BEHAVIOR OF HIGH-STRENGTH STEEL WELDED BEAM-TO-COLUMN JOINTS WITH BEAMS OF UNEQUAL HEIGHT

Sandra Jordão[a], L. Simões da Silva[b], Rui D. Simões[c]
[a,b,c]University of Coimbra, Coimbra, Portugal
[a]sjordao@dec.uc.pt, [b]luisss@dec.uc.pt, [c]rads@dec.uc.pt

ABSTRACT

Joints have a significant role on the global behavior of a steel structure. Although the most current joint typologies are already covered by several design codes, there are others, such as internal joints with beams of unequal height, which are not yet normalized. The aim of this paper is to contribute to a methodology based on the "component method" of EC 3 for this new type of node with high-strength steel S690. Finite element models calibrated with experimental results were developed, as well as analytical models tailored for the new type of joint and high-strength steel. Some modifications of state-of-the-art formulations (design rules) are proposed.

1. INTRODUCTION

The structural behavior of joints is complex in nature, due to the variety of components, geometry and phenomenon involved (plasticity, contact, non linearity, instability, etc) (Simões da Silva e Gervásio, 2007). Nevertheless, due to the huge role that connections have in construction, both in terms of structural behavior and cost (50% of the cost of the structure (Evers and Maatje, 1999)), the effort in research on this theme has grown considerably over the last 30 years (Nethercot, 2007). The subjects have been diverse, but a considerable cut has been into the development of the procedures related to the component method. In that view, there are still some important issues that haven't been addressed so far. One of them is the behavior of internal joint with beams of different heights, and the other is the updating of the Eurocode rules for high strength steel. The present paper addresses these two subjects. The procedure includes the development of finite element models (Fig. 3) calibrated with experimental results (FEM-CER) (Fig. 4), which are used in the development of analytical models for the new type of joint and S690.

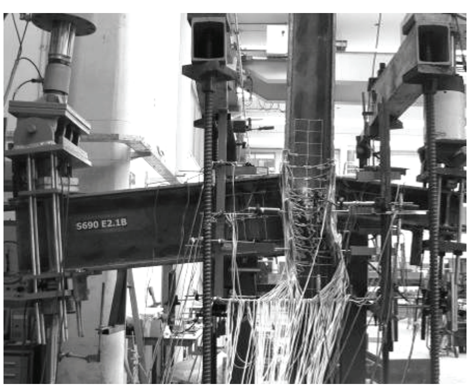


Fig. 1. Test S690E2 (M-/M-)

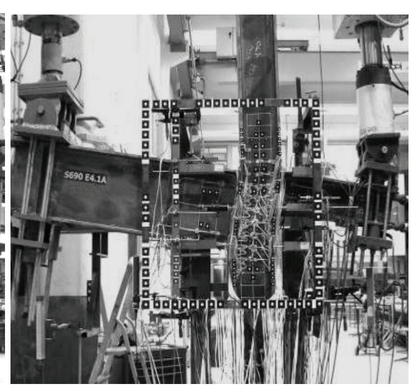


Fig. 2. Test S690E4 (M-/M+)

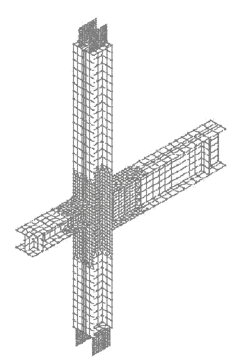


Fig. 3. FEM (E2, E4)

Thirteen full scale tests were performed (5 for S355 and 8 for S690). IPE400, HEB200 and HEB240 profiles were used for beams and column, respectively. Two types of joints were considered: i) internal node with beams of different heights (INBDH), with two different loading conditions corresponding to different shear levels in the web panel (Fig. 1 and Fig. 2); ii) external node and internal node symmetrical, to provide reference data for situations that are already in EC3 (Jordão, 2008).

2. ANALYTICAL MODEL

2.1 Analytical Jaspart Model

The rotational behavior of a welded joint can be expressed as the sum of the distortion of the web panel, associated with shear, and the load introduction rotation, associated with the forces directly transmitted through the beam flanges (Fig. 4). Atamaz and Jaspart (Atamaz and Jaspart, 1989) and Jaspart (Jaspart, 1991) defined models for each rotational component (Fig. 5 and Fig. 6) in terms of resistance and deformability, which became the basis of the component method.

