the compression zone of the beam are exceeding the limit strain.

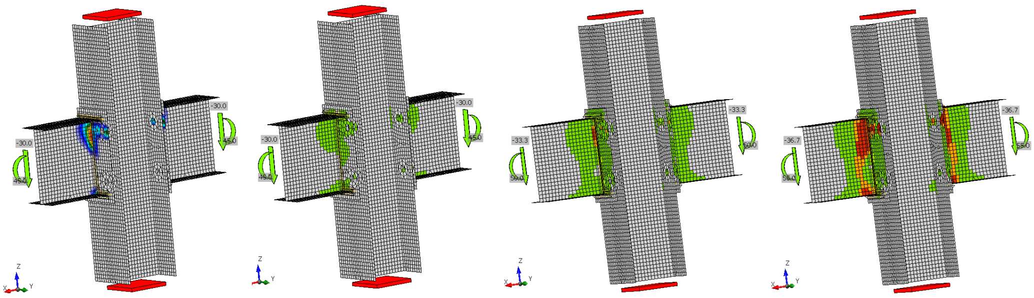


Figure 8.45: PS3, Variant D; $M_{Ed} = 45$ kNm, $M_{Ed} = 45$ kNm, $M_{Ed} = 50$ kNm, $M_{Ed} = 55$ kNm

PS3: Moment Resisting Joint (Extended), Variant E; Strain development in column
In figure 8.46 first elements are straining in the column web of the joints at a load of $M_{Ed} = 65$ kNm. Also some elements are exceeding the limit strain in the end plate. After increasing the load, straining develops in the column in tension zone and compression zone. Also at the location of end plate - beam flange connection and in the compression zone of the beam flange straining develops.

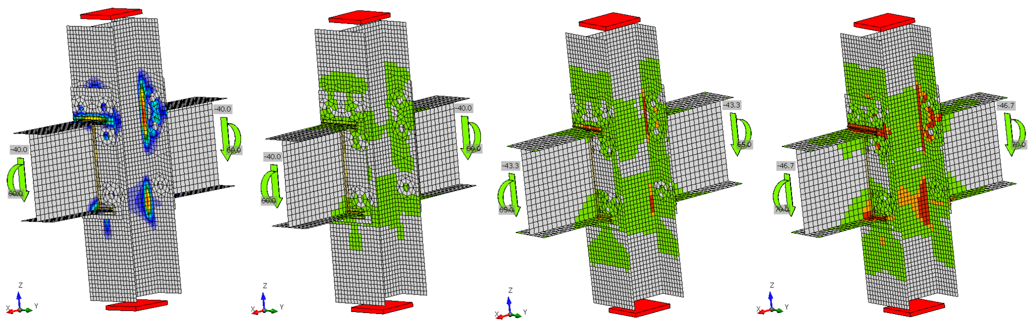


Figure 8.46: PS3, Variant E; $M_{Ed} = 60$ kNm, $M_{Ed} = 60$ kNm, $M_{Ed} = 65$ kNm, $M_{Ed} = 70$ kNm

PS3: Moment Resisting Joint (Extended), Variant F; Strain development in the beam

In figure 8.47 can be seen that first finite elements fail at the location of the beam flanges. After increasing the bending moment straining develops from the flanges in the beam to the web.

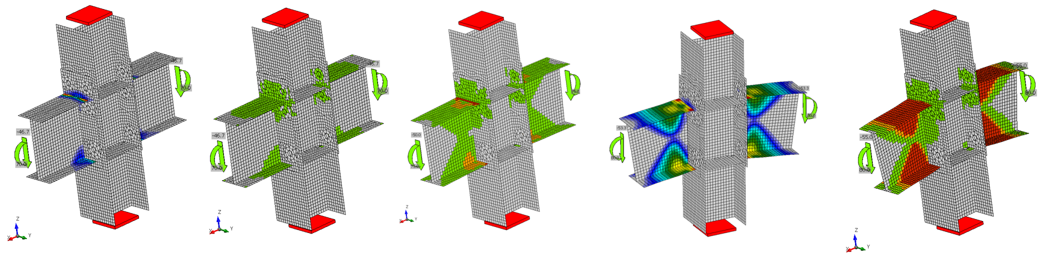


Figure 8.47: PS3, Variant F; M_{Ed} = 70 kNm, M_{Ed} = 70 kNm, M_{Ed} = 75 kNm, M_{Ed} = 80 kNm

Chapter 9

Conclusion

Hollandia Structures B.V. wanted to conduct market research of software, with or without FEM applications, specifically developed for the calculation of steel connection. IDEA Statica Connection is a suitable programme which partially uses a Finite Element Analysis (FEA) in combination with the calculation rules prescribed in the Eurocode 3 (EC3: NEN-EN 1993-1-1 and EC3: NEN-EN 1993-1-8).

9.1 Summary Results and Conclusions

This thesis explores the difference between the traditional calculation methods for steel connection and the (CB)FEM method, which is used in the IDEA Statica Connection software. The main difference in approach is that resistances of the different components are determined by effective lengths , while in the (CB)FEM method stresses and corresponding strains are calculated by the Finite Element Method (FEM). To calculate the total resistance of a connection traditionally, the governing component has to be determined, while in the CB(FEM) method the principal strain in the plated elements in the connection must remain under a specific limit value of 5% in each finite element. (2D-elements)

In this thesis the following calculation are made:

Simple Shear Joints: CA1, CA1-V, CA2, CA3
Moment Resisting Joints: CA4, CA5, CA6
Moment Resisting Joints: C11-EV, C11-EC3, C22-EV, C22-EC3

The configuration of the first two groups of joints, the simple shear joints and moment resisting joints, were calculated manually according to the Eurocode 3 (EC3). Besides, partial FEA models were made with the same joint configuration. Results from the manual calculation and FEA model were compared to each other.

The configuration of the joints of the last group was tested in a research programme. Tensile test on coupons extracted from the beams and columns were carried out. This was aimed at characterizing the actual properties of the material. These experimental values (EV) were used to calculate the resistance of the different components to manually determine the total moment resistance, without the use of the partial safety factors. The FEA was executed with bi-linear material behaviour using the yield strength values obtained from the tensile tests. Results of the manual calculation and partial FEA model were compared to the experimental results. In addition, the same calculations were executed with prescribed yield- and tensile strength values according to the EC3 with partial safety factors.

The following end results are obtained in this comparative research:

Table 9.1: Results; Manual Calculation, FEA models and Experiments

Number	**Joint**	**Type**	**MC**	**FEA**	**EXP**
CA1	*Fin Plate*	SSJ	178 kN	220 kN	-
CA1-V	*Fin Plate Variant*	SSJ	157 / 173 kN	145 kN	-
CA2	*Short End Plate*	SSJ	222 kN	250 kN	-
CA3	*Double Angle Cleats*	SSJ	340 kN		-
	- FEA model, RC1			325 kN	
	- FEA model, RC2			300 kN	
CA4	*Bolted Extended End Plate, symmetrical*	MRJ	48 kNm	45 kNm	-
CA5	*Bolted Extended End Plate, asymmetrical*	MRJ	35 kNm	44 kNm	-
CA6	*Welded Joint*	MRJ	235 kNm	300 kNm	-
CA11-EV	*Flush End Plate*	MRJ	65 kNm	80 kNm	85 kNm
CA11-EC3	*Flush End Plate*	MRJ	51 kNm	65 kNm	85 kNm
CA22-EV	*Extended End Plate*	MRJ	109 kNm	125 kNm	125 kNm
CA22-EC3	*Extended End Plate*	MRJ	82 kNm	105 kNm	125 kNm

Abbreviations in table:

CA =	Calculation	EC3 =	Eurocode 3 Values
MC =	Manual Calculation	EXP =	Experimental Results
FEA =	Finite Element Analysis Model	SSJ =	Simple Shear Joint
EV =	Experimental Values	MRJ =	Moment Resisting Joint

It can be concluded that higher resistance values are provided by the FEA models. Manual calculation using the calculation rules prescribed by the Eurocode give more conservative results. The exception is where bolts are the weakest component in the joint. In manual calculation forces on bolts are equally divided over the bolts and moment equilibrium is made relative to the assumed rotation centre in joint. In the FEA models the forces are non-uniformly distributed over the bolts. In some cases, when bolts are decisive in the joint, the FEA model will give a lower resistance value. (Some) bolts will fail earlier than will be shown from a manual calculation.
For the simple shear joints (SSJ) it holds that, in general, in manual calculation as for the FEA model the same components are governing in the joints, although the total resistance values are differing. (1) For the fin plate joint it holds that the welded fin plate is the weakest component in manual calculation Shearing is the governing failure mechanism. In the FEA model 5% limit strain is reached as governing failure mechanism. (2) For the short end plate it holds that the beam web is the governing component of the joint. Shearing is the governing failure mechanism. In the FEA model 5% limit strain is reached in the beam web of the joint as governing failure mechanism. (3) For the double angle cleat joint the weakest component is the beam web. Tearing resistance is the governing failure mode. For the FEA model the upper bolt in the bolt pattern in the angle cleats - beam web connection is governing. This difference can be explained by elastic distribution of forces over the bolts, while in manual calculation equal (plastic) distribution is assumed. Not long after exceeding the bolt resistance, the 5 % limit strain is reached in the beam web, which is also the weakest component in manual calculation. The results from manual calculation and FEA model are similar, but the deformation of the model is not realistic. In a second model with another assumption of the rotation centre, the deformation is realistic. In this case the upper bolts in the column flange - angle cleats connection are governing failing in tension-shear, caused by eccentricity due to transferring the rotation centre.

For the moment resisting joint (MRJ) in cases with a-symmetrical loading it holds that bending moment resisting values obtained are lower than is shown by the Finite Element Analyses. For the moment resisting joint which is symmetrically loaded the resistance value are similar for the manual calculation and the FEA model. The end plate is the governing component in hand calculation. In the FEA model the end plate also governing . The 5% limit strain is reached at the finite elements at the location at the governing yield line near the welded end plate top beam flange connection. This yield line is also governing in manual calculation. Both for the asymmetrically bolted joint and for the asymmetrically welded joint the moment resistance must be reduced following the prescribed rules in EC3. Reduction is needed to obtain horizontal equilibrium because of the governing compression zone or shear zone. Also, the reduction factor for interaction between tension and shearing (ω) in the column flange should be considered as well as the reduction factor for plate buckling (ρ). In the computational models reduction in moment resistance cannot be noticed. What can be noticed is a developing of a shearing in the column web. Straining occurs in the shear zone of joint during loading. For the FEA model of the bolted asymmetrically loaded moment resting connection failure of the beam web occurs by reaching 5% limit strain in the compression zone. For the manual calculation the column web in compression zone is the governing component. For the welded moment resisting joint the column web the FEA model fails simultaneous in tension and compression zone, which contrasts with the manual calculation where first compression zone should be governing prior to the tension zone because of buckling sensitivity.

A comparative study was conducted between manual calculation, FEA models and results of executed experiments. This was done for flush end plate moment resisting joint and an extended end plate moment resisting joint. First calculations were executed with realistic material properties, without usage of partial safety factors. After the prescribed EC3 values were used with partial factors from the National Annex (Dutch). For both types of joints it can be concluded that manual EC3 calculations are the most conservative. Using the experimental values, the FEA models give higher resistance values than shown in experiments. When using the prescribed EC3 design values, both the manual calculations and the FEA models remain under bending moment resistance from the experiments.

What was noticed further is that in a manual calculation it is assumed that a linear bolt pattern is developing during load increase. After reaching the full elastic capacity a plastic bolt pattern is developing. The transition from a elastic to a plastic bolt distribution cannot be noticed in the partial FEA models. In these models the bolts located in the stiffer parts of the connection take up directly more force than the bolts in the weaker parts.

Graphic representation of the final results for simple shear joints are presented in figure 8.1. The final results for the moment resisting joints are presented in figure 8.2.

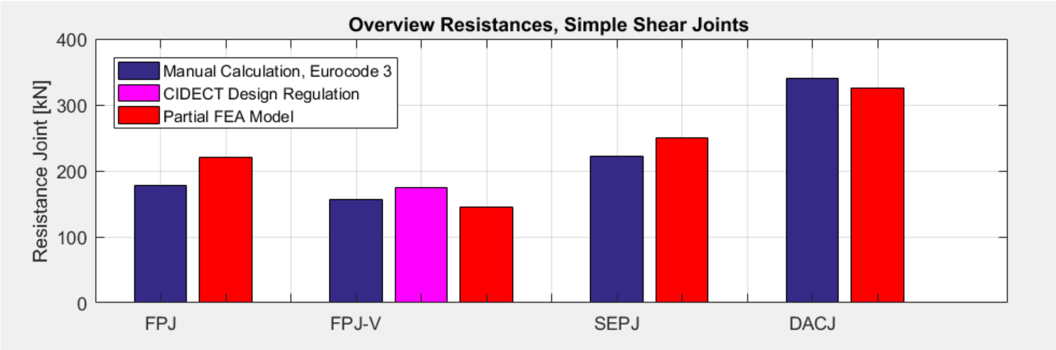


Figure 9.1: Overview resistance values of simple shear joints.

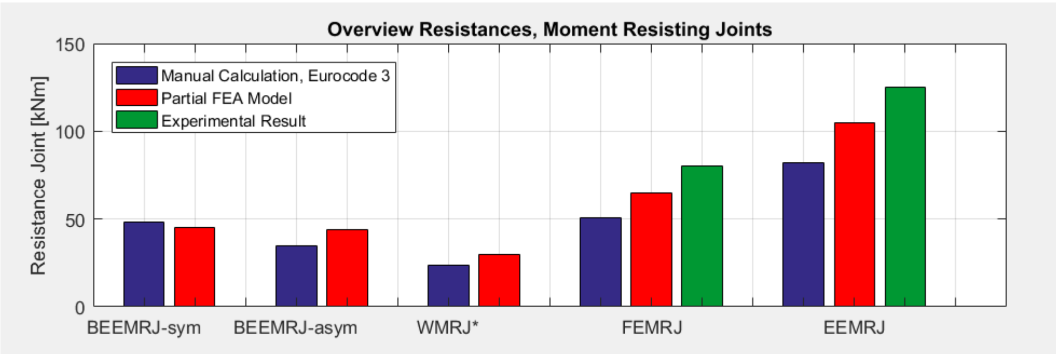


Figure 9.2: Overview resistance values of moment resisting joints, WMRJ*: Resistance Value $*10^{-1}$.

Abbreviations in figures:

FPJ =	Fin Plate Joint
FPJV =	Fin Plate Joint Variant
SEPJ =	Short End Plate Joint
DACJ =	Double Angle Cleat Joint
BEEMRJ-sym =	Bolted Extended End Plate Moment Resisting Joint *symmetrical loaded*
BEEMRJ-asym =	Bolted Extended End Plate Moment Resisting Joint *asymmetrical loaded*
WMRJ	Welded Moment Resisting Joint
FEMRJ	Flush End Plate Moment Resisting Joint
EEMRJ	Extended End Plate Moment Resisting Joint

A parametric study has been executed for two types of simple shear joints. A standard joint is designed and is loaded till failure. Based on the occurring failure mechanisms, components of the joint are strengthened. In this thesis the following parametric studies have been executed:

PS1: Fin Plate Joint
PS2: Short End Plate Joint
PS3: Moment Resisting Joint, Flush- and Extended End Plate

Results of the parametric studies can be seen in figure 8.3. In general can be concluded that manual calculation and partial FEA models give similar results, but not in every case the same components of the joint are decisive for each calculation method. In some cases bolts in the model will fail earlier in the FEA models than that will be shown in manual calculations. The force in the bolts will not be divided equally over the number of bolts in the partial FEA models. Also tensile forces may appear in bolts by deformation of the model. Consequence is that failure of the bolt by tensile shear interaction may occur earlier.

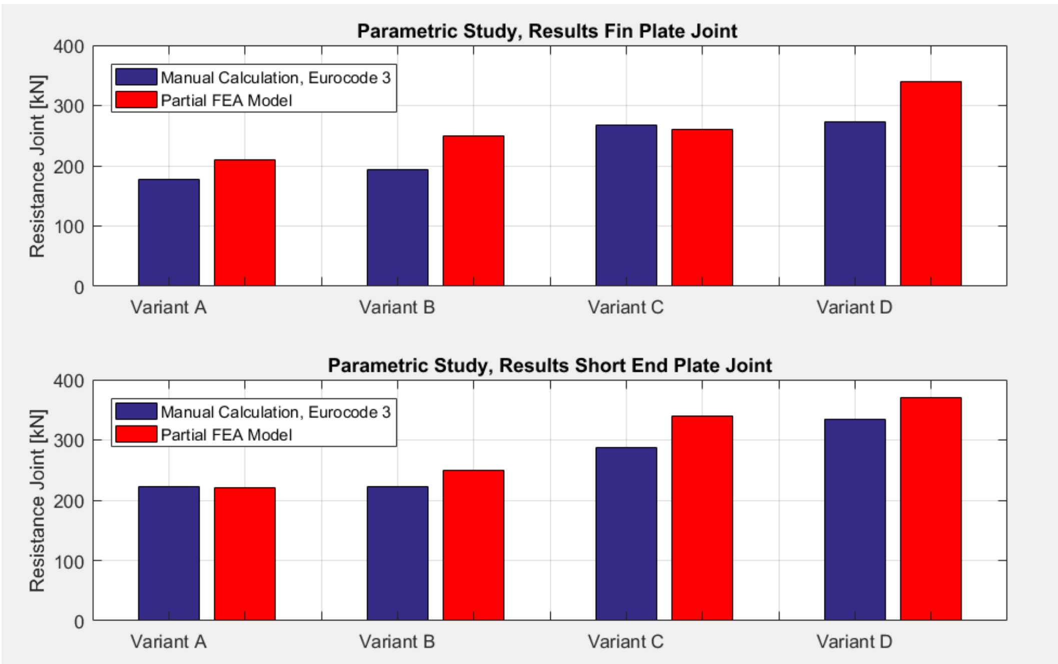


Figure 9.3: Results Parametric Study; Fin Plate Joint and Short End Plate Joint

Same procedure has been executed for moment resisting joints. Two types are are evaluated: Flush end plate joint (Variant A-D) and extended end plate joint (Variant E & F). Results of this comparative study can be found in figure 9.4.

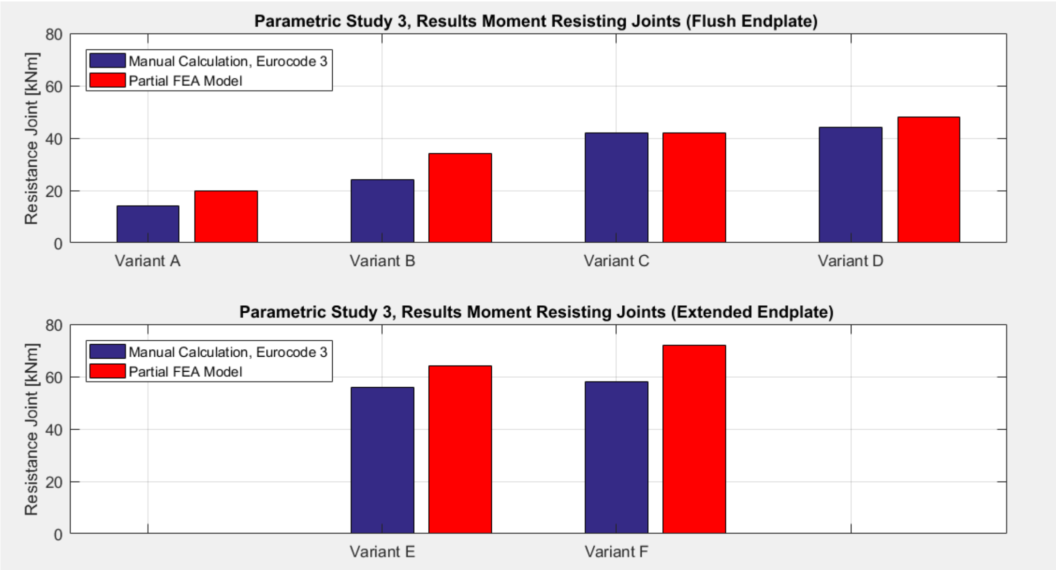


Figure 9.4: Results Parametric Study Moment Resisting Joints

9.2 Recommendations

Based on the performed work in this thesis, the following recommendations are made for future work: (1) In this thesis specific types of joints were evaluated. Other configuration can also be examined and compared to each other. MATLAB scripts, presented in Annex B, may be very useful for this purpose. (2) In addition, there are more types of joints, i.e. column splices, plated joints, composite joints and joints in 3D-space, which can be evaluated as well.

Bibliography

[1] Beton Vereniging and Bouwen met Staal. Course: Staalconstructeur MSEng, lecture 3.

[2] Betonconstructies Commissie: TBG. Eurocode 2: NEN-EN 1992-1-1: Ontwerp en berekening van betonconstructies, 2011.

[3] David Brown and David Iles. *The Green Book, Moment-Resisting Joints to Eurocode 3*. Jointly published by: The Steel Construction Institute and The British Constructional Steelwork, 2014.

[4] D.A. Hordijk and J.W.B. Stark. *Kolomvoetplaatverbindingen*. CURNET / Bouwen met Staal, 2009.

[5] Luís Simões da Silva, Pedro C.G. da. S. Vellasco, and Sebastiãno A.L. de Andrade. Behaviour of flush end-plate beam-to-column joints under bending and axial force. 2004.

[6] L.R.O de Lima, L.S. Silva, P.C.G. da Vellasco, and S.A.L. de Andrade. Experimental evaluation of extended endplate beam-to-column joints subjected to bending and axial force. *Elsevier*, 2004.

[7] Edurne Nenez (primary drafter) Moreno. *The Green Book, Joints in steel construction: simple joints to Eurocode 3*. Jointly published by: The Steel Construction Institute, The Britisch Constructional Steelwork Association, 2014.

[8] L. Gardner. *Stability of Steel Beams and Columns*. SCI, Ascot, 2011.

[9] Hollandia Structures B.V. Internal Documentation.

[10] Jean-Pierre Jaspart and Klaus Weynand. *Design of Joints in Steel and Composite Structures*. Wiley Ernst & Sohn; ECCS CECM EKS, 2016.

[11] Nederlands Normalisatie Instituut. Eurocodes in Nederland.

[12] Nederlands Normalisatie Instituut. What is normalisation?

[13] Nederlands Normalisatie Instituut. NEN-EN 1090-2, Execution of steel structures and aluminium - Part 2: Technical requirements for steel structures, 2008.

[14] Normcommissie 351001 "Technische Grondslagen voor Bouwconstructies". Eurocode 0: NEN-EN 1990, 2002.

[15] Normcommissie 351001 "Technische Grondslagen voor Bouwconstructies". Eurocode 3: NEN-EN 1993-1-8, 2006.

[16] Normcommissie 351001 "Technische Grondslagen voor Bouwconstructies". NEN-EN 1090, part 1-3, 2011.

[17] Normcommissie 351001 "Technische Grondslagen voor Bouwconstructies". Eurocode 3: NEN-EN 1993-1-5, 2012.

[18] Qec Excel-Rekenbladen. www.qec.nu.

[19] M. Saidani and D.A. Nethercot. Connection Design and Detailing. Engineering Basis Document 6. Fin plate connection with hollow section column., 1995.

[20] J.W.B. Stark. *Verbinden. Kenmerken van verbindingen in staalconstructies en het berekenen van mechanische verbindingsmiddelen en lassen volgens Eurocode 3.* Bouwen met Staal, Zoetermeer, 2012.

[21] J.W.B. Stark and J Wardernier. *Knopen. Berekenen van geboute en gelaste verbindingen in raamwerken en in buisconstructies volgens Eurocode 3.* Bouwen met Staal, 2014.

[22] IDEA STATICA. IDEA StatiCa Connection, Theoretical background. www.ideastatica.com, 2016.

[23] Steel Construction Info. http://www.steelconstruction.info/.

[24] TATA Steel. Design of Welded Joints. Technical report, Corby, 2013.

[25] TATA Steel. Welded Joints Examples. Technical report, Corby, 2013.

[26] W.H. Verburg. *Basisboek (overspannend) Staal.* Bouwen met Staal, Zoetermeer, 4 edition, 2004.

[27] Frantisek Wald. *Benchmark Cases For Advanced Design of Structural Steel Connections.* Ceska technika - nakladatelstvi CVUT, 2016.

[28] J Wardernier, J.A. Packer, X.-L Zhao, and G.J. van der Vegte. *Hollow Sections in Structural Apllications.* CIDECT, Geneva, 2nd edition, 2010.

Chapter 10

Appendices

Appendix A

Manual Calculations

The following calculation are done manually:

- CA1: SSJ, Fin Plate
- CA1-V: Fin Plate (Variant)
- CA2: SSJ, Short End Plate
- CA3: SSJ, Double Angle Cleats
- CA4: MRJ, Extended Endplate Joint symmetrical loading
- CA5: MRJ, Extended Endplate Joint unsymmetrical loading
- CA6: MRJ, Welded Joint
- CA11-EXP: MRJ, Flush End Plate Joint
- CA11-EC3: MRJ, Flush End Plate Joint
- CA12-EXP: MRJ, Extended End Plate Joint
- CA12-EC3: MRJ, Extended End Plate Joint
- PS3: MJR, Flush- and Extended End Plate Joint, Calculation Check.

Elaboration of these manual calculation (PDF's) can be found on the enclosed on the USB-flashdrive of this thesis. Only the fronts pages of the manual calculations are attached in this this thesis.

Project :

Project no.:

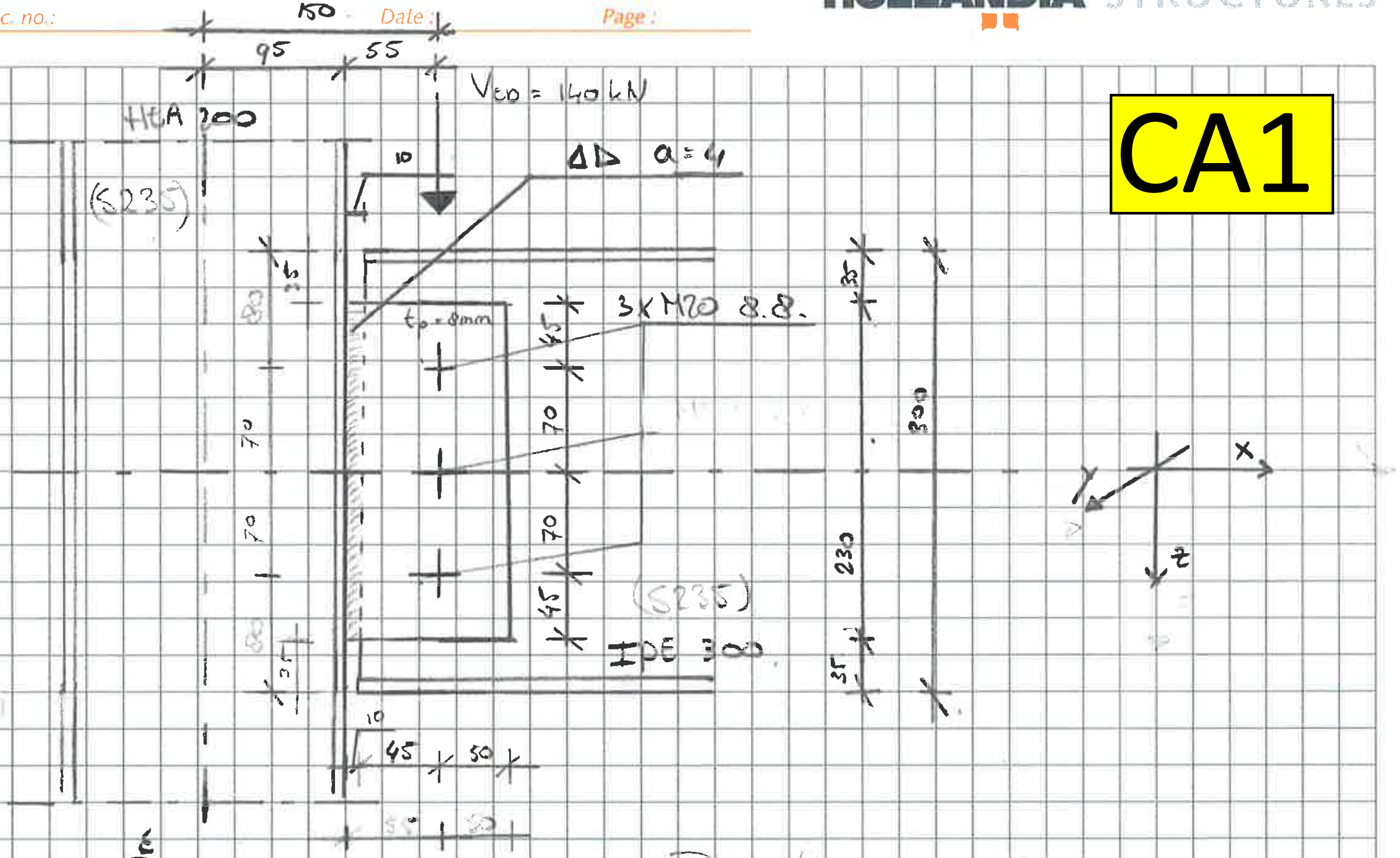
Doc. no.:

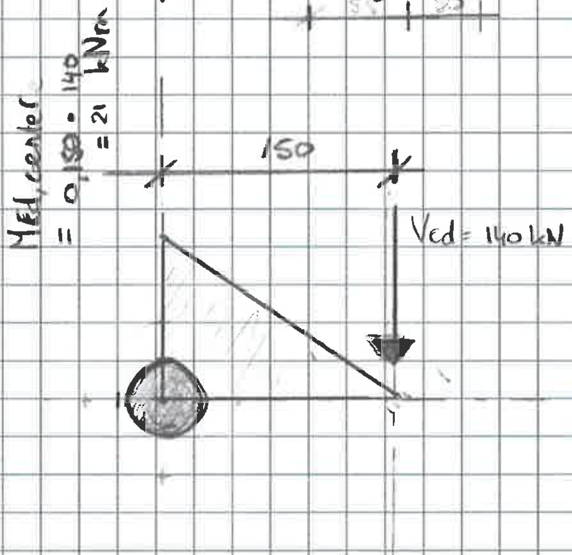
Date :

Page :

HOLLANDIA STRUCTURES

CA1





Properties

Column HEA 200 S235

h = 190 mm		f_y = 235 N/mm²	
b = 200 mm		f_u = 360 N/mm²	
t_w = 6,5 mm			
t_f = 10 mm			
r = 18 mm			
h_i = 134 mm			
A = 5380 mm²			

Beam : IPE 300 S235

h = 300 mm

b = 150 mm

t_w = 7,1 mm

t_f = 10,7 mm

r = 15 mm

h_i = 248 mm

A = 5380 mm²

Safety factors : art. 6.1 (NEN-EN 1993-1-8)

γ_{M0} = 1,0

γ_{M2} = 1,25

Bolts : M20 8.8

A_s = 245 mm² d_0 = 22 mm

f_{ub} = 800 N/mm²

f_{yb} = 640 N/mm²

Welds : Double fillet (S235)

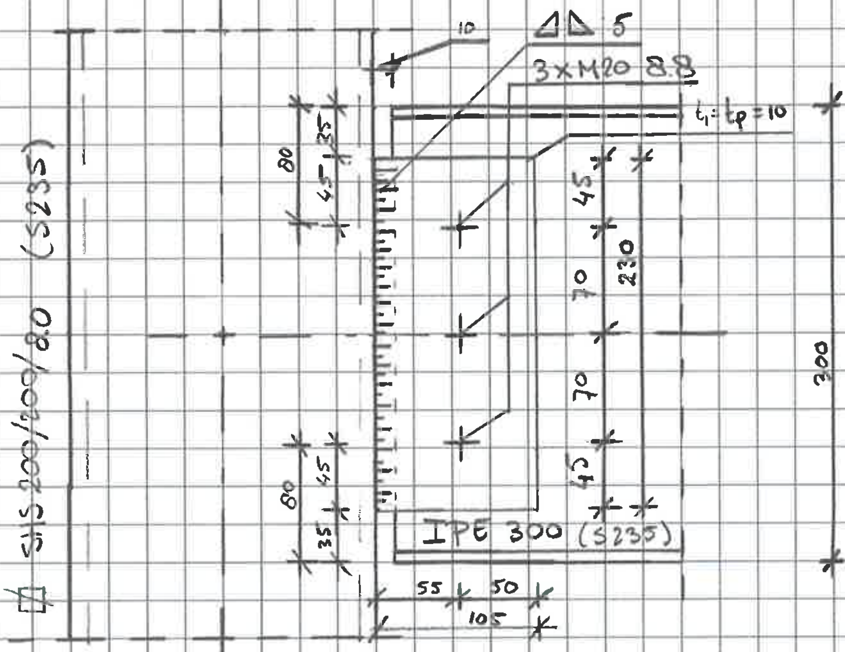
a = 4mm

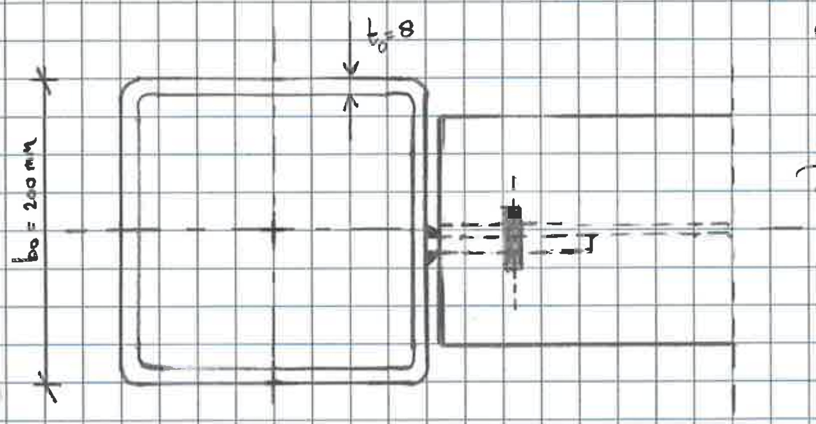
Revision:

Author:

CA1-V

CA1-V





Properties

Column SHS 200/200/8 (S235)

b_0 = 200 mm f_y = 235 N/mm²

t_0 = 8 mm f_u = 360 N/mm²

Beam IPE 300 (S235)

h = 300 mm

b = 150 mm

t_w = 7,1 mm

t_f = 10,7 mm

h_1 = 248 mm

A = 5380 mm²

Bolts : M20 8.8

A_s = 245 mm²

f_{ub} = 800 N/mm²

f_{yb} = 640 N/mm²

Welds Double Fillet.

