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Shear transfer mechanism in connections involving concrete filled steel columns under shear forces

Silvana De Nardin *1 and Ana Lucia H.C. El Debs [2a]

[1] *Department of Structural Engineering, University of Sao Paulo, Sao Carlos, Avenida Trabalhador Sãocarlense 400, Centro, São Carlos-SP - CEP: 13.566-590, Brazil*
[2] *Department of Civil Engineering, Federal University of Sao Carlos, Rod. Washington Luís km 235 - SP-310 - São Carlos, CEP 13565-905, Brazil*

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Abstract. This paper reports the experimental results of three through bolt beam-column connections under pure shear forces using modified push-out tests. The investigated specimens include extended end-plates and six through-bolts connecting square concrete-filled steel tubular column (S-CFST) to steel beams. The main goal of this study is to investigate if and how the mechanical shear connectors, such as steel angles and stud bolts, contribute to the shear transfer mechanisms in the steel-concrete interface of the composite column. The contribution of shear studs and steel angles to improve the shear resistance of steel-concrete interface in through-bolt connections was investigated using tests. The results showed that their contribution is not significant when the beam-column connection is included in the push-out tests. The specimens failed by pure shear of the long bolts, and the ultimate load can be predicted using the shear resistance of the bolts under shear forces. The predicted values of load allowed obtaining a good agreement with the tests results.

Keywords: beam-to-column connection; shear forces; through-bolt connection; steel-concrete interface; mechanical connectors

1. Introduction

Steel-concrete systems are widely accepted as structures of buildings, bridges and other engineering projects worldwide. The idea of filling steel tubes with concrete, composing the concrete filled steel tube (CFST) column, is becoming very popular in both Europe and America, including Brazil (De Nardin and El Debs 2013). The increasing use and studies of CFST columns are due to capability of this composite column to resist very high axial loads and moments with a small cross-section, high ductility, efficient fire resistance, constructability and reduction of labor cost by avoiding formworks. Despite these advantages, there are also some obstacles to the use of composite columns. The most important one lies in the connection of the CFST column to other elements composing the structural frame, thereby resisting the vertical and lateral loads. Several characteristics should be readily considered when designing a connection, such as strength, stiffness, ductility, ease of design and assembly, design assumption and cost. When it comes to beam-to-column connections, the easiest and most common way is to attach the steel beam directly to the wall of the steel tube without having components embedded in the concrete core. This method presents the advantages of being very simple, cheap and offering no internal restriction to the filling of the steel tube. However, high local stresses in the steel tube can result in excessive distortion of the walls and its eventual failure (Beutel *et al*. 2001, De Nardin and El Debs 2004, 2005). External diaphragm or T-stiffener are efficient alternatives to reduce the high local stress increasing the load capacity and deformation capacity (Shin *et al*. 2004). There are also beam-to-column connections aimed at to distributing some or all the beam forces directly into to the concrete core by embedded elements, thereby providing direct bearing on the concrete of composite column (Choi *et al*. 2010). These connections lead to disadvantages such as the more complex and expensive details and restrictions to the concrete flow into the steel tube, which could result in the formation of voids in the concrete core (Beutel *et al*. 2001, Choi *et al*. 2010). Several researchers have examined the basic behavioral characteristics of typical steel connections and have proposed specific alternatives to connect the CFST column to steel beams (De Nardin and El Debs 2004, Shin *et al*. 2004, Choi *et al*. 2010). The extended end-plate is an example of steel connection low manufacturing cost and high moment capacity that was adapted to allow the direct load transfer to the concrete core of the composite column. The end-plate combined with long bolts through the composite column were also investigated by researchers as Mclellan (1992), De Nardin and El Debs (2004) and Van-Long *et al*. (2015). For example, first Brazilian experimental program with several beam-to-column planar connections included two connections between steel beams and square concrete-filled

*Corresponding author, Associate Professor, E-mail: snardin@ufscar.br
[a] Associate Professor, E-mail: analucia@sc.usp.br

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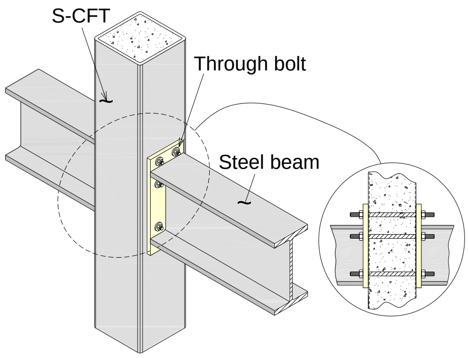


Fig. 1 Through-bolt connection for composite column and steel beam

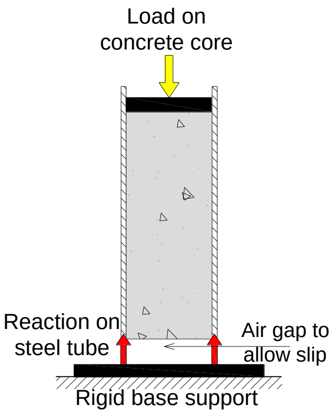
steel tube (S-CFST) columns using long bolts (De Nardin and El Debs 2004). These specimens comprised a S-CFST column connected on both sides to identical I-steel beams by extended end-plates and six through-bolts (Fig. 1). The experimental results showed that such a connection is suitable to be used for moment resistance frames. Despite this, the long bolts used to connect beams to CFST columns are still rare in the construction field and no design procedure has been included in the standard codes. This connection was chosen for the present study because field welds are not required and local deformations in the wall of steel tube are prevented. The proposed connection was firstly studied by other researchers, however the steel-concrete load transfer and the contribution of mechanical connectors to this mechanism have not been clarified yet (Mclellan 1992, De Nardin and El Debs 2004, Van-Long *et al*. 2015). The influence of bolt prestressing level and bolt embedment conditions were investigated by Mclellan (1992) and the following transfer mechanisms in the steel-concrete interfaces were identified: bolt-concrete bearing and friction between steel tube and concrete core. The concrete core works together with the column face supporting the tension forces in the bolts improving the stiffness and resistance of the joint (Van-Long *et al*. 2015).

Recently, researchers proposed a design procedure based on components method, in which joints under bending, shear and the load introduction, were investigated (Van-Long *et al*. 2015). Considering the joint under shear, some transfer mechanisms should be highlighted, namely: end-plate in bearing, end-plate in block tearing, bolts in shear, and concrete core in bearing. Some of these have already been investigated, and an analytical model was proposed and validated to predict the response of the beam-column connection by long bolts (Van-Long *et al*. 2015). Despite this, it remains uncertain whether and how the steel-concrete interface can effectively transfer the axial load from the steel tube to the concrete core of the CFST column. Besides, the load applied onto a composite column must be distributed between the steel and concrete components, primarily considering the shear resistance at the steel-concrete interface.