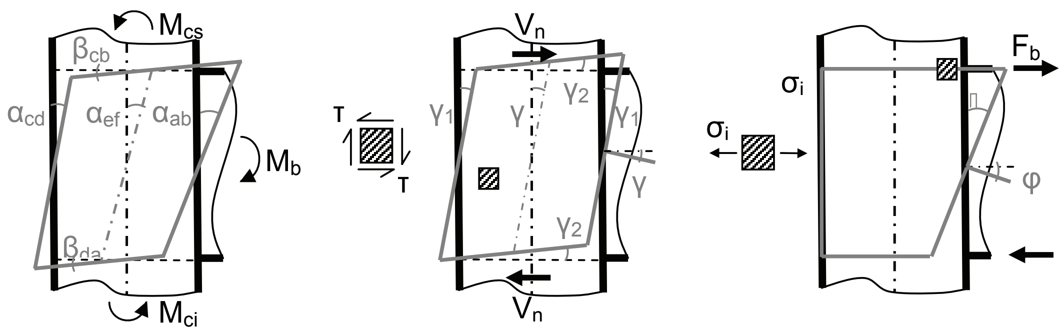


Fig. 4. Rotational behavior of a welded joint: Total, Shear and Load introduction.

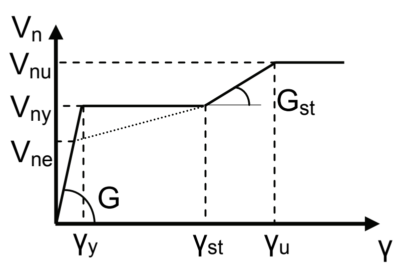


Fig. 5. Shear curve

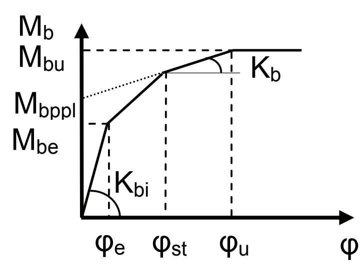


Fig. 6. Load introduction curve

The models consider the geometrical and material properties of the joint, and the von Mises yield criterion is used to account for the stress interactions. The models are valid up to failure except when instability is the failure mode. In this case the model does not mimic the negative slope end portion of the curve. The components of the stress state on the column web panel are: i) normal stresses, associated with the forces from the beams (σ_i) (localized effect); ii) shear stresses, associated with shear forces on the web panel (τ) (constant through out the web panel); and iii) normal vertical stresses, associated with column bending and axial force (σ_n), (constant through out the web panel). The relevant interactions are: τ with σ_n, and τ with σ_i.

2.2 Analytical Jaspart Modified Model

In order to deal with INBDH, the web panel is divided into two subpanels, corresponding to two areas with different shear values (Analytical-Jaspart Modified model - AJM) (Jordão, 2008) and (Jordão et al., 2007). The same assumptions already adopted in the Analytical-Jaspart model are used to account for the interaction between the internal forces on the column web panel. In the case of INBDH, the relevant combinations between σ_i, τ and σ_n lead to a higher number of possibilities that have to be accounted for. Fig. 7 and Fig. 8 show a schematic for E2.

