Domeniul de Inginerie și Instalații

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- REZUMAT -

Behaviour of beam-to-column connections with extended end-plate with four bolts-per-row

by

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CONTENTS

1. INTRODUCTION

Within the domain of steel construction, the importance of the role of steel connections is paramount, serving as the element that unites individual components into a cohesive and resilient structure. These connections, often composed of bolts and welds, play a pivotal role in transmitting forces, accommodating rotations, and ensuring overall stability. The importance of steel connections becomes particularly pronounced in the context of a steel structure's behaviour under demanding loads and conditions. They serve as critical interfaces where forces are concentrated, redistributed, or dissipated, thus directly impacting the structural response to external influences such as live loads, wind, seismic activity, and even accidental events like explosions or impacts. Understanding the behaviour of steel connections is therefore instrumental in ensuring the safety and reliability of a structure.

End-plate bolted connections are widely used in steel-frame structures for over half a century. In Europe, end-plate bolted connections are widely adopted for hot-rolled profile connections, including composite sections. The classic configuration with two bolts-per-row finds extensive use in steel structures with moment resisting frames (MRFs) due to their heightened rotational capacity, in comparison to welded connections. MRFs are a simple yet efficient architectural solution for enabling clear openings and wide spaces without disregarding the strength and stability of a given structure. The compromise is the heightened role of the main beam-to-column connections, which must provide the ensemble with a reliable means of load transfer and adequate rigidity while retaining a reasonable price tag.

While the classic two bolt-per-row configurations is considered as a standard in the industry, extensive research has demonstrated its limitations. While a common type of this connection can carry significant loads, the design of full-strength end-plate connection often leads to expensive to solutions and constrains the design of adjacent elements (imposing them to take on dissipative roles), while partial-strength connection fare poorly in situations of extreme loading, where design actions are exceeded, leading to a brittle failure of the connection and a possible structural collapse.

Motivating this study is the need to reassess longstanding assumptions. Particularly in Europe, there exists a discrepancy between conventional design assumptions and the actual performance of these connections. The generalized hypothesis that bending moments are solely transmitted by bolt rows close to the tension flange of the beam is being challenged, especially with advancements in recent research. The primary objective of this research is to gain a profound insight into the behaviour and performance of an alternative four bolts-per-row configuration of steel connections within the framework of MRF steel construction. By investigating their response under different loading scenarios, this study aims to contribute to a deeper understanding of their role in ensuring structural integrity and safety. The objectives of this study aim to contribute valuable insights to end-plate bolted beam-to-column connections. The primary objectives include:

- Investigation and optimization of the conventional two-bolts-per-row configuration prescribed by the European design code EN 1993-1-8 with additional systems in 4 bolts-per row configurations. Exploration variations in bolt disposition, end-plate thickness, bolt diameter, and flange width to identify configurations that enhance connection performance, especially under extreme loading conditions.
- Conducting an experimental study on 30 macro-component specimens representing different configurations, specifically in extended end-plate connections. Assessing the elasto-plastic behaviour of these assemblies to understand their responses to varying parameters.
- Development of an advanced FEM parametric study based on and calibrated to the behaviour results from the experimental tests, in order to further explore complementary geometries to those used for the experimental testing.
- Comparison the performance of configurations employing two bolts-per-row with those utilizing four bolts-per-row. Evaluation of the advantages and limitations of each configuration in terms of resistance, stiffness, and overall structural behaviour.
- Providing findings that have the potential to refine design procedures in steel construction, offering insights into how optimized connection configurations can contribute to the resilience and adaptability of steel structures, especially in scenarios involving extreme loading.
- Contribution to academic research by expanding the understanding of the behaviour of steel connections. This involves offering nuanced insights into the effects of various parameters on connection performance and providing a basis for further research and development in the field.

2. STATE OF THE ART

Moment-resisting frames (MRFs) are widely used in structural engineering, enabling large, opened spans, but providing adequate structural behaviour, specifically against lateral loads such as seismic and wind forces (Fig. 2-1). In this type of structural solution, the connections play a crucial role by transferring the bending moment between beam and column, through an adequate design providing them with suitable stiffness, strength, and ductility.

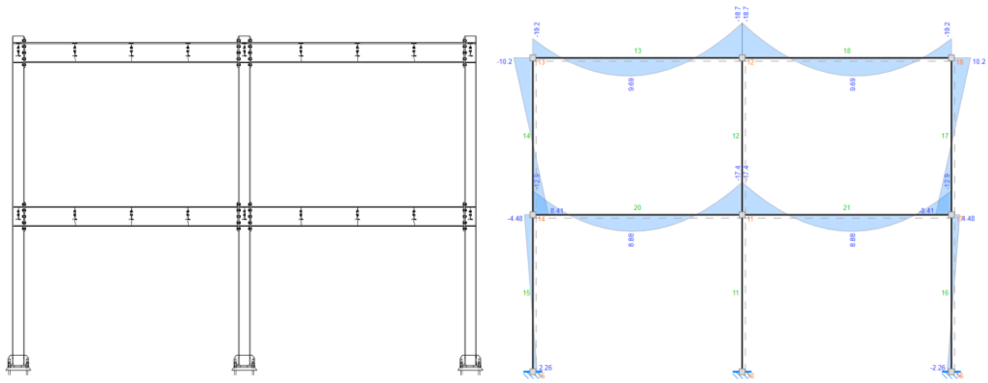


Fig. 2-1 Bending moment distribution on a moment resisting frame under vertical loads

The beam-to-column connections hold significant importance in the behaviour of framed structures and demand careful attention in their design. The widely employed configuration is the extended end-plate bolted connection with two bolts-per-row, typically designed to accommodate the elastic bending moment of the beam. However, this conventional setup has revealed its limitations, particularly when its bending capacity is reached. A notable drawback of such systems is their limited ability to carry axial forces, as highlighted in the study by Dinu, F [1].

The capability to withstand axial forces becomes crucial under special loading conditions, including seismic events or extreme loading scenarios, where a desirable post-critical behaviour is essential, as indicated by the CEN standards in 2006. During such events, it becomes necessary for the connections to retain a certain capacity even after experiencing failure. This capacity enables the redistribution of loads within the structural members, ensuring the overall structural integrity.

In this particular scenario, utilizing a configuration of four bolts per row for connections has been studied as a potential solution for enhancing structural integrity, especially in situations where the risk of progressive collapse needs to be minimized. Although it has received limited research attention, this technical approach was mentioned in outdated French standards and continues to be a viable design choice in current German standards.

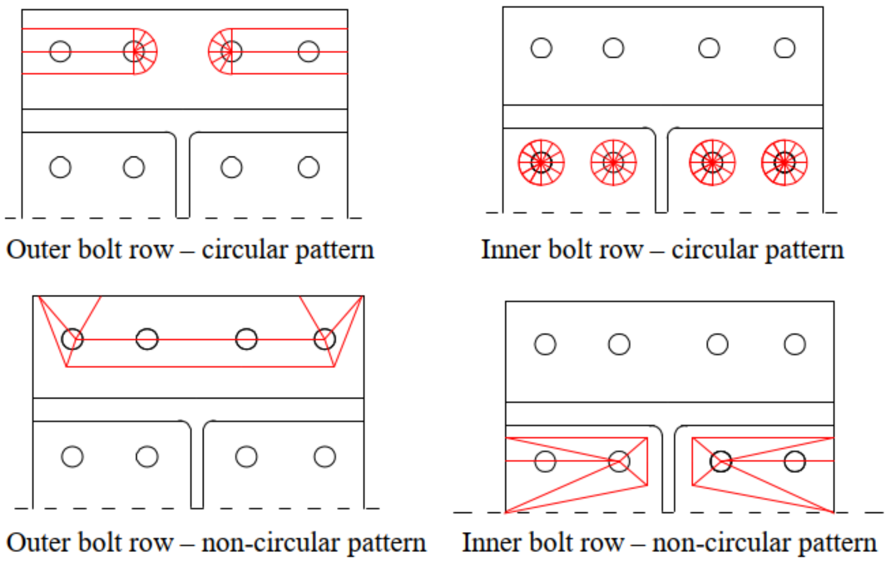


Fig. 2-2 Examples of plastic yield patterns with four bolts-per-row according to Demonceau, J-F et al. [2]

Previous research endeavours have aimed to investigate the behaviour and failure mechanisms associated with the configuration involving four bolts-per-row in steel connections. Notable contributions have been made by research teams from various institutions, such as Liège University (Fig. 2-2) [3], [4], and [2]. Additionally, Rzeszow University conducted studies by Kozolowski et al. [5]. Furthermore, a collaborative effort between Salerno and Coimbra universities also yielded enlightening results [6].

The existing body of research has primarily focused on determining the stiffness and elastic capacity of T-stubs, equivalent to EC3-1-8 [7], by adapting and refining concepts such as effective length and failure modes. However, there remains a significant gap in developing a comprehensive design methodology that accurately represents the plastic behaviour exhibited by these connections.

There is, however, renewed interest for this type of connections tied to the concept of robustness and need for adequate post-flexural behaviour. With the addition of outer bolt-rows, not only the overall strength may be improved but the added connecting elements may be used as a fail-safe in the occurrence of an initial failure, so as to avoid the row-by-row failure of the classic solutions. Previous studies suggest that the equivalent T-stub approach may not accurately describe the failure mechanism of a connection, as this configuration does not follow the same failure pattern [8]. The present study focuses on the tensioned macro-component of the bolted end-plate connections, comprising the first two bolt-rows of the connection, including the flange and the upper portion of the web of the connected beam.

2.1. **Existing standards**

In the context of German structural engineering, a well-established practice involves using end-plate connections with four bolts arranged horizontally. Back in the 1970s, the German Steelwork Association (DSTV) developed a mathematical model to predict the strength of 4 bolts-per-row end-plate connections. This tradition has its own standards, defining two types of configurations: IH 2 for narrow end-plates and IH 4 for wider end-plates. Several experiments were undertaken to refine the model, tweaking it to make the connections even stronger. Regrettably, records of those tests are unpublished. It is important to note that this mathematical model is limited. It's best suited for connections with thick end-plates that fall within a certain range. You can find the most recent strength values based on this model in [9]. In 2002, a more comprehensive model emerged, in line with Eurocode 3 standards. However, since there was not much experimental data available, the authors followed a conservative approach, which resulted in less cost-efficient outcomes compared to the earlier DSTV model [10].

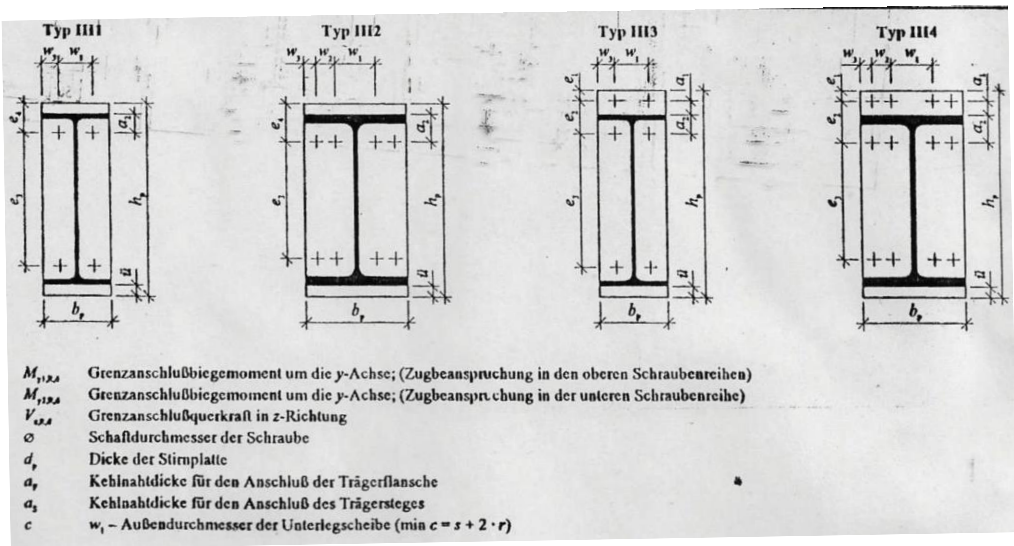


Fig. 2-3 Extracted sketch from connection design catalogue according to DSTV [10]

The Eurocode 3: Design of steel structures – Part 1-8: Design of joints [7], is the current connection design standard in Europe. The standard uses a system of classification in regards to stiffness – rigid, semi-rigid and pinned - and strength – full strength, partial strength or pinned. Moment resisting (MR) connections can be classified as either rigid or semi-rigid, and either full or partial strength. Their design method in EC3-1-8 is based on the component method which includes the evaluation of individual T-shaped elements (T-stubs) which represent the behaviour of individual bolt-rows (Fig. 2-4).