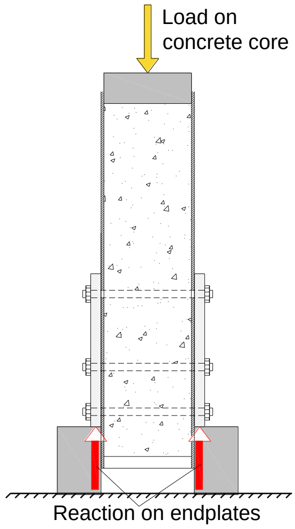
Standard codes, such as the Brazilian code (ABNT 2008) and the Eurocode 4 (BS EN 1994-1-1 2004) recommend that shear connectors should be provided in the load introduction length and in areas with changes of cross-section when the shear resistance is exceeded at the steel-concrete interface. As it is known, in real structures, the floor loads are introduced to the columns through the connections, which must ensure that both components of columns (steel and concrete) contribute to support them. As concrete-filled columns have the concrete core resisting a substantial proportion of the axial load, the vertical shear transfer from the beams to the concrete core of the column is a concern. How the loads are transferred depends on the connection detail, and the efficient loading transfer from floor to the column is responsible for the good behavior of the structure. Push-out tests are the most usual procedure to investigate the steel-concrete load transfer and the bond behavior between the steel tube and the core in CFST columns. In that, an axial compressive load is applied onto the top of the specimen only on the concrete core and it is resisted by resting the steel section on its base (Fig. 2(a)). This procedure has been carried out on several specimens of CFST columns with rectangular, square and circular cross-sections to investigate the bond strength at steel-concrete interface and the contribution of mechanical shear connectors (Shakir-Khalil 1993a, b, Parsley *et al*. 2000, Johansson 2003, De Nardin and El Debs 2007, Qu *et al*. 2013, 2015, Xu *et al*. 2009). For example, push-out tests were conducted by Petrus *et al*. (2011) to investigate the bond strength of the CFST columns with tab stiffeners. The results showed that the bond strength of the concrete-steel interface increased with decreasing as of the tab stiffener spacing decreased.

Usually, the bond stress presents three theoretical components (chemical adhesion, micro-locking and macro-locking) and most test results showed that mechanical shear connectors as angles, nails, structural bolts and headed studs did not increase the bond resistance, since these mechanical connectors only come into action when the steel-concrete bond has failed (Shakir-Khalil 1993a, b, Parsley *et al*. 2000, Johansson 2003, De Nardin and El Debs 2007).

The composite action, i.e., the effective shear stress transfer from one material to another, can be achieved either by natural bond or with the aid of mechanical connectors as bolts, angles, nails and shear studs. Experimental results from the literature indicated that the shear resistance of the steel-concrete interface is strongly dependent on the shape and size of the steel tube as well as on the steel-concrete interface conditions (Shakir-Khalil 1993a, b, Qu *et al*. 2013, 2015, Xu *et al*. 2009). Regarding the cross-section shape, friction forms uniformly around the internal perimeter of the circular tube, but in square tubes it forms mainly near the corners of a square tube (Song *et al*. 2015), and because of this, rectangular CFT columns have smaller bond strength than circular ones (Shakir-Khalil 1993, Qu *et al*. 2013, 2015). The interface conditions affect mainly the first stages of loading where the chemical adhesion and micro-locking components of bond strength are predominant. Comparison of bond strength and slip values achieved by lubricated and non-lubricated CFTS columns have shown the influence of the interface lubrication. The results of ultimate bond strength and slip from the literature have indicated that the slip of lubricated specimens is larger than that of non-lubricated one, whereas the ultimate bond strength presents a significant reduction due to lubrication



(a) Push out test



(b) Modified push out test

Fig. 2 Experimental procedures to evaluate the load transfer mechanisms in composite columns

(Shakir-Khalil 1993, Qu *et al*. 2013, 2015). There is no consensus about the influence of the concrete strength on the bond strength (Qu *et al*. 2013, 2015, Xu *et al*. 2009). Apparently, the beneficial effect of an increase in the concrete strength is accompanied by the detrimental effect of increased shrinkage. Some researchers suggest that the effect of concrete strength must be examined in conjunction with the cross-section effects, because the higher shrinkage associated with higher concrete strength has more adverse effects with an increased section size (Qu *et al*. 2015). The influence of concrete type on bond strength was investigated, and it can be seen that the expansive concrete is very effective to improve the bond strength in circular CFST columns but less effective for square CFST columns (Song *et al*. 2015). The steel tube restrains the lateral expansion of the expansive concrete and for early age concrete, such an expansion leads to the formation of pressure between the concrete and the steel tube, thereby increasing the bond strength. This effect is more significant if the expansive concrete is associated with stainless steel in circular CFST columns.

To further test the shear transfer mechanism in CFT columns, firstly was studied the influence of angles on the beam-to-column connection (De Nardin and El Debs 2004). Two planar specimens were subjected to loads on the beams ends, and the welded connection behavior was compared to a similar one stiffened by angles welded on the internal surface of the steel tube. These angles were welded at the same level as the upper and bottom flanges of an I-beam and were embedded in the concrete core of the CFST column. The angles increased the connection moment capacity, and these first results encouraged the researchers to investigate the influence of angles and headed stud bolts on the shear transfer on the steel-concrete interface of CFST columns (De Nardin and El Debs 2004). Next, the influence of shear connectors as stud bolts and angles were investigated and the experimental results showed that these mechanical connectors had a good contribution to the steel-concrete transfer mechanisms (De Nardin and El Debs 2007).

As previously mentioned, several studies have been conducted using push-out tests. Nevertheless, the bond strength between the steel tube and the concrete core including the connection, has received relatively little attention in comparison to the behavior of the CFST elements. Most of the tests conducted thus far to determine the shear transfer behavior in the steel-concrete interface have focused on the bond strength. However, a more realistic shear transfer mechanism can be utilized using specimens that include the connection detail to the CFST column. This procedure was adopted by Mollazadeh and Wang (2016) to investigate the load transfer mechanism from the shear connection to CFST columns. The authors concluded that the connection load is introduced to the concrete core through the column length above and within the connection. Moreover, changing the construction details below the connection has no effects onto the increase of the column resistance (Mollazadeh and Wang 2016).

In this background, the Department of Structural Engineering of the University of São Paulo, with the support of researchers at the Department of Civil Engineering of the São Carlos Federal University, created a research program to improve the knowledge on steel-concrete interface in concrete filled steel tube columns (CFST columns). The present paper is part of this research program and the results obtained herein may contribute to a better understanding of the load transfer mechanisms in CFST columns, including the beam-column connection.

The present paper summarizes the results of modified push-out tests (Fig. 2(b)), where short rectangular CFST columns were connected by long bolts (threaded bars) passing through the column to the beams. The primary focus of the tests was to investigate how the shear loads from the beams are transferred to axial load on the concrete core of the CFST column when a part of the joint is included in the specimens. The tested parameters include the three interface types: normal interface, interface with shear studs and interface with steel angles. Based on the findings, the effects of the mechanical connectors on the shear transfer mechanism at the steel-concrete interface were investigated, supplementing previous Brazilian research in this area (De Nardin and El Debs 2007, Araujo 2009). Modified push-out tests were performed with one group of specimens. The effects of angles and shear studs on the steel-concrete interface of the CFT columns were investigated considering a specimen without mechanical connectors as a reference.

