An innovative mechanical model for structural steel joints

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Abstract

An innovative mechanical model for the characterisation of structural steel joints is presented in this paper. The model relies on the so-called "component method" to simulate the joints through an assembly of extensional springs and rigid elements. The proposed approach solves the inconsistencies met in the existing spring models and allows bypassing the analytical assembly procedure prescribed by the current version of Eurocode 3 for the prediction of joints' behaviour. Key features systematically disregarded in classical spring models, i.e., the group effects and the variation of shear forces on the height of the column web panel, are incorporated in the proposed mechanical model. The latter allows predicting the behaviour of steel joints respecting the European normative provisions yet based on mechanical criteria only. Comparisons with experimental and analytical results show the model's code compliance and prove its accuracy in characterising the actual behaviour of structural joints.

Keywords: Steel and steel-concrete composite structures, Structural joints, Mechanical models, Component Method, Group effects, Eurocode 3

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1. Introduction

In steel and steel-concrete composite frame structures, the joints were conventionally considered to behave either as ideally pinned or fully rigid. In reality, most of structural joints exhibit neither a zero nor an infinite stiffness, hence falling into the semi-rigid category. This implies structural analyses in which joint properties are influencing not only the displacements, but also the magnitude and the distribution of internal forces. Accordingly, the behaviour of joints may significantly influence the overall structural response in terms of resistance, deformability, and ductility. In this regard, significant research effort was undertaken to propose methods for predicting the behaviour of structural joints. This led eventually to the development of analytical [1,2], numerical [3–7], and mechanical [8,9] models with different levels of complexity and reliability.

In Europe, the lack of relevant and clear normative directives for the design of semi-rigid joints emphasized the need for a generalised and practical design approach leading consequently to the development of the "component method" currently included in Eurocode 3 Part 1-8 (EC3) [10]. The method uses a hybrid analytical-mechanical approach to derive the rotational behaviour for a wide range of joint configurations. It relies on a mechanical model comprising a set of extensional springs interconnected by rigid links, thereby simulating the joint through an assembly of basic components. Each of these components represents a discrete joint part with intrinsic mechanical properties (i.e., strength and stiffness). The individual response of all active basic components is correlated through an analytical assembly procedure used to estimate the joint's resistance and stiffness characteristics. The result is an idealised moment-rotation curve

($M_j - \phi$) which is typically assigned to rotational springs used to simulate the behaviour of joints in structural analyses.

Nonetheless, the EC3 approach covers cases for which the axial force acting on the joint does not exceed a threshold set at 5% of the axial plastic resistance of the connected beam cross-section. It is assumed that axial forces below this limit have a negligible influence on the joints' rotational behaviour and so, the *M-N* interaction is disregarded. This assumption is reasonable for structural analyses of rectangular multi-storey framed buildings in which, for usual design situations, the beam-to-column joints are mainly subjected to bending moments and shear forces. Experimental observations revealed however that the 5% rule could lead to significant overestimations of the joints' resistance [11,12], the 5% limit being seemingly set without a solid scientific background. Moreover, the increasing concern over the robustness of buildings in accidental situations (e.g., a sudden column loss) highlighted the fact that, once catenary actions develop in the beams bridging over the damaged area of the structure, the beam-to-column joints may gradually shift from a bending-predominant to an axial-predominant loading [13–15]. In this case, the *M-N* interaction must be thoroughly considered in the structural analysis, yet the simplified joint modelling with rotational springs does not accommodate appropriately this feature.

Mechanical models incorporating all potentially active joint components could overcome the limitations of the analytical procedure proposed in EC3. Variances of models that account for the *M-N* interaction were previously introduced in [12,16,17]. Some researchers went even a step further and integrated numerically such mechanical models in global structural analyses [18]. Despite being undoubtably more advanced than the simplified joint modelling with rotational springs, this approach is however inconsistent with the current knowledge on the behaviour of joints since, in classical mechanical models, three key features affecting the response of joints are systematically disregarded, i.e., the group effects, the component interactions, and the effective height of the column web panel in shear.

In EC3, these issues are addressed through reduction factors at the level of component characterisation and analytical procedures used to assess the joint behaviour. For instance, the group effects that may occur in plate components subjected to transverse bolt forces are dealt with through an analytical verification of the distribution of forces within the connection with respect to the individual or group resistance of the bolt rows.

Regarding the column web panel in shear, the standard approach is to define an effective height based on a stiffness estimation, and the distribution of shear forces over the so-estimated panel height is assumed to be uniform. To account for all the forces acting on the column web panel zone, EC3 introduces a transformation parameter β that correlates the moment transferred from the connection with the shear force acting on the panel. Both stiffness of the column web panel in shear and resistance of other column components nominally active in tension or compression are somehow affected by this parameter. A realistic computation of β is however not straightforward and, as highlighted in [19], the conservative values of β recommended in EC3 could lead to substantial errors in structural analyses. Finally, the value of this parameter could change and so, should be recalculated at each loading step, which means that the behaviour laws of the springs simulating the components affected by this parameter should be adapted accordingly.

In this paper, the basic mechanical model of EC3 is comprehensively modified such that the behaviour of joints may be predicted regardless of the applied loading while accounting for the group effects and the actual deformability of the panel zone with no need for the β factor. As a result, the proposed model allows predicting the response of structural joints relying solely on mechanical principles while being in full compliance with the analytical approach provided by EC3. The proposed methodology extends the conventional applicability of classical springs models to cover generalised loading conditions, thereby paving the way for the implementation of the model as a macro-element in dedicated software. This gives prospects for reliable and code-compliant structural analyses accounting for the actual response of structural joints with relatively low computational effort.

2. Classical mechanical models for structural joints

2.1 Existing classical mechanical models

In line with the principles of the so-called "component method" [20,21], a joint is mechanically characterised by discretisation into basic individual components. Each component represents a part of the joint identified with respect to the load type acting on it, i.e., components in tension, compression, bending, or shear (see Table 1). The individual components are simulated through extensional springs with a nonlinear behaviour defined by force-displacement curves (F-Δ). The springs are interconnected by infinitely rigid pinned-end elements to form a mechanical assembly in which, under the applied loading, the equilibrium of forces and the displacement compatibility are ensured. The reliability of such models depends thus on both the characterisation of joint components (strength, stiffness, and deformation capacity) and the considered component interconnectivity.

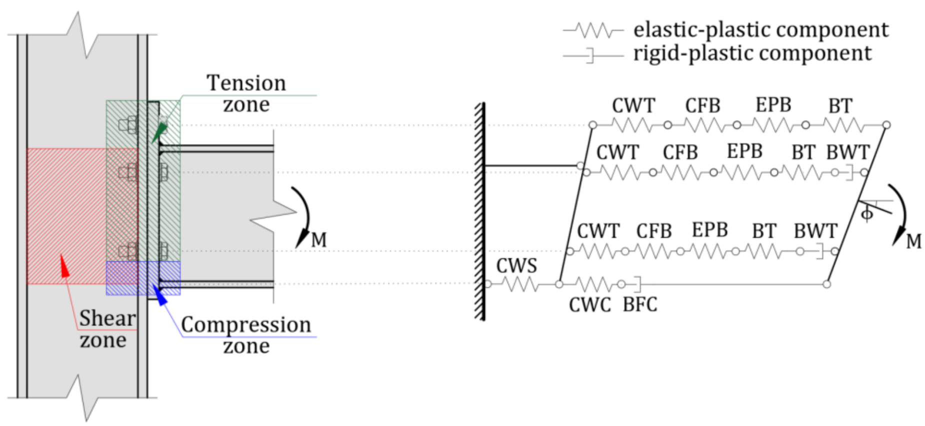

Fig. 1. Weynand et al. [23] (EC3) mechanical model for steel joints

Relying on similar mechanical principles, several mechanical models have been proposed for the characterisation of structural joints [16,22–25]. One of the most popular amongst them is the model proposed by Weynand et al. [23] which, due to its rather simple yet effective modelling approach, was included in EC3 for the stiffness estimation for a wide range of joint configurations; this model is hereinafter referred to as the "EC3 model".

