Steel and Composite Structures, Vol. 7, No. 5 (2007) 377-390
DOI: http://dx.doi.org/10.12989/scs.2007.7.5.377

Shear transfer mechanisms in composite columns: an experimental study

Silvana De Nardin†

Department of Structural Engineering, University of São Paulo at São Carlos, São Carlos, Brazil

Ana Lucia H. C. El Debs‡

Avenida Trabalhador São-Carlense 400, Centro, São Carlos-SP - CEP: 13.566-590, Brazil

(Received November 2, 2006, Accepted September 7, 2007)

Abstract. In the design of concrete filled composite columns, it is assumed that the load transfer between the steel tube and concrete core has to be achieved by the natural bond. However, it is important to investigate the mechanisms of shear transfer due to the possibility of steel-concrete interface separation. This paper deals with the contribution of headed stud bolt shear connectors and angles to improve the shear resistance of the steel-concrete interface using push-out tests. In order to determine the influence of the shear connectors, altogether three specimens of concrete filled composite column were tested: one without mechanical shear connectors, one with four stud bolt shear connectors and one with four angles. The experimental results showed the mechanisms of shear transfer and also the contribution of the angles and stud bolts to the shear resistance and the force transfer capacity.

Keywords: concrete-filled steel tube; composite action; bond; connections; load transfer; mechanical shear connectors.

1. Introduction

The increasing use of composite columns and especially concrete filled steel tubular (CFT) columns is due to several advantages attributed to this type of composite column.

Experimental research on CFT columns has been ongoing worldwide for many decades, with significant contributions having been made particularly by researchers in Australia, Europe, Asia and USA. However, the majority of these experiments have been on columns using normal and high-strength concrete, subjected to concentric or eccentric loading and only a few tests were conducted on bond strength in CFT columns. Therefore, a lack of information regarding shear transfer and bond strength of the CFT columns is one barrier to determine the shear resistance of this type of composite element.

The most common method has been utilized to study the bond strength between the concrete core and the steel tube consists in use grease or similar products on the internal surface of the steel tube reducing the steel-concrete bond. Besides, the compressive load can be applied distributed uniformly over the

†Researcher, E-mail: snardin@sc.usp.br
‡Professor, E-mail: analucia@sc.usp.br

concrete and steel tube, either on the steel section or concrete core (Cederwall *et al*. 1990, Johansson and Gylltoft 2002, Giakoumelis and Lam 2004). Considering these three types of load application, the highest load bearing capacity is obtained when the load is applied on the concrete core only and the steel-concrete interface is debonded. If the load is applied on the steel tube only and the steel-concrete interface is debonded, the column behaves as a hollow steel tube column. On the other hand, where the steel-concrete natural bond is preserved, the behavior of the column is independent of the way of load application (Cederwall *et al*. 1990). Therefore, if there is no relative movement between concrete core and steel tube the bond strength has no influence on the structural behavior of the composite column (Johansson and Gylltoft 2002).

Mechanical shear connectors are the most common way to reduce the steel-concrete slip in composite elements. Several different types of shear connection can be used since the natural bond may not be effective to obtain the composite action in composite elements. For example, connection enhancements as embossments, ribs, and shear studs can be used in composite slabs and beams, while such devices are not always added in composite columns. In this last case, mechanical shear connectors are commonly necessary only in the beam-column connection region.

Push-out tests had been an important experimental method utilized to study and understand the contribution of the mechanical shear connectors, mainly headed stud bolts, for the shear strength (Cederwall *et al*. 1990, Li and Cederwall 1996, Shim *et al*. 2004, Zhao and Li 2006). Additionally, push-out tests can be used to examine the behavior of steel–concrete interface if any mechanical shear connectors are used on the interface. In this case, the interface behavior has influence on the relative slip occurs between steel and concrete.

The mechanisms by which shear stresses can be transferred over the interface between the steel tube and the concrete core are important to ensure that the loading is introduced to the composite cross-section in a proper way. This is particularly true for single-story high column and connections of continuous columns (Johansson 2002). Since the forces from the continuous beams can be transferred to the column by direct bearing on head-plates, the load introduction is rarely a problem for the single-story column (Kilpatrick and Rangan 1999, Johansson 2002). However, the steel-concrete strain compatibility is difficult for continuous columns because the beam-column connections are commonly achieved by attaching plates to the outside of the steel tube and the load transfer between materials depends on the ability of the interface to resist shear forces. When the shear resistance is insufficient, the load transfer can be increased extending the loading plate through the steel section so that the concrete core is loaded by direct bearing (Johansson 2002).

In the reinforced concrete structural elements, as it is well-known, the bond is the most important property and it is responsible for the stress transfer and strain compatibility between the steel bar and the concrete. Therefore, in this case, the bond arises when a stress variation occurs in a portion of the steel bar. This variation is caused mainly by the external loads, cracks, variation temperature, shrinkage and creep of the concrete. The steel bars resist tension stress by bond stress and the steel bar-concrete connection controls the cracks opening (Johansson 2003).