2. Experimental study

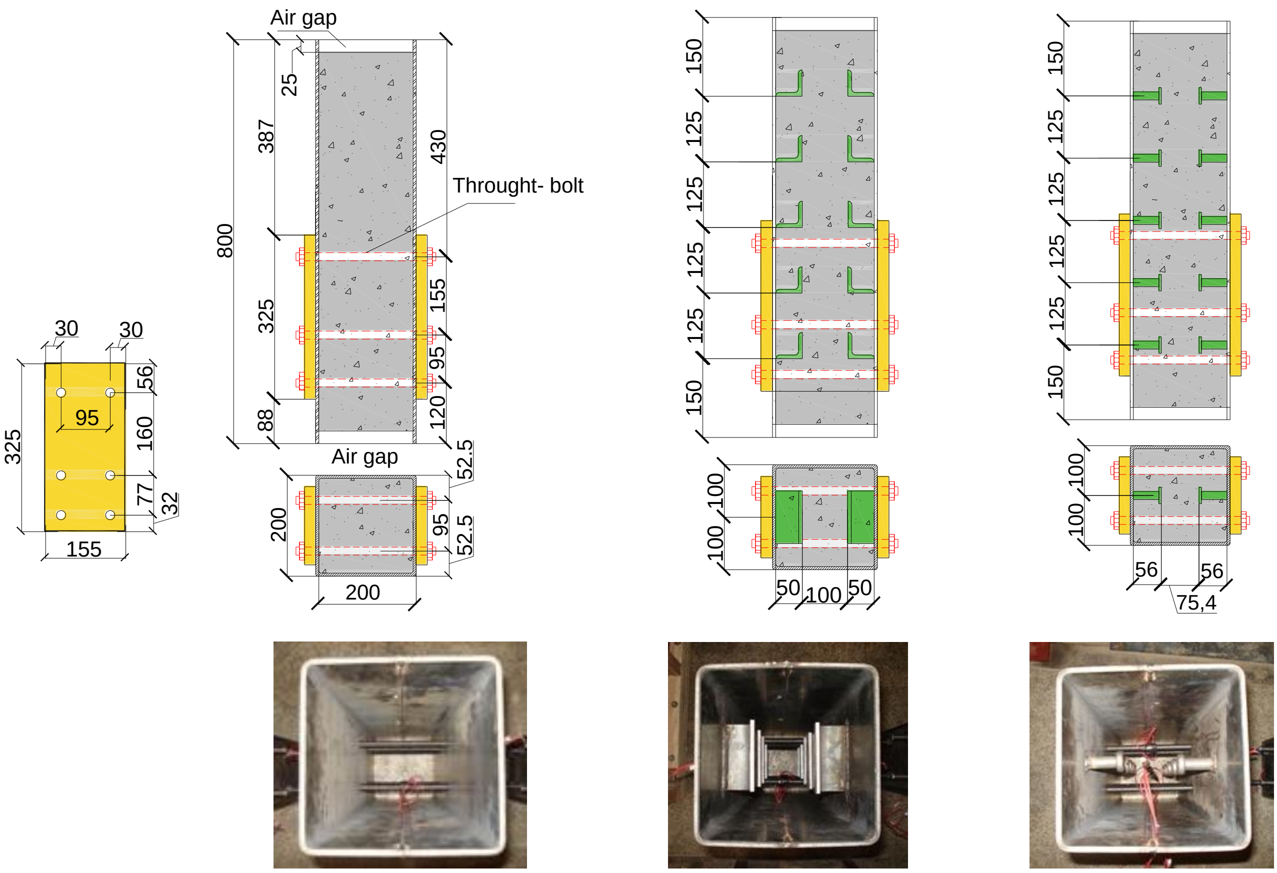
2.1 Description and dimensions of the specimens

The authors conducted a previous experimental study with push-out tests to determine the influence of shear studs and steel angles on both the load-slip response and the distribution of axial load to the CFST elements (De Nardin and El Debs 2007). The results indicated that the tested mechanical shear connectors were very efficient to decrease the concrete core slip and increase the maximum load. The cross-section of the CFST column, steel angles, and shear studs, including their spacing used in the previous study, were kept the same in the present study, however it was included the steel plate representing the beam-to-column connection and long bolts. Three specimens representing the planar beam-to-column connection between the CFST column and the steel beam were investigated in the present study. A summary of the main characteristics is given in Table 1 and details about each specimen are further described. The specimens were named starting with 'SBC', which means steel bolted connection; the letter 'W' refers to specimens without mechanical connectors (single interface); the letters 'SB' and 'A' correspond to shear studs and steel angles, respectively. No special measure was done to treat the inner surface of the steel tube, and the corresponding interface between steel tube and concrete is refer to as a single interface (specimens with letter 'W'). All specimens had a 750 mm interface length, CFST column and long bolts (threaded bars) with similar geometry.

Table 1 General characteristics of tested specimens

Group	Specimen	Connection type and forces	Steel-concrete interface
SBC	SBC-W	Steel connection under pure shear forces	Single
	SBC-SB		Shear studs
	SBC-A		Steel angles

The specimens were made of a square concrete-filled steel tube column with a height of 800 mm, six long bolts with a diameter 16 mm passing through the column, and two end-plates representing the beam-to-column connection (Fig. 3). The geometrical properties of the end-plates and CFST column are presented in Figs. 3(a) and (b),



(a) Endplate (b) SBC-W (c) SBC-A (d) SBC-SB

Fig. 3 Components geometry of SBC specimens. Units: mm respectively. The steel tube cross-section of the composite column was made by seam welding two U-shaped cold-formed profiles with a cross-section of 200 mm × 100 mm × 6.3 mm (Fig. 3). A gap of 25 mm at both ends of the specimen allowed for the compressive load to be only applied onto the concrete core of the composite column and transferred to the steel tube by the steel-concrete interface. The specimen SBC-W was tested without shear connectors (Fig. 3(b)), and the results were used as a reference to evaluate the contribution of steel angles (specimen SBC-A) and shear studs (specimen SBC-SB) to the load transfer at the beam-to-column under shear forces. Two rows of shear studs with 19 mm diameter and 56 mm length were welded on the inner surface of the steel tube along the longitudinal direction of the specimen SBC-SB (Fig. 3(d)). Steel angles with 50 mm × 50 mm × 6.3 mm and 100 mm length were welded on the inner surface of the composite column of the specimen SBC-A (Fig. 3(c)). The steel angles as well as the steel tube of composite column were made of SAE 1020 steel.

