# NUMERICAL EVALUATION OF THE BEHAVIOR OF TUBULAR COLUMNS IN THE CONNECTIONS TO I BEAMS WITH WELDED REVERSE CHANNEL

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## ABSTRACT

This paper presents results of numerical study on the behaviour of tubular columns in the connections to I beams with U-profiles, effected on the basis of a parametric variation of the most significant characteristics of tubular profiles. The parameters considered are the thicknesses, the widths and the diameters of the tubular columns, the filling with concrete. The main purpose is the characterisation of the nonlinear behaviour of this type of steel joints. The results of the parametric study with numerical models allow correlate the parameters considered with the structural behaviour of the connection, defined by the resistance, the stiffness and the rotation capacity and calibrate the experimental tests. The numerical models are developed with the software LUSAS.

*Keywords:* tubular columns, reverse channel, numerical models, resistance, stiffness, rotation capacity.

## INTRODUCTION

When compared with other steel shapes, tubular profiles show a privileged structural behaviour due to their ability to withstand axial loads, bending in several directions and torsion, besides considerable advantages in terms of maintenance and aesthetics. The welded reverse channel connection is a good solution since it allows for a bolted joint between I beam to hollow column. Furthermore, this type of joint detail has a reasonable construction cost, is easy to implement and possesses large ductility through the deformation of the U web panel combined with the tubular column. Traditional solutions, like bolted and welded joints, or are impractical because there was no access to the interior of the tubular profile or become expensive due to the welding process at work.

Due to this fact, a lesser effort has been directed to research on these typologies, and less comprehensive methods of analysis or code formulation exists for their design. Eurocode 3 provides some rules for welded connections between tubular profiles, but not for bolted connections. However, due to its strategic importance, this topic has had several significant contribution in the last decade. Among those some research initiative from the "Comité International pour le Développement et l'Étude de la Construction Tubulaire" CIDECT stands out, aiming at the development of an unified design approach for steel joints by extending the field of application of the Eurocode 3 component method.

The main purpose of this investigation is the characterisation of the nonlinear cyclic behaviour of steel joints with welded reverse channel welded to hollow section columns and bolted to I profile beams, depicted in Fig. 1.
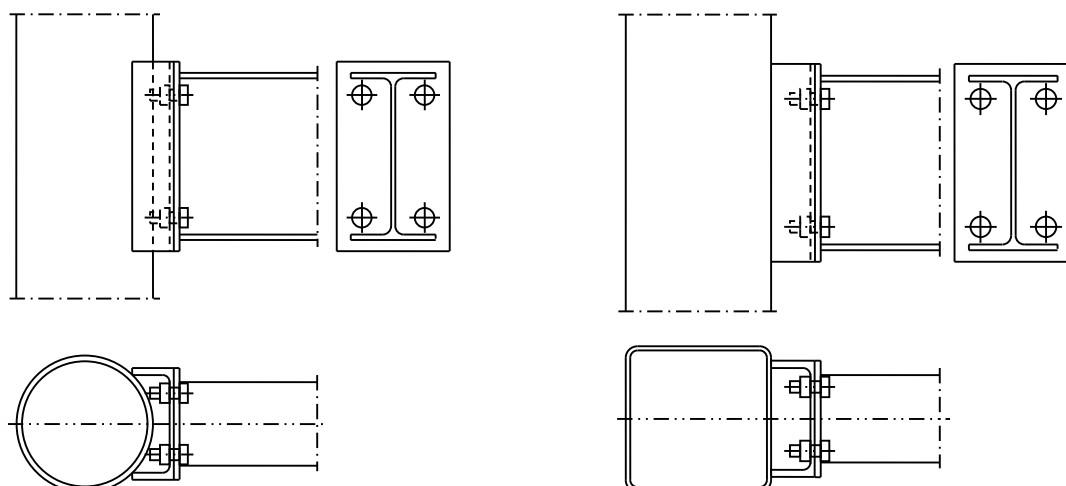
Fig.1 Beam to column CHS and SHS connections schemas

In this paper are presented the results of the numerical study on the behaviour of the columns, made with finite element models and on the basis of a parametric variation of the most significant characteristics of tubular profiles. The parameters considered in parametric variation are the thicknesses, the widths and the diameter of the tubular columns, and the filling with concrete. The numerical models were developed in the finite element program LUSAS, being used solid finite elements, spring elements and shell elements. The results of the study allow to relate the parameters considered with the structural behavior of the connection, in particular in the strength, the stiffness and the rotation capacity.

## PARAMETRIC STUDY

The various settings of the dimensions of the tubular columns selected for the numerical models correspond to parametric variation of the most influential factors in their structural behaviour:

- Thickness;
- Width (SHS columns);
- Diameter (CHS columns);
- Filling or not with concrete.

In all the numerical models, the U-profile, the beam IPE300 and the end plate always keep the same features and dimensions as listed in Table 1. The numerical model with these characteristics and dimensions, however without regard to tubular column, already has been tested experimentally, existing also an associated numeric model, which is designated as A-13, and that is mentioned in this paper to assess the behaviour of the models, whereas in this model that the tubular columns are rigid and non-deformable.

