DEVELOPMENT OF A SIMPLIFIED MODEL FOR JOINTS IN STEEL STRUCTURES

DESENVOLVIMENTO DE UM MODELO SIMPLIFICADO PARA JUNTAS VIGA-COLUNA EM ESTRUTURAS DE AÇO

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ABSTRACT

The global behaviour of a framed structure is strongly influenced by the behaviour of the beam-to-column joints. The component method coded in Eurocode 3 allows to characterize the moment-rotation curve of semi-rigid beam-column joints. However, the rigorous application of this method requires a distinction to be made between separate sources of deformability of joints: those in the connection and those in the column web panel. This paper deals with the formulation of a simplified mechanical model composed of extensional springs and rigid links able to cover beam-to-column joints with different beam depths. These models may be used for the interpretation of experimental test and for the formulation of a beam-to-column joint finite element that accurately accounts for its behaviour in global frame analysis.

RESUMO

O comportamento global de uma estrutura porticada é fortemente influenciado pelo comportamento das juntas viga-coluna. O método das componentes codificado no Eurocódigo 3 permite caracterizar as curvas momento-rotação das juntas vigas-colunas semi-rígidas. No entanto, a aplicação rigorosa deste método requer que seja feita uma distinção entre as diferentes fontes de deformação das juntas: as da ligação e as da alma do pilar. Este artigo aborda a formulação de um modelo mecânico simplificado composto por molas lineares e elementos rígidos capaz de lidar com juntas viga-coluna com vigas de diferentes alturas de secção transversal. Estes modelos podem ser usados para interpretar resultados experimentais e para a formulação de um elemento finito para juntas viga-coluna que permita ter em consideração o seu comportamento na análise global de uma estrutura porticada.

1. INTRODUCTION

The appropriate modelling of beam-to-column joints in the design of steel structures is essential not only for the accurate simulation of the overall structural behaviour but also in order to achieve economic and sustainable solutions. Accordingly, in the last decades an enormous effort was put on developing accurate and easy-to-use analysis and design procedures for beam-to-column joints, leading to the so called component method, already coded in EN 1993-1-8 (CEN 2005).

According to the component method, the joints are decomposed in several parts, called components that represent a specific part of a joint that, dependent on the type of loading, make an identified contribution to one or more of its structural properties (Simões da Silva *et al*, 2002). The constitutive relations of the components and the way they are assembled determine the joint behaviour. The relation between the components and the joint's mechanical properties is determined through equilibrium and compatibility relations.

Several approaches, with different level of complexity, can be considered for the modelling of beam-to-column joints in the framework of the component method. The traditional approach for global analysis of steel structures makes use of the component method to compute moment rotation relations for each joint and assigns these relations to rotational springs attached to the beams ends on both sides (double-side joint configuration) or just on one side (single-side joint configuration) of the column centreline. This approach was followed since the early stages of the component method because it allowed the global analysis of structures making use of ordinary Finite Element based programs taking into account the actual behaviour of beam-to-column joints without requiring new elements neither changes to the software code. Alternatively, a refined approach was also possible, whereby the joint components are explicitly and individually accounted for in the model. However, this approach is very time consuming and cumbersome, presents convergence and calibration difficulties, so that it is usually not considered in design offices and its application has been restricted to research purposes.

The need to consider the actual behaviour of beam-to-column joints in structural analysis is clear and the component method is recognized as an effective procedure to account for it. The continuous developments in structural analysis and the increased capacity of personal computers allow for more robust and rigorous implementations without increased burdens on the user.

In the field of beam-to-column joint modelling, this requirement will be accomplished in a near future through the formulation and implementation of 2D and 3D joint macro-elements developed in the framework of component method in structural analysis software packages. These macro-elements will be materialized through new structural elements suitable for global analysis of structures and will allow a refined modelling of joints effortlessly.

In this paper, the main reasons for the development of macro-component models are explained and a short state-of-the-art related to joint macro-elements is presented. Secondly, two macro-elements mechanical models suitable for symmetric and asymmetric internal steel beam-to-column joints are presented, their modelling in a general purpose nonlinear finite element program – Abaqus FEA (Simulia 2014) – is explained and the validation procedure adopted to assess the results is shown. Finally these models are inserted in a 2D frame in order to highlight its practical application and relevance.

The models presented are the first step in an ongoing research project (3DJOINTS) to validate a new finite element macro-element already being implemented in OpenSees (McKenna *et al.* 2000).

2. THE NEED FOR BEAM-TO-COLUMN JOINT MACRO-ELEMENTS

Beam-to-column joint modelling making use of rotational springs attached to the ends of the beams is an effective procedure for the simulation of joints. However this procedure also encompasses some disadvantages.