2.2 Mechanical properties of materials

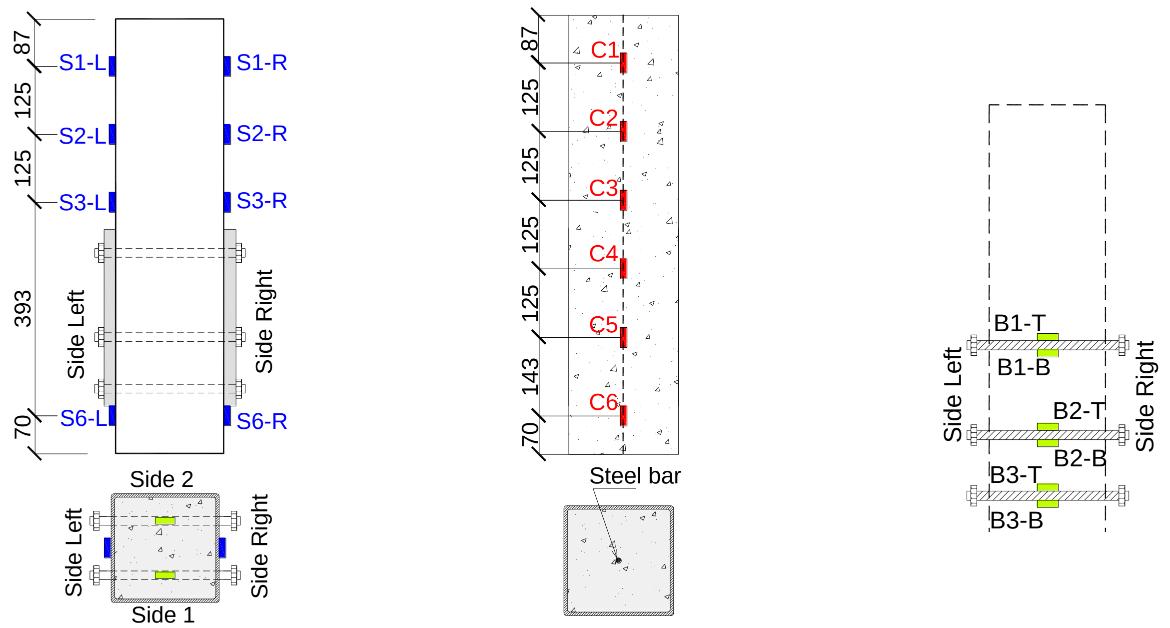
Table 2 presents the main mechanical properties of materials utilized in the tested specimens and the values corresponding to the average of the material properties. For all specimens, the end-plates were made of ASTM A-36, and the through-bolts were made of ASTM A325 steel, with the latter being, threaded steel bars. The square tubular steel section of the composite column was of SAE 1020 steel type. The materials were tested with samples of structural steel, bolts, reinforcement bars and concrete core. To characterize the steel material, standard tensile coupon tests were conducted using three coupons taken from each steel component: steel tube and end-plate. The steel samples were removed and tested according to E 8M-00 (ASTM E 8M 2000). The mechanical properties of the through-bolts and reinforcement bars were measured by tensile tests of the threaded bars with 800 mm length. Average values of yielding stress (f_y) and ultimate tensile strength (f_u) of steel components are listed in Table 2. The f_u value of the stud bolt in Table 2 corresponds to the nominal value provided by the manufacturer.

2.3 Measurement system

The measurement system was designed to allow monitoring strains and displacements and to obtain as much information as possible from each tested specimen (Figs. 4 and 5). Electrical strain gauges, linear variable displacement transducers (LVDT) and load cells were distributed on each tested specimen. Four strain gauges were placed in each external face of the steel tube (Fig. 4(a)). The concrete core strains were measured by six strain gauges fixed to a steel bar placed in the center of the composite column (Fig. 4(b)). The steel bars were kept in the position during concreting using plastic rebar concrete spacer no slippage was observed between the steel bar and the concrete core. The

Table 2 Steel and concrete mechanical properties (units: N/mm^2)

Concrete			Steel of column		Long bolts		Steel plate		Stud bolts
f_c	f_t	E_c	f_y	f_u	f_y	f_{ub}	f_y	f_u	f_u
54.9	3.7	35,100	242.2	359.3	743.9	785.4	305.2	430.5	415



(a) Steel tube (b) Concrete core (c) Through-bolts

Fig. 4 Arrangement of strain gauges

strains gauges on the steel tube and concrete core allowed evaluating the force transfer between these components. The axial strains on through-bolts were also measured by strain gauges on top and bottom faces of the bolts, located at the center of the bolt length (Fig. 4(c)).

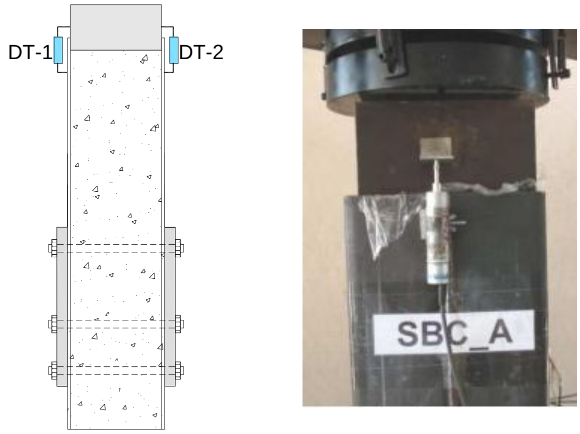


Fig. 5 Arrangement of electrical transducers

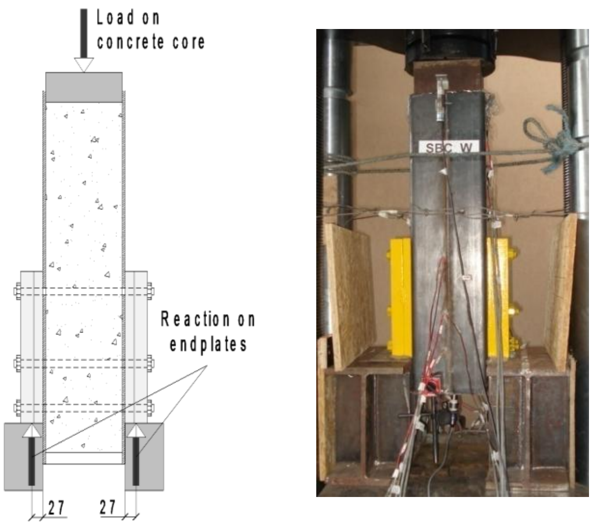


Fig. 6 Test arrangement. Units: mm

Two displacement transducers (DT-1 and DT-2, Fig. 5) were used to evaluate the slip between steel tube and the concrete core of the CFST column. The transducers were attached to the steel load plate and the steel tube near the concrete-loaded end of the specimens. These transducers were used to measure the relative displacement between the upper crosshead (concrete core) and the top of the steel tube.

2.4 Test setup and loading

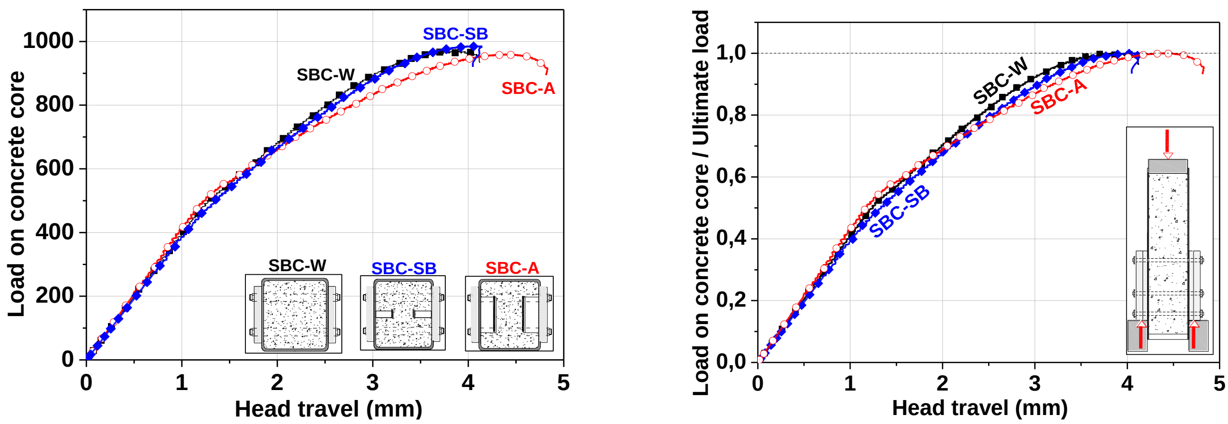
All specimens were tested using modified push-out tests (Fig. 5), i.e., a variable vertical load was applied on the concrete core of CFT column and reaction forces were applied on the end-plate of the specimens (Fig. 6). The load was applied by the spherical crosshead of the testing machine and a very rigid steel block was placed on the concrete core whose length was shorter than the steel tube. This test setup was designed to develop shear stress at the steel-concrete interface and allow evaluating the influence of angles and shear studs on the shear transfer mechanism. To help the load application on specimens, two auxiliary steel plates were bolted together with the end-plates. These auxiliary plates had 32.0 mm thickness and allowed the application of the shear forces directly on the through-bolts by reaction forces. The vertical load on the concrete core was introduced by a computer-controlled hydraulic actuator with 3,000 kN of static capacity using displacement control mode and application speed of 0.005 mm/second.

