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Plate with holes as shear connector in cold formed steel composite beams

Chapa com furos como conectores de cisalhamento em vigas mistas com perfil formado a frio




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Abstract

In search of an improved compatibility between cold-formed steel profiles and precast floor systems, this study proposes an alternative shear connector for cold-formed steel-concrete composite beams. This connector consists of a steel plate with holes placed longitudinally in the middle of the upper flange of the steel profile, aiming to maximize the support area for precast slabs during the assembly. The proposed solution was experimentally tested on I-beams under bending, composed by two cold-formed steel channels, connected to a reinforced concrete slab by the shear connector. The relative slip between the steel profile and concrete, vertical deflection of the beam, and strains at several locations of the composite section were measured. The results show that the proposed connector assures shear transfer at the interface of the composite section components and shows strength of the same magnitude as other commonly used connectors.

Keywords: composite beam, cold-formed steel profiles, shear connector, perfobond.

Resumo

Buscando compatibilizar as estruturas em perfil formado a frio com os sistemas de piso com pré-laje, foi proposta nesse trabalho uma alternativa de conector de cisalhamento para vigas mistas compostas por perfil formado a frio que visa maximizar a área disponível para apoio das pré-lajes durante o processo construtivo. Trata-se de um conector em chapa perfurada disposto longitudinalmente no centro da mesa superior do perfil. Para testar a solução proposta, foi realizado um trabalho experimental em que se submeteu à flexão uma viga de seção I composta por dois perfis U enrijecidos formados a frio conectada a uma laje de concreto armado através dos conectores propostos. Nestes ensaios foram medidos o deslizamento relativo entre o perfil de aço e o concreto, o deslocamento vertical da viga e as deformações em diversos pontos da seção mista. Analisando os resultados obtidos pôde-se concluir que o conector proposto consegue desempenhar a função de transferir o cisalhamento na interface dos componentes da seção mista apresentando resistência da mesma ordem que conectores usuais

Palavras-chave: viga mista, perfiis formados a frio, conector de cisalhamento, *perfobond*.

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1. Introduction

1.1 Initial considerations

The use of cold-formed steel (CFS) profiles for steel-concrete composite beams is still relatively uncommon, and since the headed studs, one of the most usual sort of shear connectors, is incompatible with these profiles due to their small thickness, there has been recent efforts to study connectors that properly suit these beams. Figure 1 shows examples of these connectors along with their average strengths (Q_{Rm}).

Although these connectors are compatible with CFS profiles, the large space that they occupy on the upper flange of the steel profile can interfere with precast slabs, which need a free surface for support during the assembly; therefore, the compatibility of such connectors with precast floor systems is generally difficult.

As a possible solution for this problem, use of steel plate with holes as shear connectors is suggested. Longitudinal placement of these connectors assures that they occupy less space on the upper flange of the profile, leaving the surface almost completely free for receiving the precast slabs (Figure 2).

Four point beam bending tests were performed to evaluate the proposed solution. This article presents the adopted experimental methods, the analysis of the results, and a comparison of the strength result with theoretical predictions found in the literature.

1.2 The connector used in the study

The connector used in this study, shown in Figure 3, has similar features to the *Perfobond*, an already widely studied connector, with main applications in bridges and other large structures. The *Perfobond* connector is a steel plate with longitudinally aligned circular holes that is usually thicker than 12 mm, sometimes reaching 20 mm. The connector height is about 130 mm, and the diameter of the holes is about 50 mm, as can be seen in the works of Oguejiofor & Hosain[4], Medberry & Shahrooz[5] and Al Darzi *et al.*[6]. Table 2 summarizes these dimensions. The connector used in the present work, though having a similar configuration, was adapted to be more appropriate for the reduced dimensions of the CFS beams. The dimensions of this connector are smaller than those of *Perfobond*: the thickness is set to 8 mm, height to 90 mm, and the diameter of the holes to 20 mm, as shown in Figure 4. Therefore,

