Volume 11, Number 5 (October 2018) p. 960 – 965 • ISSN 1983-4195
http://dx.doi.org/10.1590/S1983-41952018000500004

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REVISTA IBRACON DE ESTRUTURAS E MATERIAIS
IBRACON STRUCTURES AND MATERIALS JOURNAL

Crestbond shear connectors for load transfer in concrete filled tube columns

Conectores Crestbond para transferência de carga em pilares mistos preenchidos com concreto





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Abstract

This paper presents an experimental study with numerical modeling of Crestbond shear connectors in concrete filled tube columns. The Crestbond, which consists in a steel plate with regular cuttings, was originally conceived for composite beams and is now being proposed as an alternative device for load introduction and shear transfer at the steel-concrete interface in concrete filled tube columns. The results achieved in this work were very favorable to the new application proposed for the connector as high values of shear strength were obtained. Moreover, the numerical and experimental results enabled comparative analysis and investigations regarding the influence of concrete conditions and the geometry of the column section on the mechanical properties of the connector.

Keywords: crestbond shear connectors, concrete filled tube columns, push test, FEM modeling.

Resumo

Esse trabalho apresenta um estudo experimental com modelagem numérica de conectores Crestbond aplicados em pilares mistos preenchidos com concreto. O Crestbond, caracterizado por chapa de aço com recortes regulares trapezoidais e originalmente concebido para vigas mistas, é proposto como um elemento para introdução de carga e transferência de cisalhamento na interface aço-concreto de pilares mistos preenchidos com concreto. Os resultados alcançados nesse trabalho foram favoráveis à nova aplicação proposta para o conector, uma vez que foram obtidos valores elevados de resistência ao cisalhamento. Além disso, os resultados numéricos e experimentais permitiram análises comparativas e investigações acerca da influência das condições do concreto e da geometria da seção do pilar nas propriedades mecânicas do conector.

Palavras-chave: conectores de cisalhamento Crestbond, pilares mistos preenchidos com concreto, ensaio de cisalhamento, modelo em elementos finitos.

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Received: 19 Sep 2017 • Accepted: 16 Dec 2017 • Available Online: 28 Sep 2018

1. Introduction

The connectors currently found in the literature for transferring the longitudinal shear stress in Concrete Filled Tube Columns (CFTCs), such as headed studs, plates, bolts and internal ring stiffeners are not always practical or effective. These devices often show problems like rebar interference, complex installation procedure and low stiffness.
Considering the great structural efficiency of CFTCs, particularly those of circular cross section which provide greater confinement to the concrete core, and, on the other hand, the need for more effective and practical solutions for the execution of connections and shear transfer in these columns, CALDAS[1] proposed the arrangement shown in Figure 1. In this arrangement the Crestbond connectors perform the shear transfer at the steel-concrete interface while integrate the connection between beams and column. In addition, they allow free passage of concrete rebars.

2. Push tests

To study the Crestbond connector behavior when applied in CFTCs it was proposed a push test with a specific setting to reproduce the conditions observed in this sort of column. The specimens consisted of a steel tube filled with concrete up to 50 mm below the top and with Crestbond connectors positioned at diametrically opposed longitudinal openings at a given height. The connectors used had three steel dowels; an amount compatible with most practical applications considering the usual dimensions of beams and columns (Figure 1).
By applying load on the top of steel tube and supporting only the concrete at the bottom, it was assured that the entire load applied to the tube got transferred to the concrete through the connectors. Therefore, the determination of the mechanical properties of the Crestbond connectors in CFTCs was made by measuring the slip between steel and concrete using displacement transducers.
Three series of tests were conducted in laboratory namely; series B, series P and series U. Each of these series had different column cross section geometries. The purpose was to observe how the shape and diameter of the column section influences the connector's behavior. The connectors' steel dowels were named D1, D2 and D3 from bottom to top of the specimen. Each of these three series was composed by two identical specimens with geometries shown in Figure 2.
As for the mechanical properties of the materials that constitute the specimens, the average concrete strength (f_{cm}), average steel yield strength (f_y) and average steel rupture strength (f_u) were obtained by cylinder compression and coupon tensile tests. Tensile concrete strength (f_{ctm}), modulus of elasticity (E_{cm}; E_a) and poisson ratio (ν) were defined according to EN 19921-1:2004 [2], as shown in Table 1.