In the CFT columns, when the load is applied only to the steel section or only to the concrete section, the axial force in the column must be transferred over the contact surface between the concrete core and the steel tube. Therefore, in these cases, the bond strength interface influences the structural behavior and the load resistance of the CFT column. This fact explains the study of the shear transfer mechanisms in CFT columns and the importance of the mechanical shear connectors because the steel-concrete interface in CFT columns is not yet thoroughly understood.

The present study focuses on the push-out tests behavior of the square concrete-filled steel tubes columns, with the main objective of investigating the shear transfer mechanisms between concrete core and steel tube. Furthermore, the use of angles as mechanical shear connection is an interesting alternative due to the reduced cost and easiness manufacture in Brazil. To understand the structural behavior of the angles and the contribution to the shear transfer can be possible a new use of the angles.

The tests were conducted at the Structural Engineering Laboratory at University of Sao Paulo in Sao Carlos.

2. Composite action on CFT columns

2.1 Steel-concrete shear transfer

The bond stress transfer between the steel tube and the concrete core depends on the radial displacements due to the pressure of the wet concrete on the shell and the shrinkage of the concrete core, together with the rugosity of the interior surface of the steel tube (Roeder *et al*. 1999). However, the radial displacement due to Poisson effect is significant only when the composite action is not achieved, and the strains in the steel and concrete are different.

The bond strength is often studied using push-out tests and *load vs. slip curves* (see Fig. 1). Based on these curves, the behavior of the steel-concrete interface can be schematically divided into three phases: adhesion, micro-interlocking and friction. The first part of the curve presents a stiff behavior that corresponds to an initial bond provided by the concrete-steel connection and is named “adhesion” or “chemical bond”. The peak load of this phase is highly dependent on the surface quality of the steel section (τ_1 in Fig. 1) and corresponds to a small part of the bond strength.

The adhesion is active mainly at the early stage of loading, when the displacements are small and is an elastic brittle shear transfer mechanism (Johansson 2003). Results showed the adhesion contribution to transfer shear stress can be neglected if the adhesion stress is exceeded at a slip value lower than 0.01 mm or the bond stress is higher than 0.1 MPa.

The adhesion strength is exceeded with the increasing shear and a subsequent mechanism depends on the mechanical characteristics at the interface. It is composed of two features: bond steel-concrete mechanical micro-locking and bond that depends on the interface pressure and friction coefficient (Johansson 2003).

The micro-interlocking is caused by the surface roughness of the steel tube and breaks when the concrete interface reaches the compressive crushing strain of the concrete – according to Virdi and Dowling (1980). The micro-interlocking is important especially while the concrete core is close to the steel tube.

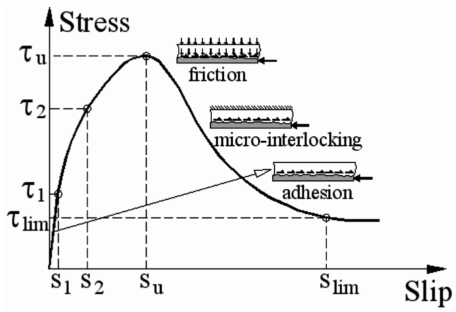


Fig. 1 Stress vs. slip behavior and idealized shear transfer mechanism in steel-concrete interface

The steel-concrete slip can occur increasing the applied load. However, the passive confinement of the concrete core prevents the separation between the steel tube and the concrete core, with the occurrence of some normal stresses that resist the slip. Therefore, the micro-interlocking can be considered a partial mechanism of friction and due to this it is rather difficult to separate the friction and the micro-interlocking portions. Steel-concrete friction stress is developed due to normal stresses and is often referred as "macro-locking". When the s*tress vs. slip* curve is almost horizontal or descending in the post-peak behavior, the bond is destroyed and the steel-concrete slip occurs with insufficient friction.

When the steel-concrete bond in concrete filled steel tube columns is not enough to achieve the required shear resistance, it is necessary to use mechanical shear connectors welded in the inner surface of the steel tube. There is a series of mechanical shear connectors varying in size, shape and methods of attachments, but the headed studs are the most widely used type, especially for columns with a large diameter and where there is necessary of space for the welding process.

Bolts can be attached from the outside by using a flow drilling process for smaller-diameter columns. Another shear connector option to prevent the separation of surfaces is the shot-fired nails, which are shot through the steel tube from the outside (Shakir-Khalil 1993a, 1993b, Johansson 2002).

However, there is no real need of connectors in CFT columns since the tube encases the concrete core and prevents separation. The use of headed connectors embedded in the concrete core permits the longitudinal shear forces to be transferred by dowel action, causing high concentrated stress in the surrounding concrete. The failure mode of the specimens with headed connectors is influenced by the concrete compressive strength. Headed stud bolt connectors and shot-fired nails show high strain capacity and need larger slip so that the mechanisms of natural bond can be activated. This aspect can limit the use of mechanical connectors in CFT columns, as they often present only smaller slips and shear connectors do not work well with the natural bond. Therefore, it is recommended the shear load transfer should be accomplished either by natural bond or entirely by mechanical shear connectors (Roeder *et al*. 1999).