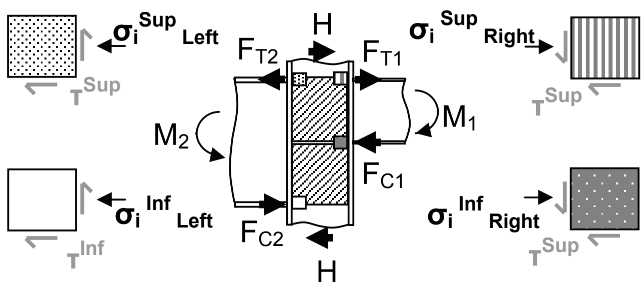


Fig. 7. $\sigma_i - \tau$ interaction

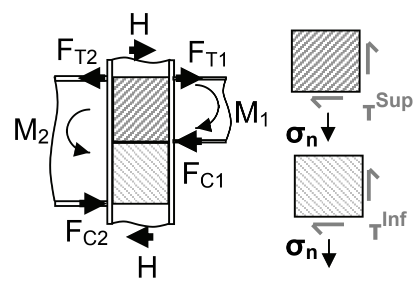
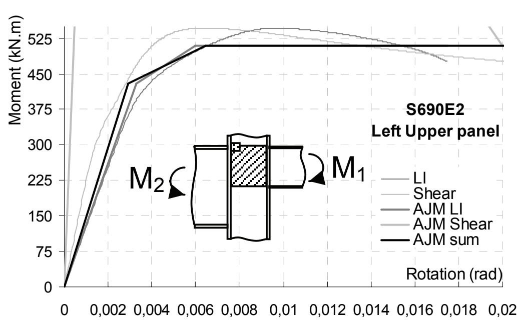
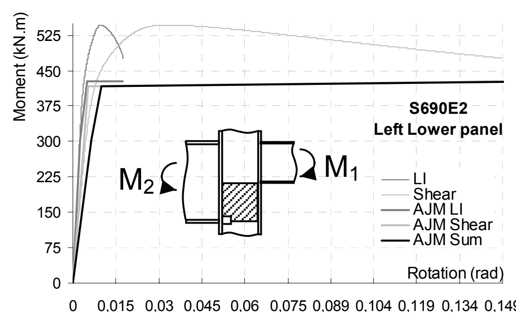


Fig. 8. $\sigma_n - \tau$ interaction

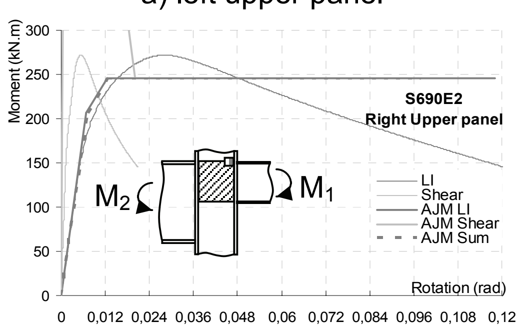
For each case, the load introduction (AJMLI) and shear (AJMShear) curves for AJM model are established. Homologous curves were obtained from FEM-CER (LI and Shear) (Fig. 9). AJMLI and AJMShear curves are added, yielding the AJM moment/ rotation curve (AJMSum), for the right and left joints, upper and lower panels (Fig. 9).



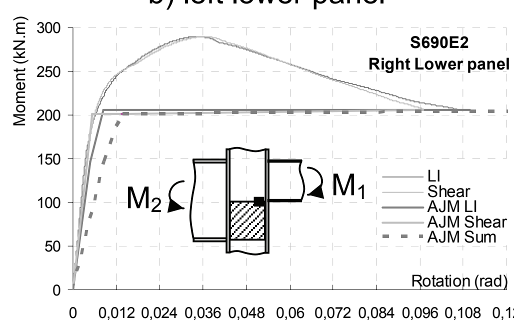
a) left upper panel



b) left lower panel



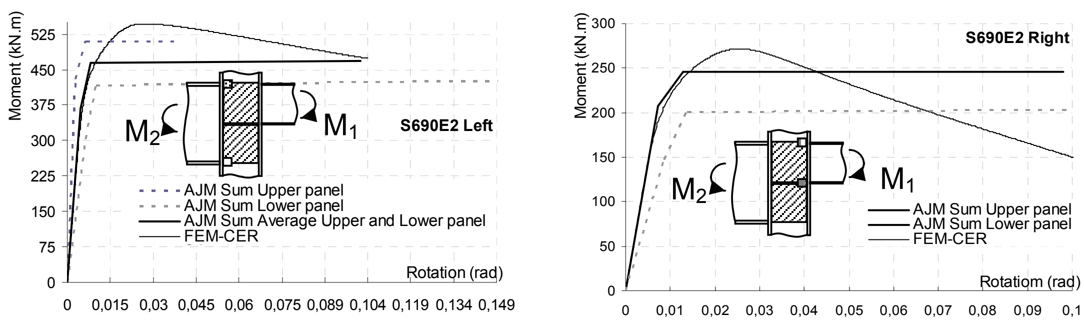
c) right upper panel



d) right lower panel

Fig. 9. M-θ curve (AJM) as the sum of LI and shear curves

Since it is sought to obtain a single curve for each side of the joint, a procedure must be established to concatenate the two curves obtained for each joint. For the left side it is assumed that the upper and the lower panels contribute equally to the global response, so the average is used (Fig. 10a)). For the right side the response of the joint is determined by the upper panel alone (Fig. 10b)). These figures illustrate the comparison of AJM *vs* FEM-CER. The good adjustment shows that the AJM model is suited for INBDH. A description of these topics may be found in (Jordão, 2008).