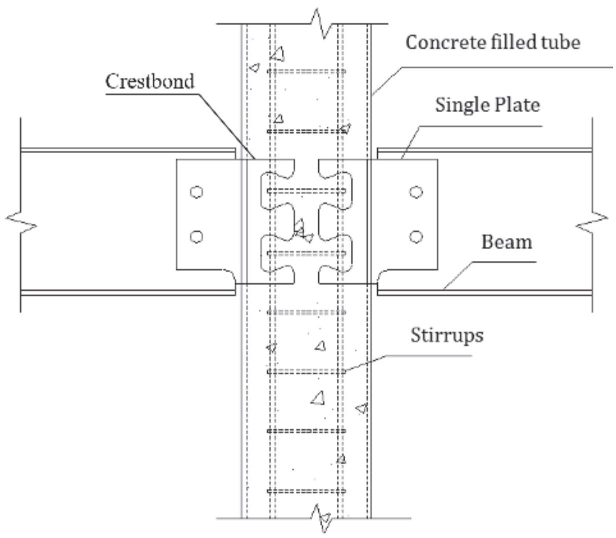

Figure 1
Proposed use of the Crestbond shear connector in concrete filled tube columns (CALDAS[1])

3. Numerical models

Using the software ABAQUS [3], a finite element model was done for each of the specimens. The models were constructed with a discrete mesh of eight node brick elements (C3D8) for both steel and concrete. The approximate size of the elements edges in the Crestbond and surroundings, where there was greater stress gradient, was 8 mm. For elements located farther from this region, their longitudinal dimensions were gradually increased to 30 mm at the top of the concrete and 50 mm at the bottom, thus reducing computing time. Only a quarter of the specimens were modeled due to double symmetry. Appropriate boundary conditions were applied in the symmetry planes. The models of the series B, P and U had 9445, 23085, and 16989 elements respectively.
Regarding the load, a concentrated force was applied to a reference point located in the geometric center of the cross section at the top of the tube. This point was connected to

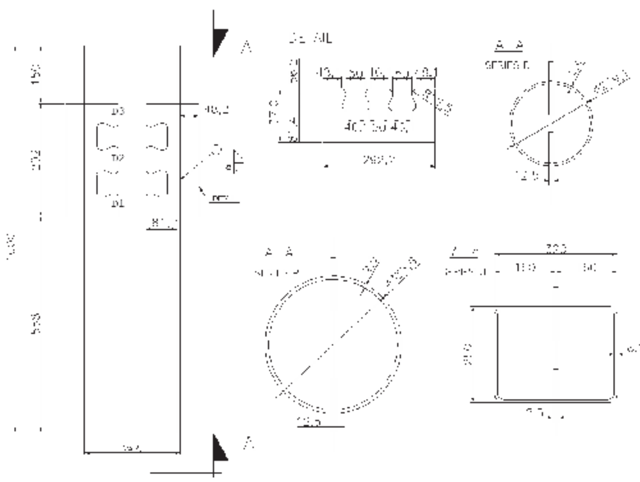

Figure 2
Column geometry and cross section (sizes in mm)

Table 1
Material properties

	Steel				Concrete	
	Tube			Crestbond		
	Series B	Series P	Series U			
f_y (MPa)	415	372	363	390	f_{cm} (MPa)	42
f_u (MPa)	570	550	467	508	f_{ctm} (MPa)*	3.14
E_a (MPa)	205000				E_{cm} (MPa)**	33837
ν	0.3				ν	0.2

* $f_{ctm} = 0.3(f_{cm} - 8)^{2/3}$, ** $E_{cm} = 22\left(\frac{f_{cm}}{10}\right)^{0.3}$ (EN 1992-1-1:2004 [2])

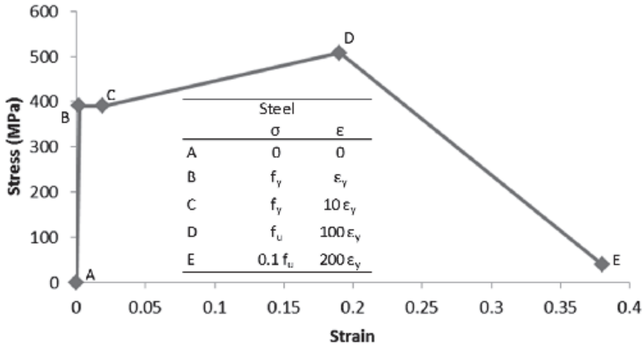

Figure 3
Stress-strain curve for steel (AGUIAR [4])

the nodes of the top surface of the tube through a rigid body constraint. This force was gradually increased by Riks incremental process.