The dimensions and characteristics of tubular profiles of the numerical models, variable parameters of the study, are indicated in Table 2.

Table 1 Dimensions of the elements of the connection with fixed parameters

| N | $p_2$ [mm] | $b_c$ [mm] | $h_c$ [mm] | $t_{wc}$ [mm] | $t_{fc}$ [mm] | Model (Reference) | End Plate | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | $b_P$ [mm] | e [mm] |
| **3** | 85 | 90 | 185 | 15 | 15 | **A-13** | 155 | 35 |

Table 2 Dimensions of the numerical models

| M | Profile | $w_c$ [mm] | $d_c$ [mm] | $t_c$ [mm] | Concrete | Model (Reference) |
|---|---------|-----------|-----------|-----------|----------|-------------------|
| 1 | | | | 8 | | S-001 |
| 2 | | 200 | | 10 | | S-002 |
| 3 | | | | 12,5 | | S-003 |
| 4 | | | | 15 | | S-004 |
| 5 | | | | 8 | | S-021 |
| 6 | | 220 | | 10 | No | S-022 |
| 7 | | | | 12,5 | | S-023 |
| 8 | | | | 15 | | S-024 |
| 9 | | | | 8 | | S-041 |
| 10 | SHS | | | 10 | | S-042 |
| 11 | | | | 12,5 | | S-043 |
| 12 | | 240 | | 15 | | S-044 |
| 13 | | | | 8 | | S-141 |
| 14 | | | | 10 | Yes | S-142 |
| 15 | | | | 12,5 | | S-143 |
| 16 | | | | 15 | | S-144 |
| 17 | | | | 8 | | S-061 |
| 18 | | 260 | | 10 | | S-062 |
| 19 | | | | 12,5 | No | S-063 |
| 20 | | | | 15 | | S-064 |
| 21 | | | | 8 | | C-021 |
| 22 | | | 219,1 | 12,5 | | C-023 |
| 23 | | | | 8 | Yes | C-121 |
| 24 | CHS | | | 12,5 | | C-123 |
| 25 | | | | 8 | No | C-041 |
| 26 | | | 244,5 | 12,5 | | C-043 |
| 27 | | | | 8 | Yes | C-141 |
| 28 | | | | 12,5 | | C-143 |

## FINITE ELEMENTS MODELING

The numerical models, represented in Fig. 2a and Fig. 2b, characterize numerically the real prototypes to be used in experimental tests.
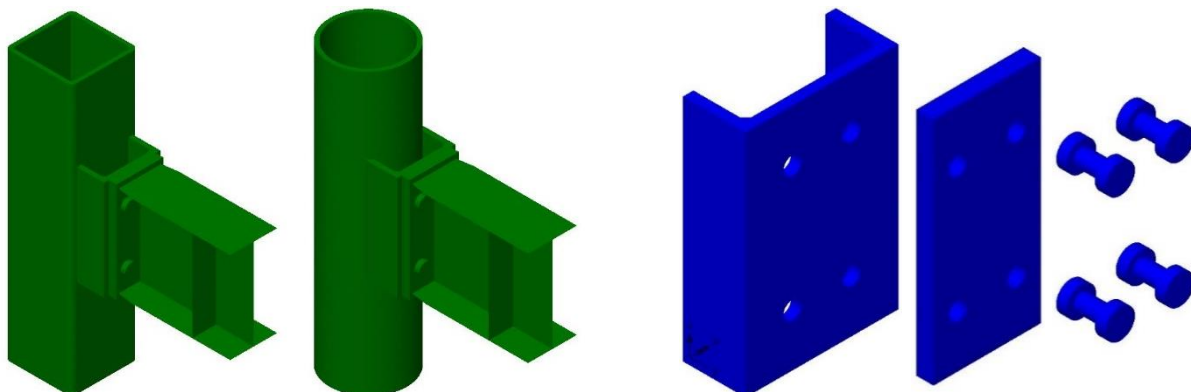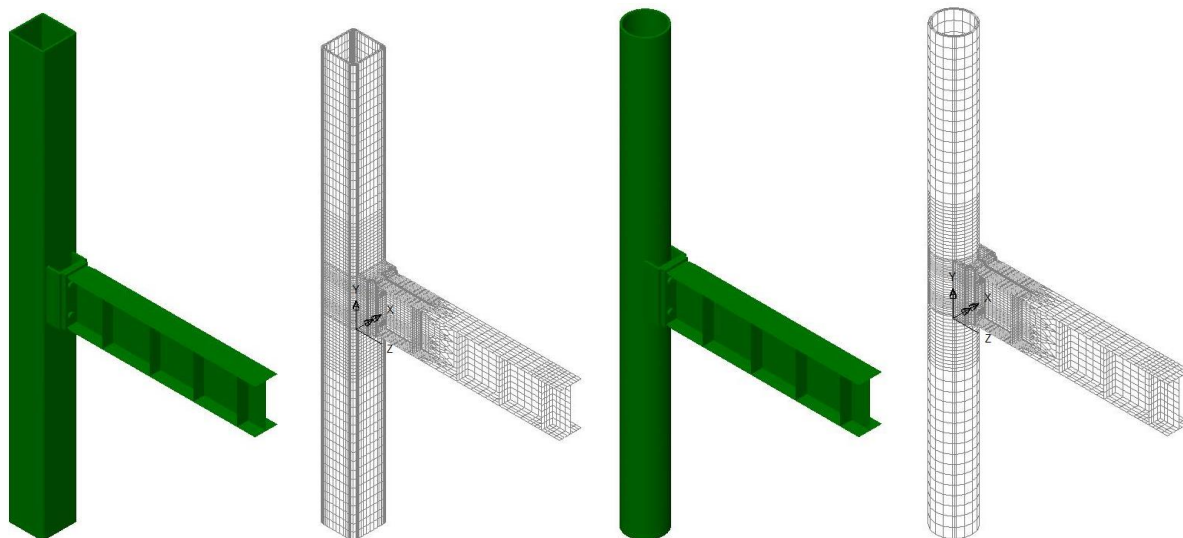