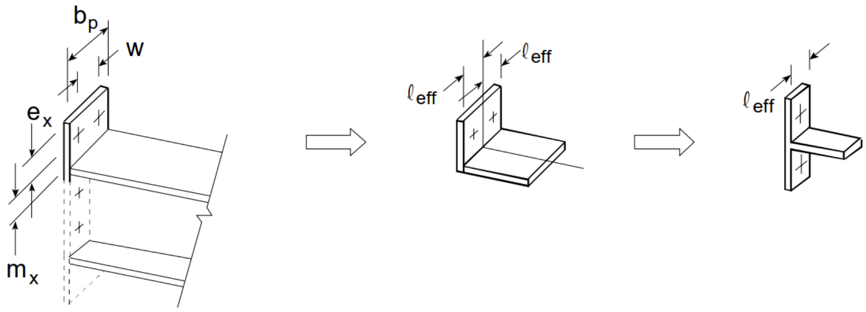


Fig. 2-4 Representation of the equivalent T-stub according to EN1993-1-8 [7]

The literature on beam-to-column connections highlights significant research gaps, including a lack of studies on beams with flanges narrower than the bolt group and the absence of a standardized method for evaluating rotational capacity. While existing research mainly focuses on elastic and pre-critical behaviour, there is insufficient exploration of post-critical behaviour, particularly in complex 4B configurations. The application of beam-to-column 4B connections in a robustness-sensitive context is also inadequately addressed. These deficiencies hinder the development of accurate predictive models and design codes, emphasizing the need for future research to prioritize these aspects. Addressing these gaps is crucial for advancing understanding, informing design practices, and establishing robust and reliable methodologies in structural engineering.

3. PRELIMINARY 4BPR APPROACH

Connections are integral components in the design and performance of steel structures, and understanding their behaviour is crucial for ensuring structural integrity [11]. One key factor that influences the performance of moment resisting steel bolted beam-to-column connections is the arrangement of bolts. In this context, a numerical study was conducted to preliminary investigate and assess the behaviour of steel connections with different bolt configurations. The study specifically focused on comparing the performance of moment-resisting beam-to-columns bolted connections with two-bolts-per-row (2B) and respectively four-bolts-per-row (4B) configurations. The objective was to analyse and evaluate the strengths, failure mechanisms, ductility, and post-flexural behaviour of these two configurations in view of further experimental testing. By examining these aspects, the study aims to provide valuable insights into the component response and performance of steel connections, contributing to the development of more efficient and reliable connection designs.

Traditionally, the two-bolts-per-row (2B) configuration represents the common solution of design of bolted end-plate connection, this solution being considered as a basis conventional arrangement, investigating its failure mechanisms and performance under various loading conditions [12]. The focus was on understanding the bending and post-flexural response of the 2B configuration.

The four-bolts-per-row, on the other hand, is a less researched alternative, despite being considered in some design standards since the 70s. A new interest has developed in the researching community since the late 2000s and a substantial headway has been made in order the correctly understand the behaviour of this configuration but there are, as previously mentioned, several unaddressed questions regarding its correct application and advantages [13].

3.1. Global behaviour of MR connections

This numerical study undertakes an examination of the mechanical performance of moment-resisting beam-to-column bolted end-plate connections subjected to extreme loading conditions. The primary objective is to evaluate the ductility of the connection as well as the post-critical behaviour to ensure the overall structural integrity of the original structure. To achieve this, various bolt geometric configurations are investigated in detail (Fig. 3-2).

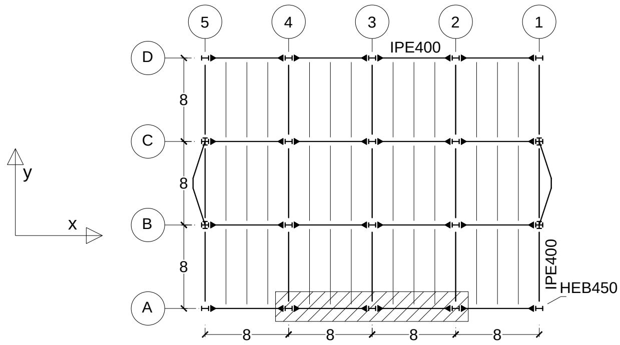
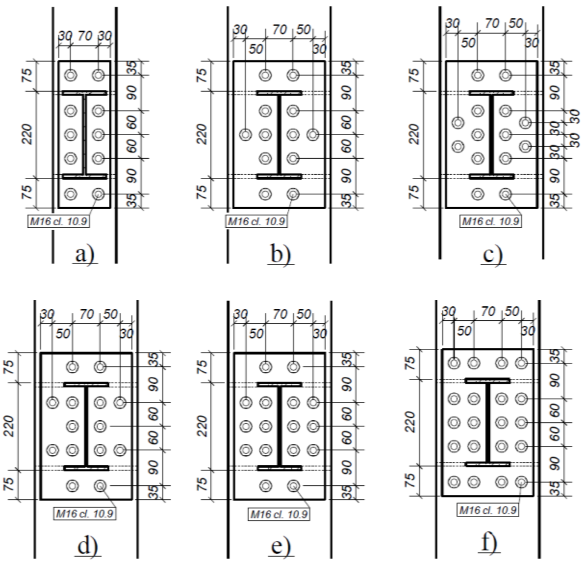


Fig. 3-1 Reference building layout plan and position of retrieved specimens for testing

For this specific study, the behaviour of the frames was analysed considering the global vertical force versus – mid-column displacement. These curves were recorded for each configuration and the failure patterns were analysed in order to assess the ductility criteria. For verification of numerical results, the experimental results are integrated, in direct comparison with the 2B_O model which replicate the real test. Thus, the comparison between the experimental and numerical results allows for an assessment of the accuracy and validity of the numerical model in capturing the actual behaviour of the connection under load. Fig. 3-3 displays the force-deformation behaviour curves. In each diagram the experimental curve and the corresponding numerical model response (2B_O) are compared.



a) 2B_O	b) 2B_1	c) 2B_2V
d) 2B_2	e) 2B_3	f) 4B

Fig. 3-2 Connection configurations (dimensions in mm)

Overall, the results demonstrate that both the addition of extra bolts and the widening of the end-plate contribute to improving the mechanical performance of the connection. These modifications enhance both the ultimate strength and ductility of the connection, ultimately increasing the robustness and reliability of the entire structural system. The configurations featuring additional outer bolts demonstrate the ability to recover a substantial percentage of their maximum strength following the initial failure, showcasing a noteworthy post-flexural rebound rate.

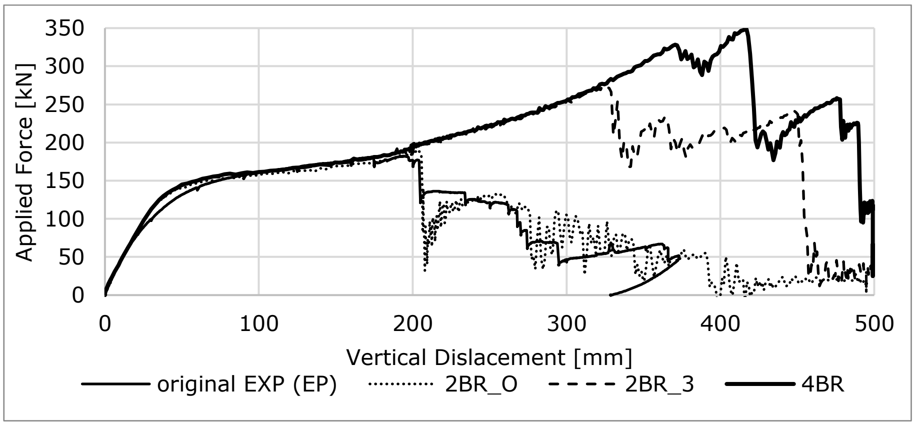


Fig. 3-3 Comparison between force-deformation curves

3.2. **Macro-component study**

In order to better understand the range of behaviour of the 4B configuration, a preliminary numerical study was performed, in order to identify the parameters which might be of interest in the upcoming experimental testing.

The T-stub equivalent model is extensively utilized for analysing end-plate steel connections. Nevertheless, when the complete tension assembly is taken into account, the behaviour of the T-stub can be altered due to the simultaneous consideration of both the beam's flange and web. Numerous studies have adopted this perspective, incorporating these components into the configuration similar to that of a T-stub [14]. This approach not only considers the behaviour of individual bolt-rows in relation to the adjacent flange and web, but also considers the interaction between multiple rows (Fig. 3-4).

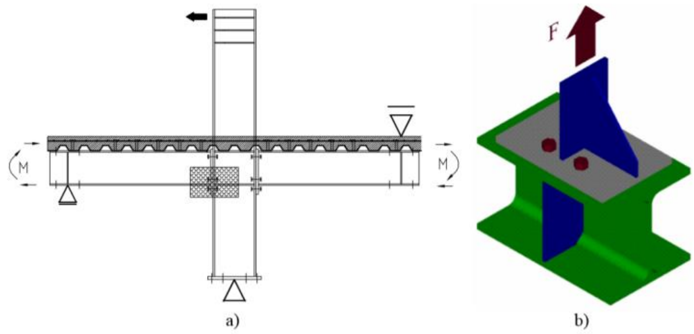


Fig. 3-4 Two-bolts-per-row macro-component retrieved from a beam-to-column connection

To examine the potential impact of adding exterior bolts, an alternative connection to an original two bolt-per-row connection from a previous study, was devised while keeping the same structural elements and retaining the original geometric characteristics. As demonstrated by the prior bolt numerical study, the key of the connection stays in the tension elements of the connection. Thus, in this study, the upper portion of the connection was isolated as a macro-component to specifically analyse its tensile behaviour (Fig. 3-5).

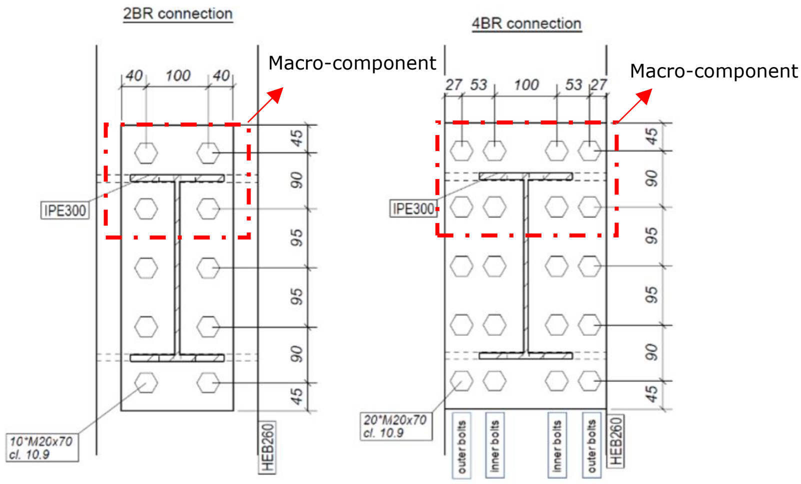


Fig. 3-5 Extraction of the macro-component

The isolation of the zone in tension comprises the two bolt-rows adjacent to the flange, but also the beam flange and the related part of the web, which contributes to the distribution of forces between both bolt-rows. The behaviour of tension zone is also the key in the rotational stiffness of the connection, in the case where the columns shear panel and flange are not being considered, as the stiffness of the compression zone is very high and assumed infinite for design purposes. In this way, the rotation of the connection can be easily obtained by multiplying the total horizontal deformation of the tension zone with the lever arm considered as the distance to the centre of rotation (middle line of the flange in compression).