(a) Bolt M12
Q_{Rm} = 44.3 kN (Lawan *et al.*[1])

(b) Arc-shaped connector
Q_{Rm} = 180.0 kN (Chaves[2])

(c) Channel connector 120x25x3
Q_{Rm} = 245.1 kN (Bremer[3])

Figure 1
Shear connectors suggested for CFS composite beams and their average strength

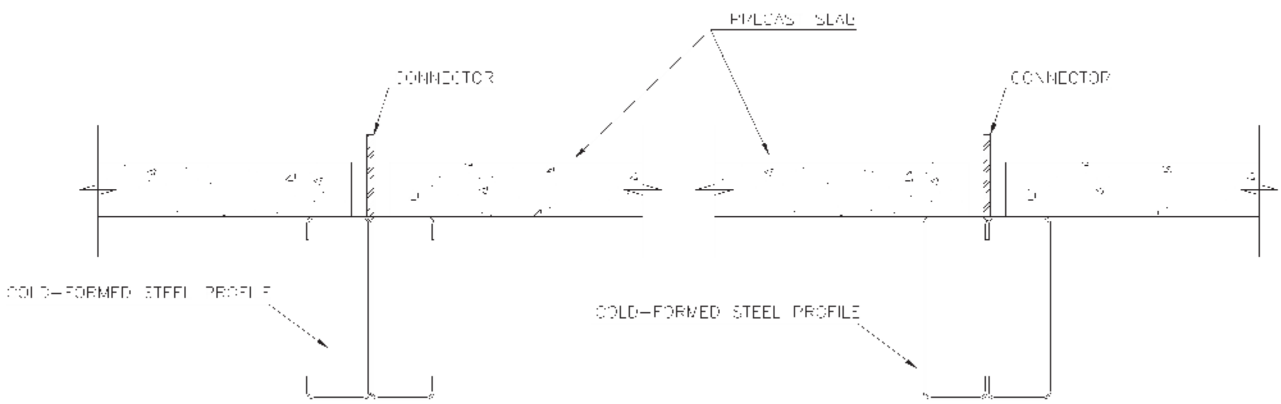

Figure 2
Steel plate with holes applied to CFS composite beams in precast floor systems

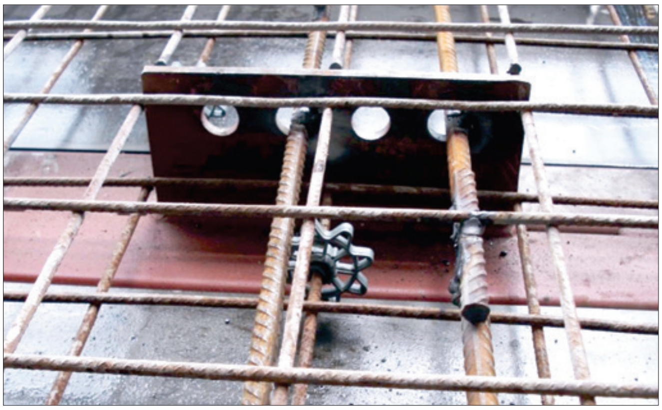

Figure 3
Tested connector prior to concrete casting

throughout this work, when referring to this connector, the terminology “plate with holes” is used.

2. Literature review

2.1 Composite beams with a cold-formed steel component

The Brazilian Standard ABNT NBR 14762:2010[7] specifies that the design of the composite steel-concrete beams constituted by a CFS profile symmetric about the plane of bending, and a concrete slab connected to the upper surface of the steel component by shear connectors, can be done based on Brazilian Standard ABNT NBR 8800:2008[8].