Fig.2a Details of the models in the connection zone

Fig.2b Complete finite element models

They are constituted by the tubular column, square (SHS) or circular (CHS) cross-section, by the U-profile of welded section (two flange plates welded to the web plate with 4 holes) welded to the column, and the I beam, in which the load is applied, with end plate that connects to the U-profile with 4 bolts.

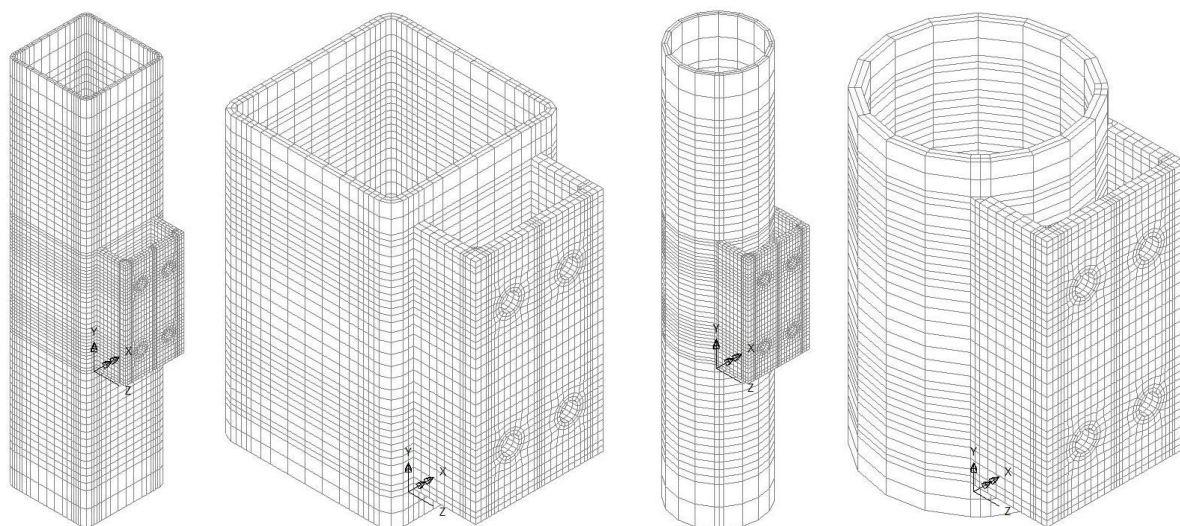The numerical models have been modeled assuming a nonlinear analysis with automatic increment of the load. The Fig. 3 presents some details of the finite elements mesh.

Fig.3 Finite elements meshes

As can be seen in Fig. 3 the finite element mesh has a tighter discretization in the profile, in the bolts, in the end plate and in the beam and column area closest to the connection, happening that in remote areas are allowed to have a more open mesh, getting with this detail lighten the models in time benefit of the numerical calculus.

For the tubular columns, the concrete, the U profile, the beam end plate and bolts elements, solid finite elements HX8M and PN6 are used. QTS4 thick shell elements are used for the beam in order to reduce the weight of the program calculation. The modelling of the contact surfaces,

between the end plate, the channel web and the bolts, is performed with spring elements JNT4, with high stiffness in compression and very low in tension. The formulation of these finite element allows to take into account the various efforts including those of membrane. The Fig. 4 outlines the finite element formulation used.
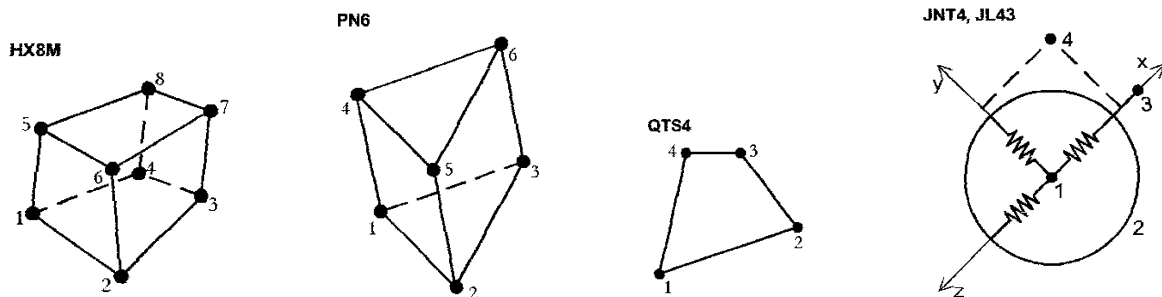