The first drawback that can be pointed out is their local kinematic behaviour. Following the analysis made by Charney and Marshal (2006), Fig. 1 shows the deflection shapes of two sub-frames with a double sided joint configuration where only the shear panel deformation is accounted for. Fig. 1(a) shows the kinematics of a Krawinkler type model (Krawinkler 1978) and Fig. 1(b) shows the rotational springs attached to beams ends joint model. From Fig. 1 it is clear that: (i) the Krawinkler type model provides a better representation of the actual behaviour of the joint; and (ii) the local kinematic behaviour of both models is different, mainly because there are no offsets between the beams and columns centrelines in Fig. 1(b).

The joint modelling making use of rotational springs attached to beams ends is also cumbersome whenever it is needed to account for the interaction between several types of internal forces transmitted to the beam-to-column joint by one or more than one adjacent element, *e.g.* when there is the need to couple the effect of the bending moment and the axial force in the connections (nonlinear analysis) or when the shear behaviour of the column web panel it to be accounted for accurately.

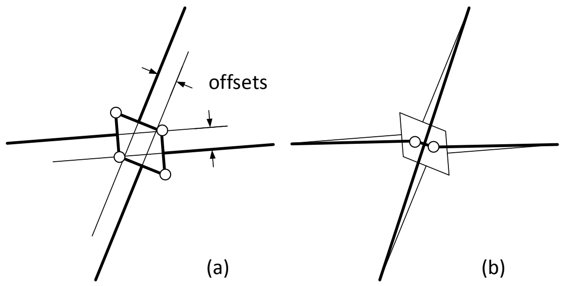

Fig. 1 – Kinematics of joint models: (a) Krawinkler type model, (b) rotational springs attached to beams ends joint model.

The latter case, *i.e.* the shear behaviour of the column web panel, is of paramount importance and it should be carefully considered. In order to account for the interaction of the internal forces transmitted to the column shear panel by the beams and columns connected to the beam-to-column joint, EN 1993-1-8 (CEN 2005) and EN 1994-1-1 (CEN 2004) define, in a simplified way (Simões da Silva *et al*, 2010) an interaction parameter called the β factor that accounts for the moments transmitted to the beam-to-column joint by beams. However, Bayo *et al.* (2006) showed that the β factor procedure (i) does not account for the beneficial effect of the columns shear force for the shear panel behaviour, (ii) requires an iterative analysis, even if a only a linear analysis is wished, (iii) may lead to substantial errors in the internal forces and (iv) may preclude the convergence of the iterative process for elastic-plastic analysis. On the other hand, the β factor procedure cannot deal with beam-to-column joints with beams with unequal depth because, in these cases, there is the need to consider two columns shear panels with different levels of shear (Jordão *et al.* 2013).

Finally, it also should be noted that although the bending deformation mode of steel joints is usually the most important deformation mode for the standard static loading conditions, in certain situations, *e.g.* fire and seismic loading, several modes become relevant and should be accounted for. Besides, robustness requirements also demand a minimum level of resistance for any arbitrary loading (Simões da Silva 2008). A joint macro-element seems to be the most effective procedure to account for several deformation modes and for the behaviour of the beam-to-column joints under arbitrary loading.

3. STATE OF THE ART

The modelling of beam-to-column joints through macro-elements in reinforced concrete (RC) structures, instead of rotational springs attached to beams ends, has received recently attention from researchers. There are two main reasons to choose this strategy in reinforced concrete frames analysis:

(i) the relative size of the joint region when compared with the length of beams and column is much higher in reinforced concrete frames than in steel frames; accordingly, a joint model which does not account for the actual joint size, *e.g.* rotational springs attached to the beams ends near the intersection of beams and columns centrelines, would be subjected to internal forces very different from the ones at the joint periphery (Costa 2013);

(ii) under seismic loads, the joint shear behaviour is one of the main sources of energy dissipation and accordingly should be carefully simulated.

Consequently, some models have been developed, mainly in the US. Two of most well know models, already implemented in OpenSees, are the model developed by Lowes and Altoontash (2003), later updated by Mitra and Lowes (2007), and the model developed by Altoontash (2004). Recently, Costa (2013) presented a model that aimed to improve the modelling of the shear behaviour in the joint panel.

Lowes and Altoontash (2003) proposed a model comprising: (i) a frame made of four rigid bi-articulated elements arranged along the periphery of the beam-to-column joint; (ii) a panel inside the frame (a plane stress shear panel); and (iii) interfaces between the beam-to-column joint and each of the adjacent beams and columns modelled by three linear springs (Fig. 2) placed between each side of the frame and a rigid element parallel to it. Two of the springs of each interface are parallel to the beam/columns centrelines and are intended to model the anchorage of the longitudinal rebars of beams and columns inside the beam-to-column joint. The third spring of the interface is orthogonal to the beam/column centreline and is intended for modelling the shear deformation at the interface. The panel in the interior of the frame aims to modelling the shear deformation in the shear panel and, according to Lowes and Altoontash (2003), can also be considered as an angular spring between two rigid elements in one of the corners of the frame. Mitra and Lowes (2007) updated this model by shifting the anchorage springs so that they become aligned with the tension and compression resultants of the beam/column ends and used a diagonal concrete strut model for the simulation of the shear panel.