As to the contacts between the parts of the model, it was applied a tie constraint between connector and tube and a "hard" surface-to-surface contact between the concrete and the steel parts, which allows minimum penetration among the surfaces and have no restraining for their separation, with a 0.5 friction coefficient between concrete and connector and zero friction between concrete and tube, as the tube was lubricated in the test.

A classic elastoplastic model was used to simulate the steel, and the concrete damaged plasticity model was used to simulate the concrete. In both cases, the material behavior is defined by providing ordered pairs which characterize the stress-strain relations, as shown in Figure 3 for the steel and in Figure 4 for the concrete.

4. Results

With the experimental and numerical results, an effort of analyzing and comparing them was done to verify the validity and accuracy of the numerical models and understand the Crestbond behavior in CFTCs.

Observing the load-slip curves obtained in the push tests (shown in Figure 5), the behavior of the connectors was characterized by an initial ascending linear phase, followed by a curved stretch with decreasing stiffness that ends with a plateau. Therefore, two resistance parameters were defined for the connectors: load P_u, which is the maximum strength and corresponds to the plateau level, and load P_1, which corresponds to the load value in which the connectors no longer display linear behavior. The average values obtained in each series for P_1 and P_u are shown in Table 2.

The overlapping of the curves shows that load P_1 varied very little between the three series, while load P_u presented values quite different. This suggests that geometric characteristics of the column section have a small influence on load P_1 and should be more strongly linked to properties of the connector itself. Load P_u, on the other hand, is more related to factors such as diameter and shape of the tube.

Regarding load P_u's behavior, it is likely to be related to the degree of confinement of the concrete, since it was precisely

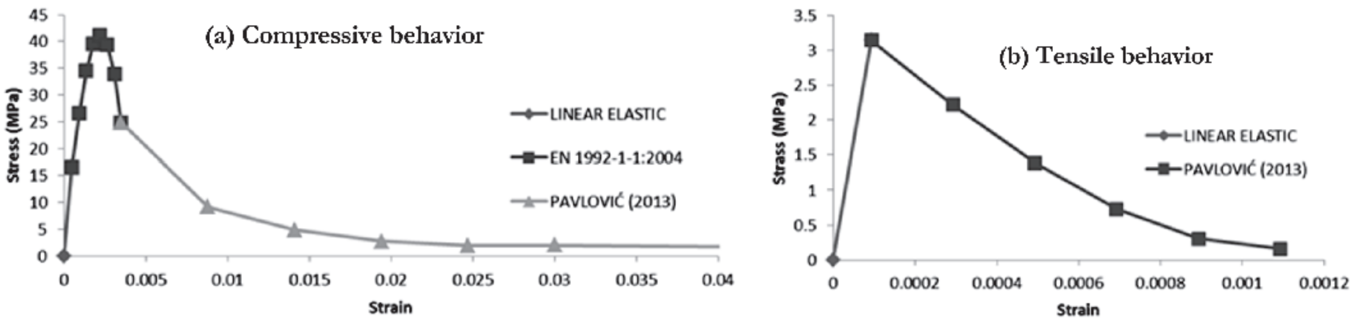

Figure 4
Stress-strain curves for concrete (AGUIAR [4])

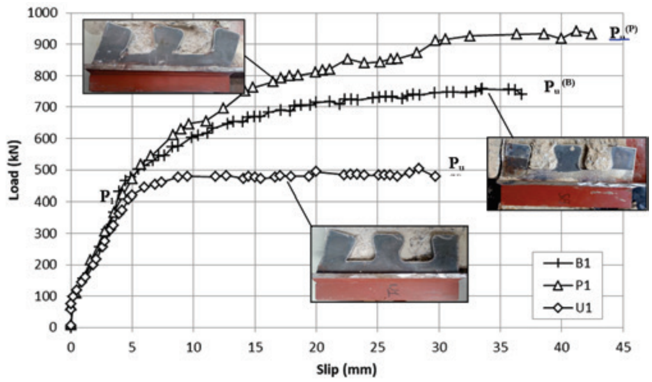

Figure 5
Overlapped load-slip curves of the three series of experimental tests

the series U, whose tubes have rectangular section (which do not provide good confinement to concrete), that differed most significantly from the others, having described a plateau shortly after the end of the initial linear phase.