3. Tests results

The experimental results were organized in graphics and commented in the next subsections. The results made it possible to evaluate the influence of mechanical connectors on the shear transfer mechanism when the connection is predominantly under shear.

3.1 Global response and ultimate load

The global response of all tested specimens can be seen at the Load on concrete core vs. Head travel (Fig. 7). The head travel measurements correspond to the vertical displacement of the machine head travel. All specimens



(a) (b)

Fig. 7 Load on concrete core vs. Head travel response

Table 3 Summary of experimental results

Specimen	F_u (kN)	Head travel (mm)	Slip (mm)	V_u (kN)	Failure mode
SBC-W	969.24	3.90	0.75	484.62	Shear of bolts
SBC-A	958.9 (-1.07%)	4.38 (+12.31%)	0.94 (+25.33%)	479.45	Shear of bolts
SBC-SB	984.86 (+1.61%)	4.09 (+4.87%)	0.71 (-5.33%)	492.43	Shear of bolts

* F_u: ultimate load applied on concrete core of composite column;
V_u: ultimate reaction force on steel plate connection

Table 4 Theoretical and experimental shear forces on bolts

Specimen	V_u (kN)	Force to shear plane (kN) $F_{v,E} = V_u / 6$	$F_{v,R}$ (kN)	$F_{v,E}/F_{v,R}$
SBC-W	484.62	80.77		1.09
SBC-A	479.45	79.91	74.22	1.07
SBC-SB	492.43	82.07		1.10
Average	485.5	80.92		1.09

presented nearly linear behavior up to 50% of the ultimate load. Later, the specimen with angles (SBC-A) presented a more sudden decrease when compared to the other specimens.

No significant difference in stiffness was observed at initial stages of loading. The difference was only strongly evident from approximately 0.7 of peak load (Fig. 7). Ultimate load results (Table 3) showed that the presence of shear studs and angles was not significant to increase the ultimate load. Considering the ultimate load, the specimens with angles (SBC-A) and stud bolts (SBC-SB) presented, respectively, values of 1.07% lower and 1.6% higher than those observed in specimen without shear connectors (SBC-W). Therefore, almost no difference was observed with the variation of the mechanical connectors, and the values of the ultimate load were probably caused by the presence of the bolts, independently of angles or shear studs. When both the column and the connection were loaded to represent a column in a multistory building, the observed failure mode was the shear of bolts. It occurred independently of the type of shear connectors in the steel-concrete interface.

The beam-to-column connections tested with through-bolts and extended endplates had bolts under pure shear forces. This fact was in accordance with the failure mode observed in the tests. Thus, the shear connection resistance can be based on the shear resistance of an individual bolt per shear plane, and can be predicted using design codes, such as the Eurocode 3 (BS EN 1993-1-8 2002). This standard code recommends the Eq. (1) to the shear resistance per shear plane when it passes through the threaded portion of the bolt

$$F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M2}} \tag{1}$$

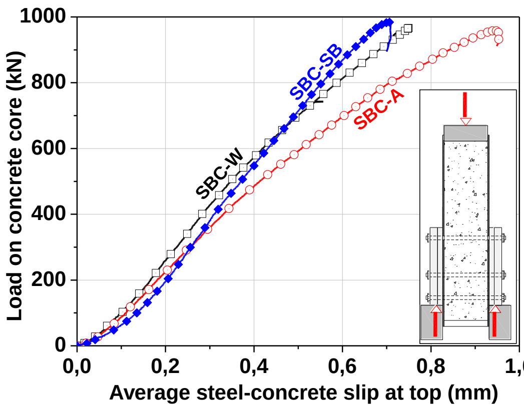
The parameter α_v is a function of strength grade of steel. In the present study, the experimental value of ultimate tensile strength (f_{ub}) of the steel bolts (Table 2) determined by tensile tests described in Section 2.2 corresponds to bolt grade 8.8 of the Eurocode 3 (BS EN 1993-1-8 2002). Because of this, α_v was considered equal to 0.6. The tensile stress area-to-gross area ratio (A/A_g) was assumed equal to 0.78, taking into account the thread type of the through-bolt. To compare the value of shear resistance per shear plane ($F_{v,Rd}$) with the experimental values of force to the shear plane ($F_{v,E}$) the partial safety factor γ_{M2} was taken equal to 1.0. This resulted in Eq. (2) that allow to estimate the shear resistance of bolts per shear plane.

$$F_{v,R} = 0.47 \cdot f_{ub} \cdot A_g \tag{2}$$

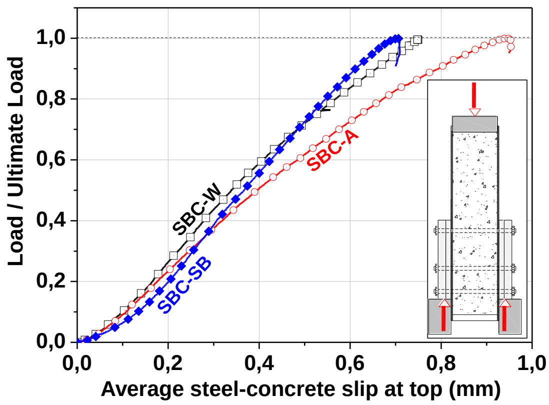
The experimental and theoretical values of force per shear plane are given in Table 4. The comparison shows a good agreement between both values for all tested specimens (Table 4). Therefore, the Eq. (2) can be used to estimate the resistance of through-bolts when the bolts are under pure shear forces. This was also concluded by other researchers who evaluated the shear resistance of long bolts without mechanical connectors in the steel-concrete interface (Van-Long *et al*. 2015). Furthermore, the results confirm that the ultimate load is highly dependent on the bolts on shear forces, so, these values were not influenced by the angles and stud bolts. The values of force to the shear plane ($F_{v,E}$, Table 4) were estimated using the experimental values on each side of the connection (V_u, Table 4) divided by the total number of bolts. The shear planes were taken equal to 6.