Altoontash (2004) suggested a beam-to-column joint model based on the model developed by Lowes and Altoontash (2003), see Fig. 3, where the beam-to-column joint was modelled by a frame made of four rigid bi-articulated elements arranged along the periphery of the beam-to-column joint (similar to the one used by Lowes and Altoontash (2003)), four angular springs arranged in the midpoints of the faces of the panel, to which the beams and the columns are connected and an angular spring between two line segments that join the midpoints of the sides of the panel. The angular springs aim to model the relative rotation between the joint faces and the end of the beams - unlike in the model proposed by Lowes and Altoontash (2003), in the model proposed by Altoontash (2004) the shear and the axial deformations at the interfaces between the beam-to-column joint and beam and columns are disregarded.

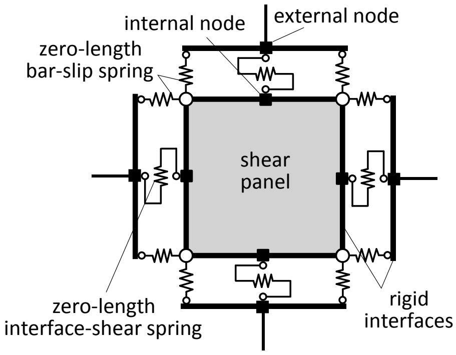

Fig. 2 – Beam-to-column joint model proposed by Lowes and Altoontash (2003).

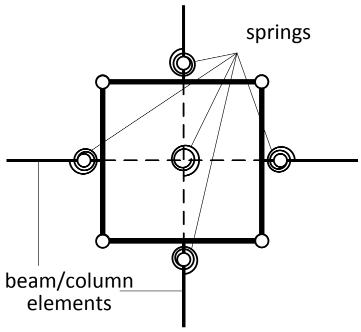

Fig. 3 – Beam-to-column joint model proposed by Altoontash (2004).

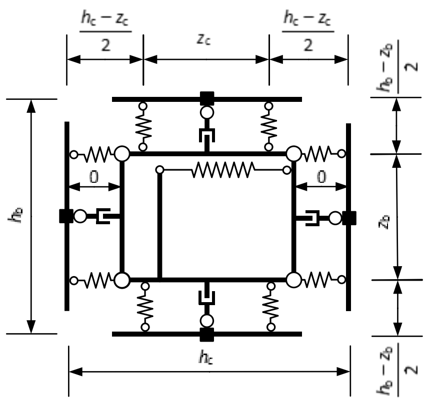

Fig. 4 – Beam-to-column joint model proposed by Costa (2013).

One of the difficulties of some RC models is the determination of the constitutive relations suitable for the shear behaviour component and the standards requirements: the shear behaviour of RC joints is usually expressed in terms of horizontal shear in the mid-height of the joint (V_{jh}) but the internal forces in these components is usually different from V_{jh}. In the model developed by Costa (2013), see Fig. 4, the geometry of the rigid frame is such that the internal force in the shear component is V_{jh}, allowing for a direct check of code requirements.

Because these models were developed for RC beam-column joints and because, according to the components method philosophy, all and only the relevant components should be considered, these models are not suitable for beam-to-column joints: (i) the components in the beam-to-column vs. column interface are not relevant in steel frames, (ii) the number of components in the beam connections are usually much higher than in RC structures and their arrangement is also different and (iii) the later models cannot deal with beam-to-column joints with beams with unequal depth.

Consequently, two models based in the findings of Jordão *et al.* (2013), suitable for steel beam-to column joints, are presented and validated in the following sections.

4. MODELS OF BEAM-TO-COLUMN JOINTS

4.1. Implementation of models in a commercial structural software

Fig. 5 refers to a mechanical model for joints with beams of equal depth while Fig. 6 shows the model in case of joints with beams of unequal depth (Jordão *et al.* (2013)).

This paper is mainly focused in the column web panel modelling. Accordingly, the components in the interface between the column web panel and the beams (left and right connections) are condensed in the model through a rotational spring. The load introduction components into the column web panel are represented as axial springs parallel to the beams centrelines and aligned with the beam flanges and the column web shear panel is represented through a diagonal spring.

The implementation of the models represented in Figs. 5 and 6 in Abaqus was made by defining the coordinates of some reference nodes and then assigning simple kinematic and static constraints between them. In Figs. 5 and 6 these constraints are represented by straight lines identified by the reference LE (link type constraint) and RE (rigid element type constraint).