a) Left joint b) Right joint

Fig. 10. M-θ curve (AJM): average of M-θ curve for upper and lower panels

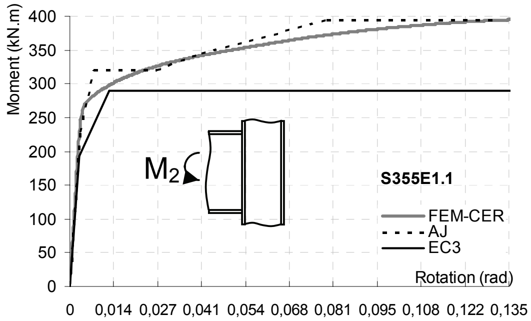
An identical study was performed for S355 models (Jordão, 2008). The same level of adjustment was reached at elastic and post-yielding range, but better agreement was reached at maximum load. This raises the question whether AJ, AJM and EC3 models are adequate for S690. This issue is discussed in the following sub section.

3. APPLICATIONS OF THE ANALYTICAL MODEL

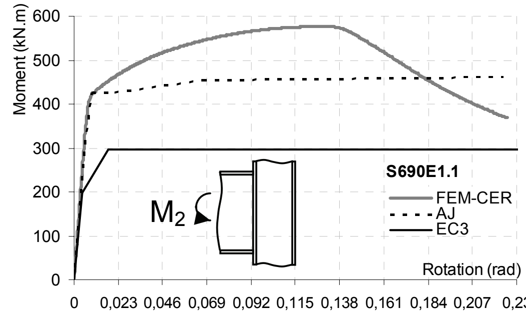
3.1 State-of-the-art analytical models: Extension for S690 steel grade

For joints with high strength steel, two different levels of evaluation can be established. Firstly, comparing the FEM-CER and the AJ model will show whether the AJ model still yields good results for high strength steel. Secondly, comparing equivalent joint typologies for S355 and S690 steels should highlight the qualitative differences that occur because of the use of different steel grades. This assessment is carried out using only the typologies that are currently covered by EC3 (E1: external and E3: internal symmetrical), in order to avoid the influence of coupled effects due to the new node configuration.

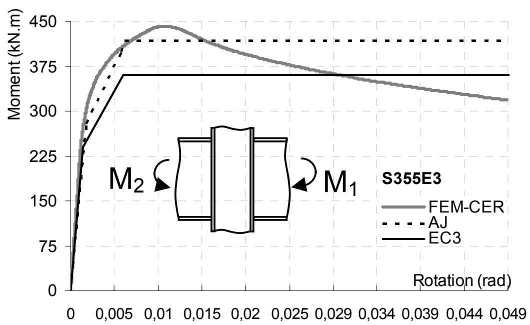
The results for S690 models are presented along with homologous results, from a similar study undertaken on S355 steel grade prototypes (Jordão, 2008). The reason for this parallel presentation is that the AJ model was established and calibrated for mild steel, and the comparative analysis will bring in information that may be used to interpret any maladjustment between FEM-CER and AJ models for S690 steel grade joints. A comparison is set between the FEM-CER and the AJ model and the FEM-CER and the EC3 model, for S355 and S690 steel grades, E1 (Fig. 11a) and Fig. 11b)) and E3 (Fig. 11c)) and Fig. 11d)) joints.