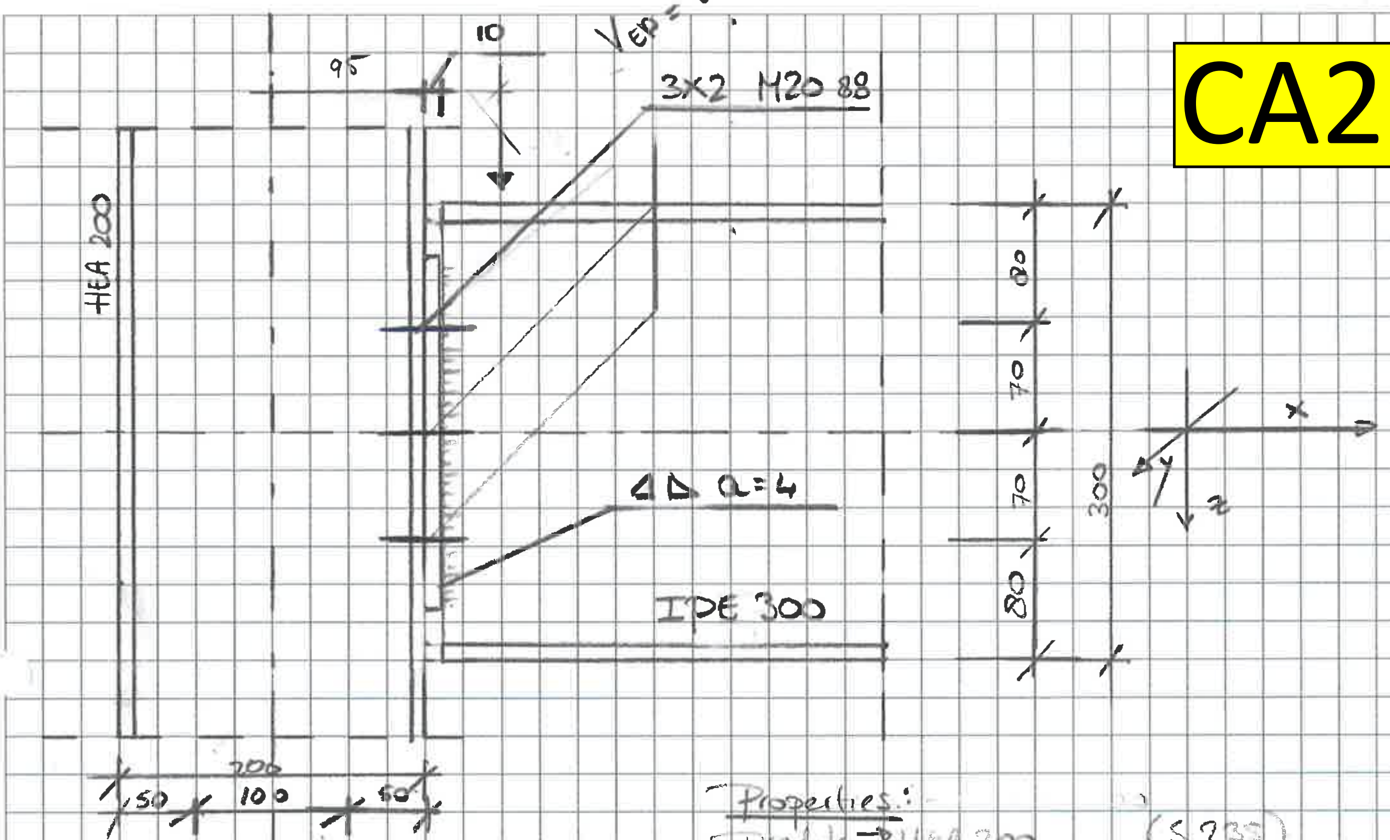
a = 5 mm

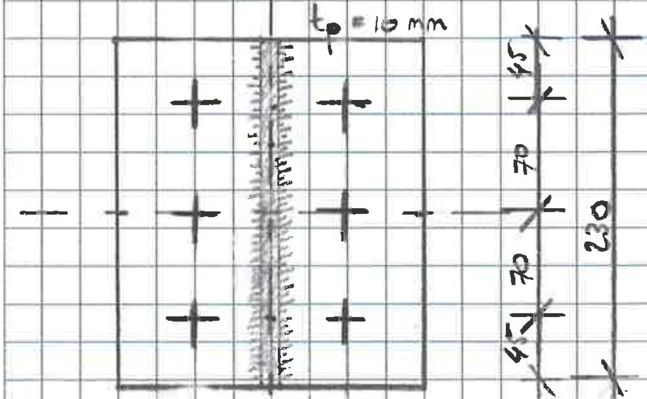
Safety Factors.

γ_{M1} = 1,0

γ_{M2} = 1,25

CA2





Properties:

Profile → HEA 200 (S 235)

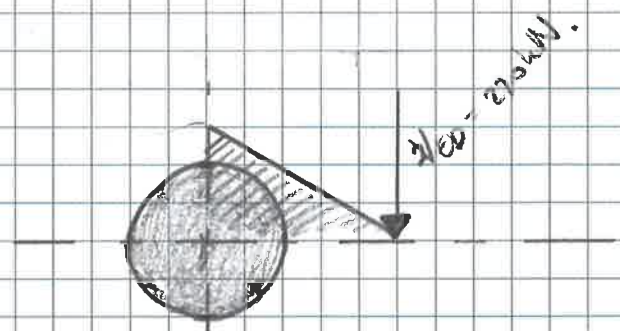
h = 190 mm
b = 200 mm
t_w = 6,5 mm
t_f = 10 mm
r = 18 mm
h_1 = 134 mm
A = 5380 mm²

Profile → IPE 300 (S235)

h = 300 mm
b = 150 mm
t_w = 7,1 mm
t_f = 10,7 mm
r = 15 mm
h_1 = 248 mm
A = 5380 mm²

Bolts → M20 8.8

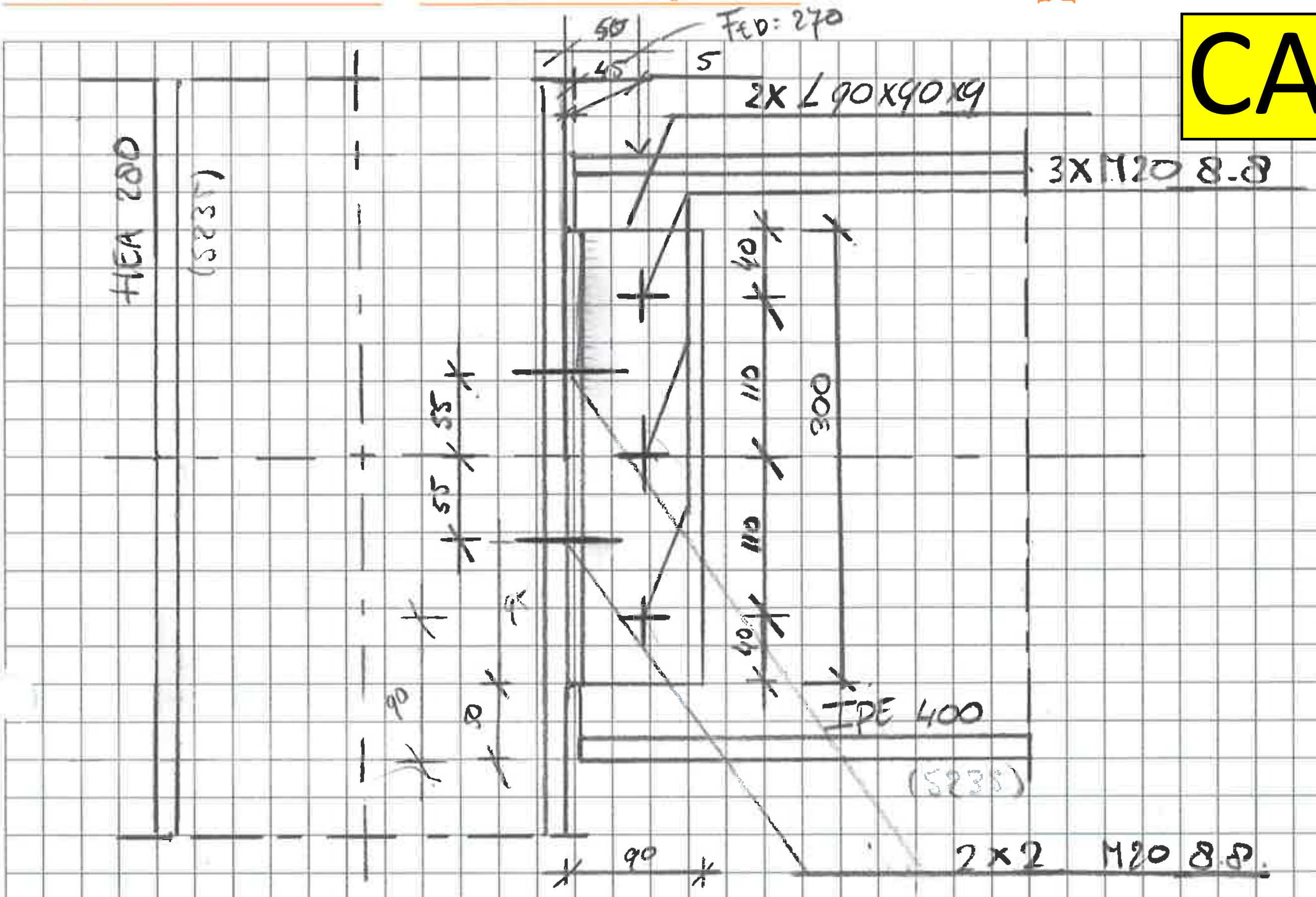
f_{ub} = 800 N/mm²
A_s = 245 mm²
d_0 = 22 mm

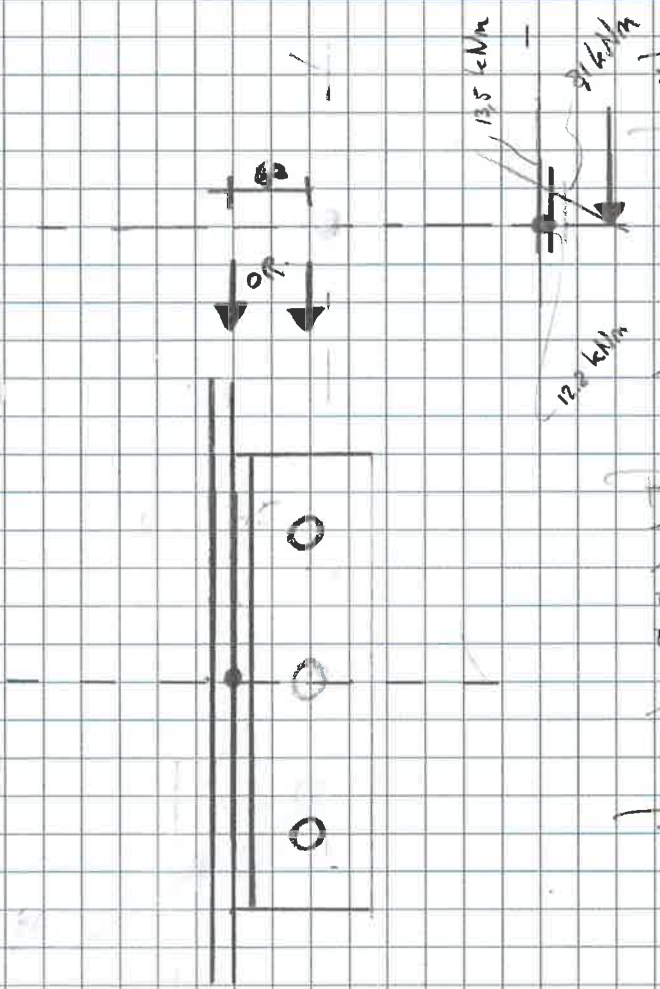


HOLLANDIA STRUCTURES

ca3

CA3





Properties

Profile IPE 400, steelquality S235

h = 400 mm

b = 180 mm

t_w = 8,6 mm

t_f = 13,5 mm

r = 21 mm

h_1 = 331 mm

A = 8450 mm²

Profile HEA 280, steelquality S235

h = 270 mm

b = 280 mm

t_w = 8,5 mm

t_f = 13 mm

r = 24 mm

h_1 = 196 mm

A = 9730 mm²

Profile L 90x90x9, steel quality S235

h = 90 mm r = 11 mm

b = 90 mm

t = 9 mm

Bolts → M20, quality 8.8

A_s = 245 mm²

f_{ub} = 800 N/mm²

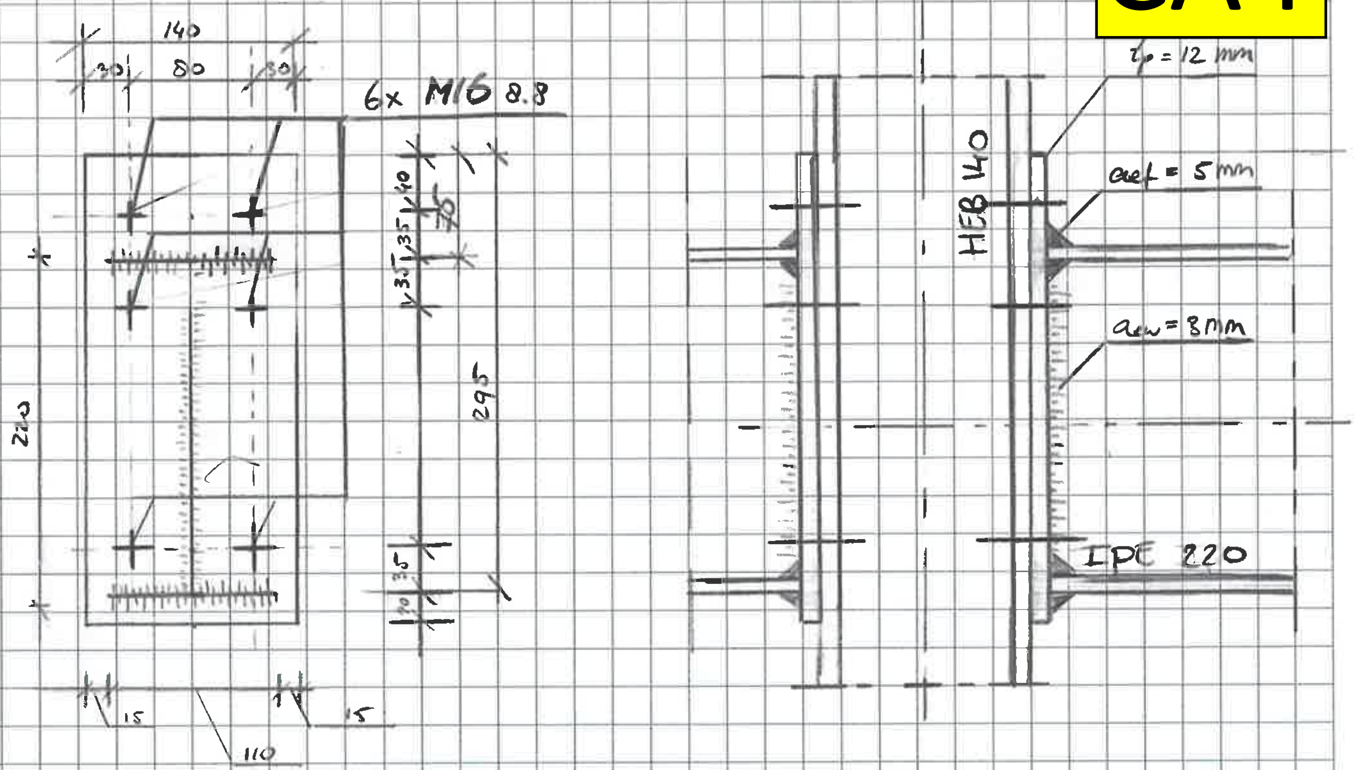
Project :
Project no.:

Doc. no.:
Date :
Page :

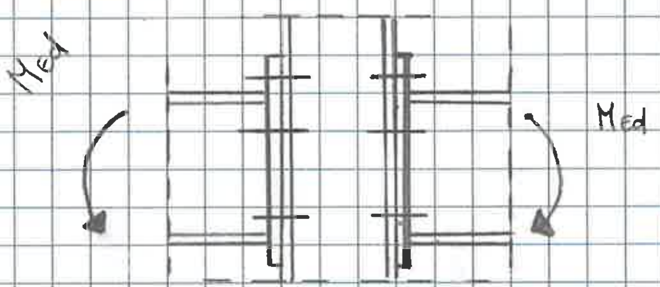
HOLLANDIA STRUCTURES

ca4

CA4



Load Situation



Geometrical Properties

Profile column S235, HEB 140

$h = 140$ mm

$b = 140$ mm

$t_w = 7$ mm

$t_f = 12$ mm

$r = 12$ mm

$A = 4300$ mm²

Profile Beam, IPE 220 (S235)

$h = 220$ mm

$b = 110$ mm

$t_w = 5{,}9$ mm

$t_f = 9{,}2$ mm

$r = 12$ mm

$A = 3340$ mm

Safety factors NEN-EN 1993-1-8 art. 6.1 (1)

$\gamma_{M0} = 1{,}0$ → resistance cross-sections

$\gamma_{M1} = 1{,}0$ → resistance stability (Buckling)

$\gamma_{M2} = 1{,}25$ → resistance tension till rupture?

Bolts 6x M16 8.8

$f_{ub} = 800$ N/mm²

$A_s = 157$ mm²

$d_0 = 18$ mm

Welds

$a_{flange} = 5$ mm

$a_{web} = 3$ mm

Revision:

Author:

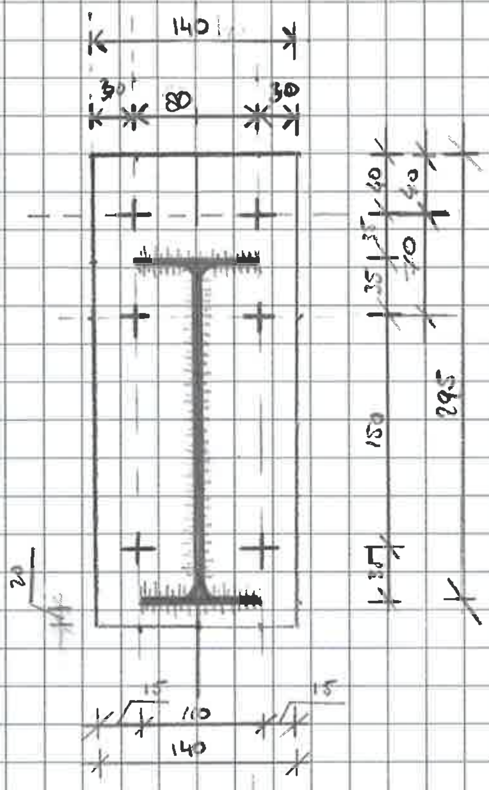
Project : Project no.:

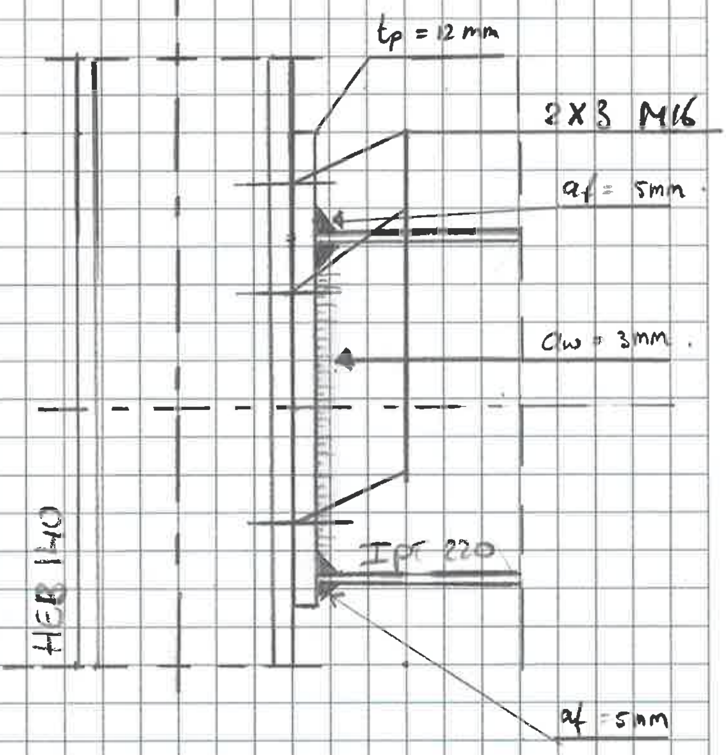
Doc. no.: Date : Page :

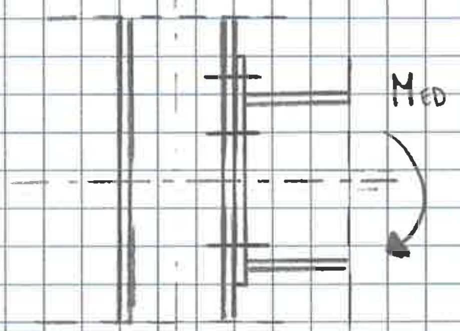
HOLLANDIA STRUCTURES

CA5

ca5







Geometrical Properties :

Column HEB140 S235

h = 140 mm

b = 140 mm

t_w = 7 mm

t_f = 12 mm

r = 12 mm

A = 4300 mm²

Safety factors NEN-EN 1993-1-8 art 6.1(1)

γ_{M0} = 1,0 → resistance cross-section

γ_{M1} = 1,0 → resistance stability buckling

γ_{M2} = 1,25 → resistance tension full rupture.

Beam IPE 220 S235

h = 220 mm

b = 110 mm

t_w = 5,9 mm

t_f = 9,2 mm

r = 12 mm

A = 3340 mm²

Welds

flange a_f = 5 mm.

web a_w = 3 mm

Bolts 2x3 M16 8.8

f_{ub} = 800 N/mm²

A_s = 157 mm²

d = 16 mm

d_0 = 18 mm.

Revision:

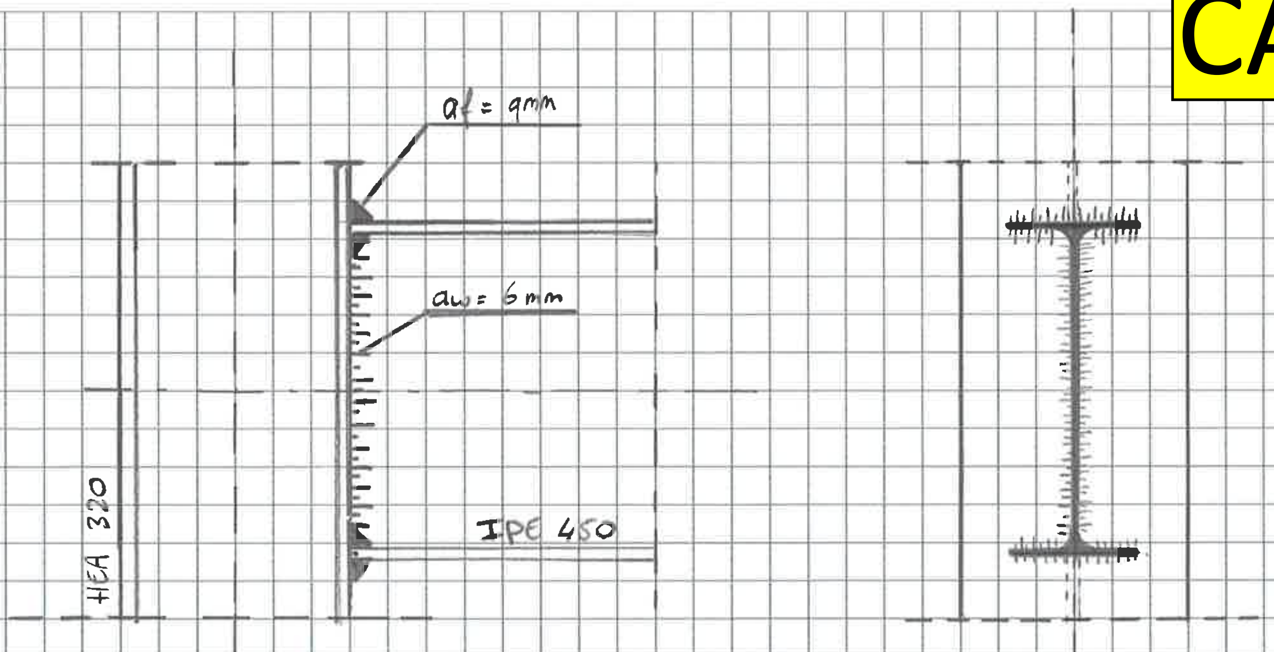
Author:

Project :
Project no.:
Doc. no.:
Date :
Page :

HOLLANDIA STRUCTURES

ca6.

CA6



Geometrical Properties

Column HEA 320 steelquality S355

h = 310 mm
b = 300 mm
t_w = 9 mm
t_f = 15,5 mm
r = 27 mm
A = 12 440 mm^2

f_y = 355 N/mm^2
f_u = 510 N/mm^2

Beam IPE 450 steelquality S355

h = 450 mm
b = 190 mm
t_w = 9,4 mm
t_f = 14,6 mm
r = 21 mm
A = 9880 mm^2

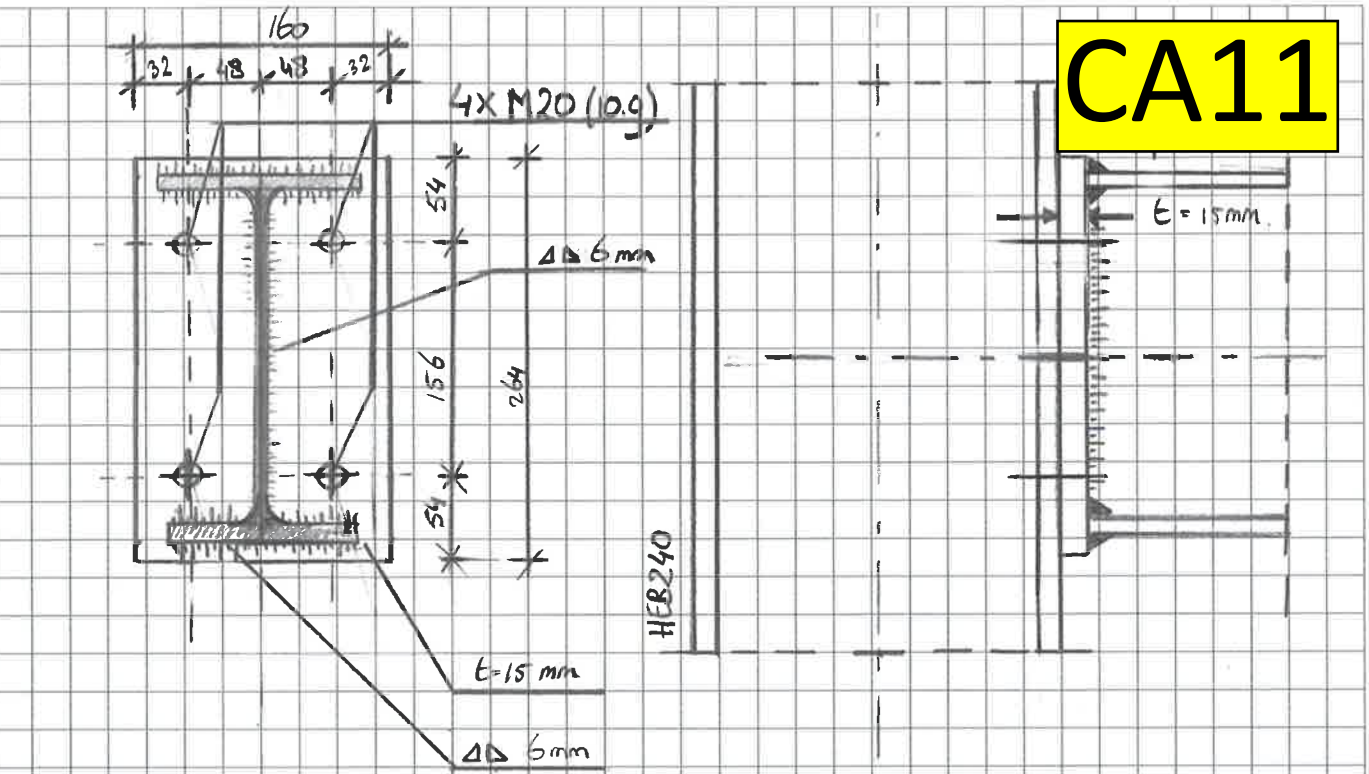
f_y = 355 N/mm^2
f_u = 510 N/mm^2

Welds

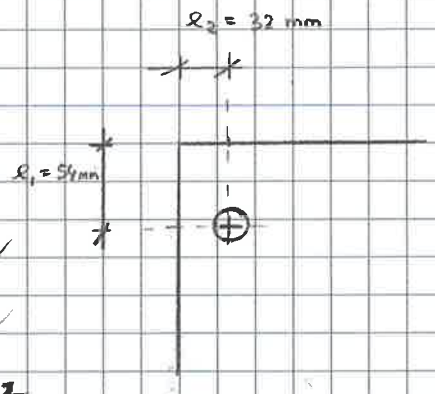
a_{flange} = 9 mm
a_{web} = 6 mm

Revision:
Author:

CA11



Geometrical Properties



Profile IPE240, S275

h = 240 mm
b = 120 mm
t_w = 6,2 mm
t_f = 9,8 mm
r = 15 mm
A = 3910 mm²
W_{pl} = 367 · 10³ mm³

f_y = ~~351,5~~ N/mm²
350 N/mm² ✓
f_u = ~~451,3~~ N/mm²
450 N/mm² ✓
E = 209 468 N/mm²
2,1 · 10⁵ N/mm²

Profile HEB240, S275

h = 240 mm
b = 240 mm
t_w = 10 mm
t_f = 17 mm
r = 21 mm
A = 10600 mm²

f_y = ~~357,5~~ N/mm²
360 N/mm²
f_u = ~~463,5~~ N/mm²
460 N/mm²
E = ~~213 864~~ N/mm²
2,1 · 10⁵ N/mm²

Bolts M20, 10.9

f_{ub} = 1000 N/mm²
d = 20 mm
d_0 = 22 mm
k = 0,9
A_s = 245 mm²

End Plate, S275

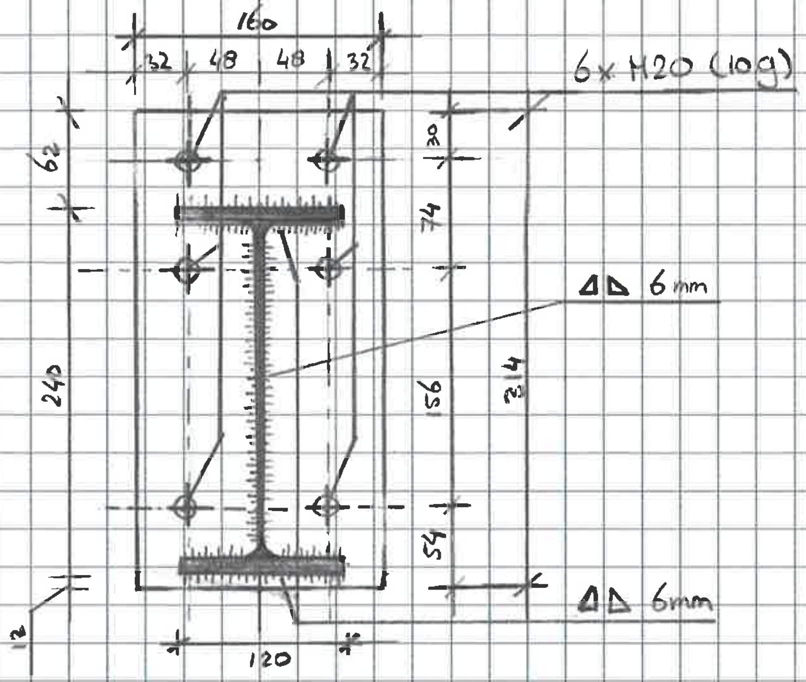
t_p = 15 mm

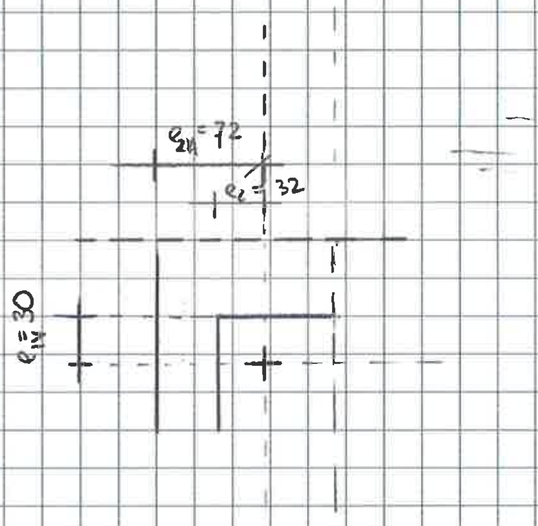
f_y = 360,4 N/mm²
370 N/mm²
f_u = 503,5 N/mm²
500 N/mm²
E = 200 248 N/mm²
2,0 · 10⁵ N/mm²

Welds

a = 6 mm

CA22





Geometrical properties

Profile IPE 240

h = 240 mm
b = 120 mm
t_w = 6,2 mm
t_f = 9,8 mm
r = 15 mm
A = 3910 mm²
W_{pl} = 367 · 10³ mm³

f_y = 351,5 N/mm²
350 N/mm²
f_u = 451,2 N/mm²
450 N/mm²
E = 209468 N/mm²
210000 N/mm²

Profile HEB 240

h = 240 mm
b = 240 mm
t_w = 10 mm
t_f = 17 mm
r = 21 mm
A = 10600 mm²

f_y = 357,5 N/mm²
360 N/mm²
f_u = 463,0 N/mm²
460 N/mm²
E = 213864 N/mm²
210000 N/mm²

End plate

t_p = 15 mm

γ_{M0} = 1,0
γ_{M1} = 1,0
γ_{M2} = 1,0

f_y = ~~369,4 N/mm²~~
370 N/mm²
f_u = 503,45 N/mm²
= 500 N/mm²
E = ~~200248~~ N/mm²
= 200000 N/mm²

Bolts M20, 10g

f_{ub} = 1000 N/mm²
d = 20 mm
d_0 = 22 mm
K_2 = 0,9
A_s = 245 mm²

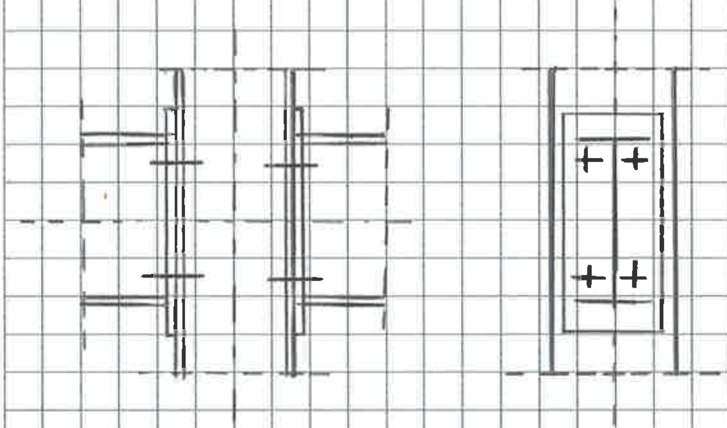
$F_{t,d}$ = 220,5 kN

Welds

a = 6 mm

PS3

PS3



Column : HEB 160

h = 160 mm
b = 160 mm
t_w = 8 mm
t_f = 13 mm
r = 15 mm
A = 5430 mm²

f_y = 235 N/mm²
f_u = 360 N/mm²

e = 40 mm

End plate

t_p = 10 mm

w = 80 mm

f_y = 235 N/mm²
f_u = 360 N/mm²

Beam : IPE 220

h = 220 mm
b = 110 mm
t_w = 5,9 mm
t_f = 9,2 mm
r = 12 mm
A = 3340 mm²

f_y = 235 N/mm²
f_u = 360 N/mm²

Bolts

2 x M12 (8.8)
d_0 = 14 mm
A_s = 84 mm²

$F_{v,Rd}$ = 32.37 kN
$F_{t,Rd}$ = 48.56 kN

Welds

a = 5

Symmetrical Loadcase

$\beta = 0 \longrightarrow \omega = 1$

Safety Factors

γ_{M0} = 1,0
γ_{M1} = 1,0
γ_{M2} = 1,25

Appendix B

MATLAB Scripts

The following scripts are made for the calculation of the following connection:

- CA1: SSJ, Fin Plate
- CA2: SSJ, Short End Plate
- CA3: SSJ, Double Angle Cleats
- CA4: MRJ, Extended Endplate Joint symmetrical loading
- CA5: MRJ, Extended Endplate Joint unsymmetrical loading
- CA6: MRJ, Welded Joint
- CA11-EXP: MRJ, Flush End Plate Joint
- CA11-EC3: MRJ, Flush End Plate Joint
- CA12-EXP: MRJ, Extended End Plate Joint
- CA12-EC3: MRJ, Extended End Plate Joint

- PS1: Fin Plate Joint
- PS2: Short End Plate Joint
- PS3a: Moment Resisting Joint, Flush End Plate Joint
- PS3b: Moment Resisting Joint, Extended End Plate Joint

The script files can be found in this appendix (PDF). MATLAB script files (.m) can be found on the USB-Flashdrive enclosed to this thesis.

