NUMERICAL INVESTIGATION OF WEAK AXIS I PROFILE CONNECTIONS

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1. ABSTRACT

Beam to column connections of I profiles along the strong axis are usually bolted end-plate ones. Although connections along the weak axis of the column are typically designed as simple ones, in certain cases it is necessary for them to be able to undertake moments. The connection of the beam directly to the web of the column may lead to construction difficulties, especially in the case that a strong axis connection exists at the same position. An alternative is to use two end plates so that the beam is connected to the two flanges of the column. In this case, however, due to the lack of guideline in the relevant Eurocode 3 part 1-8 [1], the design of such a connection may be a cumbersome procedure. An alternative design approach is to employ numerical models. The scope of this study is the investigation of the behavior of such a connection by the means of detailed numerical models, capable of taking into account the different phenomena that may arise, such as material nonlinearity and unilateral contact between the various connection components. The investigation focuses on the impact of the various parameters that affect the capacity and ductility of the studied connection.

2. INTRODUCTION

In cases of structures having columns with standard I or H steel sections, in which bracing is not possible for a variety of reasons, usually architectural ones, the required horizontal rigidity is provided by moment resisting frames. In the later, appropriate moment resisting beam-column connections are used. Typically, the beams are connected with respect to the strong axis of the columns. The design of such connections is well covered in the relative norms (Eurocode 3-part 1.8). However, the design of moment connections in which the beams are connected with respect to the weak axis of the column sections, remains an open issue and the literature provides very limited information.The most prevalent weak-axis connection configuration is when the beam, welded to an end-plate, is directly connected to the column web (Fig.1a). Initially such connections were recognized as pinned ones. However, numerous studies dating back to the last century [2-6] showed that this idealization is not in accordance with the actual structural behavior. Regarding the calculation of its ultimate moment resistance, analytical methods based on yield line analysis and plate theory have been proposed [7] and designers have managed to exploit its rigidity. The most common case of failure, is the one due to the out-of-plane deformation of the column web. In addition, the prevailing failure mode is likely to take its toll on strong-axis connection capacity, given that the column web in shear is usually a critical component. Finally, the fabrication is more difficult as in the case that a strong axis connection also exists, manufacturing conflicts appear. An alternative design scheme for weak-axis bolted connections is when the beam is attached to the two flanges of the column with the use of two end-plates. The use of this dual end-plate weak-axis connection in 3-D joints is characterized by several advantages. The connections in the two axes can be assembled easier as they do not hinder with each other and no holes on the column web are required. Furthermore, the additional plates act as stiffeners for the strong-axis connection and provide further resistance for the column web in shear, tension and compression. The first variant of this configuration consists of two different attached plates corresponding to the upper and lower beam flanges (4 in total), ideal for column section with limited heights (Fig.1b) [8],[9]. A second variant, consists of two uniform end-plates for both the column flanges and the beam, bolted to each other (Fig.1c). A series of numerical tests in 3-D joints [10], clearly showed that the additional plates in the weak-axis connections have a significant increase in the strong-axis stiffness and that the height of those plates is a key parameter to consider in 3-D joint design. In addition, the authors proposed a mechanical model [11] for a new component that appears in such connections called E-stub that accounts for the column flanges in bending and the column web and end-plates in tension. The analytical model of this new component facilitates the calculation of strong-axis stiffness of the connection in 3-D joints. Finally, from other studies in connections similar to the aforementioned [12], it has been emphasized the importance of a good welding quality in order for them to be able to distribute the internal forces in all the available components and thus perform effectively. From this review, it can be concluded that all studies to this date have focused on the coupling effects between connections in the two axes rather than to the investigation and examination of the failure mechanisms observed in individual weak-axis connections.

In this paper, the studied connection consists of a uniform attached plate, welded to the column flanges, bolted to an end-plate connected to the beam. An additional vertical stiffener has been added (Fig.1d) to increase the strength of the column-attached plates especially in column sections of significant heights (e.g. h>300mm). A parametric analysis on an individual connection has been conducted by means of numerical models in order to fully comprehend its structural capacity, ductility, vulnerability and its failure mechanisms. After presenting the procedure for the calibration of the numerical models and the relevant results, conclusions and suggestions to achieve the optimum effectiveness of the connection will be given by highlighting the main factors that affect its behaviour.

