BOLTED END-PLATE JOINTS FOR CRANE BRACKETS AND BEAM-TO-BEAM CONNECTIONS

PhD Dissertation

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Abstract

The subject of this dissertation is the analysis of innovative bolted end-plate joints. Experimental and numerical studies on end-plate connections are performed, and design methods are developed for “non standard” joint details used in the industry.

The industry needs design solutions which are cheaper, lighter and easier to erect or have higher capacities. However, in most of the cases, these innovative solutions cannot be designed according to standard design methods. In such cases the “designer” has the option to test the new design solution and measure its parameters (failure mode, load capacity, deformations, ductility, fatigue behaviour, etc.). Performing these tests required significant expenses and they provide results only for the tested arrangement i.e. geometry and load history. For this reason the aim is to develop a test based design model, which is able to extend the experimental results.

The research activity began in 1994 in Cottbus, Germany on the Department of Steel Structures where the experimental tests were performed and continued in 2002 at the Department of Structural Engineering in Budapest with completing experimental and analytical studies on the field of bolted end-plate connections. Within the confines of these studies tests were performed to determine the static and fatigue behaviour of different types of joints as crane brackets and beam-to-beam joints. The purpose was to check existing design methods, and refine and/or modify them according to the needs of design practice.

The dissertation presents two different test series with different joint arrangements. One part of the experimental programme (light crane bracket joints) was completed in the laboratory of the Brandenburg Technical University of Cottbus in Germany, whereas the other test series (beam-to-beam joints) was performed in the laboratory of the Budapest University of Technology and Economics.

The experimental part have been designed so as to determine the failure mode and the joint behaviour. The aim of the analytical study is to develop design methods and to calculate the failure mode, the load bearing capacity and stiffness of the tested joints. The developed methods are to be verified and their accuracy is to be checked against the results of the tests completed.

The developed design methods are principally based on the experimental results and the analytical and numerical studies are used in an interaction with the experiments.
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1 Introduction

1.1 Background of the research

In the following dissertation tests and design methods are presented on bolted end-plate joints. Because of the wide range of this topic the dissertation focuses on two typical joint types, i.e. crane bracket joints and beam-to-beam joints. This chapter gives an outline of the structural problems and the strategy applied in the research work.

The tested joints present innovative solutions that have been initiated by the industry and optimized for considerations related to manufacturing, erection and durability. The load bearing capacity and behaviour of these innovative joint types are not tested as they are published in the national and international literature.

Up to recently, the design of bolted joints, due to their complex behaviour and the wide variety of their arrangement (bolt number and arrangement, end-plate thickness, joint arrangement, stiffness, etc.), has only been possible by making rude approximations to the safe side. Current design standards, including the Eurocodes, offer more accurate calculation models (cf. the model behind the Eurocode 3’s component method) that consider the effect of various components of the joint upon its ultimate load-bearing capacity. An advantage of such models is that, in most cases, they are able to reflect the consequences of modification in the joint arrangement during the design process, and therefore give the freedom to the designer to choose the final layout which best suits the relevant internal forces and moments as well as the applicable geometrical constraints.

The needs of the industry, however, tend to go beyond typical arrangements covered by design standards. In some cases, if the designer wishes to justify his joint concept, he needs to redesign it so as to achieve an arrangement preferred by the standard. This approach may, in the worst case, require a modification of the structural dimensions. An alternative is to calculate the ultimate load of standardised arrangements, and then to apply such arrangements up to certain levels of internal forces and moments. The disadvantage of this latter approach is that such standardised arrangements are fixed and no alterations are possible.

While beam-to-beam joints in structures are predominantly subjected to static loading, crane brackets are usually subject to fatigue load. The different joint types require different load histories for the tests, and the applied histories were chosen according to the joint type.

In the case of both tested joint arrangements (crane bracket joints and beam-to-beam joints), an important aspect is to create a simple-to-use calculation method. The dissertation presents easy-to-use design methods that have been verified by test results for the investigated joint types.

1.1.1 Crane bracket joints

Cranes, and therefore crane brackets, have become essential in today’s industrial buildings. This calls for an interest in their efficient design.

The primary trend is that while material costs are decreasing, payroll costs are growing. There is a logical way of achieving better structural solutions of crane brackets.

The traditional crane bracket and joint arrangement is shown in Figure 1.1. The bracket is made of an I section, and the joint is conventionally reinforced with transversal stiffeners. This joint arrangement ensures high load bearing capacity and can be designed without any problem.

1.1.2 Beam-to-beam joints

Steel industrial and agricultural halls as well as multi-storey steel buildings, which are widely used in today’s Europe, are almost exclusively designed to involve beam-to-column and beam-to-beam joints with bolted end-plates. Bolted solutions are easier to install (and therefore cheaper) and faster to build than their welded counterparts.

Figure 1.2 shows the typical application field of this joint type. The traditional bolt arrangements reflect those supported by the standards, i.e. those that contain two bolts in each bolt row.
1.2 Previous studies
1.2.1 Experimental studies

Until the end the 80s the following fields were in the focus of the researchers: experimental load bearing capacity studies on different kind of joints and bolt arrangements, experimental joint behaviour studies under cyclic loading, basic research on T-stubs. Tests were carried out and theories were born all over the world by researchers such as Piazza & Turrini (1989), Lacher (1987), Thiele & Reuschel (1989), Aribert et al. (1989), Nethercot et al. (1988), Olös et al. (1989), Kato (1989).

In the 90s the joint components (bolts, welds and the compression zone), the global behaviour of semi-rigid connections and the rotation capacity were substantially analyzed. Furthermore, numerous tests were carried out to determine the joint behaviour under cyclic/seismic loading. New research fields are joints of hollow sections (RHS and CHS) and joints with cold-formed cross-sections. A brief list of some well-known researchers who worked on this topic would include Aribert & Lachal (1992), Krenk & Damkilde (1990), Sedlacek et al. (1994), Berussi et al. (1995).

Hungarian researchers have also completed tests to describe the joint behaviour. Research on tension bolted connections has started in the beginning of the 70s in the laboratory of the Department of Steel Structures of the TUB. Important work in this field has been done by Halász & Iványi (1979) and Iványi & Szabó (1989). These investigations focused on tension bolted joints and on T-stubs. Some tested joints were loaded under cyclic loading due to practical purposes. Results of this experimental analysis helped in drafting the relevant national design specification in a more accurate way. Hungarian researchers have also taken part in international research projects which aimed at improving the design methods under cyclic loading, as have been published by Dunai (1994). This research includes experimental, numerical and analytical approaches to characterize the hysteretic behaviour of the joint and its structural components. In relation to the monotonic and cyclic behaviour of bolted end-plate joints important work has been done by Dunai (1996), Ádány (2000) and Kovács (2005).

1.2.2 Analytical study and modelling

End-plate connection design has been the subject of numerous studies since the early 60s. Douty and McGuire presented in 1965 a method to determine the load carrying capacity of end-plates that took into consideration the prying force effect. As this procedure was too complicated for practical use, the aim of the next research was to develop a simple model to determine the load bearing capacity of the end-plate connection. Important work has been done in this field by Agerskov (1976), Krishnamurty (1980), Mann & Morris (1979) and Grundy et al. (1980). A very refined approach to this problem was presented by Zoetemeijer (1974).

The principles of the component method are based on Zoetemeijer’s work. Later, other researchers worked on this method to determine the mechanical properties of further components and to refine the calculation methods (Brozzetti, Nethercot, Tschemmernegg, Zandonini), in order to obtain more accuracy in the description of the mechanical behaviour.

Furthermore, many tests were carried out to validate different connection configurations. Some examples: effect of the use the Huck-Fit bolts (Aribert et al. 1994), composite connections (Nethercot 1991, Tschemmernegg 1992, Aribert et al. 1994), connection in thin-walled lattice girders (Damkinde & Krenk 1994), minor-axis joints (Gomes et al. 1994), multiplanar connection between l-beams and RHS column (Lu & Wardener 1995), multiplanar l-beam to tubular column connection (Winkel & Wardener 1995), double clip angle connections (Bursi 1990), welded RHS connections (Zhao & Hancock 1995).

The accuracy of the component method depends on the accuracy of the description of the mechanical components and on the quality of the assembling process. It is assumed that the properties of the individual components are independent from each other. However, some components do not act independently but influence each other. For hand calculation this can be accounted for in a simplified way only, because the general approach results in a complicated iterative calculation procedure.

From the 90s until today the use of the FE modelling has become more and more important. While previously individual components or joints were modelled only, nowadays researchers use FE models to carry out virtual experiments with complete structures (Komuto 2004, Vigh 2006).

More theoretical research has also been done, for example Dunai & Hegedűs (1989) looked at the numerical analysis of high strength end-plate joints using a newly developed computation method.
1.2.3 Design methods and standards

In the 70s joints were designed either as pinned or as rigid and full strength. Considerable work on connection behaviour was completed in the field of design methods in the last thirty years. The concepts of semi-rigid design and partial strength design have been developed in order to simulate more accurately the true behaviour of connections. As a result of the extended international research standard design models have been developed and implemented in design codes such as the Eurocode 3, which take into account the semi-rigidity and partial strength nature of the bolted end-plate joints. These recommendations allow to calculate the strength \( M_{j,Rd} \), rotational stiffness \( S_j \) and deformation capacity \( \Phi_j \) of moment resistant joints. This method for the determination of the mechanical properties of the joint is the component method. Recommendations for the assessment of the strength, stiffness and deformation capacity of each component are given in EN 1993-1-8, Eurocode 3.

Table 1.1 provides a brief historical overview of the development of the component method in Eurocode 3.

<table>
<thead>
<tr>
<th>Year of issue</th>
<th>Title of standard</th>
</tr>
</thead>
</table>

The Eurocode 3 (henceforth EC3) model is calibrated with static test results and has the following scope:

"(1) This part of EN 1993 gives design methods for the design of joints subject to predominantly static loading using steel grades S235, S275, S355 and S460." - EN 1993-1-8 : 2005 (E)

The concept of "predominantly static loading" means that:
- dynamic effects have no influence on the load capacity, and
- the repeated load does not cause a fatigue failure.

This definition comes from those times when beams were designed for the elastic range of loads only. In the German code for crane runways, DIN 4132, a load is defined as "predominantly static" if it consists of \( 2 \times 10^5 \) or less load cycles. Joints subjected to such loads do not need to be designed for fatigue.

In some cases, when moderate plastic deformations may develop, fatigue failure may occur at a lower number of load cycles. Construction details under high local stresses, for example bolted connections, require particular care. The full scale tests served to study the possibilities of fatigue failure and the range of use of bolted end-plate connections under repeated loading.

1.2.4 Summary of the previous studies

Numerous types of joints are being studied experimentally, but these investigations do not cover welded crane bracket joints without compression flange such as illustrated in Figure 1.3. Some design recommendations exist for joints with four bolts in a row. But there has not been found any design method that would describe a configuration that mixes two and four bolts in a row, such as shown in Figure 1.4.

The innovation in the case of the bracket joints is the welded
bracket cross-section, and in the case of the beam-to-beam joints, the bolt arrangement and the additional stiffeners. The benefits of such innovative solutions are well-known. For brackets: easier installation (by ensuring better access to the bolts); lower self-weight (by the omission of one of the flanges) and less welds (the principal benefit for the manufacturing company is the save on hand welding). For the four-bolts-in-one-row type end-plate joints as well as for joints with additional stiffeners; higher resistance of the same end-plate geometry, i.e. same beam cross-section. At the same time, the complex behaviour of this joint type makes the design difficult.

The design methods to be developed and the coefficients to be introduced need to come from and be consistent with the chosen standard.

Because of the very nature of standards the Eurocode cannot describe all joint varieties with all bolt arrangement possibilities. The component method model of the EC3 needs to be improved and adjusted to the tested joint arrangements (the light crane bracket and the HammerHead type structural joint as presented in Table 1.2 b.) and to the non-standard bolt arrangement (four bolts in one row type end-plate joints).

1.3 Purpose and scope

1.3.1 Problem statement

The subject of the thesis is to analyse the innovative bolted end-plate joints shown in Figure 1.5 and Table 1.2. The presented research programme looks at various bracket shapes and joint arrangements and examines both the static and the fatigue behaviour of the different arrangements.

Figure 1.5 shows the studied bracket arrangements. The test specimens Z1 and Z2 are conservative brackets with I cross-sections, whereas the so-called light crane brackets, specimens K1, K2_z, K2 and K3, have a cross-section without compression flange.

In the research beam-to-beam joints that are commonly used in the industry are studied, as shown in detail in Table 1.2. The presented study analyses the load bearing capacity and the bolt load distribution of the examined joint arrangements. The calculation/modelling difficulties are presented in Table 1.2.

Table 1.2 Investigated beam-to-beam joint details.

<table>
<thead>
<tr>
<th>a.) standard joint arrangement</th>
<th>b.) HammerHead joint arrangement</th>
<th>c.) joint with four bolts in one row</th>
<th>d.) HammerHead joint arrangement and joint with four bolts in one row</th>
<th>e.) joint with four bolts in one row and an additional stiffener in the first bolt-row</th>
</tr>
</thead>
<tbody>
<tr>
<td>The joint can be designed, the EC3 component model method can be used, without adjustment.</td>
<td>In the extended tension zone the end-plate has two bolt-rows and an additional flange.</td>
<td>In the end-plate the bolt-rows contain four bolts in the first and second bolt-rows.</td>
<td>The design problems indicated in b.) and c.) are combined in this joint.</td>
<td>The design problem indicated in c.) and an additional stiffener in the first (extended) bolt-row.</td>
</tr>
</tbody>
</table>
1.3.2 Purpose of the research

The purpose of the research is to perform experimental and analytical studies on innovatively designed bolted end-plate joints. The innovative joint arrangement in this case is a new type of joint with unknown behaviour and unknown design.

The experimental part has been designed so as to determine the failure mode and the joint behaviour. The aim of the theoretical study is to develop design methods compatible with the EC3 based on the standardized component method model, and to calculate the failure mode, the load bearing capacity and stiffness of the tested joints. The developed methods are to be verified and their accuracy is to be checked against the results of the tests completed.

Crane bracket joints

More specifically, the purpose of the experimental and analytical studies is as follows:
- To determine the failure mode and the load bearing capacity of different designs of crane bracket joints under static loading.
- To characterize the effect of different bolt diameters on the failure mode, load bearing capacity and fatigue behaviour of the joints.
- To study the behaviour of the different crane bracket arrangements under fatigue loading and determine the effect of the various components on the fatigue behaviour.
- To study the stiffness degradation of different end-plate arrangements.
- To create an EC3 compatible design model for crane bracket joints without compression flange. The model has to be able to calculate the load bearing capacity and the stiffness of the joint.

Beam-to-beam joints

More specifically, the purpose of the study on beam-to-beam joints is as follows:
- To determine the failure modes and the load bearing capacity of the joints.
- To study the load-deformation behaviour of the end-plate.
- To study the load distribution in the bolt rows in different joint arrangements.
- To validate experimentally the design methods for bolted end-plate joints which are differently designed from the EC3 “standard” joints, i.e. HammerHead type structural joints and four bolts in one row type end-plate joints.

1.3.3 Research strategy

The research should start at the level of physical phenomena and should arrive at practically applicable design information. Due to these requirements interacting experimental, analytical and numerical research tools had to be used. The principle of the research strategy is shown in Figure 1.6.

The experiments and the derived results have a fundamental role and give the basis to the design method. Therefore, as a first step, an experimental programme was designed and completed. In this programme a total of 38 full scale tests were carried out. These included six different crane bracket joint arrangements and eight different beam-to-beam joint arrangements. The tests on the brackets were completed in the laboratory of the Brandenburg Technical University of Cottbus in Germany, whereas the experiments on the beam-to-beam joints were done in the laboratory of the Budapest University of Technology and Economics.

The developed design methods are principally based on the experimental results. The analytical and numerical studies are used in an interaction with the experiments. In the context of the analytical model, in this case, the EC3 component model method gives a framework and the numerical model mines a non-linear 3D shell-element FE model. The design methods provided by the research harmonise with the EC3 standard.
2 Crane bracket joints

2.1 Research programme

The dissertation presents the details and results of an experimental study on bolted crane bracket joints of industrial type steel buildings that involved both monotonic and fatigue loading. The experimental programme included twenty full scale specimens and covered six different bracket arrangements. The various bracket shapes and details are shown in Figure 1.5 All test specimens were industrial made and the crane brackets were connected to the column by end-plate type bolted connections.

2.1.1 Test specimens

The tested specimens with their main dimensions are shown in Figure 2.1. The detailed geometry of the test specimens is presented in Appendix A.

![Fig. 2.1 Test specimens.](image)

The following dimensions were identical for all tested brackets: column height (1,770 mm); free bracket length (470 mm); end-plate thickness (16 mm) and the application point of the load i.e. the lever arm (250 mm), on the crane bracket.

The steel grade of all the specimens was S355 and the bolt grade was 10.9 with a pre-load according to DIN 18 800. The pre-load was 160 kN and 220 kN for M20 and M24 bolts, respectively. No special treatment was applied to the contact surfaces, i.e. they were not prepared as in the case of joints where slip-resistance at ultimate limit state is required. In the examined connection type the bolts were loaded predominantly under tension.

In all cases, the brackets were connected to the column by three bolt rows. All welds on the test specimens were double-side fillet welds. Table 2.1 shows a summary of the testing programme.
Table 2.1 Testing programme.

<table>
<thead>
<tr>
<th>test specimen</th>
<th>bracket arrangement</th>
<th>backing plate</th>
<th>test with bolt diameter M20</th>
<th>test with bolt diameter M24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>traditional bracket</td>
<td>yes</td>
<td>-</td>
<td>static and fatigue</td>
</tr>
<tr>
<td>Z2</td>
<td>traditional bracket</td>
<td>no</td>
<td>-</td>
<td>static and fatigue</td>
</tr>
<tr>
<td>K1</td>
<td>light bracket</td>
<td>yes</td>
<td>static and fatigue</td>
<td>static and fatigue</td>
</tr>
<tr>
<td>K2_z</td>
<td>light bracket</td>
<td>no</td>
<td>-</td>
<td>static and fatigue</td>
</tr>
<tr>
<td>K2</td>
<td>light bracket</td>
<td>yes</td>
<td>static and fatigue</td>
<td>static and 2 times fatigue</td>
</tr>
<tr>
<td>K3</td>
<td>light bracket</td>
<td>yes</td>
<td>fatigue</td>
<td>static and 3 times fatigue</td>
</tr>
</tbody>
</table>

Figure 1.5 outlines the test specimens. Specimens Z1 and Z2 were the traditional reference brackets with compression flange: Z1 with a tension stiffener in the column side and Z2 without stiffener but with backing plates.

The specimens identified by the letter “K” are light crane brackets without compression flanges. Specimen K2 was in all dimensions identical to specimen Z1 with the only exception that the profile of the bracket was different, as shown in Figure 1.5.

Specimen K2_z was a simplified version of K2 with the compression stiffener and backing plates of the column omitted.

Test specimens K1 and K3 have the same joint arrangement as specimen K2. These joints were stiffened in the tension zone with backing plates and with a transversal stiffener in the compression zone as shown in Figure 1.5.

Plate dimensions of brackets and columns varied over the tests. More information about plate dimensions can be found in Appendix A.

2.1.2 Test arrangement

A test frame was designed and used so as to find a simple arrangement which was flexible enough (i.e. easy to install and suitable for all tests) to allow the testing of all specimens under all load histories. Figure 2.2 shows the test frame used in most static and all fatigue tests. Figure 2.3 shows the pinned support on the top of the test specimen and the connection of the diagonal stiffener.

Two of the static tests were carried out with another test arrangement as shown in Figure 2.4. These tests were performed with specimens K1 and K2 respectively, using M20 bolts in both cases. The testing machine, a four column material testing machine, with a maximum load capacity of 1,000 kN, was built by TONI Baustoffprüfsysteme Co., Berlin. The maximum dimensions of the specimens were: length = 6.0 m; width = 3.0 m; height = 3.0 m.
The main difference between the test arrangements was the load application and the stability support.

During the TONI tests the 50 mm thick offset plate was not used and the bracket flange was not stiffened perpendicularly to the web plane as shown in Figure 2.5.

The offset plate and the lateral support of the bracket is shown in Figure 2.5.

In the test arrangement the column-bracket sub-assembly is modelled by a fixed column base and a pin at the top of the column. The bracket was loaded in the vertical axis of the crane girder by downward, and in the fatigue tests, by uplift forces using a loading system with one hydraulic actuator. In all tests representative displacements were measured by transducers and strain distribution was recorded using 12 gauges in average.

To prevent lateral torsional buckling of the bracket the construction was restrained by an additional plate as shown in Figure 2.5. The plate was linked to U-profiles, as shown in Figure 2.6 a.). This lateral support modelled the effect of the crane runway girder which effectively prevents lateral-torsional buckling.

To stiffen the test frame in the plan of the frame, two tubes were erected in the diagonal direction. One end of the tubes was fixed to the frame base, whereas the other was fixed to the frame at the level of the test column as shown in Figures 2.3 and 2.6.

The hydraulic actuator applied had a maximum capacity of 1,000 kN. To avoid local buckling in the bracket web or flange a thick plate (50 mm) was placed between the jack and the bracket flange, as shown in Figures 2.5 and 2.7. This plate simulated the flange of the runway girder.

In the middle of this plate an inductive transducer was placed in the vertical direction to measure the bracket deformations under the load, and in addition, gauges were placed on the bracket web.
Before the static tests the probable load capacities of the brackets were calculated with the developed design method and with FE simulations. The results of the FE calculations for the specimens Z2 and K3 showed that the calculated load bearing capacities were higher than 1,000 kN. However, both test arrangements had a maximum capacity of 1,000 kN. The task was to find a solution which ensured the same proportion of shear and moment in the joint and did not change the load application point. It would have been unappropriate to elongate the bracket and install a second actuator because this would have changed the moment/load ratio.

The solution was an auxiliary support as shown in Figure 2.8, where an additional load was exerted exactly in the axis of the 1,000 kN hydraulic jack. This support consisted of a 50 kN hydraulic jack, and therefore had a maximum load capacity of 250 kN. To ensure good coordination between the two jacks under the loading process they were displacement controlled.

During the fatigue tests the continuous beam effect was also taken into consideration. In case of continuous beams, there is an uplift as well as a downward force and this uplift was also simulated within the fatigue tests. For the tests an uplift load equal to 10% of the downward vertical load was assumed as explained in Figure 2.9.

### 2.1.3 Measuring system

During the static tests the data were collected and saved every second by an HBM DMC Lab plus (Hottinger Baldwin Messtechnik) data collection system. The used gauges were of the RY 41-6/120 and LY 11-6/120 type produced by HBM. Figure 2.10 shows the location of the gauges under monotonic loading.

In the load axis under the bracket an inductive transducer was placed (type IWT 402) with a maximum displacement capacity of 100 mm.

As an example Figure 2.10 shows the location of the gauges in the case of test specimen K2 with M24 bolts. For other specimens the location of the gauges can be found in Appendix A.
For the fatigue test the specimens were loaded with a frequency between 1 and 2 Hz. The same type of gauges was used as for the static tests. Figure 2.11 shows the location of the gauges under fatigue loading.

The inductive transducer was placed under the load application point within the axis of the hydraulic jack.

The implemented load spectrum curve of the fatigue tests is shown in Figure 2.12. The curve was chosen according to the recommendation of the DIN 15 018 standard. This curve was simplified by a four step approximation. The steps were chosen so as to achieve an easy control during the tests and in the post test evaluation.

The maximum fatigue load was equal to 70% of the measured static load bearing capacity.
2.1.4 Test programme

Monotonic loading

For the static tests different loading rates were applied within the limits set by the requirement of predominantly static loading. The applied values of the loading rates and the pre-load levels are presented in Table 2.2.

Table 2.2 Loading rate steps under monotonic loading.

<table>
<thead>
<tr>
<th>test specimen</th>
<th>pre-load</th>
<th>loading rates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>200 kN</td>
<td>180 kN/min (up 200 kN) 30 kN/min (up 600 kN) 2.67 kN/min (over 600 kN)</td>
</tr>
<tr>
<td>Z2</td>
<td>400 kN</td>
<td>40 kN/min (up 400 kN) 10 kN/min (up 800 kN) 4 kN/min (over 800 kN)</td>
</tr>
<tr>
<td>K1-M20</td>
<td>200 kN</td>
<td>15 kN/min (up 300 kN) 7.5 kN/min (over 300 kN)</td>
</tr>
<tr>
<td>K1-M24</td>
<td>200 kN</td>
<td>100 kN/min (up 100 kN) 15 kN/min (up 250 kN) 5.83 kN/min (over 250 kN)</td>
</tr>
<tr>
<td>K2_z</td>
<td>200 kN</td>
<td>constant 4.11 kN/min</td>
</tr>
<tr>
<td>K2-M20</td>
<td>200 kN</td>
<td>180 kN/min (up 200 kN) 30 kN/min (up 600 kN) 2.67 kN/min (over 600 kN)</td>
</tr>
<tr>
<td>K2-M24</td>
<td>200 kN</td>
<td>30 kN/min (up 400 kN) 6 kN/min (over 400 kN)</td>
</tr>
<tr>
<td>K3-M24</td>
<td>200 kN</td>
<td>33.3 kN/min (up 500 kN) 6.67 kN/min (up 900 kN) 4 kN/min (over 900 kN)</td>
</tr>
</tbody>
</table>

Fatigue loading

Under the first and second load steps (10 and 500 cycles respectively, as shown in Figure 2.12) the collection of data was continuous at 20 Hz. In steps three and four, however, there was too much data to handle. This was the reason to switch from continuous to sequential data collection, according to a rule shown in Figure 2.13.

During each load cycle measurements were taken at least 10 times so as to facilitate post test evaluation. That is, the double requirement of both a sufficient degree of accuracy at the evaluation stage and a reasonable extent of data collection have been achieved by a 20 Hz data collection system. This system, contrary to what was applied in the case of static tests, was not continuous; it was restricted to the collection of data within intervals distributed periodically within the timeframe of the test, see Figure 2.13. This system proved to be accurate enough from the point of view of post test evaluation, and at the same time, ensured a reasonable amount of data.

The fatigue tests simulated the continuous beam behaviour of the crane brackets. Therefore, the bracket flanges were subjected to both vertical downward loading and uplift (this latter equal to 10% of the downward load) as shown in Figure 2.9.
2.1.5 The design method

Before all static tests the failure mode, the load bearing capacity and the initial stiffness of the joints were calculated on the basis of the EC3 component model method modified for the particular case.

The component model method has the advantage of handling the effect of the components taken into account on the behaviour of the joint. This way the designer can find the best possible joint design and plate dimensions and overestimation of the joint resistance can be avoided. More about the EC3 model can be found in Appendix F “Summary of the Eurocode 3 model”.

The developed design method

The EC3 component model method was applied to calculate the failure mode, the design moment resistance \( M_{j,Rd} \) and the stiffness \( S_{j,I} \) of the light crane bracket joints. The standard assumes that both the column and the beam have I cross-sections. The tested brackets, however, do not have a compression flange, and they have a pentagonal web plate only, as shown in Figure 2.14 a).
location of the centre of compression one would know the exact stress distribution for all load levels in the joint. So for the purposes of the calculation it was assumed that before buckling, the centre of compression is situated at the height of the compression stiffener, as shown in Figure 2.16. The “real” centre of compression is closer to the flange if one analyzes the stress distribution in the web plate only. But the end-plate and the transversal stiffener on the column side modifies the location of this point, “pulls it down”. Along the same lines it was assumed that after buckling, the centre of compression is situated at the edge of the effective web plate, as shown in Figure 2.16.

These assumptions for the definition of the centre of compression were based on test results, and the comparison of the calculated and measured results showed that the results of the design method were always on the safe side.

- The bolts in the buckling zone have no influence on the joint moment resistance.
- The design moment resistance of the joint can be determined form the sum of the products involving the effective design tension resistances of the bolt-rows and the appropriate lever arms, as shown in Figure 2.17:

\[
M_{j,Rd} = \sum h_i \times F_{T,i,Rd}
\]  
(2.1)

where:
- \( h_i \) - is the distance from bolt-row \( i \) to the centre of compression
- \( F_{T,i,Rd} \) - is the effective design tension resistance of the bolt-row \( i \)
- \( i \) - is the serial number of the bolt-row \( i \)

Appendix H contains a calculation example according to EN 1993-1-8.

- The method calculates the shear resistance in the same way as for beam-to-column joints as defined in prEN 1993-1-1 6.2.8 Bending and shear. Regarding shear load transmission, fasteners in the buckling zone are also taken into account.
Calculation of the stiffness

The modification described below consists of the introduction of a new stiffness coefficient to be applied to the bracket web in compression, in a manner analogous to the case of a column web in compression.

- Looking first at an unstiffened web in compression, its stiffness coefficient can be calculated in the following steps:

  Equation (2.2) describes the way elastic resistance is calculated when there is a deformation $\Delta$ due to compression or tension, whereas equation (2.3) is a general representation of the resistance.

$$F_{el} = \frac{E \cdot t_{wc} \cdot \xi \cdot \Delta}{d_c}$$  \hspace{1cm} (2.2)

where:

- $E$ - is the elastic modulus
- $t_{wc}$ - is the column web thickness
- $\xi$ - is a coefficient to be calculated as the ratio of the stiffness of the column flange as regards moment to the stiffness of the web as regards compression/tension
- $\Delta$ - is the deformation of the web, as shown in Figure 2.18
- $d_c$ - is the depth of the column web, as shown in Figure 2.18

The plastic resistance is:

$$F_{pl} = b_{eff} \cdot t_{wc} \cdot f_y$$  \hspace{1cm} (2.3)

where:

- $b_{eff}$ - is the effective height of the column web
- $t_{wc}$ - is the column web thickness
- $f_y$ - is the yield strength of the web

Figure 2.18 shows the web deformations in the case of an unstiffened column as a result of a concentrated load, and Figure 2.19 illustrates the deformations of the web of light crane brackets.

Equation (2.2) can be modified by taking into account Hooke's law, i.e. $\Delta / d_c = \varepsilon_{el}$.

$$F_{el} = E \cdot \varepsilon_{el} \cdot t_{wc} \cdot \xi = \frac{E \cdot f_y}{E} \cdot t_{wc} \cdot \xi = f_y \cdot t_{wc} \cdot \xi$$  \hspace{1cm} (2.4)

For the sake of simplicity it will be assumed that the proportion between elastic and plastic resistance is 2/3. Then from equations (2.3) and (2.4) one can deduce:

$$\xi = \frac{2}{3} \cdot b_{eff}$$  \hspace{1cm} (2.5)
This is how the equation for the stiffness coefficient of an unstiffened web in compression given in EN 1993-1-8 6.3.2 Stiffness coefficients for basic joint components, Table 6.11 is obtained:

\[
k_{2,c} = \frac{0.7 \cdot b_{\text{eff}, wc} \cdot t_{wc}}{d_c}
\]

(2.6)

where:
- \(b_{\text{eff}, wc}\) is the effective width of the column web in compression (EN 1993-1-8 6.2.6.2 Column web in transverse compression)
- \(t_{wc}\) is the column web thickness
- \(d_c\) is the depth of the column web

By analogy with the column web in compression, a new stiffness coefficient was introduced in the design method which considers the stiffness of the bracket web in transverse compression, as shown in Figure 2.19.

\[
k_{2,\text{bracket}} = \frac{0.7 \cdot h_{\text{buckling}} \cdot t_{wb}}{d_{\text{eff}, wb}}
\]

(2.7)

where:
- \(h_{\text{buckling}}\) is the effective compression length of the bracket web
- \(t_{wb}\) is the thickness of the bracket web
- \(d_{\text{eff}, wb}\) is the effective web width of the bracket

The effective web width of the bracket \((d_{\text{eff}, wb})\) can be calculated from the plate geometry and from the effective length of the compression zone \((h_{\text{buckling}})\).

- According to EN 1993-1-8 6.3 Rotational stiffness, the rotational stiffness of a joint is:

\[
S_j = \frac{E \cdot z^2}{\mu \cdot \sum_{i} k_i}
\]

(2.8)

where:
- \(k_i\) is the stiffness coefficient for basic joint component \(i\)
- \(z\) is the lever arm
- \(\mu\) is the stiffness ratio

The stiffness ratio \((\mu)\) should be determined as follows:

- if \(M_{j,Ed} \leq 2/3 M_{j,Rd}\) then \(\mu = 1.0\)
- if \(2/3 M_{j,Rd} < M_{j,Ed} \leq M_{j,Rd}\) then \(\mu = (1.5 M_{j,Ed} / M_{j,Rd})^{\psi}\)

and the coefficient \(\psi\) for bolted end-plate connections is 2.7

More about the stiffness of joints \((S_j)\) can be found in Appendix F.
2.2 Static tests

2.2.1 Test results

Altogether eight static tests were carried out. For all static tests the load bearing capacity and the failure mode was pre-calculated on the basis of a design method and with a non-linear FE model, as detailed in chapter 2.3.

Specimen Z1

Specimen Z1 represents a traditional crane bracket with an I cross-section and transversal stiffener in the column compression zone such as shown in Figure 2.21. The column flange in the tension zone was reinforced with backing plates.

Figure 2.20 shows the load-deflection curves of the experiment and, in blue, the curve calculated according to the design method. For the purposes of the experimental curve, the displacements were measured under the load. In the case of the design curve, the joint stiffness was determined first and then the displacement under the load was calculated.

The first plastic deformations were observed in the tension zone at around 600 kN (Figure 2.20). Until about 600 kN the load-displacement relationship was linear and over 600 kN the material began to yield and the relationship became non-linear. The first crack was observed over the fillet weld at the height of the tension flange in the end-plate, as shown in Figure 2.22. This crack propagated as the load was further increased. The load bearing capacity was attained at about 945 kN. The failure occurred by end-plate cracking at the level of the tension bracket flange after significant deformations in the tension zone. Figures 2.21 and 2.22 show the tension and compression zone of the joint after failure.

Figure 2.20 presents the EC3 model curve (plotted in blue), which shows higher initial stiffness and an approximately 30% lower load bearing capacity than the experimental curve (shown in black). The differences in the load bearing capacity and stiffness can be explained by the calculation model applied. The resistances of the joint were calculated with the standard EC3 component model method, as if it were a normal beam-to-column joint. The test load conditions were, however, different from what the model assumed, i.e. dominant moment loading.

Legend
- "(a.m.p.)" means that the calculations were made with actual material properties. The results of the material tests can be found in Appendix B.
- The diagram “EC3 model (a.m.p.)” was calculated with the standard EC3 model with actual material properties and the partial safety factors eliminated ($\gamma_M = 1.0$, $\gamma_{M2} = 1.0$).
Specimen Z2

The geometry of specimen Z2 was identical with that of specimen Z1 with the only difference that a transversal stiffener was applied in the column tension zone rather than backing plates as in test Z1, see Figure 2.25.

The first deformations were observed at about 800 kN in the tension zone of the connection, especially in the end-plate. The failure mode was identical with that of test Z1. The failure occurred by end-plate cracking (Figures 2.27, 2.28) at the height of the tension stiffener. The crack propagated with the increase of the loading until the end-plate fractured. The load bearing capacity was achieved at about 945 kN.

Figure 2.24 shows a similar correspondence between the EC3 and the experimental curve as in the case of test specimen Z1. The diagram according to the EC3 model curve demonstrates a higher initial stiffness and an approximately 35% lower load bearing capacity. This joint was also modelled and designed as a beam-to-column joint, assuming moment as the dominant loading. According to the design method, failure was supposed to occur in the shear panel, but this failure mode was not confirmed by the experiment.

Figures 2.25 and 2.26 show the test specimen with the additional support before and after the test and Figures 2.27 and 2.28 present the joint failure.
Specimen K1-M20

Test specimen K1 was a non-conventional crane bracket having no compression flange, as shown in Figure 2.29, but with backing plates in the tension zone of the joint.

For the test with M20 bolts the so-called TONI arrangement was used, i.e. the bracket flange was not stiffened in the direction perpendicular to the web plane.

Nonlinear behaviour first occurred in the deflection diagram at about 350 kN. The test was stopped at the first sign of lateral torsional buckling of the bracket at around 527 kN.

Figure 2.30 shows the measured load-displacement curve in comparison with the results of the modified EC3 model. The EC3 model curve demonstrates a slightly higher initial stiffness than the test curve. The measured load bearing capacity is underestimated by about 30%.

Specimen K1-M24

The second test with the bracket type K1 was carried out with M24 bolts. The geometry of both tests was identical with the exception of the bolt diameters.