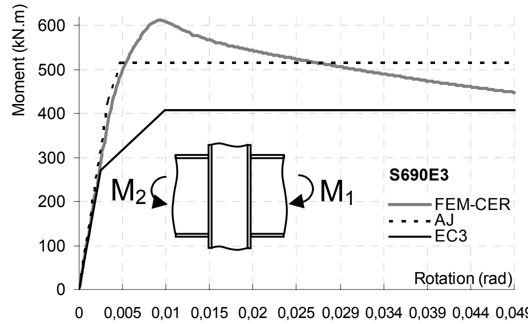
a) S355E1



b) S690E1



c) S355E3



d) S690E3

Fig. 11. FEM-CER *vs* AJ

In terms of AJ vs FEM-CER and EC3 vs FEM-CER, for both E1 and E3 S355 steel grade joints, the agreement is good for the whole curve. For steel grade S690 joints the agreement is similar, except for the maximum load, where a more conservative value is reached. The AJ model estimates the maximum load of the joint and the corresponding failure mode. For test E1 (external node), the difference between the maximum load and the predicted load, is Δ=21% (Δ=0.1%), corresponding to shear failure. The values in brackets correspond to the S355 tests, for comparison. For test E3 (internal node with symmetrical loading), this difference is Δ=17% (Δ=7.5%) and is governed by instability. The EC3 model predicts the plastic resistance of the joint. A similar comparison reveals Δ=29% (Δ=0.1%) and Δ=20% (Δ=3%) for tests E1 and E3, respectively. In order to assess the reason for this poor agreement in terms of maximum loads, for the S690 steel grade models, both instability and shear formulations are analyzed. Eq. 1 describes the shear area (EC3) for welded sections

$$A_{vc} = \eta h_{wc} . t_{wc} = \eta A_{wc} \tag{1}$$

where h_{wc} is the height and t_{wc} is the thickness of the column web, respectively, and η is equal to1.2 for steel grades S460 and below, and is given by 1.0 for higher steel grades (EC3, 2004). The calculations using the AJ model show that the value 1.0 leads to very poor agreement, so a new formula is proposed, that accounts for the actual shear area, and for the throat thickness of the weld explicitly (Eq. (2)).

$$A_{vc} = \left(h_{wc} - 2t_{fc}\right)t_{wc} + 4.a_c^2 + \left(2.a_c\sqrt{2} + t_{wc}\right)t_{fc} / 2 \tag{2}$$

t_{fc} is the thickness of the column flanges and a_c is the weld throat. Solving *Eq. (1)* with respect to η shows that this parameter is the ratio between A_{vc} and A_{wc}, so a parametric study on the influence of a_c on the η and on the shear area was performed (Table 2, Fig. 12 and Fig. 13).

a_c	$0.5.t_{wc}$ (lower bound)	$0.7.t_{wc}$ (used in prototypes)	t_{wc} (upper bound)
η	1.22	1.27	1.35

Table 1. Parameter η as function of a_c

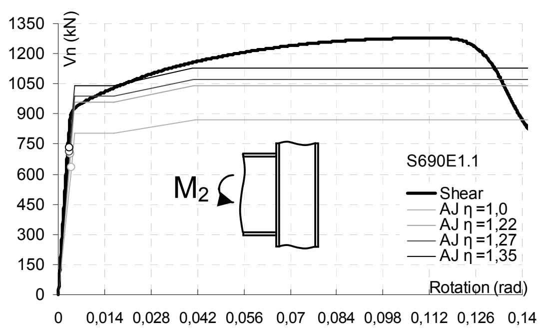


Fig. 12. η effect on shear rotation curve

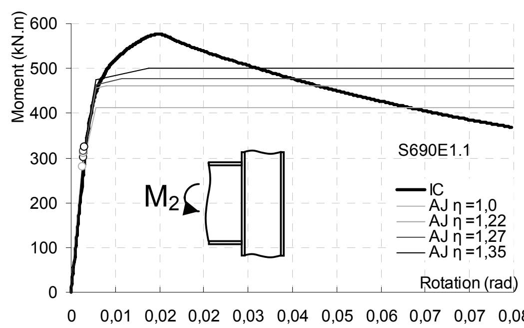


Fig. 13. η effect on LI rotation curve

Figs. 12 and 13 indicate that η influences the complete load-rotation curves, except for the elastic part of the LI curve. It also shows that the best adjustment is for values of η in the range 1.22 to 1.35. Thus, for S690 steel grade welded sections, (Eq. (2)) should be adopted for the evaluation of the shear area.