Fig.4 Finite elements

In order to define the optimum mesh discretization a mesh convergence analysis was performed. Were analyzed four meshes that divide each finite element solid in 2x2x2, 3x3x3 e 4x4x4, on the basis of the initial mesh models not divided, which is called 1x1x1, shown in the detail of the tubular column in the connection zone on the Fig. 5. The results, present in Fig. 6, show that mesh 2x2x2 is accurate enough.
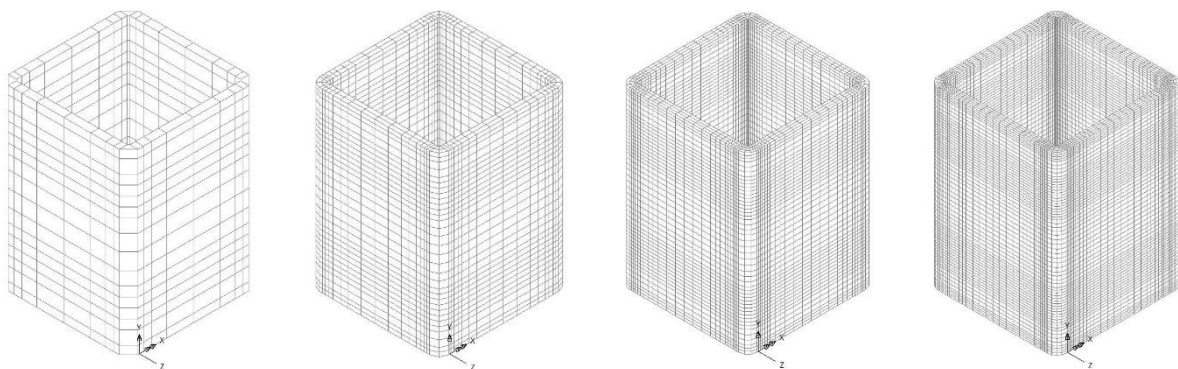


Fig.5 Finite element meshes used in convergence study (1x1x1, 2x2x2, 3x3x3 e 4x4x4)

In the modelling were considered steels with different characteristics of stress-strain behaviour, in demand to calibrate the numerical models with the experimental results. The material is described by means of multilinear curves obtained from uniaxial tension tests on coupons taken from the plates used in experimental tests. True strain and true stress values were calculated from the engineering values obtained from the tests on the material, making the curves depicted in Fig. 7.

So that the plastic deformations begin in the U-profile, all the others elements of the numerical models (column, beam, end plate and bolts) were considered with superior resistance. The tubular columns are modeled by considering a steel S355, and if they are completed, a C20/25 concrete resistance class is considered. The beam IPE300 profile, with great bending stiffness compared to the U profile, and the end plate with 25 mm of thickness, also have the characteristics of a steel S355. The bolts M24 are of resistance class 10.9.
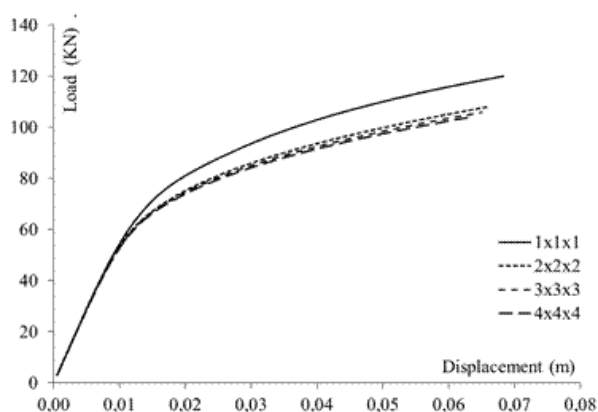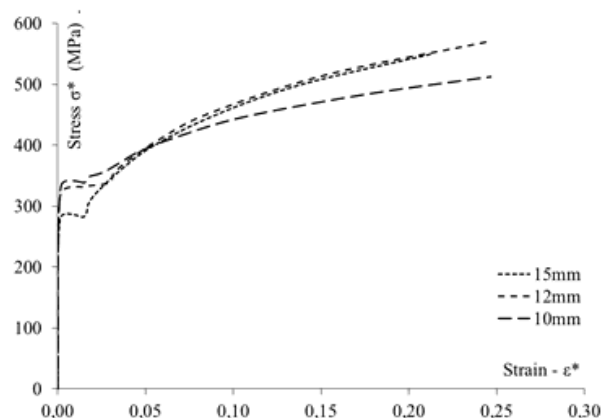
Fig.6 Load-displacement diagrams          Fig.7 Steel stress-strain diagrams