The test was stopped after the web buckled at 588 kN, as shown in Figure 2.32, the brackets failed due to plate buckling.

Figure 2.31 shows the measured and calculated load-displacement curves. The EC3 model curve underestimates the measured load bearing capacity by about 35% but predicts very well the initial stiffness.

Note that contrary to the first test with M20 bolts, in this case the upper flange of the bracket was supported by the "crane runway girder" as shown in Figure 2.7. This can be a reason for the different failure mode and the good conformity as regards stiffness.
Specimen K2_z

Test specimen K2_z has the same geometry and bolt arrangement as test specimen K2, but in this case neither the tension nor the compression zone of the joint was stiffened. The joint arrangement is shown in Figure 2.34.

For this specimen a material test was not made. Because of the missing actual material properties, a modified EC3 curve as shown in Figure 2.33 was calculated using standard material properties.

The load-displacement diagram on Figure 2.33 shows that the initial stiffness of the experimental curve is higher than that obtained from the EC3 diagram. This was the only test where the experimental curve presented significantly higher stiffness than the calculated one.

As to the load bearing capacity, the same tendency was observed. The EC3 curve underestimates the experimental values by a factor of about two, although the calculated failure mode reflected the test observations, i.e. the failure of the column web in compression.

The load bearing capacity would be higher if the actual material properties and reduced safety factors were used.

The failure mode attained and observed during the test was column web buckling in the compression zone of the joint, as shown in Figure 2.35. The low calculated load bearing capacity can be explained by the slender web plate and the missing compression stiffener.
Specimen K2-M20

The main geometrical parameters of specimen K2 were identical with those of specimen Z1, except for the fact that specimen K2 was a light crane bracket without compression flange, as illustrated in Figure 2.36. For the first static test the plates were connected with M20 bolts.

The test arrangement was the same as in the case of test K1 with M20 bolts. Likewise, the bracket flange was not supported in lateral direction as shown in Figure 2.37.

Fig. 2.36 Test specimen K2 with M20 bolts.  
Fig. 2.37 Failure by lateral torsional buckling

The failure was due to lateral torsional buckling of the bracket and occurred at about 700 kN, as shown in Figure 2.37.

The first visible deformations were detected at 350 kN. These deformations were located in the tension zone of the joint, in the column flange, in the backing plate and in the tension part of the end-plate.

Figure 2.38 presents the measured and the calculated diagrams. The load-displacement diagram shows a lower initial stiffness than the EC3 diagram and the load bearing capacity is underestimated by about 20% by the EC3 curve.

Specimen K2-M24

The second test with specimen K2 was performed with M24 bolts.

The failure mode was bracket web buckling at around 807 kN as shown in Figure 2.41.

The load-displacement diagram shows good correspondence between the initial stiffness as measured and as calculated, see Figure 2.39, but shows also that the EC3 model underestimates the load bearing capacity of the joint by about 35%.

Note that contrary to the first test with M20 bolts, in this case the upper flange of the bracket was supported by the “crane runway girder” as in test K1-M24.

The failure was due to lateral torsional buckling of the bracket and occurred at about 700 kN, as shown in Figure 2.37.
Specimen K3-M20

For test specimen K3 with M20 bolts, static test was not completed. The load bearing capacity and the failure mode was determined by the design method and by non-linear FE calculation.

Specimen K3-M24

The second test with specimen K3, with M24 bolts, was stopped at 1,250 kN because the test arrangement reached its load capacity. Figure 2.42 shows the test specimen and part of the support.

After unloading, the test specimen was checked but no visible failure was found. The load bearing capacity based on the previous test results and on the measured load-displacement diagrams was assumed to be 1,350 kN.

For the purposes of the fatigue tests this assumed capacity was used to define the load levels.

The EC3 calculated diagram shows slightly higher initial stiffness compared to the experimental curve and the load bearing capacity is by about 20% underestimated as shown in Figure 2.43.

![Fig. 2.42 Test specimen K3-M24](image)

![Fig. 2.43 Load-displacement diagrams, K3-M24.](image)

2.2.2 Evaluation of the test results

Altogether eight static tests were carried out and for all tests the load bearing capacity and failure mode was pre-calculated on the basis of a non-linear FE model and with the developed design method.

Figure 2.44 shows the measured load-displacement diagrams. The diagrams show that the bolt diameter has a slight effect on the load bearing capacity and the initial stiffness of the tested light crane bracket joints only. This phenomenon can be explained by the failure mode. Test specimens K1, K2 and K2_z failed by stability failure, i.e. plate buckling or lateral torsional buckling, failure modes in which the capacity of the bolts have minor role only.

The explanation of the deviation in the load bearing capacity and stiffness between the tests with M20 and M24 bolts, specimens K1 and K2, was the different stability support of the tested specimens, as indicated previously.

Due to their traditional design solutions, specimens Z1 and Z2 failed in the form of end-plate failure.

Figure 2.45 compares the load-displacement diagrams of three different joint arrangements (Z1, K2, K2_z). In these tests the main geometrical parameters (i.e. end-plate, backing plates, bracket web, bracket tension flange and the plates of the column) were identical, and furthermore, the bolt arrangement and diameter applied was the same. The differences were in the joint arrangement, i.e. in the shape of the bracket and in the stiffeners used on the column side.

Test specimen Z1 was the traditional solution with an I cross-section bracket, backing plates in the tension zone and a transversal stiffener in the compression zone. Specimen K2 had a T cross-section bracket without compression flange and similar stiffeners as specimen Z1. Specimen K2_z had also a T cross-section light crane bracket but neither the tension nor the compression zone of the joint was stiffened.
The diagrams on Figure 2.45 demonstrate that in spite of the different design, the initial stiffnesses were nearly identical. In terms of load bearing capacity, however, a wide range was observed. Specimen Z1 meets the requirements and has the highest capacity, followed by specimen K2 with about 15% lower capacity, and test specimen K2_z, without any stiffener, with a capacity 50% lower than that of specimen K2. The explanation was found in the failure mode of the joints. End-plate cracking (Z1), bracket web buckling (K2) and column web buckling (K2_z) was observed.

Also the load-displacement diagrams as shown in Figure 2.45 were typical for the different failure modes. In the case of end-plate cracking (specimen Z1), a plateau-like behaviour is seen. In the case of column web buckling (specimen K2_z), the load-displacement diagram is characterised by a nearly
constant, slightly descending branch. Finally, in the case of bracket web buckling (K2), the peak value was followed by a longer load uploading than in the case of the end-plate failure.

2.2.3 Verification of the developed design method

The results obtained from the design method are summarized in Table 2.4. The calculated moment resistances and stiffnesses can be found in the second and third columns of this Table. These values were evaluated with the application of the developed EC3 model using standard material and partial factors (material S355: f_y = 355 N/mm², f_u = 510 N/mm²; γ_M0 = 1.0, γ_M2 = 1.25).

The values in Table 2.4 in columns four to six were calculated with the same modified EC3 model but using the actual material properties and with partial factors γ_M0 = 1.0, γ_M2 = 1.0.

Column five lists the calculated resistances before web buckling. These were calculated with the effective bracket web cross-sections but with the assumption that the compression point is at the intersection of the compression stiffener and the bracket web, as shown in Figure 2.46.

Table 2.4 Summary of the static calculation results according to the modified EC3 model.

<table>
<thead>
<tr>
<th>test</th>
<th>moment resistance [kNm]</th>
<th>initial rotational stiffness [kNm/rad]</th>
<th>moment resistance [kNm]</th>
<th>moment resistance calculated just before the stability failure of the web [kNm]</th>
<th>initial rotational stiffness [kNm/rad]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>174.9</td>
<td>29,063</td>
<td>173.0</td>
<td>-</td>
<td>29,063</td>
</tr>
<tr>
<td>Z2</td>
<td>147.0</td>
<td>46,653</td>
<td>149.5</td>
<td>-</td>
<td>46,653</td>
</tr>
<tr>
<td>K1-M20</td>
<td>69.9</td>
<td>8,914</td>
<td>69.2</td>
<td>99.4</td>
<td>8,775</td>
</tr>
<tr>
<td>K1-M24</td>
<td>70.4</td>
<td>9,078</td>
<td>68.3</td>
<td>94.4</td>
<td>9,515</td>
</tr>
<tr>
<td>K2_z</td>
<td>61.3</td>
<td>10,446</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>K2-M20</td>
<td>100.2</td>
<td>14,958</td>
<td>109.1</td>
<td>138.4</td>
<td>14,889</td>
</tr>
<tr>
<td>K2-M24</td>
<td>112.3</td>
<td>15,347</td>
<td>109.8</td>
<td>136.5</td>
<td>15,797</td>
</tr>
<tr>
<td>K3-M20</td>
<td>130.0</td>
<td>28,714</td>
<td>166.3</td>
<td>216.9</td>
<td>29,911</td>
</tr>
<tr>
<td>K3-M24</td>
<td>188.7</td>
<td>29,590</td>
<td>220.9</td>
<td>282.2</td>
<td>26,374</td>
</tr>
</tbody>
</table>

*For specimen K2_z was no material test done, and accordingly, the adequate cells are empty.

The calculated higher initial stiffness of the arrangement using M20 bolts as compared to that using M24 bolts, as indicated in column six for specimen K3, is explained by the different failure mode. In configuration K3 with M20 bolts the resistance of the first bolt-row was calculated as individual bolt failure, whereas in K3 with M24 bolts the resistance of the first bolt-row was calculated as if it was part of a bolt group, hence the different l_eff values.

In the case of the tested light crane bracket joints it was observed that the stiffness measured in the tests for arrangements having M20 bolts was “lower” than what was calculated. At the same time, there was a good correspondence between the measured and calculated values for arrangements with M24 bolts. The explanation of this effect lies in the difference in the stability support, as explained in chapter 2.2.1.

Table 2.5 shows the comparison of the measured and calculated load bearing capacities. The calculated values are based on the modified EC3 model, actual material properties and partial factors equal to 1.0.
Table 2.5 Measured and calculated load bearing capacities.

<table>
<thead>
<tr>
<th>test</th>
<th>measured load bearing capacity [kN]</th>
<th>load bearing capacity calculated according to the developed EC3 model [kN]</th>
<th>utility [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>945</td>
<td>692</td>
<td>73.2</td>
</tr>
<tr>
<td>Z2</td>
<td>945</td>
<td>598</td>
<td>63.2</td>
</tr>
<tr>
<td>K1-M20</td>
<td>527*</td>
<td>397</td>
<td>75.3</td>
</tr>
<tr>
<td>K1-M24</td>
<td>588</td>
<td>378</td>
<td>64.3</td>
</tr>
<tr>
<td>K2-M20</td>
<td>700*</td>
<td>554</td>
<td>79.1</td>
</tr>
<tr>
<td>K2-M24</td>
<td>807</td>
<td>546</td>
<td>67.7</td>
</tr>
</tbody>
</table>

*The failure mode attained was lateral torsional buckling.

From the comparison it was found that the developed design method gives good estimations for the initial stiffness, whereas it underestimates the load bearing capacity by an average of 30%.

International research results show that the accuracy of the EC3 model as regards the design moment resistance includes a “safety margin” of about 30%. This underestimation comes from the simplifications and limitations of the method, and can partly be considered as a price paid for its easy manageability.

The few examples referred to below cover simple connections only, which can be calculated without any modification in the component method model given in EN 1993-1-8 (2005).

Girão Coelho et al (2004) have found an underestimation between 20% and 30% in their research, Schwarzlos (2005) also indicates an average of 30% underestimation, while experiments from Pasternak et al (1996b) show similar results. Hungarian researchers have also pointed out this deviation between the experimental and the calculated moment capacity. Kovács (2005) shows in her work an average 30% deviation between measured and calculated moment resistance. Finally, this dissertation also shows this tendency for the reference tests i.e. for joint arrangements which resistance can be calculated with the standardized component method model given in EN 1993-1-8 (2005), (beam-to-beam joints, series I, end-plate type IV and series II, specimen TC).

The accuracy of the developed design method fits to the accuracy of the standardized component method model and shows also about 30% underestimation.

In summary it can be stated that the EC3 compatible design method for bolted light crane bracket joints is able to provide the joint moment resistance ($M_{j,Rd}$) and stiffness ($S_j$).
2.3 FE modelling

2.3.1 General

Before performing the static tests, and in parallel to calculations with the developed design method, FE calculations were also carried out. This helped refining the load capacity and the failure mode. In the first step the calculations were done with bi-linear (linear elastic-plastic) material properties. After obtaining the results of the material tests these models were re-evaluated using the actual material properties based on the measured stress-elongation curves. The re-evaluated models also included data from the stress gauges. That is, the final model was verified on the basis of the measured stresses and their distribution.

In the following some details are given about the FE model. The profiles/plates were modelled with 4-node 2D shell elements, as shown in Figure 2.47. The bolts were 8-node volume elements. High stresses and deformations were expected in particular areas of the joint and the bracket, therefore in these areas the mesh was refined.

The accuracy of the results of the model depends to a very large extent on the behaviour of the model at the end-plate/column flange interface. In the reality these plates are connected by bolts. In the model this contact problem was solved with gap elements. These elements can deliver tension or compression only, depending in their definition, and have one degree of freedom only. The combined use of gap and bolt elements ensured an appropriate load transmission in the contact zone.

Another problem was the modelling of the backing plates. Here also the contact problem had to be solved. But the use of a lot of gap elements would have raised the calculation time and made the model unstable/vulnerable. It was found better to define the backing plates as part of the column flange, i.e. in the area of the backing plate the flange was assumed to be thicker.

The FE model included a “real” load distribution on the bracket flange, i.e. the loading was modelled as it was applied in the experiments, as shown in Figure 2.7. The thickness of the flange was defined as 50 mm and so the possibility of a local plate failure was excluded. In addition, the supports were also modelled in a manner similar to the experiments.

By a parametric study the number of the finite elements was reduced so as to optimise calculation time and accuracy of results.

Details of the analysis

The geometrically and materially non linear imperfect analysis of the light crane brackets (specimens K1, K2 and K3) was done in three steps:

- The first positive critical load parameter was determined for the bracket web in a separate calculation.
- The first buckling mode (amplitude), which belongs to the first positive critical load parameter, was scaled down and was set as a geometrical imperfection on the bracket web. The scaling factor applied was $h_{bw}/200$, i.e. bracket web height divided by 200, in conformity with the EC3.
- The final analysis was done on the imperfect model.

The load capacities of the “traditional” brackets (Z1, Z2) were calculated without imperfections because the imperfection-sensitivity of these arrangements is low.
2.3.2 Results of the FE calculations

**Specimen Z1**

Figure 2.48 a.) shows the FE mesh applied for the reference test specimen Z1.

Specimen Z1 represents the traditional crane bracket arrangement with an I cross-section.

Figure 2.48 b.) shows the failure mode, i.e. end-plate failure in the tension zone, which was confirmed by the test.

Figure 2.49 shows the deformed end-plate in the tension zone.

![Fig. 2.48 FE mesh, specimen Z1.](image1)

![Fig. 2.49 Deformed tension zone.](image2)

**Specimen Z2**

Figure 2.50 a.) shows the initial FE mesh and b.) the deformed mesh under ultimate load.

Figure 2.50 b.) shows the failure mode which was identical to that of test specimen Z1.

Figure 2.51 shows the cracking of the end-plate of the specimen after the test.

![Fig. 2.50 FE mesh, specimen Z2.](image3)

![Fig. 2.51 Specimen Z2 after test.](image4)
Specimen K1

Figure 2.52 a.) shows the optimized mesh with the applied support conditions. Test specimen K1 was a light crane bracket having no compression flange but backing plates in the tension zone of the joint.

Figure 2.52 b.) presents the failure mode, i.e. bracket web stability failure. Figures 2.53 and 2.54 show the first buckling modes for the column web and bracket web. These amplitudes were scaled down and used as geometrical imperfections for the geometrically and materially non-linear imperfect analysis.

Figure 2.55 presents the equivalent (von-Mises) stress distribution under ultimate load level. In this figure it can be seen that the highest stresses in the tension zone occurred at the level of the first bolt-row, and in the compression zone, in the bracket web.

In the case of the FE calculation of test specimen K1 with M24 bolts, mainly the same mesh was used as for the test with M20 bolts. The difference lies only in the bolt diameter.

Figure 2.56 shows the same failure mode as in the case of the smaller bolt diameter, i.e. stability failure of the bracket web. The experiments confirm the FE calculation, as shown in Figure 2.57.

The first buckling mode of the column web and bracket web were the same as in the calculation with M20 bolts and were applied as geometrical imperfections.

Figure 2.58 shows the distribution of the equivalent stresses under the ultimate load. The presented distribution is identical to that shown in Figure 2.55.
Specimen K2

Figure 2.59 shows the failure mode, i.e. web buckling of the bracket. Because of the missing lateral support in the experiment, the specimen failed due to lateral torsional buckling as shown in Figure 2.60.

Figures 2.61 and 2.62 show the first buckling modes for the column web and bracket web. These deformations were scaled down and used in the non-linear calculation as geometrical imperfections.

Figure 2.63 shows the equivalent stress (von-Mises) distribution under ultimate load. The highest stress level was detected in the bracket web under compression.

For the calculation of the failure mode and load capacity of test specimen K2 with M24 bolts, a similar mesh was used as for specimen K2 with M20. The difference was only in the applied bolt diameter.

For the non-linear calculation the same geometrical imperfections were used as for specimen K2-M20. As seen in Figure 2.64, the failure mode of specimen K2 is also the same as when using M20 bolts. The test confirmed this calculated stability failure as shown in Figure 2.65.

Figure 2.66 shows the distribution of the equivalent stresses (von-Mises) under ultimate load. The highest stresses were attained in the tension zone of the column web in the end-plate and in the compressed bracket web.
Specimen K3

Figure 2.67 shows the calculated failure mode, i.e. bolt failure in the extended bolt-row.

Figures 2.68 and 2.69 show the first buckling modes for the column and for the bracket. These deformations were scaled down and used as geometrical imperfections in the non-linear calculation.

Figure 2.70 presents the equivalent stress (von-Mises) distribution under ultimate load.

For the FE calculation of specimen K3-M24 a similar mesh was used as for specimen K3-M20. The only difference was in the applied bolt diameter.

For the non-linear calculation the same geometrical imperfections were used as in the calculations for specimen K3-M20.

Figure 2.71 shows an end-plate failure (the column flange in the tension zone was reinforced by backing plates) in the tension zone of the joint.

Figure 2.72 shows the distribution of the equivalent (von-Mises) stresses under ultimate load.
2.3.3 Evaluation of the FE results

A total of eight FE models were developed. The most important results of the FE calculations were the load bearing capacity, the failure mode and the load-displacement behaviour.

The FE calculations showed that under static loading, for the studied bracket geometries, the diameter of the bolts had only a slight effect on the load bearing capacity and the initial stiffness of the joints. These values are nearly the same for both bolt diameters used, M20 and M24. Only in test K3 did the higher bolt diameter cause an increase of the load bearing capacity of about 20%, as seen in Figure 2.73, which is explained by the different failure mode.

![Diagram showing load-displacement behavior](image)

*Fig. 2.73 Summary of the load-displacement diagrams as calculated by FE analysis.*

![Table 2.6 Failure modes as calculated by FE analysis](image)

<table>
<thead>
<tr>
<th>test specimen</th>
<th>calculated failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1-M20</td>
<td>bracket web stability failure</td>
</tr>
<tr>
<td>K1-M24</td>
<td>bracket web stability failure</td>
</tr>
<tr>
<td>K2-M20</td>
<td>bracket web stability failure</td>
</tr>
<tr>
<td>K2-M24</td>
<td>bolt failure in the extended bolt-row</td>
</tr>
<tr>
<td>K3-M20</td>
<td>end-plate failure</td>
</tr>
<tr>
<td>K3-M24</td>
<td>end-plate failure</td>
</tr>
</tbody>
</table>

Table 2.6 summarizes the failure modes according to the FE calculation. The calculated failure mode of test specimens K1 and K2 is stability failure in the web, which is independent from the applied bolt diameter, as shown in Figures 2.52, 2.56, 2.59 and 2.64. But the calculated ultimate behaviour of test specimen K3 does depend on the bolt diameter. K3 with M20 bolts failed by bolt failure, as shown in Figure 2.67, as compared to the end-plate failure in the case of bolts M24, as shown in Figure 2.71.

The FE diagrams show good conformity with the test results, as shown in Figure 2.74. Although in the FE calculations the initial stiffness depended just slightly on the bolt diameter, there were obvious differences with respect to the test results, as shown in Figure 2.74 c.) and d.), or e.) and f.). Two static tests were carried out with the TONI testing frame. The main difference between the test arrangements was the stability support of the bracket. In both specimens K1 with M20 and K2 with M20, the bracket flange was not stiffened perpendicularly to the web plane.
Fig. 2.74 Load-displacement diagrams as measured in tests and as calculated by FE analysis.

(a.) test specimen Z1

(b.) test specimen Z2

(c.) test specimen K1-M20

(d.) test specimen K1-M24

(e.) test specimen K2-M20

(f.) test specimen K2-M24

(g.) test specimen K3-M24
In addition, these test specimens failed in the form of lateral torsional bucking, contrarily to specimens K1 with M24 and K2 with M24, which failed by web buckling. It was due to the missing stability support that the measured joint stiffness was lower than in the tests with the other test arrangement, which contained a support.

Table 2.7 summarizes the measured load bearing capacities and the results of the FE calculations. The comparison shows that there were two cases when the calculations overestimated the capacities measured in the tests, and in all the other cases they underestimated them.

The calculation of the load capacity of traditional bracket arrangements (Z1, Z2) did not take account imperfections. This can be a possible explanation for the relatively high calculated capacity for test specimen Z2.

In tests K1-M20 and K2-M20 the bracket flanges were not stiffened perpendicularly to the web plane, i.e. the runway girder support was not considered, and therefore premature failure occurred in the form of lateral tensional buckling. With the appropriate stiffener, the measured load capacity would be higher.

Table 2.7 Load bearing capacities as measured and as calculated by FE analysis.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>measured load bearing capacity [kN]</th>
<th>load bearing capacity as calculated by FE analysis [kN]</th>
<th>utility [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>945</td>
<td>930</td>
<td>98.4</td>
</tr>
<tr>
<td>Z2</td>
<td>945</td>
<td>1,090</td>
<td>115.3</td>
</tr>
<tr>
<td>K1-M20</td>
<td>527</td>
<td>525</td>
<td>99.6</td>
</tr>
<tr>
<td>K1-M24</td>
<td>588</td>
<td>557</td>
<td>94.7</td>
</tr>
<tr>
<td>K2-M20</td>
<td>700</td>
<td>740</td>
<td>105.7</td>
</tr>
<tr>
<td>K2-M24</td>
<td>807</td>
<td>748</td>
<td>92.7</td>
</tr>
<tr>
<td>K3-M24</td>
<td>1,350 (assumed)</td>
<td>1,100</td>
<td>81.5</td>
</tr>
</tbody>
</table>

Table 2.7 and Figure 2.74 demonstrate that the FE calculations correspond very well to the tests results. The FE models reflected the failure mode and failure behaviour as well as the load bearing capacity with high accuracy.

It can be stated that the FE model developed and used for bolted light crane bracket joints is able to provide the load bearing capacity and initial stiffness of the joint.
2.4 Fatigue tests

2.4.1 Test results

Preparation of the test specimens
The geometry of the specimens used for the fatigue tests was the same as for the static tests. Altogether twelve tests were carried out as listed in Table 2.1. For practical reasons, the tested brackets were kept under a certain load level during the whole duration of the fatigue tests, with a minimum load level of about 20 kN. This minimum load prevented the separation of the bracket flange from the hydraulic jack.

Under normal circumstances, crane girders are built as continuous beams, which means that uplift as well as downward forces act on the bracket. In the fatigue tests this uplift was also simulated by applying a force equal to 10% of the downward vertical load, as shown in Figure 2.11. In the first and second load steps (10 and 500 cycles, see Figure 2.10) the uplift loading was applied immediately after the downward loading. In the third load step the uplift load was applied at all times after 2,000 cycles. In the transient phase between tension and compression, the loading rate was lower.

Used stress-range spectrum
For construction elements under fatigue loading, representative loads have to be defined. The frequency and magnitude of these loads give the loading class and finally the stress-range spectrum can be determined.

The German code DIN 15 018 uses the following stress-range spectra, as shown in Figure 2.75.

- \( S_0 \) = very low stress
- \( S_1 \) = low stress
- \( S_2 \) = medium stress
- \( S_3 \) = high stress

The stress-range spectrum should be determined by presenting the stress-ranges and the associated numbers of cycles in descending order.

The stress spectra define how often a maximum stress limit is achieved or exceeded. From the stress-range the stage of the saturation (\( p \)) has to be derived.

For example the stress-range spectrum \( S_2 \) has a saturation of \( p = 2/3 \), as shown in Figure 2.76.

In the fatigue tests a loading class B3 ("Beanspruchungsgruppe B3") was simulated. The only exception was test K2 with bolts M20, which was loaded continuously at the maximal load level (0.7 \( M_{\text{Rd}} \)). The chosen range of the number of load cycles \( N_1 \) (Spannungsspielbereich \( N_1 \)) was \( 2 \times 10^4 \) to \( 2 \times 10^5 \) cycles with a spectrum \( S_2 \).

<table>
<thead>
<tr>
<th>range of the number of load cycles</th>
<th>( N_1 )</th>
<th>( N_2 )</th>
<th>( N_3 )</th>
<th>( N_4 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>total of the applied number of load cycles ( N )</td>
<td>( 2 \times 10^4 ) to ( 2 \times 10^5 )</td>
<td>( 2 \times 10^5 ) to ( 6 \times 10^5 )</td>
<td>( 6 \times 10^5 ) to ( 2 \times 10^6 )</td>
<td>over ( 2 \times 10^6 )</td>
</tr>
<tr>
<td>occasional, irregular use with long pauses</td>
<td>regular use with pauses</td>
<td>regular use without pauses</td>
<td>intensive regular use without pauses</td>
<td></td>
</tr>
<tr>
<td>stress spectrum</td>
<td>( S_0 ) = very low</td>
<td>( S_1 ) = low</td>
<td>( S_2 ) = medium</td>
<td>( S_3 ) = high</td>
</tr>
<tr>
<td>loading class</td>
<td>( B_1 )</td>
<td>( B_2 )</td>
<td>( B_3 )</td>
<td>( B_4 )</td>
</tr>
</tbody>
</table>

Fig. 2.75 Stress-range spectra.

Fig. 2.76 Saturation by a stress spectra of \( S_2 \).
Table 2.8 explains the loading classes, ranges of number of load cycles and stress spectra according to the German code DIN 15 018.

The maximum fatigue load applied was equal to 70% of the static load bearing capacity. According to the recommendations of EN 1993 1.8, the behaviour is elastic up to 2/3 $M_{pl,Rd}$, and then it becomes plastic. The maximum fatigue load level was chosen so as to obtain moderate plastic deformations in the joints.

Figure 2.77 shows the load spectrum curve chosen (plotted curve in red) according to the recommendation of DIN 15 018.

First the static test was carried out and the load bearing capacity and load-deflection curve was determined. With the knowledge of the achieved load bearing capacity the load spectrum was defined.

The curve was chosen according to the recommendation of DIN 15 018 and was simplified by a four step approximation, as shown in Figure 2.77.

The used load steps were as follows:
- $1.0 \times F^{70\%}$ (10 cycles)
- $0.944 \times F^{70\%}$ (500 cycles)
- $0.856 \times F^{70\%}$ (26,000 cycles)
- $0.727 \times F^{70\%}$ (173,000 cycles)

(Where $F^{70\%} = 0.7 \times F_{load\ capacity\ of\ the\ test}$).

The fatigue tests were carried out under load control, therefore the required load level was adjusted irrespectively of the deformations in all the cycles.

\[
\frac{\sigma_u - \sigma_a}{\max\sigma_u - \sigma_a} = 1.0 = 70\% \text{ of the static load bearing capacity}
\]

\[
\sigma_u = \text{upper stress level}
\]

\[
\lg N = 6 \quad (\bar{N} = 10^6, \text{idealized stress spectrum})
\]

Fig. 2.77 Load spectrum curve and its step-like approximation.
Specimen Z1

Test specimen Z1 was a traditional crane bracket with an I cross-section and a stiffener in the column compression zone. The achieved load bearing capacity was around 945 kN, therefore the maximum fatigue load applied was 665 kN.

Two fatigue tests were carried out with this test specimen because of the early fracture, after 5,000 cycles, of the weld of the end-plate bracket.

Examination of the weld material showed slags in the weld and inadequate penetration.

In the second test, after around 7,500 cycles, an increase in the deformations was observed as shown in Figure 2.79.

The displacement vs. cycle number diagram presents the maximum displacements of each load cycle only, in the function of the number of the load cycles.

The principle of the displacement vs. cycle number diagram is explained in Figure 2.78.

Figure 2.79 shows the displacement vs. cycle number diagram for specimen Z1. The diagram shows a pronounced step at 500 cycles due to the change in the load level.

After about 11,000 cycles a higher increase of the deformations was seen. The achieved number of cycles was around 13,000, and the failure occurred in the form of end-plate fracture as shown in Figure 2.80.

In the course of the fatigue tests, the secant stiffness of each cycle can be determined, as Figure 2.81 explains, and the stiffness vs. cycle number curve can be defined.

The secant stiffness decreases during the consecutive cycles. To be able to compare the fatigue behaviour of different joints in an easier way, the secant stiffness values were normalized with the secant stiffness corresponding to the first hysteretic cycle.

The normalized stiffness curve shows the fatigue behaviour of a joint. A "rapidly falling" curve, i.e. if the curve shows a definitive decrease of the secant stiffness after a few thousand cycles, means high plastic deformations in the joint. But, if the curve is "flat", then the secant stiffnesses are about the same, which signals elastic deformations in the joint.

Figure 2.83 shows the stiffness vs. number of cycles curve of specimen Z1. This curve shows that after 10,000 cycles the secant stiffness degradation was more than 10%.

The stiffness vs. number of cycles curve can be approximated easily by a parabolic equation \( y = ax^2 + bx + c \), where the interpretation of the parameters “a”, “b”, and “c” is explained in Figure 2.82. The use of the parabolic approximation makes it easier to evaluate the results of the fatigue tests.
As a general rule it is noted that a low "c" parameter, i.e. a "widely open parabola", guarantees favourable fatigue behaviour. The approximations can be done at determined load steps, for example at 5,000, 10,000 or 20,000 cycles, to forecast the prospective load cycle number. For the fatigue tests the approximations were done numerically applying the least-mean-square technique.

Figure 2.83 shows the stiffness degradation in function of the number of cycles and Figure 2.84, the approximated stiffness vs. number of cycles curve and its parameters for the fatigue test of specimen Z1.

Figure 2.85 summarizes the approximation parameters after different cyclic numbers. Note that the changes of the parameter "c" was minimal: -1.61 x 10^-9 after 5,000 cycles; -1.64 x 10^-9 after 10,000 cycles and -1.79 x 10^-9 after 13,000 cycles. This quality of the approximation curve can be used for forecasting the fatigue behaviour and the probable number of cycles.
Specimen Z2

The achieved load bearing capacity was around 945 kN, so the maximum fatigue load applied was 665 kN. Figure 2.86 shows the displacement vs. cycle number diagram, which indicates that, after a constant level of deformations up to about 4,000 cycles, the deformations started to increase and a rapid rise occurred at about 7,500 cycles.

After 7,800 cycles the tested joint failed due to a fracture in the end-plate as shown in Figures 2.87 and 2.88.

The failure mode was identical in both specimens Z1 and Z2, as in both cases the end-plates failed in the tension zone at the height of the tension stiffener, i.e. at the level where the highest stresses occurred.

Figures 2.89 and 2.90 show the measured and the approximated stiffness vs. number of cycles curve for the test. The value of parameter “c” was -5.23 x 10^-9 after 5,000 cycles and -6.11 x 10^-9 after 7,800 cycles.
Specimen K1-M20

Because of delivery/transport problems bracket K1 for the fatigue test was connected to an S2 column. More details about the geometry of the S2 column are given in Appendix A.

The static test was stopped at a load level 527 kN and this level was assumed as the load bearing capacity. Hence the maximum fatigue load applied was 371 kN.

The first crack in the end-plate occurred after around 90,000 cycles, as shown in Figure 2.92, at the height of the web-splice in the tension flange.

The displacement vs. cycle number diagram (Figure 2.91) shows that a constant rate of change in the deflections was followed from about 94,000 cycles by a higher rate of increase.

The end-plate failed at 109,000 cycles, as shown in Figure 2.92.

After the failure of the splice material (Figure 2.93 b.)) the extended bolt-row was unloaded and the whole tension load started to act in the second bolt-row.

Because of this sudden load re-distribution one bolt failed in this bolt-row before the test was stopped. Figure 2.93 shows the broken bolt.

Figures 2.94 and 2.95 present the stiffness vs. cycle number curve and its approximation.

The stiffness vs. cycle number curve in Figure 2.94 shows that the initial secant stiffness was exceeded during the course of the test. Figure 2.95, with the single approximations curve, illustrates this observation more clearly.

This phenomenon can be explained by the different load steps, i.e. the different load levels. If there are plastic deformations in the first and/or second load step, i.e. in the first 500 cycles or so, then the secant stiffness in the following load step can be higher, as explained in Figure 2.96.

Spalte 11

Nicht-Lineare Regression (N = 35101)

\[ a + b \cdot x + c \cdot x^2 \]

Least square minimized

Iterations: 1415

Goodness of Fit:

\[ \chi = 29.4026, \text{ p = 0} \%

Parameter:

\[ a = 1.0568 \]
\[ b = 2.5413 \times 10^{-006} \]
\[ c = -3.6595 \times 10^{-11} \]

Varianz der Residuen = 0.0008

Stdabw. der Residuen = 0.0289

Korrelationskoeffizient = 0.8763
df = 35098

p< = 0 . 0 0 1%

Fig. 2.96 Evolution of the secant stiffnesses.
Specimen K1-M24

In this test, in addition to the vertical loads, a constant horizontal load (49 kN) was also applied. This horizontal load simulated the non-parallel crane wheel run. The used horizontal load was equal to 11.9%; 12.6%; 13.8% and 16.3% respectively in each load step. The load was applied at the height of the bracket flange. The jacks were harmonized so as to achieve the maximum load simultaneously.

The maximum fatigue load level applied was 413 kN.

Figure 2.97 shows the displacement vs. cycle number diagram, which shows a continuous increase of the displacement.

After around 42,000 cycles a crack in the bracket was detected, as shown in Figure 2.98, at the height of the tension flange.

The fact that a lower number of cycles was achieved in this fatigue test than that of test specimen K1 with bolts M20 be explained by the additional horizontal load applied and the relatively higher load level.

Figure 2.99 shows the stiffness vs. number of cycles curve, and Figure 2.100, its approximation.

The calculated parameter “c” was as follows: 9.52 x 10^{-11} after 10,000 cycles, -1.58 x 10^{-10} after 20,000 cycles and -2.09 x 10^{-10} after 42,000 cycles.

Note that the value of parameter “c” was positive at 10,000 cycles, which means that the secant stiffness was increasing. At the very beginning of the load history this is possible because the load levels are decreasing. It is impossible with this arrangement and load history, however, to have this tendency for a long period.
Specimen K2_z

This specimen had neither transversal stiffeners nor backing plates, as shown in Figure 2.102.

The measured load bearing capacity was 437 kN and so the chosen first fatigue load level was 306 kN.

After around 170,000 cycles the end-plate failed, as shown in Figure 2.102, at the height of the bracket tension flange.

The displacement vs. cycle number diagram in Figure 2.101 shows that at about 150,000 cycles there was a definite increase in displacement. This rapid deflection growth was due to a crack and its propagation in the end-plate.

Figures 2.103 and 2.104 show the measured and the approximated stiffness vs. number of cycles curve for this fatigue test.

The value of the parameter "c" was as follows: $-3.95 \times 10^{-11}$ after 10,000 cycles, $-1.18 \times 10^{-10}$ after 20,000 cycles, and $-1.36 \times 10^{-11}$ after 170,000 cycles.
Specimen K2-M20

For the same reasons as in the case of test specimen K1-M20, for the purposes of this fatigue test, the bracket was connected to an S3 column.

The load history for this test specimen was different in comparison to the other tests. The specimen was loaded by a constant load corresponding to 70% of the static load bearing capacity. The other specimens were loaded only in the first load step, i.e. 10 times, with this 70%.