As to the difference of P_u values between the series B and P, it may be related to the effect of confinement of a locally loaded concrete region by the concrete that surrounds it.

According to ABNT NBR 6118: 2014 [5], the confinement of a locally loaded concrete region by surrounding concrete causes a strength increase that is proportional to the ratio between the total concrete cross sectional area and the loaded area. Being this a case of locally loaded concrete, since all the load is only applied to the contact area between connectors and concrete, it was expected that series P would resist a greater load, since it has greater concrete section diameter, thus, more confining mass.

Table 2
Loads P_u and P_1

		Experimental (kN)	Numerical (kN)	Error
Series B	P_u	760	758	0.30%
	P_1	460	448	2.60%
Series P	P_u	940	883	6.10%
	P_1	500	468	6.40%
Series U	P_u	550	539	2.00%
	P_1	400	382	4.50%

Comparing the experimental test loads with the loads of the numerical models, a good approximation of the strength parameters was observed, with maximum error of 6.4%, which validates the numerical models. Table 2 shows the accuracy of the numerical models in the determination of strength parameters.

Analyzing the spread of yielding on the Crestbond increment by increment, it was observed that the reaching of P_1, *i.e.*, of the end of the linear phase, coincides with the total yielding of dowel D1's lower section. When this happens, this dowel stops resisting new load increments and starts deforming more sharply, which causes the slip to occur at an increasing rate, thus causing the load-slip curve to lean until it finally develops a plateau. Therefore, in the numerical models load P_1 was taken as the load value at the first load increment to present D1's lower section completely yielded (Figure 6).

Widespread yielding and large deformations in the Crestbond can be observed at the plateau level as load approaches P_u. Therefore, with all dowel considerably deformed and near their strength limits, the load resisting interface area becomes increasingly smaller, resulting in high stress concentrations and crushing of the concrete (Figure 7), which leads to even larger displacements.

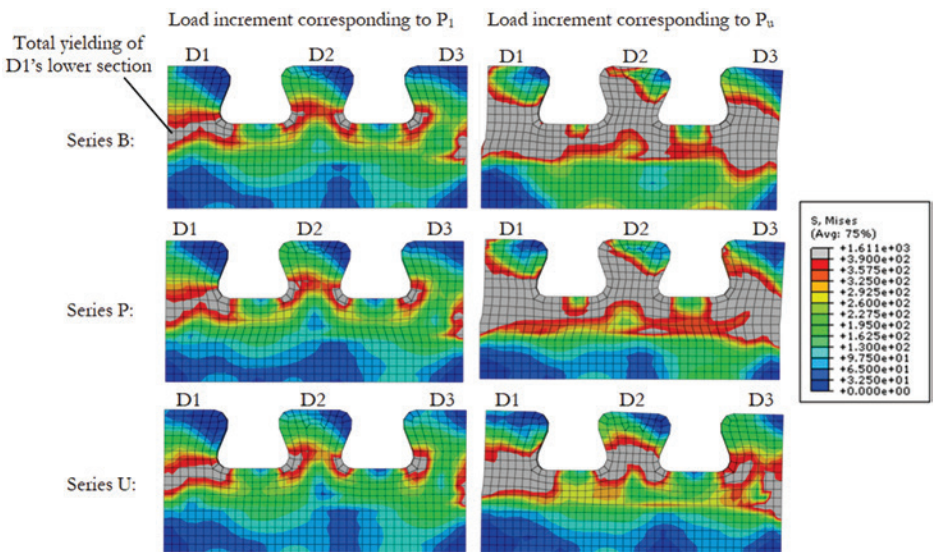

Figure 6
Distribution of von Mises stresses on the connectors of the three series for increments corresponding to loads P_1 and P_u (yielded regions in grey)