## RESULTS

Results in terms of deformation and Von Mises tensions are present in Fig. 8.
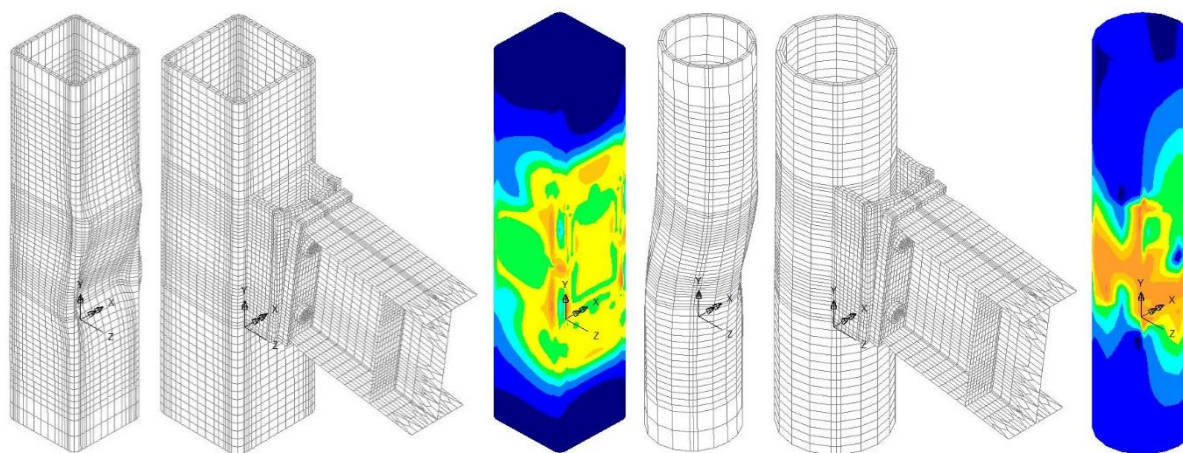


Fig.8 Deformations and Von Mises tensions for the last increment of load

We can verify that the deformation of tubular profiles is caused by the tension of the flanges of the U-profile at the upper zone and by compression at the bottom zone. In both cases we can observe significant plastic deformation, noting also that the deformation in tubular profiles are less than those caused in the soul of U profile. On the deformation of tubular profiles it is easy identify the deformations that lead to the failures modes identified in the part 1-8 of the Eurocode 3. The chord face mode and chord side wall mode are evident in SHS sections, and in CHS sections the chord shear mode is noted.

With regard to tensions is notorious that are higher in the peripheral zones of the contact with U-profile, decreasing for the contrasting faces to this profile. In the areas away from the connection to the beam the tensions are minimal.

The results can also be represented in terms of moment-rotation curves, which is shown in Fig. 9 to Fig. 12. The moment rotation curves have two different regions: The initial linear region (see the diagrams on the right side of Fig. 9) and the post-yielding region. In the linear region the behaviour of the connections may be expressed in terms of rotation (rot), initial stiffness (Sj,ini) and maximum elastic bending moment (Mj). Both values depend on the material elastic

properties and on the elastic behaviour of the joint, closely dependent on its geometry. The post-yielding region is characterized by it's stiffness ($S_{j,pl}$) and maximum rotation (not reached in the tests reported). This post yielding stiffness depends on the material hardening property and the stiffness that is mobilized on the loaded plates.