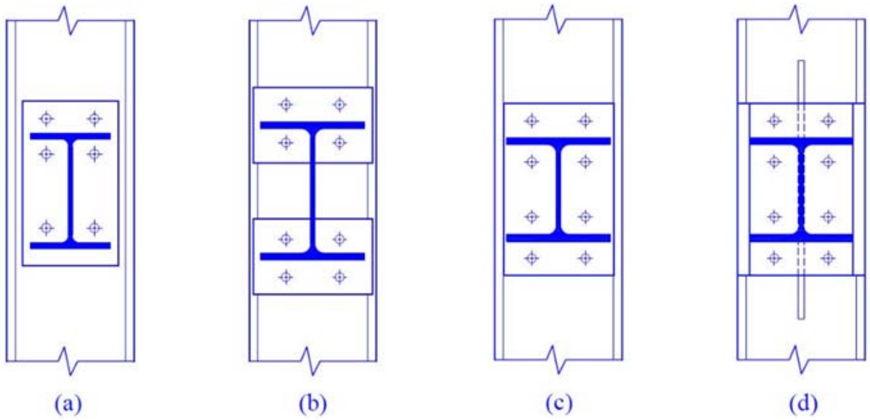

Fig. 1: Weak-axis connection configurations.

3. GEOMETRY OF THE UNDER STUDY MINOR AXIS CONNECTIONS

The geometry of the connection under investigation is presented in Fig. 2. In more detail, the connection consists of a HEB300 beam connected to a welded profile column with dimensions very close to those of the HD360x179 standard profile. The beam is attached to an end-plate with dimensions 300x500 and thickness of 20mm/25mm. The column has a plate with dimensions 370x500 attached to its flanges by means of welding. This plate is referred in the sequel as column-attached plate and for its thickness two cases were also considered (20mm/25mm). For all the studied models, full penetration welds were considered for the connection of this plate to the column flanges. A vertical stiffener of 161mm width and height/thickness of 500mm/750mm and 10mm/16mm respectively, reinforces the attached plate and connects it to the web of the column. The two assemblages (the beam with the end-plate and the column with the attached plate and the stiffener) are bolted together by means of standard ISO 4017 bolts, arranged in two columns and 4 rows, for which various alternatives were considered (see Table 1). In total, 14 different models were analyzed and assessed, resulting from a combination of the above-mentioned parameters. A full-strength connection was finally achieved.