The diagrams in Fig. 3-6 and Fig. 3-7 clearly highlight a significant difference between the two configurations in characteristic force-deformation curve responses. Specifically, the configuration with four bolts-per-row exhibits a much more pronounced plastic and post-flexural behaviour compared to the two-bolt configuration. This indicates that the four-bolt configuration demonstrates better ductility and an important strength recovery performance in terms of its ability to undergo plastic deformation without significant loss of strength. On the other hand, the elastic phase of the response remains largely unchanged between the two configurations.

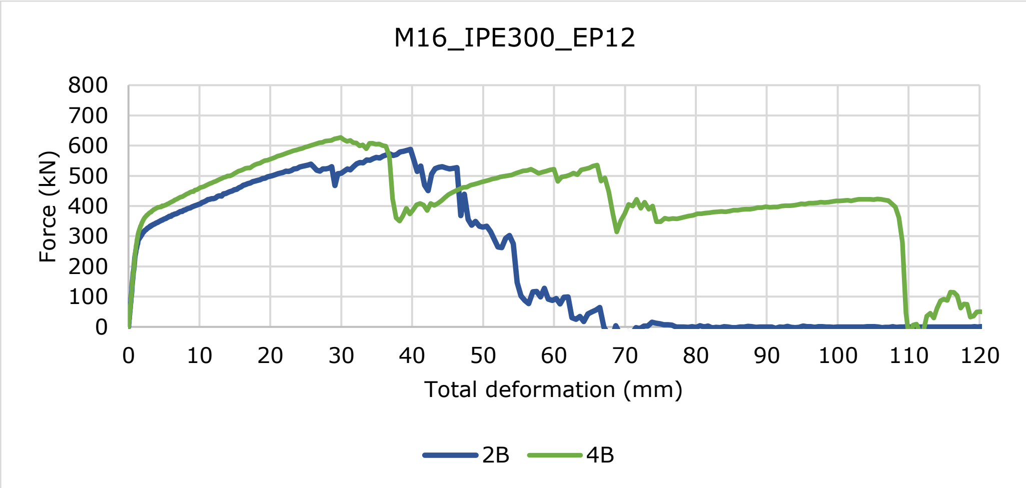


Fig. 3-6 Behaviour curves for models 2B and 4B for M16_IPE300_EP12

Overall, the four-bolts-per-row configuration exhibits advantages in post-elastic range in terms of ductility, failure mechanisms, and robustness performance, especially when compared to the classic two-bolts-per-row configurations commonly considered in design codes such as the Eurocodes.

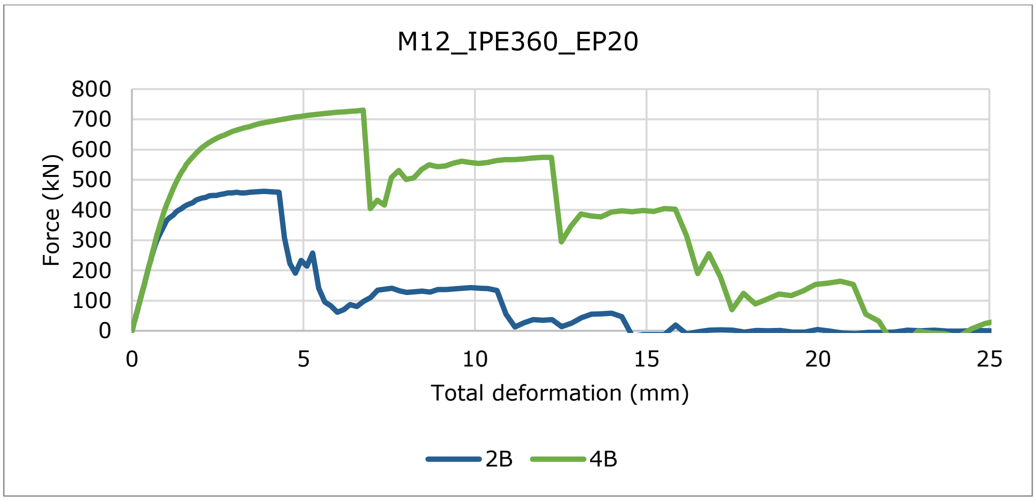


Fig. 3-7 Behaviour curves for models 2B and 4B for M12_IPE360_EP20

4. EXPERIMENTAL PROGRAM ON MACRO-COMPONENTS

The typical configuration for moment resisting steel frames utilizes a two-bolts-per-row end-plate bolted beam-to-column connection [15]. This configuration is outlined in detail in the European design code EN 1993-1-8 and is based on the assessment of resistance and stiffness of various components within the connection. However, under extreme loading conditions, there is a potential for enhancing the performance of this configuration.

The numerical studies from chapter 3 have laid-out several important indications regarding the behaviour of 4B configurations and their possible advantages in relation to the classic 2B connections. From the parameter's studies, the t/d (end-plate thickness – to bolt diameter) ratio and the beam's width specifically presented a very significant impact on the connection's behaviour. Also important for the subsequent testing was the sketching of practical limits which the experimental program should take into consideration, like the geometric limits for avoiding full beam plasticity and the compatibility between the obtained maximum forces and the test rig capacity.

Therefore, an experimental program was set out with the following objectives:

- Confirm and measure the relevance of the t/d ratio and beam's flange width;
- Observe the formation of inter-row diagonal coupling;
- Study the impact and behaviour of beam-to-end-plate welding;
- Constitute a calibrated knowledge base for broader numerical studies, such as complete connections and direct application in building structural analysis.

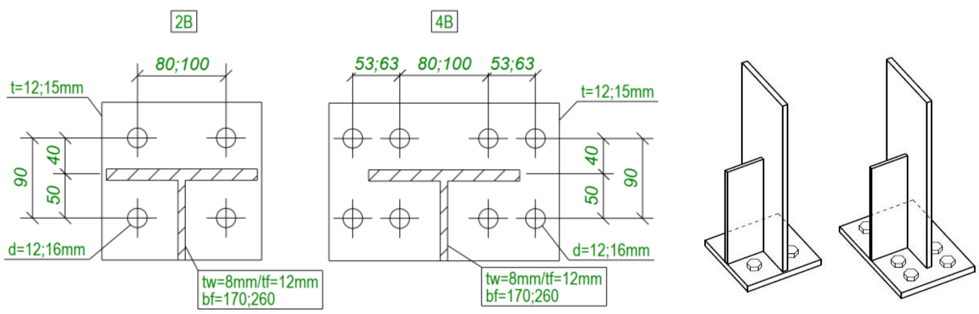


Fig. 4-1 Specimen configurations

The experimental study involved pull-out testing and analysis of 30 macro-component specimens, which represent the tensile area of a steel beam-to-column extended end-plate connection. Various parameters are investigated, including bolt diameter, end-plate thickness, flange width, and bolt arrangement. This study focused on examining the elasto-plastic behaviour of the systems and comparing their performance to the traditional two-bolts-per-row solutions.

The study specifically focuses on the outcomes derived from thirty specimens, considering both two-bolt-rows (2B) and four-bolt-rows (4B) configurations. The parameters taken into account, as shown in Fig. 4-1, include the beam profiles IPE360 and HEA260, end plate thicknesses of 12mm and 15mm, bolt diameters of 12mm and 16mm, and the bolt-disposition patterns named P1, P2, and P3.

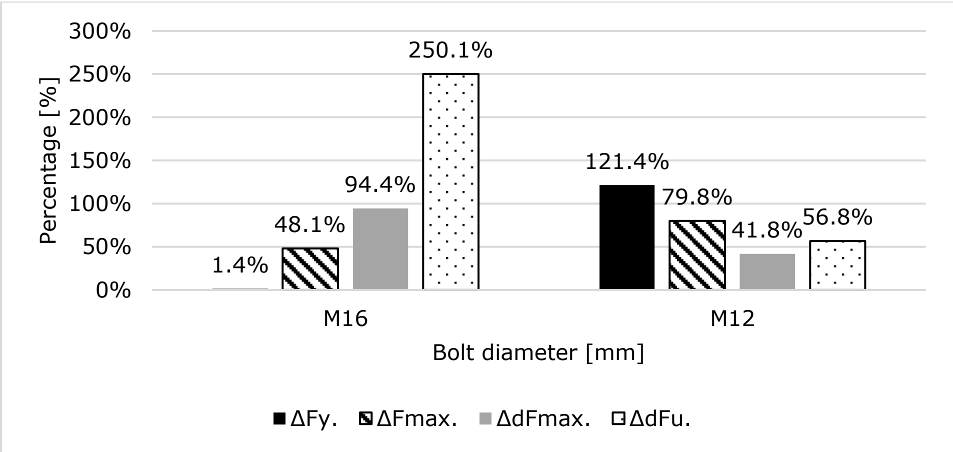


Fig. 4-2 4B relative resistance gains (I360_t15_P1) for bolt diameter variation

The variation of bolt diameter, specifically transitioning from M16 to M12, exhibits a notable increase in both yield and maximum strength variation, as depicted in Fig. 4-2. This can be attributed to the shift in the rigidity ratio between the end-plate and bolts. The relatively stiffer end-plate facilitates improved force distribution across the bolts, particularly the outer ones. However, the gain in ductility level is comparatively lower, primarily due to the connection displaying a more brittle behaviour, leaning towards mode 3 failure.

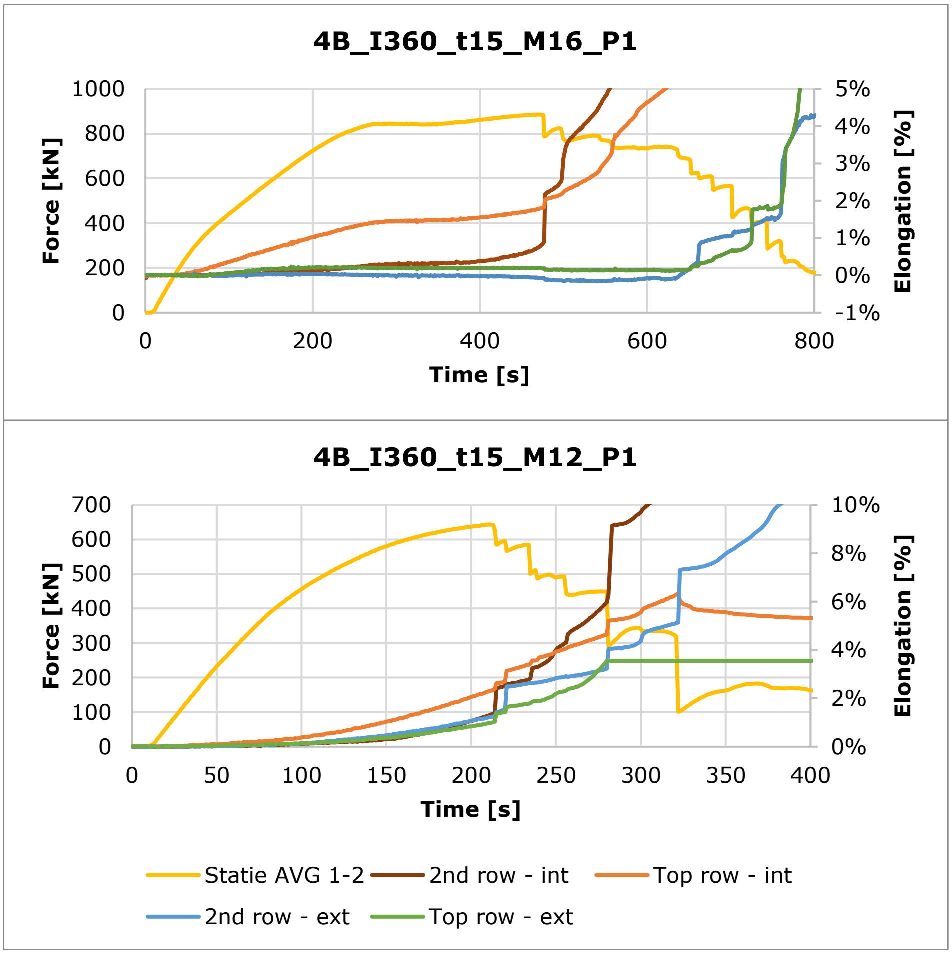


Fig. 4-3 Force & bolt elongation vs. time for 4B_I360_t15_P1

Regarding the bolt elongation, the reading in Fig. 4-3 shows a much earlier and more pronounced collaboration of the outer bolts, consistent with the increase in the t/d ratio (from M16 bolts to M12).