In Fig. 9 are presented the moment-rotation curves, for the thickness variation in the models SHS-200, SHS-240, CHS-219,1 and CHS-244,5, referenced in Table 2. The behaviour of the joint is changed with the decrease of the tubular columns thickness, particularly in models with the largest width of the columns, where the load is applied as away from the edges, the same as succeeding with circular cross-section columns, where there is a decrease in the resistance of the connection. For the upper thicknesses, the results are identical to the model A-13.
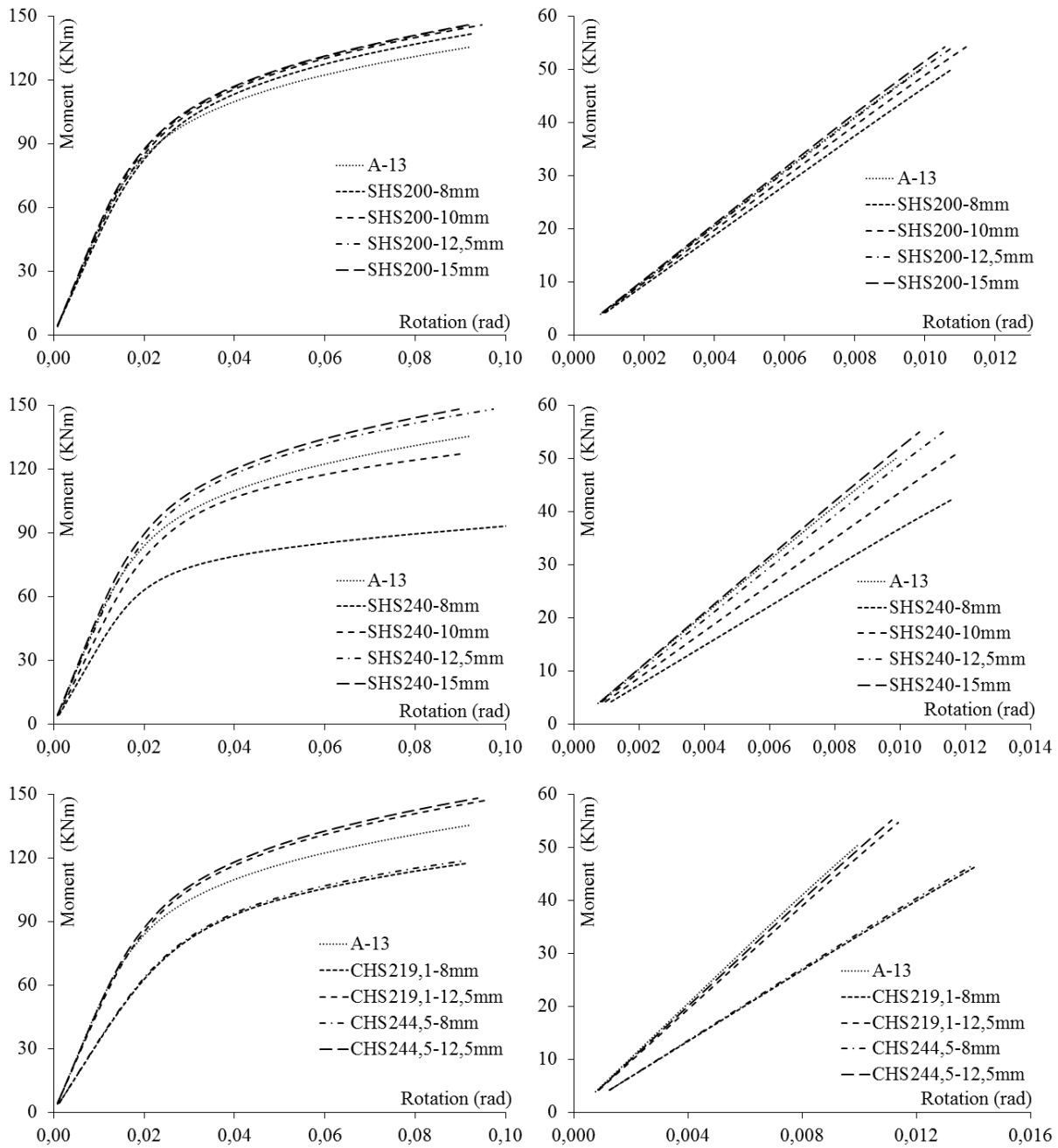


Fig.9 Moment-rotation diagrams for the variation of the parameter thickness

The Fig. 10 shows the moment-rotation curves, for the width variation in the models SHS-200, SHS-220, SHS-240, SHS-260, CHS219,1 and CHS244,5, indicated in Table 2. It turns out that the variation of the weight of the SHS tubular profiles is significant in the behavior of the connection when we consider inferior values of the thickness, not registering significant variations to great values of the thickness. On the other hand, for standard diameters considered in the study, the variation of the diameter of CHS profiles does not change the behavior of the connection.
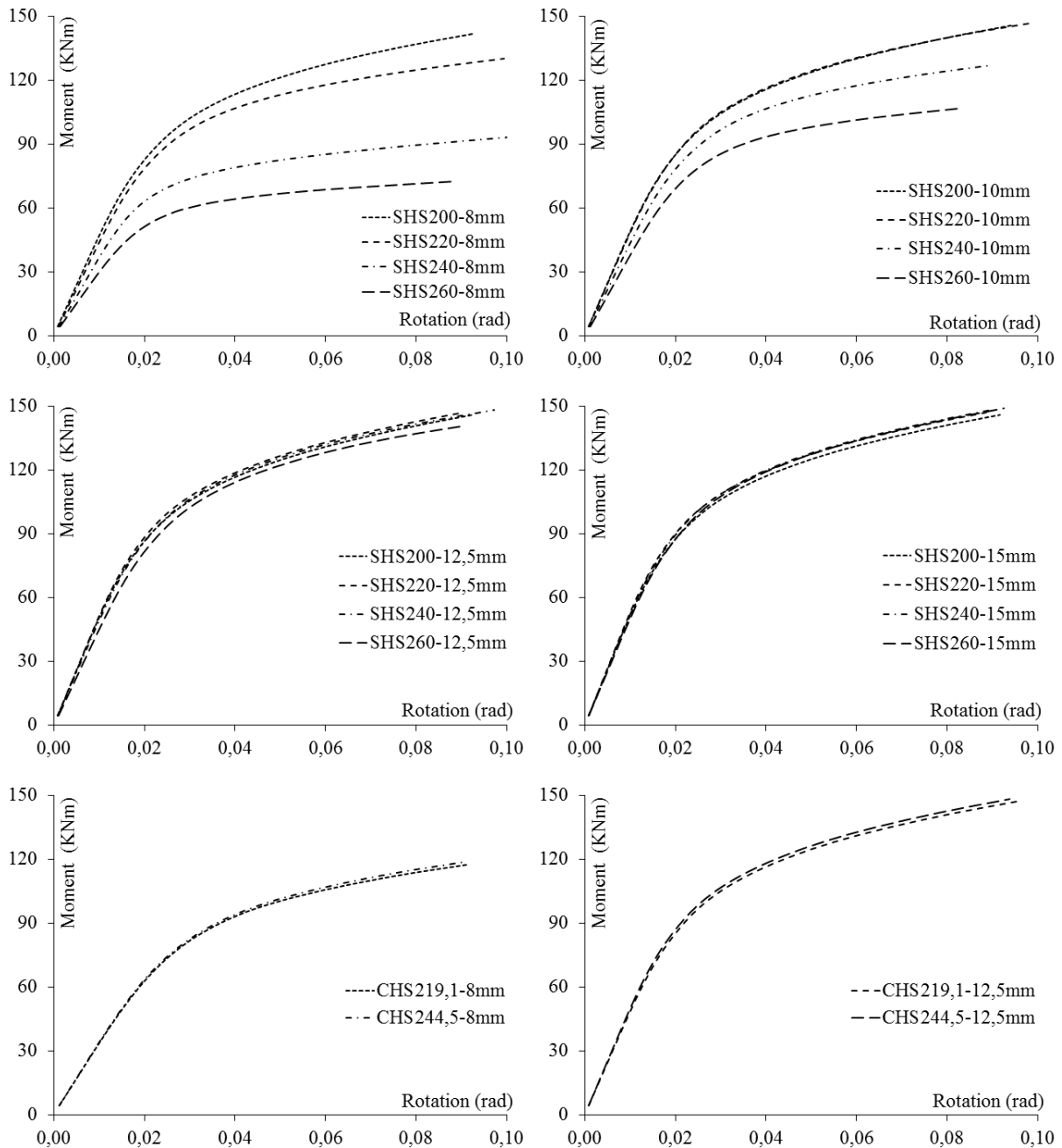
Fig.10 Moment-rotation diagrams for the variation of the weight in the profiles SHS and CHS