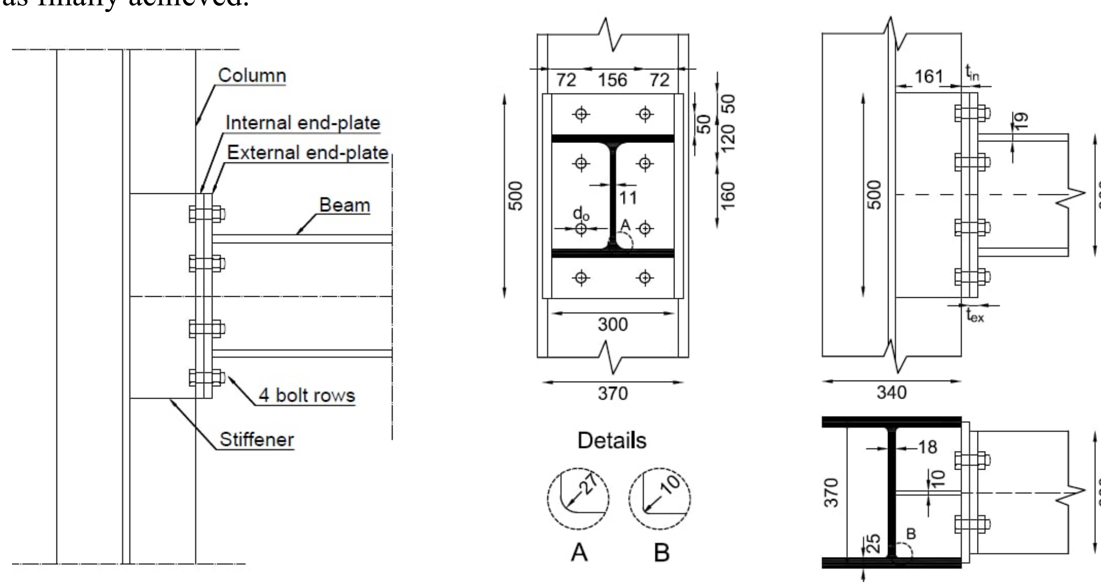

Fig. 2: Dimensions of the extended double end-plate weak axis connection

4. FORMULATION OF THE NUMERICAL MODELS

For the simulation of the various alternatives of the considered joint, 3D solid finite elements were used within the MSC Marc software environment [13]. In order to reduce the computational cost, the symmetry of the problem was utilized and thus only one half of the original connection was simulated. After a mesh sensitivity investigation that is not presented here for the sake of brevity, a combination of four-node and ten-node isoparametric tetrahedra were used for the discretization of the problem at hand. In more detail, the critical components of the connection were simulated with a finer mesh of ten-node solid elements while the less critical ones with four-node ones. The different meshes were connected using the built-in capability of MSC Marc to connect non-conforming meshes. The modelling was extended to include parts of the column below and above the connection. A typical story height of 3m was considered and half of the upper and lower parts of the column were simulated, equipped with a simple support at their ends. Concerning the modelling of the bolts, ten-node isoparametric tetrahedral elements were used. Instead of simulating the threaded part of the bolts, the shank diameter was reduced so that it corresponds to the reduced tension area. This simplification does not affect the obtained results. The welds between the beam end-plate and beam, stiffener and column were not simulated but considered as ‘glued’ for simplification purposes. The Young’s modulus was set equal to 200GPa and the Poisson’s ratio equal to 0.3 for all the different steel grades.

Material modeling

For the column, beam, beam end-plate, column attached plate and stiffener, S275 steel grade was assumed. For the material modelling, a multilinear curve based on experimental tests was adopted [14], with a fracture strain at 15% (Fig.5). For the bolts, two different grades were considered, 8.8 and 10.9. The adopted constitutive laws follow the ones proposed by Dessouki et al. [15], while the strain values after fracture were obtained from ISO 898-1, as 12% and 9% respectively (Fig. 3). Since it is expected certain model areas to develop large deformations under the applied loading, the non-linear relationship of *true stress* versus *true strain* needs to be considered. For this reason, values in Fig. 3,4 were converted from engineering to true stresses and strains according to the following expressions:

$$\sigma_{true} = \sigma_{nom}(1+\varepsilon_{nom}) \tag{1}$$

$$\varepsilon_{true} = \ln(1+\varepsilon_{nom}) \tag{2}$$

where σ_{true}, ε_{true} are the true stress and strain and σ_{nom}, ε_{nom} are nominal stress and strain.