B.1 CA1: SSJ, Fin Plate

```
%% Clear Everything
clear all
clc
close all

pause(0.5)

%%Calculation Shear Connection

%%Assumptions:
    % - Single rowed bolt patern
    % - Only a shear force is applied
    % - Plastic calculation
    % - Full strength welds


%%Results IDEA connection
IDEA_resistance_kN = 230;   %Resistance of FEA software [kN]


%%Parameters

    %Load parameters
    V_Ed_kN = 140;          %Shear Force in [kN]
    e_exc2_mm = 55;         %Excentricity till flange column [mm]

    %Geometry parameters
    %Profile beam - > IPE 300, S235
    h_b_mm = 300;       %Height of the beam [mm]
    b_b_mm = 150;       %Width of the beam [mm]
    tw_b_mm = 7.1;      %Thickness of the beam web [mm]
    tf_b_mm = 10.7;     %Thickness of the beam flange [mm]
    r_b_mm = 15;        %Radius of the beam [mm]
    A_b_mm2 = 5380;     %Area of the beam [mm]

    e11_b_mm = 80;      %Edge distance parallel to the force of the beam [mm]
    e22_b_mm = 45;      %Edge distance perpendicular to the force of the beam [mm]

    p11_b_mm = 70;      %Pitch between the bolts [mm]

    %Profile steel plate, S235
    h_p_mm = 230;       %Height of the fin plate [mm]
    t_p_mm = 8;         %Width of the fin plate [mm]
    l_p_mm = 105;       %Length of the fin plate [mm]

    e1_p_mm = 45;       %Edge distance parallel to the force [mm]
    e2_p_mm = 50;       %Edge distance perpendicular the the force [mm]

    p1_p_mm = p11_b_mm; %Pitch between the polts in the fin plate [mm]

    %Material Parameters: (1) Beam, (2) Fin Plate
    fy_b_Ndmm2 = 235;   %Yield strength beam [N/mm2]
    fu_b_Ndmm2 = 360;   %Tensile strength beam [N/mm2]
```



```
    fy_p_Ndmm2 = 235;   %Yield strength fin plate [N/mm2]
    fu_p_Ndmm2 = 360;   %Tensile strength fin plate [N/mm2]

    %Bolts -> type M20, quality 8.8
    d_mm = 20;          %Bolt diameter [mm]
    d0_mm = 22;         %Bolt hole diameter [mm]
    As_bo_mm2 = 245;    %Cross-sectional area (threaded portion) of the bolt [mm2]
    n2_bo = 3;          %Bolt (rows) in the beam [-]
    fub_bo_Ndmm2 = 800; %Tensile strenth of the bolts [N/mm2]
    alpha_v = 0.6;      %Alpha factor bolts [-]

    %Safety Factors, NEN-EN 1993-1-1, art. 6.1(1)
    gamma_M0 = 1.00;    %Factor resistance of cross-sections [-]
    gamma_M2 = 1.25;    %Factor resistance in tension till rupture [-]


%% Calcualtion

    %Derived Parameters
    e_exc2_m = e_exc2_mm / 1000;
    M_Ed_exc2_kNm = V_Ed_kN .* e_exc2_m;
    Izz_p_mm4 = (1 ./ 12) .* t_p_mm .* h_p_mm.^3;


%%A. Shear resistance of the beam web, NEN-EN 1993-1-1, art. 6.2.6
    Av_b_mm2 = A_b_mm2 - (2 .* b_b_mm .* tf_b_mm) + (tw_b_mm + 2 .* r_b_mm) .
tf_b_mm; %NEN-EN 1993-1-8, art. 6.2.6(3a)
    Avnet_b_mm2 = Av_b_mm2 - n2_bo .* (d0_mm .* tw_b_mm);

    VplRd_b_N = (Avnet_b_mm2 .* (fy_b_Ndmm2 ./ sqrt(3))) ./ gamma_M0;
    VplRd_b_kN = VplRd_b_N ./ 1000;

    %Unity Check A
    UC_A = V_Ed_kN ./ VplRd_b_kN;


%%B. Tearing resistance of the beam web, NEN-EN 1993-1-8, art. 3.10.2
    Anv_b_mm2 = (h_b_mm - 2.5 .* d0_mm - e11_b_mm) .* tw_b_mm;
    Ant_b_mm2 = (e22_b_mm - 0.5 .* d0_mm) .* tw_b_mm;

    Veff2Rd_b_N = ((0.5 .* fu_b_Ndmm2 .* Ant_b_mm2) ./ gamma_M2 ) +  ( Anv_b_mm2 .
(fy_b_Ndmm2 ./ sqrt(3)) ./ gamma_M0 ); %NEN-EN 1993-1-8, art. 3.10.(3) eq(3.10)
    Veff2Rd_b_kN = Veff2Rd_b_N ./ 1000;

    %Unity Check B
    UC_B = V_Ed_kN ./ Veff2Rd_b_kN;


%%C. Shear resistance of the bolts, NEN-EN 1993-1-8, art. 3.6.1, Table 3.4
    FvRd_N = (alpha_v .* fub_bo_Ndmm2 .* As_bo_mm2) ./ gamma_M2;
    FvRd_kN = FvRd_N ./ 1000;
    FvRd_tot_kN = n2_bo .* FvRd_kN;
```

```
    %Unity Check C
    UC_C = V_Ed_kN ./ FvRd_tot_kN;


%%D. Bearing resistance of the fin plates

    %Bearing of the inside bolts, NEN-EN 1993-1-8, art. 3.6.1, Table 3.4
    k1_inside_p_one = 2.8 .* (e2_p_mm ./ d0_mm) - 1.7;
    k1_inside_p_two = 2.5;

    k1_inside_p = min([k1_inside_p_one k1_inside_p_two]);

    alphab_inside_p_one = p1_p_mm ./ (3 .* d0_mm) - 0.25;
    alphab_inside_p_two = fub_bo_Ndmm2 ./ fu_p_Ndmm2;
    alphab_inside_p_three = 1.0;

    alphab_inside_p = min([alphab_inside_p_one alphab_inside_p_two
alphab_inside_p_three]);

    FbRd_inside_p_N = (k1_inside_p .* alphab_inside_p .* fu_p_Ndmm2 .* d_mm .*
t_p_mm) ./ gamma_M2;
    FbRd_inside_p_kN = FbRd_inside_p_N ./ 1000;

    %Bearing of the edge bolt, NEN-EN 1993-1-8, art. 3.6.1, Table 3.4
    k1_edge_p_one = 2.8 .* (e2_p_mm ./ d0_mm) - 1.7;
    k1_edge_p_two = 2.5;

    k1_edge_p = min([k1_edge_p_one k1_edge_p_two]);

    alphad_edge_p_one = e1_p_mm ./ (3 .* d0_mm);
    alphad_edge_p_two = fub_bo_Ndmm2 ./ fu_p_Ndmm2;
    alphad_edge_p_three = 1.0;

    alphab_edge_p = min([alphad_edge_p_one alphad_edge_p_two alphad_edge_p_three]);

    FbRd_edge_p_N = (k1_edge_p .* alphab_edge_p .* fu_p_Ndmm2 .* d_mm .* t_p_mm) ./
gamma_M2;
    FbRd_edge_p_kN = FbRd_edge_p_N ./ 1000;

    FbRd_tot_p_kN = ((n2_bo - 1) .* FbRd_inside_p_kN) + FbRd_edge_p_kN;

        %Unity Check D
        UC_D = V_Ed_kN ./ FbRd_tot_p_kN;


%%E Bearing of the web of the beam

    %Bearing of the inside bolts, NEN-EN 1993-1-8, art. 3.6.1, Table 3.4
    k1_inside_b_one = 2.8 .* (e22_b_mm ./ d0_mm) - 1.7;
    k1_inside_b_two = 2.5;

    k1_inside_b = min( [k1_inside_b_one k1_inside_b_two]);
```



```
    alphad_inside_b_one = p11_b_mm ./ (3 .* d0_mm) - 0.25;
    alphad_inside_b_two = fub_bo_Ndmm2 ./ fu_b_Ndmm2;
    alphad_inside_b_three = 1.0;

    alphab_inside_b = min([alphad_inside_b_one alphad_inside_b_two
alphad_inside_b_three]);

    FbRd_inside_b_N = (k1_inside_b .* alphab_inside_b .* fu_b_Ndmm2 .* d_mm .*
tw_b_mm) ./ gamma_M2;
    FbRd_inside_b_kN = FbRd_inside_b_N ./ 1000;

    %Bearing of the edge bolts, NEN-EN 1993-1-8, art. 3.6.1, Table 3.4
    k1_edge_b_one = 2.8 .* (e22_b_mm ./ d0_mm) - 1.7;

    k1_edge_b_two = 2.5;

    k1_edge_b = min([k1_edge_b_one k1_edge_b_two]);

    alphad_edge_b_one = e11_b_mm ./ (3 .* d0_mm);
    alphad_edge_b_two = fub_bo_Ndmm2 ./ fu_b_Ndmm2;
    alphad_edge_b_three = 1.0;

    alphad_edge_b = min([alphad_edge_b_one alphad_edge_b_two alphad_edge_b_three]);

    FbRd_edge_b_N = (k1_edge_b .* alphad_edge_b .* fu_b_Ndmm2 .* d_mm .* tw_b_mm)
/ gamma_M2; %NEN-EN 1993-1-8, art. 3.6.1, Table 3.4
    FbRd_edge_b_kN = FbRd_edge_b_N ./ 1000;

    FbRd_b_tot_kN = ((n2_bo - 1) .* FbRd_inside_b_kN + FbRd_edge_b_kN);

    %Unity Check E
    UC_E = V_Ed_kN ./ FbRd_b_tot_kN;


%%F Shear resistance of the fin plate, NEN-EN 1993-1-1, art. 6.2.6
Avnet_p_mm = h_p_mm .* t_p_mm - (n2_bo * d0_mm * t_p_mm);

%NEN-EN 1993-1-1, art. 6.2.6(2) eq(6.18)
VplRd_p_N = (Avnet_p_mm .* (fy_p_Ndmm2 ./ sqrt(3))) ./ gamma_M0;
VplRd_p_kN = VplRd_p_N ./ 1000;

UC_F = V_Ed_kN ./ VplRd_p_kN;

%%G. Tearing resistance of the fin plate, NEN-EN 1993-1-8, art. 3.10.2
Anv_p_mm2 = t_p_mm .* (h_p_mm - (n2_bo - 0.5) .* d0_mm - e1_p_mm);
Ant_p_mm2 = t_p_mm .* (e2_p_mm - 0.5 .* d0_mm);

Veff2Rd_p_N = (0.5 .* fu_p_Ndmm2 .* Ant_p_mm2) ./ gamma_M2 + (Anv_p_mm2 .*
(fy_p_Ndmm2 ./ sqrt(3)) ./ gamma_M0);
Veff2Rd_p_kN = Veff2Rd_p_N ./ 1000;

    %Unity Check G
```

```
    UC_G = V_Ed_kN ./ (Veff2Rd_p_kN);


%% Moment resistance of the plate
    WplRd_pcw_mm3 = (1 ./ 4) .* t_p_mm .* h_p_mm.^2;%Cross section where the plate
connects the weld (pwc)
    MplRd_pcw_Nmm = WplRd_pcw_mm3 .* fy_b_Ndmm2;

    Av_pcw = t_p_mm .* h_p_mm;
    VplRd_pcw_N = Av_pcw .* ((fy_p_Ndmm2) ./ sqrt(3)) ./ gamma_M0;%NEN-EN 1993-1-
1, art. 6.2.6(2) eq(6.18)
    VplRd_pcw_kN = VplRd_pcw_N ./ 1000;

    %NEN-EN 1993-1-1, art. 6.2.8(3), eq(6.29)
    if V_Ed_kN < (0.5 .* VplRd_pcw_kN);
        rho_if = 0;
    elseif V_Ed_kN < VplRd_pcw_kN ;
        rho_if = (((2 .* V_Ed_kN) ./ VplRd_pcw_kN)-1).^2;
    else
        rho_if = 1;
    end

    fy_p_red_Ndmm2 = (1 - rho_if).* fy_p_Ndmm2;

    MplRd_pcw_red_Nmm = MplRd_pcw_Nmm .* (fy_p_red_Ndmm2 ./ fy_p_Ndmm2);
    MplRd_pcw_red_kNm =  MplRd_pcw_red_Nmm .* 1E-6;

        %Unity Check H
        UC_H = M_Ed_exc2_kNm ./ MplRd_pcw_red_kNm;

%IDEA Resistance

        %Unity Check IDEA
        UC_IDEA =  V_Ed_kN ./ IDEA_resistance_kN;

%Matrix with the Unity Check Values and Resistance Values
UC_values_EC = [ 0; UC_A; UC_B; UC_C; UC_D; UC_E; UC_F; UC_G; UC_H; 0]
RV_values_EC    = round(   [ VplRd_b_kN;   Veff2Rd_b_kN;   FvRd_tot_kN
FbRd_tot_p_kN;      FbRd_b_tot_kN;      VplRd_p_kN;     Veff2Rd_p_kN;
] ,1)
RV_values_FEA   =           [ 0;            0;              0;              0
0;              0;              0;              IDEA_resistance_kN]
UC_fin = [  VplRd_p_kN  Veff2Rd_p_kN    IDEA_resistance_kN]
UC_IDEA

%Table
UC_values_table=round([UC_values_EC(2:9)],2)
RVT = RV_values_EC

uT = uitable;
uT.ColumnName = {'Failure mode' 'Resistance Value [kN]' ' 'Unity Check [-] '}
uT.Units='Normalized';
uT.Position = [ .1 .1 .8 .8 ];
```



```
data = {    'A. Shear beam web',        RVT(1)          ,UC_values_table(1)
            'B. Tearing beam web',      RVT(2)          ,UC_values_table(2)
            'C. Shear bolts',           RVT(3)          ,UC_values_table(3)
            'D. Bearing fin plates',    RVT(4)          ,UC_values_table(4)
            'E. Bearing of the web',    RVT(5)          ,UC_values_table(5)
            'F. Shear fin plate',       RVT(6)          ,UC_values_table(6)
            'G. Tearing fin plate',     RVT(7)          ,UC_values_table(7)
            'X. IDEA Connection',       IDEA_resistance_kN    ,UC_IDEA}
uT.Data = data;

%Bar Chart Unity Checks Eurocode

%     %Prepare unity check lines
%     UC_length = length(UC_values_EC);
%     UC_value_x = [1:1:UC_length];
%     UC_value_y = ones(1, UC_length);
%
%     %Prepare resistance line IDEA
%     IDEA_length = length(UC_values_EC);
%     IDEA_value_x = [1:1:IDEA_length];
%     IDEA_value_y = UC_IDEA .* ones(1, IDEA_length);
%
%
%     %Make Figure
%     figure;
%     bar(UC_values_EC);
%     hold on
%     plot(UC_value_x, UC_value_y,'k', 'Linewidth', 1.5)
%     hold on
%     plot(IDEA_value_x, IDEA_value_y, 'r', 'Linewidth', 1.5)
%
%
%
%     %Axis and lines
%     axis([0 11 0 1.2]);
%     xlabel('Unity Check [-]');
%     ylabel('Unity Check Value');
%     yticks([0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 1.2]);
%     grid on
%
%     %Text in figure
%     loadcase1 = V_Ed_kN;
%     title_case1 = sprintf('Unity checks for a single rowed shear connection, fo
a load V_{Ed} = %d kN', loadcase1);
%     title(title_case1);
%     set(gca, 'XTickLabel',{'','UC-A', 'UC-B', 'UC-C', 'UC-D','UC-E', 'UC-F', 'UC
G','UC-H', '', ''});
%
%     %Legend
%     legend('UC Eurocode','Unity Check Line','UC IDEA Connection');

%Bar Chart, Resistance Values Fin Plate
figure
```

```
UC_fin_EC =     [  VplRd_p_kN   Veff2Rd_p_kN        0]
UC_fin_IDEA =   [  0            0                   IDEA_resistance_kN]
bar(UC_fin_EC)
hold on
bar(UC_fin_IDEA,'r')
title('Resistance Values of the Fin Plate')
xlabel('Failure mode');
ylabel('Resistance value [kN]');
grid on
set(gca, 'XTickLabel', {'Tearing Resistance', 'Shear Resistance', 'Strain Limit
5%'})

%Bar Chart Resistance Values

figure
bar(RV_values_EC)
hold on
bar(RV_values_FEA, 'r')
grid on
loadcase2 = V_Ed_kN;
title_case2 = sprintf('Resistance Values of a Single Rowed Shear Connection')
title(title_case2)
xlabel('Component Resistances and FEA Model Resistance');
ylabel('Resistance Value [kN]');
set(gca, 'XTickLabel', {'CR1', 'CR2', 'CR3', 'CR4', 'CR5', 'CR6', 'CR7', 'TMR'})
```

B.2 CA2: SSJ, Short End Plate

B.3 CA3: SSJ, Double Angle Cleats

```
%Clear Everything
clear all
clc
close all

pause(0.5)

%%Calculation Connection_two: Shear Connection Short End Plate


%%Assumptions
    % - Full strength welds
    % - Bolts resistances for bearing are manually calculated
    % - Bolts in bearing are calculated with the elleptical method
    % - IDEA calculate the excentricity from end of the beam


%%Results IDEA connection
IDEA_resistance_model_kN =   [320]      %Bolt failure, combination tension and
shear [kN]
IDEA_resistance_bbolt_kN = [123.8]     %Bearing of the bolt failure [kN]

%%Parameters

    %Loads
    V_Ed_kN = 340;      %Load applied on connection [kN]
    e_exc1_m = 0.050;   %Excentricity of the load from the flange of the beam x
direction [mm]
    e_exc2_m = 0.0543;  %Excentricity of the load from the centre-line of the
flange of the beam in y-direction [mm]

    %Profile Beam -> IPE400, steelquality S235
    h_b_mm = 400;       %Height of the beam [mm]
    b_b_mm = 180;       %Width of the beam [mm]
    tw_b_mm = 8.6;      %Width of the beam web [mm]
    tf_b_mm = 13.5;     %With of the beam flange [mm]
    r_b_mm = 21;        %Radius of the beam [mm]
    A_b_mm2 = 8450;     %Cross sectional area of the beam [mm2]

    p1_b_mm = 110;      %Pitch, distance between the bolts in the beam parallel to
the force direction [mm]
    e111_b_mm = 90;     %Edge distance parallel to the force direction [mm]
    e222_b_mm = 45;     %Edge distance perpedicular to the force direction [mm]

    fy_b_Ndmm2 = 235;   %Yield strength of the beam [N/mm2]
    fu_b_Ndmm2 = 360;   %Tensile strength of the beam [N/mm2]


    %Profile cleats L90x90x9, steelquality S235
    hp_cl_mm = 300;     %Heigth of the cle at profile [mm]
    h_cl_mm = 90;       %Leg length (1) of the cleat [mm]
    b_cl_mm = 90;       %Leg length (2) of the cleat [mm]
    t_cl_mm = 9;        %Thickness of thet cleat [mm]
```



```
    r_cl_mm = 11;       %Radius of the cleat [mm]

    e1_cl_mm = 40;      %Edge distance (1) of the cleat, located at the beam [mm]
    e2_cl_mm = 40;      %Edge distance (2) of the cleat, located at the beam [mm]

    e11_cl_mm = 95;     %Edge distance (3) of the cleat, located at the column [mm]
    e22_cl_mm = 40;     %Edge distance (4) of the cleat, located at the column [mm]

    fy_cl_Ndmm2 = 235;  %Yield strength of the cleat [N/mm2]
    fu_cl_Ndmm2 = 360;  %Tensile strength of the cleat [N/mm2]


    %Bolts -> M20, quality 8.8
    As_bo_mm2 = 245;    %Cross sectional area of the bolt [mm2]
    fu_bo_Ndmm2 = 800;  %Tensile strength of the bolt [N/mm2]
    d_mm = 20;          %Width of the bolt [mm]
    d0_mm = 22;         %Bolt spacing [mm]
    alpha_v = 0.6;      %Factor to determine the shear resistance of the bolt
(related to steel quality) [-]

    n2b = 3;            %Rows of bolts in the beam [-]

    n1cc = 2;           %Columns of bolts in the column flange [-]
    n2cc = 2;           %Rows of bolts in the column flange [-]

    r1_c_m = 0.090;     %Distance between first the bolt row to the rotational
center in (y,z)-plane [mm]
    r2_c_m = 0.200;     %Distance between the second bolt row to rotational center
in (y,z)-plane [mm]

    %Bolt factors for bearing resistance for beam - cleat connection
    %note:  No difference between inside en end bolts same factors

        %Bolt factors for bearing resistance in vertical direction
        k1_vert_bo123 = 2.5;        %k1 factor, vertical
        alpha_b_vert_bo123 = 1;     %alpha b factor, vertical

        %Bolt factor for bearing resistance in horizontal direction for
        %bolt 1
        k1_hor_bo1 = 2.5;           %k1 factor, horizontal bolt 1
        alpha_b_hor_bo1 = 0.67;     %alpha b factor, horizontal bolt 1

        %Bolt factor for bearing resistance in horizontal direction for
        %bolt 2 and bolt 3
        k1_hor_bo23 = 2.5;          %k1 factor, horizontal bolt 2,3
        alpha_b_hor_bo23 = 1;       %alpha b factor, horizontal bolt 2,3

    %Bearing resistance of bolt partern in column - cleat connection

        %Bolt Patern Column - Cleat
        FbRd_hor_bo11_kN = 98.5;
        FbRd_vert_bo11_kN = 130;
```

```
        %Bolt Patern Column - Cleat
        FbRd_hor_bo22_kN = 98.5;
        FbRd_vert_bo22_kN = 130;


    %Safety Factors     NEN-EN 1993-1-1, art. 6.2
    gamma_M0 = 1.0      %Safety factor resistance cross-section
    gamma_M2 = 1.25     %Safety factor resistance cross-section till rupture


%%Derived parameters

    %Moment Arm, moment arm between bolts in beam (bolt 1 and bolt 3)
    a_moment_beam_mm = p1_b_mm .* (n2b - 1);
    a_moment_beam_m = a_moment_beam_mm ./ 1000;

    %Moments generated by excenticity
    M_Ed_1_kNm = V_Ed_kN .* e_exc1_m       %Moment from the beam flange (x,z)-
plane
    M_Ed_2_kNm = V_Ed_kN .* e_exc2_m / 2   %Moment is taken up by two bolt rows
(y,z)-plane


    %%Derived Loads,

    %Forces in bolts of the beam (bolt 1, bolt 2, bolt 3)
    F_Ed_vert_bo1_kN =   V_Ed_kN ./ n2b;
    F_Ed_vert_bo2_kN =   V_Ed_kN ./ n2b;
    F_Ed_vert_bo3_kN =   V_Ed_kN ./ n2b;

    F_Ed_hor_bo1_kN = M_Ed_1_kNm ./ a_moment_beam_m;
    F_Ed_hor_bo2_kN = 0;
    F_Ed_hor_bo3_kN = M_Ed_1_kNm ./ a_moment_beam_m;

    F_Ed_res_bo1_kN = sqrt(F_Ed_hor_bo1_kN.^2 + F_Ed_vert_bo1_kN.^2);
    F_Ed_res_bo2_kN = F_Ed_vert_bo2_kN;
    F_Ed_res_bo3_kN = sqrt(F_Ed_hor_bo3_kN.^2 + F_Ed_vert_bo3_kN.^2);

    %Bolts in Column (bolt 11, bolt 22, bolt 33 and bolt 44)
    F_Ed_vert_bo11_kN = V_Ed_kN ./ ( n1cc .* n2cc )
    F_Ed_vert_bo22_kN = V_Ed_kN ./ ( n1cc .* n2cc )

    F_Ed_hor_bo11_kN = M_Ed_2_kNm .* r1_c_m ./ (r1_c_m.^2 + r2_c_m .^2)
    F_Ed_hor_bo22_kN = M_Ed_2_kNm .* r2_c_m ./ (r1_c_m.^2 + r2_c_m.^2)

    F_Ed_res_bo11_kN = sqrt(F_Ed_vert_bo11_kN.^2 + F_Ed_hor_bo11_kN.^2)
    F_Ed_res_bo22_kN = sqrt(F_Ed_vert_bo22_kN.^2 + F_Ed_hor_bo22_kN.^2)


%Unity Checks

%0. Model resistance
```



```
    UC_FEA = V_Ed_kN ./ IDEA_resistance_model_kN

%A. Shear resistance of the beam
Av_mm2 = A_b_mm2 - 2 .* b_b_mm .* tf_b_mm + (tw_b_mm + 2 .* r_b_mm) .* tf_b_mm;%
NEN-EN 1993-1-1, art. 6.2.6.(3) eq(6.18)
Avnet_b_mm2 = Av_mm2 - (n2b .* d0_mm) .* tw_b_mm

VplRd_b_N = Avnet_b_mm2 .* fy_b_Ndmm2 /sqrt(3) ./ gamma_M0;    %NEN-EN 1993-1-1,
art. 6.2.6.(2) eq(6.18)
VplRd_b_kN = VplRd_b_N ./ 1000

    %Unity Check A
    UC_A = V_Ed_kN ./ VplRd_b_kN


%B. Tearing resistance of the beam
Ant_beam_mm2 = (e222_b_mm - 0.5 .* d0_mm) .* tw_b_mm;
Anv_beam_mm2 = (h_b_mm - (n2b - 0.5)*d0_mm - e111_b_mm) .* tw_b_mm;

Veff2Rd_b_N = (0.5 .* fu_b_Ndmm2 .* Ant_beam_mm2) ./ gamma_M2 + ((Anv_beam_mm2 .*
fy_b_Ndmm2) ./ sqrt(3)) ./ gamma_M0;%NEN-EN 1993-1-8, art.3.10.2(3) eq(3.10)
Veff2Rd_b_kN = Veff2Rd_b_N ./ 1000

    %Unity Check B
    UC_B = V_Ed_kN ./ Veff2Rd_b_kN


%C. Shear resistance of the bolts patern beam
FvRd_N = (alpha_v .* fu_bo_Ndmm2 .* As_bo_mm2) ./ gamma_M2;%NEN-EN 1993-1-8, art.
3.6.1 table(3.4)
FvRd_kN = FvRd_N ./ 1000;

FvRd_bobeam_tot_kN = 2 .* FvRd_kN;  %Double shear patern

    %Unity Checks C1
    UC_C1_bo1 = F_Ed_res_bo1_kN ./ FvRd_bobeam_tot_kN
    UC_C2_bo2 = F_Ed_res_bo2_kN ./ FvRd_bobeam_tot_kN
    UC_C3_bo3 = F_Ed_res_bo3_kN ./ FvRd_bobeam_tot_kN

    UC_C_max = max([UC_C1_bo1 UC_C2_bo2 UC_C3_bo3])

%D. Bearing resistance of the bolts in the beam, NEN-EN 1993-1-8, art. 3.6.1 table
(3.4)
FbRd_ver_bo123_N = (k1_vert_bo123 .* alpha_b_vert_bo123 .* fu_b_Ndmm2 .* d_mm .*
tw_b_mm) ./ gamma_M2;
FbRd_ver_bo123_kN = FbRd_ver_bo123_N ./ 1000;

FbRd_hor_bo1_N = (k1_hor_bo1 .* alpha_b_hor_bo1 .* fu_b_Ndmm2 .* d_mm .* tw_b_mm)
/ gamma_M2;
FbRd_hor_bo1_kN = FbRd_hor_bo1_N ./ 1000;

FbRd_hor_bo23_N = (k1_hor_bo23 .* alpha_b_hor_bo23 .* fu_b_Ndmm2 .* d_mm .*
tw_b_mm) ./ gamma_M2;
```

```
FbRd_hor_bo23_kN = FbRd_hor_bo23_N ./ 1000;


    %Unity Check D
    UC_D1_vert_bo1 = F_Ed_vert_bo1_kN ./ FbRd_ver_bo123_kN
    UC_D2_vert_bo2 = F_Ed_vert_bo2_kN ./ FbRd_ver_bo123_kN
    UC_D3_vert_bo3 = F_Ed_vert_bo3_kN ./ FbRd_ver_bo123_kN

    UC_D1_hor_bo1 = F_Ed_hor_bo1_kN ./ FbRd_hor_bo1_kN
    UC_D2_hor_bo2 = F_Ed_hor_bo2_kN ./ FbRd_hor_bo23_kN
    UC_D3_hor_bo3 = F_Ed_hor_bo3_kN ./ FbRd_hor_bo23_kN

    %Unity Check, Elipitcal Method BmS
    UC_D1_EM_bo1 = sqrt(UC_D1_vert_bo1.^2 + UC_D1_hor_bo1.^2)
    UC_D2_EM_bo2 = sqrt(UC_D2_vert_bo2.^2 + UC_D2_hor_bo2.^2)
    UC_D3_EM_bo3 = sqrt(UC_D3_vert_bo3.^2 + UC_D3_hor_bo3.^2)

    %Unity Check according NEN-EN 1993-1-8, art. 3.6.1(3),
        %table 3.4, Note (3)
    UC_D1_ECmax = max([UC_D1_vert_bo1 UC_D1_hor_bo1])
    UC_D2_ECmax = max([UC_D2_vert_bo2 UC_D2_hor_bo2])
    UC_D3_ECmax = max([UC_D3_vert_bo3 UC_D3_hor_bo3])


    UC_D_max = max([ UC_D1_vert_bo1 UC_D2_vert_bo2 UC_D3_vert_bo3 UC_D1_hor_bo
UC_D2_hor_bo2 UC_D3_hor_bo3 ])


%E. Shear Resistance Cleat
Av_mm2 = (hp_cl_mm - (n2b .* d0_mm)) .* t_cl_mm

VplRd_cl_N = ((Av_mm2 .* fy_cl_Ndmm2) ./ sqrt(3)) ./ gamma_M0;%NEN-EN 1993-1-1,
art. 6.2.6.(2) eq(6.18)
VplRd_cl_kN = VplRd_cl_N ./ 1000;

VplRdtot_cl_kN = 2 .* VplRd_cl_kN       %Double cleats

    %Unity Check E
    UC_E = V_Ed_kN ./ VplRdtot_cl_kN


%F. Tearing resistance of the cleat
Ant_cl1_mm2 = (e2_cl_mm - 0.5 .* d0_mm) .* t_cl_mm;
Anv_cl1_mm2 = (hp_cl_mm - e1_cl_mm - ((n2b - 0.5) .* d0_mm)) .* t_cl_mm;

Veff2Rd_cl1_N =  ((0.5 .* fu_cl_Ndmm2 .* Ant_cl1_mm2) ./ gamma_M2) + (((fy_cl_Ndmm
.* Anv_cl1_mm2) ./ sqrt(3)) ./ gamma_M0);%NEN-EN 1993-1-8, art. 3.10.2(3) eq(3.10)
Veff2Rd_cl1_kN = Veff2Rd_cl1_N ./ 1000

Veff2Rdtot_cl1_kN = 2 .* Veff2Rd_cl1_kN    %Double cleats

    %Unity Check F
    UC_F = V_Ed_kN ./ Veff2Rdtot_cl1_kN
```



```
%G Shear resisitance bolt patern column
FvRd_kN = FvRd_kN       %%NEN-EN 1993-1-8, art. 3.6.1 table(3.4)

    %Unity Checks G
    UC_G11 = F_Ed_res_bo11_kN ./ FvRd_kN
    UC_G22 = F_Ed_res_bo22_kN ./ FvRd_kN

    UC_G_max = max([ UC_G11 UC_G22 ])

    UC_G33 = UC_G11
    UC_G44 = UC_G22

%H Vertical bearing resistance

    UC_H11_hor = F_Ed_hor_bo11_kN ./ FbRd_hor_bo11_kN
    UC_H11_vert = F_Ed_vert_bo11_kN ./ FbRd_vert_bo11_kN


    UC_H22_hor = F_Ed_hor_bo22_kN ./ FbRd_hor_bo22_kN
    UC_H22_vert = F_Ed_vert_bo22_kN ./ FbRd_vert_bo22_kN


    UC_H_max = max([UC_H11_hor UC_H11_vert UC_H22_hor UC_H22_vert])

    %Bolt resistances according Elliptical Method
    UC_H11_EM = sqrt(UC_H11_hor.^2 + UC_H11_vert.^2)
    UC_H22_EM = sqrt(UC_H22_hor.^2 + UC_H22_vert.^2)
    UC_H33_EM = UC_H11_EM
    UC_H44_EM = UC_H22_EM

    %Bolt resistances according NEN-EN 1993-1-8, art. 3.6.1(3),
        %table 3.4, Note (3)
    UC_H11_ECmax = max([UC_H11_hor UC_H11_vert])
    UC_H22_ECmax = max([UC_H22_hor UC_H22_vert])
    UC_H33_ECmax = UC_H11_ECmax
    UC_H44_ECmax = UC_H22_ECmax



%I. Tearing Resistance of the Cleats
Anv_cl2_mm2 = (hp_cl_mm - e11_cl_mm - ((n2cc - 0.5) .* d0_mm)) .* t_cl_mm;
Ant_cl2_mm2 = (e22_cl_mm - 0.5 .* d0_mm) .* t_cl_mm;

VplRd_N = ((0.5 .* Ant_cl2_mm2 .* fu_b_Ndmm2) ./ gamma_M2) + ((fy_b_Ndmm2 .*
Anv_cl2_mm2) ./ sqrt(3)) ./ gamma_M0;%NEN-EN 1993-1-8, art. 3.10.2(3) eq(3.10)
VplRd_kN = VplRd_N ./ 1000

VplRdtot_kN = 2 .* VplRd_kN     %Double cleats
```

```
   %Unity Check I
    UC_I = V_Ed_kN ./ VplRdtot_kN


%J. Moment resistance of the cleat
exc_cleat_m = ((e_exc1_m * 1000) - r_cl_mm - t_cl_mm) ./ 1000
M_Ed_clpoint_kNm = V_Ed_kN .* exc_cleat_m

Wpl_mm3 = 0.25 .* t_cl_mm .* hp_cl_mm.^2
MplRd_Nmm = Wpl_mm3 .* fy_cl_Ndmm2      %NEN-EN 1993-1-1, art. 6.2.5(2) eq(6.13)
MplRd_kNm = MplRd_Nmm ./ 1000000

MplRdtot_kNm = 2 .* MplRd_kNm;

VplRd_clmres_N = (t_cl_mm .* hp_cl_mm .* (fy_cl_Ndmm2 ./ sqrt(3))) ./ gamma_M0
VplRd_clmres_kN = VplRd_clmres_N ./ 1000
VplRd_clmrestot_kN = 2 .* VplRd_clmres_kN

    %If statement for combination shear force en moment, NEN-EN 1993-1-8
    %art. 6.8.2.

    if  V_Ed_kN < 0.5 .* VplRd_clmrestot_kN
        rho_if = 0
    elseif V_Ed_kN < VplRd_clmrestot_kN
        rho_if = (((2 .* V_Ed_kN) ./ VplRd_clmrestot_kN) - 1 ).^2
    else
        rho_if = error_if_statement_not_applicable_check_V_Ed_kN
    end

fyred_cl_Ndmm2 = ( 1 - rho_if) .* fy_cl_Ndmm2

MplRd_red_kNm = MplRdtot_kNm .* (fyred_cl_Ndmm2 ./ fy_cl_Ndmm2)

    %Unity Check J
    UC_M = M_Ed_clpoint_kNm ./ MplRd_red_kNm

%IDEA Resistance

    %Unity Check IDEA
    UC_IDEA = V_Ed_kN ./ IDEA_resistance_model_kN

%Unity Check Matrix
UC_matrix_max = [ UC_A UC_B UC_C_max UC_D_max UC_E UC_F UC_G_max UC_H_max UC_I UC_↙
]

RV_matrix       =       [   0               0               ↙
0                0                0                           0           ↙
0                  0                    0;
                            VplRd_b_kN      Veff2Rd_b_kN    FvRd_bobeam_tot_k↙
FbRd_ver_bo123_kN   VplRdtot_cl_kN     Veff2Rdtot_cl1_kN      FvRd_k↙
FbRd_vert_bo11_kN      FbRd_vert_bo22_kN       VplRdtot_kN         MplRd_red_kNm;
                            0               0               ↙
0                0                0                           0           ↙
```



```
0                       0                   0               ]

UC_bo123_matrix_EC =     [   UC_D1_vert_bo1       UC_D2_vert_bo2       UC_D3_vert_bo↙
0           0               0               0;
                             UC_D1_hor_bo1        UC_D2_hor_bo2        UC_D3_hor_bo↙
0           0               0               0]

UC_bo123_matrix_EM =     [   0                    0                    ↙
0           UC_D1_EM_bo1    UC_D2_EM_bo2    UC_D3_EM_bo3;
                             0                    0                    ↙
0           0               0               0]

UC_bo14_matrix_EC    =   [   UC_H11_vert      UC_H22_vert     0       0       0;
                             UC_H11_hor       UC_H22_hor      0       0       0]

UC_bo14_matrix_EM    =   [   0                    0               0       UC_H11_E↙
UC_H22_EM;
                             0                    0               0       ↙
0           ]

R_C_com_matrix           =   [FbRd_ver_bo123_kN      0                          ↙
Veff2Rd_b_kN    0]

R_IDEA_com_matrix        =   [0                          IDEA_resistance_bbolt_kN    ↙
0               IDEA_resistance_model_kN]


UC_matrix_EM        = [  0   UC_A     UC_B    UC_C1_bo1   UC_D1_EM_bo1    UC_↙
UC_F    UC_G11      UC_H11_EM       UC_I        UC_M    0;
                         0   0        0       UC_C2_bo2   UC_D2_EM_bo2    0        ↙
UC_G22      UC_H22_EM       0           0       0;
                         0   0        0       UC_C3_bo3   UC_D3_EM_bo3    0        ↙
UC_G33      UC_H33_EM       0           0       0;
                         0   0        0       0           0               0        ↙
UC_G44      UC_H44_EM       0           0       0]

UC_matrix_EC      =   [   0   UC_A     UC_B    UC_C1_bo1   UC_D1_vert_bo1  UC_↙
UC_F    UC_G11      UC_H11_vert     UC_I        UC_M    0;
                          0   0        0       UC_C2_bo2   UC_D2_vert_bo2  0       ↙
UC_G22      UC_H22_vert     0           0       0;
                          0   0        0       UC_C3_bo3   UC_D3_vert_bo3  0       ↙
UC_G33      UC_H11_vert     0           0       0;
                          0   0        0       0           0               0       ↙
UC_G44      UC_H22_vert     0           0       0]

UC_matrix_FEA     =   [   0   0        0       0           0               0       ↙
0           0               0           0       UC_FEA]


UC_matrix_trans_EM      = transpose(UC_matrix_EM)
UC_matrix_trans_EC      = transpose(UC_matrix_EC)
RV_matrix_trans         = transpose(RV_matrix)
```

```
UC_bo123_matrix_EC_trans  = transpose(UC_bo123_matrix_EC)
UC_bo123_matrix_EM_trans  = transpose(UC_bo123_matrix_EM)

UC_bo14_matrix_EC_trans = transpose(UC_bo14_matrix_EC)
UC_bo14_matrix_EM_trans = transpose(UC_bo14_matrix_EM)

R_C_com_matrix          = transpose(R_C_com_matrix)
R_IDEA_com_matrix       = transpose(R_IDEA_com_matrix)



%Bar Chart Resistance Components
figure
bar(RV_matrix_trans)
grid on
set(gca, 'XTickLabel', {,'CR1','CR2','CR3','CR4','CR5','CR6','CR7', 'CR8',
'CR9','CR10'})
xlabel('Component')
ylabel('Resistance value [kN] or [kNm]')
title('Resistance values of double angle cleat connection')



% %Bar Chart Resistance Components Bo123
% figure
% bar(UC_bo123_matrix_EC_trans)
% hold on
% bar(UC_bo123_matrix_EM_trans, 'k')
% grid on
% xlabel('Bolts in beam')
% ylabel('Unity Check')
% title('Unity Checks of bearing capacity in beam bolt patern')
% legend('EC: Vertical Direction', 'EC: Horizontal Direction','Elliptical Method')
% set(gca, 'XTickLabel', {'Bolt 1','Bolt 2','Bolt3','','Bolt 1','Bolt 2','Bolt 3'})


% %Bar Chart Resistance Components Bo14
% figure
% bar(UC_bo14_matrix_EC_trans)
% hold on
% bar(UC_bo14_matrix_EM_trans, 'k')
% grid on
% title('Unity checks of double angle cleat connection in column bolt pattern')
% xlabel('Bolt in column')
% ylabel('Unity Check')
% legend('EC: Vertical Direction', 'EC: Horizontal Direction','Elliptical Method')
% set(gca, 'XTickLabel', {'Bolt 11/33','Bolt 22/44','','Bolt 11/33','Bolt 22/44'})

%Bar Chart Resistance Components
figure
bar(R_C_com_matrix)
hold on
```



```
bar(R_IDEA_com_matrix, 'r')
grid on
title('Overview of governing failure modes of components')
xlabel('Failure mode of component')
ylabel('Resistance Value [kN]')
legend('Manual Calcualtion, EC3 ', 'FEA Model', 'Location', 'Northwest')
set(gca, 'XTickLabel', {,'Bearing, Bolt 1','Shear Ut, Bolt 1 (Bearing)','','Web
Tearing','5% Strain Limit Web'})

% %Bar Chart EM
%
%     %Prepare unity check line
%     UC_length = length(UC_matrix_EM);
%     UC_value_x = [1:1:UC_length];
%     UC_value_y = ones(1,UC_length);
%
%     %Preprare resistance line
%     IDEA_length = length(UC_matrix_EM);
%     IDEA_value_x = [1:1:IDEA_length];
%     IDEA_value_y = UC_IDEA .* ones(1, IDEA_length);
%
%     %Make Figure
%     figure
%     bar(UC_matrix_trans_EM)
%     hold on
%     plot(UC_value_x, UC_value_y, 'k', 'Linewidth', 1.5)
%     hold on
%     plot(IDEA_value_x, IDEA_value_y, 'r', 'Linewidth', 1.5)
%     hold on
%
%     %Axis and lines
%     axis([0 13 0 1.2])
%     xlabel('Unity Check [-]')
%     ylabel('Unity Check Value')
%     yticks([0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 1.2])
%     grid on
%
%     %Text in figure
%     loadcase = V_Ed_kN;
%     resistancecase = IDEA_resistance_web_kN;
%     title_case = sprintf('Unity Checks for a double cleated shear connection fo
a load V_{Ed} = %d kN (Bolt resistance; Elliptical Method)', loadcase)
%     resistancecase = sprintf('UC IDEA Connection (V_{Rd} = %d kN)'
resistancecase)
%     set(gca, 'XTickLabel', {'','UC-A','UC-B','UC-C; 1-3','UC-D; 1-3','UC-E','UC
F','UC-G; 11-44','UC-H; 11-44','UC-I','UC-J'})
%     title(title_case)
%
%     %Legend
%     legend('UC Eurocode, subcomp. A', 'UC Eurocode, subcomp. B', 'UC Eurocode
subcomp. C', 'UC Eurocode, subcomp. D', 'Unity Check Line', resistancecase)
```

```
% %Bar Chart EC
%
%     %Prepare unity check line
%     UC_length = length(UC_matrix_EC);
%     UC_value_x = [1:1:UC_length];
%     UC_value_y = ones(1,UC_length);
%
%     %Preprare resistance line
%     IDEA_length = length(UC_matrix_EC);
%     IDEA_value_x = [1:1:IDEA_length];
%     IDEA_value_y = UC_IDEA .* ones(1, IDEA_length);
%
%     %Make Figure
%     figure
%     bar(UC_matrix_trans_EC)
%     hold on
%     plot(UC_value_x, UC_value_y, 'k', 'Linewidth', 1.5)
%     hold on
%     plot(IDEA_value_x, IDEA_value_y, 'r', 'Linewidth', 1.5)
%     hold on
%
%     %Axis and lines
%     axis([0 13 0 1.2])
%     xlabel('Unity Check [-]')
%     ylabel('Unity Check Value')
%     yticks([0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 1.2])
%     grid on
%
%     %Text in figure
%     loadcase = V_Ed_kN;
%     resistancecase = IDEA_resistance_web_kN;
%     title_case = sprintf('Unity Checks for a double cleated shear connection fo
a load V_{Ed} = %d kN (Bolt Resistance; Eurocode)' , loadcase)
%     resistancecase = sprintf('UC IDEA Connection (V_{Rd} = %d kN)'
resistancecase)
%     set(gca, 'XTickLabel', {'','UC-A','UC-B','UC-C; 1-3','UC-D; 1-3','UC-E','UC
F','UC-G; 11-44','UC-H; 11-44','UC-I','UC-J'})
%     title(title_case)
%
%     %Legend
%     legend('UC Eurocode, subcomp. A', 'UC Eurocode, subcomp. B', 'UC Eurocode
subcomp. C', 'UC Eurocode, subcomp. D', 'Unity Check Line', resistancecase)

%Bar Chart EC

    %Prepare unity check line
    UC_length = length(UC_matrix_EC);
    UC_value_x = [1:1:UC_length];
    UC_value_y = ones(1,UC_length);

    %Make Figure
    figure
    bar(UC_matrix_trans_EC)
```



```
    hold on
    bar(UC_matrix_FEA, 'r')
    hold on
    plot(UC_value_x, UC_value_y, 'k', 'Linewidth', 1.5)

    %Axis and lines
    axis([0 13 0 1.2])
    xlabel('Unity Check')
    ylabel('Unity Check Value [-]')
    yticks([0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 1.2])
    grid on

    %Text in figure
    loadcase = V_Ed_kN;
    resistancecase = IDEA_resistance_model_kN;
    title_case = sprintf('Unity Checks for a double cleated shear connection for a
load V_{Ed} = %d kN (Bolt Resistance; Eurocode)', loadcase)
    resistancecase = sprintf('UC IDEA Connection (V_{Rd} = %d kN)', resistancecase)
    set(gca, 'XTickLabel', {'','UC-A','UC-B','UC-C','UC-D','UC-E','UC-F','UC-
G','UC-H','UC-I','UC-J', 'UC-Model'})
    title(title_case)

    %Legend
    legend('UC (sub)comp. A', 'UC (sub)comp. B', 'UC (sub)comp. C', 'UC (sub)comp.
D','Model Resistance FEA', 'Unity Check Line', resistancecase, 'Location',
'Northwest')
```

