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Numerical modelling of CFST column to I beam end plate joints

Said Hicham Boukhalkhal¹, A Nacer T Ihaddoudène¹, Luis F Costa-Neves², Wafa Madi¹, Pedro CG Vellasco ³and Luciano Lima³

Correspondence

Dr. Said Hicham BOUKHALKHAL Built Environment Research Laboratory – Faculty of Civil Engineering. University of Sciences & Technology Houari Boumediene of Algiers (USTHB).

BP.32 El-Alia, Bab Ezzouar 16111 Algiers, Algeria

Email: <u>saidhichamboukhalkhal@yahoo.fr</u> and <u>sboukhalkhal@usthb.dz</u>

Affiliations

¹ Built Environment Research Laboratory -Faculty of Civil Engineering. University of Sciences & Technology Houari Boumediene of Algiers (USTHB) Algiers, Algeria.
² INESCC, Civil Engineering Department, Uni-

versity of Coimbra, Portugal. ³ Structural Engineering Department, State

University of Rio de Janeiro, UERJ, Brazil.

Abstract

The use of concrete-filled tubular structural elements around the world has been boosted in recent years. The most common geometries for these sections are circular (CHS), Square (SHS) and rectangular (RHS). They have been extensively used in multi-story buildings due to their aesthetical features and structural advantages, due to their favorable resistance to weight ratio. One of the limitations for using these geometries is the inaccessibility to the interior of the section, thus arising difficulties and added costs in materializing the connections. This paper presents a numerical study dealing with tubular columns to I beam end plate joints using welded studs as the connecting elements and aiming to evaluate their structural performance. Following a series of fullscale monotonic tests performed at the Civil Engineering Department of the University of Coimbra, Portugal, with the abovementioned geometry, several numerical models using 3D nonlinear elements have been developed to investigate the mechanical response of these joints.

The main variables were the column and beam geometries, the thickness of the hollow sections, and the presence or absence of infilling concrete. The numerical results have been compared to the tests performed under positive and negative bending moments, to validate the finite element model. The numerical model was proven to lead to accurate predictions of the initial stiffness and joint resistance. The model validation was followed by a parametric study with the abovementioned variables to investigate the influence of the governing components and behavioral features. The column face loaded out of its plane proved to be the governing component, with its width-to-thickness ratio having a significant role in determining both elastic stiffness and resistance, together with the endplate dimensions and the number of bolt rows in tension.

Keywords

Composite structures, Tubular columns, Numerical analysis, end-plate joints.

1 Introduction

Tubular structures present excellent properties which make them favourable for several applications mainly in industrial buildings and offshore structures, due to their larger durability and high compressive strength compared to those of open sections steel structures [1-4]. Due to their symmetrical shape, RHS joints sections are particularly advantageous, being more resistant to axial forces [5, 6]. Despite this fact, on site, the attached I-beam with a tubular column connection presents a particular difficulty [5].

Among several systems commonly used, the welded studs are probably the easiest and simplest way of overcoming this issue. On the other hand, the welded studs have the disadvantage of being more difficult to transport and to erect. However, the advantage of solving the problem of connecting without welds to sections whose interior is not accessible for fastening [7-9] make its use a natural solution. In addition, the use of alternative fastening techniques that do not require welding or interior fastening had a limited performance. As the studs are welded to the columns in the site, in order to be able to position the beam, one of the difficulties encountered during assembly is precisely how to find the right distance from the columns which seems not very practical. The joints responses considering this technique is one of the objectives of the present work.

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Several studies on the performance of I-beam-to-RHS column joints with or without concrete infill using different attachment techniques have been developed [10].

Fastening by welded studs was used by Maquoi et al. [9] where tests were carried out on seven types of joints (rigid, semi-rigid and flexible) subjected to monotonic loading.

Similarly, Vandegans [7] performed eight tests on different configurations. This study indicated that the dowelling technique allows very simple connections between a double tee beam and a concrete-filled steel tubular column (CFST).

Korol et al. [12] and Ghobarah et al. [13] conducted experiments and analytical studies on the behaviour and the failure modes of I-beam joints with a tubular column attached through extended end-plates using high-strength (HSBB) bolts. The results showed that, for specimens with unfilled RHS columns, the performances of the joints were classified as rigid according to the EC3 [11].