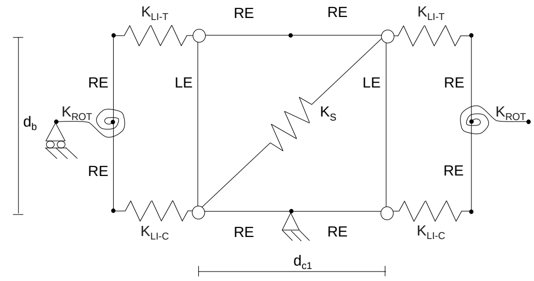

Fig. 5 – Beam-to-column joint model with beams of equal depth – single panel (SP) model.

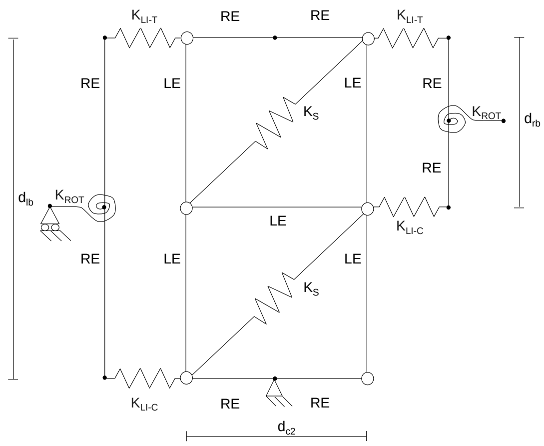

Fig. 6 – Beam-to-column joint model with beams of unequal depth – double panel (DP) model.

The LE constraint prevents the relative displacement between two reference nodes in the direction of the straight line that represents the LE constraint and imposes loads in these nodes that prevent that relative movement. The RE constraint prevents not only that same relative displacement but also the relative rotation of the reference nodes.

In Figs. 5 and 6, K_{LI-T}, K_{LI-C} and K_S represent the column web panel components in tension, compression and shear, respectively. The rotational spring K_{ROT} embodies the following components: column flange in bending, end-plate in bending, angles, bolts in tension and reinforcement in the case of composite structures. For the modeling of these components in Abaqus the following elements were used: (i) axial connectors for the shear panel component (represented by K_S), (ii) cartesian connectors for the tensile and compression behavior of the column web (represented by $K_{LI\text{-}T}$ and $K_{LI\text{-}C}$) – in this case an infinitely large stiffness is required in vertical direction – and (iii) rotation connector for the beam connection (with an infinitively large stiffness in the vertical and the horizontal directions).

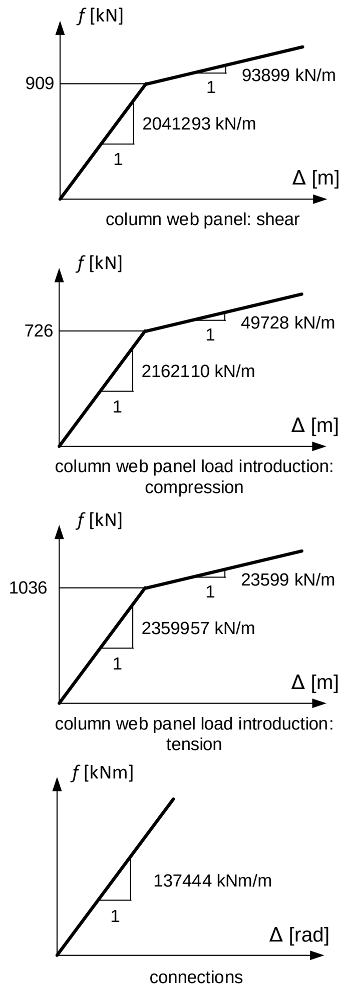

Fig. 7 – Constitutive relations of the connections and components.

A description of the components and their initial stiffness can be found in EN 1993-1-8 and EN 1994-1-1 (CEN 2005, CEN 2004). Simoes da Silva *et al.* (2002) proposed a bilinear characterization defining post-limit stiffness and ductility for the different components. In the following analyses, the behaviour shown in Fig. 7 for the components/springs was assumed. These constitutive relations are only intended for demonstration purposes and are not directly related to a specific geometry of a joint.

4.2. Model validation

In order to validate the results from Abaqus the behaviour of some isolated beam-to-column joint models was assessed making use of a simple analytical procedure implemented in algebra package Mathematica.

The numbering of the external nodal coordinates, *e.g.* the degrees of freedom (DOF), the numbering of springs and the numbering of external nodes in the beam-to-column joints models used for the validation procedure is represented in Fig. 7. Table 1 summarizes adopted geometrical dimensions for the validation procedure.

The boundary conditions for the isolated beam-to-column joint models are a double support in node 1 (DOF 1 and 2) and a vertical support in node 4 (DOF 11), see Figs. 5 and 6. Loads were applied monotonically and proportionally increased from zero to the values shown in Table 2.