The Brazilian Standard ABNT NBR 8800:2008[8] specifies that the composite beams with h/t_w ratio higher than $3.76\sqrt{E/f_y}$ have to be

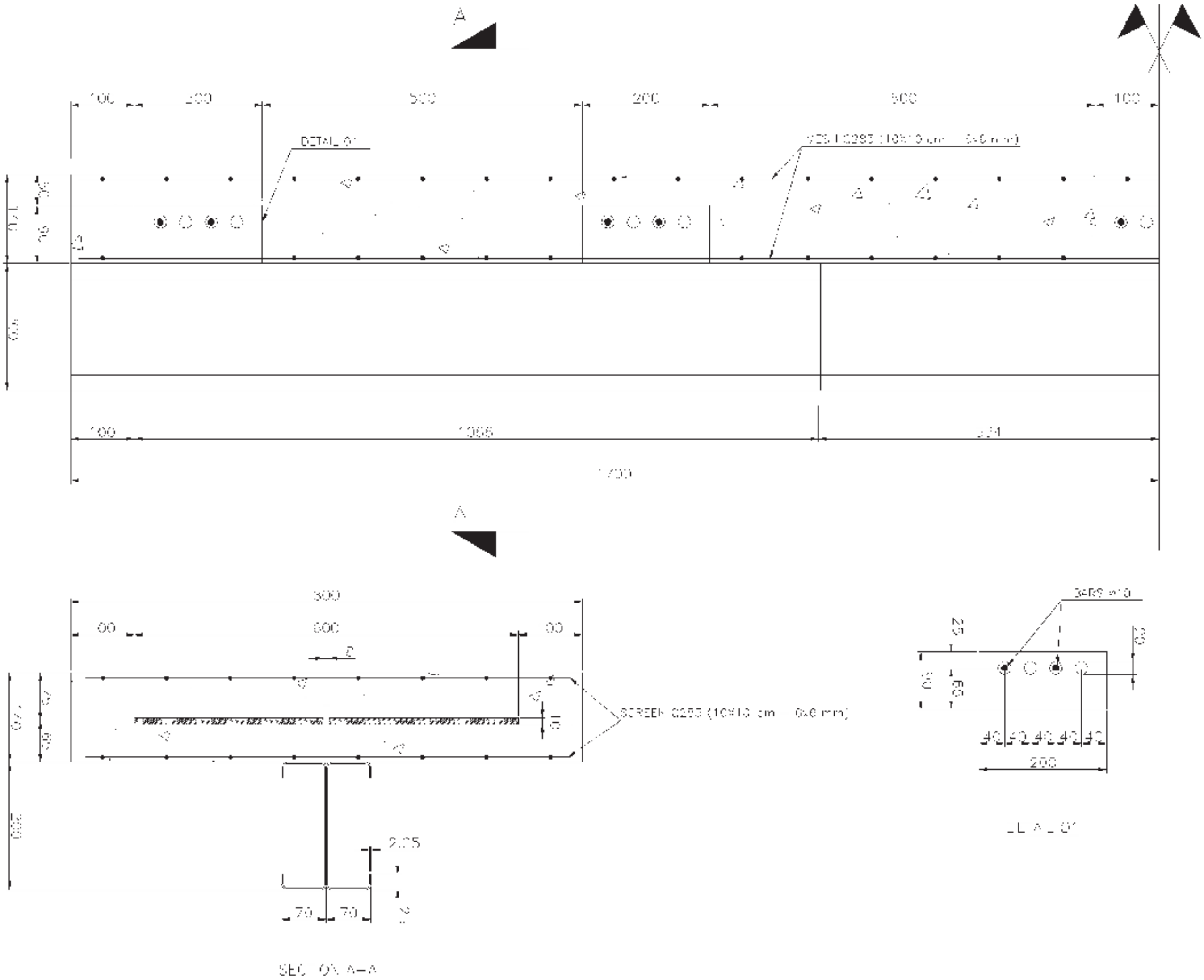

Figure 4
Geometry of the specimens, sizes in millimeters

calculated in elastic range, with normal longitudinal stresses limited to steel yield strength and concrete compressive strength[1]. Therefore, since the plates that form CFS profile have small thickness, normally varying between 1.5 and 4.75 mm, the majority of composite beams formed by CFS profiles will have to be designed with this limitation.