The failure mode for tests E3 is instability of the column web in compression. Comparison between the FEM-CER and AJ results have shown larger differences for S690 when compared to S355. This may results from the estimate of the elastic critical load of the compressed web, taken by Jaspart (Jaspart, 1991) as:

$$P_{vc} = \left(h_{wc} - 2t_{fc}\right)t_{wc}k\frac{\pi^2 E}{12\left(1-\upsilon^2\right)}\left(\frac{t_{wc}}{h_{wc} - 2t_{fc}}\right)^2 \tag{3}$$

that underestimates the theoretical solution by 27.3% for simply-supported conditions. Additional differences may also results from the definition of the “buckling length”, taken as $h_{wc} - 2\ t_{fc}$, which neglects the influence of the size of the weld for welded profiles or the root radius for rolled sections. The application of the EC3 model to S690 is much more conservative, when compared to S355. This is linked to the plate buckling curves adopted in Part 1.8 of EC3 that penalize high strength steels excessively. To illustrate this clearly, a S355 vs S690 results comparison must be set. Since S355 profiles are rolled and S690 profiles are welded, it is first necessary to calculate equivalent rolled sections for S690 joints (Jordão, 2008). The S690/S355 gain in nominal yield strength is 48.6%. Using the equivalent S690 “rolled” sections, the S690/S355 gain in the shear and the tension components is 42%, and in the compression component is 30%. This is due to the fact that λ_p (plate slenderness) (Eq. (4)) is highly penalized when $f_{y,wc}$ increases, leading to a considerable reduction in ρ (compression resistance reduction factor to account for plate buckling of the column web panel) (for example, λ_p=0.89 and ρ=0.85 for S355, and λ_p=1.14 and ρ = 0.71 for S355), that reduces the advantage of $f_{y,wc}$ on the resistance (Eq. (5)). The authors propose that new plate buckling curves should be derived for S690. Further investigation is due to clear this matter.

$$\overline{\lambda_p} = 0.932\sqrt{\frac{b_{eff,c,wc}.d_{wc} \cdot f_{y,wc}}{E.t_{wc}^2}} \tag{4}$$

$$F_{c,wc,Rd} \leq \omega.\rho.b_{eff,c,wc}.t_{wc} \cdot f_{y,wc} \tag{5}$$

3.2 β-parameters: Extension for internal nodes with beams of different heights

Part 1-8 of EC3 (EC3, 2005), presents formulation for β parameters for internal nodes symmetrical. For INBDH the code indicates that: "the actual distribution of shear stresses in the column web panel should be taken into account". In order to do so, the forces entering the web panel have to be considered in terms of magnitude and position. Fig. 14a) and Fig. 14b) show the load schematics for the two types of node. From those it is possible to withdraw formulation for β parameters for INBDH. Table 3 shows a comparison between the β formulation for EC3, and INBDH nodes.

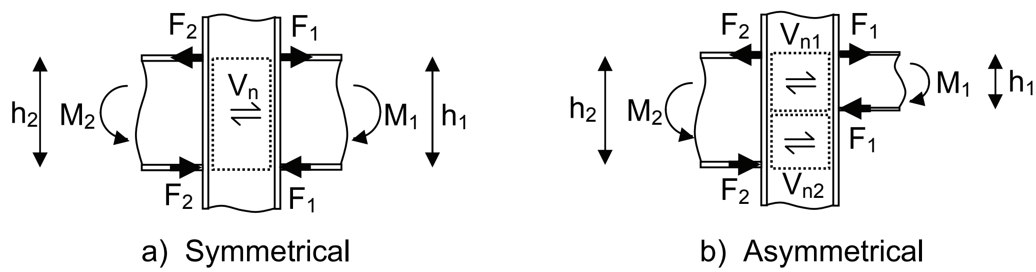
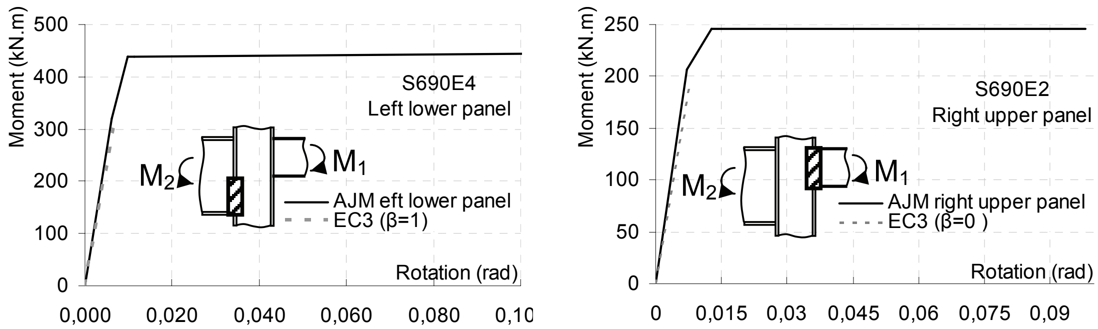