2.2 Composite action and load distribution in CFT columns

Push-out test is the main procedure used to establish the shear load resistance of the steel-concrete interface, but the real bond behavior is still uncertain especially because it is hard to correctly represent

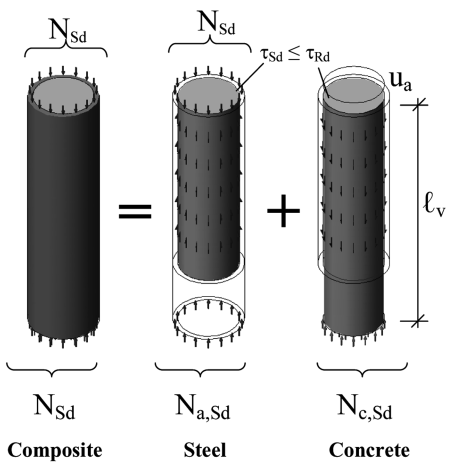


Fig. 2 Shear transfer between steel tube and concrete core (Johansson 2002)

the load introduction in the real CFT columns. Internal forces and moments applied to CFT columns should be distributed between the steel tube and the concrete core according to their response to the imposed deformations (EC 4:2004) – Fig. 2. This implies that the normal force in the ultimate limit state can be calculated based on the plastic resistance of the cross-section components, meaning that the steel contribution ratio δ can be determined for pure axial loads as

$$\delta = \frac{N_{a,Sd}}{N_{Sd}} = \frac{N_{pl,a,Sd}}{N_{pl,Sd}} = \frac{A_a \cdot f_{ya}}{A_a \cdot f_{ya} + A_c \cdot f_c} \tag{1}$$

where $N_{pl,a,Rd}$ is the nominal compressive capacity of the steel tube section, $N_{pl,Rd}$ is the total nominal capacity of the composite section, A_a is the steel tube area, A_c is the concrete core area, f_{ya} is the yield strength of the steel tube and f_c is the compressive strength of the concrete.

The remaining part of the normal force has to be carried by the concrete core ($N_{c,Sd}$):

$$\frac{N_{c,Sd}}{N_{Sd}} = 1 - \delta = 1 - \frac{N_{a,Sd}}{N_{Sd}} \tag{2}$$

In the serviceability limit state, the load distribution depends on the longitudinal stiffness of the cross-section parts. The steel component can be written as

$$\delta = \frac{N_{a,Sd}}{N_{Sd}} = \frac{A_a \cdot E_a}{A_a \cdot E_a + A_c \cdot E_c} \tag{3}$$

and the concrete part is given by

$$\frac{N_{c,Sd}}{N_{Sd}} = 1 - \delta = 1 - \frac{N_{a,Sd}}{N_{Sd}} \tag{4}$$

where E_a and E_c are modulus of elasticity of the steel and the concrete respectively.

In connection regions it can be difficult to ensure that all cross-section components of CFT column are loaded according to their resistance. Therefore, this load has to be distributed considering the interface shear resistance and which shall be provided by either bond stress and friction or mechanical shear connectors. Besides, the transfer length l_v should not be exceeding twice the relevant cross-section dimension. For example, if an external load is introduced only to the steel tube, the components of load given by Eq. (2) and Eq. (4) have to be transferred over the interface to the concrete within the introduction length.

In the absence of a well-established method to calculate the longitudinal shear stress at the steel-concrete interface, the design is often based on the mean shear stress dividing the shear force on the concrete core ($N_{c,Sd}$) by an assumed shear transfer length (see Fig. 2). The interface area is the perimeter of section u_a multiplied by l_v:

$$\tau_{Sd} = \frac{N_{c,Sd}}{u_a \cdot l_v} \tag{5}$$

where τ_{Sd} is the maximum transferable shear stress and the value must not exceed the design bond strength $\tau_{Sd} = 0.40$ MPa. The load transfer has to be achieved by mechanical connectors if τ_{Sd} exceeds τ_{Rd}.

2.3 Important aspects of the bond stress

In the design of CFT columns the full composite action is assumed up to the maximum load resistance, which means that no slip should occur in the steel-concrete interface. A series of experimental tests showed that the absence of bond is insignificant, since the load is applied to the entire cross-section (Cederwall *et al*. 1990). On the other hand, if the load is introduced entirely to either on the steel or the concrete, the longitudinal shear stress at the interface and the bond stress become important.

Some researchers have studied the importance of the internal roughness of the steel tube and the results have showed that the roughness has an influence on the bond resistance because it influences the shear flow between steel and concrete (Virdi and Dowling 1980, Shakir Khalil 1993a, 1993b). The bond resistance is a function of the micro and macrolocking between steel and concrete elements. The microlocking depends on the surface roughness and its importance is higher in the first stages of loading, contributing to a typical initial stiff part of the load-slip relationship. This portion of bond breaks when the concrete interface attains a local strain of approximately 3.5‰ (Virdi and Dowling 1980). When cyclic load is applied to the entire cross-section, the bond stress is not important but, if the load is applied only to the steel tube, the internal roughness of the steel tube is very important to the load transfer (Yoshioka 1992).

The influence of the cross-section shape is more visible when the load is very close to the failure load and the circular shape has lower influence on the bond stress (Virdi and Dowling 1980). Test results clearly showed that circular sections are much more effective than rectangular sections in resisting push out force (Shakir Khalil 1993a). This probably occurs because the circular shapes resist evenly at the slip while rectangular shapes present higher resistance in the corners of the cross-section.