Costa-Neves [14] and Costa-Neves et al. [15] studied the behaviour of end plate I-beams to RHS columns with welded studs to assess the accuracy of existing methods for predicting their strength. A mechanical model has thus been proposed to determine the rigidity of the joints when considiring the contribution of the loaded face of RHS column.

Wang [16] evaluated both the elastic and the ultimate limits of an assembly comprising a bolted end-plate of a blind bolt type to a square tubular column in the tensioned zone of the joint. A finite element model which allows visualising the details of the components of hollo-bolt was developed to better understand the local response of the joints. As a result, the strength of the joint was found to be sensitive to the thickness of the tube wall and to the gauge width of the bolt relative to the outer width of the SHS column.

The current study aims at developing a 3D numerical model to access the mechanical behaviour of an I-beam-to-CFST column joint with end-plate. The moment-rotation curves predicted by the finite element models have been compared to experimental envelopes [14] under positive (M +) and negative (M-) bending moments. A parametric study was also carried out to highlight the main geometric and mechanical parameters which might influence the behaviour of this type of joints.

2 Experimental tests

The experimental study [14] carried out on test specimens subjected to a monotonic loading comprising three different configurations I-beam-to- CFST columns joints is shown in Figs. 1 and 2. The columns are of type SHS, with variable thicknesses (6 and 16 mm) while the beams are of type IPE (240 or 300).







Config. (c)

Config. (a) Config. (b) Figure 1 RHS column-to-I-beam joints [17]



Figure 2 Configuration of the tested frame [14]

The description of the experimental program for the six tests is reported in Table 1.

Table 1 Description of the experimental program [1	17]
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	Test	Steel	Column	Beam	Loading
	E13				Monotonic M+
Config.		S 355	RHS 200*6	IPE 240	
(4)	E14				Monotonic M-
Config.	E16	\$ 255	BHS 200*6	IDE 240	Monotonic M+
(b)	E17	3 333	KH3 200°0	II E 240	Monotonic M-
	E 10	9.955	D110 200#4	IDE 200	
Config.	E19	5 355	KHS 300*6	IPE 300	Monotonic M+
(c)	E21	S 355	RHS 300*16	IPE 300	Monotonic M+

3 Numerical modelling

Based on the experimental results conducted in Coimbra on I-beamto-CFST column joints with both extended and flush end-plates, 3D non-linear finite element models have been developed to investigate the response of this type of joints.



Joint with CFST column



Figure 3 Presentation of the numerical model

3.1 Presentation of the numerical model

A detailed study was conducted to derive the parameters which could influence the behaviour of these types of joint; the finite element software ANSYS [18] was used.

The SOLID186 element was selected to represent the concrete core, IPE beam, end-plate and the RHS column, while the SOLID187 element was used to model both the studs and the nuts used in the current work (Figure 3).

A surface-to-surface type model was used to ensure an appropriate force transfer between the interacting surfaces. Two elements were used as contact surface: CONTA173 and CONTA174, while TARGE170 was used as the target surface.

For the joint components (studs, RHS column, end-plate, IPE- beam and concrete infill), the mesh was created selecting the number of division lines. The suitable number of elements was determined using alternative mesh analysis.

The boundary conditions are supposed to be fixed at the two ends (upper and lower) of the RHS column to prevent displacements (Ux = Uy = Uz = 0).

3.2 Type of analysis

The area of the loaded face of the SHS column where the studs would reach the deformation at failure is particularly targeted. Nonlinear static analysis was performed were the application of an increasing moment till failure is schematically shown in Figure 4.





heta: Joint rotation

 Δ : RHS face displacement

h: Distance between shear studs.

Figure 4 Monotonic loading

4 Validation of the numerical models

The results obtained from the numerical model analysis were compared to the experiments mainly in the terms of moment-rotation curves. Figure 5 shows the curves of the three configurations of the numerical models EN13, EN16 and EN21 and the experimental curves of tests E13, E16 and E21 [14].









Figure 5 Comparison of M-0 curves

It was possible to observe that the experimental moment-rotation curves are almost in the \pm 10% range compared to the numerical models curves with minimal margin of error, and consequently, the developed numerical models proved to be suitable for use.

5 Global behaviour of joints

The development of finite element models makes it possible to measure and evaluate the behaviour of each member of the joint in a simple way. Thus, the models were divided into levels to assess the distribution of stresses and deformations along the joint.

5.1 Column face behaviour

Figure 6 highlights the distribution of the horizontal stresses at the column face of the EN16 and the EN17 models for each level of applied moment.