3.2 Steel-concrete slip behavior

The specimens without mechanical connectors (SBC-W) and with angles (SBC-A) revealed similar initial behavior on the steel-concrete slip response (Fig. 8). On the other hand, the stiffness of specimen with stud bolts (SBC-SB) was lower than others from the initial stages of loading, until nearly 0.2 of the ultimate load. However, from approximately 0.4 of peak load, the specimen with angles (SBC-A: Fig. 8) presented higher slip than other specimens, indicating an unexpected effect of the angles on the steel-concrete forces transfer. This behavior continues until the ultimate load. Apparently, the angles did not contribute to



(a)

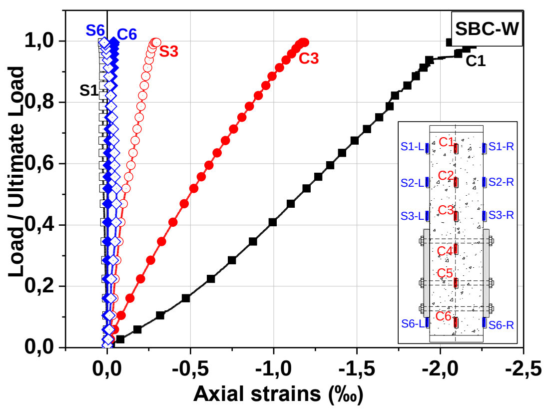


(b)

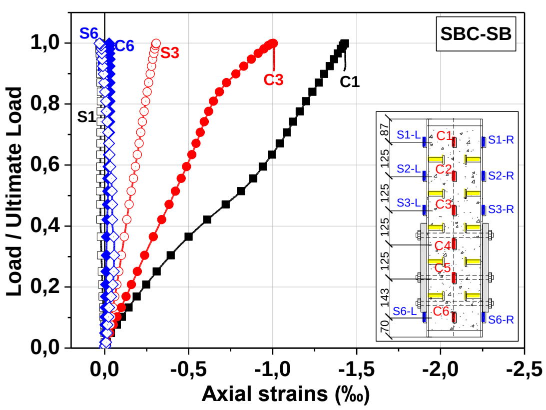
Fig. 8 Steel-concrete slip at top of specimens

reduce de steel-concrete slip resulting in values of slip, resulting in higher slip values than those recorded in the absence of mechanical connectors (SBC-W). However, this fact needs to be better investigated to evaluate the real contribution of the angles. At the ultimate load, the specimens with angles (SBC-A) presented steel-concrete slip 25.3% higher than the value recorded in the absence of shear connectors (SBC-W). Moreover, a significant difference between the slip in specimens with (SBC-SB) and without shear studs was observed. The ratio between these two slips (0.71/0.75) was 0.94 (the highest values correspond to the specimen without mechanical connector).

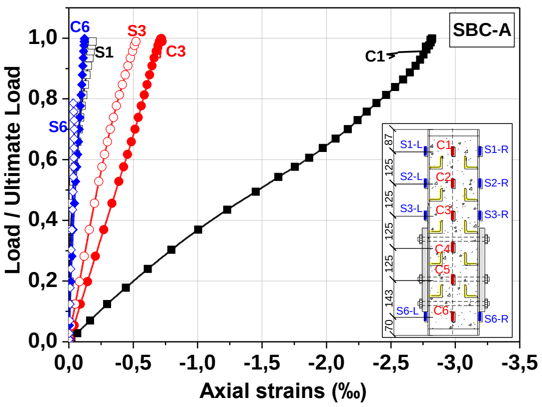
When the load is applied onto the concrete core and the reaction into the steel tube (Fig. 2(a)) there is a significant contribution of the angles to the shear forces transfer, and the failure mode of the specimens is determined by the types of mechanical connectors (De Nardin and El Debs 2007). On the other hand, the angles are less effective in transferring the forces when the load is applied onto the concrete core and the reaction at the connection (Fig. 2(b)). In this case, the failure was due to shear of bolts (Table 3) independently of the type of the shear connector.



(a) Specimen SBC-W

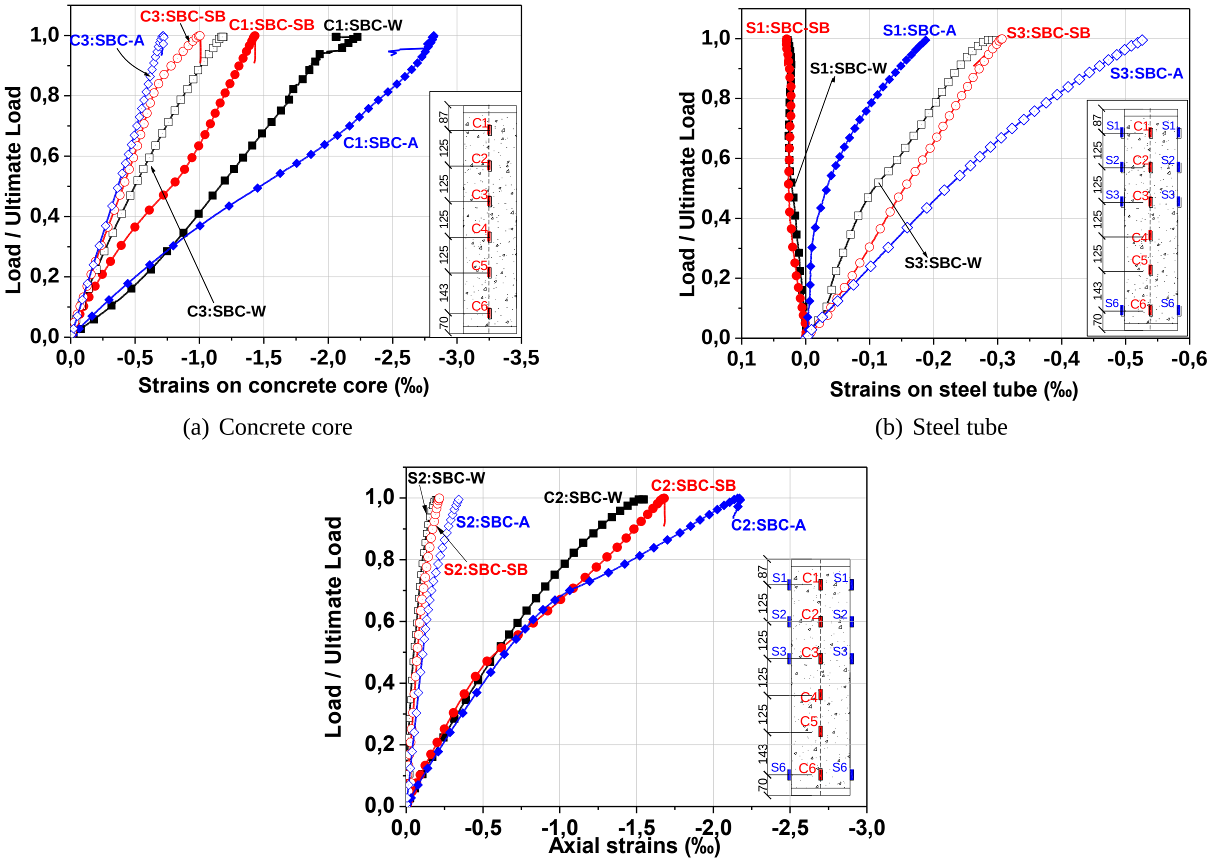


(b) Specimen SBC-SB



(c) Specimen SBC-A

Fig. 9 Strains in concrete core and steel tube: contribution of mechanical connectors