Are presented in Fig. 11 the moment-rotation curves of the models SHS220, SHS240, CHS219,1 and CHS 244,5, referenced in Table 2, concerning the comparison of the sections, square or circular. From the analysis to the moment-rotation diagrams of the Fig. 11 it is confirmed that for 8 mm of thickness and to lowers widths and diameters, the SHS profiles has more resistance than the CHS profiles, because the edges are closest to the contact with the U-profile flanges, but for largest diameters and widths the condition is opposite. When the thickness of the tubular profiles is 12.5 mm, the results of the width variation analysis are identical.
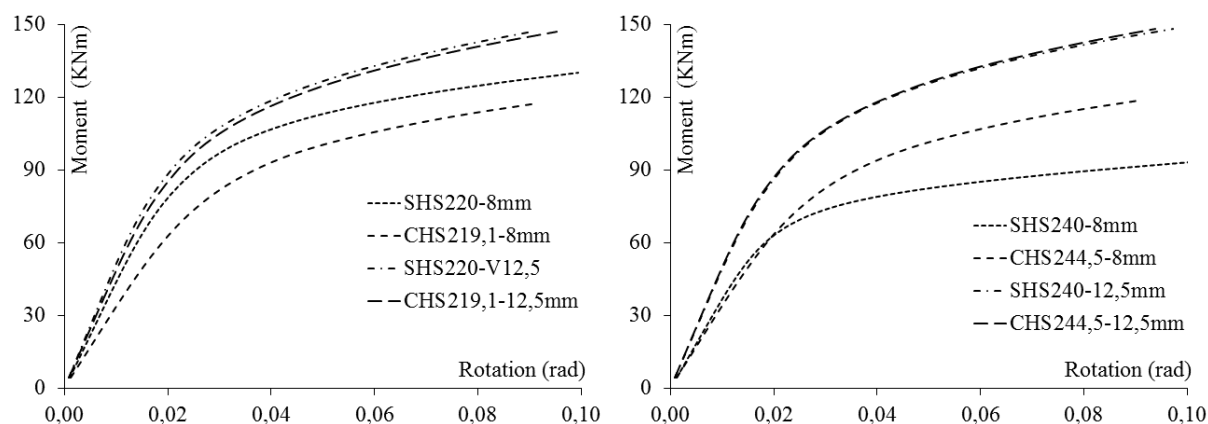


Fig.11 Moment-rotation diagrams for comparison square section with circular sections

In Fig. 12 are presented the moment-rotation curves concerning the study of the behavior of the connection when the tubular columns are filled with concrete. In both cases, SHS and CHS, the filling with concrete expressively increases the strength and the stiffness.
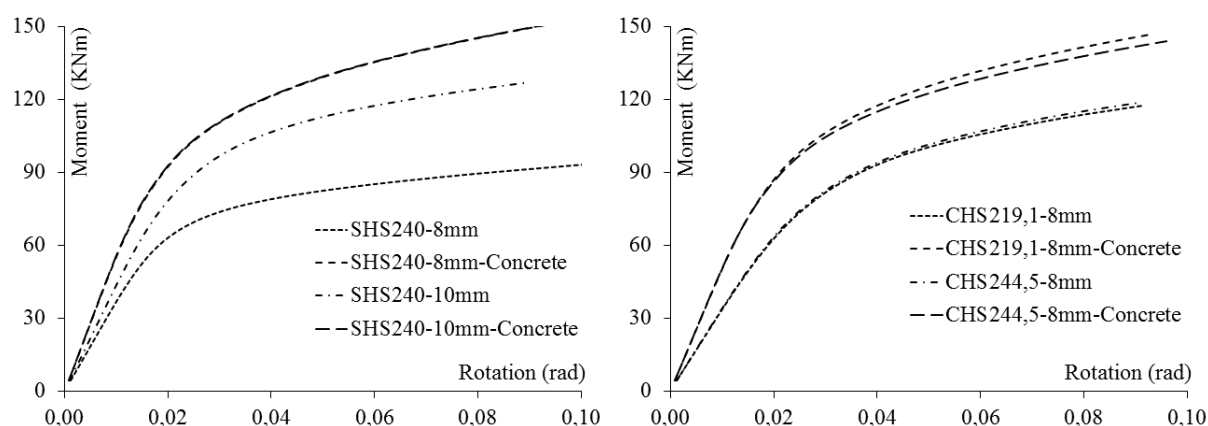