After around 13,000 cycles the bolts in the extended part of the end-plate failed, as shown in Figure 2.106.

The verification of the bracket showed no fracture or cracking. The bolts were changed and the fatigue test was restarted.

After around 10,000 cycles the bolts failed another time (Figure 2.108) and an end-plate crack occurred at the height of the tension flange, as shown in Figure 2.107.

Figure 2.105 shows the displacement vs. cycle number diagrams. On the deflections curve of the first test, plotted in blue, a constant deformation level can be seen until the failure of the bolts. This abrupt change in the diagram is a typical sign of bolt failure.

The displacement curve of the second test, plotted in black, shows a continuous increase. This deflection increment can be explained by the end-plate crack and its propagation.

Figures 2.109 and 2.110 present the stiffness vs. number of cycles curve and its approximation. The values of the calculated parameter “c” were \(-7.87 \times 10^{-10}\) for test 1 and \(-6.17 \times 10^{-10}\) for test 2.
Specimen K2-M24

The fatigue test of specimen K2 with bolts M24 was carried out twice with different load histories. When subjected to lower load steps (initiated at 41% of the load bearing capacity), the specimen achieved 293,000 load cycles; the same arrangement when subjected to higher load steps (initiated at 70%), achieved around 24,000 load cycles only.

In the first fatigue test, the end-plate cracked after around 24,000 cycles as shown in Figure 2.112.

The displacement vs. cycle number diagram of Figure 2.111 shows constantly growing deformations. At about 18,000 cycles a breaking-point can be seen in the curve. At this level one could not control the load (480 kN) safely, therefore the load level was reduced (430 kN). The end-plate was probably already cracked at this time.

In the second fatigue test the specimen was first loaded by a load corresponding to 41% of the static load bearing capacity (334 kN). After around 293,000 cycles the column web cracked, as shown in Figure 2.115. The crack began at the height of the bracket flange, and propagated in both directions.

The displacement vs. cycle number diagram of Figure 2.114 shows a constant level of deformations for the second test.

Table 2.9 summarizes the load histories with the applied load levels.

![Fig. 2.112 End-plate crack (test 1), K2-M24.](image1)

![Fig. 2.111 Displacement vs. cycle number diagram (test 1), K2-M24.](image2)

<table>
<thead>
<tr>
<th>test 1 number of cycles</th>
<th>load [kN]</th>
<th>test 2 number of cycles</th>
<th>load [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>560 (70%)</td>
<td>10</td>
<td>334 (41%)</td>
</tr>
<tr>
<td>500</td>
<td>529 (66%)</td>
<td>500</td>
<td>316 (39%)</td>
</tr>
<tr>
<td>ca. 23,000</td>
<td>480 (60%)</td>
<td>26,000</td>
<td>286 (35%)</td>
</tr>
<tr>
<td>-</td>
<td>(407) (50%)</td>
<td>ca. 266,000</td>
<td>243 (30%)</td>
</tr>
<tr>
<td>ca. ∑ 24,000</td>
<td></td>
<td>ca. ∑ 293,000</td>
<td></td>
</tr>
</tbody>
</table>

Figures 2.115 and 2.116 show the measured and the approximated stiffness vs. number of cycles curve for test 1.

Figures 2.117 and 2.118 show the measured and the approximated stiffness vs. number of cycles curve for test 2.

The value of the calculated parameter “c” was: \(-6.24 \times 10^{-10}\) for test 1 and \(-4.42 \times 10^{-12}\) for test 2.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Crane bracket joints

Fig. 2.113 Column web failure (test 2), K2-M24.

Fig. 2.114 Displacement vs. cycle number diagram (test 2), K2-M24.

Fig. 2.115 Stiffness vs. cycle number curve (test 1), K2-M24.

Fig. 2.116 Approximation curve (test 1), K2-M24.

Fig. 2.117 Stiffness vs. cycle number curve (test 2), K2-M24.

Fig. 2.118 Approximation curve (test 2), K2-M24.
Specimen K3-M20

For this test specimen configuration no static test was done. The load bearing capacity and failure mode was calculated using the developed design method and by a non-linear FE model. For the fatigue test the first load step was applied at 70% of the calculated load bearing capacity (590 kN).

In the fatigue test the bolts failed after around 26,000 cycles, as shown in Figure 2.120. The test specimen was checked for cracks and the test was continued with new bolts at the actual load step. Note that apart from the new bolts, the bracket plates and welds as well as the end-plate could have potential plastic deformations at this time.

After about 160,000 cycles (a total of 186,000 cycles) the bolts failed again, as illustrated in Figure 2.121, and an end-plate crack occurred at the height of the tension flange, as shown in Figure 2.122.

The blue plotted displacement vs. cycle number diagram of Figure 2.119 shows a constant level of deformation until the failure of the bolts. The curve is a typical bolt failure curve.

The diagram of the continued test, plotted in black, shows constant deformations until about 70,000 cycles. After this level the deformations started to grow in a continuous manner, which can be explained by the cracking of the end-plate and the propagation of these cracks.

Figures 2.123 and 2.124 show the measured and the approximated stiffness vs. number of cycles curve for test K3-M20. The value of the calculated parameter “c” is: \(-5.69 \times 10^{-11}\) for the first part of the test and \(-7.80 \times 10^{-12}\) for the continued test.
Specimen K3-M24

The fatigue test of arrangement K3 with bolts M24 was carried out three times with three different load histories as summarized in Table 2.10. Because of capacity problems of the test arrangement the load bearing capacity was assumed to be 1350 kN.

When the bracket was subjected to the chosen load steps (initiated at 70% of the load bearing capacity), the specimen achieved 13,000 load cycles; the same design when subjected to lower load steps (initiated at 53%), achieved around 154,000 load cycles and when subjected to even lower load steps (initiated at 35%), it achieved around 589,000 load cycles.

Table 2.10 summarizes the load histories with the applied load levels and the achieved number of cycles.

<table>
<thead>
<tr>
<th>Test 1</th>
<th>Load [kN]</th>
<th>Test 2</th>
<th>Load [kN]</th>
<th>Test 3</th>
<th>Load [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>945 (70%)</td>
<td>10</td>
<td>715 (53%)</td>
<td>2 x 10</td>
<td>468 (35%)</td>
</tr>
<tr>
<td>500</td>
<td>892 (66%)</td>
<td>500</td>
<td>675 (50%)</td>
<td>2 x 500</td>
<td>442 (33%)</td>
</tr>
<tr>
<td>ca. 13,000</td>
<td>809 (60%)</td>
<td>26,000</td>
<td>612 (45%)</td>
<td>2 x 26,000</td>
<td>401 (30%)</td>
</tr>
<tr>
<td>-</td>
<td>(687) (51%)</td>
<td>ca. 127,000</td>
<td>520 (38%)</td>
<td>ca. 510,000</td>
<td>340 (25%)</td>
</tr>
<tr>
<td>ca. Σ 13,000</td>
<td></td>
<td>ca. Σ 154,000</td>
<td></td>
<td>ca. Σ 563,000</td>
<td></td>
</tr>
</tbody>
</table>

In the first test the end-plate cracked after around 13,000 load cycles, as shown in Figure 2.126. The displacement vs. cycle number diagram is shown in Figure 2.126. The curve shows constant deformations until 8,000 load cycles. After that, the curve changes and deformations begin to grow. It is most likely that the end-plate cracked at this point.

Figure 2.126 shows the crack in the end-plate at the height of the bracket tension flange, as well as the crack of the column web behind the tension bolts. The failure was accompanied by high deformations.

Figures 2.127 and 2.128 present the stiffness vs. number of cycles curve and its approximation for specimen K3-M24, test 1.
In the second fatigue test (test 2) the specimen was loaded first by 53% of the static load bearing capacity (Table 2.10).

After around 154,000 load cycles cracks occurred in the end-plate as shown in Figure 2.129.

The displacement vs. cycle number diagram in Figure 2.130 shows that in the fourth load step there was a slight increase in the level of deformations until finally the end-plate cracked.

Figures 2.131 and 2.132 show the stiffness vs. number of cycles curve and its approximation for specimen K3-M24, test 2.
In the third test (test 3) the specimen was loaded first by 35% of the static load bearing capacity (468 kN).

Another different feature of this test was the higher number of cycles applied in the load steps. In each step, twice as many cycles were applied as corresponding to the chosen load spectrum curve, see Table 2.10.

After around 563,000 load cycles the test was stopped without any sign of failure. Figure 2.133 shows the deformed specimen with no cracks at all.

The displacement vs. cycle number diagram in Figure 2.134 shows a constant level of deformations.

The stiffness vs. cycle number diagram shown in Figure 2.135 exhibits a well balanced and constant stiffness level until the end of the test.

Figures 2.135 and 2.136 show the measured and the approximated stiffness vs. number of cycles curve for test 3.

The values of the calculated parameter “c” are -2.81 x 10^{-9} for test 1, -6.12 x 10^{-12} for test 2 and 5.40 x 10^{-14} for test 3.

Fig. 2.134 Displacement vs. cycle number diagram (test 3), K3-M24.

Fig. 2.135 Stiffness vs. cycle number curve (test 3), K3-M24.

Fig. 2.136 Approximation curve (test 3), K3-M24.
2.4.2 Evaluation of the test results

Table 2.11 shows a summary of the results of the fatigue testing programme within which twelve tests were carried out. The results show that the bracket load spectrum, and therefore the whole design concept, had significant influence on the fatigue behaviour of the tested brackets.

<table>
<thead>
<tr>
<th>joint arrangement</th>
<th>test</th>
<th>static load bearing capacity from test [kN]</th>
<th>rotational stiffness according to EN 1993-1-8 [kN/m/rad]</th>
<th>maximum fatigue load / static load bearing capacity [%]</th>
<th>load steps [kN]</th>
<th>load cycles achieved</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>945</td>
<td>29,063</td>
<td>70</td>
<td>665 / 628 / 569 / (483)*</td>
<td>ca. 13,000</td>
<td></td>
</tr>
<tr>
<td>Z2</td>
<td>945</td>
<td>46,653</td>
<td>70</td>
<td>665 / 628 / 569 / (483)</td>
<td>ca. 8,000</td>
<td></td>
</tr>
<tr>
<td>K1-M20</td>
<td>527</td>
<td>8,914</td>
<td>70</td>
<td>371 / 350 / 318 / 270</td>
<td>ca. 109,000</td>
<td></td>
</tr>
<tr>
<td>K1-M24</td>
<td>588</td>
<td>9,078</td>
<td>70</td>
<td>413 / 390 / 354 / 300</td>
<td>ca. 42,000</td>
<td></td>
</tr>
<tr>
<td>K2_z</td>
<td>437</td>
<td>3,680</td>
<td>70</td>
<td>306 / 289 / 262 / 222</td>
<td>ca. 170,000</td>
<td></td>
</tr>
<tr>
<td>K2-M20</td>
<td>700</td>
<td>13,897</td>
<td>70</td>
<td>480</td>
<td>ca. 13,000</td>
<td></td>
</tr>
<tr>
<td>K2-M24 (1)</td>
<td>807</td>
<td>14,237</td>
<td>70</td>
<td>560 / 529 / 480 / (407)</td>
<td>ca. 24,000</td>
<td></td>
</tr>
<tr>
<td>K2-M24 (2)</td>
<td>807</td>
<td>14,237</td>
<td>41</td>
<td>334 / 316 / 286 / 243</td>
<td>ca. 293,000</td>
<td></td>
</tr>
<tr>
<td>K3-M20</td>
<td>840</td>
<td>23,406</td>
<td>70</td>
<td>590 / 518 / 451</td>
<td>ca. 26,000</td>
<td></td>
</tr>
<tr>
<td>K3-M24 (1)</td>
<td>1,350</td>
<td>24,042</td>
<td>70</td>
<td>945 / 892 / 809 / (687)</td>
<td>ca. 13,000</td>
<td></td>
</tr>
<tr>
<td>K3-M24 (2)</td>
<td>1,350</td>
<td>24,042</td>
<td>53</td>
<td>715 / 675 / 612 / 520</td>
<td>ca. 154,000</td>
<td></td>
</tr>
<tr>
<td>K3-M24 (3)</td>
<td>1,350</td>
<td>24,042</td>
<td>35</td>
<td>468 / 442 / 401 / 340</td>
<td>ca. 563,000</td>
<td></td>
</tr>
</tbody>
</table>

(*) - load step planned but not reached

The static tests demonstrated that both traditional/conservative bracket specimens, Z1 and Z2, had the same load bearing capacity irrespective of the joint arrangement. The arrangement had an effect on the initial stiffness only. Specimen Z2 had a higher stiffness than specimen Z1, but the fatigue behaviour of specimen Z1 was more favourable.

In tests Z1 and Z2 the achieved number of load cycles was significantly lower than in the case of test K2, which was the nearest light bracket arrangement according to its geometry. In both conservative design solutions failure was achieved due to rupture in the end-plate at the height of the tension flange as shown in Figure 2.137.

Failure occurred the “classical way”, i.e. at low load cycles (13,000 and 8,000, respectively), at stress concentrations along the welds. The apparently poor fatigue behaviour of this arrangement is explained by its “excessive” rigidity, which prevents the development of an elastic response when such loading is applied.

![Fig. 2.137 Failure by rupture in the end-plate.](image-url)
Figure 2.138 shows the two load histories of test specimen K2-M24. The specimen achieved 293,000 load cycles when subjected to the lower load steps (41%) and around 24,000 load cycles when subjected to the higher load steps (70%).

This demonstrates the influence of the load spectrum on the fatigue behaviour of the joint.

Figure 2.139 presents the load history of test specimens K2_z and K2-M24. Both specimens had the same geometry, but in specimen K2_z the stiffener in the compression zone and the backing plate was omitted. Subjected to the same load spectrum, chosen according to their load capacity, the “flexible” arrangement K2_z achieved 170,000 load cycles against the 24,000 cycles of K2.

Figure 2.140 shows the load histories of test K3 with bolts M24 for three different load spectra. The diagram demonstrates the difference in fatigue behaviour. The higher the load steps of fatigue loading as compared to the static load bearing capacity (i.e. the closer actual stresses are to the yield strength), the lower the number of cycles that causes the failure of the joint.
When the joint was subjected to 70% and 53% of the static load bearing capacity respectively in the first step, joint failure was due to the rupture of the end plate at the height of the upper flange; when the same joint was initially subjected to 35% of the static load bearing capacity, testing was stopped at 563,000 cycles.

After evaluation of the results of the fatigue tests, which were performed under different load histories (K2-M24 and K3-M24), it can be concluded that the load spectrum with an initial load level corresponding to 50% of the static load bearing capacity seems to be appropriate to achieve 200,000 cycles.

For an easier evaluation of the fatigue behaviour the secant stiffness vs. number of cycles curves were approximated with parameters summarized in Table 2.12.

Table 2.12 Summary of the parameters of the approximation curves.

<table>
<thead>
<tr>
<th>joint arrangement</th>
<th>test</th>
<th>maximum fatigue load / static load bearing capacity [%]</th>
<th>number of load cycles achieved</th>
<th>parameter “a”</th>
<th>parameter “b”</th>
<th>parameter “c”</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z1</td>
<td>Z1</td>
<td>70</td>
<td>ca. 13,000</td>
<td>0.978</td>
<td>7.90 x 10^6</td>
<td>-1.79 x 10^9</td>
</tr>
<tr>
<td></td>
<td>Z2</td>
<td>70</td>
<td>ca. 8,000</td>
<td>1.067</td>
<td>3.67 x 10^5</td>
<td>-6.11 x 10^9</td>
</tr>
<tr>
<td>K1-M20</td>
<td>70</td>
<td>ca. 109,000</td>
<td>1.057</td>
<td>2.54 x 10^6</td>
<td>-3.66 x 10^11</td>
<td></td>
</tr>
<tr>
<td>K1-M24</td>
<td>70</td>
<td>ca. 42,000</td>
<td>0.979</td>
<td>4.05 x 10^6</td>
<td>-2.09 x 10^10</td>
<td></td>
</tr>
<tr>
<td>K2-z</td>
<td>70</td>
<td>ca. 170,000</td>
<td>1.052</td>
<td>1.26 x 10^6</td>
<td>-1.36 x 10^11</td>
<td></td>
</tr>
<tr>
<td>K2-M20</td>
<td>70</td>
<td>ca. 13,000</td>
<td>0.989</td>
<td>7.97 x 10^6</td>
<td>-7.87 x 10^10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>test continued</td>
<td></td>
<td></td>
<td>0.993</td>
<td>-7.63 x 10^6</td>
<td>-6.17 x 10^10</td>
</tr>
<tr>
<td>K2-M24 (1)</td>
<td>70</td>
<td>ca. 24,000</td>
<td>0.929</td>
<td>6.10 x 10^6</td>
<td>-6.24 x 10^10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>test continued</td>
<td></td>
<td></td>
<td>0.978</td>
<td>1.08 x 10^6</td>
<td>-4.42 x 10^12</td>
</tr>
<tr>
<td>K2-M24 (2)</td>
<td>41</td>
<td>ca. 293,000</td>
<td>1.019</td>
<td>7.29 x 10^7</td>
<td>-5.69 x 10^11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>test continued</td>
<td></td>
<td></td>
<td>1.012</td>
<td>8.69 x 10^7</td>
<td>-7.80 x 10^12</td>
</tr>
<tr>
<td>K3-M20</td>
<td>70</td>
<td>ca. 26,000</td>
<td>1.019</td>
<td>7.29 x 10^7</td>
<td>-5.69 x 10^11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>test continued</td>
<td></td>
<td></td>
<td>1.012</td>
<td>8.69 x 10^7</td>
<td>-7.80 x 10^12</td>
</tr>
<tr>
<td>K3-M24 (1)</td>
<td>70</td>
<td>ca. 13,000</td>
<td>1.027</td>
<td>1.65 x 10^5</td>
<td>-2.81 x 10^9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>test continued</td>
<td></td>
<td></td>
<td>1.054</td>
<td>5.59 x 10^7</td>
<td>-6.13 x 10^12</td>
</tr>
<tr>
<td>K3-M24 (2)</td>
<td>53</td>
<td>ca. 154,000</td>
<td>1.015</td>
<td>-5.54 x 10^8</td>
<td>5.4 x 10^14</td>
<td></td>
</tr>
<tr>
<td>K3-M24 (3)</td>
<td>35</td>
<td>ca. 563,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Parameter “c” refers to the “openness” of the parable, i.e. a low value of parameter “c” indicates that a high number of cycles can be achieved. The results in Table 2.12 show that if the index of “c” was lower than -10, the achieved number of load cycles was under 20,000. If the index was lower than -11, the achieved number of load cycles was between 20,000 and 42,000. When, however, the index was higher than -11, the achieved number of load cycles was higher than 100,000 cycles.

The above did not hold in two cases: for tests K2-M20 and K3-M20. When the parameters of the approximative curve, i.e. the secant stiffness, indicated bolt failure to be the cause of the failure of the specimen, favourable fatigue behaviour was forecasted.

Table 2.13 shows all tests performed, listed according to the calculated rotational stiffness and contains only those tests in which the maximum fatigue load was 70% of the static load bearing capacity. The results demonstrate that the higher the stiffness, the lower the number of the load cycles achieved.

In the Table 2.13 two exceptions are also shown: tests K2-M20 and K3-M20. Both had a relative low value of the parameter “c”, and in spite of this, they achieved a low load cycle number only. These specimens, however, failed by bolt failure contrary to the other tests, where end-plate or column web failure was observed.

In this chapter it was demonstrated by several tests that the value of parameter “c” changes to a very low extent after the first 10,000 cycles. It means that one can calculate the approximation curve after an appropriate limit, for example after 10,000 cycles, to forecast the longer-term fatigue behaviour of the bracket.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Crane bracket joints

Table 2.13 Test results according to the calculated joint stiffness.

<table>
<thead>
<tr>
<th>test</th>
<th>Maximum fatigue load / static load bearing capacity [%]</th>
<th>Calculation results with the modified EC3 model (actual material properties and partial factors $\gamma_M = 1.0$, $\gamma_K = 1.0$)</th>
<th>number of load cycles achieved</th>
<th>parameter “c”</th>
</tr>
</thead>
<tbody>
<tr>
<td>K2_z</td>
<td>70</td>
<td>3,680</td>
<td>ca. 170,000</td>
<td>$-1.36 \times 10^{-11}$</td>
</tr>
<tr>
<td>K1-M20</td>
<td>70</td>
<td>8,775</td>
<td>ca. 109,000</td>
<td>$-3.66 \times 10^{-11}$</td>
</tr>
<tr>
<td>K1-M24</td>
<td>70</td>
<td>9,515</td>
<td>ca. 42,000</td>
<td>$-2.09 \times 10^{-10}$</td>
</tr>
<tr>
<td>K2-M20</td>
<td>70</td>
<td>(bolt failure, constant 70% load step!) 13,830</td>
<td>ca. 13,000</td>
<td>$-7.87 \times 10^{-10}$</td>
</tr>
<tr>
<td>K2-M24</td>
<td>70</td>
<td>14,667</td>
<td>ca. 24,000</td>
<td>$-6.24 \times 10^{-10}$</td>
</tr>
<tr>
<td>K3-M24</td>
<td>70</td>
<td>24,042</td>
<td>ca. 13,000</td>
<td>$-2.81 \times 10^{-9}$</td>
</tr>
<tr>
<td>K3-M20</td>
<td>70</td>
<td>(bolt failure) 24,421</td>
<td>ca. 26,000</td>
<td>$-5.69 \times 10^{-11}$</td>
</tr>
<tr>
<td>Z1</td>
<td>70</td>
<td>29,063</td>
<td>ca. 13,000</td>
<td>$-1.79 \times 10^{-9}$</td>
</tr>
<tr>
<td>Z2</td>
<td>70</td>
<td>46,653</td>
<td>ca. 8,000</td>
<td>$-6.11 \times 10^{-9}$</td>
</tr>
</tbody>
</table>

2.5 Recommendations for practical design

For practical applications, the following conclusions are drawn:

- The load bearing capacity and stiffness of a bolted crane bracket joint subjected to static loading can be enhanced, even without modifying the overall geometry of the joint, with no danger within certain limits. When there is fatigue loading, however, load bearing capacity should be increased by applying larger overall dimensions rather than by introducing additional stiffeners.

- In joints under fatigue loading, abrupt premature failure such as bolt failure should be avoided by innovative solutions. The final failure, which leads to the loss of the resistance, should occur after significant plastic deformations. For configurations with a lower load bearing capacity the applied maximum fatigue load (i.e. load spectrum) was also lower, although it can still be stated that experimental observations indicated that the light crane bracket construction ensure favourable fatigue behaviour. This can be explained by elastic plate deformations of the web, which deformations eliminate energy.
  - In joints under fatigue loading, joint ductility is an important consideration; it is to ensure adequate behaviour under repeated loading.
  - It is better not to stiffen the joint under fatigue loading. Tests show that specimens with higher stiffness fail earlier, i.e. under lower load cycles.
  - Note that the combined bolt and plate failure of the T-stub (Mode 2) leads to favourable fatigue joint behaviour.

- Backing plate design has advantages over the use of welded stiffeners as its behaviour is more favourable with respect to load capacity and ensures better behaviour under fatigue loading, while easier to install and thus cheaper.

- Light crane brackets without compression flanges are useful alternatives if the web thickness is chosen adequately.
3 Beam-to-beam joints

3.1 Summary of the research programme

This chapter presents the results of an experimental study on innovative beam-to-beam joints of industrial frames. The experimental programme included eight different joint arrangements and covered eighteen specimens made by the industry.

3.1.1 Test specimens

Test series I

Figure 3.1 shows, as an example, test specimen TB2 from test series I with its main dimensions. The detailed geometry of the specimens can be found in Appendix A.

The specimens were erected between so called fixed girders as shown in Figure 3.2. These supporting beams served the economy of the fabrication of the specimens.

The following dimensions were identical for all test specimens: beam length 800 mm; beam height 900 mm; web thickness 8 mm; flanges 360 x 20 mm; end-plate width 360 mm and the load application point (i.e. the lever arm).

The steel grade of the test specimens was S355, the bolt grade 8.8; the bolt diameter M20 and M24 in the fixed connections with normal clearances and washers.

Table 3.1 shows a summary of the test specimens of series I with their bolt geometry, end-plate type and end-plate thickness.

Table 3.1 Test specimens in series I.

<table>
<thead>
<tr>
<th>end-plate arrangement</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>test specimen</td>
<td>TB2</td>
<td>TB6</td>
<td>TB10</td>
<td>TB3</td>
</tr>
<tr>
<td>end-plate type</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>end-plate thickness $t_{lp}$ [mm]</td>
<td>12</td>
<td>15</td>
<td>20</td>
<td>12</td>
</tr>
</tbody>
</table>

Fig. 3.1 Test specimen TB2.

Fig. 3.2 The test arrangement.
For the tests the bolts were preloaded. First the fixed connections were erected, the bolts were preloaded by a machine, and immediately thereafter the examined joint was made. This sequence of erection simulates the practical solutions well, where the beam has to be inserted in between existing columns.

In order to get a homogeneous preload level in the bolts of the investigated connection, a three-step preload process was used. First the bolts were strained manually; in the second step, beginning with the bolt-row farthest from the compression flange, the preload was applied by a pneumatic screwdriver; and finally, because of the end-plate deformations and other imperfections, the bolts were preloaded again with the pneumatic tool. During the preloading process the bolt forces, i.e. preload levels, were measured by the applied load cells. The exact values of the bolt preload can be found in Appendix C.

Test series II

Table 3.2 shows a summary of the test specimens in series II.

Table 3.2 Test specimens in series II.

<table>
<thead>
<tr>
<th>end-plate arrangement</th>
<th>TA</th>
<th>TB</th>
<th>TC</th>
<th>TD</th>
<th>TE</th>
<th>TF</th>
</tr>
</thead>
<tbody>
<tr>
<td>test specimen</td>
<td>TA</td>
<td>TB</td>
<td>TC</td>
<td>TD</td>
<td>TE</td>
<td>TF</td>
</tr>
<tr>
<td>end-plate thickness t₀ [mm]</td>
<td>16</td>
<td>20</td>
<td>20</td>
<td>16</td>
<td>20</td>
<td>24</td>
</tr>
</tbody>
</table>

As an example, Figure 3.3 presents test specimen TA with its main plate dimensions.

Similarly to the case of test series I, the following dimensions were identical for all test specimens: beam length 800 mm; beam height 900 mm; web thickness 8 mm; flange thickness 20 mm, dimensions of stiffeners: triangular 200 x 90 mm with 9 mm thickness, and the load application point (i.e. the lever arm) 1,000 mm from the tested joint.

In tests with specimens TA, TB, TC and TD the flange width was 300 mm, and in tests TE and TF, 420 mm.

The steel grade of the test specimens was S355, the bolt grade 10.9; the bolt diameter M20 and M24 in the fixed connections.

The erection of the specimens and the preloading process of the bolts were identical to those of test series I. From a practical point of view, the same fixed girders were used (Figure 3.2) in both series.
3.1.2 Test arrangement

For the tests a four-point-bending arrangement was used, in which the test specimens were easy to install and replace, as shown in Figure 3.2.

The static system was a simply supported beam with two concentrated loads applied in the vertical axis, by two vertical forces placed at 3,000 mm from the supports symmetrically, as shown in Figure 3.4.

Because of this load configuration the tested part, including the investigated joint, was loaded under pure bending. To prevent lateral torsional buckling of the beam, both flanges were restrained by plates close to the load introduction point, as shown in Figures 3.4 and 3.6.

Figure 3.7 illustrates the pinned support of the fixed girder.

---

3.1.3 Measuring system

During the tests representative displacements were measured by inductive transducers placed under the loads and in the cross-section of the investigated joint. The distribution of the bolt forces was registered by load cells. The calibration of the load cells is described in Appendix E.

The measured data were collected in each second, by two HBM Spider (Hottinger Baldwin Messtechnik) data collection systems.

Figure 3.8 shows the inductive transducer placed in the middle, over the investigated joint. This transducer was installed on an RHS profile, which was supported, as a simply supported beam, on the end-plates of the fixed girder as shown in Figure 3.9 (blue lines). This solution allowed to measure the relative deflection of the tested joint.
Figure 3.10 shows the point where the inductive transducer, with a maximum displacement capacity of 100 mm, was placed under the load application point. These transducers were applied under both hydraulic jacks.

Figure 3.11 shows schematically the locations of the transducers and the load cells.

3.1.4 Test programme

Before the static tests the load bearing capacities of the joints were calculated analytically and these results were used for the applied load levels. The capacities of the joints were reached in four load steps. Each specimen in the first load step was loaded by 50 kN, the second load step was the pre-calculated elastic limit, i.e. 2/3 of the calculated load capacity. The third load step was equal to the calculated load bearing capacity, and the fourth was at the level of the experimental load bearing capacity of the joint in question.

3.2 The design method

3.2.1 HammerHead arrangement and additional stiffeners

General

According to the EN 1993-1-8 “Design of joints” standard, it is up to the designer to assess the design effect of additional stiffeners between the bolt-rows. The “standard” joint configuration is limited to arrangements where the bolts are placed between the flanges and/or one bolt-row is placed in the extended end-plate part.

The HammerHead arrangement as shown in Figure 3.13, however, includes two bolt rows in the extended tension zone of the end-plate as well as an additional flange. The problem to solve was how to take into account the effect of a three-flange end-plate joint, or, more generally, how to apply the EC3 design method in the case of “freely placed” stiffeners between any two bolt-rows?

Definition of the problem

The EC3 component method model was extended to cover optional stiffeners which can be freely placed between the bolt-rows. More about the EC3 model can be found in Appendix F “Summary of the Eurocode 3 model”.

Figure 3.12 shows the possible allocation of the additional stiffeners. They can be used to avoid bolt group failure, as explained in Figure 3.12, and/or to enhance the tension resistance of the web.

In the design method the following model assumptions were made:

- The steel grade of the stiffeners is not lower than that of the member.
- The stiffeners are not taken into account in the cross-section classification of the member and do not have any influence on the moment resistance of the original member.
- The width of the stiffener has to be equal to the width of the end-plate or has to extend beyond the bolt by a distance of at least 2.0 \( \cdot \) \( d_0 \) (practical assumption in the model).
The HammerHead arrangement presented in Figure 3.13 can be designed as a beam with optional stiffeners. The following model assumptions were made:

- The steel grade of the HammerHead part is not lower than that of the member.
- The HammerHead part is not taken into account in the cross-section classification of the member and does not have any influence on the moment resistance of the original member.
- The flange width and the web thickness of the HammerHead part is not less than that of the member.
- There are exactly two bolt-rows in the HammerHead part of the joint.

**Design method**

- The resistance of each T-stub within the joint was evaluated row by row. On the basis of the end-plate geometry, bolt arrangement, position of the flanges and stiffeners the \( l_{eff} \) lengths could be calculated. In the calculation of the \( l_{eff} \) lengths the stiffeners were considered as flanges, i.e. it was assumed that both joint elements had the same effect on the \( l_{eff} \) lengths.
- The T-stub idealization is based on the assumption that the T-stub can fail according to three possible plastic collapse mechanisms: Mode 1 mechanism is characterized by complete plate yielding, Mode 2 corresponds to bolt failure with plate yielding, and the third collapse mechanism, Mode 3, involves bolt failure only.
- The T-stub resistances were calculated from the \( l_{eff} \) lengths and included the resistance of the end-plate in bending (column flange in bending) and bolts in tension.
- In view of the \( l_{eff} \) lengths the resistance of the beam web in tension (column web in tension) can be calculated row by row.
- The effective design tension resistance of bolt-row “i” is the minimum of the T-stub resistances corresponding to row “i” and the resistance of the beam web in tension calculated for row “i” (column web in tension).
- The resistance of the beam web and flange in compression (column web panel in shear, column web in transverse compression) is calculated for the whole joint, i.e. this resistance is representative for the whole joint.
- The design moment resistance of the joint can be determined from the sum of the products involving the effective design tension resistances of the bolt-rows and the appropriate lever arms.

\[
M_{j,Rd} = \sum h_i \cdot F_{T,i,Rd}
\]  

(3.1)

where:
- \( h_i \) - is the distance from bolt-row i to the centre of compression
- \( F_{T,i,Rd} \) - is the effective design tension resistance
- \( i \) - is the serial number of the bolt-row

Note that the sum of the effective design tension resistances of the bolt-rows \( \sum F_{T,i,Rd} \) cannot exceed the resistance of the beam web and flange in compression (column web panel in shear, column web in transverse compression), i.e. the minimum resistance which is representative for the whole joint.

Appendix H contains the design moment calculation as example for specimen TB5.
3.2.2 End-plate type connections with four bolts in one bolt-row

General

The EC3 applies the component method for the plastic design of bolted end-plate type joints, and the model is based on the T-stub model. In this method the bolt arrangement is assumed to be symmetrical about the web and the connection is modelled by T-stubs containing the end-plate, the web and the two bolts on both sides of the web. The tension resistance of each bolt-row is evaluated by the analysis of the T-stub.

The T-stub model of bolted end-plate type joints is illustrated in Figure 3.14.

Definition of the problem

The design method of the EC3 provides application rules for those connections only which have two bolts in each bolt-row. For this geometrical arrangement the yield line patterns are defined and the leff values (leff = the effective length of the yield line pattern) can be calculated.

But in special cases, the industry needs end-plates with four bolts in a row. The problem is that these possible yield line patterns are not defined in the EC3 standard.

The developed method is based on the EC3 component method model, but it extends its application to connections with either two or four bolts, in any combination, in each of the bolt-rows. The proposed method has to be in accordance with the T-stub model, which was developed for the design of all possible joint configurations.

Strategy of the solution

The T-stub model remains the basis of the developed design method, however, the web of the T-stub comes from the flange or the horizontal stiffener of the member. In this modified model the T-stub is rotated by 90° and its bolts are symmetrical about the flange, as shown in Figure 3.15. The model accordingly uses the T-stub derivation.

The T-stub model can be applied only when the bolts are located symmetrically about the web. This requirement is not met automatically in each T-stubs of the connection having four bolts in a row, for example when there are four bolts in the extended part of the end-plate and two bolts under the tension flange, as shown in Figure 3.16.

This problem could be solved by simple design rules, for example by the use of extra bolts or stiffeners as shown in Figure 3.17. However, this solution also generates higher costs.

The initial purpose of the development of the design method was to establish a calculation model which does not contain any restriction with regard to the bolt arrangement, i.e. this arrangement can be determined without strict regulations, while it is harmonised to the EC3 standard, i.e. uses the same basic elements, T-stubs, for the calculations.
For these reasons the application of a $\beta$-factor was recommended to take into consideration the non-symmetrical nature of the T-stub. The resistance of the bolt of the non-symmetrical T-stub ($F_{T,1/2/3,Rd}$) is multiplied by a $\beta$-factor as follows:

$$F_{T,1/2/3,Rd}^\beta = F_{T,1/2/3,Rd} \times \beta$$  \hspace{1cm} (3.2)

This empirical factor can have the values between 0.5 and 1.0. When the resistance of the T-stub is governed by the plate-type behaviour (Mode 1), then $\beta = 0.5$; when by the bolt failure (Mode 3), then $\beta = 1.0$ (EN 1993-1-8, Table 6.2).

The application of the $\beta$ factor ensures that the model takes into consideration the highest possible number of bolts in the joint during the evaluation of the moment resistance.

The following model assumptions were set up:

- In the case of the four-bolt arrangement, only those bolts have influence on the moment resistance of the connection which have a flange or a stiffener at least on one side. If two rows of four bolts are applied in the connection without stiffeners between these rows, certain bolts act only in shear as shown in Figure 3.18.

- The symmetry of the bolt distances in the T-stubs is not automatically warranted. In the developed method, if there are four bolts in a bolt-row, the bolt arrangement is controlled and the distances between the bolt-rows and the stiffeners are checked for symmetry. An asymmetrical T-stub bolt geometry situation is shown in Figure 3.19.

- The width of the flange or the stiffener has to be checked so as to make sure that it is able to play the role of the web of the T-stub.

- For the stiffeners the same assumptions were established as indicated above for the case of additional stiffeners.

- The original EC3 model, which assumes two bolts in one bolt-row, assumes uniform force distribution among the bolts, i.e. the bolt forces are assumed to be symmetrical about the web. In the case of four bolts in a row, the "outer" or "inner" location of the bolts ("outer" = towards the edge of the end-plate; "inner" = adjacent to the web) has to be taken into consideration, i.e. the forces in the "outer" and "inner" bolts are not the same.