Provided that all the components contributing to the strength and the stiffness of a joint are considered, the use of the EC3 model can be extended to determine the moment resistance as well. As depicted in Fig. 1, for a typical beam-to-column end-plated single-sided joint, such a model accounts for two types of components: elastic-plastic components contributing to both stiffness and strength, and rigid-plastic components affecting the strength only. Under the applied bending moment M, the response of the joint is governed by components associated with three distinctive zones: compression, tension, and shear zones (see Fig. 1 and Table 1). Since the springs simulating a single connection row are subjected to the same force, the model can be simplified further by replacing the series of row springs with an equivalent spring per row as shown in Fig. 8.

Table 1. Typical components in bolted beam-to-column joints

Zone	Component	Abbreviation
Shear	Column web panel in shear	CWS
Tension	Column web in tension	CWT
	Column flange in bending	CFB
	End-plate in bending	EPB
	Bolts in tension	BT
	Beam web in tension	BWT
Compression	Column web in compression	CWC
	Beam flange and web in compression	BFC

The “Innsbruck model” illustrated in Fig. 2a was proposed by Huber and Tschemmernegg [24] as an alternative to the EC3 model. The novelty of the model lies in the fact that the rotation of the connection and of the column web panel can be assessed independently, hence giving a better understanding of the joint deformability and frequently leading to more reliable predictions of the joint behaviour. Nonetheless, an iterative procedure is employed in the derivation of the joint response, and the model provides a complex numerical solution with no simple analytical alternative, which is a heavy limitation for its inclusion in a design code.

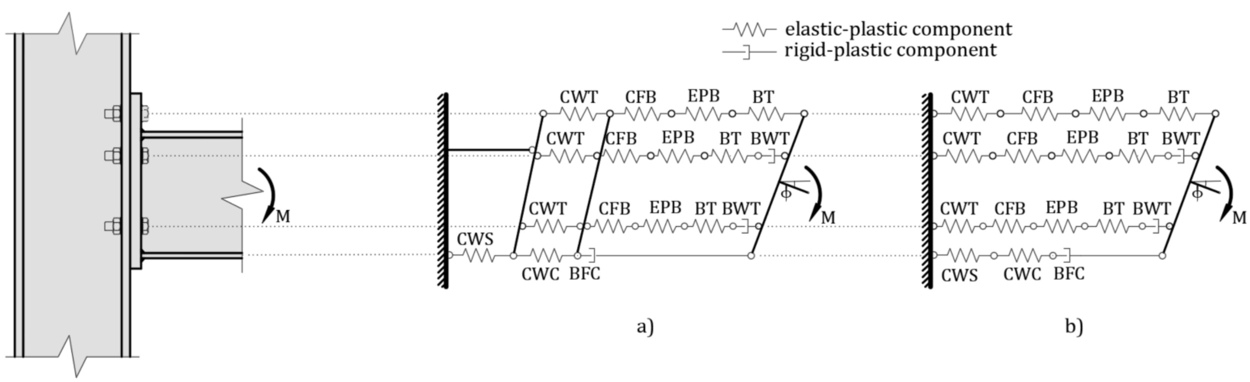

Fig. 2. Alternative mechanical models for steel joints: a) Innsbruck model; b) Coimbra model.

Girão Coelho et al. [25] proposed the “Coimbra model” which, despite operating with components identical to the ones of the EC3 model, does not allow for row spring equivalence. The limitation lies in the assumed interconnectivity of the springs that facilitates the interaction between the components of the model (Fig. 2b). This modelling choice does not lead necessarily to better results when compared to the EC3 approach; the main reason for its adoption at the University of Coimbra was the simplicity of its numerical implementation.

2.2 Limitations of existing mechanical models

Aiming at integrating the actual behaviour of joints in structural analyses, some researchers and practitioners resort to dedicated numerical tools and model explicitly the assembly of extensional springs and rigid links, thus building a numerical version of a classical mechanical model [18,26]. However, despite its complexity, the accuracy of a so-built model is still questionable as the classical spring models disregard three key features as presented in the following sections.

2.2.1 Group effects

Plastic yield mechanisms occur around the bolts in plate components subjected to transverse bolt forces, i.e., end-plate in bending (EPB) and column flange in bending (CFB). Being directly linked to the relative distance between the bolt rows, these yield mechanisms may develop around a single bolt (individual bolt mechanisms) or between several adjacent bolt rows (bolt group mechanisms) as shown in Fig. 3 and Fig. 4 for typical beam-to-column joints with bolted connections.

Under hogging moments, the three upper bolt-rows of the connection represented in Fig. 3 are subjected to tensile forces. The resistance of the CFB or of the EPB can be reached through:

– the development of isolated plastic yield patterns (Fig. 4a);
– the development of yield plastic mechanisms between consecutive bolt rows (groups R1+R2, R2+R3 or R1+R2+R3 in CFB and group R2+R3 in EPB – see Fig. 3).

EC3 provides design formulae to evaluate all the required $F_{Rd,i}$ and $F_{Rd,ik}$ resistance values, while stiffness-related aspects are addressed in a simplified manner through a conservative estimation of each bolt row stiffness assumed to be the minimum of the two values obtained successively by considering the bolt row as individual and as part of a group.

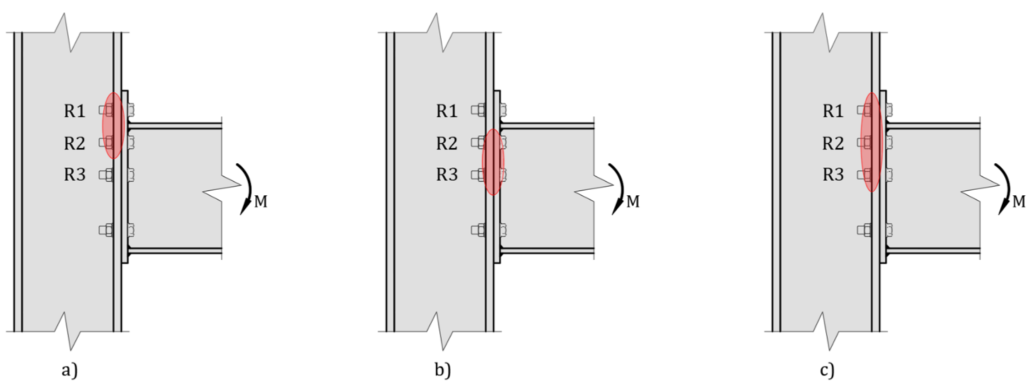

Fig. 3. Group effects in a bolted end-plate connection: a) Group R1+R2 (in CFB); b) Group R2+R3 (in CFB and EPB); c) Group R1+R2+R3 (in CFB)

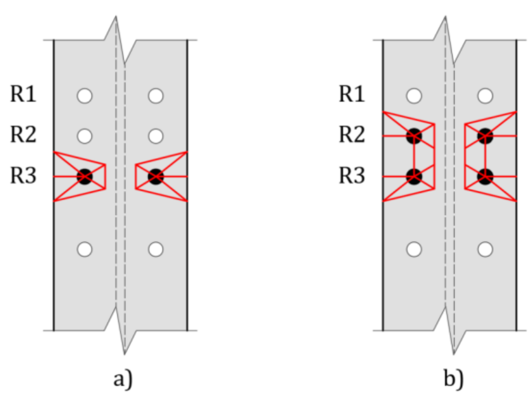

Fig. 4. CFB yield patterns: a) Individual (R3); b) Group (R2+R3)

As shown in Fig. 5, the development of such group effects can limit the maximum force $F_{Rd,i}$ that can be resisted by the i^{th} bolt row as the sum of the individual resistance of each bolt row involved in a group should always be smaller than or equal to the resistance of the group $F_{Rd,ik}$, where i and k stand for the number of the first and last bolt rows of the group.