B.4 CA4: MRJ, Extended Endplate Joint symmetrical loading

```
%Clear Everything
clear all
clc
close all

pause(0.3)

%%Calculation Moment resisting connection, Extendend end plate

%%Assumption:
    % - M16 bolts are used and programmed
    % - Redistribution of forces in the column flange allowed.
    % - Only bolts in the tension zone contribute to tension resistance.

%%Note:
    % - MANNUALLY INPUT LATER at moment resistance!!!

%%Results IDEA connection
IDEA_resistance_kNm = 45;      %Bending moment [kNm] both sides

%%Parameters

    %Load parameters
    q_Ed_kNdm = 7.5
    lsys_m = 8
    M_Ed_kNm = (1 ./ 12) .* q_Ed_kNdm .* lsys_m.^2;
    V_Ed_kN = 0;
    beta_trans = 0; %NEN-EN 1993-1-8, art. 6.2.6.2 table(6.3)
    omega_load = 1; %NEN-EN 1993-1-8, art. 5.3 table(5.4)
    kwc = 1;       %NEN-EN 1993-1-8, art.6.2.6.2 end note

    %Alpha_labda factor, NEN-EN 1993-1-8, art. 6.2.6.5 figure(6.11)
    alpha_lambda = 5.8

    %%Geometry Parameters
    %Profile beam -> IPE 220, steelquality S235
    h_b_mm = 220;           %Height of the column [mm]
    b_b_mm = 110;           %Width of the column [mm]
    tw_b_mm = 5.9;          %Web thickness of the column [mm]
    tf_b_mm = 9.2;          %Flange thickness of the column [mm]
    r_b_mm = 12;            %Radius of the column [mm]
    A_b_mm2 = 3340;         %

    Wpl_b_mm3 = 285000;     %Sectional area of the beam [mm3]

    fy_b_Ndmm2 = 235;       %Yield strength of the beam [N/mm2]
    fu_b_Ndmm2 = 360;       %Tensile strength fo the beam [N/mm2]

    %Profile column -> HEB 140, steelquality S235
    h_c_mm = 140;           %Height of the column [mm]
    b_c_mm = 140;           %Width of the column [mm]
    tw_c_mm = 7;            %Web thickness column [mm]
    tf_c_mm = 12;           %Flange thickness column [mm]
```



```
    r_c_mm = 12;            %Radius column [mm]
    A_c_mm2 = 4300;         %Cross-sectional area column [mm2]

    fy_c_Ndmm2 = 235;       %Yield strength column [N/mm2]
    fu_c_Ndmm2 = 360;       %Tensile strength column [N/mm2]

    %Profile Plate
    b_p_mm = 140;           %Plate width [mm]
    t_p_mm = 12;            %Plate thickess [mm]

    w_p_mm = 80;            %Vertical pitch [mm]
    e1_ver_mm = 40;         %Vertical end distance plate [mm]
    e2_hor_mm = 30;         %Horizontal end distance plate [mm]
    p1_ver_mm = 70;         %Vertical pitch [mm]

    nc1_up = 2;             %Bolt columns [-]
    nc2_up = 2;             %Bolt rows [-]

    fy_p_Ndmm2 = 235;       %Yield strength of the end plate [N/mm2]
    fu_p_Ndmm2 = 360;       %Tensile strength of the end plate [N/mm2]


    %Bolts -> 6 x M16, steelquality 8.8
    fub_bo_Ndmm2 = 800;     %Tensile strength of the bolts [N/mm2]
    As_bo_mm2 = 157;        %Cross-sectional area of the bolt [mm2]
    d_mm = 16;              %Diameter of the bolt [mm]
    d0_mm = 18;             %Bolt hole diameter [mm]
    alpha_v = 0.6;          %Alpha factor bolt [-]
    k2 = 0.9                %k2 factor bolt [-]

    %Welds
    aw_f_b_mm = 5;          %Welds at the flange [mm]
    aw_w_b_mm = 3;          %Welds at the web [mm]

    %Distances from the compression zone
    x1_distance_mm = 175.8; %Distance compression - inside bolt [mm]
    x2_distance_mm = 245.8; %Distance compression - outside bolt [mm]

    %Safety factor
    gamma_M0 = 1.0;         %Safety factor cross sections [-]
    gamma_M2 = 1.25;        %Safety factor cross section in tension till rupture [
]

%%Derived Parameters

    %M-distance of the end plate
    m1_hor_p_mm = 0.5 .* w_p_mm - 0.5 .* tw_b_mm - 0.8 .* sqrt(2) .* aw_w_b_mm
    m2_ver_p_mm = 0.5 .* p1_ver_mm - 0.5 .* tf_b_mm - 0.8 .* sqrt(2) .* aw_f_b_mm

    %M-distance of the column flange
    m11_hor_c_mm = 0.5 .* w_p_mm - 0.5 .* tw_c_mm - 0.8 .* r_c_mm

    %Tension resistance of the bolts (All M16)
```

```
    FtRd_M16_N = (k2 .* fub_bo_Ndmm2 .* As_bo_mm2) ./ gamma_M2;    %NEN-EN 1993-1-
8, art. 3.6.1 table(3.4)
    FtRd_M16_kN = FtRd_M16_N ./ 1000;


%%Unity Checks

%A. Column Flange Bending

    %Effective lengths A, column flange NEN-EN 1993-1-8, art. 6.2.6.4.1(3) tabl
(6.4)
    leffcp_a1_mm = 2 .* pi .* m11_hor_c_mm;
    leffcp_a2_mm = pi .* m11_hor_c_mm + p1_ver_mm;
    leffnc_a3_mm = 4 .* m11_hor_c_mm + 1.25 .* e2_hor_mm;
    leffnc_a4_mm = 2 .* m11_hor_c_mm + 0.625 .* e2_hor_mm + 0.5 .* p1_ver_mm;

    leff_a14_mm = [leffcp_a1_mm leffcp_a2_mm leffnc_a3_mm leffnc_a4_mm];
    leff_amin_mm = min([leff_a14_mm]);

    %Plastic Resitance Moment
    MplRd_cfl_Nmm = 0.25 .* leff_amin_mm .* t_p_mm.^2 * fy_c_Ndmm2;
    MplRd_cfl_kNm = MplRd_cfl_Nmm ./ 1.0E6


    %%T-stub resistance A

    %Distances
    m11_hor_c_m = m11_hor_c_mm ./ 1000;
    ne_hor_cf_m = e2_hor_mm ./ 1000;

    %Failure modes T-stub A.
    Ft1Rd_cf_fma1_kN = (4 .* MplRd_cfl_kNm) ./ m11_hor_c_m;
    Ft2Rd_cf_fma2_kN = (2 .* MplRd_cfl_kNm + 2 .* ne_hor_cf_m .* FtRd_M16_kN) .
(m11_hor_c_m + ne_hor_cf_m);
    Ft3Rd_cf_fma3_kN = 2 .* FtRd_M16_kN;

    FtRd_cf_fma_gov_kN = min([Ft1Rd_cf_fma1_kN Ft2Rd_cf_fma2_kN Ft3Rd_cf_fma3_kN])


%B1. Endplate Bending, extended plate

    %Effective lengths B, extend end plate, NEN-EN 1993-1-8, art. 6.2.6.4.2
    %table (6.5)
    leffcp_b1_mm = 2 .* pi .* m2_ver_p_mm;
    leffcp_b2_mm = pi .* m2_ver_p_mm + w_p_mm;
    leffcp_b3_mm = pi .* m2_ver_p_mm + 2 .* e2_hor_mm;
    leffnc_b4_mm = 4 .* m2_ver_p_mm + 1.25 .* e1_ver_mm;
    leffnc_b5_mm = e2_hor_mm + 2 .* m2_ver_p_mm + 0.625 .* e1_ver_mm
    leffnc_b6_mm = 0.5 .* b_p_mm;
    leffnc_b7_mm = 0.5 .* w_p_mm + 2 .* m2_ver_p_mm + 0.625 .* e1_ver_mm;

    leff_b17_mm = [leffcp_b1_mm leffcp_b2_mm leffcp_b3_mm leffnc_b4_mm leffnc_b5_mm
leffnc_b6_mm leffnc_b7_mm]
```



```
    leff_bmin_mm = min([leff_b17_mm])

    %Plastic resistance moment
    WplRd_endp_mm3 = 0.25 .* leff_bmin_mm .* (t_p_mm) .^ 2;
    MplRd_endp_Nmm = WplRd_endp_mm3 .* fy_p_Ndmm2;
    MplRd_endp_kNm = MplRd_endp_Nmm ./ 1.0E6

    %T-stub resistances

    %Distances of the extended part of the end plate in m
    m2_ver_pe_m = m2_ver_p_mm ./ 1000
    ne_ver_pe_m = min([(e1_ver_mm ./ 1000) ((1.25 .* m2_ver_p_mm)./1000)])

    %Failure modes T-stub, B1 extend.
    Ft1Rd_endp_fmb1_kN = (4 .* MplRd_endp_kNm) ./ m2_ver_pe_m;
    Ft2Rd_endp_fmb2_kN = (2 .* MplRd_endp_kNm + 2 .* ne_ver_pe_m .* FtRd_M16_kN) .
( m2_ver_pe_m + ne_ver_pe_m );
    Ft3Rd_endp_fmb3_kN = 2 .* FtRd_M16_kN;

    FtRd_endpe_fmb13_kN = [Ft1Rd_endp_fmb1_kN Ft2Rd_endp_fmb2_kN
Ft3Rd_endp_fmb3_kN]
    FtRd_endpe_fmb_kN = 0.5 .* min(FtRd_endpe_fmb13_kN)    %Only extended plate
part so devided by 2
    FtRd_endpe_fmb_gov_kN = 2.* FtRd_endpe_fmb_kN          %Two bolt rows, so
multiplied by 2


%B2. Endplate Bending, between the flanges inside

    %Distances of the inside part, between the flanges of the end plate
    m1_hor_pi_mm = 0.5 .* w_p_mm - 0.8 .* aw_w_b_mm .* sqrt(2) - 0.5 .* tw_b_mm
    m2_ver_pi_mm = 0.5 .* p1_ver_mm - 0.8 .* aw_f_b_mm .* sqrt(2) - 0.5 .* tf_b_mm
    e2_hor_pi_mm = e2_hor_mm

    m1_hor_pi_m = m1_hor_pi_mm ./ 1000;
    ne_hor_pi_m = min([ (e2_hor_pi_mm ./ 1000) (1.25 .* (m1_hor_pi_mm ./ 1000)) ])

    %Labda values, NEN-EN 1993-1-8, art. 6.2.6.4.5 figure(6.11)
       %(pi = plate inside the flanges)
    lambda_1 = m1_hor_pi_mm ./ (m1_hor_pi_mm + e2_hor_pi_mm)
    lambda_2 = m2_ver_pi_mm ./ (m1_hor_pi_mm + e2_hor_pi_mm)

    leffcp_bi1_mm = 2 .* pi .* m1_hor_pi_mm
    leffnp_bi2_mm = alpha_lambda .* m1_hor_pi_mm

    leff_bi12_mm = [leffcp_bi1_mm leffnp_bi2_mm]
    leff_bimin_mm = min([leff_bi12_mm])

    %Plastic resistance moment
    MplRd_endpi_Nmm = 0.25 .* leff_bimin_mm .* t_p_mm.^2 .* fy_p_Ndmm2
    MplRd_endpi_kNm = MplRd_endpi_Nmm ./ 1.0E6

    %Failure modes T-stub, B1 inside.
```

```
    Ft1Rd_endpi_fmb1_kN = (4 .* MplRd_endpi_kNm) ./ m1_hor_pi_m;
    Ft2Rd_endpi_fmb2_kN = (2 .* MplRd_endpi_kNm + 2 .* ne_hor_pi_m .* FtRd_M16_kN)
./ (m1_hor_pi_m + ne_hor_pi_m);
    Ft3Rd_endpi_fmb3_kN = 2 .* FtRd_M16_kN;

    FtRd_endpi_fmb13_kN = [Ft1Rd_endpi_fmb1_kN Ft2Rd_endpi_fmb2_kN
Ft3Rd_endpi_fmb3_kN]
    FtRd_endpi_fmb_gov_kN = min([FtRd_endpi_fmb13_kN])

%C Columnweb under tension

    %Effective width is the same as the effective governing
    %length of the bolt pattern in the column flange
    beffw_c_mm = 2 .* leff_amin_mm

    FtRdw_c_N = (omega_load .* beffw_c_mm .* tw_c_mm .* fy_c_Ndmm2) ./ gamma_M0;
    FtRdw_c_kN = FtRdw_c_N ./ 1000

%D. Beamweb under tension

    %Effective width is the same as the effective groverning length fo the
    %bolt patern of the inside bolts in the endplate
    beffw_b_mm = leff_bimin_mm

    FtRdw_b_N = (beffw_b_mm .* tw_b_mm .* fy_b_Ndmm2) ./ gamma_M0;
    FtRdw_b_kN = FtRdw_b_N ./ 1000

%E. Tension of the bolts
    FtRd_bo_kN = 2 .* FtRd_M16_kN       %two bolt rows


%F. Shear Resistance Column
    Av_c_mm2 = A_c_mm2 - (2 .* b_c_mm .* tf_c_mm) + (tw_c_mm + 2 .* r_c_mm) .*
tf_c_mm;
    VwpRd_c_N = (0.9 .* (fy_c_Ndmm2 .* Av_c_mm2) ./ sqrt(3)) ./ gamma_M0;
    VwpRd_c_kN = VwpRd_c_N ./ 1000;

%E. Columnweb under compression

    %Effective width: NEN-EN 1993-1-8, art. 6.2.6.2. eq(6.11)
    beffw_c_ce_mm = tf_b_mm + (2 .* sqrt(2) .* aw_f_b_mm) + 5 .* (tf_c_mm + r_c_mm)
+ 2 .* t_p_mm;

    %Compression resistance: NEN-EN 1993-1-8, art. 6.2.6.2(1) eq(6.9)
    FcRdw_c_N = (omega_load .* kwc .* beffw_c_ce_mm .* tw_c_mm .* fy_c_Ndmm2) ./
gamma_M0;
    FcRdw_c_kN = FcRdw_c_N ./ 1000;


%F. Beamflange under compression

    McRd_b_Nmm = (Wpl_b_mm3 .* fy_b_Ndmm2) ./ gamma_M0;
    McRd_b_kNm = McRd_b_Nmm ./ 1.0E6;
```



```
    FcfbRd_N = McRd_b_Nmm ./ (h_b_mm - tf_b_mm);
    FcfbRd_kN = FcfbRd_N ./ 1000

%Resistances

    %Tension
    TF_A1_kN = FtRd_cf_fma_gov_kN
    TF_A2_kN = FtRd_cf_fma_gov_kN
    TF_Atot_kN = TF_A1_kN + TF_A2_kN

    TF_B1_kN = FtRd_endpe_fmb_gov_kN
    TF_B2_kN = FtRd_endpi_fmb_gov_kN
    TF_Btot_kN = TF_B1_kN + TF_B2_kN

    TF_C1_kN = FtRdw_c_kN
    TF_Ctot_kN = TF_C1_kN

    TF_D1_kN = FcfbRd_kN
    TF_Dtot_kN = TF_D1_kN

    TF_E1_kN = FtRd_bo_kN
    TF_E2_kN = FtRd_bo_kN
    TF_Etot_kN = TF_E1_kN + TF_E2_kN

    R_tension_comp_kN =     [   TF_A1_kN    TF_A2_kN    0;
                                TF_B1_kN    TF_B2_kN    0;
                                0           0           0;
                                0           0           0;
                                TF_E1_kN    TF_E2_kN    0]

    R_tension_tot_kN =      [   0    0    TF_Atot_kN;
                                0    0    TF_Btot_kN;
                                0    0    TF_Ctot_kN;
                                0    0    TF_Dtot_kN;
                                0    0    TF_Etot_kN ]

    %Shear
    R_shear_tot_kN = VwpRd_c_kN

    %Compression
    R_compression_kN = [   0; FcRdw_c_kN;     FcfbRd_kN;   0 ]


%Bar Chart Resistances

    %Tension
    figure
    barh(R_tension_tot_kN, 'm')
    xlabel('Resistance Values [kN]')
    ylabel('Components')
    hold on
    barh(R_tension_comp_kN, 'b')
```

```
    grid on
    axis([0 400 0 6])
    set(gca, 'YTickLabel', {'Column Flange', 'Endplate', 'Column web', 'Beam web',
'Bolts'})
    title('Resistance Values of the Components in Tension Zone')
    legend('Component Resistance')
    xlabel('Resistance Value [kN]')
    ylabel('Components')

    %Compression
    figure
    barh(R_compression_kN, 'b')
    grid on
    set(gca, 'YTickLabel', {'','Column Web', 'Beam Flange / Web',''})
    title('Resistance Values of the components in Compression Zone')
    axis([0 400 0 5])
    legend('Component Resistance')
    xlabel('Resistance Values [kN]')
    ylabel('Components')


%%Moment Resistance [MANUALLY INPUT, CHECK THE COMPRESSION ZONE !!!]

%Distances
x1_distance_m = x1_distance_mm ./ 1000;
x2_distance_m = x2_distance_mm ./ 1000;

    %Determine the resistances of the tension zone, shear zone, compression
    %zone and total moment resistance.
    F_Rd_tension_kN    =  min([TF_Atot_kN TF_Btot_kN TF_Ctot_kN TF_Dtot_kN
TF_Etot_kN])

    F_Rd_shear_kN        = VwpRd_c_kN

    F_Rd_compression_kN  = min([FcRdw_c_kN FcfbRd_kN])

    M_Rd_total_kNm = [(TF_B1_kN .* x2_distance_m) + (TF_B2_kN .* x1_distance_m)]

        %Resistance matrix zones EC
        R_zone_matrix_EC = [ F_Rd_tension_kN    F_Rd_shear_kN   F_Rd_compression_kN
]

        %Resistance matrix moment resistance EC and Model
        R_moment_matrix_EC = [ 0 M_Rd_total_kNm 0 0 0 ]
        R_moment_matrix_FEM = [  0 0 0 IDEA_resistance_kNm 0 ]



    %Determining the utilization
    F_Ed_kN = M_Ed_kNm ./ ((h_b_mm - tf_b_mm) ./ 1000)

    UC_1 = F_Ed_kN ./ F_Rd_tension_kN          %Tension resistance
```



```
    UC_2 = V_Ed_kN ./ F_Rd_shear_kN            %Shear resistance
    UC_3 = F_Ed_kN ./ F_Rd_compression_kN      %Compression resistance
    UC_4 = M_Ed_kNm ./ M_Rd_total_kNm          %Total moment resistance

    UC_IDEA = M_Ed_kNm ./ IDEA_resistance_kNm  %Resistance IDEA

    UC_comp_matrix = [0 UC_1 UC_2 UC_3 0 ]
    UC_fem_matrix = [0 0 0 0 0 0 UC_IDEA 0]


%Bar Chart Resistances

    %Prepare unity check line
    UC_length = length(UC_comp_matrix);
    UC_value_x = [1:1:UC_length];
    UC_value_y = ones(1,UC_length);

    %Preprare resistance line
    IDEA_length = length(UC_comp_matrix);
    IDEA_value_x = [1:1:IDEA_length];
    IDEA_value_y = UC_IDEA .* ones(1, IDEA_length);


    %Figure Resistances Accoording EC
    figure
    bar(R_zone_matrix_EC)
    xlabel('Resistances per zone')
    ylabel('Resistance Value [kN]')
    grid on
    set(gca, 'XTickLabel', {'Tension Zone', 'Shear Zone', 'Compression Zone'})
    title('Governing Resistances Values for Each Zone')
    legend('Manual Calculation, EC3',  'Location', 'Northwest')

    %Figure Moment Resistance, Model and IDEA
    figure
    bar(R_moment_matrix_EC)
    hold on
    bar(R_moment_matrix_FEM,'r')
    xlabel('Total Moment Resistance')
    ylabel('Resistance Value [kNm]')
    grid on
    set(gca, 'XTicklabel', {'','Moment Resistance, MC','', 'Moment Resistance,
FEA',''})
    title('Moment Resistances Manual Calculation EC3 and FEA Model')
    legend('Manual Calculation EC3','FEA Model', 'Location', 'Southwest')

%     %Figure
%     figure
%     bar(UC_comp_matrix)
%     xlabel('Utilization')
%     ylabel('Utilization Value [-]')
%     title('Overview Utilization')
%     set(gca, 'XTickLabel', {'','Tension', 'Shear', 'Compression', ''
```

```
'Moment','', 'Model'})
%     axis([0 10 0 1.2])
%     hold on
%     bar(UC_fem_matrix, 'r')
%     grid on
%     hold on
%     plot(UC_value_x, UC_value_y, 'k', 'Linewidth', 1.5)
%     legend('MC: Eurocode 3', 'FEA Model')
```