- In the case of four bolts in a row, there are several possibilities for the yield line patterns to occur, which are developed and summarized in Figures 3.21 and 3.22. Due to the possible group failure in a bolt-row, more complex patterns are developed than in case of two bolts, because the bolts are separated by the web. The outer bolts form groups with the outer bolt of the previous or the next bolt-row, depending on whether or not they belong to the group of their own row.
The advantage of the developed model is that, after defining the new yield line patterns and the design rules described above, the EC3 component method model can be used and the resistance of the joint can be calculated without any further modification. Nevertheless, the definition of the Mode 2 yield line patterns is incomplete for bolt-rows below the flanges. So in these cases the design method can overestimate the effective design tension resistance \( F_{T,i,Rd} \) because the method calculates the minimum resistance from Mode 1 and Mode 3 only. Further studies have to describe the patterns that belong to Mode 2 so that this T-stub resistance can be calculated for all bolt arrangements.

**Design method**

- The resistance of each T-stub within the joint was evaluated row by row. First the \( \beta \) factor is determined for all bolts. For symmetrically placed outer bolts, as shown in Figure 3.19, and for inner bolts, \( \beta = 1.0 \), whereas for asymmetrically placed outer bolts, as shown in Figure 3.17, \( \beta = 0.5 \). Certain outer bolts, i.e. those placed between two bolt-rows without any stiffener, as shown in Figure 3.18, were considered in shear only (\( \beta = 0 \)).

  The T-stub resistances were calculated from the \( l_{eff} \) lengths. The \( l_{eff} \) lengths are determined, considering all possible yield line patterns, as shown in Figures 3.21 and 3.22. After calculating all potential individual and group failure patterns, and considering the three failure modes of the T-stub, the effective design tension resistance for each bolt can be determined. Then, from the effective design tension resistances of the bolts, the resistance for the bolt-row is calculated.

- From this point on, the calculation follows the same line as described for the design method for the HammerHead arrangement and/or for joints with additional stiffeners.
Fig 3.21 Possible yield line patterns in the extended part of the end-plate.
Fig 3.22 Possible yield line patterns inside the flange.
3.3 Test series I

3.3.1 Test results

The measured data are presented in moment vs. bolt-row force diagrams, moment-deflection diagrams as well as figures of the deformed end-plate contour-lines and surfaces. On the basis of the evaluated test results the ultimate behaviour and the failure modes are determined and classified.

In parallel with the tests design, resistance calculations have been completed and the developed EC3 based design method is compared to and verified against the experimental results.

The moment vs. bolt-row force diagrams (for example Figure 3.24) show the relationship of the measured force in the bolt-rows and the moment in the tested joint. The different bolt-row force curves are presented in different colours. The colour spectrum is fixed as follows: red, blue, green, light blue, orange and pink, beginning from the bolt-row farthest from the centre of compression.

The diagrams present data up to the last load step, i.e. until the attainment of the load bearing capacity. Because of the previous load steps and the different pre-tension levels of the bolts, the presented bolt-force curves have different starting points.

For reasons of clarity, in each diagram of the measured bolt forces, the dissertation always presents the force for the whole bolt-row. Measurements confirmed that the force differences between the bolts within a bolt-row were in most cases under 10%. In the case of four bolts in one row (end-plate types II and III) the force differences between inner and outer bolts were of course higher.

Inner bolts are the bolts closer to the web plate, and outer bolts are those that are adjacent to the edge of the end-plate. In this case the measured force distribution in the bolt-row is also presented.

The moment-deflection diagrams show the relationship of the measured deflection under the hydraulic jacks and the moment in the tested joint.

The deflection curve under the left hand side hydraulic jack is plotted in red and the one under the right hand side jack, in blue. The relative deflection in the cross-section of the joint is marked in green.

After the load bearing tests the end-plate deformations were measured in the test specimens, and the results are presented in the form of surfaces or contour-lines (Figure 3.26, Figure 3.27). The benchmark data were collected in the web direction in each 0.25 mm at an altitude accuracy of 0.001 mm. On each end-plate, eleven contour-lines were fixed in a uniform distribution. The detailed description of the measurements and the evaluation, as well as the results can be found in Appendix D.

For all tests the moment resistance, the bolt forces and the failure modes were calculated on the basis of the developed design model. The calculations assume a geometrically perfect model with standard material properties, for plated elements $f_y = 355 \text{ N/mm}^2$, $f_u = 510 \text{ N/mm}^2$ (S355) and for bolts $f_y = 640 \text{ N/mm}^2$, $f_u = 800 \text{ N/mm}^2$ (8.8). In order to get comparable results with the tests the partial factors were taken into account by the value of $1.0$ ($\gamma_M = 1.0$; $\gamma_{M1} = 1.0$; $\gamma_{M2} = 1.0$).

After the tests this model was re-evaluated using the actual material properties for plates and bolts. Twelve tensile test specimens were cut out from the end-plates (B2, B6 and B10) for the purpose of the material tests, which means four specimens for each end-plate thickness. After the material tests the obtained material properties were related to the end-plate thickness. It means that for all tests with a 12 mm thick end-plate, i.e. for all plated elements, the same yield strength and tensile strength were assumed.

The material tests on the bolts were also made in the laboratory of the BME, Department of Structural Engineering. Altogether twelve bolts were included in the material tests. For the re-calculation of the design moment resistance, the average tension resistance was utilised. More detailed information about the material properties can be found in Appendix B.

The connection type and the positions of the load cells can be seen in Figure 3.23. Note the similarities between the end-plate types. The end-plate types show differences only in the tension zone: types I and II have HammerHead extensions with two additional bolt-rows, whereas types III and IV include an extended end-plate with one additional bolt-row only. The bolts in all specimens were placed according to the same system of gridlines. The plate dimensions of the member and the HammerHead part were identical.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Beam-to-beam joints

L. Katula

End-plate type I: tests TB2, TB6, TB10
End-plate type II: tests TB3, TB7, TB11
End-plate type III: tests TB4, TB8, TB12
End-plate type IV: tests TB5, TB9, TB13

Fig. 3.23 Positions of the load cells.

3.3.1.1 End-plate type I - HammerHead connection with 2 bolts in both bolt-rows in the extended part of the end-plate

Test specimen TB2 ($t_{ep} = 12$ mm)

The moment vs. bolt-row force diagram in Figure 3.24 shows that the highest force increment was measured in bolt-row C, followed by bolt-rows B, D and A.

The diagrams present approximately the same force increments for bolt-rows adjacent to the tension flange (bolt-rows C and B).

The curves do not exhibit a definite change, which would indicate force redistribution among the bolt-rows. The absence of such a change indicates dominant plate failure (Mode 1).

The plate failure, via the evolving yield lines, works for the bolt forces in a manner similar to an elastic bedding.

Figure 3.25 presents the moment-deflection diagrams of the test: the red and blue curves, recorded deflection under the jacks, show similar characteristics. The deflection curves show linear behaviour below the level of the calculated design moment ($M_{j,Rd} = 929$ kNm), which indicates elastic joint behaviour.

The curve in green shows the relative deflections as measured in the cross-section of the tested joint (Figure 3.9) as well as the measured moment relationships.

Fig. 3.24 Moment vs. bolt-row force diagrams, test TB2.

Fig. 3.25 Moment-deflection diagrams.
Fig. 3.26 The contour-lines of the deformed end-plates.

Figure 3.26 shows the results of the measurements of the deformed end-plate and presents the deformations by contour-lines. Figure 3.27 presents the deformed shape of the end-plate calculated by linear interpolation between the contour-lines.

The surface illustrates bolt group failure in bolt-rows A and B, as well as on the other side of the flange, in bolt-rows C and D. This experimental observation confirms the results of the calculation. The calculated failure was bolt-group failure in the HammerHead part in Mode 1 and group failure involving bolt-rows C and D in Mode 2.

Note that Figures 3.26 and 3.27 show deformations at the height of the HammerHead flange. This effect shows that the support conditions in the HammerHead part were not the same as between the flanges of the member.

Figures 3.28 and 3.29 illustrate the test specimen before the test and the deformed tension zone respectively. The observed ultimate behaviour of the joint was dominant plate failure. The plate deformations, especially those close to the tension bolts, are shown in Figure 3.29.

Although Figure 3.29 shows a broken bolt, this bolt failure occurred after the complete yielding of the end-plate, i.e. after high plastic plate deformations. In this case two reasons play a role in the bolt failure: the ductility of the bolt is exhausted and/or the bolt is under bending.

Fig. 3.28 Test specimen TB2 before test.

Fig. 3.29 The tension zone after failure.
Test specimen TB6 ($t_{ep} = 15 \text{ mm}$)

The moment vs. bolt-row force diagrams in Figure 3.30 show similar bolt force behaviour as for test specimen TB2. The highest force occurs in the bolt-rows adjacent to the tension flange (rows C and B).

A slight degree of force redistribution can be seen on the diagrams at about 1,100 kNm. At this point, the rate of force increase in bolt-rows B and C decreased, whereas in rows A and D, increased. This change indicates combined plate and bolt yielding. If the trend curve of the increments changes in a “soft” manner, then mixed plate and bolt failure (Mode 2) occurs; in this case, plate yielding accounts for the change in the rise of the curve. An abrupt change in the trend curve, however, would indicate bolt failure (Mode 3).

Figure 3.31 shows the shape of the deformed end-plate. The Figure demonstrates that the failure of the joint was due to failure in bolt-rows A and B, as well as in bolt-rows C and D in the form of bolt group failure. This observation confirmed the results of the calculation. The calculated failure was bolt-group failure involving bolt-rows A and B in Mode 2 and group failure involving bolt-rows C and D in Mode 2 too.

Note that the end-plate deformations at the height of the HammerHead flange are similar to test TB2. This is due to the “weaker” support of the HammerHead flange and web.

The ultimate behaviour of the joint was combined bolt-and-plate failure (Mode 2).

Figure 3.32 illustrates the test specimen and Figure 3.33, the observed joint failure.
Test specimen TB10 (t_{ep} = 20 mm)

The diagrams in Figure 3.34 present higher force increments in the bolt-rows adjacent to the tension flange (bolt-rows B and C), but this tendency changed at ca. 1,100 kNm.

At this load level, yielding occurred in the bolts in bolt-rows B and C, and at the same time, the force in bolt-rows A and D started to grow at a higher rate. This change indicates combined plate and bolt yielding.

The recorded bolt force levels were the same in rows B and C as well as in rows A and D. The force distribution was similar to that of specimen TB6.

The behaviour of bolt-row E was also similar to that of specimen TB6.

Figure 3.35 shows the generated deformed shape of the end-plate and it indicates slight plate deformations only because of the relatively thick end-plate compared to the bolt diameter.

The calculated failure mode was bolt group failure involving bolt-rows A and B as well as bolt-rows C and D in Mode 2. The measured shape of the deformed end-plate in Figure 3.35 suggests pure bolt failure. This means that the ultimate behaviour of the joint was governed by bolt failure (Mode 3).

Figure 3.36 illustrates the joint failure in bolt-rows A, B, C and D and the broken bolts, respectively.
3.3.1.2 End-plate type II - HammerHead connection with 4 bolts in the first bolt-row and 2 bolts in the second bolt-row in the extended part of the end-plate

**Test specimen TB3 (t<sub>ep</sub> = 12 mm)**

Figure 3.37 illustrates the five measured bolt-row forces as a function of the applied moment. The values are independent from the number of bolts within each bolt-row.

The diagrams show that the bolt-force increments were higher in the bolt-rows adjacent to the tension flange (in rows B and C) than in the other tension bolt-rows.

The rise of the bolt-row force diagram for bolt-rows A (red curve) and D (light blue curve) is nearly the same. Well-defined changes cannot be seen in the diagrams. This is an indication of a plate failure (Mode 1).

Figure 3.38 shows the inner (A and A2) and outer (AA and AA2) bolt forces for bolt-row A separately. The force curves show similar levels due to the adjacent HammerHead flange. But at the same time, significantly higher forces developed in other tension bolt-rows.

Figure 3.39 illustrates the deformed shape of an end-plate. The figure shows that the bolts in bolt-rows A and B were part of a group, and the same can be said about the bolts in bolt-rows C and D. These observations confirmed the results of the calculation. The calculated failure mode was group failure in the HammerHead part in Mode 1 and bolt-group failure involving bolt-rows C and D in Mode 2.

Note that, similarly to specimens TB2 and TB6, Figures 3.39 and 3.40 show large deformations at the height of the HammerHead flange. This indicates that the support conditions of the end-plate in the HammerHead part and in the member were different.

The ultimate behaviour of the joint was dominant plate failure (Mode 1).

Figure 3.40 illustrates the observed joint failure.
Test specimen TB7 (t_{op} = 15 mm)

Figure 3.41 illustrates, independent from the number of bolts in each bolt-row, the measured bolt-row forces for the whole bolt-row.

The observed behaviour was similar to that of specimen TB3. The diagram shows that the bolt-force increments were higher in bolt-rows adjacent to the tension flange than in the other tension bolts. The rise of the bolt-row force diagrams for rows A and D was also nearly the same.

The diagrams show changes at approximately 1,400 kNm, where force redistribution began. The force increase in rows B and C slowed down, while rows A and D suffered higher increases.

Figure 3.42 shows the inner and outer bolt forces for bolt-row A separately. From a load of 1,000 kNm, about the same bolt forces can be seen in the inner and outer bolts due to the adjacent HammerHead flange.

Figure 3.43 shows the shape of the deformed end-plate. This shape shows plate failure in the HammerHead part and group failure in bolt-rows C and D.

Similarly, the calculated failure mode was bolt group failure in Mode 2 within the flanges. But the design method uses a three-flange girder assumption for the HammerHead arrangements, and therefore it does not take into account any stiffness difference between the member and the HammerHead part. This was the reason why the deformation of the end-plate at the height of the tension flange was not predicted, as shown in Figure 3.44. And for the same reason, according to the calculations, there was group failure in the HammerHead part according to Mode 1, while the experiments showed no failure in bolt-row A.

The ultimate behaviour of the joint was plate failure in the HammerHead part (in bolt-row B) and mixed plate and bolt failure in bolt-rows C and D.

Figures 3.44 and 3.45 illustrate the behaviour observed in the experiments.
Test specimen TB11 ($t_{ep} = 20$ mm)

The observed bolt forces indicated in Figure 3.46 are different from those of the other test specimens with the same end-plate type, i.e. tests TB3 and TB7. The diagrams show roughly the same bolt-force increments for all tension bolt-rows.

The highest increment was measured for bolt-row C, and the lowest, for bolt-row A.

From about 1,200 kNm a force redistribution occurred among the bolt-rows. The force increase in rows B and C slowed down, while higher increases were observed in rows A and D, in a manner similar to test TB10.

Figure 3.47 shows clearly that the force re-distribution among the bolts in row A has a slight effect only because of the strong deformation of the plate, as shown in Figures 3.48 and 3.49.

The inner bolts achieved higher force levels than the outer ones, which shows that they were “better” stiffened.

The calculated failure mode was pure bolt failure (Mode 3) in the HammerHead part and group failure in Mode 2 within the flanges. Figure 3.48 shows the shape of the deformed end-plate and confirms the bolt failure (Mode 3) in the HammerHead part, but shows also pure bolt failure in rows C and D.

The ultimate behaviour of the joint was dominant bolt failure (Mode 3).

The plate deformation at the height of the tension flange is explained by the stiffness difference between the member and the HammerHead part, similarly to test specimens TB3 and TB7.

Figures 3.49 and 3.50 illustrate the failure of the tension zone and the broken bolts, respectively.
3.3.1.3 End-plate type III - Extended end-plate connection with 4 bolts in the extended part of the end-plate and 4 bolts in the first bolt-row below the tension flange

**Test specimen TB4 (t_{ep} = 12 mm)**

The diagrams in Figure 3.51 show slightly higher force levels in the bolt-row below the tension flange, in bolt-row C, than in the bolt-row in the extended part of the end-plate, bolt-row B. The diagrams do not exhibit a definite change which would indicate force redistribution between the bolt-rows. This fact indicates dominant plate failure.

The diagrams in Figure 3.52 show the forces of the inner and outer bolts in the case of bolt-rows B and C separately. In bolt row B the outer bolts (blue curve) achieved higher force levels than the inner bolts (red curve). This phenomenon can be explained by the end-plate deformations as shown in Figure 3.53. In the extended end-plate part the yield-lines achieved higher lengths for the outer bolts than for the inner ones. And due to the large end-plate deformations as shown in Figure 3.54, additional prying forces developed.

For bolt-row C an opposite relationship was observed.

The diagrams show roughly the same bolt-force increments for all tension bolts, which can be explained by the ductility of the end-plate.

The highest increment was recorded in those bolts that were stiffened to a large extent, i.e. the inner bolts of bolt row C.

Figure 3.53 illustrates the shape of the deformed end-plate. The measured data confirmed the results of the calculation. In the calculation “horizontal” bolt group failure was obtained in the extended bolt-row and group failure was predicted involving the inner rows C and D. (“Horizontal” bolt group failure means a group failure involving bolts in the same bolt-row, in this case in the extended part of the end-plate.) The ultimate behaviour of the joint was dominant plate failure (Mode 1).

Figure 3.54 illustrates the deformations of the tension zone of the end-plate.

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**Fig. 3.51 Moment vs. bolt-row force diagrams, test TB4.**

**Fig. 3.52 Moment vs. bolt-row force diagrams, test TB4.**

**Fig. 3.53 The shape of the deformed end-plate.**

**Fig. 3.54 Plate failure in the tension zone.**
Test specimen TB8 ($t_{ep} = 15$ mm)

The bolt-force diagrams show higher bolt forces in the first bolt-row below the tension flange (bolt-row C), than in the bolt-row in the extended part of the end-plate. At the same time, Figure 3.55 shows that from a load level of about 1,500 kNm the force started to increase considerably in bolt-row B. This is explained by the full development of the yield lines in the extended part of the end-plate. The upper end-plate edges closed and a prying force occurred, as shown in Figure 3.58.

The moment vs. bolt-row force diagrams in Figure 3.56 show the measured bolt forces in the inner and outer bolts separately. The highest force levels were recorded in the inner bolts of bolt-row C (green), followed by the outer bolts of the same bolt-row.

The diagrams show that the force increments were higher within bolt-row B in the outer bolts than in the inner bolts. The explanation is the same as in the case of test specimen TB4.

The trend curves exhibit changes at the same load level, at about 1,500 kNm.

Figure 3.57 shows the shape of the deformed end-plate.

In bolt-row B the calculated and measured bolt group failure via plate failure (Mode 1), and below the tension flange the calculated failure mode was bolt failure in Mode 3, results which were partially indicated by the measured deformations.

The ultimate behaviour of the joint was plate failure (Mode 1) in the extended part of the end-plate and bolt failure below the flanges.

Figure 3.58 illustrates the end-plate deformations in tension zone.
Test specimen TB12 ($t_{ep} = 20$ mm)

The diagrams in Figure 3.59 show a behaviour characteristic of stiff end-plates. The force distribution among bolt rows reflected the distances of the individual bolt rows from the centre of compression, i.e. an elastic force distribution was observed.

The bolt-row forces increased at a constant rate with the load. A limited amount of redistribution can be observed at 1,400 kNm.

As there were no end-plate deformations, the inner and outer bolts were loaded at the same level (row B, red and blue, and row C, green and light blue), as shown in Figure 3.60.

The observed failure mode confirmed the results of the calculation. Figure 3.61 shows horizontal bolt group failure in the first bolt-row. The calculation results predict the same failure in Mode 2 for this bolt-row.

The test results also confirmed the calculated failure mode below the tension flange. Figure 3.61 shows individual bolt failure in Mode 3.

The ultimate behaviour of the joint was mixed bolt and plate failure (Mode 2) in the extended part of the end-plate and bolt failure (Mode 3) below the tension flange.

Because of the height of the load cells applied, the bolts were not long enough and their threaded part sawed only 3 or 4 threads within the cells. This fact caused premature failure in the threads of the bolts.

This means that the real load bearing capacity of the joint could have been considerably higher if bolts of appropriate length were used.

Figures 3.62 and 3.63 illustrate the end-plate deformations and the broken bolts, respectively.
3.3.1.4 End-plate type IV - Extended end-plate connection with 2 bolts in the extended part of the end-plate

**Test specimen TB5** (t_{ep} = 12 mm)

The highest degree of initial force increase was observed in bolt-row C, as shown in Figure 3.64. From about 800 kNm a redistribution occurred among the bolt forces.

Figure 3.65 shows the shape of the deformed end-plate for test TB5. In the calculation horizontal group failure in the first bolt-row (Mode 1) and group failure involving bolt-rows C and D in Mode 2 was obtained. The measured deformations confirm the results of the calculations.

The ultimate behaviour of the joint was pure plate failure (Mode 1).

![Fig. 3.64 Moment vs. bolt-row force diagrams, test TB5.](image)

![Fig. 3.65 The deformed shape of the end-plate.](image)

**Test specimen TB9** (t_{ep} = 15 mm)

Similarly to the test results of specimen TB5, the highest initial force increase was measured in bolt-row C. The force redistribution also took place at about 800 kNm, as shown in Figure 3.67.

The diagram for bolt-row B shows linear behaviour until the failure of the joint.

From the calculations horizontal group failure in the extended bolt-row (Mode 2) and group failure involving bolt-rows C and D (Mode 2) was obtained. The measured deformations, shown in Figure 3.68, confirm the results of the calculation.

The ultimate behaviour of the joint was mixed plate and bolt failure (Mode 2).

![Fig. 3.67 Moment vs. bolt-row force diagrams, test TB9.](image)
Figures 3.69 and 3.70 show the deformations in the extended end-plate part.

Test specimen TB13 (t<sub>ep</sub> = 20 mm)

The diagrams indicate stiff end-plate behaviour. The higher the distance from the centre of compression the higher the rise of the diagrams, as shown in Figure 3.71.

Slight force redistribution can be observed at 800 kNm, which involved bolt rows C and D.

Figure 3.72 shows the shape of the deformed end-plate.

In all bolt-rows the calculated failure mode was mixed bolt and plate failure (Mode 2). The shape of the deformed end-plate, however, indicated pure bolt failure in Mode 3, while the deformations below the tension flange showed typical group failure.

The ultimate behaviour of the joint was bolt failure (Mode 3).

Figure 3.7 shows the test specimen after failure. Under the ultimate load all bolts of the joint were broken.
3.3.1.5 Measured end-plate deformations in the elastic phase

Purpose and scope
The purpose of measuring the end-plate deformations in the elastic phase was to analyze the load vs. deformation relationship and to find out how plate deformations develop up until the ultimate limit state.

The EC3 gives calculation methods to determine the end-plate deformations, i.e. the yield line pattern, in the ultimate limit state, but it does not inform us about the load-deformations equilibrium in the phases when the behaviour is still elastic. This study aims at identifying the "path" that leads from elastic to plastic deformations and finding the relationships among these deformations.

General
During the tests, in the elastic phase of joint behaviour, the end-plate deformations were measured by the portable inductive transducer shown in Figure 3.76. The locations of the measuring points are presented in Figure 3.75.

Measurements were made during the calculated elastic phase of the joint behaviour and the collected deformation values were evaluated after the tests.

The measuring points were placed so as to give representative points of the expected deformation but at the same time not to disturb the development of yield lines or the load capacity of the joint. Therefore, on the one hand, an appropriate number of measuring points needed to be defined so as to achieve an adequate accuracy of the deformation values determined, while on the other hand, one needed to be careful not to place too many such points so as not to disturb the yield line pattern to develop. For this reason half of the measuring points were placed in one end plate, and the other half in the other plate, following a pattern symmetrical to the web.

Figure 3.74 shows the fabricated pattern tool, which contained the holes marked in green only, and which was then applied in both end-plates making use of the symmetry of the bolt arrangement with respect to the web. For the exact positioning the tool to the end-plates two positioning bars were used, as shown in Figure 3.75.

Figure 3.77 shows the preparation of the measuring holes and Figure 3.78 presents an example of an end-plate as prepared for the test.

A portable inductive transducer (Figure 3.76) of the type Mitutoyo, a digimatic indicator with a measuring range of 0.01 mm to 25 mm and an accuracy of 0.01 mm was applied.

The deformation diagrams presented below include the summary of deformations from both end-plates. It means that in each case the displacement between the end-plates was measured by the transducer, and it was understood that this distance was equal to the sum of the deformations of both plates.
The presented diagrams show relative deformations. The initial deformations due to the welding process and the erection were measured and taken into consideration as the reference surface for the purposes of further measurements.

The end-plate deformations in the calculated elastic phase were measured in the case of end-plate thicknesses 12 mm and 16 mm only, because calculations predicted bolt failure as dominant failure mode for specimens with 20 mm thick end-plates.

The surfaces presented in the diagrams below have been obtained by interpolation between the measured points.

To generate the deformed shapes the following assumptions were made:
- the plate cannot deform along the line of the web and of the flange,
- the tests indicated that the HammerHead flanges cannot support the plate as the flange of the girder, therefore its supporting effect was neglected for the purposes of the evaluation.

### 3.3.1.5.1 End-plate type I - HammerHead connection with 2 bolts in both bolt-rows in the extended part of the end-plate

The connection type and the positions of the load cells can be seen in Figure 3.79. Figure 3.80 shows the measured area of the end-plate with the twenty measuring points.

Test specimens TB2 and TB6

Figures 3.81-3.86 show the end-plate deformations in the form of 3D diagrams at different load levels for test specimens TB2 and TB6, respectively.

Figures 3.83 and 3.86 illustrate the measured shape of the deformed tension zone after the test (1,490 kNm for test TB2 and 1,605 kNm for test TB6).
Fig. 3.82 3D deformation diagram of the tension zone by 900 kNm, specimen TB2.

Fig. 3.83 Shape after test, specimen TB2.

Fig. 3.84 3D deformation diagram of the tension zone, specimen TB6.

Fig. 3.85 3D deformation diagram of the tension zone by 960 kNm, specimen TB6.

Fig. 3.86 Shape after test, specimen TB6.
3.3.1.5.2 End-plate type II - HammerHead connection with 4 bolts in the first bolt-row and 2 bolts in the second bolt-row in the extended part of the end-plate

The connection type and the positions of the load cells can be seen in Figure 3.87. Figure 3.88 shows the measured area of the end-plate with the measuring points.

Test specimen TB3 and TB7

Figures 3.89-3.93 show the end-plate deformations in the form of 3D diagrams at different load levels for test specimens TB3 and TB7, respectively.

Figures 3.91 and 3.93 illustrate the measured shape of the deformed tension zone after the test (1,511 kNm for test TB3 and 1,635 kNm for test TB7).
3.3.1.5.3 End-plate type III - Extended end plate connection with 4 bolts in the extended part of the end-plate and 4 in the first bolt-row below the tension flange

The connection type and the positions of the load cells are presented in Figure 3.94. Figure 3.95 shows the measured area of the end-plate with the measuring points.

Test specimen TB4 and TB8

Figures 3.96-3.101 show the end-plate deformations in the form of 3D diagrams at different load levels for test specimens TB4 and TB8, respectively. Figures 3.98 and 3.101 illustrate the measured shape of the deformed tension zone after the test (1,440 kNm for test TB4 and 1,680 kNm for test TB8).
3.3.1.5.4 Summary of the results related to end-plate deformations

For end-plate type IV joints (extended end plate connections with two bolts in the extended part) deformations were not measured in the elastic phase of the joint behaviour.

From the results presented above it is concluded that in the case of end-plate types I, II and III, the 3D deformation diagrams show similar failure modes as those corresponding to the ultimate failure already at relatively low load levels that correspond to the elastic phase of joint behaviour. This means that the governing end-plate deformations can already be identified in the phase of elastic behaviour. This observation, together with the measuring method developed could be useful for forecasting failure modes.
3.3.2 Evaluation of the test results

Table 3.3 shows a summary of the test results in comparison with the calculated moment resistance values as well as these latter values recalculated with the actual material properties.

The presented results are as follows:
- the first and second columns contain the type of the end-plate and the reference of the test specimen;
- the third column shows the achieved maximum load level and, in brackets, the moment level;
- the fourth column presents the relationship between the achieved moment level and the design bending resistance of the beam calculated according to the EC3. (The calculated bending resistance of the member was for each test specimen $M_{c,Rd} = 2,505 \text{ kNm}$, because the geometry and the material properties were the same for all beams. Additional stiffeners and the HammerHead arrangement were not considered in the resistance calculation, as explained in chapter 3.2.1. The calculation is described in detail in Appendix G.)
- the fifth column contains the reference of the bolt-row which failed first in the test;
- the sixth column presents the achieved bolt force ($F_i^{\text{max}}$) and the highest bolt force increment ($\Delta F_i^{\text{max}}$), i.e. measured bolt force without preload;
- the seventh and eighth columns indicate the calculated design moment resistance ($M_{j,Rd}$), the re-calculated design moment resistance ($M_{j,Rd}^{*}$), and, in brackets, the corresponding utilisation ratio.

Table 3.3 Summary of test results.

<table>
<thead>
<tr>
<th>end-plate type</th>
<th>specimen</th>
<th>achieved load level [$\text{kN}$]</th>
<th>$M_{Ed} / M_{c,Rd}$ [%]</th>
<th>bolt-row that failed first</th>
<th>measured maximum bolt force</th>
<th>calculated design moment resistance</th>
<th>re-calculated design moment resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$F_i^{\text{max}}$ [kN]</td>
<td>$\Delta F_i^{\text{max}}$ [kN]</td>
<td>$M_{j,Rd}$ [kNm] (%)</td>
</tr>
<tr>
<td>I</td>
<td>TB2</td>
<td>503 (1,509)</td>
<td>60.2</td>
<td>bolt-row C</td>
<td>263</td>
<td>253</td>
<td>929 (61.6)</td>
</tr>
<tr>
<td></td>
<td>TB6</td>
<td>545 (1,635)</td>
<td>65.3</td>
<td>bolt-row C</td>
<td>241</td>
<td>236</td>
<td>1,044 (63.9)</td>
</tr>
<tr>
<td></td>
<td>TB10</td>
<td>536 (1,608)</td>
<td>64.2</td>
<td>bolt-row C</td>
<td>239</td>
<td>214</td>
<td>1,082 (66.0)</td>
</tr>
<tr>
<td></td>
<td>TB3</td>
<td>504 (1,512)</td>
<td>60.4</td>
<td>bolt-row C</td>
<td>242</td>
<td>223</td>
<td>1,121 (-)</td>
</tr>
<tr>
<td></td>
<td>TB7</td>
<td>550 (1,650)</td>
<td>65.9</td>
<td>bolt-row C</td>
<td>256</td>
<td>233</td>
<td>1,453 (-)</td>
</tr>
<tr>
<td></td>
<td>TB11</td>
<td>567 (1,701)</td>
<td>67.9</td>
<td>bolt-row C</td>
<td>227</td>
<td>208</td>
<td>1,469 (-)</td>
</tr>
<tr>
<td>II</td>
<td>TB4</td>
<td>495 (1,485)</td>
<td>59.3</td>
<td>bolt-row C</td>
<td>244</td>
<td>226</td>
<td>1,022 (68.8)</td>
</tr>
<tr>
<td></td>
<td>TB8</td>
<td>623 (1,869)</td>
<td>74.6</td>
<td>bolt-row C</td>
<td>230</td>
<td>226</td>
<td>1,225 (65.5)</td>
</tr>
<tr>
<td></td>
<td>TB12</td>
<td>538 (1,614)*</td>
<td>64.4</td>
<td>bolt-row C</td>
<td>235</td>
<td>230</td>
<td>1,341 (83.1)</td>
</tr>
<tr>
<td>III</td>
<td>TB5</td>
<td>368 (1,104)</td>
<td>44.1</td>
<td>bolt-row C</td>
<td>239</td>
<td>226</td>
<td>628 (56.9)</td>
</tr>
<tr>
<td></td>
<td>TB9</td>
<td>365 (1,095)</td>
<td>43.7</td>
<td>bolt-row C</td>
<td>234</td>
<td>226</td>
<td>745 (68.0)</td>
</tr>
<tr>
<td></td>
<td>TB13</td>
<td>407 (1,221)</td>
<td>48.7</td>
<td>bolt-row C</td>
<td>244</td>
<td>239</td>
<td>810 (66.3)</td>
</tr>
</tbody>
</table>

* Premature failure in the threads of the bolts.

Table 3.3 shows for all end-plate types that the re-calculated design moment resistance (re-calculation means that both the end-plates and the bolts were taken into account in the calculation with their measured material properties) is always higher than that calculated with standard material properties.

For end-plate type II no utilisation ratio was calculated, because the test result showed that the model assumptions applied were not true. This end-plate type is evaluated in detail in chapter 3.3.3.4.

In all tests the beams (the flanges and the web plate) and the widths of the end-plates were identical and for each end-plate type three different end-plate thicknesses ($t_{ep} = 12; 15; 20 \text{ mm}$) were tested. The differences were in the number and position of the bolts and the arrangement of the end-plate extension.

On the basis of theoretical considerations it was expected that the specimens with type II end-plates, HammerHead and four bolts in bolt-row A, should have the highest moment resistance, which was only partially demonstrated by the test results. For example the test specimen TB8 (type III) had higher re-
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resistance, with identical end-plate thickens, than specimen TB7 (type II). However, Table 3.3 shows that the results for end-plate type I were almost the same. Nevertheless, from the point of view of economics, it is not always end-plate type II that gives the most efficient solution. As it is easier to fabricate and install and has a higher resistance, end-plate type III can be a competitive solution.

Note that the design moment resistance could be increased by adding a supplementary web plate which stiffens the extended part of the end-plate as shown by the results presented in chapter 3.41.

The test results showed that the calculated resistances in all joint types were lower than the measured capacities. In the case of the tested joints it was observed that the measured design resistances were in average 33% higher than the load bearing capacities calculated with standard material properties. This deviation on the safe side is internationally known. Analogous results were presented in chapter 2.2.

It can be stated that the calculated resistances were in the elastic phase of joint behaviour. This way it was confirmed that the EC3 based design method developed ensures a safe joint design.

Figures 3.103 to 3.106 show the moment-deflection diagrams for the tested end-plate types. The diagrams show the change of the joint deformations according to the end-plate thickness.

Figures 3.103 a.) to 3.106 a.) present the deflections measured under the hydraulic jack, as shown in Figure 3.11, and the measured moment relationships. The diagrams meet the requirements and shows that the end-plate thickness is inversely proportional to the deflection under the load.

The same tendency can be observed in Figures 3.103 b.) to 3.106 b.) as regards relative deflections measured in the cross-section of the tested joint, and the measured moment relationships.

Because of the nature of the test the relative deflections include the deformations of the beam as a girder. In the presented diagrams, however, these deformations were calculated and the results were adjusted accordingly, as explained in Figure 3.102. This means that the diagrams present those deformations only that are due to the “opening” of the joints.

Nevertheless it is concluded that the deflection curves in all tests showed linear behaviour up to the calculated design moment, i.e. elastic joint behaviour was measured.
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Beam-to-beam joints

L. Katula

Budapest University of Technology and Economics Department of Structural Engineering

Fig. 3.103 End-plate type I, tests: TB2, TB6, TB10.

Fig. 3.104 End-plate type II, tests: TB3, TB7, TB11.

Fig. 3.105 End-plate type III, tests: TB4, TB8, TB12.

Fig. 3.106 End-plate type IV, tests: TB5, TB9, TB13.
3.3.3 Verification of the design methods developed

3.3.3.1 End-plate type IV - Extended end-plate connection with two bolts in the extended part

Figure 3.107 shows the bolt arrangement and the positions of the load cells of the end-plate type IV. This type of joint can be considered as reference joint because the moment resistance can be calculated according to EN 1993-1-8 without modification.

Figures 3.108 to 3.110 present the distribution of the tension bolt-row forces under the load. The short dashed lines mark the re-calculated and measured resistances of the joint. The bolt-row forces corresponding to these load levels are summarized in Table 3.4 in columns three and four.

Table 3.4 contains the re-calculated and measured bolt-row forces. A calculation example, which presents the bolt-row forces, is in Appendix H to find. In the first column the bolt-rows of the tension zone were listed, the second column contains the re-calculated bolt-row forces, in third at the re-calculated resistance level measured bolt-row forces were collected and in the fourth the bolt-row forces measured in load bearing capacity.