In EC3, these coupling effects are dealt with through an analytical verification of the distribution of forces among the bolt rows with respect to their individual and group resistances, but this method can only be applied if the load acting on the joint is known in advance which is not the case when the joint is implemented in a frame subjected to varying loading paths. So, these analytical artifices are incompatible with a consistent mechanical modelling which should accommodate various loading conditions, and, currently, none of the existing mechanical models manages appropriately the group effects.

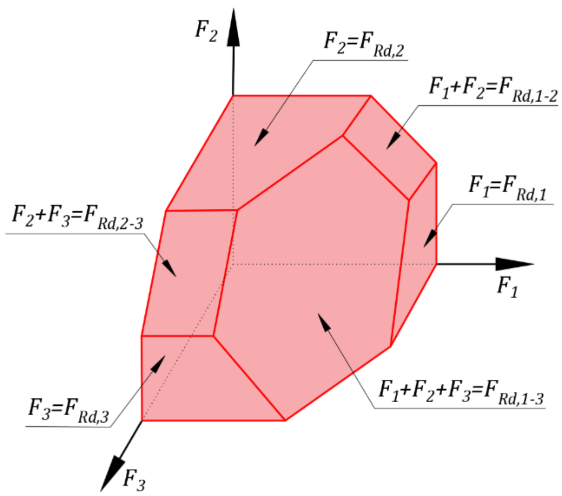

Fig. 5. Yield surface for individual and group yield patterns [21]

2.2.2 Behaviour of the column web panel in shear

The shear force V_{wp} acting on the column web panel results from all the internal forces transferred from the adjacent beams and columns (see Fig. 6). In EC3, a lever arm z is used to transform the moment M_b transferred from the beams into a shear force, and the panel shear force is computed according to Eq. (1).

$$V_{wp} = \frac{M_{b1}}{z} - \frac{V_{c1} - V_{c2}}{2} \tag{1}$$

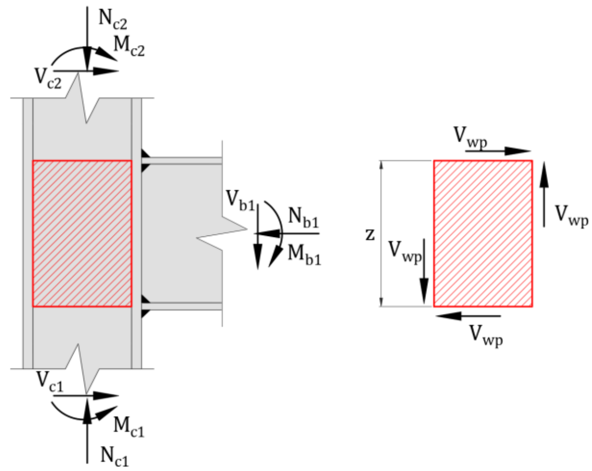

Fig. 6. Loading of the web panel (welded connection)

For joints with only two load-introduction components (i.e., welded connections or bolted connections with a single bolt row in tension), the lever arm z can be easily defined, and the shear force is constant on the entire height of the column web panel. When dealing with joints with multiple load-introduction bolt rows as the ones depicted in Fig. 7a, the definition of z becomes challenging. For such cases, EC3 recommends using an equivalent lever arm z_{eq} estimated through a stiffness calculation, which consequently leads to an estimation of a unique V_{wp} acting on the column web panel. This does not reflect appropriately the actual distribution of the shear forces along the web panel height as, in reality, V_{wp} varies along this height according to the loads introduced by connection rows. Additionally, for joints with different beam depths (Fig. 7b), an equivalent lever arm is defined for each of the connections and so, a unique V_{wp} to be compared with the resistance of the CWS cannot be properly estimated. Finally, under a varying loading applied at the level of the joint, the activated height of the web panel is also varying, what cannot be managed by the classical mechanical models.

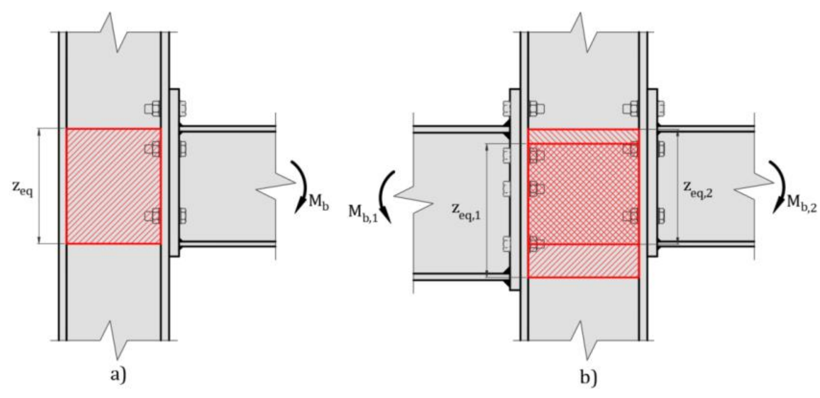

Fig. 7. Panel height according to EC3: a) Single-sided bolted joint; b) double-sided bolted joint with different beam depths

Besides the difficulties met in the definition of the effective panel height, another deficiency of the EC3 approach is that it disregards the mechanical coupling between the connection and the CWS, even though these two parts are envisaged as distinctive sources of joint deformability as illustrated in Fig. 8. The deformation of the two parts is combined accounting for the difference between the loading acting on the connection and the one acting on the column web panel through a so-called "transformation parameter" β.

This parameter relates the panel shear force to the compressive and tensile connection forces [21]. Accordingly, β is used to transform the deformability curve ($V_{wp} - \gamma$) of the CWS into a $M_b - \gamma$ curve (Fig. 8a), hence correlating the deformation of the web panel with the bending moment M_b transferred from the beam.

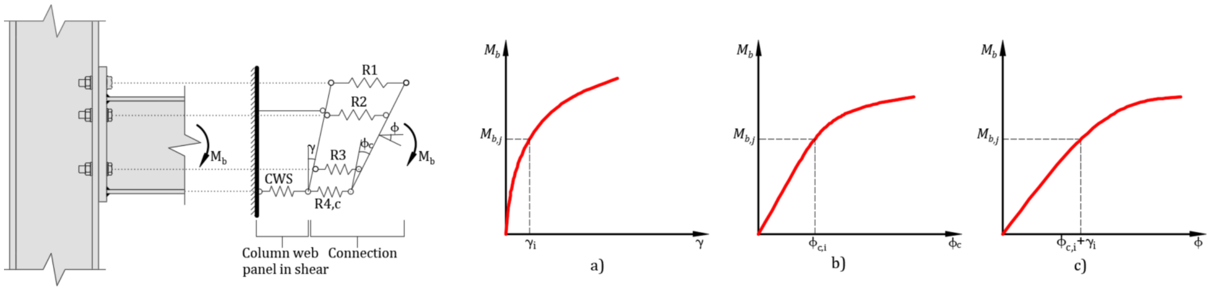

Fig. 8. Joint deformability sources: a) CWS (M_b- γ curve); b) Connection (M_b- ϕ_c curve); c) Resultant joint M_b- ϕ curve.