Figure 6 Average horizontal stresses at the column's face

The numerical model EN16 stress distribution shows that at the level of the second row of studs, the column's face attains the plastic phase for a moment equal to 25 kN.m, for the first row this moment is 32 kN.m. In addition, it can be noted that the major parts of the column's loaded face have yielded before the load reaches 40 kN.m. For EN17, it could be observed that the face of the CFST column yields for a moment equals to 22 kN.m indicated 31% reduction in comparison with the first row of studs of the model EN16. Consequently, the joint (Config. B) presents a higher resistance for EN16 (M +) and EN17 (M-).

In terms of stress concentration and yield zones, figures 7-a and 7-b depict the difference between the two configurations.

The study of the state of deformation allows the determination of the failure point of the studied element; which is supposed to be reached when the zone of interaction between the loaded face of the RHS column and the studs has reached the deformation at failure.



Figure 7 Column face: stress distribution (Von-Mises)

The comparison between the two numerical models EN13 and EN14 shows that the face of the CFST column will be damaged at moments equal to 42 kN.m and 33 kN.m respectively, with a decrease of 21.4% as shown in figure 8-a. This was due to the fact that the position and symmetry of the stud rows as well as the loading direction play a significant role in the behaviour of the joint, thus directly influencing the joint failure load.

For the numerical model EN16, the moment corresponding to the failure of the interaction zone is equal to 57 kN.m while for EN17 is 40 kN.m; this time the reduction is around 30%. While the addition of a new row of studs increases the resistance in the positive direction of loading which causes the resistance to drop in the opposite direction of loading. (Figure 8-b)

Figure 8-c shows the deformations' envelope in the CFST column face for the two models EN19 and EN21. The column face of the numerical model EN19 attains the failure phase when the moment reaches a value of 50 kN.m. On the other hand, failure in the EN21 model occurs by the studs failure due to its considerable thickness.

It is worth noting that the position of the failure point on the CFST column face of the model EN19 is located at the interaction area between the tensioned studs and the RHS column face, a similar position to that of the model EN16.

5.2 The behaviour of the plate and the I-beam

The end-plate, being a much more rigid and resistant component compared to the column face, had negligible deformations and remains in the elastic range for positive moment tests (EN13, EN16, EN19 and EN21), as illustrated in Figure 9.

For the negative moments, it could be noted that the plate yields at the extended part (EN14 and EN17), which is a direct consequence of the high level of the moment reached due to the membrane action at the column face.

For all the numerical models studied, the flanges of the beam did not attain the plastic phase, as shown in Figure 9. This reflects the fact that, in the case (EN21), the bending moment reaches only 60% of the beam plastic moment.



Figure 8 Average horizontal stresses in the column's face

6 Conclusions

A detailed study of the behaviour of each joint component under monotonic loading was carried out. The numerical results obtained allowed the following conclusions to be drawn:

The main component in the models studied was the column loaded face, which is the first element that attains the plastic phase. If the premature failure of the stud could be ignored, this component would be the most relevant joint component.

The column face stress distribution was essentially unidirectional, in the horizontal direction, while the observed variation of the stress intensity along the height of the face justifies the hypothesis of an elastic force distribution between the multiple columns rows.

The welded studs failure was caused by the failure of welds on the studs' surface, while the measured internal stresses of the studs were still in the elastic zone in the case where the RHS column thickness was quite significant.

The influence of the load direction over the maximum strength and the initial stiffness was not evident, while the end-plate type had a significant influence on the behaviour. In comparison to the joint with a flush end-plate, the extended end-plate joints could significantly improve the bearing capacity and the initial rigidity of the joint.



Extended End plate



Figure 9 End-plate and I-beam: Stress distribution (Von-Mises)