Table 3.4 shows that the re-calculated results in the first and second bolt-row were in all cases lower, except TB9 bolt-row B, than the measured values, whereas in third bolt-row the design method overestimates. Note, the bolt-forces were compared on the same moment level and the sum of the forces in tension zone i.e. bolt forces in the rows B, C and D, were the same if compared the measured and re-calculated values. The “measured/re-calculated” ratios were the following: 0.98 by TB6, 1.03 by TB8 and 1.00 by TB13. It follows that the model uses a conservative approach i.e. the force distribution, between the bolt-rows, occur in a conservative way. Because of the conservative force distribution the calculation of the moment resistance is conservative too.

The design method underestimates the moment resistance of the end-plate type IV joints in the average of 21%.
Table 3.4 Comparison of the re-calculated and measured bolt forces.

<table>
<thead>
<tr>
<th></th>
<th>test TB5 ([t_{ep} = 12\text{ mm}])</th>
<th>test TB9 ([t_{ep} = 15\text{ mm}])</th>
<th>test TB13 ([t_{ep} = 20\text{ mm}])</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>re-calculated bolt-row forces</td>
<td>measured bolt-row forces</td>
<td>measured bolt-row forces</td>
</tr>
<tr>
<td></td>
<td>(M_{j,Rd}^* = 752\text{ kNm})</td>
<td>for (M = 752\text{ kNm})</td>
<td>for (M^* = 1,060\text{ kNm})</td>
</tr>
<tr>
<td></td>
<td>bolt-row B</td>
<td>210.5 kN</td>
<td>301.5 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>238.0 kN</td>
<td>262.8 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>309.5 kN</td>
<td>310.6 kN</td>
</tr>
<tr>
<td></td>
<td>bolt-row C</td>
<td>305.5 kN</td>
<td>330.2 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>394.3 kN</td>
<td>439.9 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>451.7 kN</td>
<td>461.7 kN</td>
</tr>
<tr>
<td></td>
<td>bolt-row D</td>
<td>326.4 kN</td>
<td>360.3 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>192.3 kN</td>
<td>320.2 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>226.3 kN</td>
<td>409.1 kN</td>
</tr>
<tr>
<td></td>
<td>(\Sigma) bolt-row forces</td>
<td>842 kN</td>
<td>992 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>823 kN</td>
<td>1,023 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>988 kN</td>
<td>1,181 kN</td>
</tr>
<tr>
<td></td>
<td>re-calculated bolt-row forces</td>
<td>measured bolt-row forces</td>
<td>measured bolt-row forces</td>
</tr>
<tr>
<td></td>
<td>(M_{j,Rd}^* = 888\text{ kNm})</td>
<td>for (M = 888\text{ kNm})</td>
<td>for (M^* = 1,040\text{ kNm})</td>
</tr>
<tr>
<td></td>
<td>bolt-row B</td>
<td>301.5 kN</td>
<td>350.2 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>262.8 kN</td>
<td>362.4 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>310.6 kN</td>
<td>458.4 kN</td>
</tr>
<tr>
<td></td>
<td>bolt-row C</td>
<td>330.2 kN</td>
<td>381.5 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>439.9 kN</td>
<td>444.4 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>461.7 kN</td>
<td>471.0 kN</td>
</tr>
<tr>
<td></td>
<td>bolt-row D</td>
<td>360.3 kN</td>
<td>430.7 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>320.2 kN</td>
<td>354.2 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>409.1 kN</td>
<td>437.4 kN</td>
</tr>
<tr>
<td></td>
<td>(\Sigma) bolt-row forces</td>
<td>992 kN</td>
<td>1,162 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1,023 kN</td>
<td>1,181 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1,181 kN</td>
<td>1,367 kN</td>
</tr>
</tbody>
</table>

3.3.3.2 End-plate type III - Extended end-plate connection with 4 bolts in the extended part of the end-plate and 4 bolts in the first bolt-row below the tension flange

Figure 3.111 shows the bolt arrangement and the positions of the load cells for end-plate type III.

The bolt arrangement in rows B and C ensured the symmetry of the T-stubs because in both sides of the tension flange the bolts were placed symmetrically. This symmetrical bolt arrangement means in the design method that \(\beta = 1.0\). Different bolt forces in inner and outer bolts could occur due to the different \(l_{ef}\) lengths only.

Figures 3.112 to 3.114 present the diagrams of the bolt-row forces. In the diagrams the inner and outer bolt force levels for the rows B and C are shown separately.
Table 3.5 Comparison of the re-calculated and measured bolt forces.

<table>
<thead>
<tr>
<th>Test</th>
<th>M_{j,Rd} [kNm]</th>
<th>Re-calculated bolt-row forces</th>
<th>Measured bolt-row forces for M = 1,137 kNm</th>
<th>Measured bolt-row forces for M* = 1,440 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>inner bolts</td>
<td>outer bolts</td>
<td>inner bolts</td>
</tr>
<tr>
<td>TB4</td>
<td>1,137 kNm</td>
<td>125.6 kN</td>
<td>166.2 kN</td>
<td>244.2 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>327.8 kN</td>
<td>348.2 kN</td>
<td>380.3 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>342.3 kN</td>
<td>-</td>
<td>271.2 kN</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1,310 kN</td>
<td>1,483 kN</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test</th>
<th>M_{j,Rd} [kNm]</th>
<th>Re-calculated bolt-row forces</th>
<th>Measured bolt-row forces for M = 1,462 kNm</th>
<th>Measured bolt-row forces for M* = 1,680 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>inner bolts</td>
<td>outer bolts</td>
<td>inner bolts</td>
</tr>
<tr>
<td>TB8</td>
<td>1,462 kNm</td>
<td>180.8 kN</td>
<td>239.2 kN</td>
<td>203.4 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>448.8 kN</td>
<td>(Mode 3)</td>
<td>394.8 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>406.0 kN</td>
<td>(448.8 kN)</td>
<td>324.7 kN</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1,724 kN</td>
<td>1,553 kN</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test</th>
<th>M_{j,Rd} [kNm]</th>
<th>Re-calculated bolt-row forces</th>
<th>Measured bolt-row forces for M = 1,560 kNm</th>
<th>Measured bolt-row forces for M* = 1,560 kNm</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>inner bolts</td>
<td>outer bolts</td>
<td>inner bolts</td>
</tr>
<tr>
<td>TB12</td>
<td>1,560 kNm</td>
<td>253.0 kN</td>
<td>(267.4 kN)</td>
<td>292.0 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>424.6 kN</td>
<td>(448.8 kN)</td>
<td>423.1 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>384.1 kN</td>
<td>(406.0 kN)</td>
<td>383.9 kN</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1,821 kN</td>
<td>1,838 kN</td>
<td></td>
</tr>
</tbody>
</table>
Table 3.5 lists the re-calculated bolt-row forces compared to the measured force levels. In the case of specimen TB12 the re-calculated design moment ($M_{j,Rd} = 1,649 \text{kNm}$) was higher than the measured value because of the premature failure of the bolts. For this reason “reduced” re-calculated bolt forces were used in the evaluation. The factor of reduction was 0.946, which represents the ratio between the re-calculated and measured moment levels ($1,560 / 1,649 = 0.946$).

The experimental observations showed that the stiffer (thicker) the end-plate, the lower the difference between the inner and outer bolt forces. It is a result of the small end-plate deformations, which indicates dominant bolt failure (Mode 3).

The measured and re-calculated results show the same tendency as in the case of end-plate type IV. Generally speaking, the re-calculated bolt forces were in all cases underestimated in the first bolt-row, whereas in the second and third rows the design method overestimated the actual values. As to inner and outer bolts, the calculated distribution shows the measured tendency, i.e. for all tests, the outer bolts achieved higher force levels, which was also reflected by the calculation.

The sum of the tension forces (bolt-row forces $B + C + D$) show small deviations between the re-calculated and measured forces. The ratios were the following (“measured/re-calculated”): 1.13 for TB4, 0.90 for TB8 and 1.01 for TB12. Despite this fact we can state that the model assumes a conservative force distribution among the bolt-rows.

The average deviation of the design method in terms of moment resistance was 14%.

### 3.3.3.3 End-plate type I - HammerHead connection with 2 bolts in both bolt-rows in the extended part of the end-plate

Figure 3.115 illustrates the bolt arrangement of the end-plate type I and the positions of the load cells. Figures 3.116 to 3.118 show the distribution of the tension bolt-row forces. The short dashed lines mark the re-calculated and measured resistances of the joint.

In Table 3.6 in the first column the bolt-rows of the tension zone are listed, the second column contains the re-calculated bolt-row forces, the third, the bolt-row forces measured at the level of the re-calculated resistance, and the fourth, the bolt-row forces measured at the level of the load bearing capacity.

---

**Fig. 3.115 Positions of the load cells.**

**Fig 3.116 Bolt forces in bolt-rows A, B, C and D of the test TB2.**

**Fig 3.117 Bolt forces in bolt-rows A, B, C and D of the test TB6.**

**Fig 3.118 Bolt forces in bolt-rows A, B, C and D of the test TB10.**
Table 3.6 Comparison of the re-calculated and measured bolt forces.

<table>
<thead>
<tr>
<th>Test and End-Plate Thickness</th>
<th>Re-calculated Bolt-Row Forces</th>
<th>Measured Bolt-Row Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( M^*_{j,Rd} = 1,108 \text{ kNm} )</td>
<td>( M^*_{j,Rd} = 1,490 \text{ kNm} )</td>
</tr>
<tr>
<td><strong>Bolt-Row A</strong></td>
<td>313.4 kN</td>
<td>320.6 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row B</strong></td>
<td>252.6 kN</td>
<td>370.6 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row C</strong></td>
<td>305.5 kN</td>
<td>402.2 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row D</strong></td>
<td>326.4 kN</td>
<td>247.9 kN</td>
</tr>
<tr>
<td><strong>( \Sigma ) Bolt-Row Force</strong></td>
<td>1,198 kN</td>
<td>1,341 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test and End-Plate Thickness</th>
<th>Re-calculated Bolt-Row Forces</th>
<th>Measured Bolt-Row Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( M^*_{j,Rd} = 1,248 \text{ kNm} )</td>
<td>( M^*_{j,Rd} = 1,630 \text{ kNm} )</td>
</tr>
<tr>
<td><strong>Bolt-Row A</strong></td>
<td>330.2 kN</td>
<td>282.5 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row B</strong></td>
<td>330.2 kN</td>
<td>390.7 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row C</strong></td>
<td>330.2 kN</td>
<td>398.6 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row D</strong></td>
<td>360.3 kN</td>
<td>283.3 kN</td>
</tr>
<tr>
<td><strong>( \Sigma ) Bolt-Row Force</strong></td>
<td>1,351 kN</td>
<td>1,355 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test and End-Plate Thickness</th>
<th>Re-calculated Bolt-Row Forces</th>
<th>Measured Bolt-Row Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( M^*_{j,Rd} = 1,397 \text{ kNm} )</td>
<td>( M^*_{j,Rd} = 1,590 \text{ kNm} )</td>
</tr>
<tr>
<td><strong>Bolt-Row A</strong></td>
<td>381.5 kN</td>
<td>385.4 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row B</strong></td>
<td>381.5 kN</td>
<td>413.2 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row C</strong></td>
<td>381.5 kN</td>
<td>406.9 kN</td>
</tr>
<tr>
<td><strong>Bolt-Row D</strong></td>
<td>430.7 kN</td>
<td>365.0 kN</td>
</tr>
<tr>
<td><strong>( \Sigma ) Bolt-Row Force</strong></td>
<td>1,575 kN</td>
<td>1,571 kN</td>
</tr>
</tbody>
</table>

The results in Table 3.6 show the same tendency as was found for end-plate type III and IV, whereas the effect of the non-homogeneous support condition is also present to a certain degree. So far the observation showed that the design method underestimates the bolt forces in the first bolt-row. In the results of tests TB2 and TB6 the effect of the end-plate deformation in bolt-row A is also manifested. The deformations are shown in Figures 3.27 and 3.31. This deformation caused in the case of test TB2 that the measured and re-calculated results were about the same, and in the case of test TB6, that the re-calculated force was higher than the measured one. At the same time, in the case of test TB10, no deformations at the height of the HammerHead flange was measured, as shown in Figure 3.35, and therefore the re-calculated force was, even though only slightly, underestimated. The re-calculated bolt forces in the second and third bolt-rows (B and C) were underestimated, whereas in the fourth row (D) the design method overestimates the corresponding values in all cases. The ratio between the sum of the measured and the re-calculated tension forces (bolt-row forces A + B + C + D) shows small deviations. The ratios were the following: 1.12 for TB2, 1.00 for TB6 and 1.00 for TB10.
3.3.3.4 End-plate type II - HammerHead connection with 4 bolts in the first bolt-row and 2 bolts in the second bolt-row in the extended part of the end-plate

Figure 3.119 presents the bolt arrangement and the positions of the load cells of end-plate type II. Figures 3.120 to 3.122 show the diagrams of the bolt-row forces under the load. Table 3.7 shows the re-calculated and measured bolt-row forces in the same manner as Table 3.4.

Table 3.7 shows significant deviations on the unsafe side between the re-calculated and the measured bolt forces in the first bolt-row. The reason was the non-homogeneous support condition of the end-plate. The calculation model assumed that the supports, i.e. web and flanges, had the same stiffness. However, the test showed that for the extended HammerHead part of the end-plate, the short HammerHead web and flange cannot ensure the same degree of stiffness as an “infinitely” long flange and web, as it can be seen looking at the deformed shape of the end-plates shown in Figures 3.40, 3.44 and 3.49. This high deformation of the inadequately stiffened HammerHead part can be called “the HammerHead-effect”.

To handle this phenomenon a plausible solution could be if one reduces the effective design tension resistance of the bolts in the HammerHead part. This reduction considers the following aspects: geometry of the HammerHead part (web height, length and thickness, flange breadth and thickness), bolt diameter, material property (plate and bolt), and, last but not least, number and position of the bolts in the HammerHead part. All these parameters have influence on the bolt forces.

The low number of available test results (TB2, TB6, TB3, TB7, TB11) was not sufficient to verify a new modifying factor which could consider this HammerHead-effect. The identification of the main influential parameters and the elaboration of the modifying factor would require further tests and parametric studies.

The results in Table 3.7 show for specimens TB7 and TB11 that the re-calculated design moment was higher than the measured values because of model differences. From this reason, in Table 3.7,
“reduced” re-calculated bolt forces were assumed. The factor of reduction is 0.976 for test TB7 and 0.899 for test TB11. This reduction factor represents the ratio between the re-calculated and measured moment levels.

Table 3.7 Comparison of the re-calculated and measured bolt forces.

<table>
<thead>
<tr>
<th></th>
<th>test TB3 ([t_{ep} = 12 \text{ mm}})</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>re-calculated bolt-row forces</td>
<td>measured bolt-row forces</td>
<td>measured bolt-row forces</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(M_{j,Rd}^* = 1,298 \text{ kNm})</td>
<td>for (M = 1,298 \text{ kNm})</td>
<td>for (M^* = 1,435 \text{ kNm})</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>inner bolts</td>
<td>outer bolts</td>
<td>inner bolts</td>
<td>outer bolts</td>
<td>inner bolts</td>
</tr>
<tr>
<td>bolt-row A</td>
<td>328.0 kN</td>
<td>174.0 kN</td>
<td>192.8 kN</td>
<td>189.5 kN</td>
<td>204.2 kN</td>
</tr>
<tr>
<td>bolt-row B</td>
<td>252.6 kN</td>
<td>-</td>
<td>358.9 kN</td>
<td>-</td>
<td>404.1 kN</td>
</tr>
<tr>
<td>bolt-row C</td>
<td>305.5 kN</td>
<td>-</td>
<td>417.8 kN</td>
<td>-</td>
<td>446.2 kN</td>
</tr>
<tr>
<td>bolt-row D</td>
<td>326.4 kN</td>
<td>-</td>
<td>284.4 kN</td>
<td>-</td>
<td>331.7 kN</td>
</tr>
<tr>
<td>(\Sigma) bolt-row forces</td>
<td>1,387 kN</td>
<td>1,443 kN</td>
<td>1,596 kN</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

|                  | test TB7 \([t_{ep} = 15 \text{ mm}}\) |                  |                  |                  |                  |
|                  | re-calculated bolt-row forces        | measured bolt-row forces | measured bolt-row forces |
|                  | \(M_{j,Rd}^* = 1,675 \times 0.976 \text{ kNm}\) | for \(M = 1,635 \text{ kNm}\) | for \(M^* = 1,635 \text{ kNm}\) |
|                  | inner bolts | outer bolts | inner bolts | outer bolts | inner bolts | outer bolts |
| bolt-row A       | 438.0 kN (448.8 kN) | 267.0 kN (273.6 kN) | 243.1 kN | 233.2 kN | 243.1 kN | 233.2 kN |
| bolt-row B       | 354.8 kN (363.5 kN) | -          | 487.6 kN | -          | 487.6 kN | -          |
| bolt-row C       | 322.3 kN (330.2 kN) | -          | 458.1 kN | -          | 458.1 kN | -          |
| bolt-row D       | 351.7 kN (360.3 kN) | -          | 451.8 kN | -          | 451.8 kN | -          |
| \(\Sigma\) bolt-row forces | 1,734 kN | 1,874 kN | 1,874 kN |

|                  | test TB11 \([t_{ep} = 20 \text{ mm}}\) |                  |                  |                  |                  |
|                  | re-calculated bolt-row forces        | measured bolt-row forces | measured bolt-row forces |
|                  | \(M_{j,Rd}^* = 1,869 \times 0.899 \text{ kNm}\) | for \(M = 1,680 \text{ kNm}\) | for \(M^* = 1,680 \text{ kNm}\) |
|                  | inner bolts | outer bolts | inner bolts | outer bolts | inner bolts | outer bolts |
| bolt-row A       | 403.5 kN (448.8 kN) | 403.5 kN (448.8 kN) | 295.8 kN | 259.2 kN | 295.8 kN | 259.2 kN |
| bolt-row B       | 371.6 kN (413.4 kN) | -          | 450.0 kN | -          | 450.0 kN | -          |
| bolt-row C       | 335.7 kN (373.4 kN) | -          | 430.5 kN | -          | 430.5 kN | -          |
| bolt-row D       | 303.7 kN (337.8 kN) | -          | 478.7 kN | -          | 478.7 kN | -          |
| \(\Sigma\) bolt-row forces | 1,818 kN | 1,914 kN | 1,914 kN |

The results in Table 3.7 show that in the first bolt-row the force levels were the same in the outer and inner bolts. This phenomenon can be explained by the plate deformations as shown in Figures 3.40, 3.44 and 3.49.

The only difference between end-plate types I and II was the number of bolts in the first bolt-row (bolt-row A), two for type I and four for type II, as shown in Figures 3.115 and 3.119. Table 3.8 compares the re-calculated and measured bolt-row forces and shows the forces at the same chosen moment level.
The results in Table 3.8 corresponding to an end-plate thickness of 12 mm indicate an approximately 8% lower force level in the first bolt-row for end-plate type I, contrarily to the other bolt-rows, where higher forces were measured. The difference reflects the lever arm with respect to the compression point, i.e. the highest difference was measured in bolt-row B and the lowest in bolt-row D. In test TB2 a slight HammerHead-effect was registered, which explains this behaviour.

The differences in the bolt-row forces between TB6 and TB7 were the following: 81% for bolt-row A, 92% for bolt row B, and approximately the same forces were measured in rows C and D.

For end-plate thickness of 20 mm the results indicate slightly lower force levels in the first bolt-row for test TB10, contrarily to the other bolt-rows, where approximately the same forces were measured. Note that no HammerHead-effect was detected for test TB10.

The presented results show the complexity of the HammerHead-effect. To determine the plate deformations in the HammerHead part and due to the re-distribution of the bolt forces further studies are needed. This could give the relationship of the forces in the HammerHead part.
In view of the end-plate deformations in the HammerHead part it is worth analyzing the bending moment at the height of the tension flange of the member. Table 3.9 summarizes the moments and the deformations in the HammerHead part.

Table 3.9 Bending moments and deformations in the HammerHead part.

<table>
<thead>
<tr>
<th>end-plate thickness [mm]</th>
<th>test</th>
<th>bending moment [kNm]</th>
<th>(comment)</th>
<th>deformed tension zone</th>
<th>test</th>
<th>bending moment [kNm]</th>
<th>(comment)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>TB2</td>
<td>63.3</td>
<td></td>
<td></td>
<td>TB3</td>
<td>71.3</td>
<td>(high deformations)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(slight deformations)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>TB6</td>
<td>66.1</td>
<td></td>
<td></td>
<td>TB7</td>
<td>76.7</td>
<td>(high deformations)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(slight deformations)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>TB10</td>
<td>71.3</td>
<td></td>
<td></td>
<td>TB11</td>
<td>84.6</td>
<td>(high deformations)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(no deformations)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In the case of the tested HammerHead arrangements, in each end-plate types (I and II), the only parameter was the thickness of the end-plate. Table 3.9 summarizes the results in terms of end-plate deformations and the bending moment acting at the height of the tension flange.

The results show that in the case of the end-plate type II joints (TB3, TB7 and TB11) the higher bending moment in the HammerHead part caused high deformations. The HammerHead arrangement was not “stiff” enough, as a flange would have been, to support the edge of the end-plate.

On the basis of the studied arrangements it can be stated that for an end-plate thickness $t_{eo} = 20$ mm and end-plate type I the given HammerHead arrangement provided an appropriate support to the end-plate.

Finally it can be concluded from the results of series I that the developed method predicts the failure mode of the joint with high accuracy.

The sums of the calculated tension forces show in average less than 2% deviation from the measured forces. (This sum does not include the results obtained for end-plate type II.) The method overestimated the bolt forces in the third or fourth bolt row (depending on whether there were three or four bolt rows in the tension zone), and underestimated the bolt forces in the first, second, and, as appropriate, third bolt rows. This force distribution ensures a conservative model approach for the moment resistance of the joint.

With respect to the results of end-plate type II, further tests are needed to refine the model.

The average underestimation of the method (with end-plate type II excluded) was 20%. This deviation comes from the simplifications and limitations of the method and are a price for its easy manageability.
3.4 Test series II

3.4.1 Test results

The measured data are presented in moment vs. bolt-row force diagrams, moment-deflection diagrams, as well as figures of the deformed end-plate contour-lines and surfaces, as described in more detail in chapter 3.3.1.

Table 3.10 presents a summary of the test specimens and the load cell positions for test series II. Note the similarities between specimens TA (TB) and TD. Specimen TD is a specimen TA with an extended end-plate and an additional bolt-row A. Specimen TC can be obtained from specimen TD, and the bolt arrangement from specimen TC shows similarities with specimen TE (TF).

<table>
<thead>
<tr>
<th>end-plate arrangements with positions of load cells</th>
</tr>
</thead>
<tbody>
<tr>
<td>test specimen</td>
</tr>
<tr>
<td>end-plate thickness ( t_{ep} ) [mm]</td>
</tr>
</tbody>
</table>

Test specimen TA \( (t_{ep} = 16 \text{ mm}) \)

The moment vs. bolt-row force diagrams in Figure 3.123 show that the highest force increment occurred in bolt-row B, followed by bolt-row C.

The diagrams do not exhibit a definite change in their shape. This indicates plate failure.

Figure 3.124 illustrates the deformed end-plate and shows that the joint failure occurred in bolt-rows B and C in the form of bolt group failure. This experimental observation confirmed the results of the calculation, although the calculations predicted mixed bolt and plate failure (Mode 2).

The ultimate behaviour of the joint was plate failure (Mode 1).

Figure 3.125 shows the test specimen after the test.
Test specimen TB ($t_{ep} = 20$ mm)

The bolt-row force distribution as indicated in Figure 3.126 was similar to that of test TA.

At a load level of approximately 700 kNm, the diagrams show a change in their shape. After this change, a redistribution of the bolt forces was observed.

Figure 3.127 shows the end-plate deformations and indicates the bolt group failure involving rows B and C. This observation confirmed the results of the calculation, which predicted group failure in Mode 2.

The ultimate behaviour of the joint was mixed bolt-and-plate failure (Mode 2).

Figure 3.128 illustrates the test specimen after the test.
Test specimen TC ($t_{ep} = 20$ mm)

In Figure 3.129 the three measured bolt-row forces are plotted against the value of the applied bending moment. The observed behaviour in terms of bolt forces shows approximately the same force level in bolt-rows A and B.

The diagrams are linear without any changes in their shape or any sign of yielding.

Figure 3.130 shows the end-plate deformations after the test, and indicates bolt failure in bolt-rows A and B.

This experimental observation confirmed the results of the model calculation.

The ultimate behaviour of the joint was bolt failure (Mode 3).

Figure 3.131 illustrates the test specimen after the test.

Test specimen TD ($t_{ep} = 16$ mm)

The moment vs. bolt-row force diagrams in Figure 3.132 show slight redistribution of bolt forces at about 1,000 kNm load level involving bolt-row forces A and C.

The measured end-plate deformations in Figure 3.133 indicate individual bolt failure in bolt-row A and group bolt failure in bolt-rows B and C. This observed deformation confirmed the results of the calculation, Mode 2 failure in bolt-row A and Mode 2 failure involving bolt-rows B and C.

The ultimate behaviour of the joint was mixed bolt-and-plate failure (Mode 2).

Figure 3.134 illustrates the failure mode of the specimen.
Test specimen TE (\(t_{ep} = 20\) mm)

The bolt-row force diagrams presented in Figure 3.135 show changes in their shapes at about 1,400 kNm load level. After this load level redistribution occurred.

The diagrams show a higher rate of increase in the inner bolts than in the outer bolts.

In Figure 3.136 “horizontal” bolt group failure in bolt-row A and bolt failure in bolt-row B can be seen. “Horizontal” bolt group failure refers to a group failure involving bolts in the same bolt-row. These deformations confirmed the calculation results. In bolt-row A the calculated failure was horizontal bolt group failure in Mode 2 and in bolt-row B, bolt failure (Mode 3).

The ultimate behaviour of the joint was mixed bolt-and-plate failure (Mode 2).

Figure 3.137 shows the specimen after failure.
Test specimen TF ($t_{ep} = 24$ mm)

The moment vs. bolt-row force diagrams in Figure 3.138 show similarity with the results of test TE. The changes in the shape of the curves occurred at about 1,500 kNm load level.

The diagrams present a higher rate of increase of the bolt force in the inner bolts than in the outer bolts.

In Figure 3.139 a failure mode similar to that of specimen TE can be seen: “horizontal” group failure in bolt-row A and bolt failure in bolt-row B.

These deformations confirmed the calculation results.

The ultimate behaviour of the joint was mixed bolt-and-plate failure (Mode 2).

Figure 3.140 shows the specimen after failure.
3.4.2 Evaluation of the test results

Table 3.11 shows a summary of the test results in comparison with the calculated moment resistance values as well as these latter values recalculated with the actual material properties. The presented results are as follows:

- the first column contains the reference of the test specimen;
- the second column shows the achieved maximum load level and, in brackets, the moment level;
- the third column presents the relationship between the achieved moment level and the calculated design bending resistance of the beam. (The calculated bending resistance is $M_{c,Rd} = 2,139 \text{ kNm}$ for test specimens TA, TB, TC and TD, and $M_{c,Rd} = 2,871 \text{ kNm}$ for specimens TE and TF.)
- the fourth column contains the reference of the bolt-row which failed first during the test;
- the fifth column presents the bolt with the highest bolt force, the achieved bolt force ($F_{i,max}$) and the highest bolt force increment ($\Delta F_{i,max}$), i.e. the measured bolt force without pre-tension load;
- the sixth and seventh columns indicate the calculated design moment resistance ($M_{j,Rd}$), the re-calculated design moment resistance ($M^{*}_{j,Rd}$), and, in brackets, the corresponding utilisation ratio.

<table>
<thead>
<tr>
<th>test</th>
<th>achieved load level</th>
<th>$M_{Ed} / M_{c,Rd}$ [%]</th>
<th>bolt-row that failed first</th>
<th>measured maximum bolt force</th>
<th>$F_{i,max}$ [kN]</th>
<th>$\Delta F_{i,max}$ [kN]</th>
<th>calculated design moment resistance</th>
<th>$M_{j,Rd}$ [kNm] (%)</th>
<th>re-calculated design moment resistance</th>
<th>$M^{*}_{j,Rd}$ [kNm] (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TA</td>
<td>291 (873)</td>
<td>40.8</td>
<td>bolt-row B</td>
<td>277</td>
<td>244</td>
<td>560</td>
<td>64.1</td>
<td>655 (75.0)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TB</td>
<td>300 (900)</td>
<td>42.1</td>
<td>bolt-row B</td>
<td>291</td>
<td>282</td>
<td>649</td>
<td>72.1</td>
<td>757 (84.1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TC</td>
<td>364 (1,092)</td>
<td>51.1</td>
<td>bolt-row A</td>
<td>314</td>
<td>274</td>
<td>717</td>
<td>65.7</td>
<td>913 (83.6)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TD</td>
<td>462 (1,385)</td>
<td>64.7</td>
<td>bolt-row A</td>
<td>295</td>
<td>271</td>
<td>902</td>
<td>65.1</td>
<td>1,053 (76.0)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TE</td>
<td>626 (1,878)</td>
<td>65.4</td>
<td>bolt-row A</td>
<td>306</td>
<td>254</td>
<td>1,393</td>
<td>74.2</td>
<td>1,679 (89.4)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TF</td>
<td>634 (1,902)</td>
<td>66.2</td>
<td>bolt-row A</td>
<td>305</td>
<td>258</td>
<td>1,490</td>
<td>78.3</td>
<td>1,759 (92.5)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The test results showed that the calculated resistances in the case of all joint types were lower than the measured capacities. The comparison showed that the measured design resistances were at least 22% higher than the resistances calculated using standard material properties. The re-calculated design moment resistances were higher for all test specimens. The average deviation was 30% if calculated by assuming standard material properties and 16.6% if calculated by taking into account the measured end-plate and bolt properties.

In all specimens the geometry of the web plate (860 x 8) and the flange thickness ($t_f = 20 \text{ mm}$) was identical. The differences were as regards the width of the flanges (300 mm and 420 mm), the end-plate thickness ($t_{ep} = 16 \text{ - } 20 \text{ - } 24 \text{ mm}$), the number and position of the bolts and the arrangement of the end-plate extension and stiffeners, as shown in Table 3.10.

In accordance with the theoretical expectations, the end-plate design TF, which had the highest end-plate thickness ($t_{ep} = 24 \text{ mm}$) and four bolts in the extended part, resulted in the highest moment resistance.

Figures 3.141 to 3.146 present the measured deflections under the hydraulic jacks (red and blue curves), as shown in Figure 3.11, plotted against the moment. The vertical dashed lines indicate the calculated design moment resistance ($M_{j,Rd}$).
According to the measured deflection curves, all tests exhibit elastic behaviour up to the calculated design moment ($M_{j,Rd}$).
Comparison of the test results of tests TA and TB

Test specimens TA and TB were identical with the only exception of the end-plate thickness applied. Specimen TA included a 16 mm thick end-plate, whereas the end-plate thickness of TB was 20 mm.

Figure 3.147 shows the comparison of the bolt-row forces as a function of the measured moment. The bolt-row force diagrams of test TB (curves in light blue and orange) are steeper than those corresponding to test TA. This is due to the thicker end-plate.

While the curves related to test TA do not have a change in their shape, those corresponding to TB do. The failure occurred in both tests in the form of bolt group failure. The calculated failure mode was mixed bolt-and-plate failure (Mode 2) in both cases, but the test results for test TA indicated plate failure (Mode 1).

Comparison of the test results of tests TE and TF

The only difference between the specimens TE and TF was the applied end-plate thickness. This thickness was 20 mm for specimen TE and 24 mm for specimen TF.

Figure 3.148 presents the inner and outer bolt forces of the first bolt-row (A) for both tests.

The diagrams for inner bolts are steeper for test TF than for test TE, whereas the curves corresponding to the outer bolts (blue and light blue respectively) exhibit the same shape. This means that the end-plate deformations near the outer bolts were identical.

The change in the shape of the curves occurred at a load level of 1,400 kNm for test TE, and at 1,500 kNm for test TF.

The achieved failure in the extended end-plate part was mixed bolt-and-plate failure (Mode 2) for both specimens.
Figure 3.149 shows the inner and outer bolt forces for the second bolt-row (B) as a function of the measured moment.

The diagrams were steeper for specimen TF (curves in green and light blue) than for test TE. This observation has been explained by the thicker end-plate.

The failure occurred in both tests in the form of bolt failure (Mode 3) in bolt-row B.

The observed ultimate behaviour of the joint was in both cases mixed bolt-and-plate failure (Mode 2).

3.4.3 Verification of the developed design methods

The diagrams in Figures 3.150 and 3.151 show the bolt-row forces with the dashed lines indicating the re-calculated and measured moment resistances of the joint. The bolt-row forces corresponding to these load levels are summarized in Table 3.12.

The first column of Table 3.12 lists the bolt-rows of the tension zone, the second column contains the re-calculated bolt-row forces, the third, the measured bolt-row forces, and the fourth, the bolt-row forces measured at the load bearing capacity.

Table 3.12 shows that the re-calculated results provide the same load level for the bolts within the bolt group involving bolt rows B and C because of the symmetrical bolt arrangement. However, the measured values were higher in the first bolt-row (B) than in the second (C). The bolt-forces were compared at the same moment level and the sum of the forces in the tension zone (bolt row forces B + C) were about the same, the "measured/re-calculated" ratios were 1.03 for specimen TA and 1.07 for TB.

This observation means that the model uses a conservative approach, i.e. the distribution of the forces between the bolt-rows is assumed in a conservative way. Because of the conservative force distribution the calculation of the moment resistance is also conservative.
The results of test TC show that the triangular stiffener in the extended end-plate part reinforced the joint significantly. Figure 3.152 presents the bolt-row force diagrams and demonstrates that the force levels in bolt-rows A and B were practically the same. Due to the stiffener the force distribution was “symmetrical”.

Reference is made to the results of tests TB5, TB9 and TB13 with extended but unstiffened end-plates in chapter 3.3.3.2. Those measurements showed that the bolt forces were definitely higher in the “better” stiffened bolt-row under the tension flange than in the bolt row located in the extended part of the end plate, an observation that confirms the positive effect of the additional triangular stiffener.

Table 3.13 presents and compares the measured and re-calculated bolt forces. The calculated force levels for the bolt-row located in the extended part of the end-plate and for the second bolt-row were slightly different because of the different failure modes, Mode 2 in the extended part and Mode 3 for bolt-row B.

The design method gives an exact prediction for the bolt force in the bolt-row located in the extended part of the end-plate, and overestimates the force in the second bolt-row. As a conclusion it can be stated that the re-calculated and measured tension forces were in overall about the same (0.94) and their distribution, in view of the moment resistance, was conservative, which confirms that the design moment was calculated in a conservative way.

<table>
<thead>
<tr>
<th>Test TC</th>
<th>Re-calculated Bolt-Row Forces</th>
<th>Measured Bolt-Row Forces for M = 913 kNm</th>
<th>Measured Bolt-Row Forces for M* = 1,080 kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolt-row A</td>
<td>504.0 kN</td>
<td>506.3 kN</td>
<td>594.2 kN</td>
</tr>
<tr>
<td>Bolt-row B</td>
<td>544.2 kN</td>
<td>480.2 kN</td>
<td>545.8 kN</td>
</tr>
<tr>
<td>∑ Bolt-Row Forces</td>
<td>1,048 kN</td>
<td>987 kN</td>
<td>1,140 kN</td>
</tr>
</tbody>
</table>
The results of specimen TD show the positive effect of the triangular stiffener on the extended part of the end-plate.

Figure 3.153 presents the bolt-row forces of test TD and shows that the highest force developed in bolt-row A, followed by rows B and C.

Table 3.14 summarizes the measured and re-calculated bolt forces. The re-calculated force level in the bolt row located in the extended part of the end plate is slightly higher than that calculated in the second and third bolt-rows. The measured values show the same tendency.

The design method underestimates the force in the first and second rows and overestimates it in the third bolt-row. It can be seen that the sum of the tension forces was the same ("measured/re-calculated" ratios 1.02), the difference was in the force distribution only.

In tests TE and TF the bolt arrangement included four bolts in one row. The developed design method was described in detail in chapter 3.2.2.

The connection design included symmetrical T stubs. The reason of the different bolt forces is that the failure modes of the bolt-rows were different. The failure mode for the extended part (bolt-row A) was Mode 2 whereas for the bolts under the tension flange (bolt-row B) it was Mode 3.

![Fig 3.153 Bolt forces in bolt-rows A, B and C, of the test TD.](image)

![Fig 3.154 Bolt forces in bolt-rows A and B of test TE.](image)

![Fig 3.155 Bolt forces in bolt-rows A and B of test TF.](image)
The diagrams in Figures 3.154 and 3.155 show the bolt-row forces together with the re-calculated and measured resistances of the joint.