Normally, the accurate evaluation of β implies an iterative process since prior to the structural analysis, the internal forces acting on the joint are unknown. To avoid lengthy iterative processes in daily design practice, EC3 provides the practitioners with several fixed values for β ranging between 0 and 2 in function of the joint configuration and the presumed loading case. However, in a structural analysis, β does not remain constant but varies continuously from 0 to ∞ (at least theoretically) according to the variation of the internal forces appearing at the level of the joints. Accordingly, as revealed in [19], the fixed values of β proposed in EC3 could lead to significant miscalculations of the internal forces and moments acting on both joints and structural elements.

2.2.3 Interactions in the column web components

As schematically illustrated in Fig. 9, the column components are subjected to three types of stresses which can interact: longitudinal stresses due to bending moments and axial forces in the column (σ_n), shear stresses in the column web panel (τ), and transverse stresses associated to the concentrated compression or tension forces transferred from the connections (σ_i).

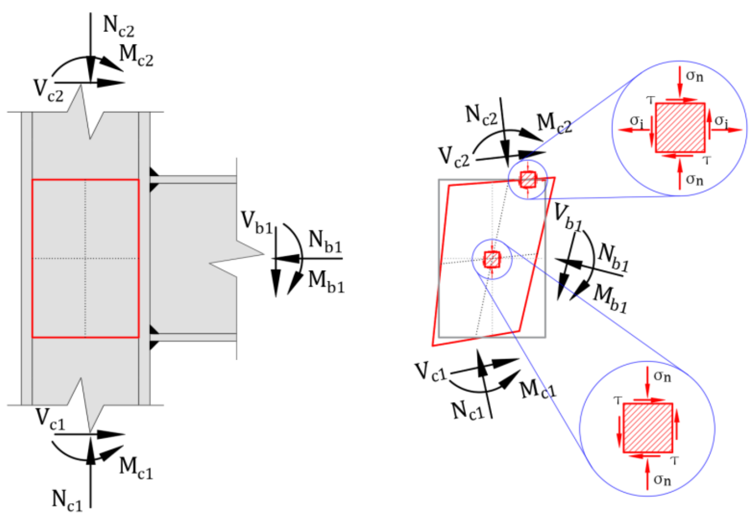

Fig. 9. Stress state in the column web panel

In EC3, the interaction between τ and σ_n in the column web panel in shear (CWS) is taken into account by affecting the component's resistance by a reduction factor of 0.9. For load-introduction components, i.e., the column web in compression (CWC) and the column web in tension (CWT), the potential stress interaction effects are dealt with through reduction factors ω that affect the resistance of these components. These penalizing factors ω are depending on the transformation parameter β which, as mentioned here-above, is assumed to have a singular value for the whole joint. In reality, β varies at each loading step and moreover, it differs from one load-introduction component to another. To incorporate these effects in a mechanical model, an iterative adjustment of the spring properties is required at each loading step so that the resistance of the components is recomputed accounting for the actual stress interaction. However, such a procedure cannot be implemented in the classical mechanical models since the latter rely on a unique and unrealistic value of β.

3. Proposed mechanical model

3.1 Connection modelling

A new connection mechanical model incorporating the group effects was proposed and validated in [28] and [29]. The authors suggest simulating the group effects using rigid-plastic springs which can be seen as "fuse" elements with a plastic strength equal to the group resistance as depicted in Fig. 10b. Once the resistance of a fuse element is reached, the latter behaves as a zero-stiffness element, and the sum of forces in the individual bolt rows within the group is equal to the group resistance. To illustrate this reasoning, the simple case of a connection with two bolt rows subjected to tension only is exemplified in Fig. 10.

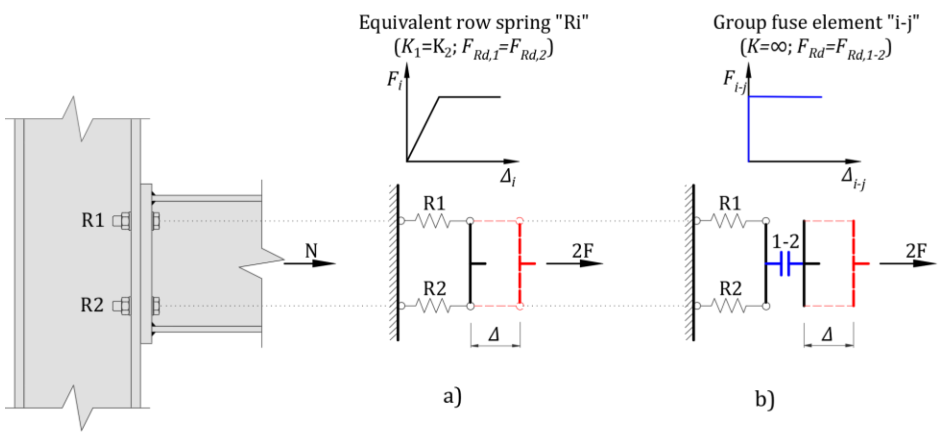

Fig. 10. Connection modelling: a) Classical connection model; b) Proposed connection model

The classical spring model (Fig. 10a) relies only on the contribution of the active components in the bolt rows simulated through equivalent row springs characterised by elastic-plastic F-Δ curves. Fig. 10b depicts the proposed mechanical model in which a fuse element (i.e., an infinitely stiff extensional spring) is added to simulate the potential group effects occurring between the two bolt rows in tension. The fuse element is tripped once the force transferred through it reaches the group resistance $F_{Rd,1\text{-}2}$.

Assuming similar mechanical properties for the equivalent row springs ($K_1 = K_2$ and $F_{Rd,1} = F_{Rd,2}$), their deformations under a progressively increasing force $2F$ will be identical ($\Delta_1 = \Delta_2$). Therefore, the properties of the connection subjected to tension may be expressed in terms of elastic stiffness and resistance as given in Eqs. (2) and (3).

$$K_{conn} = 2K_1 \tag{2}$$

$$F_{Rd,conn} = \min(2F_{Rd,1}, F_{Rd,1+2}) \tag{3}$$

This complies with the fact that the resistance of the connection may be limited either by the individual resistance of the rows or by the group resistance.

In addition, to comply with the Euler-Bernoulli theory, a parallelism between the two vertical rigid trays of the group fuse element must be kept through linear constraints. For instance, assuming the connection represented in Fig. 11 subjected to pure bending, the 3rd bolt-row (R4 in Fig. 11) may be considered as inactive since it is located very close to the compression centre. The presence of the transversal stiffener on the column web panel allows considering the stiffness of the row in compression as infinite ($K_{5,c} = \infty$), although the compression resistance of this row may be limited by the resistance of the BFC component. Under the progressively increasing bending moment, both extremities of the rigid spring simulating the group between bolt-rows R1 and R3 should rotate with the same quantity. This means that, if the group resistance ($F_{Rd,1\text{-}3}$) is reached prior to the individual bolt row resistances ($F_{Rd,1}$ or $F_{Rd,3}$) or to the connection row in compression resistance ($F_{Rd,5c}$), the connection rotates while a redistribution of forces between the bolt rows involved in the group takes place until reaching the maximum connection's bending resistance.

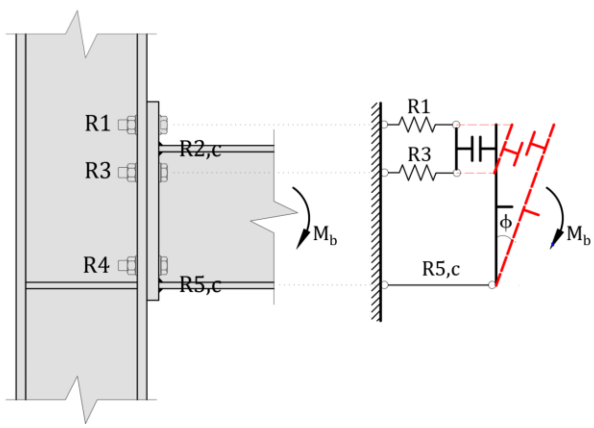

Fig. 11. Proposed model for a bolted end-plate connection in bending

In a generalised connection model, all the potential activation of components and of the associated rows must be identified and characterised to allow simulating the behaviour of the joint regardless of the loading type. This principle is illustrated in Fig. 12 for a single-sided joint in which two rows are active in compression only (R2,c and R5,c) and three rows are active in tension (R1, R3 and R4). The group effects that could potentially occur between the rows in tension should be integrated as well, and particular emphasis is given to the way the group elements transfer the forces within the mechanical model.