In all tested specimens, the failure sequence consistently began with initial yielding of the end-plate, followed by bolt failure. The bolts predominantly experienced failure by thread stripping, which is characteristic of the HV bolts subjected to traction.

However, both bolt and end-plate failure are strongly correlated with the t/d ratio of the connection. In the case of smaller t/d values (pointing to mode 2 towards 1, whose threshold is around t/d≈0.29), large deformations were registered in the end-plate, which in turn exerted a relevant bending moment on the bolts, leading to an asymmetrical thread stripping (Fig. 4-4). Due to the high deformation of the end-plate, a failure of the welds was also observed at the edge of the beam's flange, in the IPE360 models (Fig. 4-5).

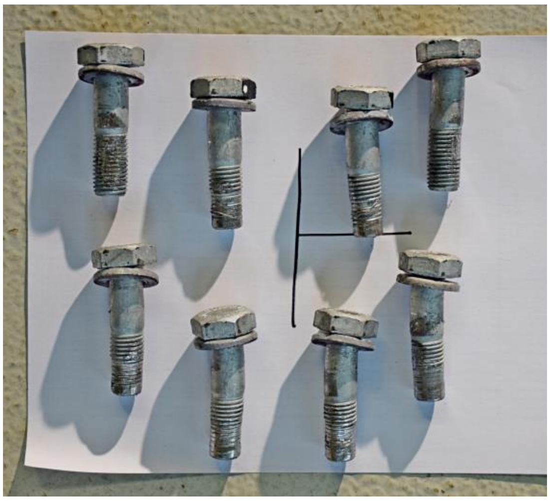


Fig. 4-4 Tested bolts from 4B_I360_t12_M16_P3

Fig. 4-5 End-plate deformation and weld failure from 4B_I360_t12_M16_P3

The 4B cases specifically exhibited more complex and staggered bolt failures. The bolts failed in symmetric pairs, and the failure sequence generally followed the patterns observed in previous numerical simulations [16]: (i) failure of the inner bolts in the second row, (ii) failure of the inner bolts in the first row, (iii) failure of the outer bolts in the second row, and (iv) failure of the outer bolts in the first row. In specimens with larger bolt diameters, a partial failure of the welds occurred between steps (ii) and (iii) due to the eccentric loading of the welds (Fig. 4-6).

Fig. 4-6 Resulting deformation of specimens

In the case of the 2B configuration, the yielding progression of the end-plate occurs linearly, row by row, following the bolt disposition. However, due to the complex failure mechanism observed in the 4B specimens, the yielding phenomena become more intricate. The yielding integrates the demand on both the external and internal sides of the end plate, resulting in a greater degree of complexity (Fig. 4-6).

The analysis of the experimental results focused on evaluating the influence of the additional outer bolts on the performance of the macro-component. The following trends were observed:

- The resistance of the macro-component with 4B configuration showed an improvement in both the elastic force, associated with the yield strength, and the maximum strength.
- The deformation corresponding to the maximum strength exhibited an increase in the case of the 4B configuration.

The results of the study indicate that the 4B specimens exhibited a significant increase in both elastic resistance and maximum strength compared to the 2B specimens. However, the increase in resistance was not directly proportional to the number of bolts (100%). The 4B specimens also demonstrated higher deformability and joint rotations compared to the 2B specimens, indicating improved ductility. This improvement can be attributed to the post-critical behaviour of the macro-component, where the failure of bolts in the 4B configuration occurred progressively and in a staggered manner.

The failure mechanism of the 4B specimens was found to be significantly more complex and influenced by various geometric and mechanical factors such as end-plate characteristics, bolt diameter, and distance between bolts. After the first bolt failure, the end-plate played a crucial role in delaying yielding, resulting in a reserve of resistance and ductility. The experimental results revealed a difference in the yield mechanism, with the presence of outer bolts leading to a more intricate progression of bolt failure, starting from the inner rows towards the outer ones.

In comparison with the precursory numerical study from chapter 3, although the absolute values cannot be confirmed due to the differences in material characteristics, several general tendencies have been confirmed, such as:

- The bolt failure patterns obtained in the numerical study are consistent with the experimental observations;
- The 4B configurations present significant increase strength, specifically F_{max}, depending on the rigidity of the end-plate;
- Important gains in ductility where indeed observed in components with a t/d ratio resulting in mode 2 towards 3;

Based on the findings of the study, it can be concluded that the addition of outer bolts-per-row improves the post-elastic behaviour of steel beam-column joints with extended bolted end-plates. This improvement is attributed to the dynamic redistribution of joint forces, which is influenced by the joint's geometric and mechanical parameters and the observed partial failure modes. The main advantage of incorporating additional bolts lies in the enhanced ductility and maximum strength of the joint.

5. NUMERICAL STUDIES

The experimental results provided by chapter 4 provide valuable insights into the response and performance of the 4B ed-plate tensile macro-components, particularly in comparison to the traditional two-bolts-per-row (2B) configurations. The investigation also showed the influence in behaviour of various parameters, such as bolt diameter, end-plate thickness, flange width or the bolt arrangement.

To further enhance the understanding in the behaviour of the tested specimens and explore the influence of other different parameters, one delves into the numerical modelling of the considered connections using advanced 3D Finite Element Analysis (FEA). Numerical modelling offers several advantages in design and research, including the ability to simulate complex behaviour, investigate various design scenarios, detail analysis of the modelled elements and predict the response of structural elements under different loading conditions.

This chapter aims to present the FEA numerical models developed to simulate the behaviour of the experimented specimens. The numerical models were constructed using ABAQUS software packages [17] capable of accurately represent the mechanical response of the steel connections. The FEA models were calibrated and validated against the experimental results to ensure their accuracy and reliability.

5.1. **Parametric study**

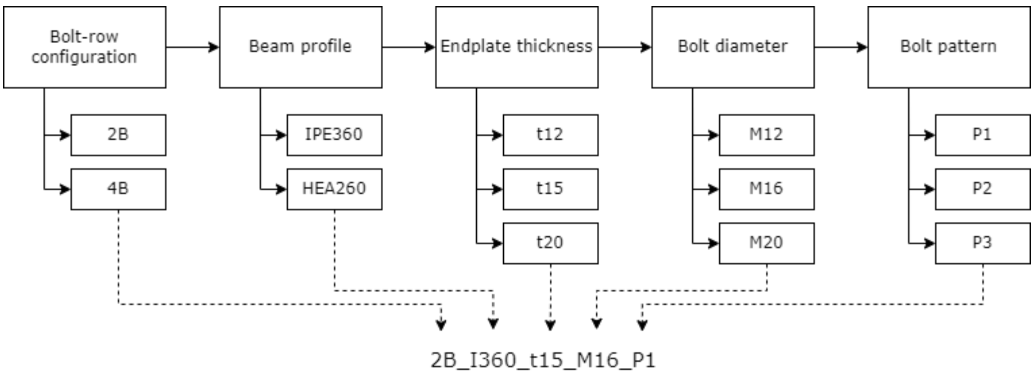


Fig. 5-1 Variables for the parametric study

To gain a deeper understanding of the behaviour of the new configuration, several relevant variables were identified and systematically modified. This approach aimed to investigate the impact of these variables on the overall behaviour of the connection through finding their influence on key aspects of the performance of the macro-component. Fig. 5-1 provides a visual representation of the variables parameters that were selected for modification, enabling a clearer understanding of their significance in the study:

- Number of bolt-rows: 2/4
- Beam profile: IPE360/HEA260
- End-plate thickness: 12mm/15mm/20mm
- Bolt diameter: M12/M16/M20
- Bolt pattern: P1=53x80x53/P2=63x80x63/P3=53x100x53

To validate the FEA models, the simulated results were calibrated against the experimental ones presented in the previous chapter. The force-deformation curves, with special focus on the maximum force, and deformations at critical points, were evaluated. The comparison between the FEA predictions and experimental results allowed the assessment of the accuracy of the numerical models in capturing the overall behaviour and response of the connections (Fig. 5-2 and Fig. 5-3).

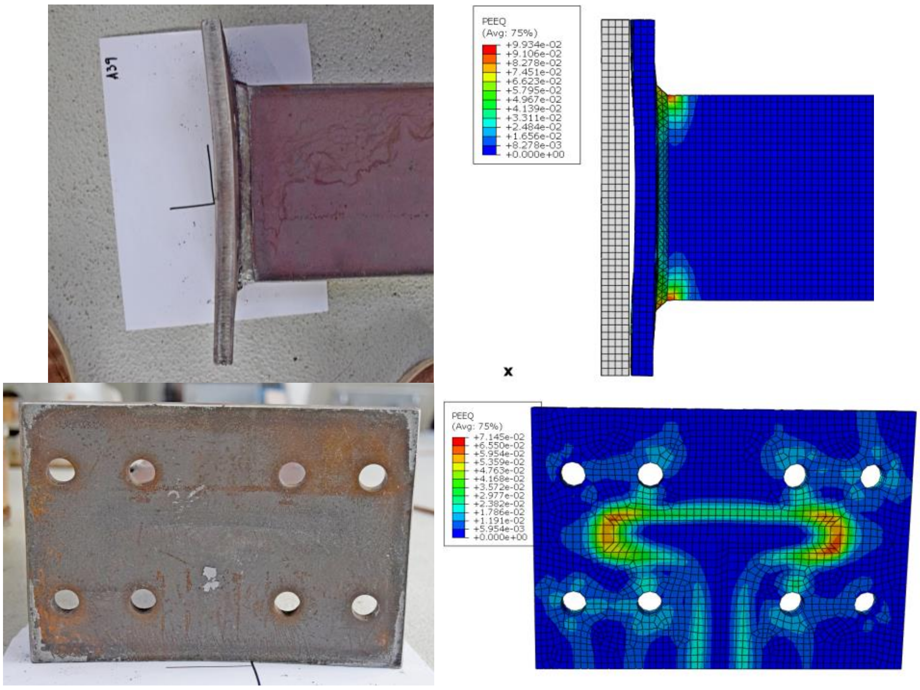


Fig. 5-2 Comparison between experimental and numerical results for 4B_P3, with recorded PEEQ

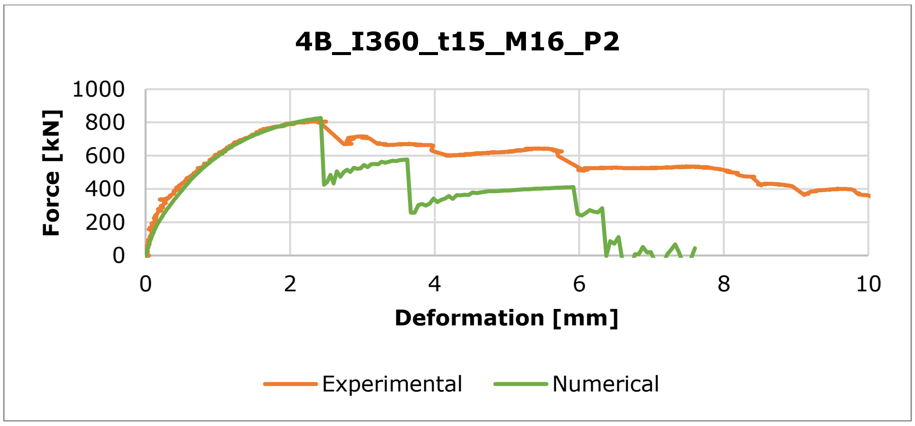


Fig. 5-3 Force-deformation 4B_I360_t15_M16_P2 obtained in numerical validation

Taking into consideration the spectral nature of the failure modes (where mode 2 can be closer to mode 1 or mode 3), the indicative $\delta_{2\text{-}3}$ is calculated, ilustrating how close mode 3 is, comparing to mode 1:

$$\delta_{2-3} = \frac{|F_{T.2.Rd} - F_{T.1.Rd}|}{|F_{T.2.Rd} - F_{T.3.Rd}| + |F_{T.2.Rd} - F_{T.1.Rd}|} \quad (5\text{-}1)$$

In the given context, when referring to a value near 0%, it indicates a tendency towards mode 1, while a value near 100% suggests a tendency towards mode 3. Specifically, a value close to 0% signifies a dominant occurrence of mode 1 behavior, while a value near 100% indicates a predominant occurrence of mode 3 behavior.