However, if, on one hand, the reduced thickness of CFS profiles makes them prone for local buckling, on the other hand, the smaller sectional area of these profiles brings the neutral axis of the composite beams to be located on concrete in most of the cases. Hence leaving the profile completely in tension and, therefore, free of instability.

2.2 The Perfobond

2.2.1 General properties

Perfobond is a rigid connector developed in 1987 by German company Leonhardt, Andrä und Partner for use in composite bridges and other structures subjected to fatigue. The connector consists of a plate with longitudinally aligned circular holes that are filled by concrete and can be penetrated by reinforcement bars. The concrete and transversal bars inside these holes improve the shear strength and prevent uplift (vertical separation between the beam and the slab). The *Perfobond* is embedded into composite structures by welding it to the steel profile prior to casting the concrete slab.

2.2.2 Failure mode in Perfobond tests

According to Oguejiofor[10], failure mode of the concrete slab in Perfobond specimens that don't have transverse reinforcement bars is brittle. Starting from cracks along the connector line, the slab suddenly splits.

When transverse reinforcement bars are present within the holes, splitting is resisted by reinforcement and failure is characterized by large cracks along the connector line and crushing of concrete in

Table 1
Analytical models for prediction of *Perfobond* strength

Authors	Model	Equation
Oguejiofor & Hosain[4]	$Q_R = 4.50.h_{sc}.t_{sc}.f_{ck} + 0.91.A_{tr}.f_y + 3.31.n.D^2.\sqrt{f_{ck}}$	(1)
Medberry & Shahrooz[5]	$Q_R = 0.747.b.h.\sqrt{f_{ck}} + 0.413.b_f.L_c + 0.9.A_{tr}.f_y + 1.66.n.D^2.\sqrt{f_{ck}}$	(2)
Al Darzi *et al.*[6]	$Q_R = 255.31 + 7.62.10^{-4}.h_{sc}.t_{sc}.f_{ck} - 7.59.10^{-7}.A_{tr}.f_y + 2.53.10^{-3}.A_{sc}.\sqrt{f_{ck}}$	(3)

h_{sc} = connector height;
t_{sc} = connector thickness;
f_{ck} = characteristic compressive strength of concrete in MPa;
A_{tr} = area of transverse reinforcement;
f_y = yield strength of transverse reinforcement steel;
n = number of holes in the connector;
D = diameter of the holes;
b = thickness of the concrete slab;
h = length of the concrete slab in front of the connector (considered as the average distance between the connectors in this work);
b_f = width of the upper flange of the profile;
L_c = length of the contact region between the slab and the profile (considered as the average distance between the connectors in this work);
A_{sc} = total area of the concrete dowels in the holes.

Table 2
Applicability limits and values used in obtaining the analytical models

Oguejiofor & Hosain[4]		Medberry & Shahrooz[5]		Al Darzi *et al.*[6]	
Parameters	Limits	Parameters	Limits	Parameters	Limits
t_{sc} (mm)	13	t_{sc} (mm)	12.7 – 19.05	h_{sc} (mm) × t_{sc} (mm)	1200 – 4500
h_{sc} (mm)	127	h_{sc} (mm)	139.7	A_{tr} (mm²) × f_y (MPa)	20000 – 1100000
f_{ck} (MPa)	20.91 – 41.43	f_{ck} (MPa)	39.6 – 45.5	A_{sc} (mm²) × f_{ck} (MPa)	5000 – 43000
A_{tr} (mm²)	0,0 – 817.4	A_{tr} (mm²)	0.0 – 999.3	–	–
f_y (MPa)	406.4 – 426.6	f_y (MPa)	Not stated	–	–
n	0 – 4	n	0 – 3	–	–
D (mm)	50	D (mm)	50.8	–	–

t_{sc} = connector thickness;
h_{sc} = connector height;
f_{ck} = characteristic compressive strength of concrete in MPa;
A_{tr} = area of transverse reinforcement;
f_y = yield strength of transverse reinforcement steel;
n = number of holes in the connector;
D = diameter of the holes;
A_{sc} = total area of the concrete dowels in the holes.