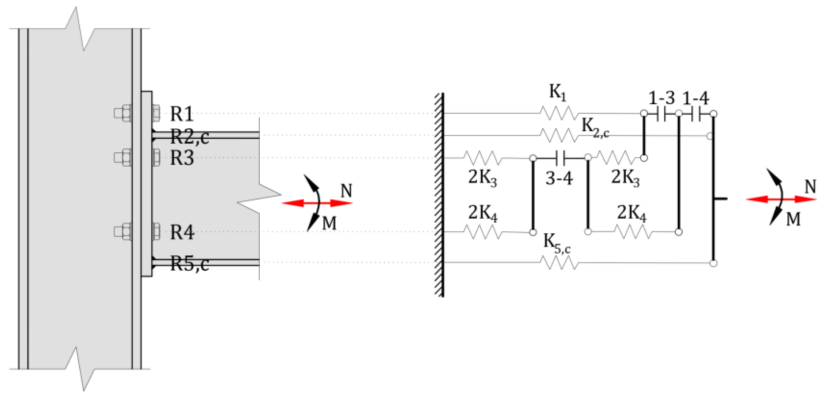

Fig. 12. Generalised mechanical model of a bolted end-plate connection

To avoid any conflicts between the bolt group elements, it is imperative to ensure that a specific group *i-j* is subjected exclusively to the forces acting in the i^{th} to j^{th} bolt rows. For instance, the group element 3-4 should only carry the forces in bolt rows R3 and R4, while the group element 1-4 should carry the forces from bolt rows R1, R3 and R4 (R2,c is not activated as it is working in compression only). This compatibility is ensured in the proposed model by splitting some equivalent row springs into several extensional springs so that group elements are integrated as depicted in Fig. 12 for rows R3 and R4. Since these springs work in series, their stiffness must be affected accordingly to ensure that the overall row stiffness is identical to the one of the initial equivalent row spring before splitting.

3.2 Column web panel modelling

As presented in Section 2.2.2, the characterisation of the web panel adopted in the classical spring models assumes a uniform shear force distribution over the entire height of the panel while, in reality, the shear force is varying over the panel's height according to the forces transferred from the active connection components and to the general *M-N* loading acting on the joint. Accordingly, since each load-introduction component induces a distortion in the stress state developed in the column web panel, the latter can be divided into several subpanels according to the number of connection rows. With such a subpanel modelling, the variation of the shear force is accounted for and reflects the actual stress state induced by all the forces concurring at the joint level. Additionally, the actual deformation of the whole panel zone is properly taken into consideration while bypassing the difficulties related to the definition of the panel height in the classical spring models.

Fig. 13 exemplifies these assumptions for a joint subjected to pure bending in which five potential load-introduction connection rows are identified on the height of the column web panel. The latter is thus divided into four subpanels according to the transfer of tensile forces from the bolt rows (F_1, F_3 and F_4) and the compression forces transferred from the beam flanges (F_2 and F_5) that equilibrate the connection.

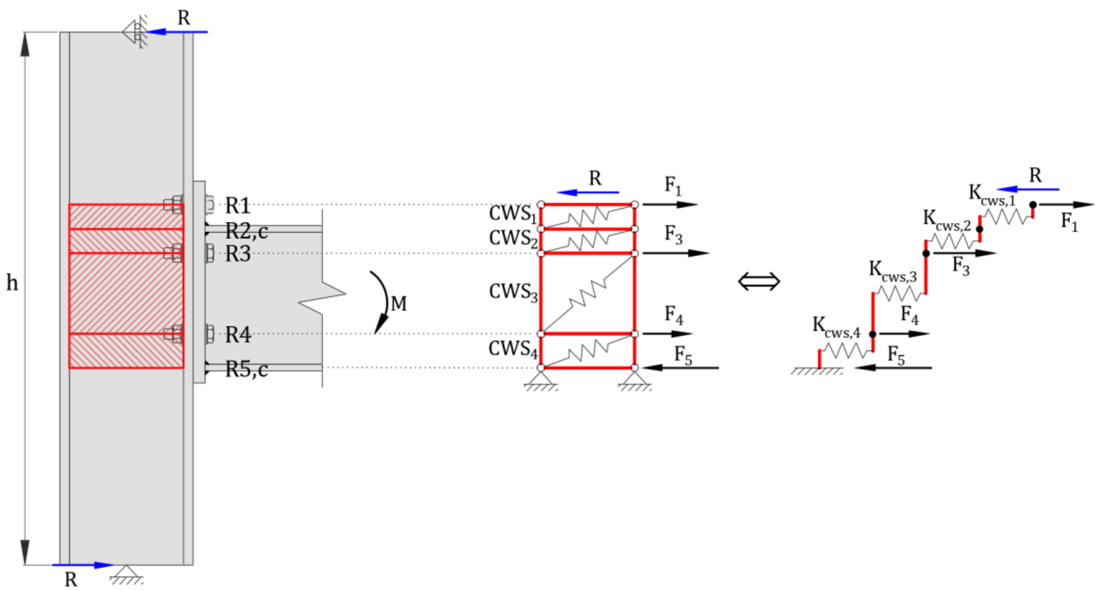

Fig. 13. Proposed model for the column web panel in shear

In the loading situation depicted in Fig. 13, the column region above the column panel is subjected to a resultant shear force R which can be computed as:

$$R = \frac{M}{h} \tag{4}$$

where M is the bending moment applied on the connection and h is the column height.

As a result, the shear force V_{wp} varies in each subpanel of the CWS accounting for both the forces in the connection rows and the column shear resultant simulated through R. Under these circumstances, a transformation parameter is no longer required to estimate the stiffness of the subpanels in shear which can be simply determined by removing β from the analytical expression specified in EC3 as follows in Eq. 5:

$$K_{cws,i} = E\frac{0.38A_{vc}}{z_i} \tag{5}$$

where z_i is the height of the i^{th} subpanel, A_{vc} is the shear area of the column section, and E is the modulus of elasticity of steel.

According to EC3, this stiffness corresponds to a horizontal extensional spring. Hence, to comply with the standard, the conventionally diagonal shear springs are replaced by horizontal springs with a stiffness $K_{cws,i}$ as shown in Fig. 13. The deformation of the whole panel region is simulated by allowing only horizontal displacements (elongation/shortening) in the extensional springs while the rotation of the vertical elements connecting the shear springs is restrained.

3.3 Generalised mechanical model for structural joints

A generalised mechanical model is finally built by combining the two models proposed above for the two major joint parts contributing to the joint's deformability as depicted in Fig. 14. The so-defined mechanical model accounts for the group effects and for the actual deformability of the panel zone with respect to the connection load introduction.

Provided that all the connection rows are properly characterised in terms of equivalent stiffness and strength, and assuming a similar behaviour in tension and compression for the extensional springs simulating the CWS subpanels, the so-built model allows predicting the behaviour of structural joints under *M*-*N* interaction, i.e., under generalised loading conditions.