As mentioned before, the models with stiffer end-plate showed significantly better results in terms of strength. In Fig. 5-4, the evolution of the yield and maximum forces is correlated with the closeness to either failure mode 1 or 3. For specimens which retain characteristics with tendency towards mode 1, the gains between 2B and 4B are small, or even negative. In the other side of the spectrum, however, both strength values register a clear increase for the cases tending towards failure mode 3.

The ductility of the tensile macro-components serves as a crucial factor in the design of semi-continuous connections to accidental loads. The ability of these components to undergo plastic deformation allows them to absorb energy and redistribute forces within the structure. This deformation capacity plays a significant role in enhancing the overall structural performance and resilience.

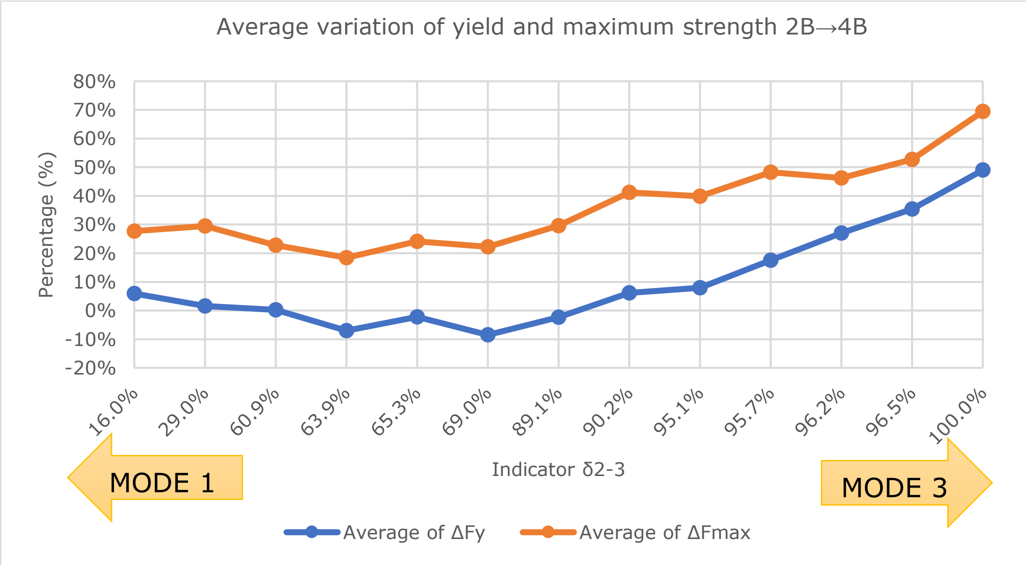


Fig. 5-4 Average 2B-4B gain in yield and maximum strength in relation to mode tendency

In the extreme loading scenario, where the robustness of the structure must be considered, the post-critical behaviour of the connection may be decisive in avoiding a partial total collapse. As observed throughout this study, the 4B configuration consistently registered significant resilience after first bolt failure by comparison with the 2B variants.

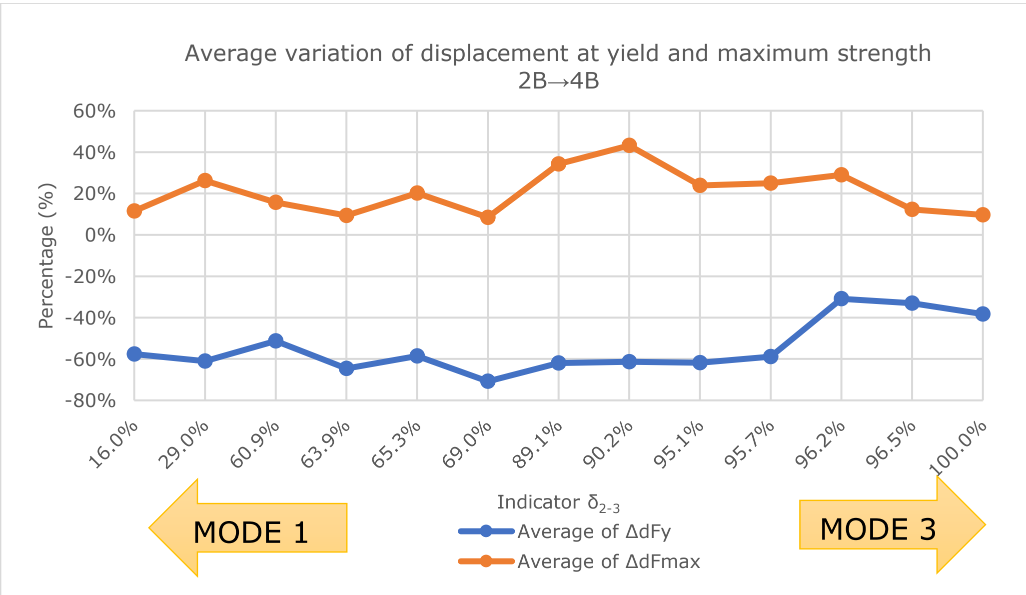


Fig. 5-5 Average deformation gains 2B-4B at yield and maximum strength

In Fig. 5-6 the average variation of deformation at 20% drop of maximum load from 2B to 4B ($d_{80\%Fmax,4B}$/ $d_{80\%Fmax,2B}$) is drawn against the indicator $\delta_{2\text{-}3}$, for each beam profile. In the case of IPE360 profile, the connection presents a loss in resilience. Equally significant is the considerable increase in resilience for models approaching mode 3 in comparison with the 2B configuration.

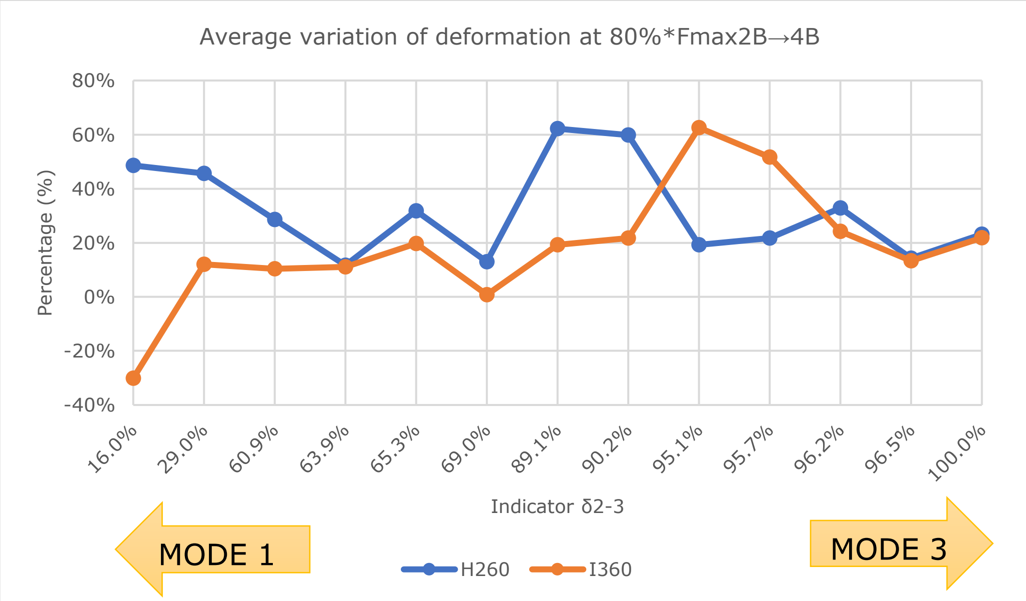


Fig. 5-6 Average variation of deformation at 80%F_{max} after first failure 2B-4B

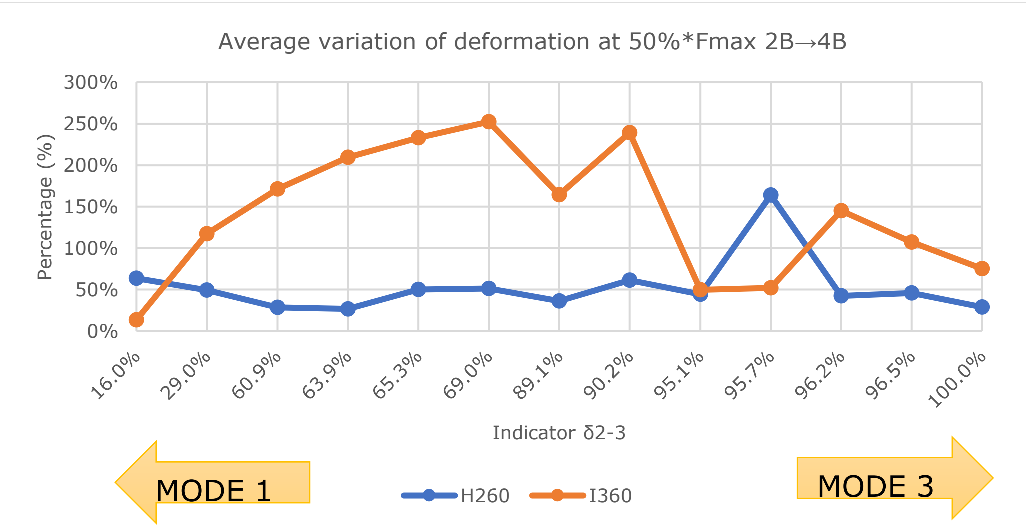


Fig. 5-7 Average variation of deformation at 50%Fmax after first failure

In regard to beam profile HEA260 the gains (Fig. 5-6) are general comprised between +11.7% and +31.7% with two exceptions: (i) For models close to mode 1, the models reported gains of up to +48.5%, in contrast with their IPE360 counterparts; (ii)For models with $\delta_{2\text{-}3}$ between 89% and 91%, there was registered an increase up to 61%.

Considering the ultimate force to be at a lower level – 50% - in relation to the maximum force, the ductility/resilience result change considerably (see Fig. 5-7). The models with the wider flange (HEA260 profile) registered a regular increase throughout the spectrum, with some exceptions.

In the case of the narrower flange (IPE360 profile), the recorded results are much more expressive: the increase in deformation progressively increases to +252.4% for models at $\delta_{2\text{-}3}$=69.0%. It then progressively decreases to +75.2 for model in mode 3.