According to Virdi and Dowling 1980, the diameter/thickness ratio (or wall slenderness) of the circular cross-section had no influence on the bond strength. However, test results showed that the bond strength increases with smaller wall slenderness of the rectangular cross-section (Parsley *et al*. 2000).

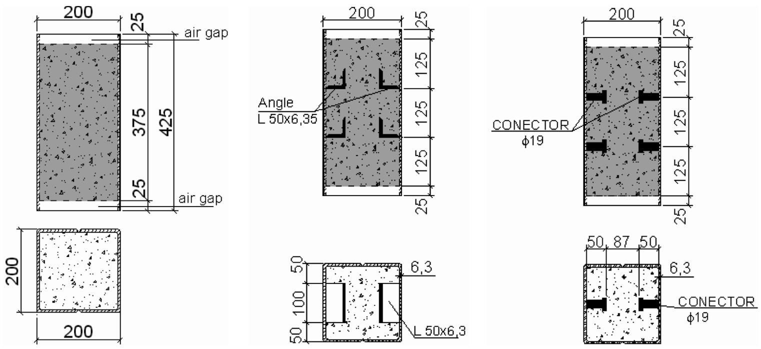
3. Experimental study

Three specimens of CFT columns have been carried out in the laboratory at the Structures Engineering Department, Sao Paulo University. To limit the scope of the tests, the influence of the mechanical shear connectors was the only variable considered. Therefore, the experimental program made it possible to evaluate the effect of the mechanical connectors on both the load-slip response and the distribution of axial load to the cross-section components. Besides, the use of angles as mechanical shear connector is an important aspect of this experimental study.

A total of three specimens of CFT columns was tested until failure in a type of push out test and a summary of the dimensions and material properties is presented in Table 1. All specimens were square

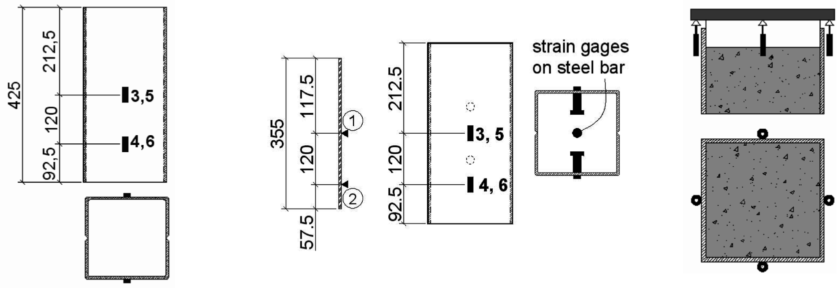
Table 1 Properties of tested specimens

Specimen	Section (mm)	Length (mm)	Concrete (kN/cm^2)	fy (kN/cm^2)	Mechanical connector
CFT-S CFT-SB CFT-A	Square 200 × 200 × 6.3	425	f_c = 4.8 E_c = 3325	25.33	Without Stud bolts Angles
Angle	L 50 × 6.3	100	-	45.40	-
Stud bolt	19 mm diameter	50	-	f_u = 41.5	-



(a) CFT-S (b) CFT-A (c) CFT-SB

Fig. 3 Details of the tested specimens



(a) On the external steel tube (b) In the concrete core (c) Transducers

Fig. 4 Strain gages and displacement transducers arrangements

CFT with the same length-to-width ratio and the inside surfaces of the steel tube were not treated – Fig. 3. Two of the specimens had mechanical shear connectors welded to the inner steel surface. The CFT-SB and CFT-A specimens had two headed bolts and two angles welded in two opposite faces, respectively. The CFT-S specimen did not have any mechanical shear connectors. The connectors spacing was chosen similar that used in steel-concrete composite beams.

All specimens were instrumented with six electrical strain gages and four linear displacement transducers (Fig. 4). The transducers were attached to the steel tube near the concrete-loaded end of the specimen and measured the relative displacement between the upper crosshead and the top of the steel tube (Fig. 4c). A steel bar was instrumented with two strain gages, placed at the centroid of the concrete core of the specimens and measured the longitudinal strains of the concrete (Fig. 4b). As shown in Fig. 4(a), the strain gages were also placed on the outer surface of the steel tube in the same instrumented planes of the concrete core.

The load was applied by the spherical crosshead of the testing machine and a steel plate was placed on the portion of the concrete core whose length is shorter than the steel tube. This test setup was designed to develop shear stress between the steel and the concrete in the specimens; therefore load was applied only on the concrete core and this component of the composite section can slip in relation to the steel tube (Fig. 5). At the base, the load was only resisted by the steel tube because the concrete core was recessed before the bottom end. Therefore, only the steel tube rest on the steel base of the testing machine.

Fig. 5 Test arrangement

The vertical force on the concrete core was introduced by a computer controlled hydraulic actuator with 3000 kN capacity and the speed of load application was 0.005 mm/second in all specimens.

4. Test results

The main results of push out tests were organized in the form of a load-slip response, strains in concrete core and steel tube, and axial load distribution to the steel tube and the concrete. The detailed test results for each specimen and comparison between main results are also presented.