Table 1 – Geometry

d_b [mm]	d_{c1} [mm]	d_{lb} [mm]	d_{rb} [mm]	d_{c2} [mm]
273.6	400	400	200	240

Table 2 – Loads.

loads	Node	Load (kNm)
symmetric load	6	400
	12	-400
	1,…,5,7,…,11	0
asymmetric load	6	-400
	12	-200
	1,…,5,7,…,11	0

In the following, **F** (12x1) is the nodal forces vector, according to the node coordinates numbering in Fig. 8, **f** is the vector of internal forces and **Δ** is the vector of the deformations in the components (both 7x1 in the single panel (SP) model and 8x1 in the double panel (DP) model) also according to the components numbering in Fig. 8. F_i and d_i are the nodal force and the nodal displacement, respectively, in DOF i and f_i and Δ_i are the internal force and the deformation, respectively, in spring i.

$$\mathbf{F}=\begin{bmatrix}F_1\\ \vdots\\ F_{12}\end{bmatrix},\ \mathbf{f}^{(SP)}=\begin{bmatrix}f_1\\ \vdots\\ f_7\end{bmatrix},\ \mathbf{f}^{(DP)}=\begin{bmatrix}f_1\\ \vdots\\ f_8\end{bmatrix},\quad \boldsymbol{\Delta}^{(SP)}=\begin{bmatrix}\Delta_1\\ \vdots\\ \Delta_7\end{bmatrix}\ \text{and}\ \ \boldsymbol{\Delta}^{(DP)}=\begin{bmatrix}\Delta_1\\ \vdots\\ \Delta_8\end{bmatrix} \qquad (1)$$

The procedure comprises four steps:

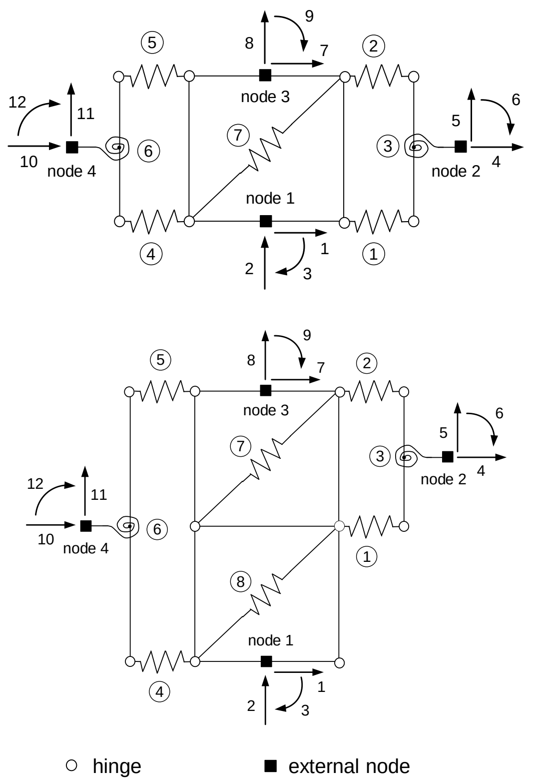

Fig. 8 – External degrees of freedom, springs and external nodes numbering for SP model (top) and for the DP model (bottom).

Step 1 - Compute the internal forces in all of the springs. The support reactions were computed making use of statics leading to

$$\begin{bmatrix}F_1\\ F_2\\ F_{11}\end{bmatrix}=\begin{bmatrix}-F_{10}-F_4-F_7\\ -F_5-F_8-F_{11}\\ \dfrac{2}{z_c}\left(\begin{array}{c}-F_3-F_6-F_9-F_{12}-F_7 z_b-\\ \left(F_{10}+F_4\right)\dfrac{z_b}{2}+F_5\dfrac{z_c}{2}\end{array}\right)\end{bmatrix} \qquad (2)$$

for the single panel isolated beam-to-column joint model and

$$\begin{bmatrix}F_1\\ F_2\\ F_{11}\end{bmatrix}=\begin{bmatrix}-F_{10}-F_4-F_7\\ -F_5-F_8-F_{11}\\ \dfrac{2}{z_c}\left(\begin{array}{c}-F_3-F_6-F_9-F_{12}-F_7 z_{bL}-\\ F_{10}\dfrac{z_{bL}}{2}-F_4\left(z_{bL}-\dfrac{z_{bR}}{2}\right)+F_5\dfrac{z_c}{2}\end{array}\right)\end{bmatrix} \qquad (3)$$

for the double panel isolated beam-to-column joint model.