Fig.12 Comparative moment-rotation diagrams of the use of concrete

In Table 3 are indicated the values obtained for the bending moment, rotation and stiffness. The results of the numerical models listed in Table 3, indicate that the parameters with more influence on the global behavior of the joint are the thickness and the width of the square section columns. When the thickness decreases the values of the moment and of the rigidities decline. The variation of the width of SHS columns involves significant variations of the stiffness and

elastic moment, when it keeps constant the thickness. Specifically, in the CHS columns, the diameter is a parameter that does not cause modifications in terms of strength and stiffness. The models with higher resistance are the models that have superior values of the thickness, and the models with higher stiffness are those that are filled with concrete, being most evident in SHS.

Table 3 Values of the elastic bending moment, rotation and stiffness of the models

| Mod. | $M_j$ [KNm] | Rot. [rad] | $S_{j,ini}$ [KNm] | $S_{j,pl}$ [KNm] | Mod. | $M_j$ [KNm] | Rot. [rad] | $S_{j,ini}$ [KNm] | $S_{j,pl}$ [KNm] |
|---|---|---|---|---|---|---|---|---|---|
| S-001 | 50,0 | 0,0107 | 4651,4 | 397,8 | S-143 | 50,8 | 0,0089 | 5679,3 | 414,6 |
| S-002 | 54,2 | 0,0112 | 4851,9 | 411,1 | S-144 | 55,0 | 0,0097 | 5687,2 | 415,5 |
| S-003 | 54,2 | 0,0108 | 5027,4 | 421,5 | S-061 | 29,8 | 0,0100 | 2969,9 | 129,5 |
| S-004 | 54,2 | 0,0106 | 5134,6 | 435,9 | S-062 | 46,9 | 0,0124 | 3767,7 | 217,4 |
| S-021 | 46,2 | 0,0105 | 4392,7 | 273,8 | S-063 | 55,4 | 0,0124 | 4482,8 | 351,7 |
| S-022 | 50,4 | 0,0105 | 4811,7 | 372,9 | S-064 | 55,4 | 0,0112 | 4963,1 | 432,9 |
| S-023 | 50,4 | 0,0098 | 5136,0 | 420,7 | C-021 | 46,2 | 0,0140 | 3291,6 | 334,4 |
| S-024 | 54,6 | 0,0103 | 5297,7 | 403,8 | C-023 | 54,6 | 0,0114 | 4800,3 | 409,1 |
| S-041 | 42,3 | 0,0116 | 3640,2 | 180,0 | C-121 | 46,2 | 0,0092 | 5038,7 | 415,7 |
| S-042 | 50,8 | 0,0117 | 4329,9 | 288,5 | C-123 | 50,4 | 0,0098 | 5148,1 | 419,3 |
| S-043 | 55,0 | 0,0113 | 4859,2 | 381,0 | C-041 | 46,6 | 0,0140 | 3330,5 | 327,6 |
| S-044 | 55,0 | 0,0106 | 5188,3 | 413,3 | C-043 | 55,1 | 0,0111 | 4949,9 | 418,3 |
| S-141 | 46,5 | 0,0083 | 5596,1 | 412,0 | C-141 | 50,8 | 0,0101 | 5052,0 | 386,1 |
| S-142 | 50,8 | 0,0090 | 5627,6 | 413,3 | C-143 | 55,1 | 0,0108 | 5108,1 | 377,7 |

## CONCLUSION

The geometry of the tubular column, as well as the transmission area of the load by the U-profile flanges, are important parameters on the behaviour of the joint to I beams, because there are significant changes in this behaviour to the parametric variation considerer in the study.

The resistance and the stiffness of the connection are lower when the thickness is less.

Larger relationships between sections and widths of the loaded area, imply lowers resistances and stiffness. This is justified by the fact that for low relations the compression and the traction forces transmitted by the flanges to be supported by the sides of the columns, giving greater rigidity to the connection, while in the opposite situation, the strength and the stiffness are conditioned by the bending of the face welded to U-profile.

The variation of diameter of CHS columns does not affect the behavior of the connection.

The concrete filling of the columns increases the resistance and the stiffness of the joint.

On models with greater thickness, with smaller width and when are filled with concrete, the behavior of the column can be considered rigid, because the results are similar to those obtained for the model A-13.

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