The diagrams present the effect of the triangular stiffener in the extended part of the end-plate. It can be seen that the highest force increments were measured in the bolts of the first row (A). Comparing these findings with the results in chapter 3.3.3.4 one can be convinced of the positive effect of the triangular stiffener.

<table>
<thead>
<tr>
<th>Table 3.15 Comparison of the re-calculated and measured bolt forces.</th>
</tr>
</thead>
<tbody>
<tr>
<td>test TE</td>
</tr>
<tr>
<td>re-calculated bolt-row forces</td>
</tr>
<tr>
<td>$M_{1,Rd} = 1.679 \text{ kNm}$</td>
</tr>
<tr>
<td>measured bolt-row forces</td>
</tr>
<tr>
<td>for $M = 1.679 \text{ kNm}$</td>
</tr>
<tr>
<td>measured bolt-row forces</td>
</tr>
<tr>
<td>for $M^* = 1.830 \text{ kNm}$</td>
</tr>
<tr>
<td>inner bolts</td>
</tr>
<tr>
<td>--------------</td>
</tr>
<tr>
<td>bolt-row A</td>
</tr>
<tr>
<td>bolt-row B</td>
</tr>
<tr>
<td>$\Sigma$ bolt-row forces</td>
</tr>
<tr>
<td>test TF</td>
</tr>
<tr>
<td>re-calculated bolt-row forces</td>
</tr>
<tr>
<td>$M_{1,Rd} = 1.759 \text{ kNm}$</td>
</tr>
<tr>
<td>measured bolt-row forces</td>
</tr>
<tr>
<td>for $M = 1.759 \text{ kNm}$</td>
</tr>
<tr>
<td>measured bolt-row forces</td>
</tr>
<tr>
<td>for $M^* = 1.790 \text{ kNm}$</td>
</tr>
<tr>
<td>inner bolts</td>
</tr>
<tr>
<td>--------------</td>
</tr>
<tr>
<td>bolt-row A</td>
</tr>
<tr>
<td>bolt-row B</td>
</tr>
<tr>
<td>$\Sigma$ bolt-row forces</td>
</tr>
</tbody>
</table>

The bolt-row forces are summarized in Table 3.15. The re-calculated force levels for the inner bolts were approximately the same as the measured values, the maximum difference being 9%. For the outer bolts the method underestimates the measurements in the extended part, and to about the same extent, it also overestimates the measurements in the second row.

Similarly to the other tests in test series II the sums of the re-calculated and measured tension forces were the same with the "measured/re-calculated" ratios 0.99 for TE and 1.05 for TF. The design method uses a conservative approach to calculate the distribution of the forces between the bolt-rows, therefore the force levels in the first bolt-row were underestimated, and those in the second bolt-row were overestimated. Because of this conservative force distribution the calculation of the moment resistance is also conservative.

The final conclusion is that the developed method provides the failure mode of the joint with a high accuracy. The sum of the re-calculated tension forces shows in average less than 5% deviation from the measured forces. The bolt-row forces of the model were underestimated in the first and second bolt rows, and overestimated in the third row. This force distribution ensures a conservative model approach for the moment resistance of the joint.
3.5 Recommendations for practical design

For practical applications, the following conclusions are drawn:

- In the case of HammerHead type joints, a long HammerHead end-plate and web plate is recommended to ensure a higher stiffness of the extended end-plate part and so to make sure that bolt forces are higher in this part. In the studied HammerHead connections, the bolts should be placed as close as possible to the “stiffened joint elements”, i.e. the tension flange. In the case of what is called thin end-plate design, i.e. when plate failure is expected (Mode 1), it is suggested to place four bolts instead of two in the first and the second bolt-rows.

- The four bolts in one row joint arrangement is a competitive solution if the bolt design is chosen adequately. From the point of view of load capacity, it is recommended to prefer this arrangement rather than solutions such as the redesign of the bolt arrangement or increasing the overall dimensions of the beam. On the basis of the experimental results it is suggested to place the bolts symmetrical to the tension flange so as to achieve the most favourable utilization ratio in the bolts.

- The triangular stiffeners are efficient and have positive influence on the load bearing capacity of the joint because of their local effect. They can help avoiding bolt group failure and enhancing the tension resistance of the web. In the tests, the use of the stiffeners in the extended part of the end-plate showed a positive effect on the bolt force distribution. (See chapter 3.4.3 for a comparison between tests TC and end-plate type IV and between tests TE (TF) and end-plate type III.)

- Bolts having quality 8.8 ensure a “good-natured” ultimate behaviour, i.e. a behaviour in which the load-deflection diagrams of the joint show definite yielding points.
4 Conclusions

4.1 New scientific results

As a result of the presented research the following new scientific results are concluded.

4.1.1 The theses of the PhD dissertation in English

**Thesis 1**
I designed and performed an experimental programme on full scale bolted end-plate type light crane bracket joints (without compression flange) under static and fatigue loading in bending, with the purpose of defining the static load bearing capacity and fatigue behaviour of the studied joint type.

On the basis of the experimental results I characterized the static and fatigue behaviour of the joints as follows:

a.) I determined the load bearing capacity of the tested joints and characterized the different failure modes (web buckling and end-plate failure). Based on the results I described the effect of the bolt diameter and stiffeners (transversal stiffeners and backing plates) on the load bearing capacity.

b.) I determined the fatigue failure modes of the studied joints and described the main parameters (bolt diameter, web thickness and type of stiffener) that influence the resistance, the stiffness and their degradation during the test.

c.) I introduced a parabolic curve approximation of the secant stiffness vs. number of cycles diagram so as to classify the fatigue behaviour of the joints, and I defined the relevant coefficients on the basis of experiments.

**Thesis 2**
I performed analytical studies on the basis of the experimental research on light crane bracket joints:

a.) As the arrangement in question is innovative in the sense that it lacks the compression flange, the Eurocode 3 does not provide recommendations for the evaluation of the design resistance and the stiffness of its web plate component. I developed an analytical calculation method to evaluate the load bearing capacity and I introduced a new stiffness coefficient to calculate the initial stiffness of the joint. I verified the developed analytical method on the basis of test results and justified its accuracy.

b.) I developed and verified a geometrically and materially non-linear FE model based on shell-elements for bolted end-plate type light crane bracket joints. I used this advanced FE model to predict the load bearing capacity and failure modes of the specimens before the experimental tests were done. After the completion of the tests I examined the accuracy of the model by comparing its results to the experimental results.

**Thesis 3**
I designed and performed an experimental programme on full-scale bolted end-plate type steel beam-to-beam joints under static loading in bending, with the aim of defining the load bearing capacity and the load vs. deformation relationship. The investigated joint types were joints with additional stiffeners between the bolts as well as the so-called HammerHead joints and end-plate type joints with four bolts in a row, connection types that are out of the scope of the Eurocode 3 standard design method. I analyzed and characterized the static behaviour of the joints on the basis of the experimental results, and came to the following conclusions:

a.) On the basis of the measured bolt forces and end-plate deformations I demonstrated that the stiffness distribution within the joint has a significant influence on the bolt forces.

b.) I developed a method to measure the end-plate deformations both in the elastic and plastic behaviour phase.

c.) On the basis of the measured load vs. deformation relationship of the end-plate I proved that the governing end-plate deformations can already be identified in the elastic phase.
The Eurocode 3 does not include explicit rules for additional stiffeners and does not give recommendations as to the evaluation of the design resistance of the HammerHead joints and the four bolts in one row type end-plate joints, I performed analytical studies on these joint types based on the component method (same as used by the Eurocode 3). The results of these analytical studies are summarized as follows:

a.) I developed a design method to evaluate the moment resistance of joints with additional stiffeners between the bolts. I verified and validated the design method using the test results.

b.) I developed a design method to evaluate the moment resistance of the HammerHead type structural joints. I verified the design method using the test results and showed the limitations of the method.

c.) I developed a Eurocode 3 compatible design method for the four bolts in one row type end-plate joints. With the application of the developed method the design resistance of the joints including the cases when there are two and/or four bolts in each bolt-row can be evaluated. I verified the model by comparing the evaluated bolt-row forces to the experimental measurements, and on the basis of the test results I verified the accuracy of the model developed.

Based on a comparison of the analytical and experimental results I derived practical design rules that ensure favourable fatigue and static behaviour and resistance.

a.) I demonstrated that, within certain limits, the load bearing capacity and stiffness of a bolted crane bracket joint subjected to static loading can be enhanced with additional stiffeners without any danger. When there is fatigue loading, however, load bearing capacity should be increased by applying larger overall dimensions rather than by introducing additional stiffeners.

b.) In relation to the HammerHead joint design, I pointed out that, as far as possible, the bolts should be placed close to the “stiffened joint elements”, i.e. the tension flange.

- On the basis of the results obtained I showed that the four bolts in one row joint arrangement is a competitive solution from the point of view of load capacity with respect to solutions such as the redesign of the bolt arrangement or increasing the overall dimensions of the beam. On the basis of the experimental results I suggested to place the bolts symmetrical to the tension flange so as to achieve the most favourable utilization ratio in the bolts.

- On the basis of the experimental results I pointed out that triangular stiffeners are efficient and can help avoiding bolt group failure and enhancing the tension resistance of the web.
4.1.2 The theses of the PhD dissertation in Hungarian

1. tézis

Megterveztem és végrehajtottam egy kísérleti programot acélszerkezetű, homloklemezes, csavarozott, könnyű darupályakonzolok (nyomott öv nélküli kialakítás) statikus és fárasztó nyomatéki terhelés alkalmazásával történő vizsgálatára, a szerkezeti kialakítás statikus teherbírásának és fáradási viselkedésének meghatározása céljából.

A kísérleti eredmények alapján a következőképpen határoztam meg a kapcsolatok statikus és fáradási viselkedési módját:

a.) Meghatároztam a vizsgált kapcsolatok statikus teherbírását, és jellemeztem a különböző tönkremeneteli módokat (gerinchorpadás és homloklemeztorés). Az eredményekre támaszkodva leírtam a csavaratmérő és az alkalmazott merevítési kialakítások (gerincmerekítés, övhízaló lemez) hatását.

b.) Meghatároztam a fárasztó igénybevételhez tartozó tönkremeneteli módokat, és jellemeztem az ellenállást és a merészetet, illetve az ezek leépülését befolyásoló főbb tényezőket (csavaratómérő, gerincvastagság, kapcsolat merevítettségének módja).

c.) A terhelés alatti merevségváltozás jellemzésére bevezettem egy másodfokú közelítést, és kísérleti eredményekből meghatároztam a közelítő függvény paraméterét.

2. tézis

Analitikus vizsgálatokat végeztem a könnyű darupályakonzolok vizsgálata során nyert eredmények alapján:

a.) A konzol innovatív kialakítása következtében (nincs nyomott öv) az Eurocode 3 szabvány nem tartalmaz eljárást a statikus teherbírás és a gerincmerés merészetének számítására. Kifejlesztettem egy analitikus számítási módszert a statikus teherbírás meghatározására, valamint bevezettem egy új merészetleni tényezőt a kapcsolat kezdetei merészetének számításához. A kísérleti eredmények felhasználásával ellenőriztem és igazoltam a kifejlesztett számítási módszert.

b.) Kidolgoztam és ellenőriztem egy nemlineáris végeselemes modellt csavarozott, homloklemezes, könnyű darupályakonzolok számítására. A kifejlesztett modellt felhasználtam a statikus teherbírás és a várható tönkremeneteli mód kísérleteket megelőző meghatározására. A kísérletek elvégzése után összehasonlítottam a számított és a mért eredményeket, és igazoltam a modell helyességét.

3. tézis

Megterveztem és végrehajtottam egy kísérleti programot acélszerkezetű, homloklemezes, csavarozott gerenda-gerenda kapcsolatok statikus teherbírás vizsgálatára, valamint az erő-alakváltozás vizsgálatára nyomatéki terhelés alkalmazása mellett. A kísérletekben vizsgált kapcsolatok a csavarosok között kiegészítő merevítőkkel ellátottak, valamint úgynevezett kalapácsfej ("HammerHead") kialakításuk voltak, illetve soronként négy csavart tartalmaztak. Ezek a kapcsolati kialakítások kívül esnek az Eurocode 3 szabvány által lefedett területen.

Kiértékeltem és jellemeztem a statikus terhelés során mért kapcsolati viselkedést, és a mérési eredmények alapján az alábbi megállapításokat tettem:

a.) A mért csavarerők és homloklemez-alakváltozások alapján kimutattam, hogy a kapcsolaton befülı merevségeloszlás jelentősen befolyásolja a csavarokban ébredő erőket.

b.) Módszert dolgoztam ki a homloklemez alakváltozásainak mérésére a lemez rugalmas és képlekény állapotában.

c.) A mérési eredményeket felhasználva kimutattam, hogy a tönkremenetelhez tartozó homloklemez-deformációk már a rugalmas zónában felismerhetőek.
4. tézis

Mivel az Eurocode 3 nem tesz ajánlást a teherbírás meghatározására kalapácsfej típusú kapcsolatokra, valamint nem tartalmaz explicit eljárást csavarok között elhelyezett merevítőkre, illetve az egy csavarsorban négy csavart tartalmazó kapcsolatokra, kidolgoztam egy analitikus eljárást ilyen kapcsolati kialakításokra a szabvány komponensmódszerét felhasználva. Az analitikus vizsgálatok eredményeit az alábbiakban foglalom össze:

a.) Kidolgoztam egy méretezési eljárást csavarok között kiegészítő merevítőket tartalmazó kapcsolatok nyomatéki ellenállásának számítására. A mérési eredmények segítségével ellenőriztem és igazoltam a kidolgozott eljárás helyességét.

b.) Kidolgoztam egy méretezési eljárást kalapácsfej típusú kapcsolatok nyomatéki ellenállásának számítására. A méretezési eljárást a mérési eredmények segítségével igazoltam és megmutattam a modell alkalmazási határait.

c.) Kifejlesztettem egy Eurocode 3-alapú méretezési eljárást az egy csavarsorban négy csavart tartalmazó homoloklemezes kapcsolatok számítására. A kidolgozott eljárás alkalmazásával az egy csavarsorban két és/vagy négy csavart tartalmazó homoloklemezes kapcsolatok nyomatéki ellenállása határozható meg. Az eljárást a mért és a számított csavarerők összehasonlításával ellenőriztem, majd az eredményekre támaszkodva igazoltam a kidolgozott eljárás helyességét.

5. tézis

Az analitikus vizsgálatok és a kísérleti eredmények összehasonlítása alapján tervezési szabályokat határoztam meg, melyek kedvező fáradási és statikus viselkedésű, valamint ellenállású kialakításokat tesznek lehetővé:

a.) - Kimutattam, hogy a statikusan terhelt csavarozott darupályakonzolok teherbírása és merevsége kiegészítő merevítőkkel, bizonyos határok között, növelhető. Amennyiben azonban a terhelés fárasztó jellegű, a teherbírás növelésének célja a befoglaló méretek növelése, semmint kiegészítő merevítők alkalmazása.

- Rámutattam fárasztó terhelésnek kitett kapcsolatoknál a kapcsolat duktilitását jelentős szerepét a merevítők elhelyezésének következményeinek elkerülésében. A kidolgozott eredmények igazoltak, hogy a nagyobb merevségű kapcsolatok alacsonyabb ismétlészámúan működnek.

- Rámutattam fárasztó terhelésnek kitett kapcsolatoknál a hirtelen, idő előtt bekövetkező tönkremeneteket, mint a csavarból, kerülni kell.

- A darupályakonzolokon végzett kísérletek eredményei alapján megmutattam, hogy az övhizlaló lemezes kialakítás fárasztó terhelés esetén előnyösebben viselkedik, mint a behegesztett gerenda kapcsolati megoldás.

b.) - Kalapácsfej kialakítású kapcsolatoknál kimutattam, hogy a csavarokat, a lehetőségek szerint, a „merev” kapcsolati elemei közéletel, azaz a húzott övre közelében kell elhelyezni.

- Az elvégzett kísérletek eredményei alapján megmutattam, hogy az egy sorban négy csavart tartalmazó kapcsolatok megfelelően megválasztott csavarkiosztás mellett a kapcsolati teherbírás szempontjából versenyképes megoldást nyújtanak például a kapcsolat újratevezésével vagy a gerenda befoglaló méretek növelésével szemben. A kísérleti eredményekre támaszkodva a húzott övre szimmetrikus csavarképet javaslom, mellyel a csavarok előnyös kihasonlítása érhető el.

- Kísérleti eredményekkel igazolva kimutattam a háromszög alakú merevítők hatékonyságát, melyek alkalmazásával meggátolható a csavarok csoportos tönkremenetele, és növelhető a gerinc húzási ellenállása.
4.2 Publications on the subject of the thesis

Books


Papers in book


International journal papers


International conference papers


Technical/University Reports


4.3 Proposed directions for further research

The results of the research presented above can be summarized as follows:

Experimental tests as well as numerical studies were performed on innovatively designed bolted end-plate joints. The test results were evaluated and the developed design methods were verified and their accuracy checked against the results of the tests completed.

Crane bracket joints

A design method for light crane brackets has been developed to calculate the failure mode, the load bearing capacity and the stiffness of the joint. With the help of the proposed parabolic approximation and its parameters the fatigue behaviour of the tested brackets can be predicted.

Further tests could be performed, with different geometrical dimensions and joint arrangements, so as to check the accuracy of, and generalize, the design method and the validity of the parable approximation.

Beam-to-beam joints

The developed design methods can be used to calculate the failure mode and the load bearing capacity of joints with additional stiffeners, HammerHead type structural joints and four bolts in one row type end-plate joints.

It is planned to continue the experimental programme for other bolt and joint arrangements so as to further verify the developed design methods. There are also further tests and parametric studies planned on HammerHead type structural joints with a view of clarifying the main parameters that influence of the HammerHead effects and developing the modifying factor. For four bolts in one row type end-plate joints, the research aim is to study further yield line patterns, in particular those that belong to Mode 2, and, on this basis, to refine the design method.

In parallel with the experiments, advanced numerical models will be developed to simulate the joint behaviour and calculate the load bearing capacity and the failure mode.
Bolted end-plate joints for crane brackets and beam-to-beam connections

References

Standards

[Background Documentation] Design of Steel Structures, Background Documentation, Chapter 6, Document 6.09, Beam to column connections, March 1983.


[DIN 15 018] DIN 15.018 Teil 1: Krane; Grundsätze für Stahlbauwerke, Berechnung, 11/84; Teil 2: Krane; Stahlbautragwerke, Grundsätze für die bauliche Durchbildung und Ausführung, 11/84.

[DIN 18 800] DIN 18.800 Stahlbauten Teil 1 Bemessung und Konstruktion, Deutsches Institut für Normung e. V., 11/90; Teil 2 Stabilitätsfälle, 12/90.


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References


References


References


Bolted end-plate joints for crane brackets and beam-to-beam connections

References


Bolted end-plate joints for crane brackets and beam-to-beam connections

References

L. Katula

Budapest University of Technology and Economics Department of Structural Engineering


[Zoetemeijer 1990] Zoetemeijer, P.: “Summary of the research on bolted beam-to-column connections”, Delft University of Technology, Faculty of Civil Engineering, Stevin Laboratory report 6-90-02, 1990. (This report is also published as a background report for Eurocode 3, Chapter 6).
APPENDICES

BOLTED END-PLATE JOINTS FOR CRANE BRACKETS
AND BEAM-TO-BEAM CONNECTIONS

PhD Dissertation

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<th>Title</th>
<th>Page</th>
</tr>
</thead>
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<tr>
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<td>p. 8.</td>
</tr>
</tbody>
</table>
Geometry of the test specimens

Location of the gauges under static loading

Location of the gauges under fatigue loading

Fig. A1 Test specimen Z1.
Location of the gauges under static loading

Location of the gauges under fatigue loading

Fig. A2 Test specimen Z2.
Fig. A3 Test specimen K1.

Location of the gauges under static loading

Location of the gauges under fatigue loading
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix A

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Fig. A4 Test specimen K2.
Fig. A5 Test specimen K2_z.
Location of the gauges under static loading | Location of the gauges under fatigue loading
---|---
**test specimen K3-M20** | no test

**test specimen K3-M24**

---

Fig. A6 Test specimen K3.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Fig. A7 Test specimen TB2.

Fig. A8 Test specimen TB3.
Fig. A9 Test specimen TB4.

Fig. A10 Test specimen TB5.
Fig. A11 Test specimen TB6.

Fig. A12 Test specimen TB7.
Fig. A13 Test specimen TB8.

Fig. A14 Test specimen TB9.
Fig. A15 Test specimen TB10.

Fig. A16 Test specimen TB11.
Fig. A17 Test specimen TB12.

Fig. A18 Test specimen TB13.
Fig. A19 Test specimen TB1, fixed beam.
Material tests

Test on crane brackets

The steel grade of the specimens was S355. To verify this material test specimens were prepared. Table B1 shows the locations where the specimens were cut out and the measured main material properties.

The material properties of the tension tests were accomplished according to the standard EN 1002-1.

![Material test of test specimen Z1.](image1)

![Material test of the test specimen K2 with M20 bolts, bracket web.](image2)

As an example, Figure B2 shows the determination of the yield point and Young's modulus.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix B

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Budapest University of Technology and Economics Department of Structural Engineering

Fig. B3 Stress-elongation diagram of test specimen K1 with M20 bolts, column web.

Fig. B4 Stress-elongation diagram of test specimen K1 with M20 bolts, column flange.

Fig. B5 Stress-elongation diagram of test specimen K1 with M20 bolts, bracket web.

Fig. B6 Stress-elongation diagram of test specimen K1 with M20 bolts, bracket flange.

Fig. B7 Stress-elongation diagram of test specimen K1 with M20 bolts, end-plate.

Fig. B8 Stress-elongation diagram of test specimen K2 with M20 bolts, bracket web.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix B

L. Katula
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Fig. B9 Stress-elongation diagram of test specimen K2 with M20 bolts, bracket flange.

Fig. B10 Stress-elongation diagram of test specimen K2 with M20 bolts, end-plate.

Fig. B11 Stress-elongation diagram of test specimen K3 with M20 bolts, column web.

Fig. B12 Stress-elongation diagram of test specimen K3 with M20 bolts, column flange.

Fig. B13 Stress-elongation diagram of test specimen K3 with M20 bolts, bracket web.

Fig. B14 Stress-elongation diagram of test specimen K3 with M20 bolts, bracket flange.
Table B1 Actual material properties.

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<th>tensile strength [N/mm²]</th>
<th>Young modulus E [N/mm²]</th>
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Fig. B15 Stress-elongation diagram of test specimen K3 with M20 bolts, end-plate.
Tests on beam-to-beam joints

Series I

Altogether test specimens were cut from six end-plates, and these specimens were subjected to the material tests. Figures B16-B19 show the process.

Fig. B16 Marked end-plate.

Fig. B17 Cutting of the test specimen.

Fig. B18 Cut surface.

Fig. B19 End-plate after cutting the test specimen.

Four specimens were cut out, as the follows: The 12 mm thick test specimens were cut out form the specimen TB2, the 15 mm thick test specimen form item TB6 and the 15 mm thick test specimen from TB10.

Fig. B20 Parallel-side milling of the test specimens.

Fig. B21 Raw test specimens.
The test specimens were milled in the Laboratory for Testing of Structures of the Department of Structural Engineering. The material tests were carried out also in the same laboratory.
Fig. B28 Load-deformation diagram, specimen A1 (tep = 12 mm).

Fig. B29 Load-deformation diagram, specimen A2 (tep = 12 mm).

Fig. B30 Load-deformation diagram, specimen A3 (tep = 12 mm).

Fig. B31 Load-deformation diagram, specimen A4 (tep = 12 mm).

Fig. B32 Load-deformation diagram, specimen B1 (tep = 15 mm).

Fig. B33 Load-deformation diagram, specimen B2 (tep = 15 mm).
Fig. B34 Load-deformation diagram, specimen B3 (tep = 15 mm).

Fig. B35 Load-deformation diagram, specimen B4 (tep = 15 mm).

Fig. B36 Load-deformation diagram, specimen C1 (tep = 20 mm).

Fig. B37 Load-deformation diagram, specimen C2 (tep = 20 mm).

Fig. B38 Load-deformation diagram, specimen C3 (tep = 20 mm).

Fig. B39 Load-deformation diagram, specimen C4 (tep = 20 mm).
Table B2 Measured geometrical and material properties.

<table>
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<th>specimen</th>
<th>nominal cross-section [mm]</th>
<th>measured cross-section [mm]</th>
<th>a x b [mm²]</th>
<th>base length</th>
<th>base length after test [mm]</th>
<th>elongation [%]</th>
<th>yield point [N/mm²]</th>
<th>tensile strength [N/mm²]</th>
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<tr>
<td></td>
<td>a* b*</td>
<td>a</td>
<td>b</td>
<td>A</td>
<td>L₀</td>
<td>%</td>
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<td>-</td>
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Table B3 shows the measured main material properties.

Table B3 Material properties.

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<tr>
<th>test 1</th>
<th>test 2</th>
<th>test 3</th>
<th>test 4</th>
<th>average</th>
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<td>fᵤ [N/mm²]</td>
<td>fᵧ [N/mm²]</td>
<td>fᵤ [N/mm²]</td>
<td>fᵧ [N/mm²]</td>
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<td>509</td>
<td>328</td>
<td>511</td>
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Series II

In total specimens were cut form three end-plates, and these specimens were subjected to the material tests.

From each end-plate thickness (16 mm; 20 mm; 24 mm) four specimens were cut. The 16 mm thick specimens were cut from specimen D, the 20 mm thick specimens from test specimen E, and the 24 mm thick specimens from test specimen F.
The specimens were prepared and the material tests carried out in the laboratory of the BME, Department of Structural Engineering.

Fig. B46 Milling of the specimens.

Fig. B47 Completed test specimens.

Fig. B48 Test specimen failure 1 (B3).

Fig. B49 Test specimen failure 2 (B3).

Fig. B50 Test specimens after the test.
Fig. B51 Load-deformation diagram, specimen A1b (tep = 16 mm).

Fig. B52 Load-deformation diagram, specimen A2b (tep = 16 mm).

Fig. B53 Load-deformation diagram, specimen A3b (tep = 16 mm).

Fig. B54 Load-deformation diagram, specimen A4b (tep = 16 mm).

Fig. B55 Load-deformation diagram, specimen B1b (tep = 20 mm).

Fig. B56 Load-deformation diagram, specimen B2b (tep = 20 mm).
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix B L. Katula

Budapest University of Technology and Economics Department of Structural Engineering

Fig. B57 Load-deformation diagram, specimen B3b ($t_{ep} = 20$ mm).

Fig. B58 Load-deformation diagram, specimen B4b ($t_{ep} = 20$ mm).

Fig. B59 Load-deformation diagram, specimen C1b ($t_{ep} = 24$ mm).

Fig. B60 Load-deformation diagram, specimen C2b ($t_{ep} = 24$ mm).

Fig. B61 Load-deformation diagram, specimen C3b ($t_{ep} = 24$ mm).

Fig. B62 Load-deformation diagram, specimen C4b ($t_{ep} = 24$ mm).
Table B4 Measured geometrical and material properties.

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<tr>
<th>specimen</th>
<th>nominal cross-section [mm]</th>
<th>measured cross-section [mm]</th>
<th>a x b [mm²]</th>
<th>base length L₀ [mm]</th>
<th>base length after test L*₀ [mm]</th>
<th>elongation [%]</th>
<th>yield point fᵧ [N/mm²]</th>
<th>tensile strength fᵤ [N/mm²]</th>
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<td>180</td>
<td>29</td>
<td>374.6</td>
<td>556.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B4b</td>
<td>19.5 29.7 579.2</td>
<td>179</td>
<td>28</td>
<td>366.7</td>
<td>550.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C1b</td>
<td>23.4 29.5 690.3</td>
<td>202</td>
<td>30</td>
<td>338.9</td>
<td>530.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C2b</td>
<td>23.2 29.6 688.7</td>
<td>201</td>
<td>29</td>
<td>345.1</td>
<td>534.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C3b</td>
<td>23.5 29.5 693.3</td>
<td>201</td>
<td>30</td>
<td>358.4</td>
<td>538.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C4b</td>
<td>23.5 29.6 695.6</td>
<td>201</td>
<td>30</td>
<td>352.8</td>
<td>537.9</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table B5 shows the measured main material properties.

<table>
<thead>
<tr>
<th>test 1</th>
<th>test 2</th>
<th>test 3</th>
<th>test 4</th>
<th>average</th>
</tr>
</thead>
<tbody>
<tr>
<td>fᵧ [N/mm²]</td>
<td>fᵤ [N/mm²]</td>
<td>fᵧ [N/mm²]</td>
<td>fᵤ [N/mm²]</td>
<td>fᵧ [N/mm²]</td>
</tr>
<tr>
<td>plate thickness</td>
<td>16</td>
<td>363.9</td>
<td>564.6</td>
<td>363.7</td>
</tr>
<tr>
<td>tₑ₀ [mm]</td>
<td>20</td>
<td>373.6</td>
<td>555.1</td>
<td>376.3</td>
</tr>
<tr>
<td>plate thickness</td>
<td>24</td>
<td>338.9</td>
<td>530.8</td>
<td>345.1</td>
</tr>
</tbody>
</table>
Test on bolts

Altogether twelve bolts were subjected to tensile tests as part of the material tests, six from bolt grade 8.8 and six from 10.9. The material tests were performed in the laboratory of the BME, Department of Structural Engineering with the ZD 100 RENEW testing machine, which has a maximum load capacity of 1,000 kN.

Figures B63 and B64 show the tested bolts before and after test.

![B bolt grade 10.9 before and after test](image)

a.) bolts before test  

b.) bolts after test 

Fig. B63 Bolt grade 10.9.

![B bolt grade 8.8 before and after test](image)

a.) bolts before test  

b.) bolts after test 

Fig. B64 Bolt grade 8.8

Figure B65 shows the testing machine, and Figure B66 presents an example of a bolt after failure.

![B testing machine](image)

Fig. B65 The testing machine ZD 100.

![B bolt failure](image)

Fig. B66 Bolt failure.
Figures B67 and B68 present load-displacement curves measured during the material tests. Table B6 shows the measured maximal bolt resistances.

Table B6 Results of the material test.

<table>
<thead>
<tr>
<th>bolt</th>
<th>B1</th>
<th>B2</th>
<th>B3</th>
<th>B4</th>
<th>B5</th>
<th>B5</th>
<th>average [kN]</th>
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<tr>
<td>10.9</td>
<td>273.5</td>
<td>265.4</td>
<td>265.2</td>
<td>277.3</td>
<td>272.7</td>
<td>278.7</td>
<td><strong>272.1</strong></td>
</tr>
<tr>
<td>8.8</td>
<td>225.8</td>
<td>223.3</td>
<td>224.4</td>
<td>222.8</td>
<td>223.9</td>
<td>226.0</td>
<td><strong>224.4</strong></td>
</tr>
</tbody>
</table>
Pre-tensioning of the bolts

**Test series I**

Table C1 Measured preloads in the bolts.

<table>
<thead>
<tr>
<th>bolt position</th>
<th>test series I</th>
<th>name of the specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TB2</td>
<td>TB3</td>
</tr>
<tr>
<td>A [kN]</td>
<td>48.3</td>
<td>60.6</td>
</tr>
<tr>
<td>AA [kN]</td>
<td>-</td>
<td>67.1</td>
</tr>
<tr>
<td>A2 [kN]</td>
<td>43.3</td>
<td>47.9</td>
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<td>AA2 [kN]</td>
<td>-</td>
<td>64.6</td>
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<td>B [kN]</td>
<td>35.7</td>
<td>65.0</td>
</tr>
<tr>
<td>BB [kN]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B2 [kN]</td>
<td>36.8</td>
<td>40.4</td>
</tr>
<tr>
<td>BB2 [kN]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>C [kN]</td>
<td>35.6</td>
<td>59.2</td>
</tr>
<tr>
<td>CC [kN]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>C2 [kN]</td>
<td>44.6</td>
<td>67.8</td>
</tr>
<tr>
<td>CC2 [kN]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>D [kN]</td>
<td>45.2</td>
<td>58.2</td>
</tr>
<tr>
<td>D2 [kN]</td>
<td>41.5</td>
<td>52.4</td>
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<tr>
<td>E [kN]</td>
<td>49.1</td>
<td>44.2</td>
</tr>
<tr>
<td>E2 [kN]</td>
<td>50.9</td>
<td>-</td>
</tr>
<tr>
<td>F [kN]</td>
<td>27.8</td>
<td>-</td>
</tr>
<tr>
<td>F2 [kN]</td>
<td>35.4</td>
<td>-</td>
</tr>
</tbody>
</table>

Fig C1 Positions of the load cells.
Test series II

Table C2 Measured preloads in the bolts.

<table>
<thead>
<tr>
<th>test series II</th>
<th>name of the specimen</th>
<th>TA</th>
<th>TB</th>
<th>TC</th>
<th>TD</th>
<th>TE</th>
<th>TF</th>
</tr>
</thead>
<tbody>
<tr>
<td>A [kN]</td>
<td>-</td>
<td>-</td>
<td>65.0</td>
<td>65.4</td>
<td>116.5</td>
<td>107.1</td>
<td></td>
</tr>
<tr>
<td>AA [kN]</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>114.2</td>
<td>106.4</td>
<td></td>
</tr>
<tr>
<td>A2 [kN]</td>
<td>-</td>
<td>-</td>
<td>76.3</td>
<td>77.1</td>
<td>90.4</td>
<td>87.9</td>
<td></td>
</tr>
<tr>
<td>AA2 [kN]</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>75.8</td>
<td>98.4</td>
<td></td>
</tr>
<tr>
<td>B [kN]</td>
<td>42.1</td>
<td>36.7</td>
<td>69.0</td>
<td>74.0</td>
<td>90.2</td>
<td>88.0</td>
<td></td>
</tr>
<tr>
<td>BB [kN]</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>97.7</td>
<td>85.6</td>
<td></td>
</tr>
<tr>
<td>B2 [kN]</td>
<td>53.7</td>
<td>39.8</td>
<td>64.5</td>
<td>75.2</td>
<td>101.1</td>
<td>109.3</td>
<td></td>
</tr>
<tr>
<td>BB2 [kN]</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>90.6</td>
<td>97.6</td>
<td></td>
</tr>
<tr>
<td>C [kN]</td>
<td>54.2</td>
<td>33.2</td>
<td>-</td>
<td>88.6</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>C2 [kN]</td>
<td>67.3</td>
<td>51.0</td>
<td>-</td>
<td>85.8</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>D [kN]</td>
<td>66.1</td>
<td>50.5</td>
<td>73.3</td>
<td>105.2</td>
<td>121.4</td>
<td>99.7</td>
<td></td>
</tr>
<tr>
<td>D2 [kN]</td>
<td>85.5</td>
<td>57.1</td>
<td>73.2</td>
<td>98.1</td>
<td>124.4</td>
<td>115.4</td>
<td></td>
</tr>
</tbody>
</table>

Fig C2 Positions of the load cells.

The bolts were preloaded in each test. The M20 bolts and the M24 bolts - in the fixed connection - were pre-tensioned by a maximum preload of 200 Nm and 450 Nm, respectively.

In order to get a homogeneous preload level in the bolts of the investigated connection, a three-step preload process was used. First the bolts were strained by hand, then, beginning with the bolt-row farthest from the compression flange, the bolts were preloaded by a pneumatic screwdriver, and in the third step, because of the end-plate deformations and other imperfections, the bolts were preloaded again with the pneumatic screwdriver.

Due to the following effects the achieved pre-load levels were not exactly the same:
- friction differences of the bolts (imperfect form, different coating thickness of the bolts),
- the bolt nut and/or bolt head was not exactly perpendicular to the end-plate (erection imperfection),
- the stiffness distribution in the end-plate was not homogeneous, i.e. the connected plate parts adjacent to the flange had higher stiffness than the parts near the edge.
Measuring the end-plate surface

A special equipment were developed to measure the deformation of the end-plate after the test, as shown in Figures D1, D2, D3 and D4.

Fig. D1 The testing bench.

Fig. D2 Specimens with the testing bar.

Fig. D3 Measuring wheel with inductive transducer.

Fig. D4 Magnetic scale to measure the length.

Benchmark data were collected in the web direction at each 0.25 mm, with an accuracy of 0.001 mm in terms of altitude. To measure the length a magnetic scale with an accuracy of 0.01 mm was used. The magnetic scale is shown in Figure D4.