B.5 CA5: MRJ, Extended Endplate Joint unsymmetrical loading

```
%Clear Everything
clear all
clc
close all

pause(0.3)

%Calculation Moment Resisting Connection, singular extended endplate
%connection
    % - M16 bolts are used and programmed
    % - Redistribution of forces in the column flange allowed
    % - Only bolts in the tension zone contribute to the tension
    % resistance.

%% FEA Model input
    MjRd_FEA_kNm = 44;


%% Load Parameters
    M_Ed_kNm = 40;
    V_Ed_kN = 0;
    beta_trans = 1; %NEN-EN 1993-1-8, art. 6.2.6.2 table(6.3)
    kwc = 1.0;      %NEN-EN 1993-1-8, art. 6.2.6.2 end note

    %%Alpha_lambda_factor, NEN-EN 1993-1-8, art. 6.2.6.5 figure(3.11)
    alpha_lambda = 5.8;



%%Geometry Parameters
%Profile Beam -> IPE 220, steelquality S235
h_b_mm = 220;
b_b_mm = 110;
tw_b_mm = 5.9;
tf_b_mm = 9.2;
r_b_mm = 12;
A_b_mm2 = 3340;

Wpl_b_mm3 = 285000;

fy_b_Ndmm2 = 235;
fu_b_Ndmm2 = 360;

%Profile Column -> HEB 140, steelquality S235
h_c_mm = 140;
b_c_mm = 140;
tw_c_mm = 7;
tf_c_mm = 12;
r_c_mm = 12;
A_c_mm2 = 4300;

fy_c_Ndmm2 = 235;
fu_c_Ndmm2 = 360;
```



```
%Profile Plate
b_p_mm = 140;
t_p_mm = 12;

w_p_mm = 80;
e1_ver_mm = 40;
e2_hor_mm = 30;
p1_ver_mm = 70;

nc1_up = 2;
nr2_up = 2;

fy_p_Ndmm2 = 235;
fu_p_Ndmm2 = 360;

%Bolts 2x3 M16 8.8
fub_bo_Ndmm2 = 800;
As_bo_mm2 = 157;
d_bo_mm = 16;
d0_bo_mm = 18;
alpha_v = 0.6;
k2 = 0.9

x1_max_distance_mm = 246
x2_min_distance_mm = 176

%Welds
aw_f_b_mm = 5;
aw_w_b_mm = 3;

%Safety factors NEN-EN 1993-1-8, art. 6.1(1)
gamma_M0 = 1.0;
gamma_M1 = 1.0;
gamma_M2 = 1.25;


%%Derived Parameters

    %M-distance of the end plate
    m1_hor_p_mm = 0.5 .* w_p_mm - 0.5 .* tw_b_mm - 0.8 .* sqrt(2) .* aw_w_b_mm
    m2_ver_p_mm = 0.5 .* p1_ver_mm - 0.5 .* tf_b_mm - 0.8 .* sqrt(2) .* aw_f_b_mm

    %M-distance of the column flange
    m11_hor_c_mm = 0.5 .* w_p_mm - 0.5 .* tw_c_mm - 0.8 .* r_c_mm

    %Tension resistance of the bolts (All M16)
    FtRd_M16_N = (k2 .* fub_bo_Ndmm2 .* As_bo_mm2) ./ gamma_M2;    %NEN-EN 1993-1-↙
8, art. 3.6.1 table(3.4)
    FtRd_M16_kN = FtRd_M16_N ./ 1000;


%A. Column Flange Bending (NOTE: Check of the Ne=e < 1.25*m must be done
```

```
%manually)

    %Effective lengths A, column flange NEN-EN 1993-1-8, art. 6.2.6.4.1(3) tabl
(6.4)
    leffcp_a1_mm = 2 .* pi .* m11_hor_c_mm;
    leffcp_a2_mm = pi .* m11_hor_c_mm + p1_ver_mm;
    leffnc_a3_mm = 4 .* m11_hor_c_mm + 1.25 .* e2_hor_mm;
    leffnc_a4_mm = 2 .* m11_hor_c_mm + 0.625 .* e2_hor_mm + 0.5 .* p1_ver_mm;

    leff_a14_mm = [leffcp_a1_mm leffcp_a2_mm leffnc_a3_mm leffnc_a4_mm];
    leff_amin_mm = min([leff_a14_mm])

    %Plastic Resitance Moment
    MplRd_cfl_Nmm = 0.25 .* leff_amin_mm .* t_p_mm.^2 * fy_c_Ndmm2;
    MplRd_cfl_kNm = MplRd_cfl_Nmm ./ 1.0E6


    %T-stub resistance A

    %Distances
    m11_hor_c_m = m11_hor_c_mm ./ 1000;
    ne_hor_cf_m = e2_hor_mm ./ 1000;

    %Failure modes T-stub A.
    Ft1Rd_cf_fma1_kN = (4 .* MplRd_cfl_kNm) ./ m11_hor_c_m;
    Ft2Rd_cf_fma2_kN = (2 .* MplRd_cfl_kNm + 2 .* ne_hor_cf_m .* FtRd_M16_kN) .
(m11_hor_c_m + ne_hor_cf_m);
    Ft3Rd_cf_fma3_kN = 2 .* FtRd_M16_kN;

    FtRd_cf_fma_gov_kN = min([Ft1Rd_cf_fma1_kN Ft2Rd_cf_fma2_kN Ft3Rd_cf_fma3_kN]);

    FtRd_cf_fma_totgov_kN = nc1_up .* FtRd_cf_fma_gov_kN;


%B1. Endplate Bending, extended plate

    %Effective lengths B, extend end plate, NEN-EN 1993-1-8, art. 6.2.6.4.2
    %table (6.5)
    leffcp_b1_mm = 2 .* pi .* m2_ver_p_mm;
    leffcp_b2_mm = pi .* m2_ver_p_mm + w_p_mm;
    leffcp_b3_mm = pi .* m2_ver_p_mm + 2 .* e2_hor_mm;
    leffnc_b4_mm = 4 .* m2_ver_p_mm + 1.25 .* e1_ver_mm;
    leffnc_b5_mm = e2_hor_mm + 2 .* m2_ver_p_mm + 0.625 .* e1_ver_mm
    leffnc_b6_mm = 0.5 .* b_p_mm;
    leffnc_b7_mm = 0.5 .* w_p_mm + 2 .* m2_ver_p_mm + 0.625 .* e1_ver_mm;

    leff_b17_mm = [leffcp_b1_mm leffcp_b2_mm leffcp_b3_mm leffnc_b4_mm leffnc_b5_mm
leffnc_b6_mm leffnc_b7_mm]
    leff_bmin_mm = min([leff_b17_mm])

    %Plastic resistance moment
    WplRd_endp_mm3 = 0.25 .* leff_bmin_mm .* (t_p_mm) .^ 2;
    MplRd_endp_Nmm = WplRd_endp_mm3 .* fy_p_Ndmm2;
```



```
    MplRd_endp_kNm = MplRd_endp_Nmm ./ 1.0E6

    %T-stub resistances B1

    %Distances of the extended part of the end plate in m
    m2_ver_pe_m = m2_ver_p_mm ./ 1000
    ne_ver_pe_m = min([(e1_ver_mm ./ 1000) ((1.25 .* m2_ver_p_mm)./1000)])

    %Failure modes T-stub, B1 extend.
    Ft1Rd_endp_fmb1_kN = (4 .* MplRd_endp_kNm) ./ m2_ver_pe_m;
    Ft2Rd_endp_fmb2_kN = (2 .* MplRd_endp_kNm + 2 .* ne_ver_pe_m .* FtRd_M16_kN) .
( m2_ver_pe_m + ne_ver_pe_m );
    Ft3Rd_endp_fmb3_kN = 2 .* FtRd_M16_kN;

    FtRd_endpe_fmb13_kN = [Ft1Rd_endp_fmb1_kN Ft2Rd_endp_fmb2_kN
Ft3Rd_endp_fmb3_kN]
    FtRd_endpe_fmb_kN = 0.5 .* min(FtRd_endpe_fmb13_kN)    %Only extended plate
part so devided by 2
    FtRd_endpe_fmb_gov_kN = 2.* FtRd_endpe_fmb_kN          %Two bolt rows, so
multiplied by 2

    %B2. Endplate Bending, between the flanges inside

    %Distances of the inside part, between the flanges of the end plate
    m1_hor_pi_mm = 0.5 .* w_p_mm - 0.8 .* aw_w_b_mm .* sqrt(2) - 0.5 .* tw_b_mm
    m2_ver_pi_mm = 0.5 .* p1_ver_mm - 0.8 .* aw_f_b_mm .* sqrt(2) - 0.5 .* tf_b_mm
    e2_hor_pi_mm = e2_hor_mm

    m1_hor_pi_m = m1_hor_pi_mm ./ 1000;
    ne_hor_pi_m = min([ (e2_hor_pi_mm ./ 1000) (1.25 .* (m1_hor_pi_mm ./ 1000)) ])

    %Labda values, NEN-EN 1993-1-8, art. 6.2.6.4.5 figure(6.11)
        %(pi = plate inside the flanges)
    lambda_1 = m1_hor_pi_mm ./ (m1_hor_pi_mm + e2_hor_pi_mm)
    lambda_2 = m2_ver_pi_mm ./ (m1_hor_pi_mm + e2_hor_pi_mm)

    leffcp_bi1_mm = 2 .* pi .* m1_hor_pi_mm
    leffnp_bi2_mm = alpha_lambda .* m1_hor_pi_mm

    leff_bi12_mm = [leffcp_bi1_mm leffnp_bi2_mm]
    leff_bimin_mm = min([leff_bi12_mm])

    %Plastic resistance moment
    MplRd_endpi_Nmm = 0.25 .* leff_bimin_mm .* t_p_mm.^2 .* fy_p_Ndmm2
    MplRd_endpi_kNm = MplRd_endpi_Nmm ./ 1.0E6

    %Failure modes T-stub, B1 inside.
    Ft1Rd_endpi_fmb1_kN = (4 .* MplRd_endpi_kNm) ./ m1_hor_pi_m;
    Ft2Rd_endpi_fmb2_kN = (2 .* MplRd_endpi_kNm + 2 .* ne_hor_pi_m .* FtRd_M16_kN
./ (m1_hor_pi_m + ne_hor_pi_m);
    Ft3Rd_endpi_fmb3_kN = 2 .* FtRd_M16_kN;

    FtRd_endpi_fmb13_kN = [Ft1Rd_endpi_fmb1_kN Ft2Rd_endpi_fmb2_kN
```

```
Ft3Rd_endpi_fmb3_kN]
    FtRd_endpi_fmb_gov_kN = min([FtRd_endpi_fmb13_kN])


%C Columnweb under tension (NOTE: Depending on omega_load, omega_load =
%omega_load_one for beta = 1.

    %Effective width is the same as the effective governing
    %length of the bolt patern in the column flange
    beffw_c_mm = 2 .* leff_amin_mm


    %Determining omega_load, depending on beta trans, NEN-EN 1993-1-8:
    %art. 6.2.6.2
    beffcwc_mm = tf_b_mm + (2 .* sqrt(2) .* aw_f_b_mm) + 5 .* ( tf_c_mm + r_c_mm) +
2 .* t_p_mm
    Av_c_mm2 = A_c_mm2 - (2 .* b_c_mm .* tf_c_mm) + (tw_c_mm + 2 .* r_c_mm) .*
tf_c_mm

    omega_load_one = 1 ./ sqrt( 1 + 1.3 .* ((beffcwc_mm .* tw_c_mm) ./ Av_c_mm2)
^2)
    omega_load = omega_load_one

    FtRdw_c_N = (omega_load .* beffw_c_mm .* tw_c_mm .* fy_c_Ndmm2) ./ gamma_M0;
    FtRdw_c_kN = FtRdw_c_N ./ 1000


%D. Beam web in tension
    beffw_b_mm = leff_bimin_mm
    FtRdw_b_N = (beffw_b_mm .* tw_b_mm .* fy_b_Ndmm2) ./ gamma_M0;
    FtRdw_b_kN = FtRdw_b_N ./ 1000


%E. Bolts in tension -> Already done in T-stub calcualtions

%F. Welds in tension -> Full strenth welds


%G. Shear Resistance
    VwpRd_c_N = (0.9 .* (fy_c_Ndmm2 .* Av_c_mm2) ./ sqrt(3)) ./ gamma_M0;
    VwpRd_c_kN = VwpRd_c_N ./ 1000;

%H. Columnweb under compression

    %Effective width: NEN-EN 1993-1-8, art. 6.2.6.2. eq(6.11)
    beffw_c_ce_mm = tf_b_mm + (2 .* sqrt(2) .* aw_f_b_mm) + 5 .* (tf_c_mm + r_c_mm)
+ 2 .* t_p_mm;

    %Compression resistance: NEN-EN 1993-1-8, art. 6.2.6.2(1) eq(6.9)
    FcRdw_c_N = (omega_load .* kwc .* beffw_c_ce_mm .* tw_c_mm .* fy_c_Ndmm2) ./
gamma_M0;
    FcRdw_c_kN = FcRdw_c_N ./ 1000
```



```
%I. Beamflange under compression

    McRd_b_Nmm = (Wpl_b_mm3 .* fy_b_Ndmm2) ./ gamma_M0;
    McRd_b_kNm = McRd_b_Nmm ./ 1.0E6;

    FcfbRd_N = McRd_b_Nmm ./ (h_b_mm - tf_b_mm);
    FcfbRd_kN = FcfbRd_N ./ 1000


%%Overview resistances

%Tension
    %Column Flange
    TR_A1_kN = FtRd_cf_fma_gov_kN
    TR_A2_kN = FtRd_cf_fma_gov_kN
    TR_Atot_kN = TR_A1_kN + TR_A2_kN

    %End Plate
    TR_B1_kN = FtRd_endpe_fmb_gov_kN
    TR_B2_kN = FtRd_endpi_fmb_gov_kN
    TR_Btot_kN = TR_B1_kN + TR_B2_kN

    %Column web
    TR_C1_kN = FtRdw_c_kN
    TR_Ctot_kN = TR_C1_kN

    %Beam Flange
    TR_D1_kN = FtRdw_b_kN
    TR_Dtot_kN = TR_D1_kN

    %Bolts
    TR_E1_kN = nc1_up .* FtRd_M16_kN
    TR_E2_kN = nc1_up .* FtRd_M16_kN
    TR_Etot_kN = TR_E1_kN + TR_E2_kN

    R_tension_comp_kN = [   TR_A1_kN    TR_A2_kN    0;
                            TR_B1_kN    TR_B2_kN    0;
                            0           TR_C1_kN    0;
                            0           TR_D1_kN    0;
                            TR_E1_kN    TR_E2_kN    0]

    R_tension_tot_kN =  [   0           0       TR_Atot_kN;
                            0           0       TR_Btot_kN;
                            0           0       TR_Ctot_kN;
                            0           0       TR_Dtot_kN;
                            0           0       TR_Etot_kN]

%Shear
    R_shear_tot_kN = [0 ; VwpRd_c_kN; 0]

%Compression
    R_compression_kN =  [   FcRdw_c_kN  FcfbRd_kN   ]
```

```
%%Bar Chart Resistances

    %Bar Chart Tension
    figure
    barh(R_tension_tot_kN,'b')
    hold on
    barh(R_tension_comp_kN,'g')
    grid on
    title('Resistance values in tension zone')
    legend('Total resistance component')
    axis([0 400 0 6])
    xlabel('Resistance Values [kN]')
    ylabel('Components')
    set(gca, 'YTickLabel', {'Column Flange', 'Endplate', 'Column web', 'Beam web',
'Bolts'})

    %Bar Chart Shear zone
    figure
    barh(R_shear_tot_kN)
    grid on
    title('Resistance value in shear zone')
    legend('Manual Calculation EC3')
    axis([0 400 0 4])
    xlabel('Resistance Values [kN]')
    ylabel('Components')
    set(gca, 'YTickLabel', {'','Beam web',''})

    %Bar Chart Compression zone
    figure
    barh(R_compression_kN, 'k')
    grid on
    title('Resistance values in compression zone')
    legend('Manual Calculation EC3')
    axis([0 400 0 3])
    xlabel('Resistance Values [kN]')
    ylabel('Components')
    set(gca, 'YTickLabel', {'Column web', 'Beam Flange'})


%%J. Total moment resistance[MANUALLY INPUT, CHECK THE SHEAR AND THE COMPRESSIO
ZONE
%%!!!]

%Determining the governing resistance value for each zone
FRd_tension_kN = min([TR_Atot_kN TR_Btot_kN TR_Ctot_kN TR_Dtot_kN TR_Etot_kN])
FRd_shear_kN = min([VwpRd_c_kN])
FRd_compression_kN = min([FcRdw_c_kN FcfbRd_kN])

FRd_zone_matrix_kN = [  FRd_tension_kN;
                        FRd_shear_kN;
                        FRd_compression_kN; ]
```



```
    %Bar Chart Resistance for tension, shear and compression zone
    figure
    bar(FRd_zone_matrix_kN)
    grid on
    title('Governing resistance for compression, shear and tension zone')
    legend('Manual Calculation EC3')
    xlabel('Zones')
    ylabel('Resistance value [kN]')
    axis([0 4 0 250])
    set(gca, 'XTickLabel', {'Tension zone', 'Shear zone', 'Compression zone'})


%Determining the moment resistance of the connection  [MANUALLY IMPUT !!!]
x1_max_distance_m = x1_max_distance_mm ./ 1000;
x2_min_distance_m = x2_min_distance_mm ./ 1000;

%NOTE: Command line should be only exectuded if shear or compression is
%governing.

    %Determining if shear or compression is governing
    FRd_shearcomp_gov_kN = min([FRd_shear_kN FRd_compression_kN]);
    FRd_reduction_tension_zone_kN = FRd_tension_kN - FRd_shearcomp_gov_kN;

    FRd_bo1_kN = TR_B1_kN;
    FRd_bo2_kN = TR_B2_kN - FRd_reduction_tension_zone_kN;

    MjRd_total_kNm = FRd_bo1_kN .* x1_max_distance_m + FRd_bo2_kN .
x2_min_distance_m;

    %Assebly of the moment resistance matrix
    MR_manualcomp_kNm =     [0 MjRd_total_kNm   0   0               0   ]
    MR_model_kNm =          [0 0                0   MjRd_FEA_kNm    0   ]


%Bar chart moment resistances, manual calculation and FEA model
figure
bar(MR_manualcomp_kNm)
hold on
bar(MR_model_kNm, 'r')
title('Moment resistances Manual Calculation EC3 and FEA Model')
legend('Manual Calculation EC3', 'FEA Model', 'Location', 'Southwest')
grid on
xlabel('Moment Resistance')
ylabel('Resistance Values [kNm]')
set(gca, 'XTicklabel', {'','Moment Resistance, MC', '', 'Moment Resistance, FEA',
''})
```

B.6 CA6: MRJ, Welded Joint

```
%Clear Everything
clear all
clc
close all

pause(0.3)

%%Calculation Welded Moment Resisting Connection

%%Results Partial FEA model
FEA_modelres_kNm = 300;

%%Assumptions
    % Omega load = omega load one, assymetrical loading
    % For resistance column web in compression, kwc = 1.0

%Note: Check if tension or compression is governing!!!

%%Parameters

    %Load parameters
    kwc = 1.0;

    %Geometry Parameters
    %Beam -> IPE 450
    h_b_mm = 450;
    b_b_mm = 190;
    tw_b_mm = 9.4;
    tf_b_mm = 14.6;
    r_b_mm = 21;
    A_b_mm2 = 9380;
    Wpl_b_mm3 = 1702000;

    fy_b_Ndmm2 = 355;
    fu_b_Ndmm2 = 510;

    %Column -> HEA 320
    h_c_mm = 310;
    b_c_mm = 300;
    tw_c_mm = 9;
    tf_c_mm = 15.5;
    r_c_mm = 27;
    A_c_mm2 = 12440;

    fy_c_Ndmm2 = 355;
    fu_c_Ndmm2 = 510;
    E_c_Ndmm2 = 210000;

    %Welds
    af_b_mm = 9;
    aw_b_mm = 6;

    %Safety factors
```



```
    gamma_M0 = 1.0;

%%Derived Parameters
    z_arm_mm = h_b_mm - tf_b_mm
    z_arm_m = z_arm_mm ./ 1000

%A. Column Flange in tension
beffcf_c_mm = tw_c_mm + 2 .* r_c_mm + 7 .* tf_c_mm;

FcftRd_c_N = fy_c_Ndmm2 .* tf_b_mm .* beffcf_c_mm;
FcftRd_c_kN = FcftRd_c_N ./ 1000


%B. Column web in tension
befftwc_ct_mm = tf_b_mm + 2.* af_b_mm .* sqrt(2) +  5 .* (tf_c_mm + r_c_mm)
Avc_mm2 = A_c_mm2 - 2 .* b_c_mm .* tf_c_mm + (tw_c_mm + 2 .* r_c_mm) .* tf_c_mm

%NEN-EN 1993-1-8, art. 6.2.6.2 table(6.3)
omega_one = 1 ./  sqrt(1 + 1.3 .* ((( befftwc_ct_mm .* tw_c_mm ) ./ Avc_mm2).^2))
omega_load = omega_one

%NEN-EN 1993-1-8, art. 6.2.6.3, eq(6.15)
FcwtRd_c_N = (omega_load .* befftwc_ct_mm .* tw_c_mm .* fy_c_Ndmm2) ./ gamma_M0;
FcwtRd_c_kN = FcwtRd_c_N ./ 1000

%C. Shear resistance
%NEN-EN 1993-1-8, art. 6.2.6.1 eq(6.7)
VwpRd_N = ( 0.9 .* fy_c_Ndmm2 .* ((Avc_mm2) ./ sqrt(3)) ) ./ gamma_M0
VwpRd_kN = VwpRd_N ./ 1000


%D. Columnweb in compression [Manually check !!!]
%NEN-EN 1993-1-8, art. 6.2.6.2 eq(6.12)
befftwc_cc_mm = tf_b_mm + 2 .* af_b_mm .* sqrt(2) + 5 .* (tf_c_mm + r_c_mm);
dwc_c_mm = h_c_mm - 2 .* r_c_mm - 2 .* tf_c_mm;

%NEN-EN 1993-1-8, art. eq(6.13a)
lambda_bar = 0.932 .* sqrt((befftwc_cc_mm .* dwc_c_mm .* fy_c_Ndmm2) ./ (E_c_Ndmm2
.* (tw_c_mm).^2))

if lambda_bar < 0.72
    rho_if = 1.0;
elseif lambda_bar > 0.72
    rho_if = ((lambda_bar - 0.2) ./ (lambda_bar).^2)
end

rho_lambda = rho_if

%NEN-EN  1993-1-8, art. 6.2.6.2, eq(6.9)
FcwcRd_c_N = ( omega_load .* kwc .* rho_lambda .* befftwc_cc_mm .* tw_c_mm .*
fy_c_Ndmm2 ) ./ gamma_M0;
FcwcRd_c_kN = FcwcRd_c_N ./ 1000
```

```
%E. Beamweb in compression
McRd_b_Nmm = ( Wpl_b_mm3 .* fy_b_Ndmm2) ./ gamma_M0;
McRd_b_kNm = McRd_b_Nmm ./ 1.0E6;

%NEN-EN 1993-1-8, art. 6.2.6.7 (eq. 6.21)
FcfbRd_b_N = McRd_b_Nmm ./ (h_b_mm - tf_b_mm)
FcfbRd_b_kN = FcfbRd_b_N ./ 1000


%Resistances

    %Resistance matrices
    R_tension_matrix_kN = [ FcftRd_c_kN; FcwtRd_c_kN ]
    R_shear_matrix_kN = [ VwpRd_kN ]
    R_compression_matrix_kN = [ FcwcRd_c_kN; FcfbRd_b_kN ]

    %Governing restances
    FRd_tension_kN = min( [ FcftRd_c_kN; FcwtRd_c_kN ] )
    VwpRd_kN
    FRd_compression_kN = min( [FcwcRd_c_kN; FcfbRd_b_kN] )

    FRd_matrix_kN = [ FRd_tension_kN   VwpRd_kN   FRd_compression_kN ]

%Bar Chart Resistances

    %Overview resistances: tension, shear and compression zone

    R_resistance_overview = [ FcftRd_c_kN;  FcwtRd_c_kN; 0; VwpRd_kN; 0↙
FcwcRd_c_kN; FcfbRd_b_kN]
    R_resistance_overview_mirror = [ FcfbRd_b_kN; FcwcRd_c_kN; 0; VwpRd_kN; 0↙
FcwtRd_c_kN; FcftRd_c_kN ]
    barh(R_resistance_overview_mirror)
    grid on
    xlabel('Resistance Value [kN]')
    ylabel('Component')
    title('Overview Resistance Values')
    axis([ 0 1500 0 8])
    set(gca,  'YTickLabel',  {'Beam web in compression', 'Columnweb in↙
compression', '', 'Column web in shear', '', 'Column web in tension', 'Column↙
flange in bending'})
    legend('Manual Calculation EC3')

    %Overview governing resistances for each zone
    figure
    bar(FRd_matrix_kN)
    axis([ 0 4 0 800])
    xlabel('Zones')
    ylabel('Resistance Value [kN]')
    grid on
    title('Overall resistance for each zone')
    set(gca, 'XTickLabel', {'Tension Zone', 'Shear Zone', 'Compression Zone'})
    legend('Manual Calculation EC3')
```



```
%Total moment resistance
FRdgov_moment_resistance_kN = min([ FRd_tension_kN   VwpRd_kN↙
FRd_compression_kN])
MjRd_total_kNm = FRdgov_moment_resistance_kN .* z_arm_m;

    %Matrix Moment Resistance
    MR_manualcal_kNm  = [   MjRd_total_kNm  0   0                    ]
    MR_partialFEA_kNm = [   0               0   FEA_modelres_kNm     ]


    %Bar chart overview total moment resistances
    figure
    bar(MR_manualcal_kNm)
    hold on
    bar(MR_partialFEA_kNm, 'r')
    axis([ 0 4 0 400])
    ylabel('Moment Resistance Value [kNm]')
    xlabel('Total Moment Resistances')
    grid on
    title('Total Moment Resistance Manual Calculation and FEA Model)
    set(gca, 'XTicklabel', {'Moment Resistance, MC','', 'Moment Resistance, FEA'})
    legend('Manual Calculation, EC3', 'FEA model')
```

B.7 CA11-EXP: MRJ, Flush End Plate Joint

B.8 CA11-EC3: MRJ, Flush End Plate Joint

Same MATLAB scriptfile as CA11-EXP but with other the following yield strength ,tensile strength and partial factors:

Table B.1: Values for manual calculation

Column	yield strength	$f_{y,c}$	$275N/mm^2$
	tension strength	$f_{u,c}$	$430N/mm^2$
Beam	yield strength	$f_{y,b}$	$275N/mm^2$
	tension strength	$f_{u,b}$	$430N/mm^2$
End Plate	yield strength	$f_{y,ep}$	$275N/mm^2$
	tension strength	$f_{y,ep}$	$430N/mm^2$
	E-modulus	E_{ep}	$21000N/mm^2$
Partial Factors	resistance cross-sections	γ_{M0}	1.0
	resistance on stability	γ_{M1}	1.0
	resistance cross-section in tension till rupture	γ_{M2}	1.25

B.9 CA12-EXP: MRJ, Extended End Plate Joint

```
%Clear Everything
clear all
clc
close all

pause(0.3)

%%Calculation Valdiation Momement Resisting Connection, Extended End Plate
    % - Script designed to calculate two bolt columns and two bolt rows
    % - Script designed to calculate to calculate the resistance against a
    % unsymmetrical load
    % - M20 bolts, quality 10.9
    % - Only the upper bolts contribute to the resistance against an
    % unsymmetrical load. (2x2)
    % - System length (lsystem) is estimated from drawing where from the
    % shear force is derived.

%%Load parameters
    MEd_kNm =   40;         %Load applied [kNm]
    lsys_m =    1.150;      %System length [m]

%%Alpha_lambda factor: NEN-EN 1993-1-8, art. 6.2.6.5 figure (3.11)
    %[Manually Input Depening on lambda1 and lambda2 !!!]
    alpha_lambda = 5.8;     %Alpha factor [-]

%%Results partial FEA model
    MRd_FEA_kNm = 125;      %Resistance value of the FEA model [kNm]

%%Results Experiments
    MRd_EXP_kNm = 125;      %Experimental bending moment at 50mrad [kNm]

%Load parameters
%NEN-EN 1993-1-8, art. 5.3, table(5.4)
beta_trans = 1;             %Transformation parameter
%NEN-EN 1993-1-8, art. 6.2.6.2(2), note
kwc = 1.0;                  %Reduction factor

%Profile Beam -> IPE240 (S275), properties by test results
h_b_mm = 240;               %Height of the beam [mm]
b_b_mm = 120;               %Width of the beam [mm]
tw_b_mm = 6.2;              %Web thickness of the beam [mm]
tf_b_mm = 9.8;              %Flange thickness of the beam [mm]
r_b_mm = 15;                %Radius of the beam [mm]
A_b_mm2 = 3910;             %Cross sectional area of the beam [mm2]
Wpl_b_mm3 = 367000;         %Sectional are of the beam [mm3]

fy_b_expv_Ndmm2 = 350;      %Experimental value, yield strength of the beam [N/mm2]
fu_b_expv_Ndmm2 = 450;      %Experimental value, tensile strength of the beam
[N/mm2]

%Profile Colmn -> HEB240 (S275), properties by test results
h_c_mm = 240;               %Height of the column [mm]
b_c_mm = 240;               %Width fo the column [mm]
```



```
tw_c_mm = 10;               %Web thickess of the column [mm]
tf_c_mm = 17;               %Flange thickness of the column [mm]
r_c_mm = 21;                %Radius of the column [mm]
A_c_mm = 10600;             %Cross sectional area of the column [mm]

e22_c_mm = 72;              %end distance (horizontal) of the column [mm]

fy_c_expv_Ndmm2 = 360;      %Experimental value, Yield strength of the column
[N/mm2]
fu_c_expv_Ndmm2 = 460;      %Experimental value, Tensile strength of the column
[N/mm2]
E_c_expv_Ndmm2 = 210000;    %Experimental value, E-modulus of the column [N/mm2]

%Extended End Plate (S275), properties by test results
b_p_mm = 160;           %Width of the end plate [mm]
t_p_mm = 15;            %Thickness of the end plate [mm]

p_mm = 74;              %Vertical pitch of the end plate [mm]
w_mm = 96;              %Horizontal pitch of the end plate [mm]

e1_ver_p_mm = 30;       %End distance (vertical) of the end plate [mm]
e2_hor_p_mm = 32;       %End distance (horizontal) of the end plate [mm]

distance1_p_mm = 62;    %Distances in end plate [mm]
distance2_p_mm = 104;   %Distances in end plate [mm]

fy_p_expv_Ndmm2 = 370;  %Yield strength of the end plate [N/mm2]
fu_p_expv_Ndmm2 = 500;  %Tensile strength of end plate [N/mm2]


%Bolts 2x2 M20, quality 10.9
fub_bo_Ndmm2 = 1000;    %Tensile strength of the bolts [N/mm2]
d_bo_mm = 20;           %Diameter of the bolt [mm]
d0_bo_mm = 22;          %Bolt hole diameter [mm]
k2_bo = 0.9;            %k2 factor [-]
As_bo_mm2 = 245;        %Cross sectional area of the bolt [mm2]

r_out_mm = 257.1        %Distance outside bolt - compression point [mm]
r_in_mm = 193.1         %Distance inside bolt - compression point [mm]

%Welds
a_weld_mm = 6;          %Troath thickness of the weld [mm]

%Safety factors NEN-EN 1993-1-8, art. 6.1(1)
gamma_M0 = 1.0;             %Safety factor, not applied
gamma_M1 = 1.0;             %Safety factor, not applied
gamma_M2 = 1.0;             %Safety factor, not applied


%%Derived parameters

    %Tension ristance bolts M20, steel quality 10.9
    %NEN-EN 1993-1-8, art. 3.6.1, table(3.4)
```

```
    FtRd_M20_q109_N = (k2_bo .* fub_bo_Ndmm2 .* As_bo_mm2) ./ gamma_M2;
    FtRd_M20_q109_kN = FtRd_M20_q109_N ./ 1000;

    %M-distance of the column flange
    m22_hor_c_mm = 0.5 .* w_mm - 0.5 .* tw_c_mm - 0.8 .* r_c_mm;
    m22_hor_c_m = m22_hor_c_mm ./ 1000

    %M-distance of the extended endplate
    %M-Outside
    m1_out_ver_p_mm = distance1_p_mm - e1_ver_p_mm - 0.8 .* sqrt(2) .* a_weld_mm
    m1_out_ver_p_m = m1_out_ver_p_mm ./ 1000;

    %M-Inside
    m1_in_ver_p_mm = distance2_p_mm - distance1_p_mm - tf_b_mm - 0.8 .* sqrt(2) .↙
a_weld_mm
    m1_in_ver_p_m = m1_in_ver_p_mm ./ 1000;

    m2_in_hor_p_mm = 0.5 .* w_mm - 0.5 .* tw_b_mm - 0.8 .* a_weld_mm .* sqrt(2);
    m2_in_hor_p_m = m2_in_hor_p_mm ./ 1000;


%%Components

%A. Column flange in bending

    %Determining effective lengths
    leffcp_cfb1_c_mm = 2 .* pi .* m22_hor_c_mm
    leffcp_cfb2_c_mm = pi .* m22_hor_c_mm + p_mm
    leffnc_cfb3_c_mm = 4 .* m22_hor_c_mm + 1.25 .* e22_c_mm
    leffnc_cfb4_c_mm = 2 .* m22_hor_c_mm + 0.625 .* e22_c_mm + 0.5 .* p_mm

    leffgov_cfb_c_mm = min([leffcp_cfb1_c_mm leffcp_cfb2_c_mm leffnc_cfb3_c_m↙
leffnc_cfb4_c_mm])


    %Determing ne for T-stub resistances
    e22_c_mm
    e22_limit_cfb_c_mm = 1.25 .* m22_hor_c_mm
    ne_cfb_c_mm = min([e22_c_mm e22_limit_cfb_c_mm])
    ne_cfb_c_m = ne_cfb_c_mm ./ 1000;

    %Determing Plastic Moment Resistance
    MplRd_c_Nmm = 0.25 .* leffgov_cfb_c_mm .* (tf_c_mm).^2 .* fy_c_expv_Ndmm2
    MplRd_c_kNm = MplRd_c_Nmm ./ 1000000

    %T-Stub Resistances Column Flange in Bending
    %NEN-EN 1993-1-8, art. 6.2.4.1, table(6.2)
    Ft1Rd_cfb_c_kN = (4 .* MplRd_c_kNm) ./ m22_hor_c_m
    Ft2Rd_cfb_c_kN = (2 .* MplRd_c_kNm + 2 .* ne_cfb_c_m .* FtRd_M20_q109_kN) ./  ↙
m22_hor_c_m + ne_cfb_c_m)
    Ft3Rd_cfb_c_kN = 2 .* FtRd_M20_q109_kN

    FtRdgov_cfb_c_kN = min([Ft1Rd_cfb_c_kN Ft2Rd_cfb_c_kN Ft3Rd_cfb_c_kN])
```



```
    FtRdgov_tot_cfb_c_kN = 2 .* FtRdgov_cfb_c_kN

%B. End Plate in Bending

    %B1 End Plate Outside Part

    %Determining the effective lengths
    leffcp_epb1_out_p_mm = 2 .* pi .* m1_out_ver_p_mm;
    leffcp_epb2_out_p_mm = pi .* m1_out_ver_p_mm + w_mm;
    leffcp_ebp3_out_p_mm = pi .* m1_out_ver_p_mm + 2 .* e2_hor_p_mm
    leffnc_ebp4_out_p_mm = 4 .* m1_out_ver_p_mm + 1.25 .* e1_ver_p_mm
    leffnc_ebp5_out_p_mm = e2_hor_p_mm + 2 .* m1_out_ver_p_mm + 0.625 .↙
e1_ver_p_mm;
    leffnc_ebp6_out_p_mm = 0.5 .* b_p_mm
    leffnc_ebp7_out_p_mm = 0.5 .* w_mm + 2.* m1_out_ver_p_mm + 0.625 .↙
e1_ver_p_mm;

    leff_epb_matrix_p_mm = [    leffcp_epb1_out_p_mm;
                                leffcp_epb2_out_p_mm;
                                leffcp_ebp3_out_p_mm;
                                leffnc_ebp4_out_p_mm;
                                leffnc_ebp5_out_p_mm;
                                leffnc_ebp6_out_p_mm;
                                leffnc_ebp7_out_p_mm]

    leff_epb_p_mm = min([leff_epb_matrix_p_mm])

    %Determing Plastic Moment Resistance
    MplRd_out_p_Nmm = 0.25 .* leff_epb_p_mm .* (t_p_mm).^2 .* fy_p_expv_Ndmm2
    MplRd_out_p_kNm = MplRd_out_p_Nmm ./ 1000000

    %Determing ne (outside) for T-stub resistances
    e1_ver_p_mm;
    e1_ver_limit_epb_p_mm = 1.25 .* m1_out_ver_p_mm;
    ne_epb_out_p_mm = min( [e1_ver_p_mm e1_ver_limit_epb_p_mm])
    ne_epb_out_p_m = ne_epb_out_p_mm ./ 1000;

    %T-Stub Resistances End plate in bending
    %NEN-EN 1993-1-8, art. 6.2.4.1, table(6.2)
    FT1Rd_epb_out_kN = (4 .* MplRd_out_p_kNm) ./ m1_out_ver_p_m
    FT2Rd_epb_out_kN = (2 .* MplRd_out_p_kNm + 2 .* ne_epb_out_p_m .↙
FtRd_M20_q109_kN) ./ ( m1_out_ver_p_m + ne_epb_out_p_m )
    FT3Rd_epb_out_kN = 2 .* FtRd_M20_q109_kN

    FTRdgov_epb_out_kN = min([FT1Rd_epb_out_kN FT2Rd_epb_out_kN FT3Rd_epb_out_kN])
    FTRdgov_ht_epb_out_kN = (FTRdgov_epb_out_kN ./ 2)      %Half T-stub

    FTRdgov_tot_epb_out_kN = 2 .* FTRdgov_ht_epb_out_kN


    %B2 End Plate Inside Part
```

```
    %Determining lambda factors
    lambda1 = m2_in_hor_p_mm ./ ( m2_in_hor_p_mm + e2_hor_p_mm )
    lambda2 = m1_in_ver_p_mm ./ ( m2_in_hor_p_mm + e2_hor_p_mm )

    %Effective lengths
    leffcp_epb1_in_mm = 2 .* pi .* m2_in_hor_p_mm;
    leffcm_ebp2_in_mm = alpha_lambda .* m2_in_hor_p_mm;

    leffgov_epb_in_mm = min([leffcp_epb1_in_mm leffcm_ebp2_in_mm])

    %Determing Plastic Moment Resistance
    MplRd_in_p_Nmm = 0.25 .* leffgov_epb_in_mm .* (t_p_mm).^2 .* fy_p_expv_Ndmm2;
    MplRd_in_p_kNm = MplRd_in_p_Nmm ./ 1000000

    %Determing ne (inside) for T-stub resistance
    e2_hor_p_mm
    e2_hor_limit_p_mm = 1.25 .* m2_in_hor_p_mm;
    ne_epb_in_mm = min([e2_hor_p_mm e2_hor_limit_p_mm])
    ne_epb_in_m = ne_epb_in_mm ./ 1000

    %Determing T-stub resistance
    %NEN-EN 1993-1-8, art. 6.2.4.1, table(6.2)
    FT1Rd_epb_in_kN = (4 .* MplRd_in_p_kNm ) ./ m2_in_hor_p_m
    FT2Rd_epb_in_kN = (2 .* MplRd_in_p_kNm + 2 .* ne_epb_in_m .* FtRd_M20_q109_kN
./ (m2_in_hor_p_m + ne_epb_in_m)
    FT3Rd_epb_in_kN = (2 .* FtRd_M20_q109_kN)

    FTRdgov_tot_epb_in_kN = min([FT1Rd_epb_in_kN FT2Rd_epb_in_kN FT3Rd_epb_in_kN])


%C Column web in Tension

    %Effective length
    beff_cwt_mm = 2 .* leffgov_cfb_c_mm

    %Omega factor
    beff_column_c_mm = tf_b_mm + 2 .* sqrt(2) .* a_weld_mm + 5 .* (tf_c_mm
r_c_mm) + 2 .* t_p_mm
    Avc_c_mm2 = A_c_mm - (2 .* b_c_mm .* tf_c_mm) + (tw_c_mm + 2 .* r_c_mm) .
tf_c_mm
    omega_load = 1 ./ sqrt( 1 + 1.3 .* ((beff_column_c_mm .* tw_c_mm) .
(Avc_c_mm2)).^2)

    %Determining resistance, NEN-EN 1993-1-8, art. 6.2.6.3, eq(6.15)
    FtwcRdgov_cwt_N = (omega_load .* beff_cwt_mm .* tw_c_mm .* fy_c_expv_Ndmm2) .
gamma_M0
    FtwcRdbov_cwt_kN = FtwcRdgov_cwt_N ./ 1000


%D Beam web in Tension

    %Effective width
    beff_bwt_b_mm = leffcm_ebp2_in_mm
```



```
    %Determining resistance, NEN-EN 1993-1-8, art. 6.2.6.8 eq(6.2)
    FtwbRdgov_bwt_c_N = (beff_bwt_b_mm .* tw_b_mm .* fy_c_expv_Ndmm2) ./ gamma_M0
    FtwbRdgov_bwt_c_kN = FtwbRdgov_bwt_c_N ./ 1000


%E Bolts in Tension

    %Determining bolt resistance
    FtRd_boltrow_kN = 2 .* FtRd_M20_q109_kN
    FtRdgov_tot_bo_kN = 2 .* FtRd_boltrow_kN

%F Columnweb in Shear

    %Determmining the shear resistance
    %NEN-EN 1993-1-8, art. 6.2.6.1, eq(6.7)
    VwpRdgov_c_N = 0.9 .* (Avc_c_mm2 ./ sqrt(3)) .* fy_c_expv_Ndmm2 ./ gamma_M0
    VwpRdgov_c_kN = VwpRdgov_c_N ./ 1000;

%G Column web in compression

    %Effective width
    beff_cwc_c_mm = tf_b_mm + 2 .* sqrt(2) .* a_weld_mm + 5 .* (tf_c_mm + r_c_mm)
2 .* t_p_mm

    %Determining resistance without buckling factor
    %NEN-EN 1993-1-8, art. 6.2.6.2, eq(6.10)
    FcwcRd_nonbuc_c_kN = omega_load .* kwc .* beff_cwc_c_mm .* tw_c_mm .
fy_c_expv_Ndmm2 ./ gamma_M0

    %Determining buckling factor
    dwc_mm = h_c_mm - 2 .* (tf_c_mm + r_c_mm)

    %NEN-EN 1993-1-8, art. 6.2.6.2, eq(6.13)
    lambda_bar = 0.932 .* sqrt( (beff_cwc_c_mm .* dwc_mm .* fy_c_expv_Ndmm2) .
(E_c_expv_Ndmm2 .* (tw_c_mm).^2))

    %If-statement for buckling factor
        if lambda_bar < 0.72
            rho_red = 1.0;
        elseif lambda_bar > 0.72
            rho_red = (lambda_bar - 0.2) ./ (lambda_bar.^2);
        else
            rho_red = 0;
        end

    %Determining resistance with buckling factor
    FcwcRd_N = (omega_load .* kwc .* rho_red .* beff_cwc_c_mm .* tw_c_mm .
fy_c_expv_Ndmm2) ./ gamma_M1
    FcwcRd_buc_c_kN = FcwcRd_N ./ 1000;

    %Determing the governing risitance for column flange under bending
    FcwcRdgov_c_kN = min([FcwcRd_nonbuc_c_kN FcwcRd_buc_c_kN])
```

```
%H Beam flange and beam web under compression
McRd_b_Nmm = (Wpl_b_mm3 .* fy_b_expv_Ndmm2) ./ gamma_M0;
McRd_b_kNm = McRd_b_Nmm ./ 1000000

%NEN-EN 1993-1-8, art. 6.2.6.7 (eq. 6.21)
FcfbRd_b_N = McRd_b_Nmm ./ (h_b_mm - tf_b_mm);
FcfbRd_b_kN = FcfbRd_b_N ./ 1000


%Tension resistances

%Overview of the tension resistances
FRd_tension_one_column_flange = FtRdgov_tot_cfb_c_kN
FRd_tension_two_end_plate = FTRdgov_tot_epb_out_kN + FTRdgov_tot_epb_in_kN
FRd_tension_three_column_web = FtwcRdbov_cwt_kN
FRd_tension_four_beam_web = FtwbRdgov_bwt_c_kN
FRd_tension_five_bolts = FtRdgov_tot_bo_kN

    %Tension Matrix Total
    R_tension_kN =          [   FRd_tension_one_column_flange;
                                FRd_tension_two_end_plate;
                                FRd_tension_three_column_web;
                                FRd_tension_four_beam_web;
                                FRd_tension_five_bolts]

    %Tension zone Matrix
    R_tension_total_kN =     [          0        0
FRd_tension_one_column_flange;
                                        0        0       FRd_tension_two_end_plate;
                                        0        0
FRd_tension_three_column_web;
                                        0        0       FRd_tension_four_beam_web;
                                        0        0       FRd_tension_five_bolts]

    R_tension_separate_kN  =       [    0
0;
                                        FTRdgov_tot_epb_in_kN
FTRdgov_tot_epb_out_kN      0;
                                        0
0;
                                        0
0;
                                        FtRd_boltrow_kN              FtRd_boltrow_kN
0]

    %Figure in tension zone
    figure
    barh(R_tension_total_kN, 'FaceColor', [0 153/256 51/256])
    hold on
    barh(R_tension_separate_kN, 'FaceColor', [0 51/256 153/256])
    grid on
    set(gca, 'YTicklabel', {'Column flange in bending', 'Endplate in bending',
```



```
'Column web in tension', 'Beam web in tension', 'Bolts in tension'})
    ylabel('Component in Tension')
    xlabel('Resistance Value [kN]')
    legend('Resistance Component', 'location', 'southeast')


%Overview of the shear resistance
FRd_shear_one_column_web = VwpRdgov_c_kN

    %Shear Matrix
    R_shear_kN =     [   0;
                         VwpRdgov_c_kN;
                         0]



%Overview of the compression resistance
FRd_compression_one_column_web = FcwcRdgov_c_kN
FRd_compresion_one_beam_flange = FcfbRd_b_kN

    %Compression Matrix
    R_compression_kN = [    FRd_compression_one_column_web;
                            FRd_compresion_one_beam_flange]

%Total moment resistance
R_overall_kN = [        R_compression_kN;
                        R_shear_kN;
                        R_tension_kN]

%Overall Bar Chart
figure
barh(R_overall_kN)
xlabel('Resistance value [kN]')
ylabel('Components in tension, shear and compression')
grid on
set(gca, 'YTickLabel', {'Column web in compression', 'Beam flange in compression',
'', 'Column web in shear','', 'Column flange in bending', 'Endplate in bending',
'Column web in tension', 'Beamweb in tension', 'Bolts in tension'})
title('Resistances Components')
legend('Manual Calculation, EC3')

%Governing resistance for each zone
figure
R_overallgov_kN =   [       min(R_tension_kN) ;
                            R_shear_kN(2)
                            min(R_compression_kN);]

bar(R_overallgov_kN)
xlabel('Governing resistances in tension, shear and compression zone')
ylabel('Resistance value [kN]')
grid on
set(gca, 'XTicklabel', {'Tension Zone', 'Shear Zone', 'Compression Zone'})
legend('Manual Calculation, EC3')
```

```
%Determine the moment resistance [Manually Input !!!]
Dif_tension_kN = FRd_tension_two_end_plate - min(R_tension_kN)

FTRd_reduced_tot_epb_out_kN = FTRdgov_tot_epb_out_kN - (Dif_tension_kN ./ 2)
FTRd_reduced_tot_epb_in_kN = FTRdgov_tot_epb_in_kN - (Dif_tension_kN ./ 2)

z_out_m = r_out_mm ./ 1000;
z_in_m = r_in_mm ./ 1000;

MRdj_out_kNm = FTRd_reduced_tot_epb_out_kN .* z_out_m
MRdj_in_kNm = FTRd_reduced_tot_epb_in_kN .* z_in_m

MRdj_tot_kNm = MRdj_out_kNm + MRdj_in_kNm

    %Matrix Moment Resistances
    MRd_matrix_MC_kNm =     [   MRdj_tot_kNm        0               0];

    MRd_matrix_FEA_kNm =    [   0                   MRd_FEA_kNm     0];

    MRd_matrix_EXP_kNm =    [   0                   0               MRd_EXP_kNm];


    %Bar Chart moment resistances, manual calculation, fea model and
    %experiment
    figure
    bar(MRd_matrix_MC_kNm)
    hold on
    bar(MRd_matrix_FEA_kNm, 'r')
    hold on
    bar(MRd_matrix_EXP_kNm, 'g')
    grid on
    title('Overview Moment Resistances, Experimental Values')
    legend('Manual Calculation, EC3', 'FEA Model', 'Experimental Value',
'Location', 'Southeast')
    xlabel('Experiment / Calculation Procedure')
    ylabel('Moment Resistance Value [kNm]')
    set(gca, 'XTicklabel', {'Manual Calculation', 'FEA model', 'Experiment'})
```