Fig 14. Load in the web panel in symmetrical and asymmetrical internal nodes

EC3 node configurations	$V_n = \beta_1 F_1$ $V_n = \beta_2 F_2$	$\beta_1 \frac{M_1}{z} = \frac{M_1}{z} - \frac{M_2}{z}$	$\beta_1 = \left\lvert 1 - \frac{M_2}{M_1} \right\rvert$ $\beta_2 = \left\lvert 1 - \frac{M_1}{M_2} \right\rvert$
INBDH		$\beta_1 \frac{M_1}{z_1} = \frac{M_1}{z_1} - \frac{M_2}{z_2}$	$\beta_1 = \left\lvert 1 - \frac{M_2 z_1}{M_1 z_2} \right\rvert$ $\beta_2 = \left\lvert 1 - \frac{M_1 z_2}{M_2 z_1} \right\rvert$

Table 2. Parameter β: formulation for EC3 node configurations and INBDH

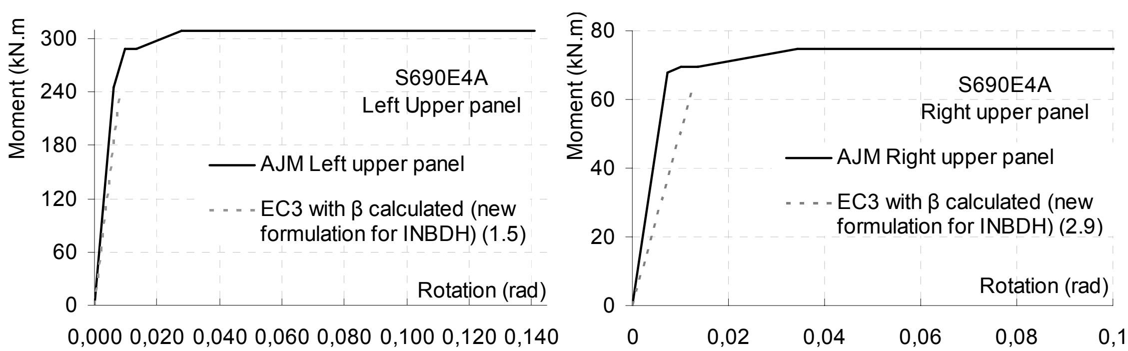
The β values calculated with INBDH formulation, concern only the upper panel. In the lower panel, the forces from the right beam cancel each other out, so the lower panel may be considered equivalent to an external node (β=1). To assess the quality of INBDH β formulation, a comparison was set between the initial stiffness of AJM and of EC3 model (β calculated with NBDH formulation). For all the joints studied, the referred comparison, established for the lower panel with β =1, yields a fair agreement (ex. in Fig. 15a) for S690E4), which confirms that the lower panel may be considered similar to an external node. In the case of E2 models, the forces entering the panel are similar on both sides, meaning that the shear value is null, so β should be also null. When considering this value, the agreement between the initial stiffness of the two models is reasonable (example in Fig. 15b) for S690E2).



a) Left lower panel (S690E4) b) Right upper panel (S690E2)

Fig. 15. Comparison between the initial stiffness of AJM and EC3 model

In the case of E4 joints, the shear value, in both the upper and lower panels is high, so these are the best cases to completely test the INBDH β formulation. Fig. 16a) shows an example for the left upper panel, where a reasonable adjustment is achieved. In the case of the right upper panel (Fig. 16b)), the value yielded from the new formulation leads to an initial stiffness that lies far from the one of AJM. This is due to the fact that the difference between the moments on both beams is considerable, which leads to non significative values for the right joint. The EC3 formulation prevents this situation by limiting the β value to 2.



a) Left upper panel (S690E4) b) Right upper panel (S690E4)

Fig 16. Comparison between the initial stiffness of AJM and EC3 model

Table 4 shows the results of the comparison between the initial stiffness of AJM and EC3 curve (β evaluated by INBDH formulation). The letters *c* and *a* stand for calculated, (β calculated by INBDH formulation) and adjusted (value that β parameter should read so that the initial stiffness of AJM and EC3 curves would be the same). If *a* and *c* are alike it means that INBDH formulation yields good results.