[1] *According to ANSI/AISC 360-16[9], section of comments, item I3-2a, adopting first yield as flexural strength limit is a conservative specification that accounts for possible web buckling when web is slender and has large portion in compression.*

Table 3
Strength of the materials

	Steel				Concrete
	Profiles and plates (USI SAC300)	Screen Q283 (CA-60)	Transverse bars (CA-50)		
f_y (MPa)	369	690	575	f_{cm} (MPa)	20.12
f_u (MPa)	453	725	633		

front of the connector. In this case, failure process is slower due to the action of transverse reinforcement and aggregate interlock, which enables the cracked slab continue resisting to shear until failure of the connector or transverse reinforcement.

2.3.3 Strength of Perfobond

Since the invention of *Perfobond* various analytical models were proposed to predict the strength of this shear connector. The models of Oguejiofor & Hosain[4], Medberry & Shahrooz[5] and Al Darzi *et al.*[6] were chosen as a reference to verify the experimental results in this study due to a simpler and more direct relation of the terms of the equations to the geometry of the proposed experimental model. Table 1 summarizes the equations for the analytical models.
The equation proposed by Oguejiofor & Hosain[4] has three terms, the first one related to the contribution of the frontal contact between the connector and concrete, the second one related to the transverse reinforcement contribution, and the third one related to the shear of the concrete dowels formed through the holes of the connector. The equation of Medberry & Shahrooz[5] also considers these three contributions; however, it also takes into account the term related to the adhesion between the concrete and the upper surface of the steel profile. The equation of Al Darzi *et al.*[6], besides the three components, includes also one constant.
The three analytical models were obtained from shear tests and parametric studies, and their validity is limited to the range of values shown in Table 2.

The connector used in this study is an adaptation of the *Perfobond* and has reduced dimensions which don't comply with the limits of the parameters t_{sc}, h_{sc}, D and $h_{sc} \times t_{sc}$ adopted by the authors above.

3. Experimental program

3.1 Bending tests

The specimens used for the tests are composite beams with the span of 3200 mm, consisting of a 140 mm thick and 800 mm wide slab, and one I-beam, composed by coupled back-to-back oriented stiffened U channels (Ue200x70x25x2.25). The slab is reinforced by welded mesh Q 283 (10x10cm – 6x6mm) on top and bottom faces. Five connectors were used to join the components of the composite section. The connectors are CH8x90x200 mm plates with four longitudinally aligned holes of 20 mm diameter; steel bars of 10 mm diameter pass through two of these holes. Figure 4 shows the detailed geometry of the specimens.
Table 3 presents average yield strength (f_y) and ultimate strength (f_u) for the steel and average compressive strength (f_{cm}) for the concrete used in the specimens, which were obtained by tensile tests and cylinder compression tests respectively.
Three specimens were tested in a four point bending configuration, as shown in Figure 5. Load was applied by hydraulic press through a load distributing beam. Twenty five load cycles between 15 kN and 60 kN were applied before test, as recommended by EN 1992 11:2004[11] for tests on shear connectors.