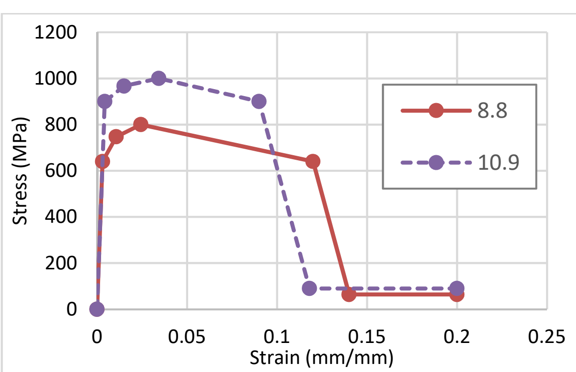

Fig.3: Stress-strain curves for 8.8-10.9 bolts

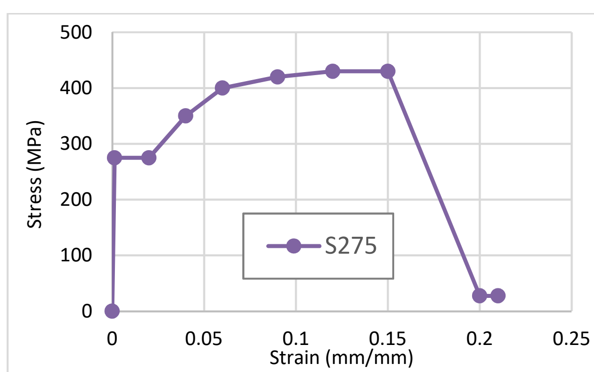

Fig. 4: Stress-strain curve for S275 steel

Load application and analysis

All the analyses were displacement controlled. A vertical displacement was applied at the free edge of the 1.5 m long beam, leading to a moment capacity of the beam which does not need to be reduced due to the presence of shear (the shear stresses that develop in the beam web are comparatively low). All the models were analyzed using a large strain total Lagrange formulation. The two main assemblages and the eight bolts were considered as different deformable bodies that may come into unilateral contact. Between the parts that come into contact, Coulomb friction is considered with a friction coefficient of 0.6. The computational solution strategy employed incremental load application with a relative convergence criterion based on the residual forces.

5. RESULTS

The first two models of the analysis, were examined in order to evaluate the significance of the additional vertical stiffener. The results showed a considerable improvement when the vertical stiffener is used. The absence of a stiffener (Model 0) can cause in a typical model a 17% decrease in the moment resistance (Fig.5). Models 1-5 exhibited a rather similar behavior with significant plastic deformations in the two upper bolt rows and in the beam end-plate, at the location where it is welded with the flanges of the beam. These models failed due to bolt rupture either from inadequacy of the bolts themselves (Models 2,4) or the development of prying forces between the two plates that caused an additional tension to the bolts (Models 1,3,5). When a thick beam end-plate (25mm) was combined with strong bolts (M30 10.9), the plastic strains began to be accumulate at the column's stiffener as well as the beam and column web (Fig.6). In the next three models and in order to curtail these plastic strains, given that this could affect badly a second connection in the strong-axis of the column, an increased height and thickness for the column stiffener was considered. This modification (Models 7,8,9) did not have a significant impact on the connection capacity but rather led to the decrease of the level of stressing of the column web. In addition, significant plastic strains were also observed in the column-attached plate. Models 10,11,12 were analyzed to examine the impact of an increased column-plate thickness from 20mm to 25mm. The results were satisfactory with a slight increase in the ultimate moment capacity and again a shift of the internal forces to the upper bolt rows and the beam end-plate. Finally, using a 32mm beam end-plate (Model 13) a moment capacity of 540.55kNm was obtained, that consequently renders the connection as a full-strength one. A plastic hinge was observed at beam (Fig.7) and the connection reached its limit with a simultaneous fail of the upper bolt row.