	S690E2A		S690E2B		S690E4A		S690E4B	
	c	a	c	a	c	a	c	a
Left ^{Upper}	0	0	0	0	1.5	1.25	1.5	1.2
Left ^{Lower}	1	1	1	1	1	1	1	1
Right ^{Upper}	0	0	0	0	2.9	1.6	2.9	1.4

Table 3. Parameter β (INBDH formulation): calculated and adjusted values

The adjustment is reasonable for all the cases, except for the right upper panel, due to the high difference between the moments on the joints, as explained above.
The implementation of the β formulation (INBDH) yields two different values for the joint in the left (upper and lower panel). Since it is sought to obtain a single value for β for each joint, a procedure must be established to concatenate the two β values obtained for the joint in the left. The approach used consists in determining the value of β that would make the initial stiffness of EC3 curve be similar to that of the AJM or FEM-CER, and relate that value to those obtained for left joint/upper panel and left joint/lower panel. The referred procedure is illustrated in Fig. 17 for S690E4A, and Table 5 summarizes the homologous results for the other studied models.

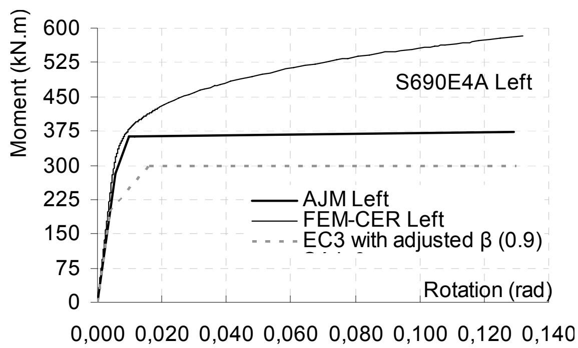


Fig. 17. β parameter for the left joint by adjusting the initial stiffness of EC3 curve to that of AJM

	S690E2		S690E4	
	c	a	c	a
Left ^{Upper}	0	0	1.5	1.2
Left ^{Lower}	1	1	1	1
Average	0.5	0.5	1.3	1.1
Left ^{Joint} (adjusted)	0.57		0.95	
Drift from average (%)	12	12	27	14

Table 4. β parameters for the left joint: upper and lower panels and the whole joint

For the configurations E2 (M-/M-), the β value obtained for the whole joint on the left, has a difference of 12% to the average of the β values obtained for the upper and lower panels, for both calculated and adjusted values.

For the configurations E4 (M+/M-), the β value obtained for the whole joint on the left, has a difference of 14% or 27% to the average of the β values obtained for the upper and lower panels, for calculated and adjusted values, respectively.

The β value obtained for the whole joint on the left seems to have a steady relation to the average of the β values, obtained for the upper and lower panels, except for the calculated values in E4 model. It is not yet possible to establish a relation between the parameters for the whole joint and those of the sub-panels. Further investigation is due.

4. CONCLUSIONS

Based on experimental and numerical evidence resulting from a series of tests on welded joints (external nodes, internal nodes with beams of similar height and internal nodes with beams of different height) in steels grades S690 and S355, it was possible to propose a model to predict the behavior of the column web panel with beams of different height. This model is an extension of the Analytical-Jaspart model that led to the EC3 rules for the evaluation of the behavior of the column web panel.

It is worth highlighting that the modified model (AJM) yields good results for S355 steels, reflecting the quality of the Analytical -Jaspart model for these steel grades.

Secondly, the adjustment for S690 steels is not so good. It is clear that some adjustments are necessary in terms of the shear resistance of the web panel and the evaluation of the stability of the compressed web. The implementation of final design rules to improve these aspects requires further work, notably a thorough parametric study to widen the limits of application.

An extension to the EC3 β parameters formulation is proposed for the case of internal nodes with beams of different heights. The application of the Analytical-Jaspart modified model allowed testing the quality of the referred extension. Some importance conclusions have been drawn concerning this topic, but clearly, further work is needed.

The problems presented are currently actively being looked into by the authors, and further developments and conclusions are expected.

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