References

- FADDEN, M.; MCCORMICK, J. (2014) Finite element model of the cyclic bending behavior of hollow structural sections, JOURNAL OF CONSTRUCTIONAL STEEL RESEARCH 94 64–75.
- [2] MANCO, T.; RIGUEIRO MC.; MARTINS, JP.; DA SILVA, LS. (2016) Comparative assessment of the design of tubular elements according to offshore design standards and Eurocode 3. STEEL CONSTRUCTION 9, NO. 4, PP 266-278.
- [3] NIZER, L.; VELLASCO, DE LIMA/P. DA S.; DE ANDRADE, E.; DA S. GOULART, A.; DA SILVA, L.; COSTA-NEVES LF. (2016) Experimental and numerical assessment of RHS T-joints. STEEL CONSTRUCTION 9, NO. 4, PP 315-322.
- [4] WARDENIER, J.; DUTTA, D.; YEOMANS, N.; PACKER, J. A.; BUCAK, O. (1995) Design Guide for Structural Hollow Sections in Mechanical Applications. CIDECT (ED.), T-V RHEINLAND, COLOGNE.
- [5] BOUKHALKHAL, SH.; IHADDOUDÈNE, A.; DA COSTA NEVES, L.; MADI, W. (2019) Dynamic behavior of concrete filled steel tubular columns. INTERNATIONAL JOURNAL OF STRUCTURAL INTEGRITY, VOL. 10 NO. 2, PP. 244-264.
- [6] BOUKHALKHAL, SH.; IHADDOUDÈNE, A.; DA COSTA NEVES, L. (2019) Effect of concrete filling on the bending behaviour of end plate I-beams to tubular columns connections. THE FIRST INTERNATIONAL CONFERENCE ON MATERIALS, ENVIRONMENT, MECHANICAL AND INDUSTRIAL SYSTEMS ICMEMIS'19, DJELFA (ALGERIA).
- [7] VANDEGANS D. (1995) Liaison entre poutres métalliques et colonnes en profils creux remplis de béton, basée sur la technique du goujonnage (goujons filetés) - CRIF MT 193, BRUXELLES.
- [8] JASPART, J. P.; AND WEYNAND, K. (2001) Extension of the component method to joints in tubular construction. PROC., 9TH INT.

SYMP. AND EUROCONFERENCE ON TUBULAR STRUCTURES, TUBULAR STRUCTURES IX, R. PUTHLI AND S. HERION, EDS., A. A. BALKEMA, ROTTERDAM, NETHERLANDS, 517–523.

- [9] MAQUOI, R.; NAVEAU, X.; RONDAL, J. (1984) Beam-column welded stud connections. J. CONSTR. STEEL RES., 4(1), 3–26.
- [10] WANG, JF.; WANG, H. (2018) Cyclic Experimental Behavior of CFST Column to Steel Beam Frames with Blind Bolted Connections. STRUCTURES. <u>HTTPS://DOI.ORG/10.1007/s13296-018-0089-8</u>
- [11] EC3 PART 1-8 (2005). Eurocode 3. Design of steel structures, design of joints. European Committee for Standardization (CEN). BRUSSELS, BELGIUM.
- [12] KOROL, R.M.; GHOBARAH, A.; MOURAD, S. (1993) Blind bolting Wshape beams to HSS columns. J. STRUCT. ENG. 119 (12), PP 3463– 3481.
- [13] GHOBARAH, A.; MOURAD, S.; KOROL, R. M. (1996) Moment-rotation relationship of blind bolted connections for HSS columns. J. CONSTR.STEEL RES. VOL.40 NO.1, PP.63-91,.

- [14] COSTA-NEVES, L.F. (2004). Monotonic and cyclic behavior of minor-axis and tubular joints in steel and steel and concrete composite structures. PH.D. THESIS. UNIVERSITY OF COIMBRA. PORTUGAL. (IN PORTUGUESE).
- [15] COSTA-NEVES L.F, SILVA L.A.P.S, VELLASCO P.C.G.S. (2003) Experimental Behaviour of End plate I-Beam to Concrete-Filled Rectangular Hollow Section Column Joints under Cyclic Loading PROC. OF THE FOURTH INTERNATIONAL CONF. ON THE BEHAVIOUR OF STEEL STRUCTURES IN SEISMIC AREAS (STESSA '03), NAPLES ITALY 9-12 JUNE 2003, PP 277-283.
- [16] WANG Z.Y, WANG Q.Y. (2016). Yield and ultimate strengths determination of a blind bolted endplate connection to square hollow section column. ENGINEERING STRUCTURES 111 345–369.
- [17] Costa-Neves L.F., Silva L.A.P.S, Vellasco P.C.G.S. (2002): Experimental Behaviour of End plate I-Beam to Concrete-Filled Rectangular Hollow Section Column Joint - Proc. of the Third International Conf. on Advances in Steel Structures (ICASS '02), Hong Kong, 9-11 December 2002, pp 253-260.
- [18] ANSYS Release 15.0 (2015). Finite element program system, ANSYS Inc.