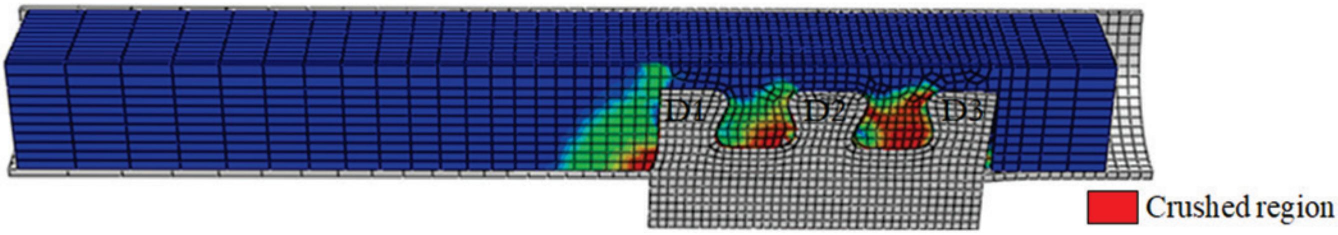

Figure 7
Compression damage in the concrete and final deformed state of series B numerical model

5. Stiffness of the connector

While the Crestbond's numerical strength parameters (loads P_1 and P_u) showed maximum error of 6.4% in regard to experimental values, the stiffness value of the connector showed, at first, large discrepancy between experimental and numerical models.
After investigating the problem it was found that the discrepancy was due to certain conditions in the concrete core of the specimens that were facilitating the slip and consequently causing very low stiffness values. These conditions were gaps, as shown in Figure 8.
The gaps probably resulted from concrete's plastic settlement, which according to NEWMAN [6], tend to cause voids under obstructions within the formwork, and may have been aggravated due to poor consolidation. That caused reduction of the contact interface between connector and concrete, thus generating higher