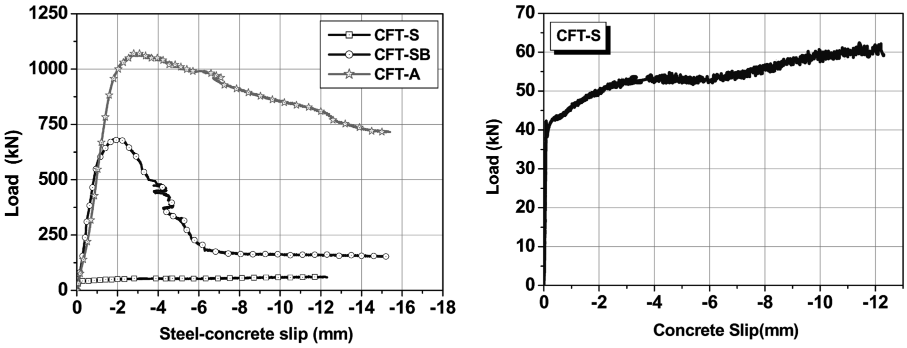
4.1 Maximum load

The maximum load resistance and concrete slip values are shown in Table 2. These values evidence the contribution of the headed stud bolts (CFT-SB) and angles (CFT-A) restraining the slip of the concrete core.

The load-slip curves (Fig. 6) show that stud bolts and angles used as mechanical connectors were very effective in transferring the load between the concrete core and the steel hollow section. Concerning the addition of the four stud bolts with 19 mm diameter, 50 mm length and 125 mm distance, it seems that the maximum load increases about 11 times in relation to the specimen without mechanical connectors (CFT-S). Besides, these stud bolts decrease the slip at the maximum load approximately 4 times the maximum load of the specimen without mechanical connectors (Table 2 and Fig. 6). Four angles fixed in the steel section along two opposite sides contribute increasing the maximum load in relation to the other specimens. The slip at the maximum load decreases 3 times in comparison with the specimen without mechanical connectors. Therefore, stud bolts or angles are very efficient to increase the capacity of the steel-concrete interface to resist the separation.

Table 2 Results of push out tests

Specimen	Maximum load resistance (kN)	Slip (mm)
CFT-S	63.1	11.48
CFT-SB	684.5	2.65
CFT-A	1071.9	3.66



(a) All specimens tested (b) CFT-S Specimen

Fig. 6 Load vs. concrete core slip

4.2 Load-slip diagrams and concrete-steel separation

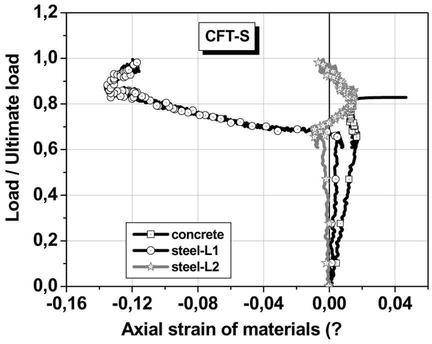
Fig. 6 shows the load applied-slip relationship for all specimens tested. In the early stages of loading, the stiffness shown by the curves was high and had the same order of magnitude of the specimens with mechanical connectors, although the specimen with angles had lower stiffness than CFT-SB specimen. The load applied-slip relationship to the CFT-S (specimen without mechanical connectors) evidences the break of the adhesion at about 42 kN and this rupture is characterized by the concrete slip showed in Fig. 6(b). The rupture of the adhesion was not clearly identified for the specimens with mechanical connectors (CFT-SB and CFT-A in Fig. 6a). In CFT-S specimen, as soon as the adhesion had been lost, the friction began to work and in this phase, it is predominant in the load transfer mechanism.

It can also be seen that the presence of the mechanical connectors contributes to increasing the maximum load and changes the load applied-slip relationship for both, ascending and descending branches. The ascending behavior is very similar for two specimens with mechanical connectors but the descending behavior was very different.

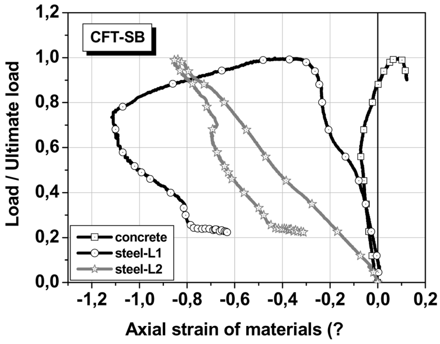
While CFT-SB specimen presented a sudden load decrease after the peak load, the descending behavior of the CFT-A specimen was characterized by a gradual decrease of the load and increase of the slip, characterizing a residual bond strength that corresponds to 150 kN. For the specimen without mechanical connectors, the displacements increased after the break of the adhesion and continued increasing at constant load. Then, when the CFT-SB specimen reached the residual bond strength, the slip increased gradually without reduction of the applied load. For CFT-A specimen, the residual bond strength was not clearly identified in the load-slip relationship. The behavior of the CFT-SB specimen presented some abrupt variation in the post-peak branch, compared to the CFT-A specimen. In this specimen, the observed post-peak behavior can be explained by two reasons: the fact that the angles are stiffer than the studs and the higher bearing area of the angles. In fact, these two reasons are related, because the stiffness of the angles is not only its own stiffness, but it also counts with the contribution of the “encased” concrete. Therefore, angle and concrete working together as one resistant mechanism, the whole region becomes stiffer. In this way, the contribution of the steel tube can also be mobilized, as observed in the failure mode of the specimen CFT-A.