(a) Concrete core

(b) Steel tube

(c) Strains at level 2

Fig. 10 Strains in concrete core and steel tube: contribution of mechanical connectors

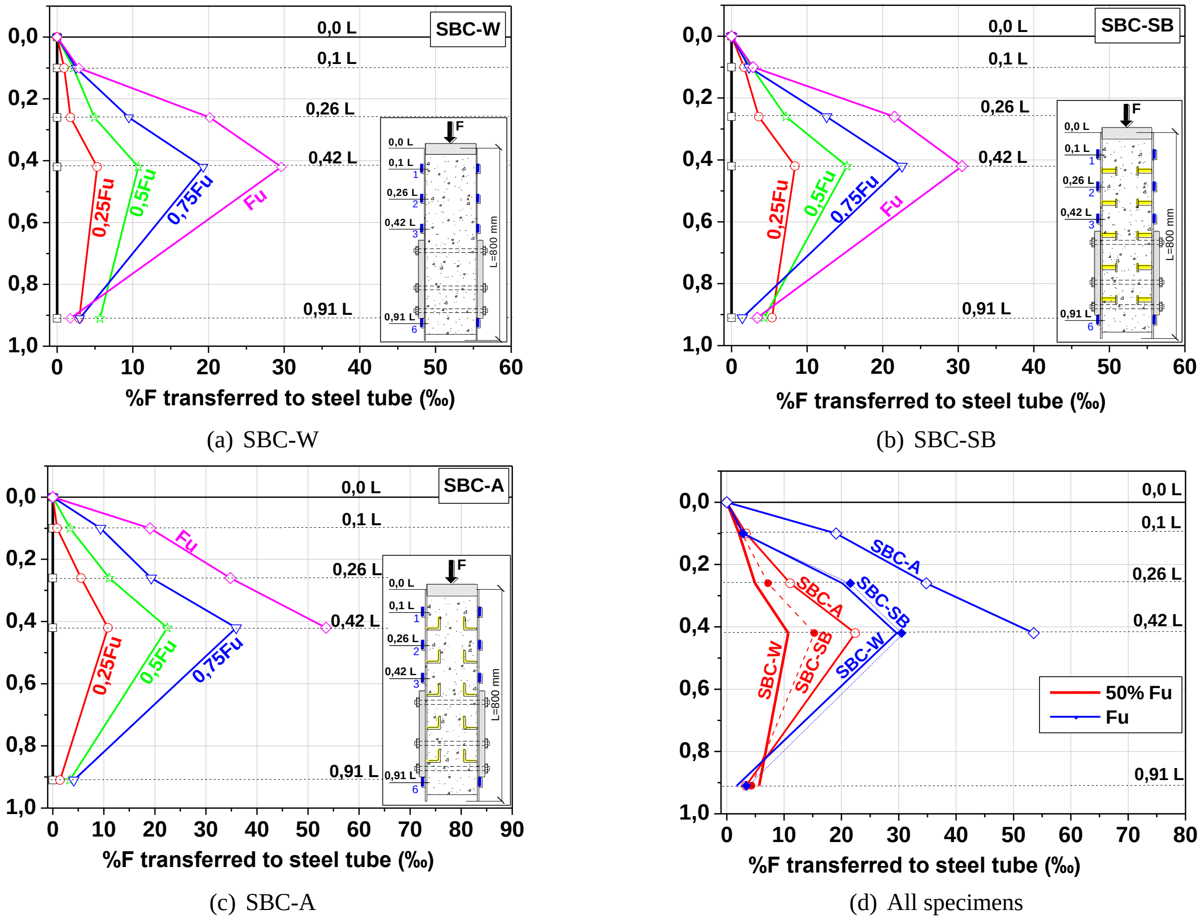
Apparently, including angles in the steel-concrete interface merely transfers the force from the concrete to the steel tube without increasing the total load applied. This fact probably occurs because the angles are very rigid and their contribution is effective only when the strains are significant. However, these first results should be confirmed with complementary tests and numerical simulations.

3.3 Axial strains on components of composite column

The variation of strains along the steel tube and the concrete core as well as the strain responses on components of the composite columns are summarized in Figs. 9 and 10. Both the axial strains in concrete core and steel tube allowed understanding how the load is transferred between these components and the contribution of angles and shear studs to the steel-concrete shear transfer. To this, the strains of the steel tube were recorded along two opposite sides and the values presented here correspond to the average values recorded on points placed at the left and right sides of the tube (Fig. 4). The specimen with angles (SBC-A, Fig. 9(c)) presented the highest values of compressive strains in the concrete core (point C1). The specimen without mechanical connectors (SBC-W, Fig. 9(a)) and the specimen with shear studs presented values of strain near 2.2 ‰ and 1.5 ‰, respectively. These values were significantly lower than those of the specimen with angles (2.9 ‰). The0 compressive concrete strains recorded at the point C3 (Fig. 9) were not similar on the tested specimens. Apparently, the angles were the most efficient mechanical connector to transfer the applied load to the steel tube. The strains at point C3 were much lower in specimen SBC-A while similar values were found in the specimens SBC-W and SBC-SB (Figs. 9(a) and (b)). This fact is confirmed by the higher values of strains recorded at point S3 (steel tube) of the specimen SBC-A in comparison with the other specimens. These higher values indicate higher stresses in the steel tube of this specimen.

As the load was introduced onto the composite column by the concrete core, the strains recorded in this component decreased gradually from the top towards the base (point C1 to C6). This decrease was noticeable for all specimens (Fig. 10(a)). On the other hand, the strains on the steel tube increased gradually from the top towards the base, confirming the load transfer between the concrete and the steel tube (Fig. 10(b)). Therefore, the load transfer was observed in all tested specimens, and it was more intense in the specimen with angles (SBC-A) where the highest strains were recorded at points S1 and S3. The strains at levels 2 and 3 presented similar behavior (Fig. 10(c)).

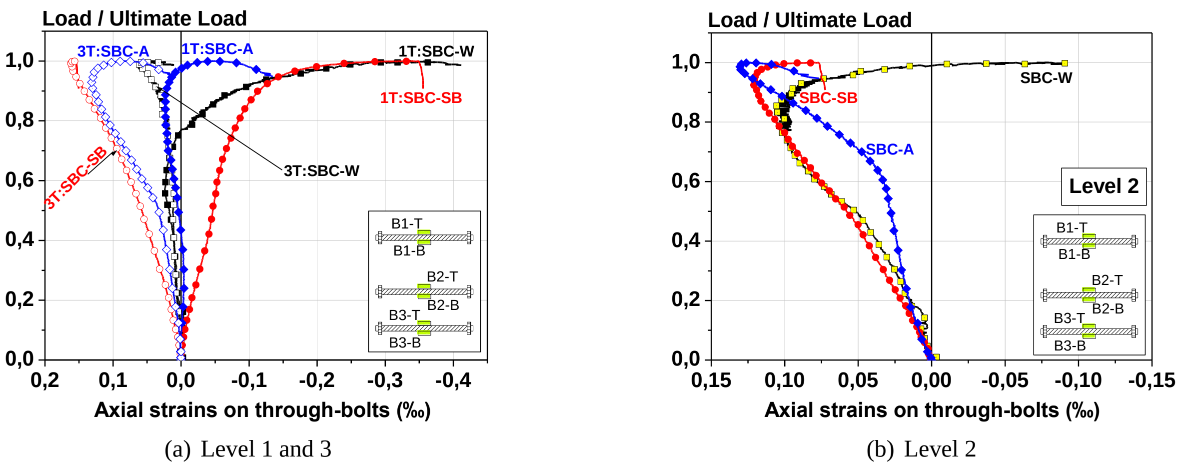
The specimens with angles and shear studs exhibited similar responses at point S3 only to 0.2 of peak load (Fig. 10(b)). Just after this point, non-linear responses of strains were clearly identified in the specimens without mechanical connectors (SBC-W) and with angles (SBC-A). The