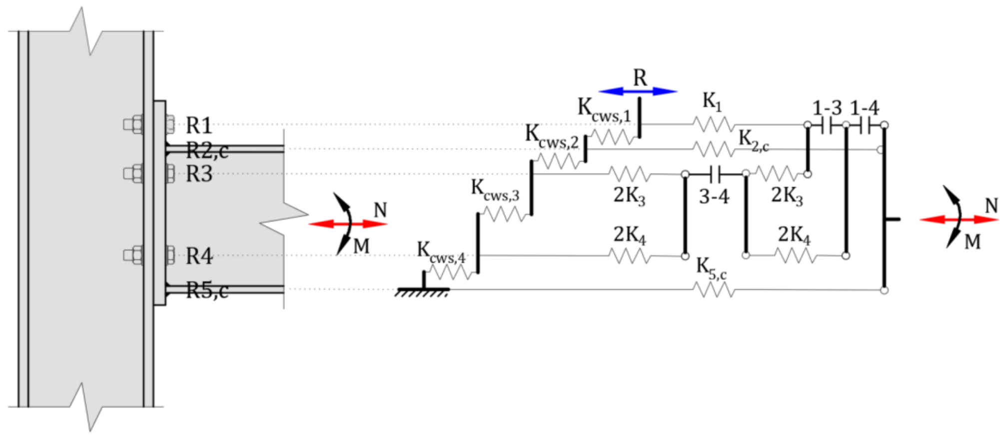

Fig. 14. Generalised model for a single-sided joint

By using the analytical formulae recommended in EC3 for the characterisation of basic joint components, the proposed model leads to code-compliant predictions. It can be therefore considered as a practical alternative to the analytical assembly procedure prescribed by the design standard. In addition, the applicability of the model can be readily extended for research purposes since the physical parameters influencing its performances concern only the characterisation of components. So, refined non-linear behaviour laws may be assigned to the extensional springs simulating the components enabling the model to characterise the full range behaviour of joints up to their failure.

Particularities related to the proper incorporation of component interactions are still to be addressed. The comprehensive methodology proposed in this paper allows implementing the model in numerical software as a macro-element. Accordingly, the stress interaction in the column components can be included through an automatic routine that updates the resistance of the corresponding springs with respect to the actual stress state at a specific loading step.

Furthermore, since the beam-to-column joints are subjected to shear forces in addition to bending moments and/or axial forces, the shear resistance of connections should also be considered in the mechanical characterisation of joints. In EC3, this issue is addressed through a simplified analytical verification based on the distribution of forces in the bolt rows when the bending capacity of the whole joint is reached. Accordingly, the shear resistance of the connection is estimated by considering that the bolt rows located in the compression zone transfer a shear force equal to their shear resistance, whereas the bolt rows subjected to tension are conservatively considered to transfer a shear force equal to only 28.6% of their shear resistance [21]. In the proposed mechanical model, a more accurate and less conservative check of the shear resistance of the connection is envisaged. Similar to the component interaction, the available resistance of bolt rows in shear may be calculated at each iteration step based on the distribution of forces within the joint and accounting for the actual shear-tension (*N*-*V*) interaction in the bolts. Further improvements aiming at incorporating additional elements that simulate the contribution of reinforced concrete slabs in steel-concrete composite joints are also envisaged.

4. Model validation

4.1 Modelling assumptions

Based on the approach presented in Section 3, the proposed mechanical model was numerically built in the FINELG FE software [30] for seven steel joint configurations previously tested at the University of Liège [31,32]. FINELG is a FE software developed at the University of Liège in collaboration with Greisch Engineering office and allows performing different types of analyses (e.g., elastic, nonlinear, static/dynamic) with account for geometric and material nonlinearities. In the conducted investigations, emphasis was given to joints with bolted connections, the behaviour of which is influenced by phenomena addressed in this paper.

Nonlinear extensional spring elements were used to simulate the connection rows in tension/compression and the column web subpanels in shear. The compatibility of displacements between the springs in series/parallel and the validity of the Euler-Bernoulli theory at the end-section of the model were ensured through linear constraints.

Analytical formulae provided by EC3 were used to characterise all active components of the analysed joints in terms of strength and stiffness, except for the stiffness of the CWS subpanels which was evaluated according to Eq. (5). To account for a more realistic behaviour in the post-elastic range, tri-linear behaviour laws in agreement with EC3 recommendations were defined as depicted in Fig. 15a-c, where F_{el} and F_{pl} represent the elastic and the plastic resistances of the components respectively with F_{el} considered as $2/3F_{pl}$. Rigid-perfectly plastic laws (Fig. 15d) with a plastic resistance F_{pl} representing the group resistance were assigned to the fuse elements simulating the group effects.

Given the fact that, in EC3, the strain hardening of the steel material is not considered in the characterisation of components' behaviour, only the plastic moment resistance ($M_{R,pl}$) of the joints can be evaluated. Consequently, the derived M_j - ϕ curves will exhibit a plateau at the level of $M_{R,pl}$ which generally is significantly lower than the ultimate resistance ($M_{R,u}$).

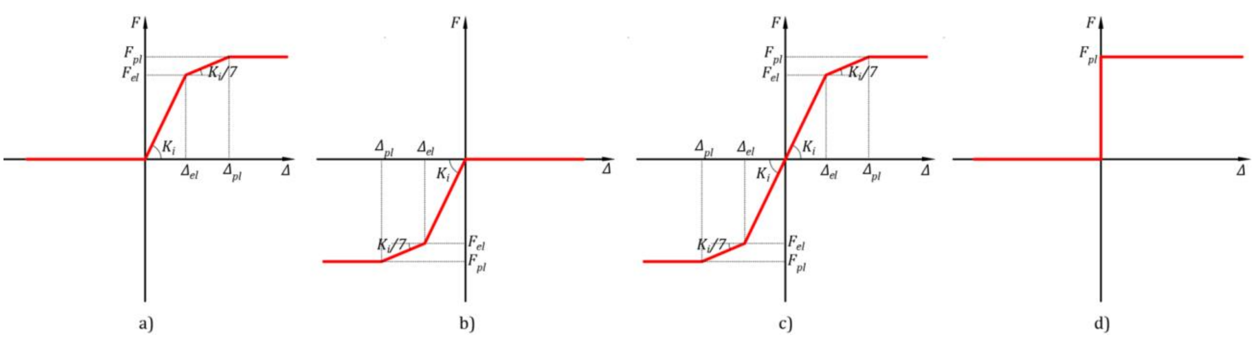

Fig. 15. Typical behaviour laws: a) Components in tension; b) Components in compression; c) Subpanels in shear; d) Group elements

It is worth noting that the current version of EC3 deals with aspects related to the ductility of components in an exclusively qualitative manner, i.e., CWS, BWT, EPB, and CFB are assumed to be infinitely ductile, CWC and BFC exhibit a limited ductility, and BT are "brittle" components. If ductile components are activated plastically, the failure of a joint is reached after internal plastic redistribution of forces occurs amongst the connection rows. There are however cases when the resistance and/or the rotation capacity of the joints is limited by the non-ductile response of brittle components, the failure of which prevents any further plastic redistribution. Therefore, although no specific limit was assigned herein to the deformation capacity of components, the attainment of the plastic deformation in brittle components was systematically checked throughout the numerical simulations.

4.2 Results and discussion

The compliance of the proposed mechanical model with the analytical approach proposed in EC3 is demonstrated for single-sided and double-sided beam-to-column joints tested under monotonic and cyclic loading. It is worth mentioning that, to achieve consistent results, the M_j - ϕ curves were derived according to EC3 accounting for the actual transformation parameter β estimated from the experimental setup, the dimensions of the specimens, and the applied loading. Accordingly, the stress interaction in the CWT and CWC components was accounted for through the so calculated β (used to estimate the reduction factors ω) in both the EC3 approach and the proposed model.