4.3 Strain on steel tube and concrete core

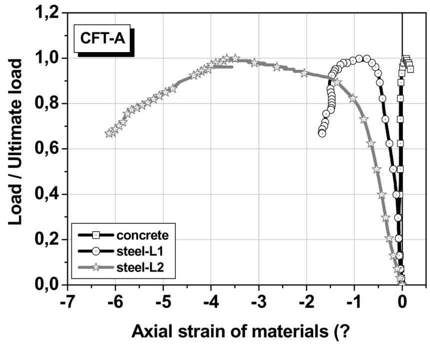
Fig. 7 shows the axial strains of the steel tube and concrete core. The measurements were taken at



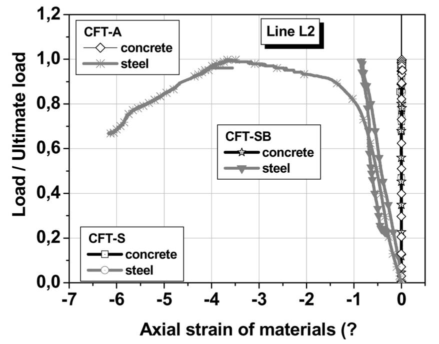
(a) Specimen CFT-S



(b) Specimen CFT-SB



(c) Specimen CFT-A



(d) All tested specimens

Fig. 7 Strains on steel tube and concrete core

212.5 mm and 332.5 mm from the end of the specimen, where the concrete core was loaded. From the readings of the axial strain it is possible to evaluate the transfer of forces between the concrete core and the steel tube. At initial loading stages, the strain of the concrete core of CFT-S specimen is larger than the steel tube (Fig. 7a). At this stage, up to 42 kN of applied load, the adhesion part of the bond had been not broken yet (Fig. 6b). Once the adhesion is exceeded, considerably larger strains start to appear on the steel tube, indicating that the remaining transfer mechanisms of friction and mechanical bond are being mobilizedFig. 7 Load-strains relationship of the steel tube and concrete core (Fig. 7a).

From Fig. 7(b), it is possible to observe the stud-bolts contribution to the shear force transfer (model CFT-SB). As the first strain-gage is placed below the first connector, at 212.5 mm from the loaded end, the axial strains in the concrete and the steel tube indicate the contribution of the connectors since the earlier stages of loading. At that position, called L1, the strains in the concrete core and in the steel tube are similar with the load has reached 45% of the maximum value. For higher values, the strain in the steel tube increases substantially (Fig. 7b).

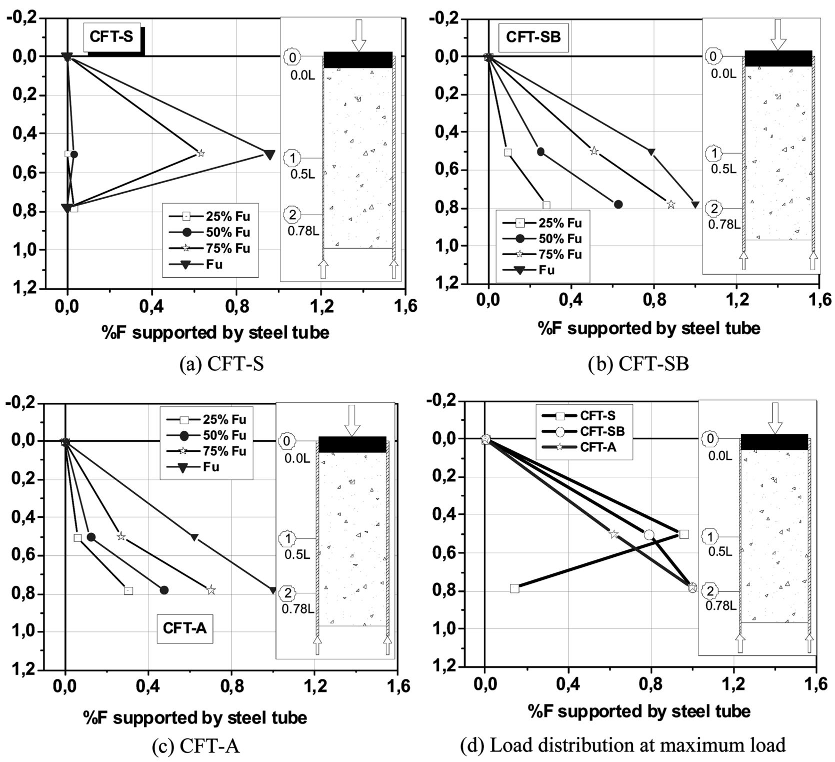
At level L2, 332.5 mm distant from the loaded end, the strain in the steel tube is much larger than the ones registered in the concrete core, since the beginning of the loading process. The variation of the strains can be a sign that some transfer of the applied load occurs. Comparing the strains measured in specimens CFT-S and CFT-SB, it is possible to observe the efficiency of the shear connectors, since the corresponding readings in the model that does not have connectors are very small, showing almost no transfer of forces. The angles contribution to the transfer mechanism can be observed in Fig. 7c, specimen CFT-A. As in the specimen without connectors, at line L1 the steel tube and the concrete core presented similar strain, until 40% of the maximum load. At line L2, a significant strain was observed in the steel tube, meaning the force transfer occurs since the loading process starts.