The zero altitude level was chosen for all end-plates at the intersection of the web and the compression flange, as shown in Figure D5.

Test series I

On each end-plate eleven contour-lines in uniform distances were designated. Figure D5 shows the positions of the measured contour-lines and the identification codes.

The presented surfaces were determined by linear interpolation between the contour-lines.

Fig. D5 Measured lines and identification codes.
1.) End-plate type I
Test specimen TB2

Fig. D6 Deformed end-plate contour-lines.

Fig. D7 The shapes of the deformed end-plates.

Test specimen TB6

Fig. D8 Deformed end-plate contour-lines.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix D  L. Katula

Budapest University of Technology and Economics Department of Structural Engineering

Fig. D9 The shapes of the deformed end-plates.

Test specimen TB10

Fig. D10 Deformed end-plate contour-lines.

Fig. D11 The shapes of the deformed end-plates.
2.) End-plate type II
Test specimen TB3

Fig. D12 Deformed end-plate contour-lines.

Fig. D13 The shapes of the deformed end-plates.

Test specimen TB7

Fig. D14 Deformed end-plate contour-lines.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Fig. D15 The shapes of the deformed end-plates.

Test specimen TB11

Fig. D16 Deformed end-plate contour-lines.

Fig. D17 The shapes of the deformed end-plates.
3.) End-plate type III
Test specimen TB4

Fig. D18 Deformed end-plate contour-lines.

Fig. D19 The shapes of the deformed end-plates.

Test specimen TB8

Fig. D20 Deformed end-plate contour-lines.
Fig. D21 The shapes of the deformed end-plates.

Test specimen TB12

Fig. D22 Deformed end-plate contour-lines.

Fig. D23 The shapes of the deformed end-plates.
4.) End-plate type IV
Test specimen TB5

Test specimen TB9

Fig. D24 Deformed end-plate contour-lines.

Fig. D25 The shapes of the deformed end-plates.

Fig. D26 Deformed end-plate contour-lines.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix D

L. Katula

Budapest University of Technology and Economics Department of Structural Engineering

Fig. D27 D The shapes of the deformed end-plates.

Test specimen TB13

Fig. D28 Deformed end-plate contour-lines.

Fig. D29 The shapes of the deformed end-plates.
Test series II

The shape of the end-plates was measured before and after the tests. For all end-plates, the zero altitude level was chosen in the line of the web, on the border with the end-plate, i.e. right below the compression flange.

![Diagram](image)

Fig. D30 Measured lines and identification on the diagrams.

Figure D30 shows the positions of the measured contour-lines and the identification for the following contour-lines diagrams. The contour-lines were measured over the web, at the end-plate border, in the axis of the bolts and at uniform distances in-between.
Test specimen TA

Fig. D31 Deformed end-plate contour-lines before test.

Fig. D32 The shapes of the end-plates before test.

Fig. D33 Deformed end-plate contour-lines after test.

Fig. D34 The shapes of the deformed end-plates after test.
The short dashed contour-lines were measured on the end-plate before the test, the solid lines after.

Fig. D35 Contour-lines before and after the test.

Fig. D36 Difference between the contour-lines as measured before and after test.

Fig. D37 Difference between the shapes as measured before and after test.
Test specimen TB

Fig. D38 Deformed end-plate contour-lines before test.

Fig. D39 The shapes of the end-plates before test.

Fig. D40 Deformed end-plate contour-lines after test.

Fig. D41 The shapes of the deformed end-plates after test.
Fig. D42 Contour-lines before and after the test.

Fig. D43 Difference between the contour-lines as measured before and after test.

Fig. D44 Difference between the shapes as measured before and after test.
Test specimen TC

Fig. D45 Deformed end-plate contour-lines before test.

Fig. D46 The shapes of the end-plates before test.

Fig. D47 Deformed end-plate contour-lines after test.

Fig. D48 The shapes of the deformed end-plates after test.
Fig. D49 Contour-lines before and after the test.

Fig. D50 Difference between the contour-lines as measured before and after test.

Fig. D51 Difference between the shapes as measured before and after test.
Test specimen TD

Fig. D52 Deformed end-plate contour-lines before test.

Fig. D53 The shapes of the end-plates before test.

Fig. D54 Deformed end-plate contour-lines after test.

Fig. D55 The shape of the deformed end-plate after test.
Fig. D56 Contour-lines before and after the test.

Fig. D57 Difference between the contour-lines as measured before and after test.

Fig. D58 Difference between the shapes as measured before and after test.
Test specimen TE

Fig. D59 Deformed end-plate contour-lines before test.

Fig. D60 The shapes of the end-plates before test.

Fig. D61 Deformed end-plate contour-lines after test.

Fig. D62 The shapes of the deformed end-plates after test.
Fig. D63 Contour-lines before and after the test.

Fig. D64 Difference contour-lines between the measured lines before and after test.

Fig. D65 Difference between the shapes as measured before and after test.
Test specimen TF

Fig. D66 Deformed end-plate contour-lines before test.

Fig. D67 The shapes of the end-plates before test.

Fig. D68 Deformed end-plate contour-lines after test.

Fig. D69 The shapes of the deformed end-plates after test.
Fig. D70 Contour-lines before and after the test.

Fig. D71 Difference between the contour-lines as measured before and after test.

Fig. D72 Difference between the shapes as measured before and after test.
Load cell calibrations

The load cells were calibrated before test series I. The calibration load was 250 kN. The theoretical load capacity of the studied bolts (M20, 8.8) was $245 \times 800 = 196$ kN.

After test series I and before test series II the load cells were re-calibrated with a load of 300 kN. The theoretical load capacity of the studied bolts (M20, 10.9) was $245 \times 1,000 = 245$ kN.

In each load cell 4 strain gauges were arranged in the form of a halfbridge, which measures the strains. From the measured strains the stress and the load were calculated.

The following diagrams show the calibration curve of the used cells. The load and the output data from the cell were measured in mV/V. The diagrams show the measured data and the regression curve (regression line), which was used to convert the measured data to loads.

The calculated regression lines demonstrate that the used load cells have good elastic behaviour in the applied load-range.

The use of the load cells slightly modifies the bolt load distribution, because the application of the cells enlarges the elongation lengths of the bolts. This effect modifies the stiffness of the joint to a small extent and has minimal effect on the measured bolt-loads.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix E

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Fig. E5 Calibration curve of load cell 2.

Fig. E6 Re-calibration curve of load cell 2.

Fig. E7 Calibration curve of load cell 3.

Fig. E8 Re-calibration curve of load cell 3.

Fig. E9 Calibration curve of load cell 4.

Fig. E10 Re-calibration curve of load cell 4.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix E

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Fig. E11 Calibration curve of load cell 5.

Fig. E12 Re-calibration curve of load cell 5.

Fig. E13 Calibration curve of load cell 8.

Fig. E14 Re-calibration curve of load cell 8.

Fig. E15 Calibration curve of load cell 9.

Fig. E16 Re-calibration curve of load cell 9.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix E

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Fig. E17 Calibration curve of load cell 11.

Fig. E18 Re-calibration curve of load cell 11.

Fig. E19 Calibration curve of load cell 12.

Fig. E20 Re-calibration curve of load cell 12.

Fig. E21 Calibration curve of load cell 13.

Fig. E22 Re-calibration curve of load cell 13.
Fig. E23 Calibration curve of load cell 14.

Fig. E24 Re-calibration curve of load cell 14.

Fig. E25 Calibration curve of load cell 15.

Fig. E26 Re-calibration curve of load cell 15.

Fig. E27 Calibration curve of load cell 16.

Fig. E28 Re-calibration of curve load cell 16.
Appendix E L. Katula

Budapest University of Technology and Economics Department of Structural Engineering

Fig. E29 Calibration curve of load cell 17.

Fig. E30 Calibration curve of load cell 18.

Fig. E31 Calibration curve of load cell 19.

Fig. E32 Calibration curve of load cell 20.
Summary of the Eurocode 3 model

Introduction

It is well known that the rotational stiffness of the structural joints is between the two extreme cases of the rigid and the pinned behaviour.

The joint is classified as rigid when all parts of the joint have sufficient rotational stiffness (i.e. their stiffness can be assumed to be infinite) and the rotations of each connected member of the joint are identical to each other, see Figure F1a. Rigid-body type rotation can be observed in the joint, called as nodal rotation in the commonly used analysis methods of frame structures.

In the case of nominally pinned joints, it is assumed that the joint has no stiffness and the beam works as a simply supported system, independently from the behaviour of any other connected member, as shown in Figure F1b.

For the intermediate case (when the stiffness is neither zero nor infinite) the joint is classified as semi-rigid, and the transferred moment is a result of the different absolute rotations of the two connected members, as shown in Figure F1c.

The simplest way to represent the rotational stiffness of the joint is a rotational (spiral) spring between the ends of the two connected members. The rotational stiffness "S" of this spring is the parameter which describes the relationship between the transformed moment $M_j$ and the relative rotational difference ($\phi$) between the two connected members.

When this rotational stiffness is equal to zero ($S = 0$), or when it has a relatively small value, the joint belongs to the class of nominally pinned joints. In the opposite case, when the rotational stiffness is infinite or when it is relatively large, the joint is classified as rigid.

In all the intermediate cases, the joint belongs to the semi-rigid class.

In the case of semi-rigid joints the external loads cause both a bending moment ($M_j$) and a relative rotational difference ($\phi$) between the connected members. The moment and the relative rotation are related to each other through a constitutive law, which depends on the joint properties, as it is illustrated in Figure F2, where, for simplification, the global analysis is assumed to be performed assuming linear elastic behaviour.

The Eurocode 3 (EC3) requirements and the desire to model the behaviour of the structure in a more realistic way lead to the consideration of the semi-rigid behaviour, when it is necessary.

At the global analysis stage, if we take into consideration the semi-rigid nature of the joints - instead of the rigid or pinned joint behaviour - it causes a modification not only of the displacements, but also of the distribution of the internal forces within the structure.
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix F

L. Katula

Budapest University of Technology and Economics Department of Structural Engineering

Fig. F2 Modelling of joints (elastic global analysis).

It is to be noted that the concept of rigid and pinned joints is still nevertheless included in the EC3. It is acceptable in the design process that a joint which is not fully rigid or pinned is indeed considered as fully rigid or pinned. The decision whether a joint can be considered as rigid, semi-rigid or pinned, depends on the comparison of the joint stiffness and the beam stiffness (which in turn depends on the second moment of area and the length of the beam).

Definitions of joint configuration, joint and connection

Building frames consist of beams and columns, usually made of H or I sections, which are assembled together by means of connections. These connections are between two beams, two columns, a beam and a column or a column and the foundation. The possible connection types are illustrated in Figure F3.

Fig. F3 Different types of connections in a building frame.

A connection is defined as the set of the physical components which mechanically fasten the connected elements.

When the connection as well as the corresponding zone of interaction between the connected members are considered together, the wording joint is used, as shown in Figure F5.
Depending on the number of in-plane elements connected together, single-sided and double-sided joint configurations are defined, as shown in Figure F4. In a double-sided configuration two joints, left and right, have to be considered.

The definitions illustrated in Figure F4 and F5 are also valid for other joint configurations and connection types.

As explained previously, the joints which are traditionally considered as rigid or pinned and are designed accordingly, possess, in reality, their own degree of finite flexibility or finite stiffness resulting from the deformability of all the constitutive components.

Classification of joints

Stiffness classification

The stiffness classification into rigid, semi-rigid and pinned joints is performed by comparing simply the design joint stiffness to the two stiffness boundaries (Figure F6). The stiffness boundaries have been derived so as to allow a direct comparison with the initial design joint stiffness, whatever type of joint idealization is used afterwards in the analysis.
**Strength classification**

The strength classification simply consists of comparing the joint design moment resistance to the boundaries of the "full-strength" and "pinned" behaviour (Figure 7).

![Strength classification diagram](image)

**Fig. F7 Strength classification boundaries.**

**Boundaries for classification**

It is worthwhile to stress that a classification based on the experimental M-ϕ characteristics of a joint is not allowed, as only design properties are of concern.

The stiffness and strength boundaries for the joint classification are given as follows:

**Classification by stiffness**

- **Rigid joint**
  
  \[ S_{j,ini} \geq 25 \frac{EI}{L} \] (unbraced frames)
  
  \[ S_{j,ini} \geq 8 \frac{EI}{L} \] (braced frames)

- **Semi-rigid joint**
  
  \[ 0,5 \frac{EI}{L} < S_{j,ini} < 25 \frac{EI}{L} \] (unbraced frames)
  
  \[ 0,5 \frac{EI}{L} < S_{j,ini} < 8 \frac{EI}{L} \] (braced frames)

- **Pinned joint**
  
  \[ S_{j,ini} \leq 0,5 \frac{EI}{L} \]

**Classification by strength**

- **Full-strength joint**
  
  \[ M_{j,Rd} \geq M_{\text{full-strength}} \]

- **Partial strength joint**
  
  \[ 0,25 M_{\text{full-strength}} < M_{j,Rd} < M_{\text{full-strength}} \]

- **Pinned joint**
  
  \[ M_{j,Rd} \leq 0,25 M_{\text{full-strength}} \]

where

- \( E \) = elastic modulus
- \( I \) = the second moment of area of the member
- \( L \) = the system length of the member
- \( M_{\text{full-strength}} \) = the design moment resistance of the weaker of the connected members

**Joint modelling**

Joint behaviour affects the structural frame response and shall therefore be modelled, just as for beams and columns, for the frame analysis and design. Traditionally, the following types of joint modelling are considered:

**For rotational stiffness:**

- rigid
- pinned

**For resistance:**

- full-strength
- partial-strength
- pinned
When the joint rotational stiffness is of concern, the wording rigid means that no relative rotation occurs between the connected members whatever is the applied moment. The wording pinned assumes the existence of a perfect hinge between the members. Indeed rather flexible but not fully pinned joints and rather stiff but not fully rigid joints may be considered as effectively pinned and sufficiently rigid, respectively.

For joint resistance, a full-strength joint is stronger than the weaker of the connected members, which is in contrast to a partial-strength joint. In the everyday practice, partial-strength joints are used whenever the joints are designed to transfer the internal forces but not to resist the full capacity of the connected members. A pinned joint is considered to transfer no moment.

Consideration of rotational stiffness and joint resistance properties leads to three significant joint models:
- rigid & full-strength;
- rigid & partial-strength;
- pinned.

However, as far as the joint rotational stiffness is considered, joints designed for economy may be neither rigid nor pinned but semi-rigid. There are thus new possibilities for joint modelling:
- semi-rigid & full-strength;
- semi-rigid & partial-strength.

As a simplification we can introduce three joint models (Table F1):

<table>
<thead>
<tr>
<th>stiffness</th>
<th>resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>rigid</td>
<td>full-strength</td>
</tr>
<tr>
<td>semi-rigid</td>
<td>partial-strength</td>
</tr>
<tr>
<td>pinned</td>
<td>pinned</td>
</tr>
</tbody>
</table>

| continuous   | covering the rigid/full-strength case only, i.e. the joint ensures a full rotational continuity between the connected members |
| semi-continuous | covering the rigid/partial-strength, the semi-rigid/full-strength and the semi-rigid/partial-strength cases, i.e. the joint ensures only a partial rotational continuity between the connected members |
| simple       | covering the pinned case only, i.e. the joint prevents any rotational continuity between the connected members |

The interpretation to be given to these wordings depends on the type of frame analysis to be performed. In the case of an elastic global frame analysis, only the stiffness properties of the joint are relevant for the joint modelling. In the case of a rigid-plastic analysis, the main joint feature is the resistance. In all the other cases, both the stiffness and resistance properties govern the manner in which the joints should be modelled. These possibilities are illustrated in Table F2.

<table>
<thead>
<tr>
<th>modelling</th>
<th>type of frame analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td>continuous</td>
<td>rigid-full-strength</td>
</tr>
<tr>
<td>semi-continuous</td>
<td>rigid/partial-strength, semi-rigid/full-strength</td>
</tr>
<tr>
<td>simple</td>
<td>pinned</td>
</tr>
</tbody>
</table>

The interpretation to be given to these wordings depends on the type of frame analysis to be performed. In the case of an elastic global frame analysis, only the stiffness properties of the joint are relevant for the joint modelling. In the case of a rigid-plastic analysis, the main joint feature is the resistance. In all the other cases, both the stiffness and resistance properties govern the manner in which the joints should be modelled. These possibilities are illustrated in Table F2.
Joint characterization

An important step when designing a frame consists of the characterization of the rotational response of the joints, i.e. the evaluation of the mechanical properties in terms of stiffness, strength and ductility. Three main approaches may be followed:

- experimental,
- numerical,
- analytical.

The only practical option for the designer is the analytical approach. Analytical procedures have been developed which enable a prediction of the joint response based on the knowledge of the mechanical and geometrical properties of the joint components, termed component method. It applies to any type of steel or composite joints, whatever is the geometrical configuration, the type of loading (axial force and/or bending moment, ...) and the type of member sections.

Introduction to the component method

The component method considers any joint as a set of individual basic components. The components are the following:

Compression zone:
- column web in compression;
- beam flange and web in compression;

Tension zone:
- column web in tension;
- column flange in bending;
- bolts in tension;
- end-plate in bending;
- beam web in tension;

Shear zone:
- column web panel in shear.

Each of these basic components possesses its own strength and stiffness either in tension or in compression or in shear. The column web is subject to coincident compression, tension and shear. This co-existence of several components within the same joint element can obviously lead to stress interactions that are likely to decrease the resistance of the individual basic components. To derive the mechanical properties of the whole joint from those of all the individual constituent components requires a preliminary distribution of the forces acting on the joint into internal forces acting on the components in a way that satisfies equilibrium.

The application of the component method requires the following steps:

- identification of the active components in the joint being considered;
- evaluation of the stiffness and/or resistance characteristics for each individual basic component;
- assembly of all the constituent components and evaluation of the stiffness and/or resistance characteristics of the whole joint.

The application of the component method requires a sufficient knowledge of the behaviour of the basic components. Those covered by EC3 are listed in Table F3.

The combination of these components allows one to cover a wide range of joint configurations, which should be sufficient to satisfy the needs of practitioners as far as beam-to-column joints and beam splices in bending are concerned.
Table F3/a List of components covered by Eurocode 3 (EC3 EN 1993-1-8: 2005).

<table>
<thead>
<tr>
<th>Component</th>
<th>Design resistance</th>
<th>Stiffness coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Column web pane in shear</td>
<td>V_{Ed}</td>
<td>6.2.6.1 6.3.2</td>
</tr>
<tr>
<td>2. Column web in transverse</td>
<td>F_{c,Ed}</td>
<td>6.2.6.2 6.3.2</td>
</tr>
<tr>
<td>compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Column web in transverse</td>
<td>F_{LEd}</td>
<td>6.2.6.3 6.3.2</td>
</tr>
<tr>
<td>tension</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Column flange in bending</td>
<td>F_{LEd}</td>
<td>6.2.6.4 6.3.2</td>
</tr>
<tr>
<td>5. End-plate in bending</td>
<td>F_{LEd}</td>
<td>6.2.6.5 6.3.2</td>
</tr>
<tr>
<td>6. Flange cleat in bending</td>
<td>F_{LEd}</td>
<td>6.2.6.6 6.3.2</td>
</tr>
</tbody>
</table>
### Table 3/b List of components covered by Eurocode 3 (EC3 EN 1993-1-8: 2005).

<table>
<thead>
<tr>
<th>Component</th>
<th>Design resistance</th>
<th>Stiffness coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>7. Beam or column flange and web in compression</td>
<td></td>
<td>6.2.6.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.3.2</td>
</tr>
<tr>
<td>8. Beam web in tension</td>
<td></td>
<td>6.2.6.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.3.2</td>
</tr>
<tr>
<td>9. Plate in tension or compression</td>
<td></td>
<td>In tension: EN 1993-1-1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>In compr.: EN 1993-1-1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.3.2</td>
</tr>
<tr>
<td>10. Bolts in tension</td>
<td></td>
<td>With column flange: 6.2.6.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>with end-plate 6.2.6.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>with flange cleat: 6.2.6.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.3.2</td>
</tr>
<tr>
<td>11. Bolts in shear</td>
<td></td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.3.2</td>
</tr>
<tr>
<td>12. Bolts in bearing (on beam flange, column flange, end-plate or cleat)</td>
<td></td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.3.2</td>
</tr>
</tbody>
</table>
Joint idealization

The non-linear behaviour of the isolated flexural spring which characterizes the actual joint response does not lend itself towards everyday design practice. However the moment-rotation characteristic curve may be idealized without significant loss of accuracy. One of the most simple idealizations possible is the elastic-perfectly plastic relationship. This modelling has the advantage of being quite similar to that used traditionally for the modelling of member cross-sections subject to bending.

The initial stiffness $S_{j,ini}$ is derived from the elastic stiffness of the components. The elastic behaviour of each component is represented by an extensional spring. The force-deformation relationship of this spring is given by:

$$ F_i = k_i E \Delta_i $$

where:
- $F_i$ is the force in the spring $i$;
- $k_i$ is the stiffness coefficient of the component $i$;
- $E$ is the Young modulus;
- $\Delta_i$ is the deformation of the spring $i$.

The spring components in a joint are combined into a spring model. As an example the spring model for an unstiffened welded beam-to-column joint is shown in Figure F8.

Fig. F8 Spring model for an unstiffened welded joint.

The force in each spring is equal to $F$. The moment $M$ acting on the spring model is equal to $F \cdot z$, where $z$ is the distance between the centre of tension (for welded joints, located in the centre of the upper beam flange) and the centre of compression (for welded joints, located in the centre of the lower beam flange). The rotation $\phi$ in the joint is equal to $(\Delta_1 + \Delta_2 + \Delta_4) / z$. In other words:

$$ S_{i,ini} = \frac{M}{\phi} = \frac{Fz^2}{\Sigma \Delta_i / z} = \frac{Fz^2}{E \Sigma \frac{1}{k_i}} = \frac{Ez^2}{\Sigma \frac{1}{k_i}} $$
The spring model adopted for end-plate joints with two or more bolt-rows in tension is shown in Figure F9. It is assumed that the bolt-row deformations for all rows are proportional to the distance to the point of compression, but that the elastic forces in each row are dependent on the stiffness of the components.

![Spring model for a beam-to-column end-plate joint with more than one bolt-row in tension.](image)

Strength assembly

For the connection represented in Figure F10 the distribution of internal forces is quite easy to obtain: the compressive force is transferred at the centroid of the beam flange, while the tension force is at the level of the upper bolt-row. The resistance possibly associated with the lower bolt-row is usually neglected as it contributes in a very modest way to the transfer of bending moment in the joint (small level arm).

![Joint with one bolt-row in tension.](image)

The design resistance of the joint $M_{j,Rd}$ is associated with the design resistance $F_{Rd}$ of the weakest joint component which can be one of the following:
- the beam and web in compression,
- the beam web in tension,
- the plate in bending or
- the bolts in tension.

For the two last components (plate and bolts), reference is made to the concept of "idealized T-stub" introduced in EC3. The bending resistance becomes:

$$M_{j,Rd} = F_{Rd} \cdot z$$

where $z = h$ = the lever arm.

When more than one bolt-row has to be considered in the tension zone (Figure F11), the distribution of internal forces is more complex.
Assume, initially, that the design of the joint leads to the adoption of a particularly thick end-plate in comparison to the bolt diameter – see Figure F12. In this case, the distribution of internal forces between the different bolt-rows is linear according to the distance from the centre of compression. The compression force $F_c$ which equilibrates the tension forces acts at the level of the centroid of the lower beam flange.

The design resistance $M_{j,Rd}$ of the joint is reached as soon as the bolt-row subjected to the highest stresses - in reality that which is located the farthest from the centre of compression - reaches its design resistance in tension $2B_tRd$.

Because of a limited deformation capacity of the bolts in tension no redistribution of forces is allowed to take place between bolt-rows.

It is assumed here that the design resistance of the beam flange and web in compression is sufficient to transfer the compression force $F_c$. The tensile resistance of the beam web is also assumed not to limit the design resistance of the joint. $M_{j,Rd}$ is so expressed as:

$$M_{j,Rd} = \frac{F_{Rd}}{h_i} \sum h_i^2$$

For thinner end-plates, the distribution of internal forces requires much more attention. When an initial moment is applied to the joint, the forces distribute between the bolt-rows according to the relative stiffnesses of the T-stubs. This stiffness is namely associated to that of the part of the end-plate adjacent to the considered bolt-row. In the particular case of Figure F13, the upper bolt-row is characterized by a higher stiffness because of the presence of the beam flange and the web welded to the end-plate.

Because of the higher stiffness, the upper bolt-row is capable of transferring a higher load than the lower bolt-rows – illustrated in Figure F13/b.
The design resistance of the upper bolt-row may be associated with one of the following components:

- the end-plate only (Mode 1),
- the bolts-plate assembly (Mode 2) or
- the bolts only (Mode 3),
- the beam web in tension.

If its failure mode is ductile, a redistribution of forces between the bolt-rows can take place: as soon as the upper bolt-row reaches its design resistance, any additional bending moment applied to the joint will be carried by the lower bolt-rows, each of which in their turn may reach its own design resistance.

The failure of a T-stub may occur in three different ways as shown Table F4.

**Table 4 Failure modes of a T-stub.**
The plastic redistribution of the internal forces extends to all bolt-rows when they have sufficient deformation capacity.

The design moment resistance \( M_{j,Rd} \) is expressed as - see Figure F14:

\[
M_{j,Rd} = \sum_i F_{Rd,i} h_i
\]

Fig. F14 Plastic distribution of internal forces.

The plastic forces \( F_{Rd,i} \) vary from one bolt-row to another according to the failure modes.

EC3 considers that a bolt-row possesses a sufficient deformation capacity to allow a plastic redistribution of internal force to take place when:

- \( F_{Rd,i} \) is associated to the failure of the beam web in tension;
- \( F_{Rd,i} \) is associated to the failure of the T-sub and:

\[
F_{Rd,i} \leq 1.9 B_{t,Rd}
\]

The plastic redistribution of forces is interrupted because of the lack of deformation capacity in the last bolt-row \( k \) which has reached its design resistance.

In the bolt-rows located lower than bolt-row \( k \), the forces are then linearly distributed according to their distance to the point of compression (Figure F15).

\[
M_{j,Rd} = \sum_{i=1,k} F_{Rd,j} h_i + \frac{F_{Rd,k}}{h_k} \sum_{j=k+1,n} h_j^2
\]

Fig. F15 Elasto-plastic distribution of internal forces.

where: \( n \) is the total number of bolt-rows; \( k \) is the number of the bolt-row, the deformation capacity of which is not sufficient.

In this case, the distribution is "elasto-plastic".

The plastic or elasto-plastic distribution of internal forces is interrupted because the compression force \( F_c \) attains the design resistance of the beam flange and web in compression. The moment resistance \( M_{j,Rd} \) is evaluated with the formula

\[
M_{j,Rd} = \sum_{i=1,k} F_{Rd,j} h_i + \frac{F_{Rd,k}}{h_k} \sum_{j=k+1,n} h_j^2
\]

In which, obviously, only a limited number of bolt-rows are taken into consideration. These bolt rows are such that:

\[
\sum_{i=1,n} F_i = F_{c,Rd}
\]
where: $m$ is the number of the last bolt-row transferring a tensile force;
$F_t$ is the tensile force in bolt-row number $t$;
$F_{c,Rd}$ is the design resistance of the beam flange and web in compression.

The application of the above-described principles to beam-to-column joints is quite similar. The design moment resistance $M_{Rd}$ is, as for the beam splices, likely to be limited by the resistance of:
- the end-plate in bending,
- the bolts in tension,
- the beam web in tension,
- the beam flange and web in compression,
but also by that of:
- the column web in tension,
- the column flange in bending,
- the column web in compression,
- the column web panel in shear.

To each of these mechanisms are associated specific design resistances.

The resistance, which can be evaluated for a certain bolt row assuming the failure of the group of fasteners, is always smaller than the resistance of that certain bolt row in single failure mode.

Table F5 shows examples of yield line patterns for individual bolt-row failure and for the failure of groups of bolt-rows.

<table>
<thead>
<tr>
<th>a) Individual:</th>
<th>$l_{eff,1} = 4,m + 1,25,e$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Individual Mechanism" /></td>
<td><img src="image2" alt="Individual Mechanism" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>b) Group mechanism:</th>
<th>$l_{eff,g} = 4,m + 1,25,e + 2,p$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image3" alt="Group Mechanism" /></td>
<td><img src="image4" alt="Group Mechanism" /></td>
</tr>
</tbody>
</table>
Classification of cross-section according to EN 1993-1-1

**Input data:**

- **material properties:** $S_{355}$, $f_y = 355 \text{ N/mm}^2$, $E = 210000 \text{ N/mm}^2$, $\nu = 0.3$, $G = \frac{E}{2(1 + \nu)}$
- **partial factor:** $\gamma_{M.0} = 1.0$
- **loads:** $M_{Ed} = 100 \text{kN\,m}$, $N_{Ed} = 0 \text{kN}$, $V_{Ed} = 0 \text{kN}$

The dimensions of the welds are in this calculation neglected!

**Dimensions:**

- $a := 360 \text{mm}$, $e := 360 \text{mm}$, $x := 0 \text{mm}$
- $b := 20 \text{mm}$, $f := 20 \text{mm}$
- $c := 0 \text{mm}$, $g := 860 \text{mm}$
- $d := 0 \text{mm}$, $h := 8 \text{mm}$
- $g_1 := \frac{x - d}{2}$, $g_1 = 0 \text{mm}$
- $g_2 := g - x - \frac{d}{2}$, $g_2 = 860 \text{mm}$
- $h_b := b + g + f$, $h_b = 900 \text{mm}$

$A_n := a \cdot b + (c - h) \cdot d + e \cdot f + g \cdot h$ 
$A_n = 212.8 \text{ cm}^2$

$S_y_n := (a \cdot b) - \frac{b}{2} + (c - h) \cdot d \cdot x + e \cdot f \left( g + \frac{f}{2} \right) + g_1 \cdot h \cdot \frac{g_1}{2} + g_2 \cdot h \left( x + \frac{d}{2} + \frac{g_2}{2} \right)$

$S_y_n = 9150.4 \text{ cm}^3$

$\text{sy}_n := \frac{S_y_n}{A_n}$, $\text{sy}_n = 430 \text{ mm}$

$y_2_n := g + f - \text{sy}_n$, $y_2_n = 450 \text{ mm}$

$y_{1,n} := g + 2 \cdot f - y_2_n$, $y_{1,n} = 450 \text{ mm}$

$y_{\max,n} := \max(\text{sy}_n, y_2_n)$, $y_{\max,n} = 450 \text{ mm}$

$S_{y,n,\text{half}} := a \cdot b \left( y_{1,n} - \frac{b}{2} \right) + h \cdot \frac{\text{sy}_n}{2}$, $S_{y,n,\text{half}} = 3907.6 \text{ cm}^3$

$I_{y,n} := a \cdot \frac{b^3}{12} + a \cdot b \left( \frac{b^2}{2} + c \cdot \frac{d^3}{12} + c \cdot d \cdot x^2 + e \cdot \frac{f^3}{12} + e \cdot f \left( g + \frac{f}{2} \right) + h \cdot \frac{g_1^3}{12} + g_1 \cdot h \left( \frac{g_1}{2} \right)^2 + h \cdot \frac{g_2^3}{12} + g_2 \cdot h \left( x + \frac{d}{2} + \frac{g_2}{2} \right)^2 - A_n \cdot \frac{\text{sy}_n^2}{2}$

$I_{y,n} = 321235.733 \text{ cm}^4$

$W_{y,n} := \frac{I_{y,n}}{y_{\max,n}}$, $W_{y,n} = 7138.572 \text{ cm}^3$

$W_{y,\text{pl},n} := 2 \cdot S_{y,n,\text{half}}$, $W_{y,\text{pl},n} = 7815.2 \text{ cm}^3$

$A_v := g \cdot h$, $A_v = 68.8 \text{ cm}^2$
Cross-section under compression (N)

classification

upper flange:
\[
\varepsilon := \frac{235 - \frac{N}{\text{mm}^2}}{f_y}\]
\[
\varepsilon = 0.814 \quad C := \frac{a}{2} \quad C = 18 \text{ cm} \quad \frac{C}{b} = 9
\]

\[
\text{upper_flange}_N :=
\begin{cases}
1 & \text{if } \frac{C}{b} \leq 9 \cdot \varepsilon \\
2 & \text{if } 9 \cdot \varepsilon < \frac{C}{b} \leq 10 \cdot \varepsilon \\
3 & \text{if } 10 \cdot \varepsilon < \frac{C}{b} \leq 14 \cdot \varepsilon \\
4 & \text{otherwise}
\end{cases}
\]

\text{upper_flange}_N = 3 \quad \text{class 3}

web plate:
\[
\frac{g}{h} = 107.5
\]

\[
\text{web_plate}_N :=
\begin{cases}
1 & \text{if } \frac{g}{h} \leq 33 \cdot \varepsilon \\
2 & \text{if } 33 \cdot \varepsilon < \frac{g}{h} \leq 38 \cdot \varepsilon \\
3 & \text{if } 38 \cdot \varepsilon < \frac{g}{h} \leq 42 \cdot \varepsilon \\
4 & \text{otherwise}
\end{cases}
\]

\text{web_plate}_N = 4 \quad \text{class 4}

lower flange:
\[
\varepsilon := \frac{235 - \frac{N}{\text{mm}^2}}{f_y}\]
\[
\varepsilon = 0.814 \quad C := \frac{e}{2} \quad C = 18 \text{ cm} \quad \frac{C}{f} = 9
\]

\[
\text{lower_flange}_N :=
\begin{cases}
1 & \text{if } \frac{C}{f} \leq 9 \cdot \varepsilon \\
2 & \text{if } 9 \cdot \varepsilon < \frac{C}{f} \leq 10 \cdot \varepsilon \\
3 & \text{if } 10 \cdot \varepsilon < \frac{C}{f} \leq 14 \cdot \varepsilon \\
4 & \text{otherwise}
\end{cases}
\]

\text{lower_flange}_N = 3 \quad \text{class 3}

cross_section_compression := \max(\text{upper_flange}_N, \text{web_plate}_N, \text{lower_flange}_N)

cross_section_compression = 4 \quad \text{class 4}
**Cross-section under bending (M)**

**classification**

upper flange is under tension $\rightarrow$ upper\_flange\_M := 1 class 1

web plate: $\frac{g}{h} = 107.5$

\[
\text{web\_plate\_M} :=
\begin{cases} 
 1 & \text{if } \frac{g}{h} \leq 72 \cdot \varepsilon \\
 2 & \text{if } 72 \cdot \varepsilon < \frac{g}{h} \leq 83 \cdot \varepsilon \\
 3 & \text{if } 83 \cdot \varepsilon < \frac{g}{h} \leq 124 \cdot \varepsilon \\
 4 & \text{otherwise}
\end{cases}
\]

web\_plate\_M = 4 class 4

lower flange is under compression. $\rightarrow$ lower\_flange\_M := 3 class 3

cross\_section\_bending := \max(upper\_flange\_M, web\_plate\_M, lower\_flange\_M)

\[
\text{cross\_section\_bending} = 4 \text{ class 4}
\]

**Resistances for class 1 and 2 cross-sections**

**shear:**

\[
\eta := 1.2 \quad A_y := \eta \cdot g \cdot h
\]

\[
V_{pl.Rd.I} := \frac{A_y f_y}{2\sqrt{3} \cdot \gamma_{M.0}} \quad V_{pl.Rd.I} = 846.072 \text{ kN}
\]

**bending:**

\[
M_{c.Rd.I} := \frac{W_{y.pl.n} f_y}{\gamma_{M.0}} \quad M_{c.Rd.I} = 2774.396 \text{ kN m}
\]

**compression / tension:**

\[
N_{c.Rd.I} := \frac{A_n f_y}{\gamma_{M.0}} \quad N_{c.Rd.I} = 7554.4 \text{ kN}
\]

**Resistances for class 3 cross-sections**

**bending:**

\[
M_{c.Rd.III} := \frac{W_{y.n} f_y}{\gamma_{M.0}} \quad M_{c.Rd.III} = 2534.193 \text{ kN m}
\]

**compression:**

\[
N_{c.Rd.III} := \frac{A_n f_y}{\gamma_{M.0}} \quad N_{c.Rd.III} = 7554.4 \text{ kN}
\]
Effective cross-section under compression (N)