B.10 CA12-EC3: MRJ, Extended End Plate Joint

Same MATLAB scriptfile as CA11-EXP but with other the following yield strength ,tensile strength and partial factors:

Table B.2: Values for manual calculation

Column	yield strength	$f_{y,c}$	$275 N/mm^2$
	tension strength	$f_{u,c}$	$430 N/mm^2$
Beam	yield strength	$f_{y,b}$	$275 N/mm^2$
	tension strength	$f_{u,b}$	$430 N/mm^2$
End Plate	yield strength	$f_{y,ep}$	$275 N/mm^2$
	tension strength	$f_{y,ep}$	$430 N/mm^2$
	E-modulus	E_{ep}	$21000 N/mm^2$
Partial Factors	resistance cross-sections	γ_{M0}	1.0
	resistance on stability	γ_{M1}	1.0
	resistance cross-section in tension till rupture	γ_{M2}	1.25

B.11 PS1: Fin Plate Joint

MATLAB script is similar to script of CA1.

B.12 PS2: Short End Plate Joint

MATLAB script is similar to the script of CA2.

B.13 PS3a: Moment Resisting Joint, Flush End Plate

```
%Clear Everything
clear all
clc
close all

pause(0.3)

%%Calculation Validation Moment Resisting Connection, Flush End Plate

%% Assumptions
    % - Script designed to calculate two bolt rows
    % - Script designed to calculate the resistance for symmetrical load
    % load.
    % - Bolt quality 8.8
    % - Only upper bolts contribute to tension resistance
    % - system length (lsystem) is estimated from drawing where from the
    % shear force is derived.


%loop statement!

for n = [1:1:4]
     if n == 1
%% Load parameters
    MEd_kNm = 40;           %Applied load [kNm]
    lsys_m = 1.5;           %System length [m]

%%Alpha_lambda_factor: NEN-EN 1993-1-8, art. 6.2.6.5 figure (3.11)
    alpha_lambda =5.8;        %Alpha factor [-]

%%Parameters

%Load parameters
%NEN-EN 1993-1-8, art. 5.3, table(5.4)
beta_trans = 0          %Transformation parameter
omega_load_one = 1      %Reductionfactor
%NEN-EN 1993-1-8, art. 6.2.6.2(2), note
kwc = 1.0;              %Reduction factor kwc,

%Profile Beam -> IPE220 (S235), properties by test results
h_b_mm = 220;           %Height of the beam [mm]
b_b_mm = 110;           %Width of the beam [mm]
tw_b_mm = 5.9;            %Thickness of the beam web [mm]
tf_b_mm = 9.2;           %Thickness of the beam flange [mm]
r_b_mm = 12;            %Radius of the beam [mm]
A_b_mm2 = 3340;         %Cross sectional area of the beam [mm2]
Wpl_mm3 = 285000;       %Sectional area of the beam [mm3]

fy_b_test_Ndmm2 = 235;  %Yield strength of the beam [N/mm2]
fu_b_test_Ndmm2 = 360;  %Tensile strength of the beam [N/mm2]
E_b_test_Ndmm2 = 210000;%E-modulus of the beam [N/mm2]

%Profile Column -> HEB160 (S235), properties by test results
```



```
h_c_mm = 160;               %Height of the column [mm]
b_c_mm = 160;               %Width of the column [mm]
tw_c_mm = 8;                %Thickness of the column web [mm]
tf_c_mm = 13;               %Thickness of the column flange [mm]
r_c_mm = 15;                %Radius of the column [mm]
A_c_mm2 = 5430;             %Cross sectional area of the column [mm2]

fy_c_test_Ndmm2 = 235;      %Yield strength of the column [N/mm2]
fu_c_test_Ndmm2 = 360;      %Tensile strength of the column [N/mm2]
E_c_test_Ndmm2 = 210000;    %E-modulus of the beam [N/mm2]

e_c_mm = 40;                %End distance (horizontal) of the column [mm]

%End Plate (S235), properties by test results
t_p_mm = 10;            %Plate thickness of the end plate [mm]

e1_p_mm = 59.6;         %Vertical end distance of the endplate [mm]
e2_p_mm = 30;           %Horizontal end distance of the endplate [mm]

w_mm = 80;              %Horizontal pitch in the end plate [mm]

fy_p_test_Ndmm2 = 235;      %Yield strength of the end plate [N/mm2]
fu_p_test_Ndmm2 = 360;      %Tensile strength of the end plate [N/mm2]
E_p_test_Ndmm2 = 210000;    %E-modulus of the end plate [N/mm2]

distance1_bolt_mm = 35;     %Distance geometry, bolt position [mm]


distance5_mm = 175.8;      %Distance leverage arm, compression point - bolt [mm]

%Bolts 4x M12, quality 8.8
fub_bo_Ndmm2 = 800;         %Tensile strength bolt [N/mm2]
d_bo_mm = 12;               %Diameter of the bolt [mm]
d0_bo_mm = 14;              %Bolt hole diameter [mm]
k2_bo = 0.9;                %k2 factor [-]
As_bo_mm2 = 84;             %Cross-sectional area bolt [mm2]

%Welds
a_weld_mm = 5;              %Troath thickenss weld [mm]

%Safety factors NEN-EN 1993-1-1, art. 6.1(1)
gamma_M0 = 1.0;             %Safety factor, not applied
gamma_M1 = 1.0;             %Safety factor, not applied
gamma_M2 = 1.25;            %Safety factor, not applied


    elseif n == 2
        %Bolts 4x M12, quality 8.8
        fub_bo_Ndmm2 = 800;         %Tensile strength bolt [N/mm2]
        d_bo_mm = 20;               %Diameter of the bolt [mm]
        d0_bo_mm = 22;              %Bolt hole diameter [mm]
        k2_bo = 0.9;                %k2 factor [-]
        As_bo_mm2 = 353;
```

```
    elseif n == 3
        %Bolts 4x M20, quality 8.8
        fub_bo_Ndmm2 = 800;         %Tensile strength bolt [N/mm2]
        d_bo_mm = 20;               %Diameter of the bolt [mm]
        d0_bo_mm = 22;              %Bolt hole diameter [mm]
        k2_bo = 0.9;                %k2 factor [-]
        As_bo_mm2 = 353;

        %End Plate (S235), properties by test results
        t_p_mm = 15;            %Plate thickness of the end plate [mm]
     elseif n == 4
        %Bolts 4x M20, quality 8.8
        fub_bo_Ndmm2 = 800;         %Tensile strength bolt [N/mm2]
        d_bo_mm = 20;               %Diameter of the bolt [mm]
        d0_bo_mm = 22;              %Bolt hole diameter [mm]
        k2_bo = 0.9;                %k2 factor [-]
        As_bo_mm2 = 353;

        %End Plate (S235), properties by test results
        t_p_mm = 15;            %Plate thickness of the end plate [mm]

        %Profile Column -> HEM160 (S235), properties by test results
        h_c_mm = 180;               %Height of the column [mm]
        b_c_mm = 166;               %Width of the column [mm]
        tw_c_mm = 14;                %Thickness of the column web [mm]
        tf_c_mm = 23;               %Thickness of the column flange [mm]
        r_c_mm = 15;                %Radius of the column [mm]
        A_c_mm2 = 9710;             %Cross sectional area of the column [mm2]

        fy_c_test_Ndmm2 = 235;      %Yield strength of the column [N/mm2]
        fu_c_test_Ndmm2 = 360;      %Tensile strength of the column [N/mm2]
        E_c_test_Ndmm2 = 210000;    %E-modulus of the beam [N/mm2]

        e_c_mm = 43;                %End distance (horizontal) of the column [mm]
    end

%%Derived parameters

    %Tension resistance bolts M12, steelquality 8.8
    %NEN-EN 1993-1-8, art. 3.6.1, table(3.4)
    FtRd_bolt_q88_N = ( k2_bo .* fub_bo_Ndmm2 .* As_bo_mm2 ) ./ gamma_M2;
    FtRd_bolt_q88_kN = FtRd_bolt_q88_N ./ 1000;

    %M-distance of the column flange
    m22_hor_mm = 0.5 .* w_mm - 0.5 .* tw_c_mm - 0.8 .* r_c_mm;
    m22_hor_m = m22_hor_mm ./ 1000;

    %M-distance of the beam web
    m1_ver_mm = distance1_bolt_mm - 0.5 * tf_b_mm - 0.8 .* a_weld_mm .* sqrt(2);
    m1_ver_m = m1_ver_mm ./ 1000;
```



```
    m2_hor_mm = 0.5 .* w_mm - 0.5 .* tw_b_mm - 0.8 .* a_weld_mm .* sqrt(2);
    m2_hor_m = m2_hor_mm ./ 1000;

    %Leverage arm upperbolt
    zmax_mm = distance5_mm - 0.5 .* tf_b_mm;
    zmax_m = zmax_mm ./ 1000;

    %Shear force derived from system length
    VEd_kN = MEd_kNm ./ lsys_m


%A. Column flange in bending
    leffcp_c_mm = 2 .* pi .* m22_hor_mm;
    leffncp_c_mm = 4 .* m22_hor_mm + 1.25 .* e_c_mm;

    leff_c_mm = min([leffcp_c_mm leffncp_c_mm])

    ne1_c_mm = e_c_mm;
    ne2_c_mm = 1.25 .* m22_hor_mm;
    ne_c_mm = min([ne1_c_mm ne2_c_mm])
    ne_c_m = ne_c_mm ./ 1000;

    MplRd_c_Nmm = 0.25 .* leff_c_mm .* tf_c_mm.^2 .* fy_c_test_Ndmm2
    MplRd_c_kNm = MplRd_c_Nmm ./ 1000000

    FT1Rd_c_kN = ( 4 .* MplRd_c_kNm ) ./ m22_hor_m
    FT2Rd_c_kN = ( 2 .* MplRd_c_kNm + 2 .* ne_c_m .* FtRd_bolt_q88_kN ) ./ ↙
m22_hor_m + ne_c_m)
    FT3Rd_c_kN = 2 .* FtRd_bolt_q88_kN

    FTRdgov_c_kN = min([FT1Rd_c_kN FT2Rd_c_kN FT3Rd_c_kN])


%B. Endplate in bending, NEN-EN 1993-1-8, art. 6.2.6.5
    lambda1 = m2_hor_mm ./ (m2_hor_mm + e2_p_mm)
    lambda2 = m1_ver_mm ./ (m2_hor_mm + e2_p_mm)

    leffcp_p_mm = 2 .* pi .* m2_hor_mm
    leffnc_p_mm = alpha_lambda .* m2_hor_mm

    leff_p_mm = min([leffcp_p_mm leffnc_p_mm])

    nea_p_mm = e2_p_mm
    neb_p_mm = 1.25 .* m2_hor_mm

    ne_p_mm = min([nea_p_mm neb_p_mm])
    ne_p_m = ne_p_mm ./ 1000;

    MplRd_p_Nmm = 0.25 .* leff_p_mm .* t_p_mm.^2 .* fy_p_test_Ndmm2;
    MplRd_p_kNm = MplRd_p_Nmm ./ 1000000

    %T-stubs
    FT1Rd_p_kN = 4 .* MplRd_p_kNm ./ (m2_hor_m)
```

```
    FT2Rd_p_kN = (2 .* MplRd_p_kNm + 2 .* ne_p_m .* FtRd_bolt_q88_kN) ./ (m2_hor_
+ ne_p_m)
    FT3Rd_p_kN = 2 .* FtRd_bolt_q88_kN

    FT1_Rd_p_matrix_kN = ([FT1Rd_p_kN FT2Rd_p_kN FT3Rd_p_kN])

    FTRdgov_p_kN = min([FT1Rd_p_kN FT2Rd_p_kN FT3Rd_p_kN])

%C. Column web in tension

    befftwc_c_mm = 2 .* leff_c_mm % two times effective length from column flange

    %NEN-EN 1993-1-8, art. 6.2.6.3, eq(6.15)
    FtwcRd_N = omega_load_one .* befftwc_c_mm .* tw_c_mm .* fy_c_test_Ndmm2;
    FtwcRd_kN = FtwcRd_N ./ 1000

%D. Beam web in tension
    befftwb_b_mm = leff_p_mm     %Effective length from end plate

    %Determining resistance, NEN-EN 1993-1-8, art. 6.2.6.8 eq(6.2)
    FtwbRd_b_N = (befftwb_b_mm .* tw_b_mm .* fy_b_test_Ndmm2) ./ gamma_M0;
    FtwbRd_b_kN = FtwbRd_b_N ./ 1000

%E. Bolts resistances
    FtRd_bolts_kN = 2 .* FtRd_bolt_q88_kN;

%G. Column web on compression
    %NEN-EN 1993-1-8, art. 6.2.6.2, eq(6.10)
    beffcwc_cwoc_mm = tf_b_mm + 2 .* sqrt(2) .* a_weld_mm + 5 .* (tf_c_mm + r_c_mm)
+ 2 .* t_p_mm
    FcwcRd_c_N = (omega_load_one .* kwc .* beffcwc_cwoc_mm .* tw_c_mm .*
fy_c_test_Ndmm2) ./ gamma_M0
    FcwcRd_c_kN = FcwcRd_c_N ./ 1000

    dwc_c_mm = h_c_mm - 2 .* tf_c_mm - 2 .* r_c_mm;

    %NEN-EN 1993-1-8, art. 6.2.6.2, eq(6.13)
    lambda_bar_c = 0.932 .* sqrt(((beffcwc_cwoc_mm .* dwc_c_mm .* fy_c_test_Ndmm2)
./ (E_c_test_Ndmm2 .* (tw_c_mm) .^ 2)))

        if lambda_bar_c < 0.72
        rho_red = 1.0
        elseif lambda_bar_c > 0.72
        rho_red = ( lambda_bar_c - 0.2 ) ./ ((lambda_bar_c).^2)
        else
        rho_red = 0
        end

    %NEN-EN 1993-1-8, art. 6.2.6.7 eq(6.21)
    FcwcRd_red_N = (omega_load_one .* kwc .* rho_red .* beffcwc_cwoc_mm .* tw_c_mm
.* fy_c_test_Ndmm2);
    FcwcRd_red_kN = FcwcRd_red_N ./ 1000;
```



```
    %H. Column web on compression
    McRd_Nmm = (Wpl_mm3 .* fy_b_test_Ndmm2) ./ gamma_M0

    %NEN-EN 1993-1-8, art. 6.2.6.7 (eq. 6.21)
    FcfbRd_N = McRd_Nmm ./ (h_b_mm - tf_b_mm)
    FcfbRd_kN = FcfbRd_N ./ 1000

%Overview of the tension resistances
FRd_tension_one_column_flange = FTRdgov_c_kN
FRd_tension_two_end_plate = FTRdgov_p_kN
FRd_tension_three_column_web = FtwcRd_kN
FRd_tension_four_beam_web = FtwbRd_b_kN
FRd_tension_five_bolts = FtRd_bolts_kN

%Overview of the compression resistances
FRd_compression_one_column_web = FcwcRd_red_kN
FRd_compression_two_beam_webflange = FcfbRd_kN

%Matrix
R_total_barchart_kN =    [   FRd_compression_two_beam_webflange;
                             FRd_compression_one_column_web;
                             0;
                             FRd_tension_five_bolts;
                             FRd_tension_four_beam_web;
                             FRd_tension_three_column_web;
                             FRd_tension_two_end_plate;
                             FRd_tension_one_column_flange ]

 OV_R_total_barchart_kN = zeros(1,4)



    %Overall Bar Chart
    %figure



        %Title if
        if n == 1
        gov_res_kN = zeros(4,1)

        figure

        subplot(2,1,n)
        barh(R_total_barchart_kN)
        title('Parametric Study, Resistances Components MRJ, Variant A')
        axis([0 1000 0 9])
        xlabel('Resistance value [kN]')
        ylabel('Components in tension and compression')
        grid on
        set(gca, 'YTickLabel', {'Beam flange', 'Column web', '', 'Bolts',
```

```
'Beamweb', 'Columnweb', 'Endplate', 'Column flange'})
        legend('Manual Calculation, EC3')

        gov_res_kN(1,n) = min([R_total_barchart_kN(4:8,1)])

        elseif n == 2
        subplot(2,1,n)
        barh(R_total_barchart_kN)
        title('Parametric Study, Resistances Components MRJ, Variant B')
        axis([0 1000 0 9])
        xlabel('Resistance value [kN]')
        ylabel('Components in tension and compression')
        grid on
        set(gca, 'YTickLabel', {'Beam flange', 'Column web', '', 'Bolts',↙
'Beamweb', 'Columnweb', 'Endplate', 'Column flange'})
        legend('Manual Calculation, EC3')

        gov_res_kN(1,n) = min([R_total_barchart_kN(4:8,1)])

        elseif n == 3
        figure
        subplot(2,1,(n-2))
        barh(R_total_barchart_kN)
        title('Parametric Study, Resistances Components MRJ, Variant C')
        axis([0 1000 0 9])
        xlabel('Resistance value [kN]')
        ylabel('Components in tension and compression')
        grid on
        set(gca, 'YTickLabel', {'Beam flange', 'Column web', '', 'Bolts',↙
'Beamweb', 'Columnweb', 'Endplate', 'Column flange'})
        legend('Manual Calculation, EC3')

        gov_res_kN(1,n) = min([R_total_barchart_kN(4:8,1)])

        else n == 4
        subplot(2,1,(n-2))
        barh(R_total_barchart_kN)
        title('Parametric Study, Resistances Components MRJ, Variant D')
        axis([0 1000 0 9])
        xlabel('Resistance value [kN]')
        ylabel('Components in tension and compression')
        grid on
        set(gca, 'YTickLabel', {'Beam flange', 'Column web', '', 'Bolts',↙
'Beamweb', 'Columnweb', 'Endplate', 'Column flange'})
        legend('Manual Calculation, EC3')

        gov_res_kN(1,n) = min([R_total_barchart_kN(4:8,1)])

        end

    pause(0.25)
```



```
end

    Moment_Resistances_kNm = gov_res_kN .* (distance5_mm ./ 1000)
```

B.14 PS3b: Moment Resisting Joint, Extended End Plate

```
%Clear Everything
clear all
clc
close all

pause(0.3)

%%Calculation Valdiation Momement Resisting Connection, Extended End Plate
    % - Script designed to calculate two bolt columns and two bolt rows
    % - Script designed to calculate
    % - M20 bolts, quality 8.8
    % - Only the upper bolts contribute to the resistance against an
    % unsymmetrical load. (2x2)

%%Load parameters
    MEd_kNm =   40;            %Load applied [kNm]
    lsys_m =    1.5;           %System length [m]

%%Alpha_lambda factor: NEN-EN 1993-1-8, art. 6.2.6.5 figure (3.11)
    %[Manually Input Depening on lambda1 and lambda2 !!!]
    alpha_lambda = 6.3;        %Alpha factor [-]

%Load parameters
%NEN-EN 1993-1-8, art. 5.3, table(5.4)
beta_trans = 0;                %Transformation parameter
%NEN-EN 1993-1-8, art. 6.2.6.2(2), note
kwc = 1.0;                     %Reduction factor

%Profile Colmn -> HEB160 (S235), properties by test results
h_c_mm = 160;                  %Height of the column [mm]
b_c_mm = 160;                  %Width fo the column [mm]
tw_c_mm = 8;                   %Web thickess of the column [mm]
tf_c_mm = 13;                  %Flange thickness of the column [mm]
r_c_mm = 15;                   %Radius of the column [mm]
A_c_mm = 5430;                 %Cross sectional area of the column [mm]

e22_c_mm = 40;             %end distance (horizontal) of the column [mm]

fy_c_expv_Ndmm2 = 235;      %Experimental value, Yield strength of the column
[N/mm2]
fu_c_expv_Ndmm2 = 360;      %Experimental value, Tensile strength of the column
[N/mm2]
E_c_expv_Ndmm2 = 210000;    %Experimental value, E-modulus of the column [N/mm2]


%Profile Beam -> IPE220 (S235), properties by test results
h_b_mm = 220;               %Height of the beam [mm]
b_b_mm = 110;               %Width of the beam [mm]
tw_b_mm = 5.9;              %Web thickness of the beam [mm]
tf_b_mm = 9.2;              %Flange thickness of the beam [mm]
r_b_mm = 12;                %Radius of the beam [mm]
A_b_mm2 = 3340;             %Cross sectional area of the beam [mm2]
Wpl_b_mm3 = 285000;         %Sectional are of the beam [mm3]
```



```
fy_b_expv_Ndmm2 = 235;      %Experimental value, yield strength of the beam [N/mm2]
fu_b_expv_Ndmm2 = 360;      %Experimental value, tensile strength of the beam
[N/mm2]


%Extended End Plate (S235)
b_p_mm = 140;              %Width of the end plate [mm]
t_p_mm = 15;               %Thickness of the end plate [mm]

p_mm = 70;                 %Vertical pitch of the end plate [mm]
w_mm = 80;                 %Horizontal pitch of the end plate [mm]

e1_ver_p_mm = 44.6;        %End distance (vertical) of the end plate [mm]
e2_hor_p_mm = 30;          %End distance (horizontal) of the end plate [mm]

distance1_p_mm = 75;       %Distances in end plate [mm]
distance2_p_mm = 114.6;        %Distances in end plate [mm]

fy_p_expv_Ndmm2 = 235;  %Yield strength of the end plate [N/mm2]
fu_p_expv_Ndmm2 = 360;  %Tensile strength of end plate [N/mm2]


%Bolts 2x2 M20, quality 8.8
fub_bo_Ndmm2 = 800;    %Tensile strength of the bolts [N/mm2]
d_bo_mm = 20;              %Diameter of the bolt [mm]
d0_bo_mm = 22;             %Bolt hole diameter [mm]
k2_bo = 0.9;               %k2 factor [-]
As_bo_mm2 = 245;           %Cross sectional area of the bolt [mm2]

r_out_mm = 245.8               %Distance outside bolt - compression point [mm]
r_in_mm = 175.8                %Distance inside bolt - compression point [mm]

%Welds
a_weld_mm = 5;             %Troath thickness of the weld [mm]

%Stiffner, double symmetrical welded
ts_s_mm = 10;
bsg_s_mm = 76;
bsn_s_mm = 55;
fy_s_Ndmm2 = 235;
fu_s_Ndmm2 = 360;


%Safety factors NEN-EN 1993-1-8, art. 6.1(1)
gamma_M0 = 1.0;                %Safety factor, cross-sections
gamma_M1 = 1.0;                %Safety factor, stability
gamma_M2 = 1.25;               %Safety factor, bolts and welds


%%Derived parameters

    %Tension ristance bolts M20, steel quality 8.8
```

```
    %NEN-EN 1993-1-8, art. 3.6.1, table(3.4)
    FtRd_M20_q88_N = (k2_bo .* fub_bo_Ndmm2 .* As_bo_mm2) ./ gamma_M2;
    FtRd_M20_q88_kN = FtRd_M20_q88_N ./ 1000;

    %M-distance of the column flange
    m22_hor_c_mm = 0.5 .* w_mm - 0.5 .* tw_c_mm - 0.8 .* r_c_mm;
    m22_hor_c_m = m22_hor_c_mm ./ 1000

    %M-distance of the extended endplate
    %M-Outside
    m1_out_ver_p_mm = distance1_p_mm - e1_ver_p_mm - 0.8 .* sqrt(2) .* a_weld_mm
    m1_out_ver_p_m = m1_out_ver_p_mm ./ 1000;

    %M-Inside
    m1_in_ver_p_mm = distance2_p_mm - distance1_p_mm - tf_b_mm - 0.8 .* sqrt(2) .*
a_weld_mm
    m1_in_ver_p_m = m1_in_ver_p_mm ./ 1000;

    m2_in_hor_p_mm = 0.5 .* w_mm - 0.5 .* tw_b_mm - 0.8 .* a_weld_mm .* sqrt(2);
    m2_in_hor_p_m = m2_in_hor_p_mm ./ 1000;


%%Components

%A. Column flange in bending

    %Determining effective lengths
    leffcp_cfb1_c_mm = 2 .* pi .* m22_hor_c_mm
    leffcp_cfb2_c_mm = pi .* m22_hor_c_mm + p_mm
    leffnc_cfb3_c_mm = 4 .* m22_hor_c_mm + 1.25 .* e22_c_mm
    leffnc_cfb4_c_mm = 2 .* m22_hor_c_mm + 0.625 .* e22_c_mm + 0.5 .* p_mm

    leffgov_cfb_c_mm = min([leffcp_cfb1_c_mm leffcp_cfb2_c_mm leffnc_cfb3_c_mm
leffnc_cfb4_c_mm])


    %Determing ne for T-stub resistances
    e22_c_mm
    e22_limit_cfb_c_mm = 1.25 .* m22_hor_c_mm
    ne_cfb_c_mm = min([e22_c_mm e22_limit_cfb_c_mm])
    ne_cfb_c_m = ne_cfb_c_mm ./ 1000;

    %Determing Plastic Moment Resistance
    MplRd_c_Nmm = 0.25 .* leffgov_cfb_c_mm .* (tf_c_mm).^2 .* fy_c_expv_Ndmm2
    MplRd_c_kNm = MplRd_c_Nmm ./ 1000000

    %T-Stub Resistances Column Flange in Bending
    %NEN-EN 1993-1-8, art. 6.2.4.1, table(6.2)
    Ft1Rd_cfb_c_kN = (4 .* MplRd_c_kNm) ./ m22_hor_c_m
    Ft2Rd_cfb_c_kN = (2 .* MplRd_c_kNm + 2 .* ne_cfb_c_m .* FtRd_M20_q88_kN) ./
m22_hor_c_m + ne_cfb_c_m)
    Ft3Rd_cfb_c_kN = 2 .* FtRd_M20_q88_kN
```



```
    FtRdgov_cfb_c_kN = min([Ft1Rd_cfb_c_kN Ft2Rd_cfb_c_kN Ft3Rd_cfb_c_kN])

    FtRdgov_tot_cfb_c_kN = 2 .* FtRdgov_cfb_c_kN

%B. End Plate in Bending

    %B1 End Plate Outside Part

    %Determining the effective lengths
    leffcp_epb1_out_p_mm = 2 .* pi .* m1_out_ver_p_mm;
    leffcp_epb2_out_p_mm = pi .* m1_out_ver_p_mm + w_mm;
    leffcp_ebp3_out_p_mm = pi .* m1_out_ver_p_mm + 2 .* e2_hor_p_mm
    leffnc_ebp4_out_p_mm = 4 .* m1_out_ver_p_mm + 1.25 .* e1_ver_p_mm
    leffnc_ebp5_out_p_mm = e2_hor_p_mm + 2 .* m1_out_ver_p_mm + 0.625 .*
e1_ver_p_mm;
    leffnc_ebp6_out_p_mm = 0.5 .* b_p_mm
    leffnc_ebp7_out_p_mm = 0.5 .* w_mm + 2.* m1_out_ver_p_mm + 0.625 .*
e1_ver_p_mm;

    leff_epb_matrix_p_mm = [    leffcp_epb1_out_p_mm;
                                leffcp_epb2_out_p_mm;
                                leffcp_ebp3_out_p_mm;
                                leffnc_ebp4_out_p_mm;
                                leffnc_ebp5_out_p_mm;
                                leffnc_ebp6_out_p_mm;
                                leffnc_ebp7_out_p_mm]


    leff_epb_p_mm = min([leff_epb_matrix_p_mm])

    %Determing Plastic Moment Resistance
    MplRd_out_p_Nmm = 0.25 .* leff_epb_p_mm .* (t_p_mm).^2 .* fy_p_expv_Ndmm2
    MplRd_out_p_kNm = MplRd_out_p_Nmm ./ 1000000

    %Determing ne (outside) for T-stub resistances
    e1_ver_p_mm;
    e1_ver_limit_epb_p_mm = 1.25 .* m1_out_ver_p_mm;
    ne_epb_out_p_mm = min( [e1_ver_p_mm e1_ver_limit_epb_p_mm])
    ne_epb_out_p_m = ne_epb_out_p_mm ./ 1000;

    %T-Stub Resistances End plate in bending
    %NEN-EN 1993-1-8, art. 6.2.4.1, table(6.2)
    FT1Rd_epb_out_kN = (4 .* MplRd_out_p_kNm) ./ m1_out_ver_p_m
    FT2Rd_epb_out_kN = (2 .* MplRd_out_p_kNm + 2 .* ne_epb_out_p_m .*
FtRd_M20_q88_kN) ./ ( m1_out_ver_p_m + ne_epb_out_p_m )
    FT3Rd_epb_out_kN = 2 .* FtRd_M20_q88_kN

    FTRdgov_epb_out_kN = min([FT1Rd_epb_out_kN FT2Rd_epb_out_kN FT3Rd_epb_out_kN])
    FTRdgov_ht_epb_out_kN = (FTRdgov_epb_out_kN ./ 2)      %Half T-stub

    FTRdgov_tot_epb_out_kN = 2 .* FTRdgov_ht_epb_out_kN


    %B2 End Plate Inside Part
```

```
    %Determining lambda factors
    lambda1 = m2_in_hor_p_mm ./ ( m2_in_hor_p_mm + e2_hor_p_mm )
    lambda2 = m1_in_ver_p_mm ./ ( m2_in_hor_p_mm + e2_hor_p_mm )

    %Effective lengths
    leffcp_epb1_in_mm = 2 .* pi .* m2_in_hor_p_mm;
    leffcm_ebp2_in_mm = alpha_lambda .* m2_in_hor_p_mm;

    leffgov_epb_in_mm = min([leffcp_epb1_in_mm leffcm_ebp2_in_mm])

    %Determing Plastic Moment Resistance
    MplRd_in_p_Nmm = 0.25 .* leffgov_epb_in_mm .* (t_p_mm).^2 .* fy_p_expv_Ndmm2;
    MplRd_in_p_kNm = MplRd_in_p_Nmm ./ 1000000

    %Determing ne (inside) for T-stub resistance
    e2_hor_p_mm
    e2_hor_limit_p_mm = 1.25 .* m2_in_hor_p_mm;
    ne_epb_in_mm = min([e2_hor_p_mm e2_hor_limit_p_mm])
    ne_epb_in_m = ne_epb_in_mm ./ 1000

    %Determing T-stub resistance
    %NEN-EN 1993-1-8, art. 6.2.4.1, table(6.2)
    FT1Rd_epb_in_kN = (4 .* MplRd_in_p_kNm ) ./ m2_in_hor_p_m
    FT2Rd_epb_in_kN = (2 .* MplRd_in_p_kNm + 2 .* ne_epb_in_m .* FtRd_M20_q88_kN) ↙
/ (m2_in_hor_p_m + ne_epb_in_m)
    FT3Rd_epb_in_kN = (2 .* FtRd_M20_q88_kN)

    FTRdgov_tot_epb_in_kN = min([FT1Rd_epb_in_kN FT2Rd_epb_in_kN FT3Rd_epb_in_kN])


%C Column web in Tension

    %Effective length
    beff_cwt_mm = 2 .* leffgov_cfb_c_mm

    %Omega factor
    beff_column_c_mm = tf_b_mm + 2 .* sqrt(2) .* a_weld_mm + 5 .* (tf_c_mm ↙
r_c_mm) + 2 .* t_p_mm
    Avc_c_mm2 = A_c_mm - (2 .* b_c_mm .* tf_c_mm) + (tw_c_mm + 2 .* r_c_mm) .↙
tf_c_mm
    omega_load = 1 ./ sqrt( 1 + 1.3 .* ((beff_column_c_mm .* tw_c_mm) .↙
(Avc_c_mm2)).^2)

    %Determining resistance, NEN-EN 1993-1-8, art. 6.2.6.3, eq(6.15)
    FtwcRdgov_cwt_N = (omega_load .* beff_cwt_mm .* tw_c_mm .* fy_c_expv_Ndmm2) .↙
gamma_M0
    FtwcRdbov_cwt_kN = FtwcRdgov_cwt_N ./ 1000


%D Beam web in Tension

    %Effective width
```



```
    beff_bwt_b_mm = leffcm_ebp2_in_mm

    %Determining resistance, NEN-EN 1993-1-8, art. 6.2.6.8 eq(6.2)
    FtwbRdgov_bwt_c_N = (beff_bwt_b_mm .* tw_b_mm .* fy_c_expv_Ndmm2) ./ gamma_M0
    FtwbRdgov_bwt_c_kN = FtwbRdgov_bwt_c_N ./ 1000


%E Bolts in Tension

    %Determining bolt resistance
    FtRd_boltrow_kN = 2 .* FtRd_M20_q88_kN
    FtRdgov_tot_bo_kN = 2 .* FtRd_boltrow_kN

%G Column web in compression

    %Effective width
    beff_cwc_c_mm = tf_b_mm + 2 .* sqrt(2) .* a_weld_mm + 5 .* (tf_c_mm + r_c_mm) ↙
2 .* t_p_mm

    %Determining resistance without buckling factor
    %NEN-EN 1993-1-8, art. 6.2.6.2, eq(6.10)
    FcwcRd_nonbuc_c_kN = omega_load .* kwc .* beff_cwc_c_mm .* tw_c_mm .↙
fy_c_expv_Ndmm2 ./ gamma_M0

    %Determining buckling factor
    dwc_mm = h_c_mm - 2 .* (tf_c_mm + r_c_mm)

    %NEN-EN 1993-1-8, art. 6.2.6.2, eq(6.13)
    lambda_bar = 0.932 .* sqrt( (beff_cwc_c_mm .* dwc_mm .* fy_c_expv_Ndmm2) .↙
(E_c_expv_Ndmm2 .* (tw_c_mm).^2))

    %If-statement for buckling factor
        if lambda_bar < 0.72
            rho_red = 1.0;
        elseif lambda_bar > 0.72
            rho_red = (lambda_bar - 0.2) ./ (lambda_bar.^2);
        else
            rho_red = 0;
        end

    %Determining resistance with buckling factor
    FcwcRd_N = (omega_load .* kwc .* rho_red .* beff_cwc_c_mm .* tw_c_mm .↙
fy_c_expv_Ndmm2) ./ gamma_M1
    FcwcRd_buc_c_kN = FcwcRd_N ./ 1000;

    %Determing the governing risitance for column flange under bending
    FcwcRdgov_c_kN = min([FcwcRd_nonbuc_c_kN FcwcRd_buc_c_kN])

%H Beam flange and beam web under compression
McRd_b_Nmm = (Wpl_b_mm3 .* fy_b_expv_Ndmm2) ./ gamma_M0;
McRd_b_kNm = McRd_b_Nmm ./ 1000000

%NEN-EN 1993-1-8, art. 6.2.6.7 (eq. 6.21)
```

```
FcfbRd_b_N = McRd_b_Nmm ./ (h_b_mm - tf_b_mm);
FcfbRd_b_kN = FcfbRd_b_N ./ 1000


%Tension resistances
%Overview of the tension resistances
FRd_tension_one_column_flange = FtRdgov_tot_cfb_c_kN
FRd_tension_two_end_plate = FTRdgov_tot_epb_out_kN + FTRdgov_tot_epb_in_kN
FRd_tension_three_column_web = FtwcRdbov_cwt_kN
FRd_tension_four_beam_web = FtwbRdgov_bwt_c_kN
FRd_tension_five_bolts = FtRdgov_tot_bo_kN

    %Tension Matrix Total
    R_tension_kN =          [   FRd_tension_one_column_flange;
                                FRd_tension_two_end_plate;
                                FRd_tension_three_column_web;
                                FRd_tension_four_beam_web;
                                FRd_tension_five_bolts]

    %Tension zone Matrix
    R_tension_total_kN =     [          0        0
FRd_tension_one_column_flange;
                                        0        0       FRd_tension_two_end_plate;
                                        0        0
FRd_tension_three_column_web;
                                        0        0       FRd_tension_four_beam_web;
                                        0        0       FRd_tension_five_bolts]

    R_tension_separate_kN  =       [    0
0;
                                        FTRdgov_tot_epb_in_kN
FTRdgov_tot_epb_out_kN      0;
                                        0
0;
                                        0
0;
                                        FtRd_boltrow_kN             FtRd_boltrow_kN
0]

    %Figure in tension zone
    figure
    barh(R_tension_total_kN, 'FaceColor', [0 153/256 51/256])
    hold on
    barh(R_tension_separate_kN, 'FaceColor', [0 51/256 153/256])
    grid on
    set(gca, 'YTicklabel', {'Column flange in bending', 'Endplate in bending',
'Column web in tension', 'Beam web in tension', 'Bolts in tension'})
    ylabel('Component in Tension')
    xlabel('Resistance Value [kN]')
    legend('Resistance Component', 'location', 'southeast')


%Overview of the compression resistance
```



```
FRd_compression_one_column_web = FcwcRdgov_c_kN
FRd_compresion_one_beam_flange = FcfbRd_b_kN

    %Compression Matrix
    R_compression_kN = [    FRd_compression_one_column_web;
                            FRd_compresion_one_beam_flange]

%Total Matrix
R_overall_kN = [        R_compression_kN;
                        0;
                        R_tension_kN]

%Overall Bar Chart
figure
barh(R_overall_kN)
xlabel('Resistance value [kN]')
ylabel('Components in tension and compression')
grid on
set(gca, 'YTickLabel', {'Column web in compression', 'Beam flange in compression',
'', 'Column flange in bending', 'Endplate in bending', 'Column web in tension',
'Beamweb in tension', 'Bolts in tension'})
title('Parametric Study, Resistances Components MRJ, Variant E')
legend('Manual Calculation, EC3')

%Determining Resistance [Manually Input]!
Difference_kN = min(R_tension_kN) - min(R_compression_kN)

FRd_bo1_out_kN = FTRdgov_tot_epb_out_kN
FRd_bo1_in_kN = min(R_compression_kN) - FTRdgov_tot_epb_out_kN

MRd_bo1_out_kNm = FRd_bo1_out_kN .* (r_out_mm ./ 1000)
MRd_bo1_in_kNm  = FRd_bo1_in_kN .* (r_in_mm ./ 1000)

MRd_tot1_kNm = MRd_bo1_in_kNm + MRd_bo1_out_kNm

controle1 = min(R_compression_kN) - FRd_bo1_in_kN - FRd_bo1_out_kN

FRd_bo2_out_kN  = FRd_compression_one_column_web ./ 2
FRd_bo2_in_kN   = FRd_compression_one_column_web ./ 2

MRd_bo2_out_kNm = FRd_bo2_out_kN .* (r_out_mm ./ 1000)
MRd_bo2_in_kNm  = FRd_bo2_in_kN .* (r_in_mm ./ 1000)

MRd_tot2_kNm = MRd_bo2_in_kNm + MRd_bo2_out_kNm

controle2 = min(R_compression_kN) - FRd_bo2_in_kN - FRd_bo2_out_kN


%Overall Bar Chart
figure
subplot(1,2,1)
barh(R_overall_kN)
xlabel('Resistance value [kN]')
```

```
ylabel('Components in tension and compression')
axis([0 900 0 9])
grid on
set(gca, 'YTickLabel', {'Column web in compression', 'Beam flange in compression',
'', 'Column flange in bending', 'Endplate in bending', 'Column web in tension',
'Beamweb in tension', 'Bolts in tension'})
title('Parametric Study, Resistances Components MRJ, Variant E')
legend('Manual Calculation, EC3')


%Stiffened Column
epsilon_s = sqrt(235 ./ fy_s_Ndmm2)

Aseff_s_mm2 = ((30 .* epsilon_s .* tw_c_mm + ts_s_mm) .* tw_c_mm) + ( 2 .* bsg_s_mm
.* ts_s_mm)
Is_s_mm4 = (1 ./ 12) .* ts_s_mm .* (2 .* bsg_s_mm + tw_c_mm) .^ 3
is_s_mm = sqrt( Is_s_mm4 ./ Aseff_s_mm2)

l_cbl_mm = h_c_mm - (2 .* tf_c_mm)

lambda_one = 93.9 .* epsilon_s
lambda_bar = l_cbl_mm ./ ( is_s_mm .* lambda_one )

if lambda_bar < 0.2
    NcRd_s_N = (Aseff_s_mm2 .* fy_s_Ndmm2) ./ gamma_M1
else
    NcRd_s_N = 0;
end

NcRd_s_kN = NcRd_s_N ./ 1000;

FRd_compression_extra_stiff = NcRd_s_kN

pause(0.5)

%Stiffner
    R_tension_stiff_kN =                    [   FRd_tension_one_column_flange;
                                                FRd_tension_two_end_plate;
                                                FRd_tension_three_column_web;
                                                FRd_tension_four_beam_web;
                                                FRd_tension_five_bolts]

    R_compression_stiff_kN = [                  FRd_compression_extra_stiff
                                                0;
                                                FRd_compresion_one_beam_flange]

%Total Matrix
R_overall_stiff_kN = [          R_compression_stiff_kN;
                                0;
                                R_tension_stiff_kN]


%Overall Bar Chart
```



```
subplot(1,2,2)
barh(R_overall_stiff_kN)
xlabel('Resistance value [kN]')
ylabel('Components in tension and compression')
grid on
axis([0 900 0 10])
set(gca, 'YTickLabel', {'Compression stiffner','Column web in compression', 'Beam
flange in compression', '', 'Column flange in bending', 'Endplate in bending',
'Column web in tension', 'Beamweb in tension', 'Bolts in tension'})
title('Parametric Study, Resistances Components MRJ, Variant F')
legend('Manual Calculation, EC3')
```