When comparing the strain readings for the three tested specimens (Fig. 7d), one can observe that in the specimen with angles (CFT-A), the steel tube presented much larger strains than the other specimens. Therefore, the angles showed to be a good alternative to promote the shear transfer from the concrete core to the steel tube, for square filled columns.

4.4 Load distribution

The distribution of load along the length of the specimens at applied loads of 0.25, 0.50, 0.75 and maximum load is shown in Fig. 8. At the upper end of the steel tube, the load is applied to the concrete core and at the bottom end (1.0 L) the load is applied only to the steel tube, as showed in Fig. 4 and Fig. 5. This load distribution at upper and bottom ends is due to the load arrangement adopted on the test setup, where the load is applied only on the concrete core at the upper end and the bottom end of the steel tube rest on the base of the testing machine.

The load distribution was obtained using the load applied to the concrete core and the strain measured by the strain gages. The presence of the mechanical shear connectors changes the load distribution along the length of the specimens (Fig. 8). If any mechanical shear connectors were used in the specimen (see CFT-S in Fig. 8a), the load transfer from the concrete to the steel tube would occurred only when the load applied reached the load that breaks the adhesion.



(a) CFT-S
(b) CFT-SB
(c) CFT-A
(d) Load distribution at maximum load

Fig. 8 Load distribution along length of CFT specimens

The shear connectors (specimen CFT-SB – Fig. 8b) significantly improve the force transfer and, below the first line of connectors, about 80 % of the load applied to concrete core is transferred to the steel tube. For line L2, at 0.78L, the strains measured in the steel tube indicate the load resisted by the steel tube (F_s) exceeds the applied load. The reason for that is the deformation of the tube close to the next connector, where the strain-gage is placed. The specimen with angles (specimen CFT-A – Fig. 8c) showed this behavior in the tests even more clearly.

A general overview of the forces distribution along the length of the tested specimens for the maximum applied load is presented in Fig. 8(d), where the contribution of the connectors and angles is also showed. It can be seen in Fig. 8(d) the influence of the mechanical connectors on the load distribution between the concrete core and the steel tube.

4.5 Failure modes

The final configuration of the tested specimens is shown in Fig. 9. The failure mode of the CFT-S specimen occurred by a slip between the steel tube and the concrete core of about 11.5 mm at the maximum load. This slip could be observed at the top of the specimen (Fig. 9a). The CFT-SB specimen presented a smaller slip than CFT-S specimen, which was also observed on the top. Besides, the stud bolt did not fail, but they rotated deforming the wall of the steel tube. As seen in Fig. 9(c), the CFT-A specimen also showed visible deformations due to the rotation of the angles and deformed the tube wall where they were welded. Probably, the angles did not fail due to the steel-concrete slip.

4.6 Bond strength

The peak bond strength of each of the three specimens was calculated dividing the maximum load (Table 2) by the contact area and considering the length of the steel-concrete interface was equal to 375 mm. The results are shown in Table 3.

The peak bond strength value of the specimen without mechanical connectors was 0.22 MPa, but the maximum transferable shear stress, t_{Sd}, must not exceed the design bond strength t_{Rd} = 0.4 MPa. When the action shear stress exceeds the admissible value, the load transfer has to be achieved by shear mechanical connectors.

By adding shear mechanical connectors, higher values of peak bond strength were obtained, as the load applied on concrete core was transferred from the concrete to the steel tube through the stud bolts and angles welded in the internal surface of the tubes. If the value corresponding to the CFT-S specimen

(a) CFT-S (b) CFT-SB (c) CFT-A

Fig. 9 Final configuration of the tested specimens

Table 3 Peak bond strengths tests and shear strength capacity of the specimens

Specimen	u_a (cm)	$F_{c,\max}$ (kN)	τ_u (MPa)	EC4 (kN)	Brazilian Code (kN)
CFT-S		63.1	0.22	-	-
CFT-SB	74.96	684.5	2.44	376.4	470.8
CFT-A		1071.9	3.81	-	716.4

u_a: perimeter of the steel-concrete interface
The values correspondents to EC4 and Brazilian Code are shear resistance design values

(without mechanical connectors) was removed from the total value of the peak bond strength of the other specimens, it would be possible to evaluate the contribution of the stud bolts and angles. Thereby, the increases in the peak bond strength attributed to each stud bolt and angle are 0.55 MPa and 0.90 MPa to specimens CFT-SB and CFT-A, respectively, meaning that stud bolts and angles were very effective to increase the load transfer between the concrete and the steel tube.