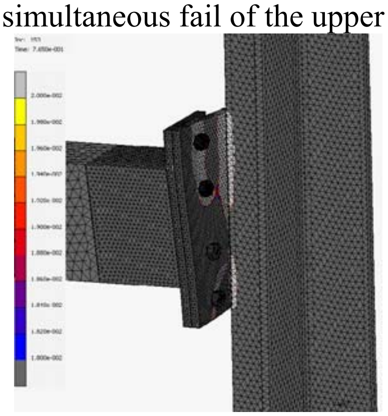

Fig. 5: Model 0-plastic strains

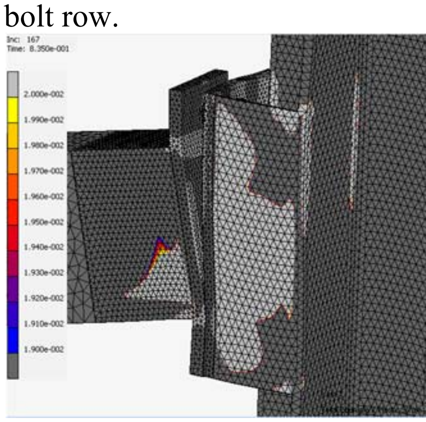

Fig. 6: Model 6-plastic strains

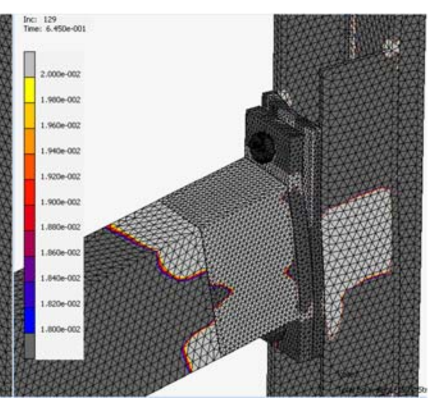

Fig. 7: Model 13-plastic strains

Fig. 8 presents the moment-rotation curves of all the considered connections.

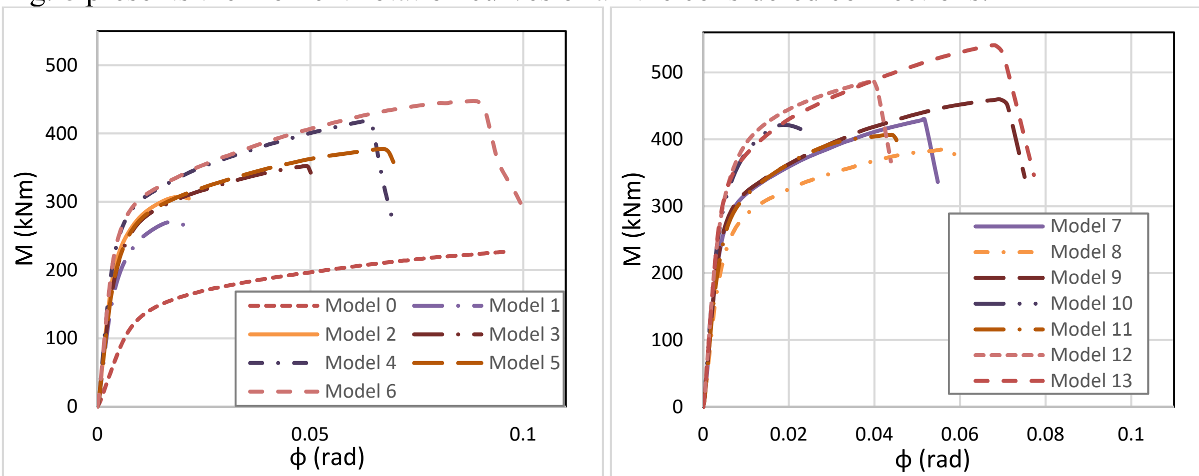

Fig. 8: Moment-rotation curves of the studied models

Table 1. Geometric details and overall response of the studied models

	Bolt type	End-plate thickness (mm)	Column-plate thickness(mm)	Column stiffener dimensions (mm)	M_{jRd} (kNm)	$M_{jRd}/M_{pl,HEB300}$ (%)
Model 0	M24 8.8	20	20	-	226.66	44.10
Model 1	M24 8.8	20	20	161x500x10	273.04	53.12
Model 2	M24 8.8	25	20	161x500x10	312.46	60.79
Model 3	M30 8.8	20	20	161x500x10	348.77	67.86
Model 4	M30 8.8	25	20	161x500x10	419.36	81.59
Model 5	M30 10.9	20	20	161x500x10	377.40	73.43
Model 6	M30 10.9	25	20	161x500x10	447.69	87.10
Model 7	M30 8.8	25	20	161x750x16	429.67	83.60
Model 8	M30 10.9	20	20	161x750x16	385.82	75.07
Model 9	M30 10.9	25	20	161x750x16	459.59	89.42
Model 10	M30 8.8	25	25	161x750x16	427.69	83.21
Model 11	M30 10.9	20	25	161x750x16	406.78	79.14
Model 12	M30 10.9	25	25	161x750x16	487.66	94.88
Model 13	M36 10.9	32	25	161x750x20	540.55	105.17