**upper flange:**

\[
B := \frac{a}{2} - \frac{h}{2} \quad t := b \quad B = 176 \text{ mm} \quad \psi := 1 \quad k_\sigma := 0.43
\]

\[
\lambda_p := \frac{B}{t} = 28.4 \sqrt{k_\sigma} \quad \lambda_p = 0.581
\]

\[
\rho := \begin{cases} 
\frac{\lambda_p - 0.188}{\lambda_p^2} & \text{if } \frac{\lambda_p - 0.188}{\lambda_p^2} > 1 \\
1, \frac{\lambda_p - 0.188}{\lambda_p^2} & \text{otherwise}
\end{cases} \quad \rho = 1
\]

\[
b_{\text{eff}} := \rho \cdot B \quad b_{\text{eff}} = 176 \text{ mm} \quad a := 2 \cdot b_{\text{eff}} + h
\]

\[
a := \begin{cases} 
\text{if } \text{upper\_flange\_N} < 4, a_n, a \end{cases} \quad a = 360 \text{ mm}
\]

**web plate:**

\[
B := g \quad t := h \quad \psi := 1 \quad k_\sigma := 4.0
\]

\[
\lambda_p := \frac{B}{t} = 28.4 \sqrt{k_\sigma} \quad \lambda_p = 2.326
\]

\[
\rho := \begin{cases} 
\frac{\lambda_p - 0.055 (3 + \psi)}{\lambda_p^2} & \text{if } \frac{\lambda_p - 0.055 (3 + \psi)}{\lambda_p^2} > 1 \\
1, \frac{\lambda_p - 0.055 (3 + \psi)}{\lambda_p^2} & \text{otherwise}
\end{cases} \quad \rho = 0.389
\]

\[
b_{\text{eff}} := \rho \cdot B \quad b_{\text{eff}} = 334.742 \text{ mm}
\]

\[
g := \frac{b_{\text{eff}}}{2} \quad g = 167.371 \text{ mm}
\]

\[
g := \begin{cases} 
\text{if } \text{web\_plate\_N} < 4, g_n, g \end{cases} \quad g = 167.371 \text{ mm}
\]

\[
k := \frac{b_{\text{eff}}}{2} \quad k = 167.371 \text{ mm}
\]

\[
k := \begin{cases} 
\text{if } \text{web\_plate\_N} < 4, k_n, k \end{cases} \quad k = 167.371 \text{ mm}
\]

**lower flange:**

\[
B := \frac{e}{2} - \frac{h}{2} \quad t := 1 \quad \psi := 1 \quad k_\sigma := 0.43
\]

\[
\lambda_p := \frac{B}{t} = 28.4 \sqrt{k_\sigma} \quad \lambda_p = 0.581
\]
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix G L Katula

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\[
\rho := \begin{cases} 
\frac{\lambda - 0.188}{\lambda^2} > 1, & \frac{\lambda - 0.188}{\lambda^2} > 1, \\
1, & 1
\end{cases} \quad \rho = 1
\]

\[
b_{\text{eff}} := \rho \cdot B
\]

\[
e := 2 \cdot b_{\text{eff}} + h
\]

\[
e := \text{if}(\text{lower_flange}_N < 4, e_n, e)
\]

The dimensions of the effective cross-section

\[
a = 360 \text{ mm} \quad b = 20 \text{ mm} \quad x_1 := 0 \text{ mm} \quad x_2 := g_n
\]

\[
e = 360 \text{ mm} \quad f = 20 \text{ mm}
\]

\[
g = 167.371 \text{ mm} \quad h = 8 \text{ mm} \quad k = 167.371 \text{ mm}
\]

\[
i := 0 \text{ mm} \quad j := 0 \text{ mm}
\]

\[
A_{\text{eff},N} := a \cdot b + c \cdot d + e \cdot f + (g + i + j + k) \cdot h
\]

\[
S_{y,\text{eff},N} := \sqrt{(a \cdot b) \cdot b - c \cdot d \cdot x_1 + e \cdot f \cdot \left(x_2 + \frac{f}{2}\right) + g \cdot h \cdot \left(x_1 - \frac{d}{2} - \frac{i}{2}\right) + j \cdot h \cdot \left(x_1 + \frac{d}{2} + \frac{j}{2}\right) + k \cdot h \cdot \left(x_2 - \frac{k}{2}\right)}
\]

\[
S_{y,N} := \frac{S_{y,\text{eff},N}}{A_{\text{eff},N}} \quad s_{y,N} = 430 \text{ mm}
\]

\[
l_{y,N,IV} := a \cdot \frac{b^3}{12} + a \cdot b \cdot \left(\frac{b}{2}\right)^2 + c \cdot \frac{d^3}{12} + c \cdot d \cdot x_1 + e \cdot \frac{f^3}{12} + e \cdot f \cdot \left(x_2 + \frac{f}{2}\right)^2 + g \cdot h \cdot \frac{g}{12} + h \cdot g \cdot \left(\frac{g}{2}\right)^2 + h \cdot \frac{i^3}{12}
\]

\[
y_2 := x_2 + f - s_{y,N} \quad y_2 = 450 \text{ mm}
\]

\[
y_{\text{max},\text{eff},N} := \max(s_{y,N}, y_2) \quad y_{\text{max},\text{eff},N} = 450 \text{ mm}
\]

\[
W_{y,\text{eff},N} := \frac{l_{y,N,IV}}{y_{\text{max},\text{eff},N}} \quad W_{y,\text{eff},N} = 6923.88 \text{ cm}^3
\]

Resistance of the effective cross-section under compression

\[
N_{c,Rd,IV} := \frac{A_{\text{eff},N} \cdot f_y}{\gamma M.0} \quad N_{c,Rd,IV} = 6062.667 \text{ kN}
\]

\[
\text{Budapest University of Technology and Economics Department of Structural Engineering}
\]
Effective cross-section under bending (M)

\[ a := a_n \quad b = 20 \text{ mm} \]
\[ e := e_n \quad f = 20 \text{ mm} \]
\[ g := g_n \quad h = 8 \text{ mm} \]
\[ g_1 := y^2_n - b \quad g_2 := y^1_n - f \]

stresses:

from bending (\(M_{Ed}\))

\[ \sigma_{Mf.f} := \frac{-M_{Ed}}{I_{y,n}} (g_1 + b) \quad \sigma_{Mf.f} = -1.401 \frac{\text{kN cm}^2}{\text{cm}^2} \] (stress in upper flange)

\[ \sigma_{Mf.a} := \frac{M_{Ed}}{I_{y,n}} (g_2 + f) \quad \sigma_{Mf.a} = 1.401 \frac{\text{kN cm}^2}{\text{cm}^2} \] (stress in lower flange)

\[ \sigma_{Mw1} := \frac{-M_{Ed}}{I_{y,n}} (g_1) \quad \sigma_{Mw1} = -1.339 \frac{\text{kN cm}^2}{\text{cm}^2} \] (max. compr. stress in web plate)

\[ \sigma_{Mw2} := \frac{M_{Ed}}{I_{y,n}} (g_2) \quad \sigma_{Mw2} = 1.339 \frac{\text{kN cm}^2}{\text{cm}^2} \] (max. tension stress in web plate)

upper flange:

\[ B := \frac{a}{2} - \frac{h}{2} \quad t := b \quad \psi := 1 \quad k_p := 0.43 \]

\[ \lambda_p := \frac{B}{t} \quad 28.4 \cdot e \sqrt{k_p} \quad \lambda_p = 0.581 \]

\[ \rho := \frac{\lambda_p - 0.188}{\lambda_p^2} \quad \rho = 1.164 \]

\[ \delta := \text{if} (\rho < 1, \rho, 1) \]

\[ b_{eff} := \begin{cases} B & \text{if} \quad \sigma_{Mf.f} \leq 0 \\ \rho \cdot B & \text{otherwise} \end{cases} \quad b_{eff} = 176 \text{ mm} \]

\[ a := 2 \cdot b_{eff} + h \quad a = 360 \text{ mm} \]

\[ a := \text{if} (\text{upper_flange}_M < 4, a_n, 2 \cdot b_{eff} + h) \quad a = 360 \text{ mm} \]

web plate:

\[ B := g \quad t := h \quad \sigma_{Mw2} = 1.339 \frac{\text{kN cm}^2}{\text{cm}^2} \quad \sigma_{Mw1} = -1.339 \frac{\text{kN cm}^2}{\text{cm}^2} \]

\[ \psi := \frac{\sigma_{Mw1}}{\sigma_{Mw2}} \quad \psi = -1 \]
\[k := \begin{cases} 4.0 & \text{if } \psi = 1 \\ 8.2 & \text{if } 1 > \psi > 0 \\ 1.05 + \psi & \text{if } \psi = 0 \\ 7.81 - 6.29 \cdot \psi + 9.78 \cdot \psi^2 & \text{if } 0 > \psi > -1 \\ 23.9 & \text{if } \psi = -1 \\ 5.89 \cdot (1 - \psi)^2 & \text{if } -1 > \psi > -2 \end{cases} \]

\[k_\sigma = 23.9\]

\[\lambda := \frac{B}{t} \sqrt{28.4 \cdot \varepsilon \cdot k_\sigma} \quad \lambda_p = 0.952\]

\[\rho := \frac{\lambda_p - 0.055 \cdot (3 + \psi)}{\lambda_p^2} \quad \rho = 0.929\]

\[\rho := \begin{cases} (\rho < 1, \rho > 1) \end{cases}\]

\[b_c := \frac{B}{1 - \psi} \quad b_c = 430 \text{ mm}\]

\[b_t := B - b_c \quad b_t = 430 \text{ mm}\]

\[b_{eff} := \rho \cdot b_c \quad b_{eff} = 399.623 \text{ mm}\]

\[b_{e.1} := 0.4 \cdot b_{eff} \quad b_{e.1} = 159.849 \text{ mm}\]

\[b_{e.2} := 0.6 \cdot b_{eff} \quad b_{e.2} = 239.774 \text{ mm}\]

\[g := \begin{cases} b_{e.1} & \text{if } \sigma_{Mw1} > 0 \\ b_t + b_{e.2} & \text{otherwise} \end{cases} \quad g = 669.774 \text{ mm}\]

\[g := \begin{cases} \text{web_plate_M < 4, g_n, g} \end{cases} \quad g = 669.774 \text{ mm}\]

\[k := \begin{cases} b_{e.1} & \text{if } \sigma_{Mw2} > 0 \\ b_t + b_{e.2} & \text{otherwise} \end{cases} \quad k = 159.849 \text{ mm}\]

\[k := \begin{cases} \text{web_plate_M < 4, 0mm, k} \end{cases} \quad k = 159.849 \text{ mm}\]

lower flange:

\[B := \frac{a}{2} \quad h := \frac{a}{2} \quad t := f \quad \sigma_2 := 1 \quad \sigma_1 := 1 \quad \psi := 1\]

\[\lambda := \frac{B}{t} \sqrt{28.4 \cdot \varepsilon \cdot k_\sigma} \quad \lambda_p = 0.078 \quad k_\sigma = 0.43\]
The dimensions of the effective cross-section

\[
a = 360 \text{ mm} \quad b = 20 \text{ mm} \quad x_1 := 0 \text{ mm} \quad x_2 := g_n
\]
\[
e = 360 \text{ mm} \quad f = 20 \text{ mm}
\]
\[
g = 669.774 \text{ mm} \quad h = 8 \text{ mm} \quad i := 0 \text{ mm} \quad j := 0 \text{ mm} \quad k = 159.849 \text{ mm}
\]
\[
A_{\text{eff},M} := a \cdot b + c \cdot d + e \cdot f + (g + i + j + k) \cdot h
\]
\[
S_{y,\text{eff},M} = (a \cdot b) \cdot \frac{-b}{2} + c \cdot d \cdot x_1 + e \cdot f \cdot \left( x_2 + \frac{f}{2} \right) + g \cdot h \cdot \frac{g}{2} + i \cdot h \cdot \left( x_1 - \frac{d}{2} - \frac{i}{2} \right) \quad \cdots
\]
\[
s_{y,M} = \frac{S_{y,\text{eff},M}}{A_{\text{eff},M}} = 427.055 \text{ mm}
\]
\[
l_{y,\text{M.IV}} = a \cdot \frac{b^3}{12} + a \cdot b \cdot \frac{b}{2} + c \cdot \frac{d^3}{12} + c \cdot d \cdot x_1^2 + e \cdot f \cdot \left( x_2 + \frac{f}{2} \right)^2 + e \cdot f \cdot h \cdot \frac{g}{2} + \frac{g}{2} \cdot h \cdot \frac{g}{2} + \frac{g}{2} \cdot h \cdot \frac{g}{2} \cdot \frac{i}{2} \quad \cdots
\]
\[
l_{y,\text{M.IV}} = 319635.875 \text{ cm}^4
\]
\[
y_1 := s_{y,M} + b
\]
\[
y_1 = 447.055 \text{ mm} \quad y_1 + y_2 = 900 \text{ mm}
\]
\[
y_{\text{max},M} := \max(y_1, y_2) \quad y_{\text{max},M} = 452.945 \text{ mm}
\]
\[
W_{y,\text{eff},M} = \frac{l_{y,\text{M.IV}}}{y_{\text{max},M}}
\]
\[
W_{y,\text{eff},M} = 7056.832 \text{ cm}^3
\]
\[
M_{c,Rd.IV} := \frac{W_{y,\text{eff},M} \cdot y}{\gamma_{M,0}}
\]
\[
M_{c,Rd.IV} = 2505.175 \text{ kN} \cdot \text{m}
\]
Resistance of the effective cross-section under bending (M)

\[
M_{c,Rd} := \begin{cases} 
M_{c,Rd.I} & \text{if } \text{cross\_section\_bending} = 1 \\
M_{c,Rd.I} & \text{if } \text{cross\_section\_bending} = 2 \\
M_{c,Rd.III} & \text{if } \text{cross\_section\_bending} = 3 \\
M_{c,Rd.IV} & \text{if } \text{cross\_section\_bending} = 4 
\end{cases}
\]

\[M_{c,Rd} = 2505.175 \text{ kN\cdot m}\]

Resistance of the effective cross-section under axial force (N)

\[
N_{c,Rd} := \begin{cases} 
N_{c,Rd.I} & \text{if } \text{cross\_section\_bending} < 4 \\
N_{c,Rd.IV} & \text{if } \text{cross\_section\_bending} = 4 
\end{cases}
\]

\[N_{c,Rd} = 6062.667 \text{ kN}\]
Design moment calculations example according to EN 1993-1-8

Input data:

material properties: 
\[ f_y := 393 \frac{N}{mm^2} \]  
\[ f_u := 552 \frac{N}{mm^2} \]  
\[ f_{u,b} := 1018 \frac{N}{mm^2} \]  
\[ E := 210000 \frac{N}{mm^2} \]  
\[ v := 0.3 \]  
\[ G := \frac{E}{2(1 + v)} \] 

partial factor: 
\[ \gamma_{M.0} := 1.0 \]  
\[ \gamma_{M.1} := 1.0 \]  
\[ \gamma_{M.2} := 1.0 \] 

resistance of the section: 
\[ M_{c,Rd} := 2755 \cdot kN \cdot m \]

The dimensions of the welds are: 
\[ a_{w,f} := 4.5 \cdot mm \]  
\[ a_{w,w} := 4.5 \cdot mm \]

Geometry:

Bolts: 
\[ d := 20 \cdot mm \]  
\[ A_s := 245 \cdot mm^2 \]  
\[ k_2 := 0.9 \]

End-plate: 
\[ t_{ep} := 12 \cdot mm \]  
\[ b_{ep} := 360 \cdot mm \]  
\[ w := 80 \cdot mm \]  
\[ P_{CD} := 80 \cdot mm \]  
\[ P_{DE} := 640 \cdot mm \]

\[ m_B := 40 \cdot mm - 0.8 \cdot a_{w,f} \cdot \sqrt{2} \]  
\[ m_B = 34.9088 \cdot mm \]  
\[ e_B := 35 \cdot mm \]  
\[ n_{mB} := 1.25 \cdot m_B \]  
\[ n_B := \min(e_B, n_{mB}) \]  
\[ n_B = 35 \cdot mm \]

\[ m_C := 40 \cdot mm - 4 \cdot mm - 0.8 \cdot a_{w,w} \cdot \sqrt{2} \]  
\[ m_C = 30.9088 \cdot mm \]  
\[ e_C := 140 \cdot mm \]  
\[ n_{mC} := 1.25 \cdot m_C \]  
\[ n_C := \min(e_C, n_{mC}) \]  
\[ n_C = 38.636 \cdot mm \]

\[ m_D := 40 \cdot mm - 4 \cdot mm - 0.8 \cdot a_{w,w} \cdot \sqrt{2} \]  
\[ m_D = 30.9088 \cdot mm \]  
\[ e_D := 140 \cdot mm \]  
\[ n_{mD} := 1.25 \cdot m_C \]  
\[ n_D := \min(e_D, n_{mD}) \]  
\[ n_D = 38.636 \cdot mm \]
Calculation of the effective lengths

**bolt-row B:**

\[ l_{\text{eff.B.ind.1}} := 2 \cdot \Pi \cdot m_B \]
\[ l_{\text{eff.B.ind.1}} = 219.3389 \text{ mm} \]

\[ l_{\text{eff.B.ind.2}} := \Pi \cdot m_B + 2 \cdot 140 \text{ mm} \]
\[ l_{\text{eff.B.ind.2}} = 389.6694 \text{ mm} \]

\[ l_{\text{eff.B.ind.2.1}} := 4 \cdot m_B + 1.25 \cdot e_B \]
\[ l_{\text{eff.B.ind.2.1}} = 183.3853 \text{ mm} \]

\[ l_{\text{eff.B.gr.1.1}} := \Pi \cdot m_B + w \]
\[ l_{\text{eff.B.gr.1.1}} = 189.6694 \text{ mm} \]

\[ l_{\text{eff.B.gr.1.2}} := \Pi \cdot m_B + \frac{w + e_B}{2} \]
\[ l_{\text{eff.B.gr.1.2}} = 129.8347 \text{ mm} \]

\[ l_{\text{eff.B.gr.2.1}} := 2 \cdot m_B + 0.625 \cdot e_B + \frac{w}{2} \]
\[ l_{\text{eff.B.gr.2.1}} = 131.6927 \text{ mm} \]

\[ l_{\text{eff.B.gr.2.2}} := w + 2 \cdot 140 \text{ mm} \]
\[ l_{\text{eff.B.gr.2.2}} = 360 \text{ mm} \]

\[ l_{\text{eff.B.ind.2}} := l_{\text{eff.B.ind.2.1}} \]
\[ l_{\text{eff.B.ind.1}} := \min\{l_{\text{eff.B.ind.1.1}}, l_{\text{eff.B.ind.1.2}}, l_{\text{eff.B.ind.2}}\} \]
\[ l_{\text{eff.B.ind.1}} = 183.3853 \text{ mm} \]

\[ l_{\text{eff.B.gr.2}} := \min\{l_{\text{eff.B.gr.2.1}}, l_{\text{eff.B.gr.2.2}}\} \]
\[ l_{\text{eff.B.gr.1}} = 129.8347 \text{ mm} \]

\[ M_{\text{B.pl.ind.1.Rd}} = \frac{1}{4} l_{\text{eff.B.ind.1}} t_{\text{ep}}^2 \frac{f_Y}{\gamma_{M.0}} \]
\[ M_{\text{B.pl.ind.1.Rd}} = 2.5945 \text{ kN m} \]

\[ M_{\text{B.pl.ind.2.Rd}} = \frac{1}{4} l_{\text{eff.B.ind.2}} t_{\text{ep}}^2 \frac{f_Y}{\gamma_{M.0}} \]
\[ M_{\text{B.pl.ind.2.Rd}} = 2.5945 \text{ kN m} \]

\[ M_{\text{B.pl.gr.1.Rd}} = \frac{1}{4} l_{\text{eff.B.gr.1.1}} t_{\text{ep}}^2 \frac{f_Y}{\gamma_{M.0}} \]
\[ M_{\text{B.pl.gr.1.Rd}} = 1.8369 \text{ kN m} \]

\[ M_{\text{B.pl.gr.2.Rd}} = \frac{1}{4} l_{\text{eff.B.gr.2.1}} t_{\text{ep}}^2 \frac{f_Y}{\gamma_{M.0}} \]
\[ M_{\text{B.pl.gr.2.Rd}} = 1.8632 \text{ kN m} \]
Design resistance of the T-stub

Bolt in tension:

\[ F_{T,Rd} = \frac{k_2 f_{u,b} A_s}{\gamma_{M.2}} \]

Bolt-row considered individually:

Mode 1:

\[ F_{T,Rd.1,B,ind} = \frac{4 M_{B,pl,ind.1,Rc}}{m_B} \]

\[ F_{T,Rd.1,B,ind} = 297.2927 \text{ kN} \]

Mode 2:

\[ F_{T,Rd.2,B,ind} = \frac{2 M_{B,pl,ind.2,Rd} n_B}{m_B + n_B} 2 F_{T,Rd} \]

\[ F_{T,Rd.2,B,ind} = 298.988 \text{ kN} \]

Mode 3:

\[ F_{T,Rd.3} = 2 F_{T,Rc} \]

\[ F_{T,Rd.3} = 448.938 \text{ kN} \]

\[ F_{T,Rd,B,ind} = \min(F_{T,Rd.1,B,ind}, F_{T,Rd.2,B,ind}, F_{T,Rd.3}) \]

\[ F_{T,Rd,B,ind} = 297.2927 \text{ kN} \]

Bolt-row considered as "horizontal" group:

Mode 1:

\[ F_{T,Rd.1,B,gr} = \frac{4 M_{B,pl,gr.1,Rd}}{m_B} \]

\[ F_{T,Rd.1,B,gr} = 210.4799 \text{ kN} \]

Mode 2:

\[ F_{T,Rd.2,B,gr} = \frac{2 M_{B,pl,gr.2,Rd} n_B}{m_B + n_B} 2 F_{T,Rd} \]

\[ F_{T,Rd.2,B,gr} = 278.0651 \text{ kN} \]

Mode 3:

\[ F_{T,Rd.3} = 448.938 \text{ kN} \]

\[ F_{T,Rd,B,gr} = \min(F_{T,Rd.1,B,gr}, F_{T,Rd.2,B,gr}, F_{T,Rd.3}) \]

\[ F_{T,Rd,B,gr} = 210.4799 \text{ kN} \]

\[ F_{T,Rd,B} = \min(F_{T,Rd,B,ind}, F_{T,Rd,B,gr}) \]

\[ F_{T,Rd,B} = 210.4799 \text{ kN} \]
Calculation of the effective lengths

**bolt-row C:**

\[ l_{\text{eff.C.ind.1.1}} := 2 \Pi \cdot m_C^2 \]
\[ l_{\text{eff.C.ind.1.1}} = 156.507 \text{ mm} \]

\[ l_{\text{eff.C.ind.1.2}} := \Pi \cdot m_C^2 + 2 \cdot e_C \]
\[ l_{\text{eff.C.ind.1.2}} = 358.2535 \text{ mm} \]

\[ \lambda_1 := \frac{m_C}{m_C + e_C} \quad \lambda_1 = 0.1808 \]

\[ \lambda_2 := \frac{m_C^2}{m_C + e_C} \quad \lambda_2 = 0.1457 \]

\[ \alpha := 8.0 \quad \text{(from EC3 1-8, Fig. 6.11)} \]

\[ l_{\text{eff.C.ind.2.1}} := \alpha \cdot m_C \]
\[ l_{\text{eff.C.ind.2.1}} = 247.2706 \text{ mm} \]

\[ l_{\text{eff.C.ind.2.2}} := 4 \cdot m_C + 1.25 \cdot e_C \]
\[ l_{\text{eff.C.ind.2.2}} = 298.6353 \text{ mm} \]

\[ l_{\text{eff.C.gr.1.1}} := \Pi \cdot m_C^2 + P_{CD} \]
\[ l_{\text{eff.C.gr.1.1}} = 158.2535 \text{ mm} \]

\[ l_{\text{eff.C.gr.1.2}} := \Pi \cdot m_C^2 + e_C + \frac{P_{CD}}{2} \]
\[ l_{\text{eff.C.gr.1.2}} = 228.5515 \text{ mm} \]

\[ l_{\text{eff.C.gr.2.1}} := \alpha \cdot m_C + \frac{P_{CD}}{2} - \left(2 \cdot m_C + 0.625 \cdot e_C\right) \]
\[ l_{\text{eff.C.gr.2.1}} = 137.953 \text{ mm} \]

\[ l_{\text{eff.C.gr.2.2}} := 2 \cdot m_C + 0.625 \cdot e_C + \frac{P_{CD}}{2} \]
\[ l_{\text{eff.C.gr.2.2}} = 189.3177 \text{ mm} \]
Bolted end-plate joints for crane brackets and beam-to-beam connections

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\[ l_{\text{eff.C.ind.2}} = \min \left( l_{\text{eff.C.ind.2.1}} \cdot l_{\text{eff.C.ind.2.2}} \right) \]
\[ l_{\text{eff.C.ind.1}} = \min \left( l_{\text{eff.C.ind.1.1}} \cdot l_{\text{eff.C.ind.1.2}} \cdot l_{\text{eff.C.ind.2}} \right) \]
\[ l_{\text{eff.C.gr.2}} = \min \left( l_{\text{eff.C.gr.2.1}} \cdot l_{\text{eff.C.gr.2.2}} \right) \]
\[ l_{\text{eff.C.gr.1}} = \min \left( l_{\text{eff.C.gr.1.1}} \cdot l_{\text{eff.C.gr.1.2}} \cdot l_{\text{eff.C.gr.2}} \right) \]

\[ M_{\text{C.pl.ind.1.Rd}} = \frac{1}{4} l_{\text{eff.C.ind.1}} \cdot t_{\text{ep}} \cdot f_y \cdot \gamma_{M.0} \]
\[ M_{\text{C.pl.ind.2.Rd}} = \frac{1}{4} l_{\text{eff.C.ind.2}} \cdot t_{\text{ep}} \cdot f_y \cdot \gamma_{M.0} \]
\[ M_{\text{C.pl.gr.1.Rd}} = \frac{1}{4} l_{\text{eff.C.gr.1}} \cdot t_{\text{ep}} \cdot f_y \cdot \gamma_{M.0} \]
\[ M_{\text{C.pl.gr.2.Rd}} = \frac{1}{4} l_{\text{eff.C.gr.2}} \cdot t_{\text{ep}} \cdot f_y \cdot \gamma_{M.0} \]

Design resistance of the T-stub

Bolt-row considered individually:

Mode 1:
\[ F_{\text{T.Rd.1.C.ind}} = \frac{4 \cdot M_{\text{C.pl.ind.1.Rc}}}{m_{C2}} \]
\[ F_{\text{T.Rd.1.C.ind}} = 355.5784 \text{ kN} \]

Mode 2:
\[ F_{\text{T.Rd.2.C.ind}} = \frac{2 \cdot M_{\text{C.pl.ind.2.Rd}}}{m_{C} + n_{C}} \cdot F_{\text{T.Rd}} \]
\[ F_{\text{T.Rd.2.C.ind}} = 305.018 \text{ kN} \]

Mode 3:
\[ F_{\text{T.Rd.C.ind}} = \min \left( F_{\text{T.Rd.1.C.ind}}, F_{\text{T.Rd.2.C.ind}}, F_{\text{T.Rd.3}} \right) \]
\[ F_{\text{T.Rd.C.ind}} = 350.018 \text{ kN} \]

Bolt-row considered as "horizontal" group:

Mode 1:
\[ F_{\text{T.Rd.1.C.gr}} = \frac{4 \cdot M_{\text{C.pl.gr.1.Rd}}}{m_{C2}} \]
\[ F_{\text{T.Rd.1.C.gr}} = 313.4244 \text{ kN} \]

Mode 2:
\[ F_{\text{T.Rd.2.C.gr}} = \frac{2 \cdot M_{\text{C.pl.gr.2.Rd}}}{m_{C} + n_{C}} \cdot F_{\text{T.Rd}} \]
\[ F_{\text{T.Rd.2.C.gr}} = 305.5395 \text{ kN} \]

Mode 3:
\[ F_{\text{T.Rd.C.gr}} = \min \left( F_{\text{T.Rd.1.C.gr}}, F_{\text{T.Rd.2.C.gr}}, F_{\text{T.Rd.3}} \right) \]
\[ F_{\text{T.Rd.C.gr}} = 305.5395 \text{ kN} \]

\[ F_{\text{T.Rd.C}} = \min \left( F_{\text{T.Rd.C.ind}}, F_{\text{T.Rd.C.gr}} \right) \]
\[ F_{\text{T.Rd.C}} = 305.5395 \text{ kN} \]
**Calculation of the effective lengths**

**Bolt-row D:**

\[ l_{\text{eff.D.ind.1.1}} := 2 \cdot \Pi \cdot m_D \]
\[ l_{\text{eff.D.ind.1.1}} = 194.2061 \text{ mm} \]

\[ l_{\text{eff.D.ind.1.2}} := \Pi \cdot m_D + 2 \cdot e_D \]
\[ l_{\text{eff.D.ind.1.2}} = 358.2535 \text{ mm} \]

\[ l_{\text{eff.D.ind.2.1}} := 4 \cdot m_D + 1.25 \cdot e_D \]
\[ l_{\text{eff.D.ind.2.1}} = 298.6353 \text{ mm} \]

**Group failure**

\[ l_{\text{eff.D.gr.1.1}} := \Pi \cdot m_D + P_{CD} \]
\[ l_{\text{eff.D.gr.1.1}} = 177.1031 \text{ mm} \]

\[ l_{\text{eff.D.gr.1.2}} := \Pi \cdot \frac{m_D}{2} + e_D + \frac{P_{CD}}{2} \]
\[ l_{\text{eff.D.gr.1.2}} = 228.5515 \text{ mm} \]

\[ l_{\text{eff.D.gr.2.1}} := 2 \cdot m_D + 0.625 \cdot e_D + \frac{P_{CD}}{2} \]
\[ l_{\text{eff.D.gr.2.1}} = 189.3177 \text{ mm} \]

\[ l_{\text{eff.D.ind.2}} \leq l_{\text{eff.D.ind.2.1}} \]
\[ l_{\text{eff.D.ind.1}} \leq \min(l_{\text{eff.D.ind.1.1}}, l_{\text{eff.D.ind.1.2}}, l_{\text{eff.D.ind.2}}) \]
\[ l_{\text{eff.D.gr.2}} \leq l_{\text{eff.D.gr.2.1}} \]
\[ l_{\text{eff.D.gr.1}} \leq \min(l_{\text{eff.D.gr.1.1}}, l_{\text{eff.D.gr.1.2}}, l_{\text{eff.D.gr.2}}) \]
Design resistance of the T-stub

Bolt-row considered individually:

Mode 1:
\[ F_{T,Rd.1,D,ind} = \frac{4 \cdot M_{D,pl,ind.1,Rd}}{m_D} \]
\[ F_{T,Rd.1,D,ind} = 355.5784 \, kN \]

Mode 2:
\[ F_{T,Rd.2,D,ind} = \frac{2 \cdot M_{D,pl,ind.2,Rd}^+ \cdot n_D^2 \cdot F_{T,Rd}}{m_D + n_D} \]
\[ F_{T,Rd.2,D,ind} = 370.917 \, kN \]

Mode 3:
\[ F_{T,Rd.3} = 448.938 \, kN \]

Bolt-row considered as "horizontal" group:

Mode 1:
\[ F_{T,Rd.1,D,gr} = \frac{4 \cdot M_{D,pl,gr.1,Rd}}{m_D} \]
\[ F_{T,Rd.1,D,gr} = 324.2638 \, kN \]

Mode 2:
\[ F_{T,Rd.2,D,gr} = \frac{2 \cdot M_{D,pl,gr.2,Rd}^+ \cdot n_D^2 \cdot F_{T,Rd}}{m_D + n_D} \]
\[ F_{T,Rd.2,D,gr} = 326.4384 \, kN \]

Mode 3:
\[ F_{T,Rd.3} = 448.938 \, kN \]

\[ F_{T,Rd,CD,gr} = \min\{F_{T,Rd.1,C,gr}^+, F_{T,Rd.1,D,gr} F_{T,Rd.2,C,gr}^+ F_{T,Rd.2,D,gr}^2 F_{T,Rd.3}\} \]
\[ F_{T,Rd,CD,gr} = 631.9779 \, kN \]

\[ F_{T,Rd.2,C,gr}^+ F_{T,Rd.2,D,gr}^+ = 631.9779 \, kN \]

\[ F_{T,Rd.D,gr} = F_{T,Rd.2,D,gr}^+ \]
\[ F_{T,Rd.D,gr} = 326.4384 \, kN \]

\[ F_{T,Rd.D} = \min\{F_{T,Rd.D,ind} F_{T,Rd.D,gr}\} \]
\[ F_{T,Rd.D} = 326.4384 \, kN \]
Bolted end-plate joints for crane brackets and beam-to-beam connections

Appendix H

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**T-stub forces**

\[ F_{T,Rd.B} = 210.4799 \text{kN} \]

Mode 1, horizontal group failure

\[ F_{T,Rd.C} = 305.5395 \text{kN} \]

Mode 2, group failure

\[ F_{T,Rd.D} = 326.4384 \text{kN} \]

Mode 2, group failure

**Beam flange and web in compression**

\[ M_{c,Rd} := 2755 \text{kN} \cdot \text{m} \]

\[ h := 900 \text{ mm} \]

\[ t_{fb} := 20 \text{ mm} \]

\[ F_{c.fb,Rd} :=\frac{M_{c,Rd}}{h - t_{fb}} \]

\[ F_{c.fb,Rd} = 3130.6818 \text{kN} \]

**Web in transverse tension**

\[ t_{wb} := 8 \text{ mm} \]

\[ F_{t.wb,Rd.B} := l_{eff.B.gr.1} \cdot t_{wb} \cdot \frac{f_y}{\gamma M.0} \]

\[ F_{t.wb,Rd.B} = 408.2004 \text{ kN} \]

\[ F_{t.wb,Rd.C} := l_{eff.C.gr.1} \cdot t_{wb} \cdot \frac{f_y}{\gamma M.0} \]

\[ F_{t.wb,Rd.C} = 433.7242 \text{ kN} \]

\[ F_{t.wb,Rd.D} := l_{eff.D.gr.1} \cdot t_{wb} \cdot \frac{f_y}{\gamma M.0} \]

\[ F_{t.wb,Rd.D} = 595.2147 \text{ kN} \]

The effective design tension resistance of the bolt-rows B, C and D

\[ F_{t.wb,Rd.B} = 408.2004 \text{ kN} \quad > \quad F_{T,Rd.B} = 210.4799 \text{ kN} \quad F_{tr.Rd.B} := F_{T,Rd.E} \]

\[ F_{t.wb,Rd.C} = 433.7242 \text{ kN} \quad > \quad F_{T,Rd.C} = 305.5395 \text{ kN} \quad F_{tr.Rd.C} := F_{T,Rd.C} \]

\[ F_{t.wb,Rd.D} = 595.2147 \text{ kN} \quad > \quad F_{T,Rd.D} = 326.4384 \text{ kN} \quad F_{tr.Rd.D} := F_{T,Rd.C} \]

and

\[ F_{tr.Rd.B} + F_{tr.Rd.C} + F_{tr.Rd.D} = 842.4578 \text{ kN} \quad > \quad F_{c.fb,Rd} = 3130.6818 \text{ kN} \]

The design moment resistance of the joint

because of the symmetry of the bolt arrangement

\[ F_{tr.Rd.E} := F_{tr.Rd.C} \]

\[ F_{tr.Rd.E} = 326.4384 \text{ kN} \quad \text{Mode 2, group failure} \]

\[ F_{tr.Rd.F} := F_{tr.Rd.C} \]

\[ F_{tr.Rd.F} = 305.5395 \text{ kN} \quad \text{Mode 2, group failure} \]

\[ h_B := 930 \text{ mm} \quad h_C := 840 \text{ mm} \quad h_D := 760 \text{ mm} \quad h_E := 120 \text{ mm} \quad h_f := 40 \text{ mm} \]

\[ M_{j,Rd} = F_{tr.Rd.B} \cdot h_B + F_{tr.Rd.C} \cdot h_C + F_{tr.Rd.D} \cdot h_D + F_{tr.Rd.E} \cdot h_E + F_{tr.Rd.F} \cdot h_f \]

\[ M_{j,Rd} = 751.8869 \text{ kN} \cdot \text{m} \]