4.7 Shear transfer mechanisms

The experimental program was developed to study the contribution of the stud bolts and angles to transfer shear forces. Results have confirmed that the mechanisms of force transfer along the concrete-steel interface are adhesion, interface interlocking and friction. As shown in Fig. 6(b), the adhesion contributed to shear transfer only during the initial loading and was lost as soon as the concrete core had slipped in relation to the steel tube. The adhesion was clearly observed only in the CFT-S specimen. The descending branch behavior indicated the contribution of the mechanical connectors to interface interlocking and friction mechanisms, but is very hard to separate.

The real contribution of the mechanical connectors to the maximum load was taken as a sum of the steel-concrete bond strength and the loading carrying capacity of the mechanical connectors. Besides, the shear resistance of the headed stud was predicted using EC4 and the Brazilian code (NBR 8800:2006) recommendations. The experimental and predicted values are shown in Table 3 and the partial safety factors were adopted as being equal to 1.0.

All values of the shear strength capacity predicted by standard codes are lower than the values experimentally obtained from the two tests carried out in specimens with mechanical connectors.

5. Conclusions

The main objective of the experimental study was to evaluate the contribution of the headed stud bolts and angles to the mechanism of the shear forces transfer between the concrete core and the steel tube of the concrete-filled steel tubes columns. The test results indicated that the mechanical shear connectors tested were very efficient to decrease the concrete slip and increase the maximum load capacity. Based on some experimental results, the angles were more efficient than the stud bolts increasing the load capacity and reducing the slip at maximum load applied. Probably, the greater stiffness of the angle in relation to the stud bolt is the responsible by the higher shear force transferred in the Specimen CFT-A.

In addition, in the specimen without mechanical shear connectors, it was observed that as soon as the maximum load had been reached a concrete-steel separation occurred. Using mechanical shear connectors the relative slip was also recorded, but it was much smaller in these specimens. Besides, specimens with mechanical shear connectors presented residual strength in the post-peak branch.

Finally, however these conclusions are based on limited experimental results, the obtained results are in agreement with those of other researchers.

Acknowledgements

The authors would like to acknowledge to Fapesp (Research Support Foundation of Sao Paulo State) for the financial support given to this research.

References

Cederwall, K. and Engstrom, B. and Grauers, M. (1990), "High-strength concrete used in composite columns", *Proceedings of High-strength Concrete: Second International Symposium*, Detroit-USA, 195-214 (ACI SP-121).

European Committee for Standardization (ECS). (2004), "Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings", Brussels, EN 1994-1-1:2004, 122p.

Giakoumelis, G. and Lam, D. (2004), "Axial capacity of circular concrete-filled tube columns", *J. Constr. Steel Res.*, **60**(7), 1049-1068.

Jeong, Y.-Ju and Kim, H.-Y. and Kim, S.-H. (2005), "Partial-interaction analysis with push-out tests", *J. Constr. Steel Res.*, **61**(9), 1318-1331.

Johansson, M. (2002), "Composite action and confinement effects in tubular steel-concrete columns", Doctoral Thesis, Department of Structural Engineering, Chalmers University of Technology, Sweden, 204p.

Johansson, M. and Kent Gylltoft (2002), "Mechanical behavior of circular steel–concrete composite stub columns", *J. Struct. Eng.*, **128**(8), 1073-1081.

Johansson, M. (2003). "Composite action in connection regions of concrete-filled steel tube columns", *Steel. Compos. Struct.*, **3**(1).

Kilpatrick, E. and Rangan, B. V. (1999), "Influence of interfacial shear transfer on behavior of concrete-filled steel tubular columns", *ACI Struct. J.*, **96**(4), 642-647.

Li, A. and Cederwall, K. (1996), "Push-out tests on studs in high strength and normal strength concrete", *J. Construct. Steel Res.*, **36**(1), 15-29.

Parsley, M. A. and Yura, J. A. and Jirsa, J. O. (2000), "Push-out behavior of rectangular concrete-filled steel tubes", *Composite and Hybrid Systems*, Riyad S. Aboutaha and Joseph M. Bracci ed., ACI., 87-107 (ACI SP-196).

Roeder, C. W. and Cameron, C. and Brown, C. B. (1999), "Composite action in concrete filled tubes", *J. Struct. Eng.*, **125**(5), 477-484.

Shakir-Khalil, H. (1993a), "Pushout strength of concrete-filled steel hollow sections", *The Struct. Eng.*, **71**(31), 230-233.

Shakir-Khalil, H. (1993b), "Resistance of concrete-filled steel tubes to pushout forces", *The Struct. Engineer*, **71**(13), 234-243.

Shim, C.-S, Lee, P.-G. and Yoon, T.-Y. (2004), "Static behavior of large stud shear connectors", *Eng. Struct.*, **26**(12), 1853-1860.

Virdi, K. S. and Dowling, P. J. (1980), "Bond strength in concrete filled steel tubes", *Proceedings of IABSE-International Association for Bridge and Struct, Eng.*, August, **3**, 125-137.

Yoshioka, Y. (1992), "State of art of composite steel tube and concrete structures in Japan", *Proceedings of Japan Workshop on Composite and Hybrid Structures*, Berkeley, 119-130.

Zhao, G. and Li, Y. (2006), "Bond capacity of steel reinforced concrete composite short columns", *Proceedings of 2nd International Fib Congress*, June, Session 5 – Composite and hybrid structures, Naples-Italy, ID5-7.

CC