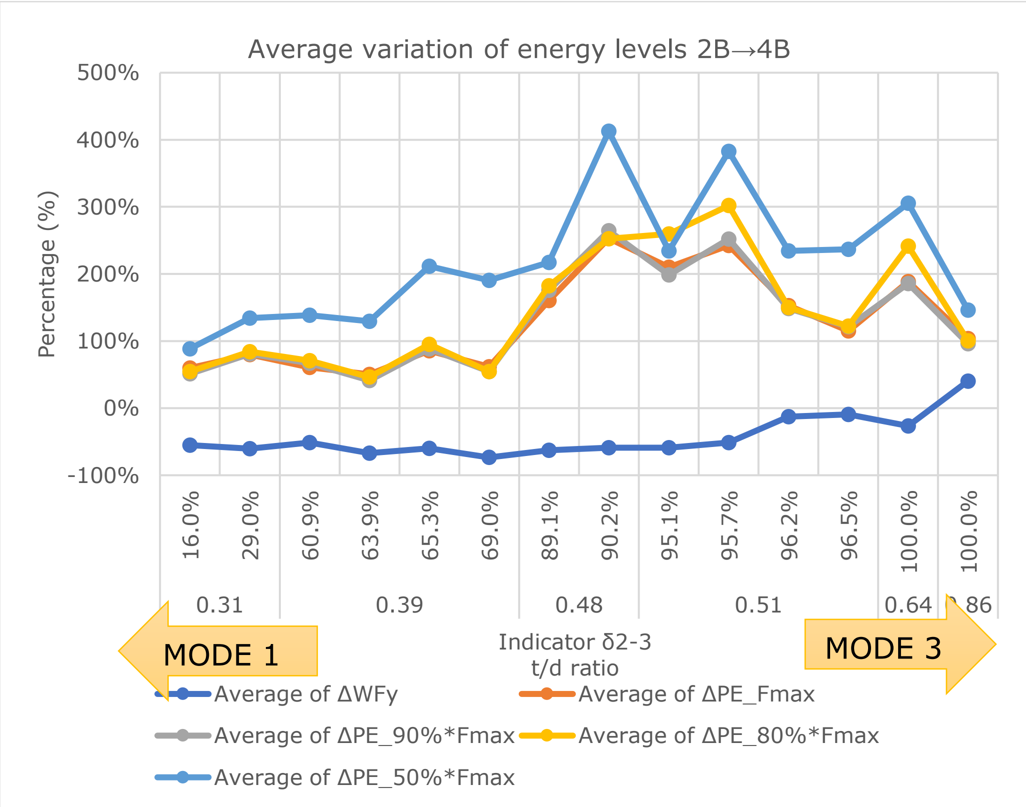


Fig. 5-8 Energy level as work at yield point (Wy) and as plastic strain energy (PE) for Fmax, 90%Fmax, 80%Fmax and 50%Fmax

Lastly, in order to assess the overall performance of the connections regarding both strength and ductility, the level of mechanical work was computed for each POI:

$$W_i = \int_0^{\delta_i} F d\delta \qquad (5\text{-}2)$$

where

δ_i is the deformation at each POI

In order to obtain the level of dissipated energy by the connection, the cumulative plastic strain energy (PE) was calculated by removing the mechanical work performed during the elastic phase:

$$PE_i = W_i - W_y \qquad (5\text{-}3)$$

In Fig. 5-8, the average variation between 2B and 4B for the values of the mechanical work at yield point (W_y) and PE for Fmax, 90%Fmax, 80%Fmax and 50%Fmax are plotted against the indicator $\delta_{2\text{-}3}$ and the t/d ratio.

5.2. **Rotational capacity**

Following the Eurocode provisions, a map of the observed ductility from the 4B models from numerical study can be drawn against the t/d ratio (according to the EN1993-1-8), illustrating the thresholds specified in sections 6.2.7.2(9) and 6.4.2(2) for "unlimited rotation" and "insufficient rotation" (Fig. 5-9).

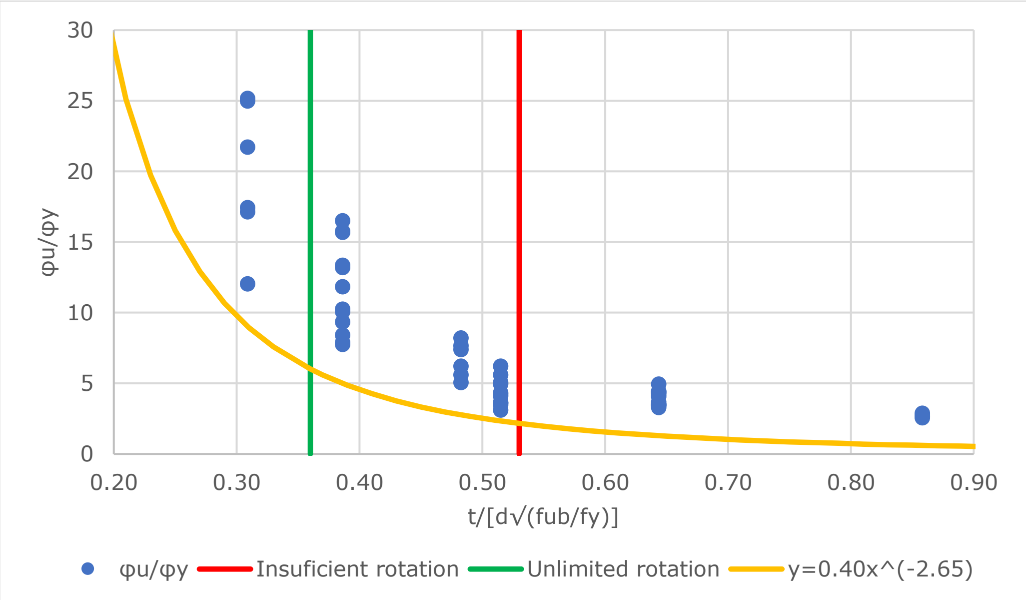


Fig. 5-9 Regression curve for minimum values plus 25% safety reduction for ductility at φ_u proportional to t/d ratio

Despite the EN1993-1-8 not giving specific values for rotational capacity, the numerical data confirms the trend of loss of ductility for higher t/d ratios and exponentially better ductility for smaller t/d ratios. This allows for an empiric approach to delineate an acceptable threshold in function of the t/d ratio, through the charting of a polynomial regression. In Fig. 5-9, a polynomial curve defines the minimum envelope of the ductility obtained from numerical analysis, with a safety coefficient of 1.25, analogous to the partial coefficient for resistance of connections γ_{M2}. Thus, the relation between the relative ductility and the t/d ration can be expressed as:

$$\frac{\theta_u}{\theta_y} = 0.40 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-2.65} \qquad but \qquad \frac{\theta_u}{\theta_y} < 9 \qquad (5\text{-}4)$$

where
f_y is the yield strength of the end-plate
f_{ub} is the ultimate strength of the bolts

For small t/d values, the values for relative ductility should be limited to the minimum obtained value, plus the safety margin.

Similarly, the same approach can be used to set a threshold for the ultimate stages of the post-critical phase: 90%*F_{max}, 80%*F_{max} and 50%*F_{max}. The ductility limit can then be summarized using the following expressions:

$$\frac{\theta_{max90}}{\theta_y} = 0.43 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-2.62} \qquad but \qquad \frac{\theta_{max90}}{\theta_y} < 9.35 \qquad (5\text{-}5)$$

$$\frac{\theta_{max80}}{\theta_y} = 0.45 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-2.62} \qquad but \qquad \frac{\theta_{max80}}{\theta_y} < 9.86 \qquad (5\text{-}6)$$

$$\frac{\theta_{max50}}{\theta_y} = 0.48 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-3.04} \qquad but \qquad \frac{\theta_{max50}}{\theta_y} < 17.15 \qquad (5\text{-}7)$$

5.3. **Influence of flange width variation in 4B configurations**

While the results from the experimental program generally indicated an overall improvement in the characteristics of the connections, the specific gains between specimens with different configurations varied depending on the changed parameter. In other words, the impact of the altered parameter differed across the various configurations, leading to varying degrees of improvement.

In the case of the variation of the width of the beam's flange, the observed behaviour of the end-plate was significantly different (Fig. 5-10): while in the case of the HEA260, the end-plate developed a final yield pattern similar to the predicted non-circular patterns from Demonceau, J-F., in the case of the IPE360 beam, the end-plate presented a marked plastic yield in its lateral extension, presenting a notable warping along the vertical line of the edge of the flange.

Fig. 5-10 End-plate deformed shape for HEA260 (left) and IPE360 (right)

Given the observed differences, further investigation is necessary to gain a clearer understanding of the impact of the flange width on the overall behaviour of the connection. In line with this objective, a supplementary parametric study was conducted, involving a progressive widening of the flange width from an original width b=150mm (characteristic for an IPE300 profile to a value of 240mm. This study aimed to systematically analyse and evaluate the effects of varying the flange widths on the connection's performance.

Regarding the deformation at key points such as F_y, F_{max}, and F_u (Fig. 5-11), each model exhibits a slightly different pattern with the widening of the flange width. Both d_{Fmax} (deformation at maximum force) and d_{Fu} (deformation at ultimate force) demonstrate a clear decrease as the flange widens, irrespective of the end-plate thickness (Fig. 5-12). This decrease can be attributed to the stiffening effect of the wider flange on the end-plate, which reduces the number of yield lines and subsequently limits the overall ductility of the connection. On the other hand, the deformation at which F_y occurs (d_{Fy}) shows only slight changes, but with an increasing trend for the t12 end-plate and a decreasing trend for the thicker t15 end-plate.

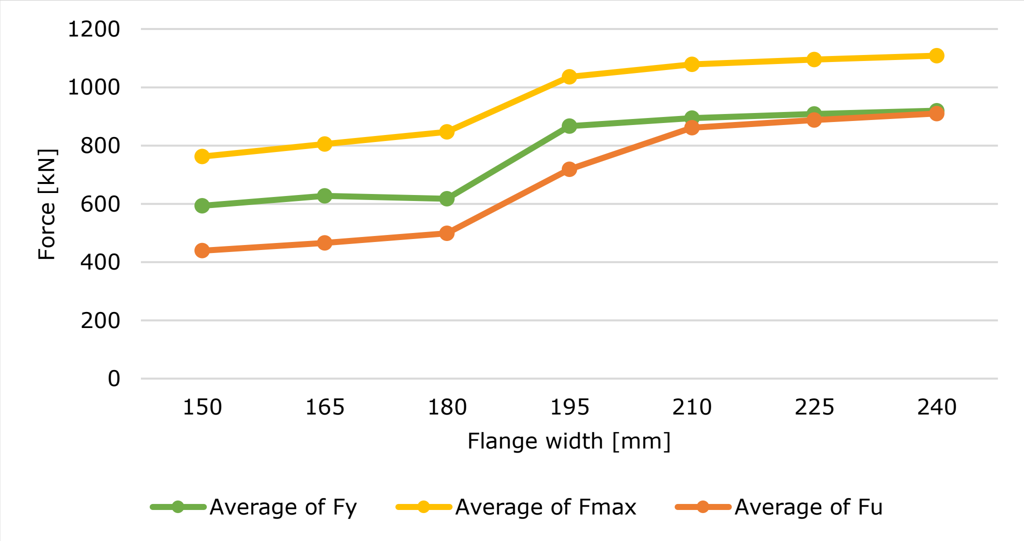


Fig. 5-11 Evolution of F_y, F_{max} and F_u with the width of the flange

The evolution of the presented force and deformation values is presented in the Fig. 5-11 and Fig. 5-12. In the diagrams, both the force and deformation curves exhibit a noticeable change in slope around the 180mm to 195mm mark. This corresponds to the total width of the bolt group, which is 186mm (b_{bg}). The abrupt slope change indicates a transition point where the behaviour of the connection is influenced by the relative position of the flange edge and the position of the outer bolts.