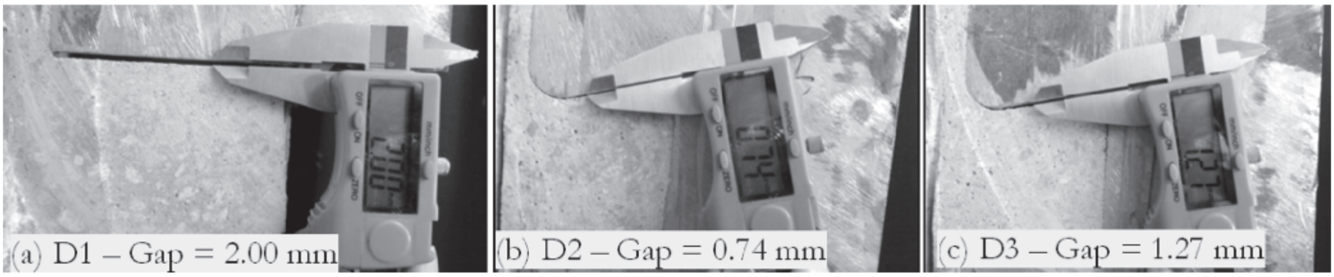

Figure 8
Settlement gaps between concrete and connector in an untested specimen

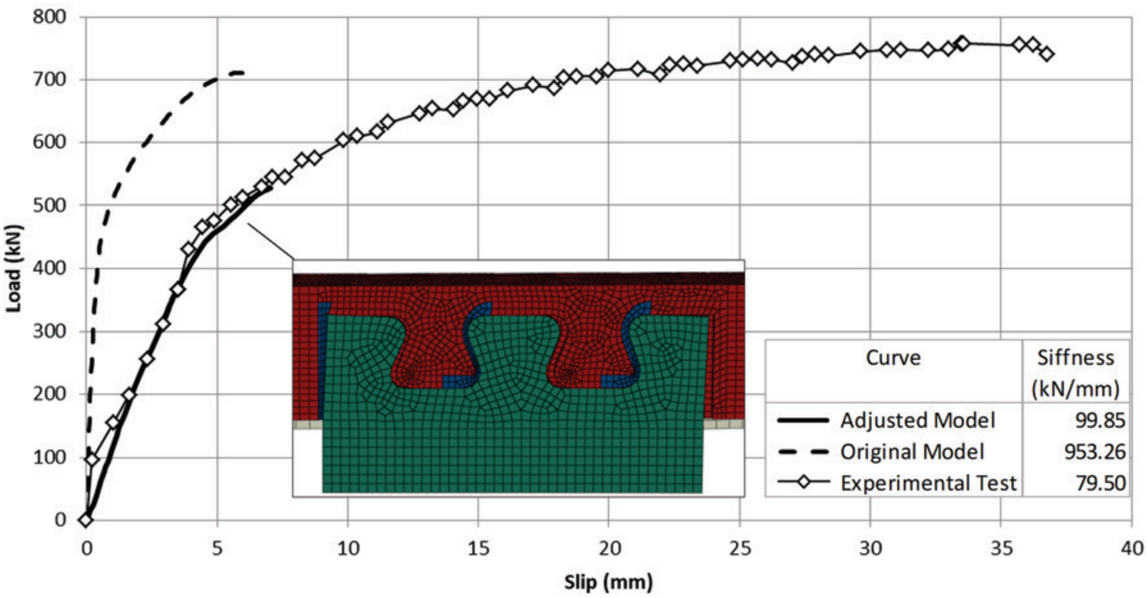

Figure 9
Matching of FEM model and test's load-slip curves through the adjustment of the modulus of elasticity of a set of elements in the concrete (elements shown in blue)

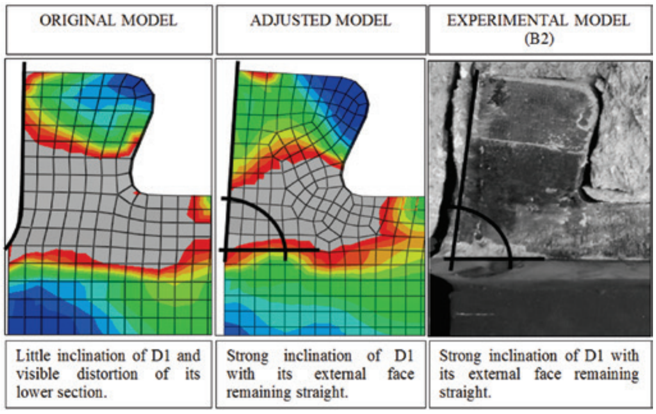

Figure 10
Deformed configuration of D1 for the different models

stress concentration, accelerating the concrete crushing at the contact points and leading to greater displacements, especially at initial load.
Seeking to simulate the conditions observed in the concrete in order to match the numerical and experimental results and therefore make sure that the concrete irregularities were indeed responsible for the discrepancy, an attempt was made to reproduce the irregularities of the concrete at the interfaces with the Crestbond by inserting highly flexible material bands (shown in blue color in Figure 9) in the numerical models. The modulus of elasticity of that material was adjusted in order to match the load-slip curves of the numerical and experimental models.
Analyzing the results of these models it was noted an agreement of the curves (Figure 9), with the stiffness difference lowering from 1099% in the first model to 25% in the adjusted model, and a deformed configuration similar to observed in tests (Figure 10). Thus indicating that the simulation of the concrete irregularities at the interface with the connectors was exactly what was needed to obtain a more accurate result. As shown in Figure 9, the adjusted model didn't converge up to the curve's plateau, due to excessive distortion of the elements of the flexible bands.
Given the impact that irregularities at the interface with the connectors had on the connector's stiffness, it may be necessary to consider measures to minimize plastic settlement when applying the connection here proposed. According to CCAA T41/HB64 [7], that may be achieved by designing a concrete mix with lower slump and higher cohesion and fully compacting the concrete.

6. Conclusion

The shear strength parameters obtained for the Crestbond connector proposed in this work varied between 400 kN and 500 kN for load P_1 and 550 kN and 940 kN for P_u. These values are generally higher than the strength of a typical single plate connection with compatible size, thus favoring the viability of this new application.
The numerical models have proven to be reliable in predicting loads P_1 and P_u, presenting a maximum error of 6.4% with respect to test, which attests to their validity. Also, they have enabled a better understanding of the Crestbond behavior, elucidating the mechanisms that occur during the push tests, the influence of the column section geometry and the impact of the concrete conditions in the connector's stiffness.
The gaps between the concrete and the connector may have resulted from settlement of the concrete and had significant impact on the connector stiffness, therefore, when applying the connection here proposed it may be necessary to take measures to minimize this effect, given that the displacements caused by the gaps at the connection could compromise service limit state.

7. Acknowledgments

The authors thank FAPEMIG (Fundação de Amparo à Pesquisa do Estado de Minas Gerais), CAPES (Coordenação de Aperfeiçoamento de Pessoal de Nível Superior), CNPq (Conselho Nacional de Desenvolvimento Científico e Tecnológico) and Vallourec for their support in this research project.

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