Step 2 - From the free body diagram of the beam-to-column joint models, the internal forces in the components were computed again only making use of statics leading to

$$\mathbf{f}=\begin{bmatrix}-\dfrac{F_6}{z_b}+\dfrac{F_4}{2}\\ \dfrac{F_6}{z_b}+\dfrac{F_4}{2}\\ -F_6\\ \dfrac{F_{12}}{z_b}-\dfrac{F_{10}}{2}\\ -\dfrac{F_{12}}{z_b}-\dfrac{F_{10}}{2}\\ F_{12}\\ \dfrac{\sqrt{z_c^2+z_b^2}}{z_c}\left(F_7+\dfrac{F_{10}+F_4}{2}+\dfrac{F_{12}+F_6}{z_b}\right)\end{bmatrix} \qquad (4)$$

for the single panel isolated beam-to-column joint model (Fig. 5) and

$$\mathbf{f}=\begin{bmatrix}-\dfrac{F_6}{z_{bR}}+\dfrac{F_4}{2}\\ \dfrac{F_6}{z_{bR}}+\dfrac{F_4}{2}\\ -F_6\\ \dfrac{F_{12}}{z_{bL}}-\dfrac{F_{10}}{2}\\ -\dfrac{F_{12}}{z_{bL}}-\dfrac{F_{10}}{2}\\ F_{12}\\ \dfrac{\sqrt{z_c^2+z_{bR}^2}}{z_c}\left(F_7+\dfrac{F_{10}+F_4}{2}+\dfrac{F_6}{z_{bR}}+\dfrac{F_{12}}{z_{bL}}\right)\\ \dfrac{\sqrt{z_c^2+\left(z_{bL}-z_{bR}\right)^2}}{z_c}\left(\dfrac{F_{12}}{z_{bL}}-\dfrac{F_{10}}{2}-F_1\right)\end{bmatrix} \qquad (5)$$

for the double panel isolated beam-to-column joint model (Fig. 6).

Step 3 - Compute the deformations in all the components. The deformations in the components were computed making use of the internal forces and the uniaxial constitutive relations shown in Fig. 7.

Step 4 - Compute the nodal displacements of external nodes. The displacements were computed making used of Second Castigliano's Theorem. This theorem applied for the isolated beam-to-column joint models (where the only deformable elements are the springs) states that the displacement in any of the external DOF represented in Fig. 8 can be computed through the sum of the product of the deformation in each spring caused by the actual load and the internal force in that same spring caused by a unit load applied in the DOF where the displacement is wanted. For instance the displacement in DOF j may be computed through eq. (5).

$$d_{\mathrm{j}} = \sum_{i=1}^{7(\mathrm{SP})\text{ or }8(\mathrm{DP})} \left(\Delta_i^{(\mathrm{Load})} \times f_i^{(F_{\mathrm{j}}=1)} \right) = \left(\mathbf{\Delta}^{(\mathrm{Load})} \right)^{\mathrm{T}} . \mathbf{f}^{\,(F_{\mathrm{j}}=1)} \tag{5}$$

The former procedure is suitable for statically determinate structures for the elastic and for the post-elastic range when the behaviour of the components is holonomic and has no softening.

Fig. 9 and Fig. 11 illustrate the bending moment-rotation curve of right side (node 5 in Fig. 8) of SP and DP joint model under symmetric and asymmetric loading conditions, respectively. For both cases, the yielding of the first component (column web compression) and the yielding of the second component (column web tension) are noted.

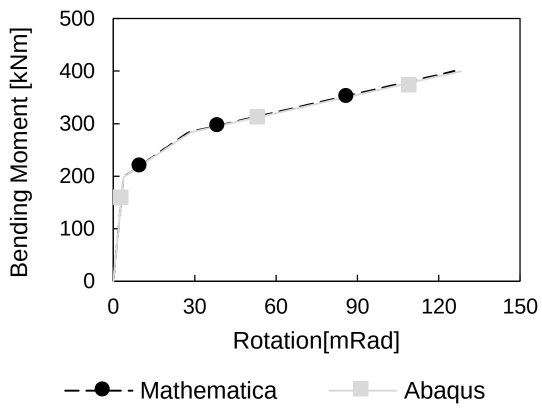

Fig. 9 – Moment-rotation curve for right side of SP model under symmetric load condition (node 5 in Fig. 8).

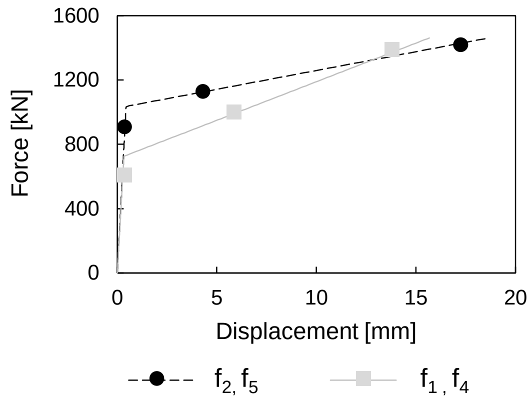

Fig. 10 – Force-displacements relationship for tension (f_2, f_5) and compression (f_1, f_4) components in SP model.