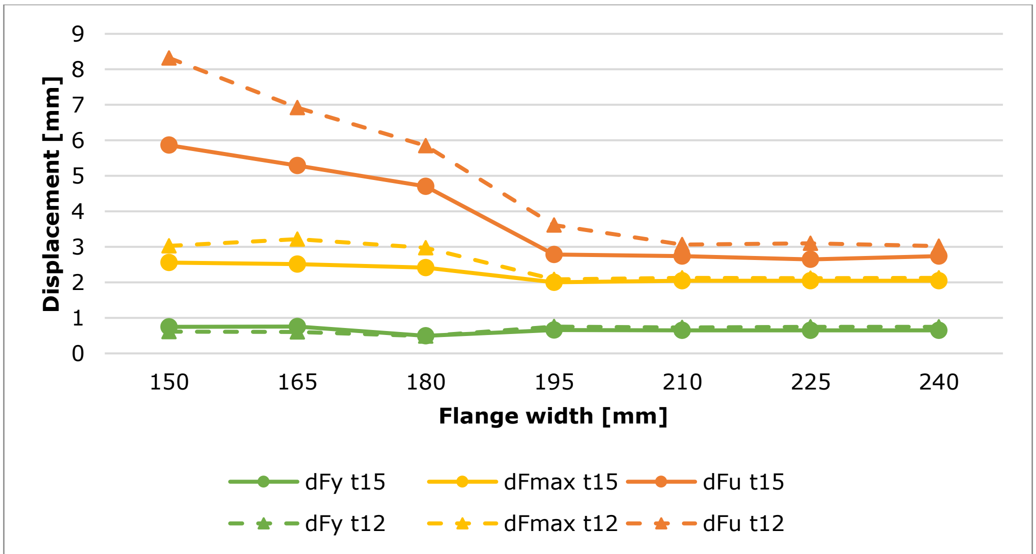


Fig. 5-12 Evolution of dF_y, dF_{max} and dF_u with the width of the flange

5.4. **Validity parameter definition**

Several parameters have a direct impact on the behaviour of 4B connections. The following guidelines should be taken into consideration when designing with such a configuration:

- In function of the desired objective, different widths of the beam flange lead to different behaviours. While a beam flange wider than the bolt group width b_{bg} assures a better outer bolt contribution and higher strength results, it also reduces ductility and a beneficial post-critical behaviour; The inner bolts should be placed inside the limits of the flange width;
- The values of the t/d ratio and the δ_{2-3} indicator also reflect different behaviour tendencies. While high values of $t/d > 0.51$ and $\delta_{2-3} > 95\%$ assure a good strength output, for higher ductility, t/d values between 0.47 and 0.49 and δ_{2-3} between 88% and 91% give optimal results for wider beams ($b \geq b_{bg}$) and t/d values between 0.50 and 0.52 and δ_{2-3} between 94% and 96% give optimal results for narrower beams ($b < b_{bg}$);
- The assumed rotational capacity at maximum force and following post-critical stages could be safely estimated as:

$$\frac{\theta_u}{\theta_y} = 0.40 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-2.65} \quad but \quad \frac{\theta_u}{\theta_y} < 9 \qquad (5\text{-}5)$$

$$\frac{\theta_{max90}}{\theta_y} = 0.43 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-2.62} \quad but \quad \frac{\theta_{max90}}{\theta_y} < 9.35 \qquad (5\text{-}6)$$

$$\frac{\theta_{max80}}{\theta_y} = 0.45 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-2.62} \quad but \quad \frac{\theta_{max80}}{\theta_y} < 9.86 \qquad (5\text{-}7)$$

$$\frac{\theta_{max50}}{\theta_y} = 0.48 \cdot \left(\frac{t}{d}\sqrt{\frac{f_y}{f_{ub}}}\right)^{-3.04} \quad but \quad \frac{\theta_{max50}}{\theta_y} < 17.15 \qquad (5\text{-}8)$$

- In all the design process, the welds should be designed to sustain the potential stress concentration points, specifically at the edges of the flange, where force is transferred to the outer bolts;

5.5. **Conclusion**

In terms of strength and behaviour, the 4-bolt-per-row (4B) configuration consistently exhibited higher maximum force (F_{max}) values compared to the 2-bolt-per-row (2B) configuration. This effect was particularly pronounced when the 4B configuration was paired with wider beam profiles and stiffer end-plates in relation to the bolts. Notably, connections with stiffer end-plates demonstrated significantly improved strength and post-critical behaviour. However, wider beam profiles, while contributing to higher strength values, showed limitations in terms of post-critical ductility. Additionally, the use of larger bolt diameters led to early plastic deformation in beam flanges, resulting in decreased overall yield strength.

Regarding ductility through deformation, the study found that the deformation values at both yield and maximum strength varied among different configurations. Generally, the 4B configuration exhibited higher deformation, indicating enhanced ductility. Combining stiffer end-plates with smaller bolt diameters resulted in higher ductility values and improved post-critical behaviour. The wider beam profile typically contributed to improved ductility, except in cases where a wider bolt distribution reduced the connection's post-critical reserve strength. Following a statistical analysis, expressions for the three stages of post-critical ductility are put forward.

Overall, the findings underscore the intricate interplay between various parameters in the design of bolted connections. While the 4B configuration offers enhanced strength, achieving an optimal balance between strength and ductility but necessitates careful consideration of end-plate stiffness, beam profile, and bolt distribution. The results suggest that configurations favouring a mode 2 tendency provide a favourable compromise between strength and post-critical behaviour.

The evolution of the results in relation to the indicative $\delta_{2\text{-}3}$ is significant. Configurations showing a $\delta_{2\text{-}3}$ value between 89.1% and 96.2%, indicating a mode 2 leaning towards mode 3, exhibited better outcomes in terms of both strength and ductility. The $\delta_{2\text{-}3}$ ratio appears to serve as an important indicator for achieving a desirable balance between strength and ductility across different configurations.

The experimental program and numerical simulations revealed several key findings regarding the influence of flange width on yield strength, maximum strength, deformation, and failure modes:

- The behaviour of the end-plate showed significant differences for HEA260 and IPE360 beams. The wider end-plate of HEA260 profile exhibited a yield pattern consistent with predicted non-circular patterns, while the IPE360 profile end-plate experienced marked plastic yield and warping along the vertical line of the flange edge. This indicates that flange width affects the deformation characteristics of the end-plate.
- The numerical models demonstrated that a systematic increasing the flange width from 150 mm to 240 mm resulted in higher values for capacity factor, yield strength and maximum strength, but this is joined by decrease of total deformations at maximum force and ultimate force decreased due to the stiffening effect of the wider flange on the end-plate.
- The models with a more dispersed bolt group (relative to the flange) are limited by the material constraints of the welded region. Despite their potential for further deformation, the maximum achievable ductility is restricted by the limitations of the welded area.

6. ROBUSTNESS OF MRF STRUCTURES WITH RESILIENT CONNECTIONS

The adoption of a resilient configuration for the connection may prove to be an adequate solution for exceptional design situations in which added robustness is important. In this chapter a robustness analysis is performed on a multi-story building in a classic design with no specific provisions for accidental situations other than low seismicity. The analysis is broadened by the variation of several parameters like the beam type and the connection configuration.

The adoption of a resilient configuration for the connection may prove to be a reasonable solution even in design situations where the design strength of the connection is higher than the resistance of the connected members. Even though in the design phase, the strength of the connection may seem safe above that of the beams, in reality, there are several situations in which the real strength of the beam may be higher than that of the connection:

i. Discrepancies in material overstrength (excessive in the beam and/or deficient in the connection components);
ii. Unwarranted lateral restraints or excessive rigidity;
iii. Overestimation of the real buckling length
iv. Underestimation of the connection’s rigidity as defined in the numerical model, leading to low bending moment values.

6.1. Robustness assessment

An established method to evaluate the robustness performance of a building is the notional removal of a supporting column. In the design of a new building, specific measures should be applied in order to safeguard it against progressive collapse following extreme accidental action, in function of the building consequence class. In the analysis of the proposed column removal scenario, the structure is expected to have adequate strength and ductility so that alternative load paths can develop and avoid the collapse of the structure.

In the event of a column loss, the adjacent beams (and respective beam-column connections) should be able to both hold the columns in place and enable catenary action to develop. While steel beams sections prove typically ductile to coupled tension and flexure loads, the same is not usually the case for semi-continuous bolted connections. According to Dinu et al. this type of connection exhibits an inadequate behaviour for the correct development of catenary forces.

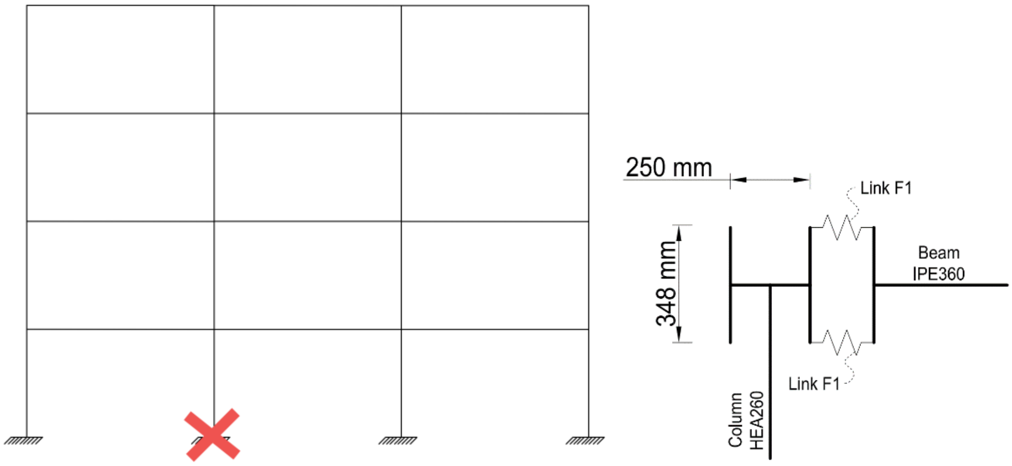


Fig. 6-1 Column support removal and connection layout

In the proposed column-loss scenario, the behaviour of the aforementioned frames is compared, using the classic 2B connection configuration and the alternative four bolts-per-row configuration. The analysis considers a 2D frame analysis, with one of the central columns of the marginal MR frame of the building proposed for removal. In this analysis both the time-dependent dynamic behaviour and geometric non-linearity are taken into account by considering the mass of the material (and associated inertial effects) and the recalculation of the stiffness matrix at each incremental step.

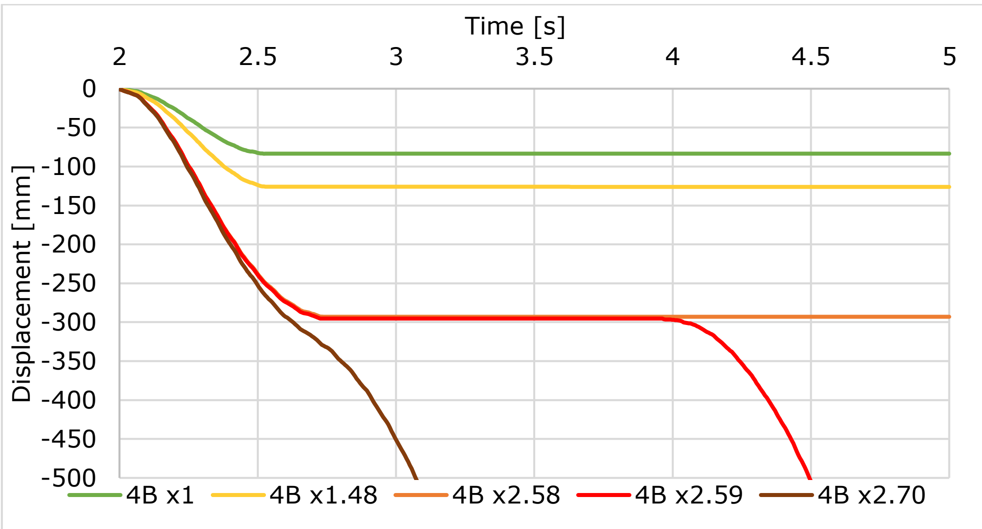


Fig. 6-2 Downward displacement evolution for 4B_HEA260_t15_M20

For each overload value, the vertical displacement at the point of the column removal was recorded and plotted vs. time. The gravitational loading in the exceptional combination was incremented by the overload factor – Ω. The stages considered relevant were the base loading (Ω=1), the point of structural collapse, and three separate points in the connection behaviour for the most deformed: i. the elastic threshold (θ_y); ii. the plastic threshold (θ_u); and iii. the failure threshold (θ_{max}).