The experimental M_j - ϕ curves were extracted from the test recordings assuming that the bending moment acting on the joint M_j is the moment resulted at the interface between the beam-end (end-plate) and the column flange. The joint rotation ϕ was estimated by extracting the elastic deformation of the beam from the vertical displacement measured throughout the tests at the level of the load application point. To ensure consistent comparisons, the M_j - ϕ curves predicted with the proposed model were generated in a similar manner by calculating the applied bending moment at the same location and estimating the joint rotation ϕ based on the slope of the rigid element provided at the extremity of the connection model.

Fig. 16 and Fig. 17 show the predicted M_j - ϕ curves for two joints tested in [31]. A good agreement between the predictions is observed for both plastic resistance and initial stiffness. However, there are noticeable differences in the shape of the post-elastic curve mainly due to the simplified estimation of the post-elastic stiffness according to EC3 ($S_{j,ini}$/7 for bolted connections). Moreover, contrary to the EC3 approach in which the elastic resistance of a joint is assumed equal to 2/3$M_{R,pl}$, the proposed model allows engaging gradual yielding in the rows of the connections. Hence, the predicted M_j - ϕ curve shapes a strongly nonlinear post-elastic behaviour in a better agreement with the experimental response.

In terms of failure modes, the model indicates the failure of the column web panel in shear (plasticity reached in the two lower subpanels of the CWS) for the O1 specimen (Fig. 16) and the failure of the column web in compression for the O13 specimen (Fig. 17). Both failure modes are in line with the EC3 predictions and serve as an additional validation criterion.

A slight difference between the predicted initial stiffness is observed in Fig. 17, yet the perfect match between the response of the mechanical model and the experimental results confirms that the modelling adopted for the panel zone is more realistic when compared to the one-panel modelling considered in EC3. It becomes evident that the discretisation into several subpanels with respect to the connection load introduction leads to a realistic assessment of the deformability of the panel zone improving thus the accuracy in predicting the rotational behaviour of semi-rigid joints.

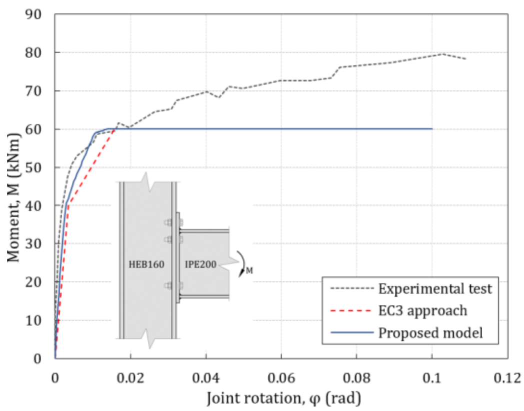

Fig. 16. M_j - ϕ curve: O1 specimen

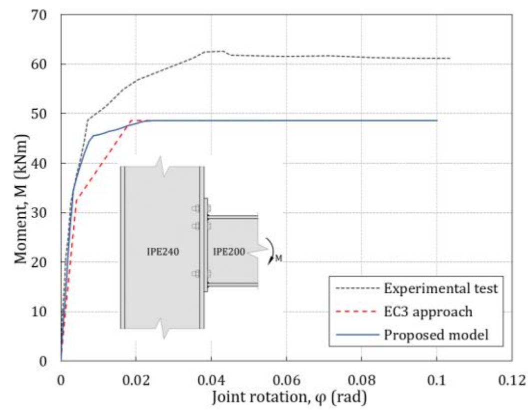

Fig. 17. M_j - ϕ curve: O13 specimen

Fig. 18 shows the predicted response for a single-sided joint tested in [32] under monotonic loading. A significant overestimation of the plastic moment resistance is observed for this specimen. Since the component governing the response of the joint is the CWS, this overestimation can be ascribed to the limitations of the analytical expression used to evaluate the resistance of this component in EC3. The validation of the formulae prescribed by the standard falls outside the scope of this paper, yet this issue is currently under investigation in Liège and improvements for the characterisation of the panel zone were proposed in [27] and [33] with promising results representing prospective changes to the analytical expressions prescribed by the code.

Nevertheless, the M_j - ϕ curve predicted with the proposed model is in a good agreement with the EC3 prediction, although, due to previously mentioned reasons, a minor discrepancy is observed in the post-elastic range of the curve. The refined discretisation of the CWS into subpanels leads to an initial rotational stiffness marginally higher than the one obtained with the analytical approach, but otherwise in a good agreement with the actual stiffness of the specimen given by the slope of the experimental unloading-loading cycle performed in the plastic range.

For the double-sided E2-XW-P specimen tested in [32] under cyclic loading with unbalanced moments on each side of the joint, Fig. 19 shows that the mechanical model exhibits a plastic moment resistance ($M_{R,pl}$) 6% higher than the EC3 prediction. One may find this discrepancy somewhat counterintuitive since identical values for the strength of individual components were used in both the EC3 approach and the proposed model. The increase of $M_{R,pl}$ lies in the fact that, according to EC3, the component limiting the plastic resistance of the joint is the CWS; the failure occurring in the proposed mechanical model confirms this prediction. However, contrary to the one-spring panel model of EC3, the multi-layered springs simulating the panel zone in the proposed model allow activating progressive plasticity in the subpanels subjected to shear. As a result, given the unbalanced moments applied on the beams (right and left connections under hogging and sagging respectively), the central subpanel of the CWS reaches its plastic resistance under a bending moment M=526 kNm which, as a matter of fact, corresponds to $M_{R,pl}$ predicted by EC3. At this instance, a plastic redistribution occurs between the subpanels of the CWS, which consequently induces a redistribution of forces within the connection. This leads to an increase of the applied bending moment until plasticity has spread to the adjacent CWS subpanels, preventing thus further plastic redistribution, and shaping the plateau depicted in Fig. 19.

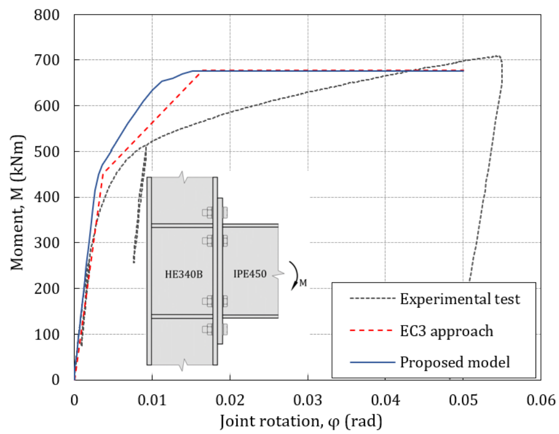

Fig. 18. M_j - ϕ curve: E2-TB-E specimen

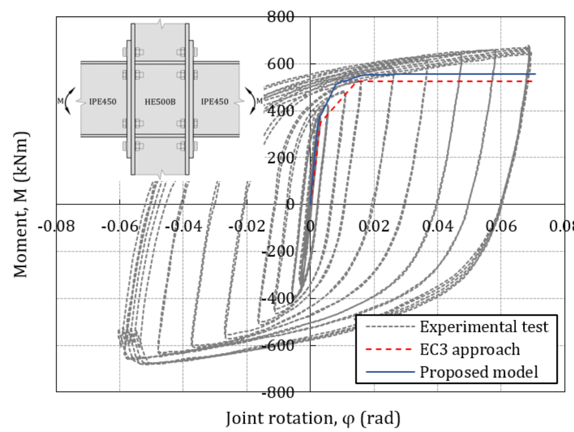

Fig. 19. M_j - ϕ curve: E2-XW-P specimen (right joint)

The M_j - ϕ curves reported in Fig. 20 (E3-TB-E specimen) and Fig. 21 (E3-XW-P specimen) show a good agreement between the performance of the proposed model and the EC3 approach for all validation criteria: initial stiffness, plastic resistance, and failure mode. The use of tri-linear behaviour laws for the characterisation of joints' basic components improved the prediction for the nonlinear post-elastic response of the joints. Despite the neglection of strain-hardening effects, the two predictions converge in terms of plastic moment resistance after significant plastic rotations are reached mainly due to the prolonged plastic redistribution that occurs amongst the connection rows in the proposed model.