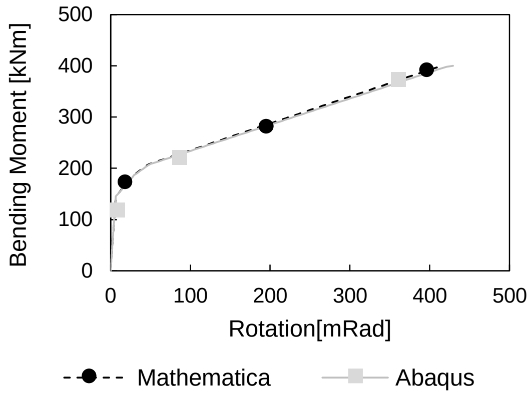

Fig. 11 – Moment-rotation curve for right side of DP model under asymmetric load conditions (node 5 in Fig. 8).

In Fig. 10 and Fig. 12, the behaviour of the individual components is highlighted in case of SP and DP models respectively. It can be seen from Fig. 9 to Fig. 12 that the values obtained with Abaqus match the analytical results from Mathematica.

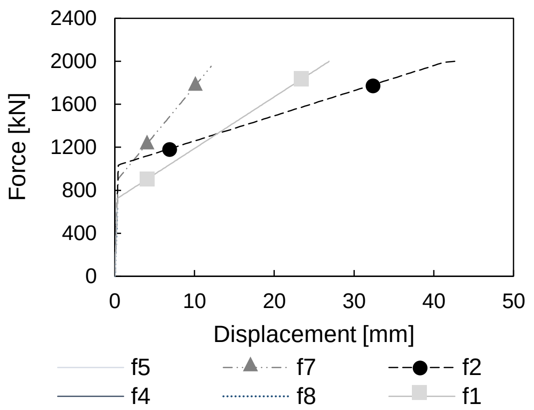

Fig. 12 – Force-displacements relationship for tension (f_2, f_5), compression (f_1, f_4) and shear (f_7, f_8) components in DP model.

Fig. 13 depicts the undeformed (gray lines) and typical deformed patterns (orange lines) of the beam-to-column joint models. As would be expected, the SP joint model under symmetric load conditions does not show any shear deformation and the DP joint model shows shear deformations irrespective of the loading condition considered.

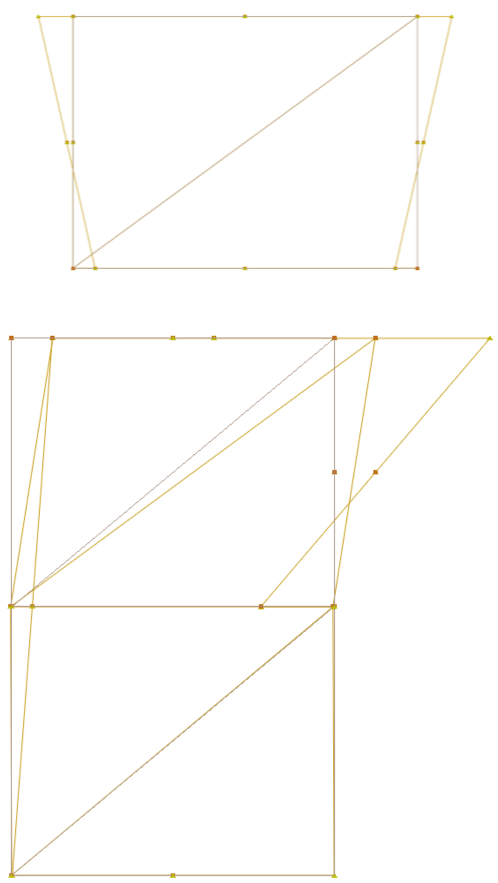

Fig. 13 – Deformed shape for the SP model for symmetric load (top) and for the DP model for asymmetric load (bottom).

5. CASE STUDY

In order to assess the differences in the results of the structural analysis of complete frames when the beam-to-column joints are is properly modelled, the steel frame shown in Fig. 14 was analysed for the load conditions shown in Table 3 and Fig. 14.

The geometric characteristics of the beam-to-column joints are depicted in Table 4 and the joints numbering is illustrated in Fig. 15 together with a typical deformed configuration of the frame.

Table 3 – Load Combinations

	F_1 [kN]	F_2 [kN]	p [kN/m]
LC1	10	20	2.5
LC2	25	50	6.25
LC3	40	80	10.0

Table 4 – Joint geometric characteristics

	N4 N7	N5A N8A	N5B N8B	N6 N9
d_c [mm]	160	180	180	160
d_b [mm]	300	300	200	200

The constitutive relations used for the joints springs were the ones represented in Fig. 7 and the beams and columns were assumed to have a linear and elastic behavior.

Two modeling strategies were considered: (i) Model A: SP joints model have been used for all the joints, even for joints with beams of unequal depth (d_b was assumed equal to 200 mm in Nodes 5 and 8) and (ii) Model B: SP joints model have been implemented in Nodes 4, 6, 7 and 9 and DP joint models have been used for Node 5 and 8.

The bending moments at the beams' ends, resulting from the structural analysis of the frame, are shown in Table 5 (*a* refers to the left beam and *b* refers to the right beam).