Appendix C

FEA Models

The following partial FEA models are made:

- CA1: SSJ, Fin Plate Joint
- CA2: SSJ, Short End Plate Joint
- CA3: SSJ, Double Angle Cleats Joint
- CA4: MRJ, Extended Endplate Joint symmetrical loading
- CA5: MRJ, Extended Endplate Joint unsymmetrical loading
- CA6: MRJ, Welded Joint
- CA11-EXP: MRJ, Flush End Plate Joint
- CA11-EC3: MRJ, Flush End Plate Joint
- CA12-EXP: MRJ, Extended End Plate Joint
- CA12-EC3: MRJ, Extended End Plate Joint

- PS1: Fin Plate Joint
- PS2: Short End Plate Joint
- PS3: Moment Resisting Joint

These models can be found on the USB-Flashdrive enclosed to this thesis.

Appendix D

Technical Drawings

Technical drawings of the Flush End Plate Joint and Extended End Plate Joint, which where tested during experiments, can be found in this Appendix. Also the variants of the parametric studies can be found in this Appendix.

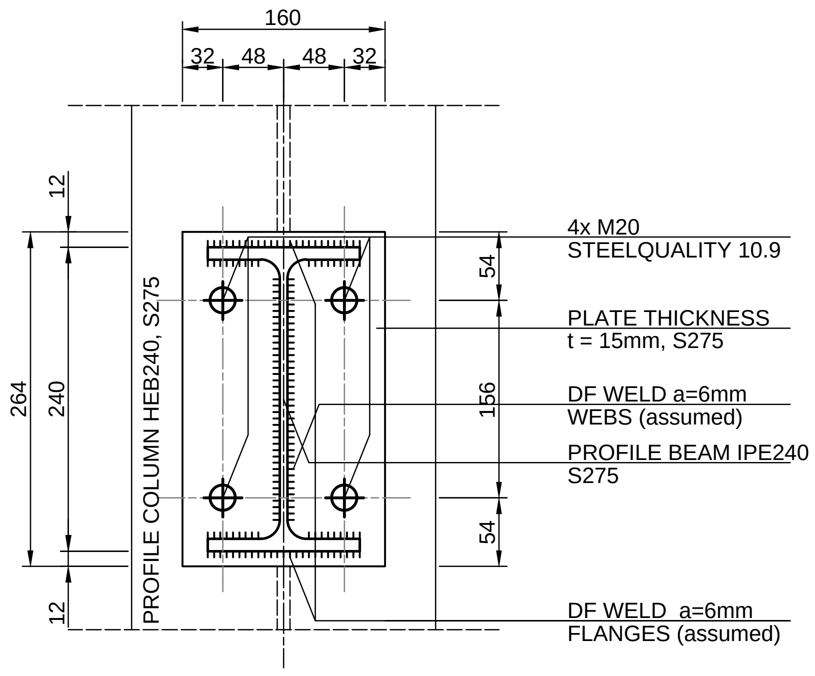
This appendix contains the following drawings:

- Flush End Plate Moment Resisting Joint (CA11)
- Extended End Plate Moment Resisting Joint (CA22)
- Fin Plate Joint (PS1)
- Short End Plate Joint (PS2)
- Moment Resisting Joint, Variant A & B (PS3)
- Moment Resisting Joint, Variant B & C (PS3)
- Moment Resisting Joint, Variant D & E (PS3)

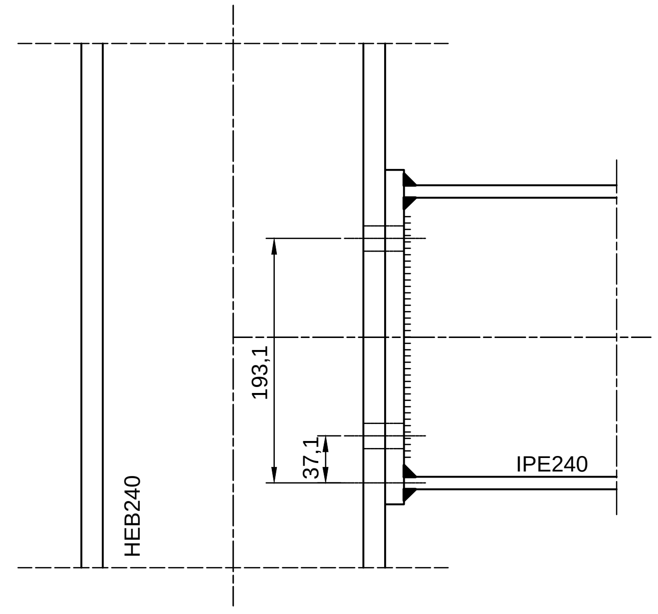
FLUSH ENDPLATE JOINT (CA11)

PROPERTIES JOINT ELEMENTS, FLUSH ENDPLATE

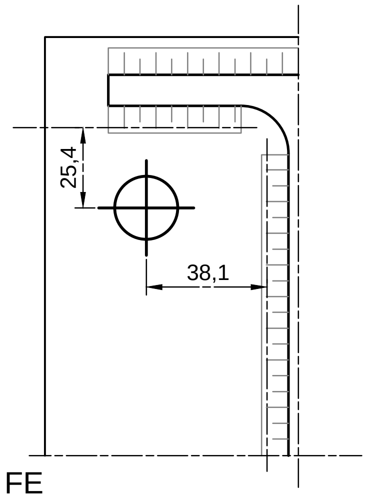
IPE240	EXPERIMENTAL VALUES			DERIVED CALCUALTION VALUES		
YIELD STRENGTH	fy	351.7	N/mm2	fy	350	N/mm2
TENSILE STRENGTH	fu	451.3	N/mm2	fu	450	N/mm2
E-MODULUS	E	209468	N/mm2	E	210000	N/mm2
HEB240						
YIELD STRENGTH	fy	357.5	N/mm2	fy	360	N/mm2
TENSILE STRENGTH	fu	463.0	N/mm2	fu	460	N/mm2
E-MODULUS	E	213864	N/mm2	E	210000	N/mm2
END PLATE						
YIELD STRENGTH	fy	369.4	N/mm2	fy	370	N/mm2
TENSILE STRENGTH	fu	503.5	N/mm2	fu	500	N/mm2
E-MODULUS	E	200248	N/mm2	E	20000	N/mm2



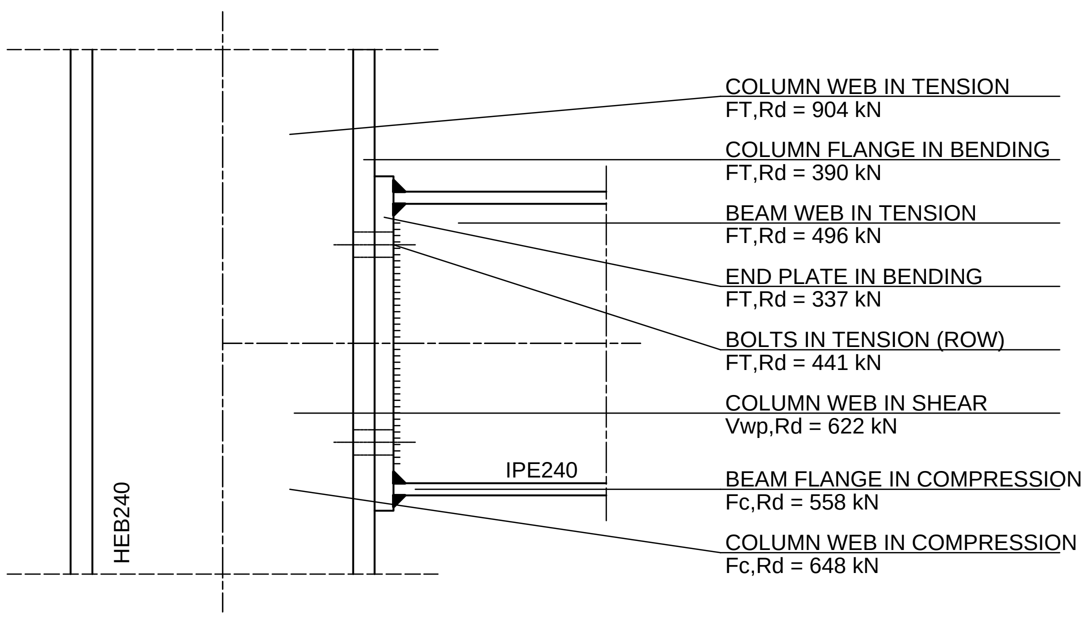
MOMENT RESISTING JOINT
FLUSH ENDPLATE
CROSS-SECTIONAL VIEW, SCALE 1:5



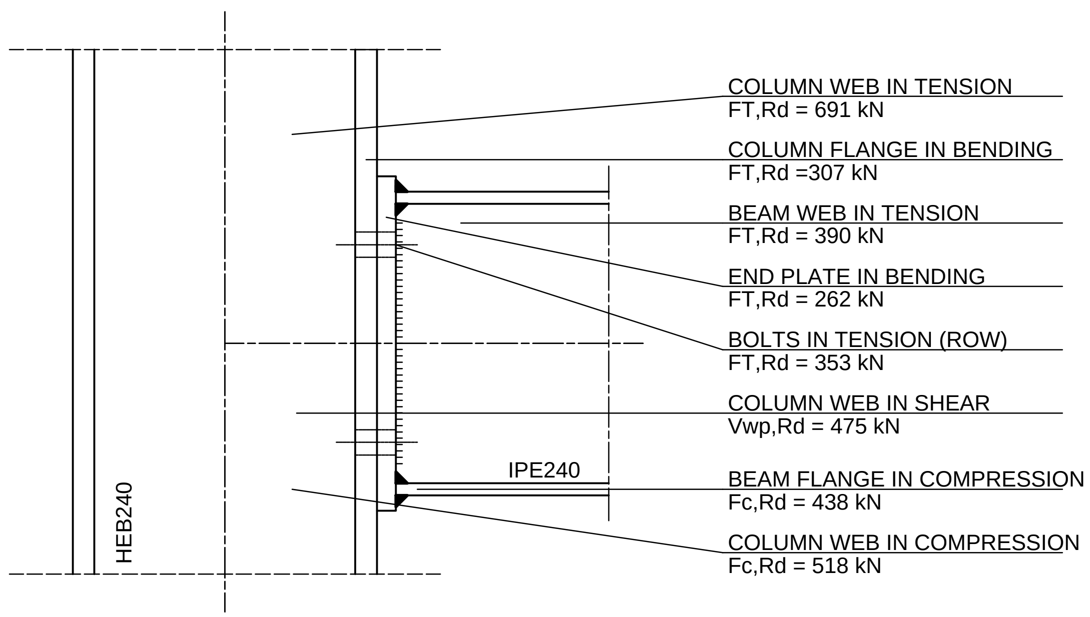
MOMENT RESISTING JOINT
FLUSH ENDPLATE
SIDE VIEW, SCALE 1:5



DETAIL 2
M-DISTANCES FE
SCALE 1:2



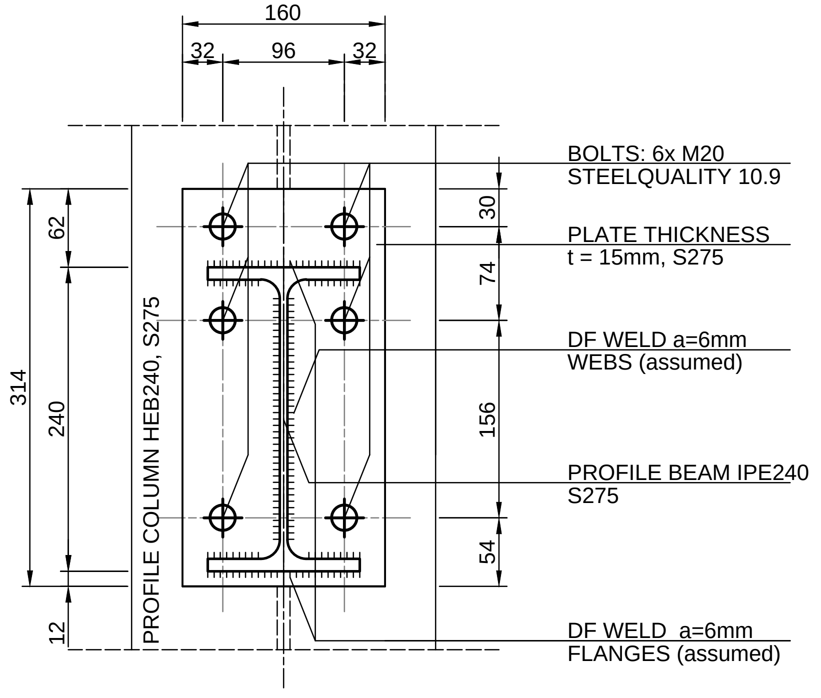
RESISTANCE VALUES COMPONENTS
EXPERIMENTAL VALUES
SIDE VIEW, SCALE 1:5



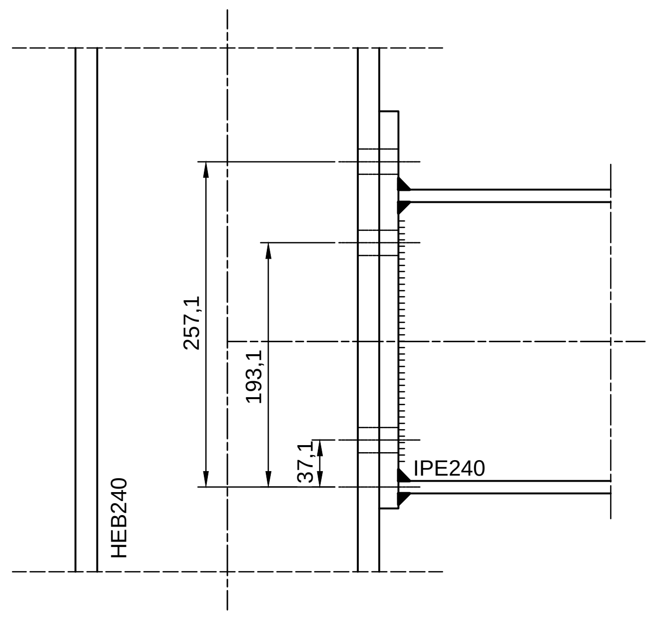
RESISTANCE VALUES COMPONENTS
EC3 VALUES
SIDE VIEW, SCALE 1:5

Surface Treatment:
Remarks:
Rev. | Date | Drawn | Checked | Cause | Description
HOLLANDIA
Client -
Project -
Title -
Drawn by | Checked | Initials | Date | Scale | Format A0
PROJECT No. **.***
DRG. No. 000
REV.

EXTENDED ENDPLATE JOINT (CA22)



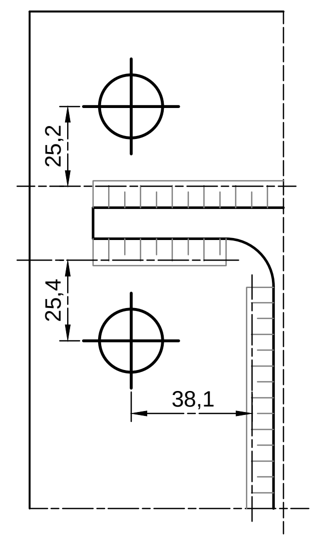
MOMENT RESISTING JOINT
EXTENDED ENDPLATE
CROSS-SECTIONAL VIEW, SCALE 1:5



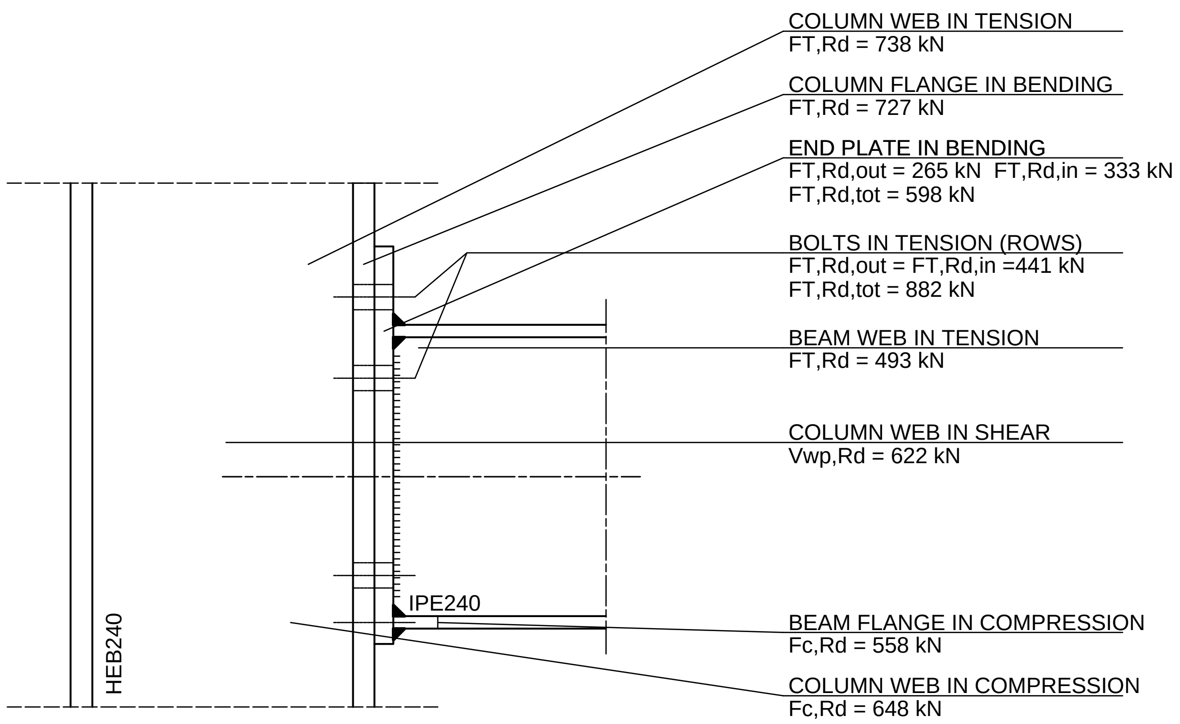
MOMENT RESISTING JOINT
EXTENDED ENDPLATE
SIDE VIEW, SCALE 1:5

PROPERTIES JOINT ELEMENTS, EXTENDED ENDPLATE

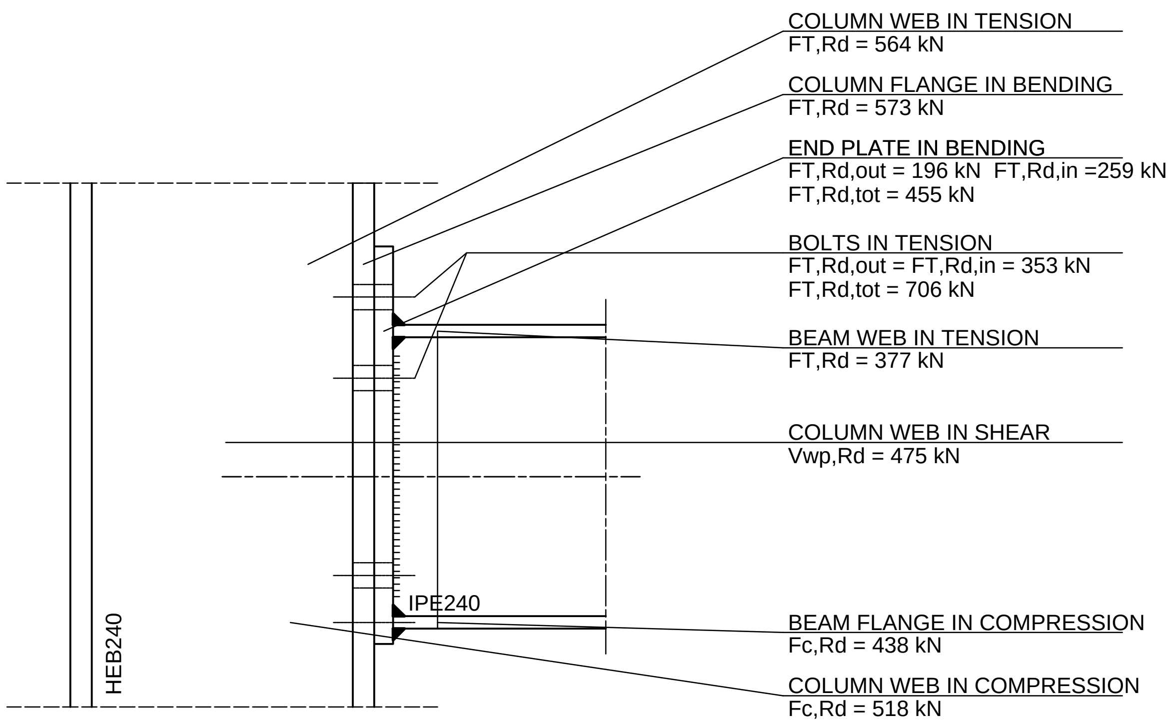
IPE240	EXPERIMENTAL VALUES			DERIVED CALCUALTION VALUES		
YIELD STRENGTH	fy	351.5	N/mm2	fy	350	N/mm2
TENSILE STRENGTH	fu	451.2	N/mm2	fu	450	N/mm2
E-MODULUS	E	209468	N/mm2	E	210000	N/mm2
HEB240						
YIELD STRENGTH	fy	357.5	N/mm2	fy	360	N/mm2
TENSILE STRENGTH	fu	463.0	N/mm2	fu	460	N/mm2
E-MODULUS	E	213864	N/mm2	E	210000	N/mm2
END PLATE						
YIELD STRENGTH	fy	369.4	N/mm2	fy	370	N/mm2
TENSILE STRENGTH	fu	503.5	N/mm2	fu	500	N/mm2
E-MODULUS	E	200248	N/mm2	E	200000	N/mm2



DETAIL 2
M-DISTANCES EE
SCALE 1:2



RESISTANCE VALUES COMPONENTS
EXPERIMENTAL VALUES
SIDE VIEW, SCALE 1:5



RESISTANCE VALUES COMPONENTS
EC3 VALUES
SIDE VIEW, SCALE 1:5

Surface Treatment:
Remarks:
Rev. | Date | Drawn | Checked | Cause | Description

HOLLANDIA
Hollandia b.v.
P.O. Box 12
2920 AA Krimpen a/d IJssel

Client	-				
Project	-				
Title	-				
	Initials	Date	Scale	PROJECT No.	DRG. No. / REV.
Drawn by	**	**-**-****			
Checked	**	**-**-****	Format A0	**.***	000

VARIANTS FOR PARAMETRIC STUDY, FIN PLATE JOINT

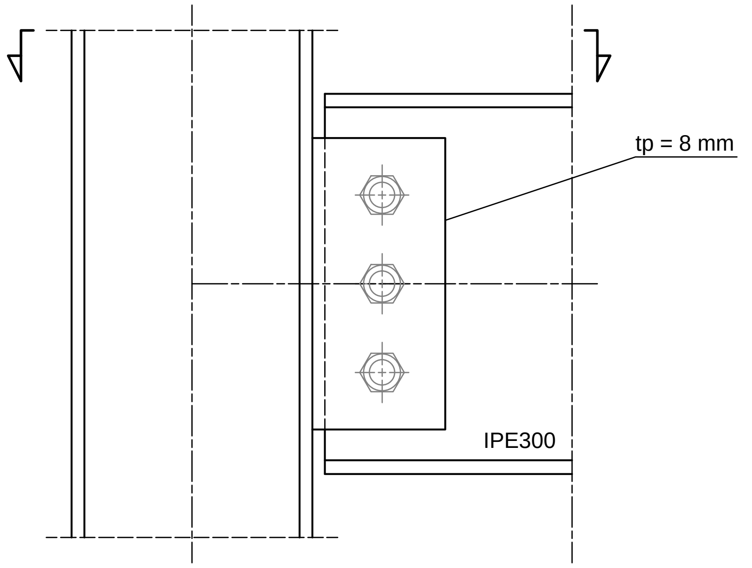
VARIANT A

BEAM: IPE300
- h = 300 mm
- b = 150 mm
- tw = 7.1 mm
- tf = 10.7 mm
- r = 15 mm
- A = 5380 mm2
- e11 = 80 mm

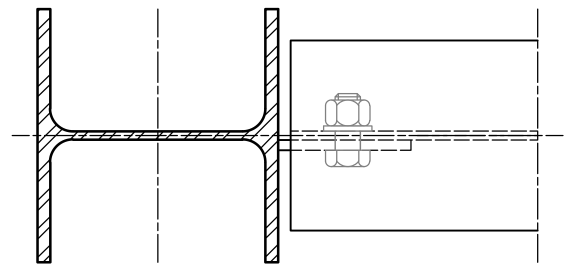
FIN PLATE
- hp = 290 mm
- tp = 8 mm
- e1 = 45 mm
- p1 = 70 mm

BOLTS
- type: 3x M20
- quality: 8.8

RESISTANCE
- VRd;EC3 = 178 kN
- VRd;model = 210 kN



VARIANT A, SIDE VIEW



VARIANT A, CROSS-SECTIONAL VIEW

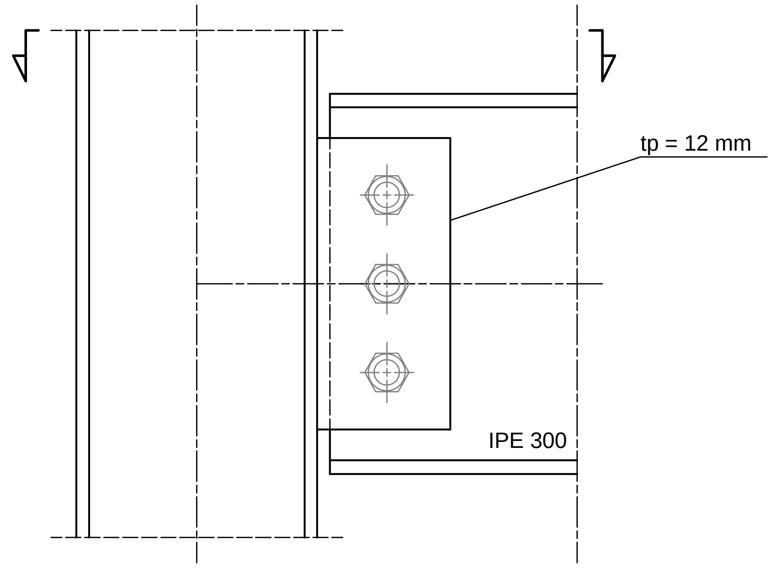
VARIANT B

BEAM: IPE300
- h = idem
- b = idem
- tw = idem
- tf = idem
- r = idem
- A = idem
- e11 = idem

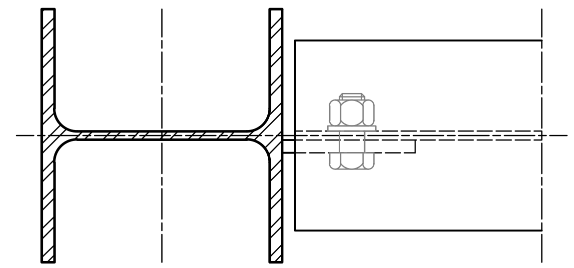
FIN PLATE
- hp = idem
- tp = 12 mm
- e1 = idem
- p1 = idem

BOLTS
- type: idem
- quality: idem

RESISTANCE
- VRd;EC3 = 194 kN
- VRd;model = 250 kN



VARIANT B, SIDE VIEW



VARIANT B, CROSS-SECTIONAL VIEW

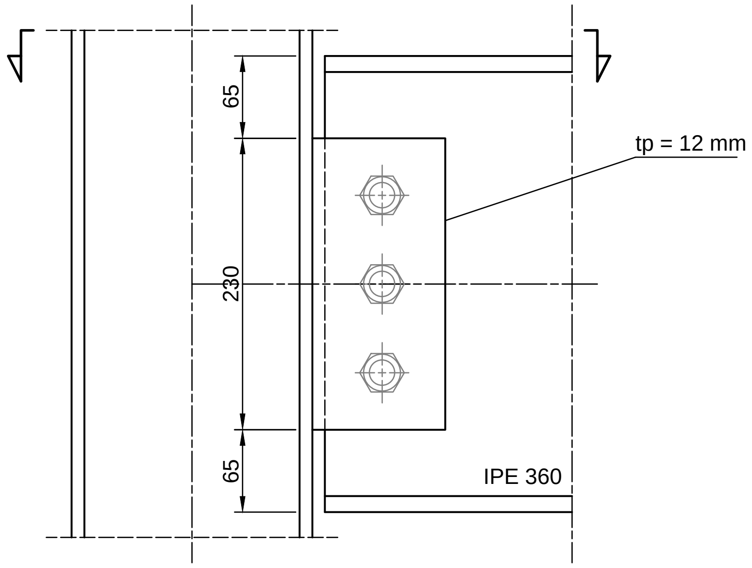
VARIANT C

BEAM: IPE360
- h = 360 mm
- b = 170 mm
- tw = 8.0 mm
- tf = 12.7 mm
- r = 18 mm
- A = 7270 mm2
- e11 = 110 mm2

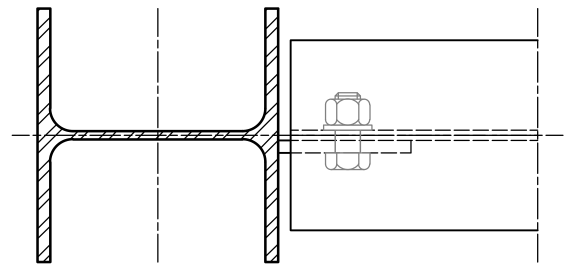
FIN PLATE
- hp = idem
- tp = 12 mm
- e1 = idem
- p1 = idem

BOLTS
- type: idem
- quality: idem

RESISTANCE
- VRd;EC3 = 267 kN
- VRd;model = 260 kN



VARIANT C, SIDE VIEW



VARIANT C, CROSS-SECTIONAL VIEW

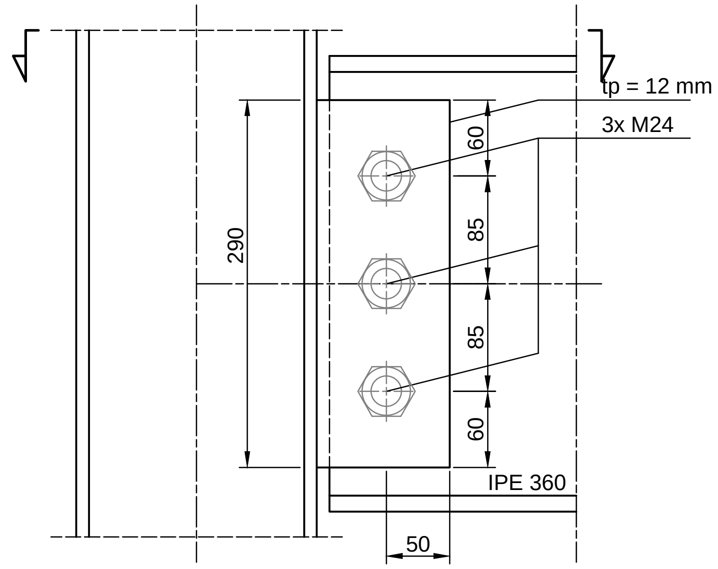
VARIANT D

BEAM: IPE360
- h = 360 mm
- b = 170 mm
- tw = 8.0 mm
- tf = 12.7 mm
- r = 18 mm
- A = 7270 mm2
- e11 = 95 mm2

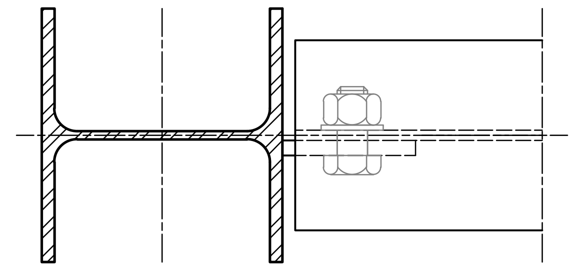
FIN PLATE
- hp = 290 mm
- tp = 12 mm
- e1 = 60 mm
- p1 = 85 mm

BOLTS
- type: M24
- quality: 8.8

RESISTANCE
- VRd;EC3 = 273 kN
- VRd;model = 340 kN



VARIANT D, SIDE VIEW



VARIANT D, CROSS-SECTIONAL VIEW

Surface Treatment:

Welds, unless otherwise noted a 4 | All bolts quality 8.8 hot dip galvanised

Remarks:

Rev.	Date	Drawn	Checked	Cause	Description

HOLLANDIA

Hollandia b.v.
P.O. Box 12
2920 AA Krimpen a/d IJssel
Phone: +31 (0) 180 - 540 540
Fax: +31 (0) 180 - 519 956
Tekenkamer Dintelmond: tel. 0167-527100
Emailadres: Secr.engineering.heijningen@hollandia.biz

Client -

Project -

Title -
-

R is as drawn, L is opp. hand.