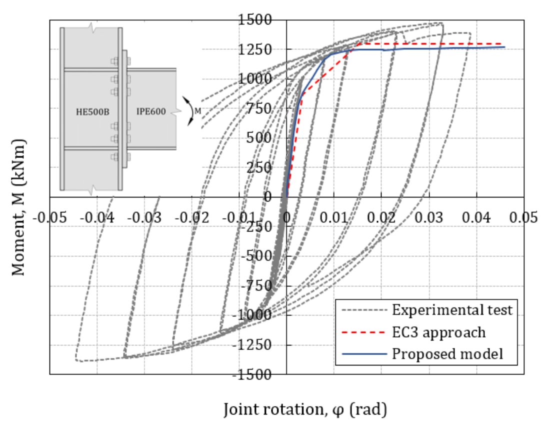

Fig. 20. M_j - ϕ curve: E3-TB-E specimen

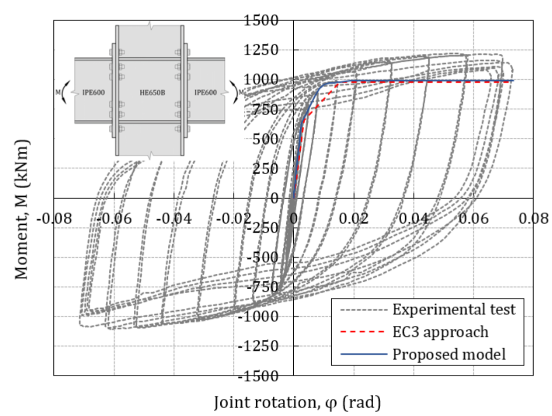

Fig. 21. M_j - ϕ curve: E3-XW-P specimen (right joint)

The results reported hereabove prove the compliance of the mechanical model with the analytical approach prescribed by EC3. Nonetheless, given the specific conditions in which the group mechanisms develop, these had no influence on the response of the tested specimens. In particular, the group effects were prevented either by large spacing between the bolt rows (O3, O13, E2-TB-E, and E2-XW-P specimens) or by relatively thick plates of the components subjected to transverse bolt forces (E3-TB-E and E3-XW-P specimens). Furthermore, limited number of relevant tests featuring the group effects has been performed until the present day mainly due to the large-scale specimens with multiple adjoining bolt rows that would be required for this purpose. In the daily design practice, however, this type of joints is a common solution for large-span beams or for structures designed to resist seismic loads. The E3-TB-P joint depicted in Fig. 22 falls into the second category and was tested under cyclic loading simulating the seismic action.

It is worth mentioning that the E3-TB-E and E3-TB-P specimens were identical in terms of geometrical and material properties excepting the end-plate thickness, which was reduced from 25 mm (E3-TB-E) to 20 mm (E3-TB-P). The activation of group mechanisms in the E3-TB-P specimen reflects the high dependence of such effects on the joints' detailing.

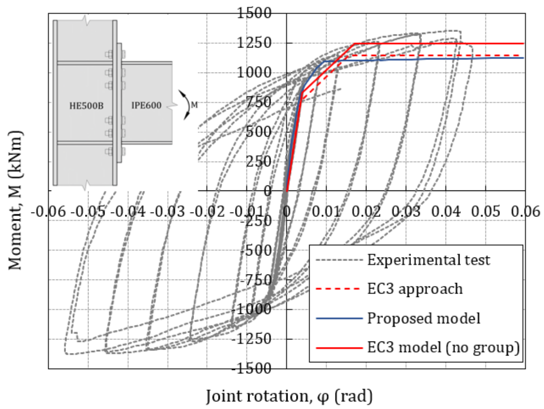

Fig. 22. M_j - ϕ curve: E3-TB-P specimen

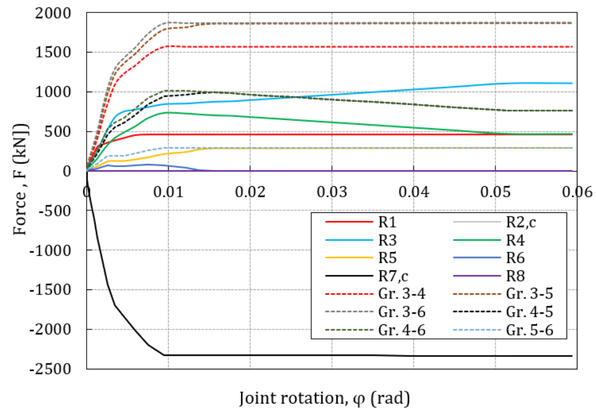

Fig. 23. Variation of forces in the connection rows

A good agreement between the M_j - ϕ curves derived through the EC3 approach and with the proposed model is observed in Fig. 22. The results also show that by relying on a classical model with no account for the group effects, the plastic bending resistance of the joint would be significantly overestimated (red continuous line in Fig. 22). The distribution of forces in the connection rows depicted in Fig. 23 points out the influence of the group mechanisms on the overall response of the joint with several key observations to be made:

- At first, the plastic individual resistance of bolt row R1 is reached for a bending moment M=1041 kNm and a rotation ϕ=7.5 mrad. Given the ductile post-plastic behaviour of the components working in series in this row, a further increase in the applied bending moment is envisaged.
- The resistance of the group comprising bolt rows R3 and R4 (Gr. 3-4 in Fig. 23) is reached for a joint rotation ϕ=10 mrad and a corresponding bending moment M =1093 kNm. The group yield patterns develop in the EPB component which fails further to the yielding of the flange of the equivalent T-stub [10].
- Subsequently, a plastic redistribution of forces leads to the overloading of bolt row R3 and a decrease in the tensile force in R4 respectively. This trend continues up to the attainment of the plastic resistance of R3 which prevents a further plastic redistribution of forces within group 3-4.
- The mechanical coupling ensured through the fuse element simulating the group 3-4 limits the sum of forces in the two bolt rows to the overall plastic resistance of the group. Moreover, the decrease in the tensile force in R4 affects the distribution of forces in groups 4-5 and 4-6.

These observations validate the assumptions made on the modelling of the group effects and point out the efficiency of the proposed model for predicting the joint's behaviour with account for the group mechanisms in compliance with the analytical assembly procedure prescribed by EC3.

5. Conclusions and perspectives

An innovative generalised mechanical model for the characterisation of structural joints is proposed in this paper. The model addresses the group effects and the actual deformability of the column web panel zone – aspects systematically disregarded in classical spring models. While operating with components identical to the ones identified in EC3, the proposed model overcomes the limitations of other existing component-based models through the addition of a group component and through a refined discretisation of the panel zone with respect to the connection's load introduction.

The validation against experimental evidence proves the effectiveness of the model in characterising realistically the behaviour of steel joints. The comparison with analytical results based on the EC3 approach demonstrates the model's compliance with the current normative requirements.

The proposed methodology enables one to integrate the model in structural analyses conducted in full accordance with EC3 while avoiding the inherent iterative process implied by the standard for the estimation of the transformation parameter β. Moreover, the supressed need for the calculation of β and the realistic modelling of the column web panel set the way forward for the appropriate incorporation of component interactions in the proposed model. These represent some promising perspectives for the adoption of the model as a macro-element in dedicated software for reliable and time efficient structural analyses.

Given the fact that the model accounts for all potentially active components within a joint, its use can be directly extended for generalised loading conditions providing an adequate understanding of the influence of complex load interactions. As perspectives, the accuracy of the model could be enhanced by refining the characterisation of basic components through complex non-linear behaviour laws accounting for the strain hardening and/or softening of components.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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