(a) SBC-W (b) SBC-SB

(c) SBC-A (d) All specimens

Fig. 11 Load distribution along length of SBC group specimens



(a) Level 1 and 3 (b) Level 2

Fig. 12 Influence of mechanical connectors on through-bolts strains

specimen with shear studs (SBC-SB) exhibited linear response almost until the peak load (Fig. 10(b)). By means of concrete strain response (Fig. 10(a)), it was observed that in all tested specimens the strains at level 1 (C1) were much higher than at level 3 (C3). This confirms the load transfer to the steel along the tube length. In the specimen without mechanical connectors (SBC-W), at point C3, placed immediately above the end-plate, higher strains were recorded. Regarding strains recorded in concrete core at level 1 (C1), specimens with angles (SBC-A) and without mechanical connectors (SBC-W) presented almost the same response for values up to 0.3 of peak load (Fig. 10(a)). The specimen with shear studs (SBC-SB) presented lower strains for all stages of loading. This response exhibited by the specimens of SBC group suggests that shear studs contribute to steel-concrete transfer since the initial stages of loading while the angles only become efficient after 0.3 of peak load (Fig. 10(a)). Besides, the strain responses and values at point C3 were very similar for specimens with angles (SBC-A) and shear studs (SBC-SB). A probable cause for this is that the angle stiffness affects the load transfer mechanism. Apparently, the most effective contribution of the angles from of a certain level of the load has a direct relationship with the more rigid connection offered by these mechanical connectors in comparison to the stud bolts. However, these results should be confirmed with complementary tests and numerical simulations.

3.4 Load distribution in composite columns

The load distribution along the composite columns length for applied loads of 0.25, 0.5, 0.75 and ultimate load (F_u, Table 3) are shown in Fig. 11. In all specimens, at the upper end of the steel tube the load was applied only to the concrete core and the reaction forces were applied on the endplate connection (Fig. 6). The load distribution was obtained using the load applied on the concrete core and the axial strains measured in steel tubes of composite columns (Fig. 10). The distributed load on the steel tube was calculated considering the Hooke's law relating axial stresses and corresponding axial strains. This load was represented by the force on steel tube divided by the load applied on the concrete core. The highest values of load transferred from the concrete core to the steel tube were recorded below the second line of mechanical connectors, at 0.42 of the column length, for both tested groups. The presence of mechanical connectors on the steel-concrete interface of the composite columns affects the transfer of forces to the steel tube. In both groups, specimens with angles exhibited the highest values of the load transferred to steel tube. This value was 53.3% of the load applied on the concrete core to SBC-A specimen, while the specimen without mechanical connectors exhibited 29.6%. The load distribution along the length of specimen SBC-W (Fig. 11(a)) allowed evaluating the shear transfer in the steel-concrete interface by adhesion and the mechanical characteristics of the interface. The results of the transferred load at F_u (Fig. 11(d)) lead to conclude that in the absence of mechanical connectors the steel-concrete interface was able to transfer 29% of the applied load when the connection was subjected predominantly to pure shear (SBC group).

3.5 Strains on through-bolts

Strain gauges placed at several points on top and bottom of each through-bolt of the SBC group specimens (Fig. 4(c)) and made it possible to evaluate the influence of the angles and shear studs on the through-bolt strains (Fig. 12). At level 2, the strains on specimens with shear studs (SBC-SB) and without shear connectors (SBC-W) exhibited similar behavior up to 0.8 of peak load (Fig. 12(b)). From this point, both specimens presented abruptly change on the load-strain response with an apparent reversal of strains, and this was more intense in specimen SBC-W. Like what was shown for level 2, the highest strains at level 1 (Fig. 12(a)) were recorded on the specimen with shear studs (SBC-SB). Moreover, in the absence of shear connectors or in the presence of angles, the strains were similar also up to 0.7 peak load. Therefore, the presence of mechanical connectors, such as the angles, changed the response and values of the through-bolt strains at level 1.

Table 5 General characteristics of tested specimens

Specimen	F_u (kN)	Slip (mm)	Failure mode
CFT-S*	63.1	11.48	Steel-concrete slip
CFT-A*	1071.9	3.66	Steel-concrete slip and steel tube deformation
CFT-SB*	684.5	2.65	Rotation of the angles and steel tube deformation
SBC-W	969.24	0.75	Shear of bolts
SBC-A	958.9	0.94	Shear of bolts
SBC-SB	984.86	0.71	Shear of bolts

* De Nardin and El Debs (2007)

4. Influence of beam-column connection

The influence of the steel plate connection to the shear transfer mechanism can be evaluated comparing the results of the present study with previous results of De Nardin and El Debs (2007). The specimens CFT-S, CFT-A and CFT-SB were dedicated to the characterization of the slip resistance between the steel tube and the concrete core. As it can be seen in the test results summarized in Table 5, significant loads were reached when angles (CFT-A) and shear studs (CFT-SB) were included in the steel-concrete interface, in particular for the specimen with steel angles (F_u = 1071.9 kN in comparison with 63.1 kN and 684.5 kN, respectively, without mechanical connectors and with stud bolts). In addition, a significant reduction in the values of the steel-concrete slip was recorded, a consequence of the angles and stud bolts (Table 5). When the steel plate was introduced (Fig. 6), the maximal load (F_u), the failure mode, and the steel-concrete slip were almost the same for the tests indicating that there is no significant effect of the angles and stud bolts on the bolt shear resistance. Furthermore, the application of the shear through the concrete core and the steel plate support is very significant and significantly affects the shear transfer mechanisms and the failure modes.

5. Conclusions

This paper presented an experimental investigation of beam-to-column connection using through-bolts and the influence of steel angles and shear studs were investigated using modified push-out tests. According to the short test program outlined here, several conclusions can be drawn and are summarized as follow:

- Specimens under pure shear force (SBC group) presented an increase in the load transferred to the steel tube when angles were welded on the steel tube of the composite column. However, no significant contribution of the mechanical connectors to reduce the steel-concrete slip was recorded;

- Furthermore, the peak load values were not influenced by the angles and shear studs on the steel-concrete interface, and shear of long bolts was founded to be the failure mode recorded in all specimens;
    - The consideration of the steel plate connection modified the failure mode and the maximum applied load of the tested specimens in comparison with the traditional push-out test (De Nardin and El Debs 2007);
- Further studies are suggested to elucidate the shear transfer mechanism when the beam-column connection is considered in the tests especially in relation to the angles contribution.

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