6. CONCLUSIONS

From the above investigation, one can assume that the use of the vertical stiffener is essential in order to avoid extreme deformations in the column web and in the attached plate, stress concentration in the welds between the column and its attached plate and therefore an early failure. The connections that correspond to Models 1-12, are classified as partially strength ones. The prevailing failure mode of the models was the one due to bolt rupture in the upper row. The reduced moment capacity of 20mm beam end-plate models can be partially attributed to the development of prying forces between the two end-plates that caused the early failure. However, when bolts are designed stronger, stress is mainly concentrated on the column stiffener due to its inadequate height and thickness and on the column web. By increasing the stiffener's height

and thickness, the plastic strains are observed in the column-attached plate and stress is once again shifted to the bolts and the beam end-plate. It can be concluded that the critical components by which such connection can be designed is the bolts in the upper row in tension and the T-stub in bending formed by the beam and its attached end-plate. For this reason and a when a full-strength connection is needed, attention should be given to these components.
More studies must be performed in order to provide additional insights of the overall performance of such connections and also examine the interaction in the presence of a second one in the strong axis.

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ΑΡΙΘΜΗΤΙΚΗ ΠΡΟΣΟΜΟΙΩΣΗ ΣΥΝΔΕΣΕΩΝ ΑΣΘΕΝΟΥΣ ΑΞΟΝΑ ΔΙΑΤΟΜΩΝ ΔΙΠΛΟΥ Τ

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ΠΕΡΙΛΗΨΗ

Οι συνδέσεις δοκού-υποστυλώματος διατομής διπλού ταυ κατά τον ισχυρό άξονα του υποστυλώματος συνήθως υλοποιούνται μέσω κοχλιωτής σύνδεσης με μετωπική πλάκα. Παρόλο που οι συνδέσεις στον ασθενή άξονα του υποστυλώματος συνήθως θεωρούνται απλές, επιτρέποντας την ανάπτυξη στροφών που προκύπτουν λόγω των δράσεων σχεδιασμού, ενίοτε η ανάγκη παραλαβής ροπής και στους δύο άξονες του υποστυλώματος (ισχυρό και ασθενή) γίνεται επιτακτική. Καθώς η σύνδεση της δοκού απευθείας στον κορμό του υποστυλώματος επισύρει πιθανές κατασκευαστικές δυσκολίες, ιδιαίτερα στην περίπτωση που υπάρχει σύνδεση στην ίδια θέση κατά τον ισχυρό άξονα, εναλλακτική λύση αποτελεί η σύνδεση της δοκού στα πέλματα του υποστυλώματος με την χρήση δύο μετωπικών πλακών. Εξαιτίας της περιορισμένης βιβλιογραφίας πάνω στο εν λόγω ζήτημα αλλά και της έλλειψης οδηγιών σχεδιασμού από τον σχετικό κανονισμό (Ευρωκώδικας 3, μέρος 1.8), ο προσδιορισμός της αντοχής και της συνολικής συμπεριφοράς είναι σχετικά δύσκολος. Εναλλακτική προσέγγιση υπολογισμού μιας τέτοιας σύνδεσης αποτελεί η χρήση αριθμητικών προσομοιωμάτων και ο προσδιορισμός της συμπεριφοράς μέσω της μεθόδου των Πεπερασμένων Στοιχείων. Σκοπός της παρούσας εργασίας, είναι η μελέτη της συμπεριφοράς και των μηχανισμών αστοχίας μίας τέτοιας σύνδεσης στον ασθενή άξονα μέσω τρισδιάστατων αριθμητικών προσομοιωμάτων. Η μελέτη επικεντρώνεται στην επιρροή του πάχους των μετωπικών πλακών, των διαστάσεων της ενισχυτικής λεπίδας του υποστυλώματος αλλά και της διαμέτρου και ποιότητας των κοχλιών, τα οποία καθορίζουν σε μεγάλο βαθμό την τελική αντοχή της σύνδεσης.