Projection

Units: mm

Tolerances acc. to: EN ISO 13920 classes B&F

Welds acc. to: EN 22553

	Initials	Date	Scale	PROJECT No.	DRG. No.	REV.
Drawn by	**	**-**-****		**.***	000	
Checked	**	**-**-****	Format A0			

VARIANTS FOR PARAMETRIC STUDY, SHORT END PLATE JOINT

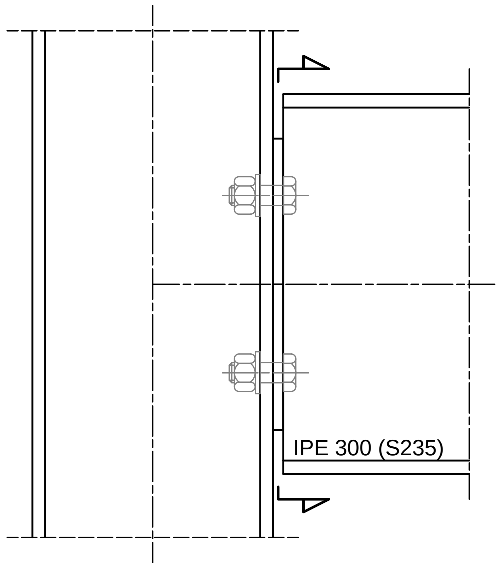
VARIANT A

BEAM: IPE300
- h = 300 mm
- b = 150 mm
- tw = 7.1 mm
- tf = 10.7 mm
- r = 15 mm
- A = 5380 mm2
- Steel Quality: S235

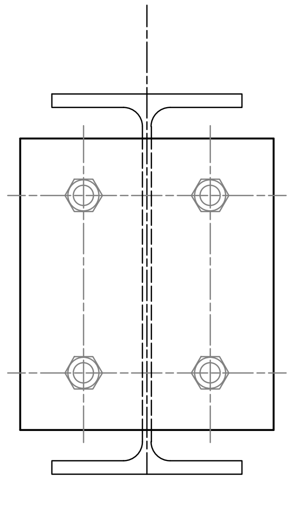
END PLATE
- tp = 6 mm
- e1 = 45 mm
- e2 = 50 mm
- p1 = 140 mm
- w = 100 mm
- Steel Quality: S235

BOLTS
- type: 4x M16
- quality: 8.8

RESISTANCE
- VRd;EC3 = 222 kN
- VRd;model = 220 kN



VARIANT A, SIDE VIEW



VARIANT A,
CROSS-SECTIONAL VIEW

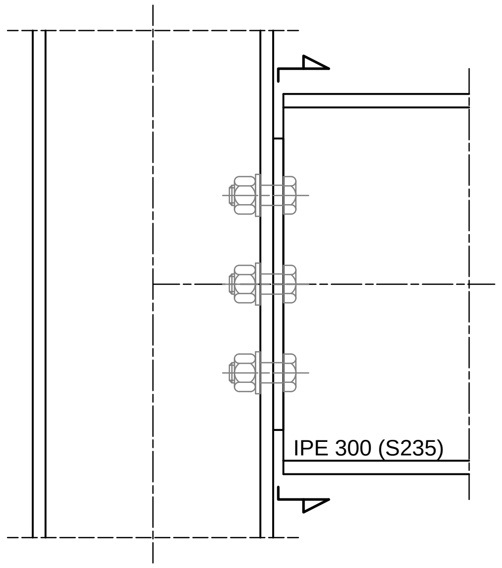
VARIANT B

BEAM: IPE300
- h = idem
- b = idem
- tw = idem
- tf = idem
- r = idem
- A = idem
- Steel Quality: idem

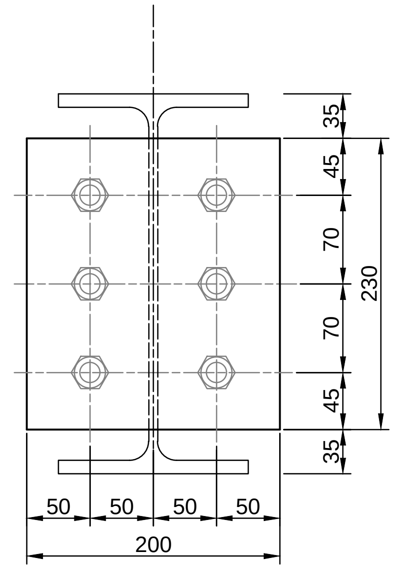
END PLATE
- tp = idem
- e1 = idem
- e2 = idem
- p1 = 70 mm
- w = idem
- Steel Quality: idem

BOLTS
- type: 6x M16
- quality: 8.8

RESISTANCE
- VRd;EC3 = 222 kN
- VRd;model = 250 kN



VARIANT B, SIDE VIEW



VARIANT B,
CROSS-SECTIONAL VIEW

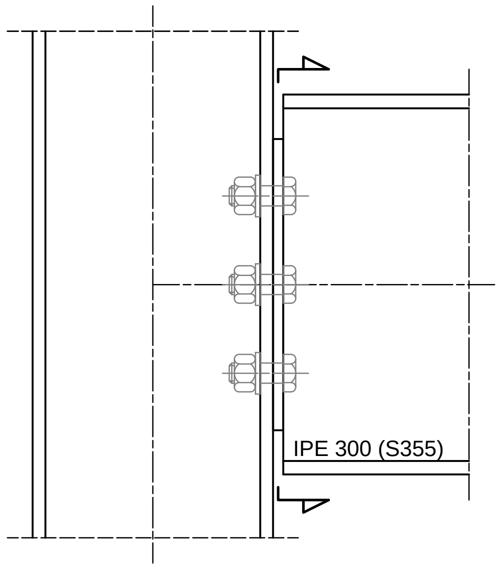
VARIANT C

BEAM: IPE300
- h = idem
- b = idem
- tw = idem
- tf = idem
- r = idem
- A = idem
- Steel Quality: S355

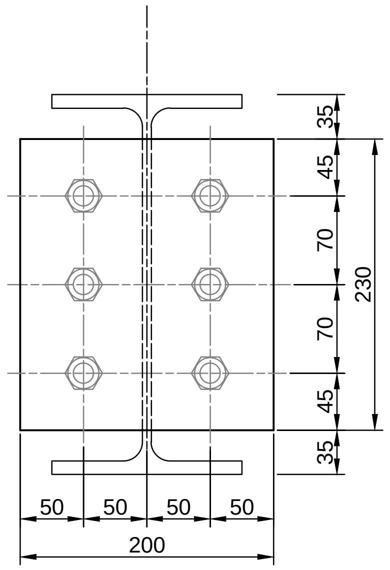
END PLATE
- tp = idem
- e1 = idem
- e2 = idem
- p1 = 70 mm
- w = idem
- Steel Quality: idem

BOLTS
- type: 6x M16
- quality: 8.8

RESISTANCE
- VRd;EC3 = 287 kN
- VRd;model = 340 kN



VARIANT C, SIDE VIEW



VARIANT C,
CROSS-SECTIONAL VIEW

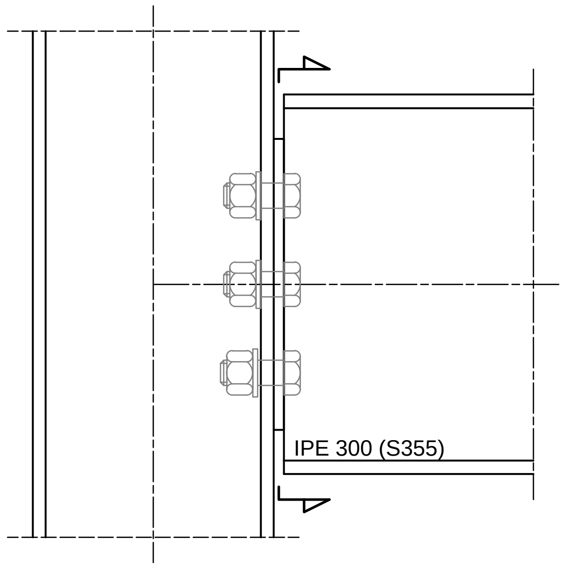
VARIANT D

BEAM: IPE300
- h = idem
- b = idem
- tw = idem
- tf = idem
- r = idem
- A = idem
- Steel Quality: S355

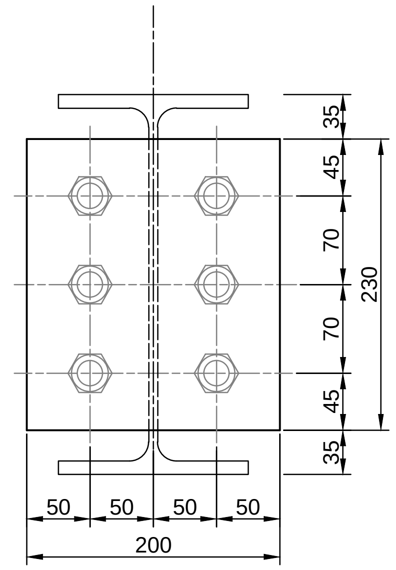
END PLATE
- tp = idem
- e1 = idem
- e2 = idem
- p1 = 70 mm
- w = idem
- Steel Quality: S355

BOLTS
- type: 6x M20
- quality: 8.8

RESISTANCE
- VRd;EC3 = 335 kN
- VRd;model = 370 kN



VARIANT D, SIDE VIEW



VARIANT D,
CROSS-SECTIONAL VIEW

VARIANTS FOR PARAMETRIC STUDY, MOMENT RESISTING JOINTS

VARIANT A

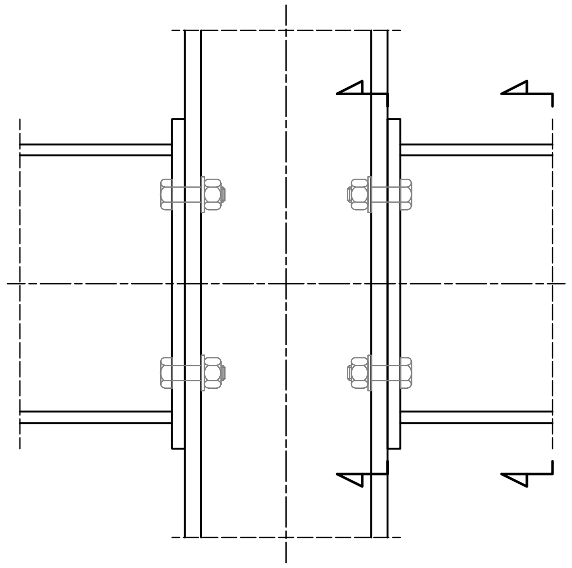
COLUMN: HEB160, S235
- h = 160 mm
- b = 160 mm
- tw = 8 mm
- tf = 13 mm
- r = 15 mm
- A = 5430 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

BEAM: IPE220
- h = 220 mm
- b = 110 mm
- tw = 5.9 mm
- tf = 9.2 mm
- r = 12 mm
- A = 3340 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

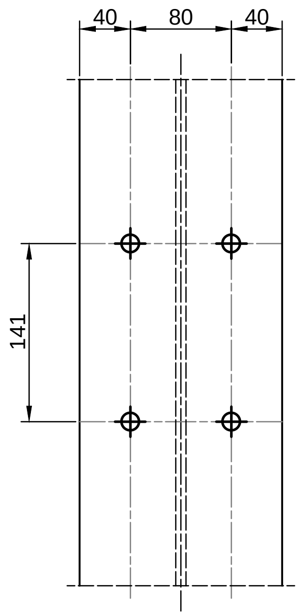
END PLATE (FLUSH), S235
- tp = 10 mm
- fy = 235 N/mm2
- fu = 360 N/mm2

WELDS
- Full Strength
- a = 5 mm

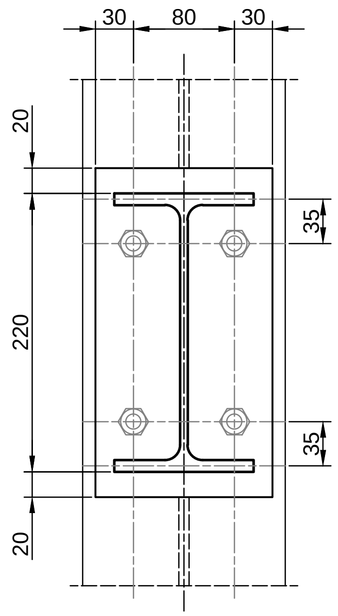
BOLTS
- type: 2x M12
- quality: 8.8
- fub = 800 N/mm2
- As = 84 mm2



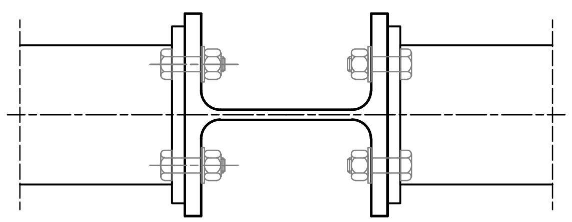
VARIANT A, SIDE VIEW



VARIANT A, CROSS-SECTION
COLUMN



VARIANT A, CROSS-SECTION
BEAM SECTION



VARIANT A, TOP VIEW

VARIANT B

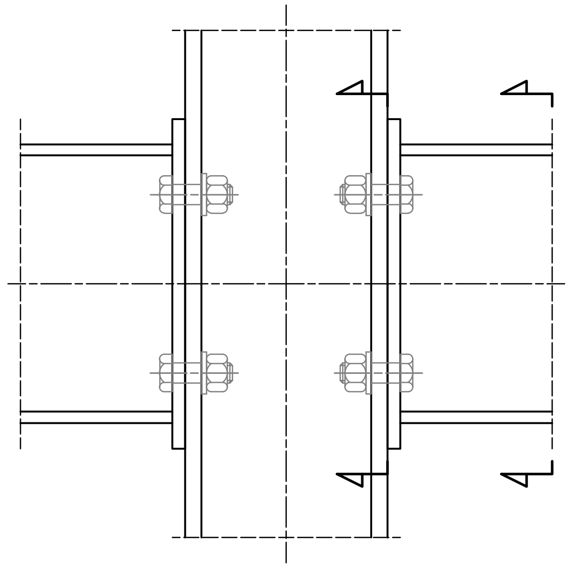
COLUMN: HEB160
- h = 160 mm
- b = 160 mm
- tw = 8 mm
- tf = 13 mm
- r = 15 mm
- A = 5430 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

BEAM: IPE220
- h = 220 mm
- b = 110 mm
- tw = 5.9 mm
- tf = 9.2 mm
- r = 12 mm
- A = 3340 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

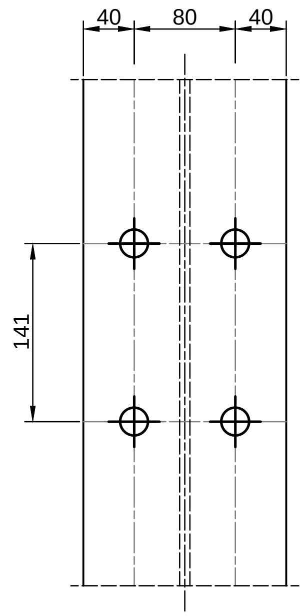
END PLATE (FLUSH), S235
- tp = 10 mm
- fy = 235 N/mm2
- fu = 360 N/mm2

WELDS
- Full Strength
- a = 5 mm

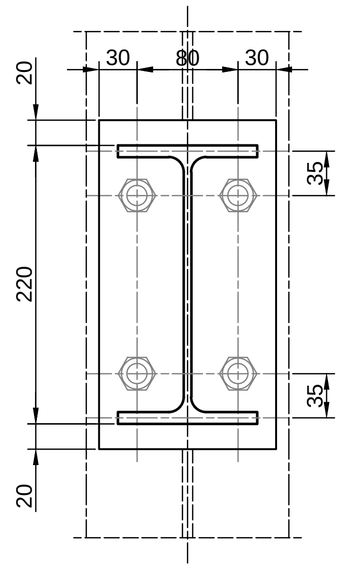
BOLTS
- type: 2x M20
- quality: 8.8
- fub = 800 N/mm2
- As = 245 mm2



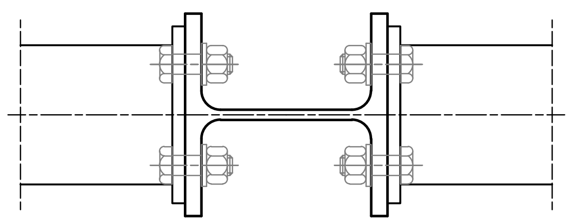
VARIANT B, SIDE VIEW



VARIANT B, CROSS-SECTION
COLUMN



VARIANT B, CROSS-SECTION
BEAM SECTION



VARIANT B, TOP VIEW

VARIANTS FOR PARAMETRIC STUDY, MOMENT RESISTING JOINTS

VARIANT C

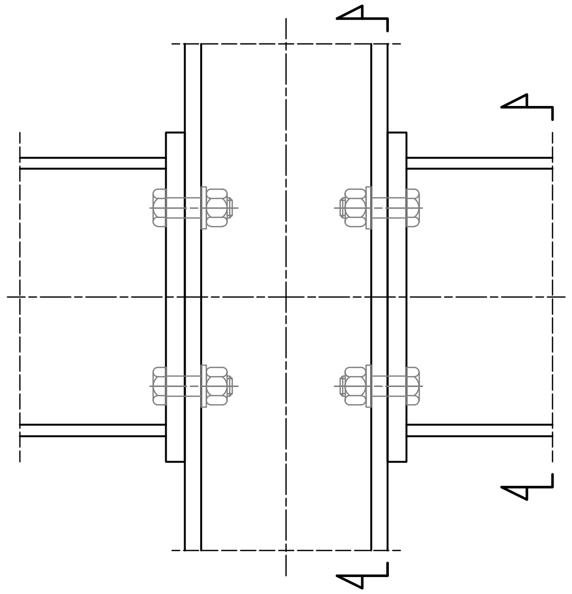
COLUMN: HEB160, S235
- h = 160 mm
- b = 160 mm
- tw = 8 mm
- tf = 13 mm
- r = 15 mm
- A = 5430 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

BEAM: IPE220, S235
- h = 220 mm
- b = 110 mm
- tw = 5.9 mm
- tf = 9.2 mm
- r = 12 mm
- A = 3340 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

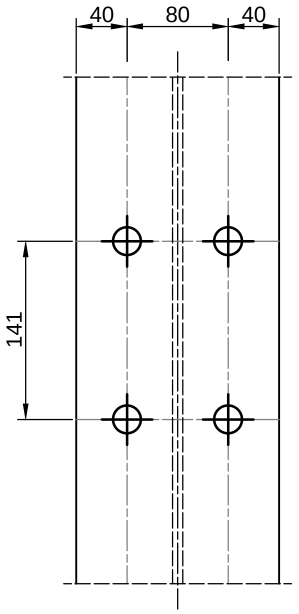
END PLATE (FLUSH), S235
- tp = 15 mm
- fy = 235 N/mm2
- fu = 360 N/mm2

WELDS
- Full Strength
- a = 5 mm

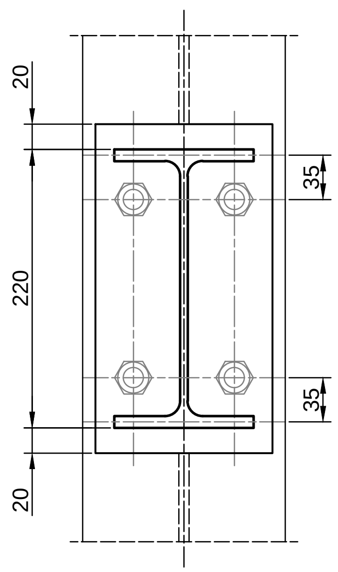
BOLTS
- type: 2x M20
- quality: 8.8
- fub = 800 N/mm2
- As = 245 mm2



VARIANT C, SIDE VIEW



VARIANT C, CROSS-SECTION
COLUMN



VARIANT C, CROSS-SECTION
BEAM SECTION

VARIANT D

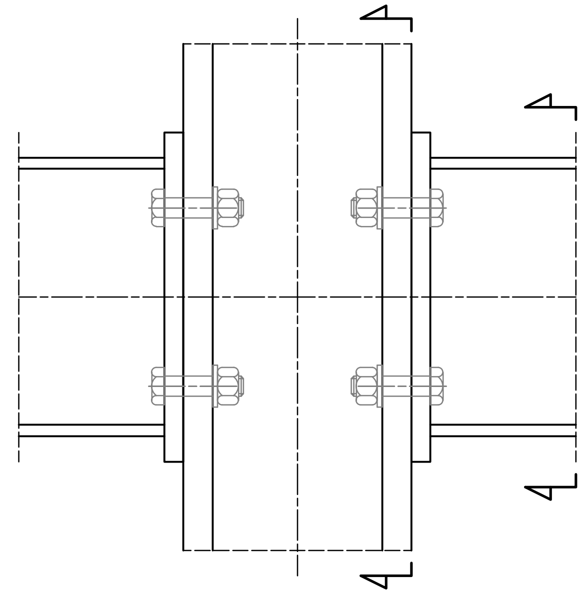
COLUMN: HEB160, S235
- h = 180 mm
- b = 166 mm
- tw = 14 mm
- tf = 23 mm
- r = 15 mm
- A = 9710 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

BEAM: IPE220, S235
- h = 220 mm
- b = 110 mm
- tw = 5.9 mm
- tf = 9.2 mm
- r = 12 mm
- A = 3340 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

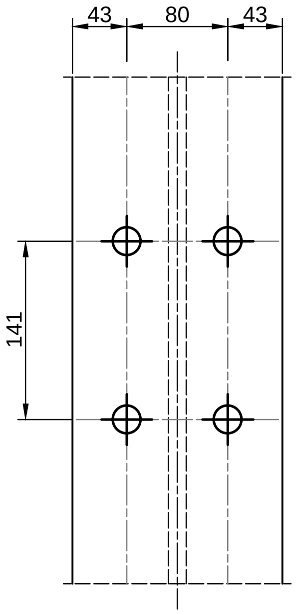
END PLATE (FLUSH), S235
- tp = 15 mm
- fy = 235 N/mm2
- fu = 360 N/mm2

WELDS
- Full Strength
- a = 5 mm

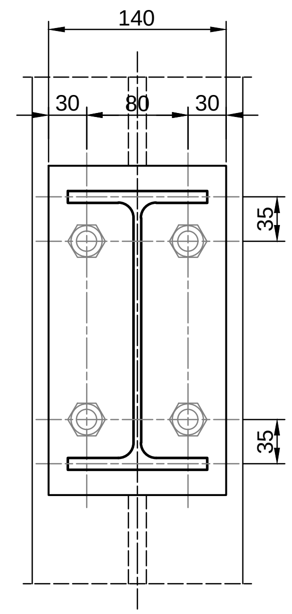
BOLTS
- type: 2x M20
- quality: 8.8
- fub = 800 N/mm2
- As = 245 mm2



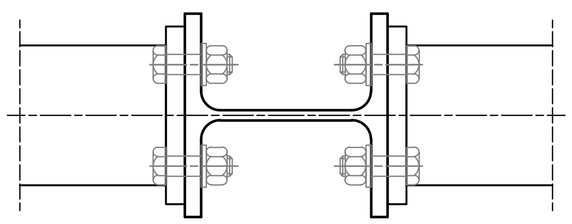
VARIANT D, SIDE VIEW



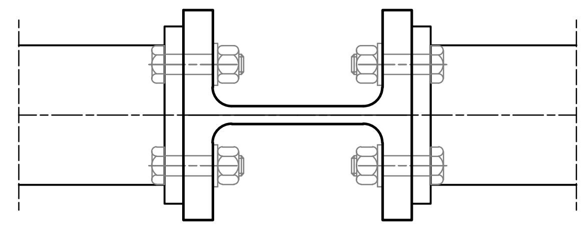
VARIANT D, CROSS-SECTION
COLUMN



VARIANT D, CROSS-SECTION
BEAM SECTION



VARIANT C, TOP VIEW



VARIANT D, TOP VIEW

VARIANTS FOR PARAMETRIC STUDY, MOMENT RESISTING JOINTS

VARIANT E

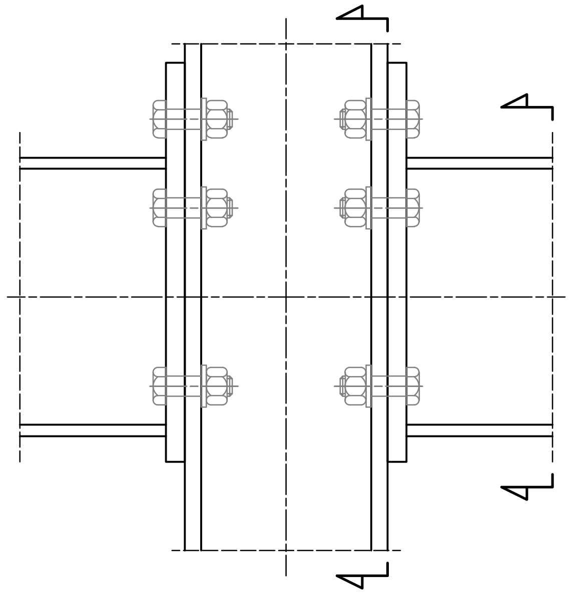
COLUMN: HEB160, S235
- h = 160 mm
- b = 160 mm
- tw = 8 mm
- tf = 13 mm
- r = 15 mm
- A = 5430 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

BEAM: IPE220, S235
- h = 220 mm
- b = 110 mm
- tw = 5.9 mm
- tf = 9.2 mm
- r = 12 mm
- A = 3340 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

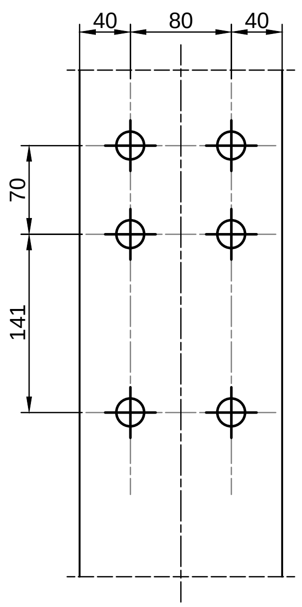
END PLATE (FLUSH), S235
- tp = 15 mm
- fy = 235 N/mm2
- fu = 360 N/mm2

WELDS
- Full Strength
- a = 5 mm

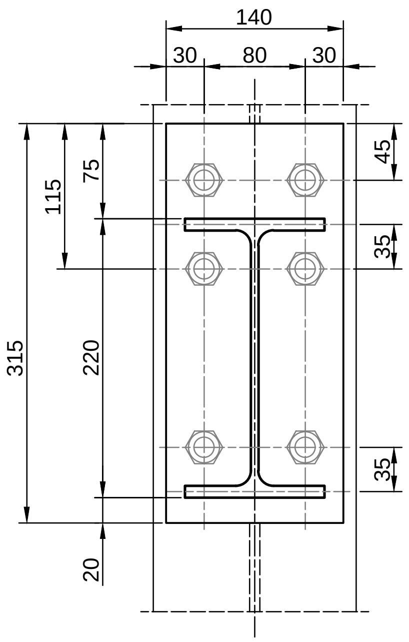
BOLTS
- type: 4x M20
- quality: 8.8
- fub = 800 N/mm2
- As = 245 mm2



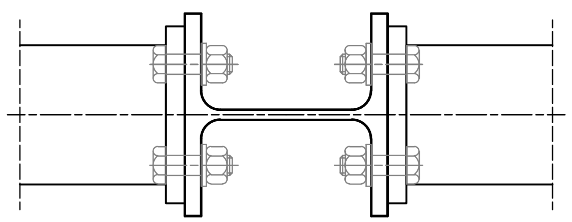
VARIANT E, SIDE VIEW



VARIANT E, CROSS-SECTION
COLUMN



VARIANT E, CROSS-SECTION
BEAM SECTION



VARIANT E, TOP VIEW

VARIANT F

COLUMN: HEB160, S235
- h = 160 mm
- b = 160 mm
- tw = 8 mm
- tf = 13 mm
- r = 15 mm
- A = 5430 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

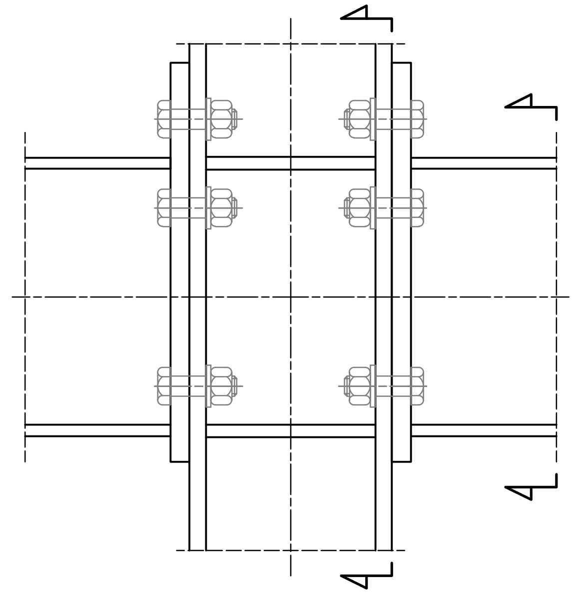
BEAM: IPE220, S235
- h = 220 mm
- b = 110 mm
- tw = 5.9 mm
- tf = 9.2 mm
- r = 12 mm
- A = 3340 mm2
- fy = 235 N/mm2
- fu = 360 N/mm2

END PLATE (FLUSH), S235
- tp = 15 mm
- fy = 235 N/mm2
- fu = 360 N/mm2

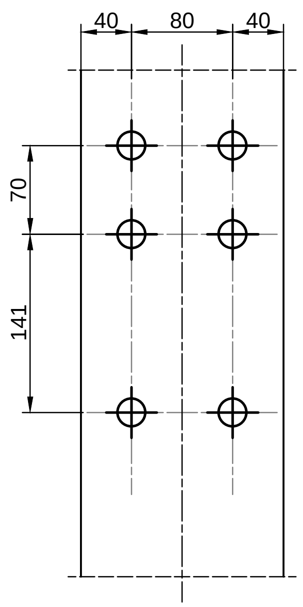
WELDS
- Full Strength
- a = 5 mm

BOLTS
- type: 4x M20
- quality: 8.8
- fub = 800 N/mm2
- As = 245 mm2

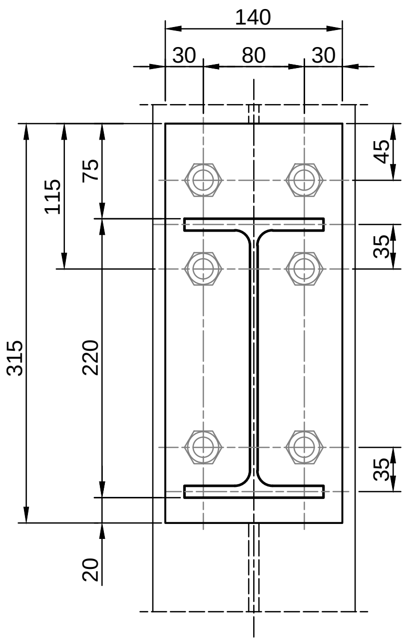
STIFFNER, S235
- ts = 10 mm
- fy = 235 N/mm2
- fu = 360 N/mm2



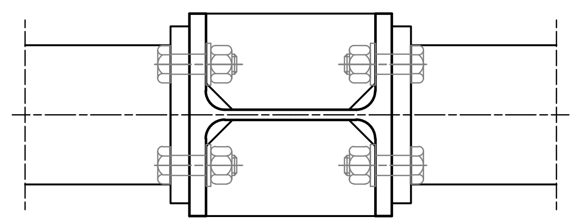
VARIANT F, SIDE VIEW



VARIANT F, CROSS-SECTION
COLUMN



VARIANT F, CROSS-SECTION
BEAM SECTION



VARIANT F, TOP VIEW