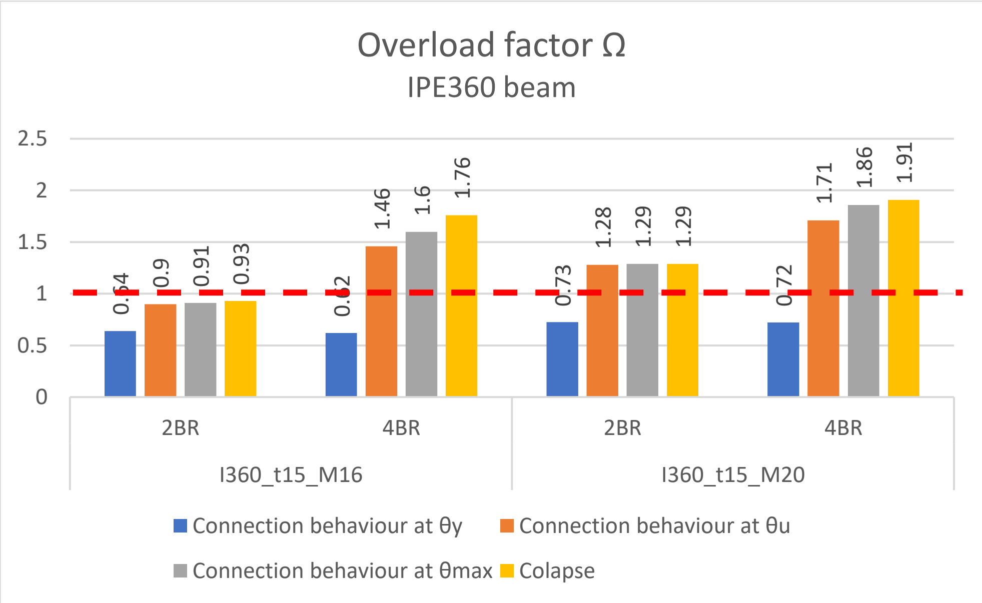


Fig. 6-3 Overload factor values for IPE360 configurations

In Fig. 6-3 and Fig. 6-4 is possible to compare the values of Ω across configurations. Regarding the original IPE360 variant, model 2B_I360_t15_M16, it can be observed that all Ω values are smaller than 1 (represented by the red-dash line), meaning that the structure would collapse under the accidental design loads. In the 4B variant, however, would need an increase of 46% of the design load in order to reach the critical point in the connection, and 76% increase to reach collapse. In the M20 variant, the increase in performance is similar for 2B and 4B configuration, and the 2B variant can now avoid failure of the connection (Ω>1). However, in contrast with its 4B counterpart, it lacks resilience sustaining only an additional 1% of load between the critical point and collapse. In contrast, the 4B variant shows that 20% of the load can be supported after the first yield point before the structure collapses.

The HEA260 original configuration, model 2B_HEA260_t15_M20 presents a reserve of 14% for the first connection yielding and 30% to collapse, which is an important (but limited) ductility reserve. The corresponding 4B configuration, however, sustained an additional 48% of load until critical point, and a considerable 159% reserve until collapse. In the variant with M16 bolts (smaller), the 2B variant collapsed before reaching the design target load (Ω_{pc}=0.88) but its 4B configuration showed an important resilience level: despite having connections reaching the yield point before reaching the design load, the structure was able to sustain an additional 46% of the design load before collapsing.

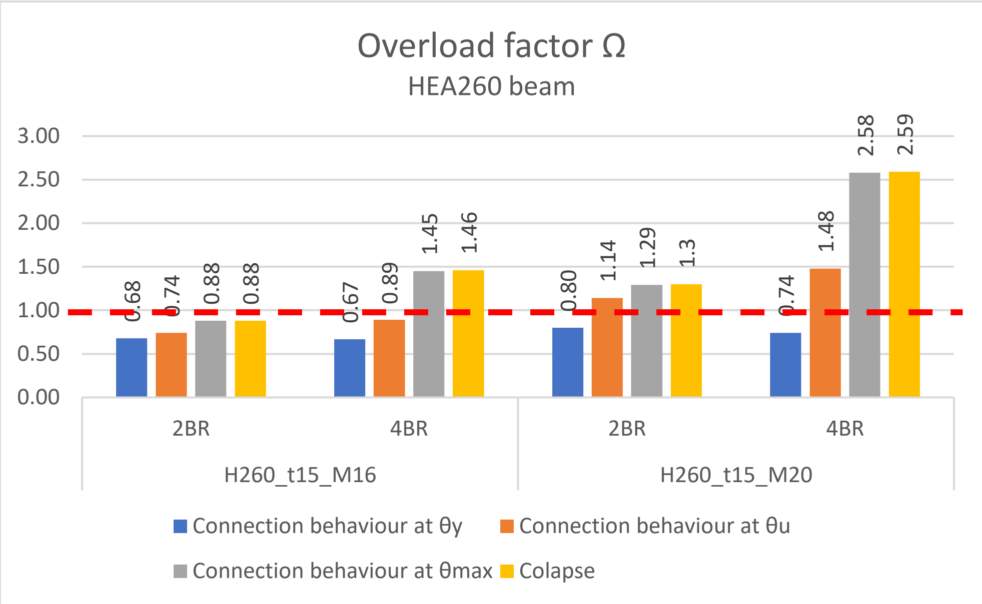


Fig. 6-4 Overload factor values for HEA260 configurations

6.2. **Conclusions**

In the elastic phase, both original models considering connections with 2B configuration and end-plate thickness of 15mm 2B_t15_M16 configuration present subunit load multiplier values, indicating the lack of strength of an optimized connection design to allow the transfer and redistribution of forces within the structure and thus avoiding its collapse. This was also the case for the 4B_H260_t15_M16 configuration, even though it sustained 20% more load than its 2B counterpart. Also in this elastic phase, even though the 4B configuration consistently presents higher values, of up to 62.2%, the difference is small compared to post-elastic behaviour.

In the plastic phase (θ_u) the influence of the added exterior bolt rows is much more pronounced, providing the structure with a significant strength and ductility reserve, increasing up to 249% comparing to the elastic phase, while the classic solution only allows for an increase of maximum 78%. The difference between 2B and 4B is more pronounced in the case of HEA260 beam where the wider flange allows for end-plate added rigidity and a better distribution of the forces towards the outer bolts.

In the post-critical phase (θ_{max}) the gains remained high, at 248% for the 4B comparing to 161% for the 2B. However, the relative gains between post-critical point and failure point were very limited in the 4B_HEA260 variant (1%) comparing to the 4B_IPE360 (16%). The reserve between failure of a connection and the structural collapse was insignificant, signalling the sensitivity of the structure to a failed connection in an already element-loss scenario.

After the first failure, the retained strength reserve in the connections have a reduced impact in the overall result. But while the effect of this reserve is inexistent in the case of 2B connections, in the particular case of 4B configuration with narrow beam profiles (IPE360), this reserve brings a small but significant contribution on the structural behaviour. In this case, the more deformable end-plate allows for a more effective redistribution of the forces between bolts after the first bolt failure occurs. This can add up to 16% to the load multiplier value and further delay the event of a progressive collapse.

The performance of connection with a 4B configuration in a complex context of a structure can add important levels of resilience to the structure overall. The performance indicators for these connections are tightly linked to their t/d ratio, providing enhanced performance within specific values, depending on geometrical characteristics of the connection components.

7. FINAL CONCLUSIONS AND CONTRIBUTION

In general, the 4-bolt-per-row (4B) configuration consistently exhibited higher strength, particularly with wider beams and stiffer end-plates. For HEA260 beams, this increase was consistent, while for IPE360 beams, it varied for different failure modes. The yield strength differences between 2B and 4B configurations were notable, especially with smaller bolts and thicker end-plates. Stiffer end-plates improved strength and post-critical behaviour, but wider beam profiles showed limitations in post-critical ductility. Larger bolt diameters led to early plastic deformation on the interface between beam profile and end-plate.

As a general trend, the end-plate deformation values at yield for 4B configuration were smaller than in case of 2B systems, indicating an increased stiffness in the elastic phase. The deformation at maximum strength varied among configurations, with the 4B configuration generally exhibiting higher ductility. Achieving an optimal balance between strength and ductility requires careful consideration of end-plate stiffness, beam profile, and bolt distribution.

Contextualizing in the Eurocode design frame, configurations favouring a mode 2 towards 3 provided a favourable compromise between strength and ductility. Flange width variation in connection design also influences the end-plate deformation characteristics: systematically increasing flange width enhances capacity factor, yield strength, and maximum strength but reduces total deformations due to the stiffening effect on the end-plate, the relation between width of the flange and width of the bolt group being key in the behaviour transition. Configurations with more dispersed bolt groups are limited by material constraints in the welded region, restricting ductility.

In general, from the extended numerical and experimental studies performed, two main conclusions can be drawn:

- For 2B connection configurations characterized by a prevalent failure towards mode 1 and 2, no notable differences are observed between both 2B and 4B configurations in the elastic phase. In the plastic and post-critical phase however, significant performance improvements are registered, both in terms of strength and in terms of ductility, by successive and significant strength recoveries.
- For 2B connection configurations characterized by a prevalent failure akin to mode 3, the 4B configuration allows for a significant increase in strength, while the failure mechanism remains brittle, characterized by the failure of the bolts, presenting low levels of ductility.

Finally, several design guidelines are outlined in order to aid the configuration of a ductile 4B connection.

The robustness response and performance of a MRF structure was also analysed under a column-loss scenario, specifically focusing on the behaviour of beam-to-column connections. Two selected configurations of connections, t15_M16 (t/d ratio of 0.48) and t15_M20 (t/d ratio of 0.38), were examined for their strength and ductility capacities. In the elastic phase, both original models (2B) showed suboptimal load transfer capabilities, leading to structural vulnerability. Despite the 4B_H260_t15_M16 configuration sustaining more load than its 2B counterpart, the difference was limited.

In the plastic phase, the addition of exterior bolt rows significantly enhanced strength and ductility reserves, particularly in the 4B configuration. The HEA260 beam variant demonstrated a more pronounced difference due to its wider flange. In the post-critical phase, the gains in strength remained high, especially for the 4B configuration. However, the sensitivity to failed connections became evident, with limited reserve between connection failure and structural collapse.

After the first failure, the retained strength reserve had a reduced impact overall, but the 4B configuration, especially with narrow beam profiles (IPE360), demonstrated a slight contribution to structural resilience. The deformable end-plate facilitated effective force redistribution, potentially delaying progressive collapse by up to 16%. Overall, the performance of connections with a 4B configuration contributed significantly to the structural resilience, with performance indicators closely tied to the t/d ratio within specific geometric values.

7.1. **Further research**

In light of the results obtained in the present study, several research paths have been opened linked to connection with four bolts-per-row:

- The behaviour of individual bolts and welds play a crucial part in the post-critical behaviour of a connection. In the specific type of connection studied in this thesis, the accurate prediction of its post-critical behaviour is a necessity if they are to play a role in avoiding partial or complete structural collapses. The type and timing of the failure of the bolts needs to be clarified in situation where important deformations of the end-plate apply an eccentric axial force in the bolt. Also due to the significant role of the end-plate's stiffness in the behaviour of the connection, the behaviour of the welds to bending moment and torsion should be investigated, especially around points of stress concentration and geometric discontinuity;
- The macro-component approach, already adopted by several other researchers, requires experimental validation in the context of full-scale connections;

- The harmonization with the current design standards, although, initiated, still needs further definition, especially regarding the limits of its application, such as the width of the flange, t/d ratio correlation and limitation or technologic impositions such as welding demands;
- The reliant method for evaluating the rotational capacity is central to the correct prediction of the connection's behaviour in extreme loading conditions. Starting with the insufficiencies of the present standards, a dependable approach needs to be developed in order characterize the post-critical behaviour of steel connection for application in design. The expressions put forward in this regard need to be confirmed by a significant range of experimental testing;
- A fastener assembly with two nuts instead of one can reduce the probability of thread failure, which may further increase the resilience of the 4B connections, avoiding a brittle failure which can cause undesirable dynamic effect.

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