Table 5 shows that the inaccurate modelling of the beam-to-column joints may lead to significant errors in the internal forces when beams with unequal height are used. To illustrate these differences, the bending moment vs. rotation of the rotational spring 3 representing the right connection of the beam-to column joints' models in Fig. 8 is shown in Fig. 16 for models A and B for all the load conditions considered.

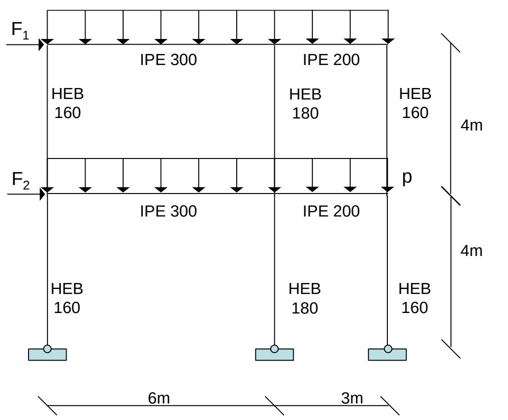

Fig. 14 – Steel frame: dimensions, sections, loads

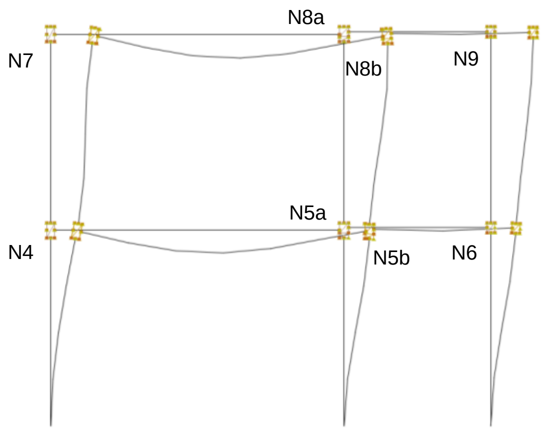

Fig. 15 – Deformed shape of steel frame and joints' numbering.

Table 5 – Bending moments (kNm).

	LC1		LC2		LC3	
	Mod. A	Mod. B	Mod. A	Mod. B	Mod. A	Mod. B
N4	18.3	19.1	61.3	47.8	60.5	78.5
N5a	71.2	76.8	150.7	192.1	174.7	254.4
N5b	9.2	9.7	13.5	24.3	13.0	20.3
N6	18.0	16.3	51.6	40.9	101.2	77.0
N7	19.8	21.1	48.0	52.8	65.0	81.8
N8a	58.6	61.8	138.8	154.7	166.3	244.7
N8b	20.4	21.1	45.5	52.8	37.9	80.2
N9	7.2	5.8	22.6	14.5	57.4	29.1

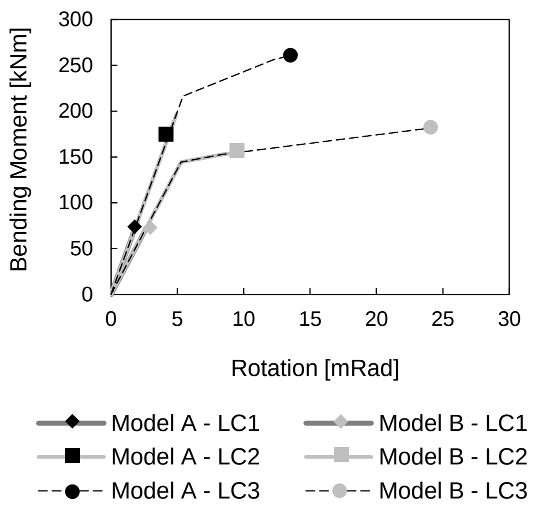

Fig. 16 – Bending moment vs. rotation of spring 3.

Fig. 16 shows that spring 3 (i) in LC1 remains in the elastic range in both models, (ii) in LC3, the most severe combination, determines the development of plasticity in the connection for both models but (iii) on the other hand, in LC2 determines that plasticity occurs only in model A, *i.e.* in this later loading condition the non-linearity in is only caused by the inaccuracy of the model.

6. CONCLUSIONS

The paper has emphasized the need of macro-elements for beam-to-column joint modeling. In was shown that, when compared with the use of springs attached to the ends of beams, this modeling strategy would allow to: (i) reduce the computational cost; (ii) overcome numerical difficulties due to nonlinearities; and (iii) provide a more rigorous modeling of the beam-to-column joints.

Two macro-models suitable for steel beam-to-column joints with beams of equal and unequal depth were presented and their modeling in Abaqus was explained.

These models were validated by means of an analytical procedure, and later were included in a 2D steel frame.

The structural analysis of the steel frame highlighted the potentialities of the proposed models showing that the inaccurate modelling of the beam-to-column joints may lead to significant errors in the results.

This work will carry on to the formulation of a new finite element suitable for steel beam-to-column joints.

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