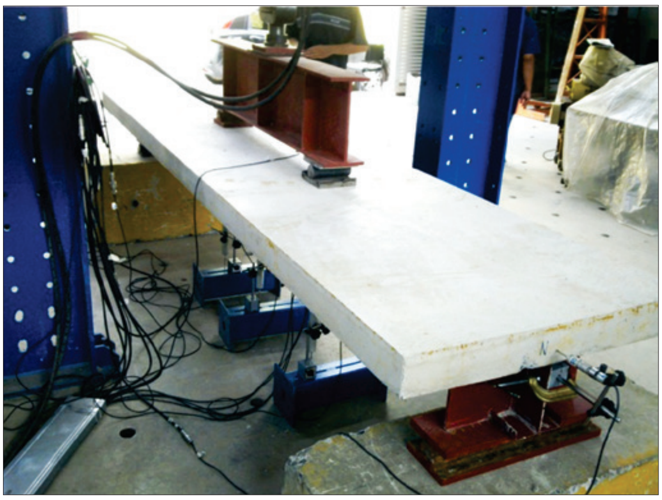

Figure 5
Experimental set-up

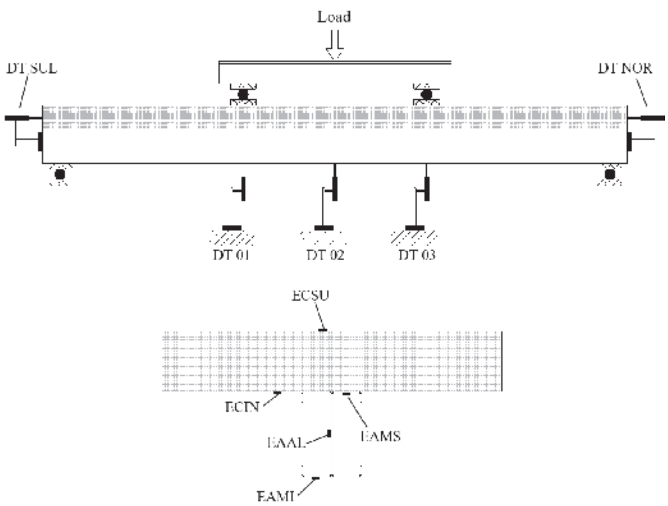

Figure 6
Strain gauges and displacement transducers

Table 4
Ultimate load and corresponding moment for each specimen

	Ultimate load (kN)	Corresponding bending moment (kN.cm)
Specimen 01	172.51	9200.60
Specimen 02	177.94	9489.90
Specimen 03	169.61	9045.65
Average	173.35	9245.38

Vertical deflection was measured under the loading points by displacement transducers named DT 01 and DT 03 and at mid-span by transducer DT 02. The slip of the steel at the concrete interface was measured at the ends of the specimen by transducers DT NOR and DT SUL. Strain gauges were placed on the central section: on the top face of the concrete slab (strain gauge ECSU); on the bottom face of the concrete slab (strain gauge ECIN); on the bottom face of the upper flange of steel profile (strain gauge EAMS); in the middle of the profile web (strain gauge EAAL); and on the bottom face of the bottom flange of steel profile (strain gauge EAMI). Instrumentation set up is shown in Figure 6.

3.2 Results

The ultimate load and corresponding bending moment for each of the three specimens are shown in Table 4.
The specimens showed the first cracks on the bottom face of the slab between loads 70 and 114 kN, thereafter cracking increased rapidly and longitudinal cracks appeared on the top face along the connectors' line. Concrete failure was first observed near the outermost connectors at the end of the tests.
With the displacement transducers and strain gauges installed on the three specimens, it was possible to obtain curves load × deflection at three points along the beam's span, load × slip at both ends of the beam, and load × strain at different points of mid-span

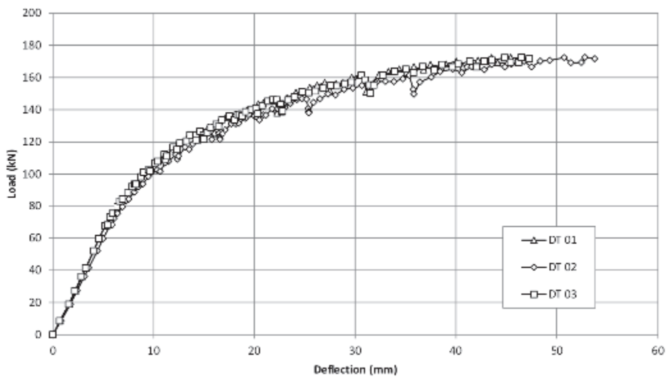

Figure 7
Load (kN) × vertical deflection (mm) under the loading points (DT 01 e DT 03) and on the central section (DT02) of Specimen 01

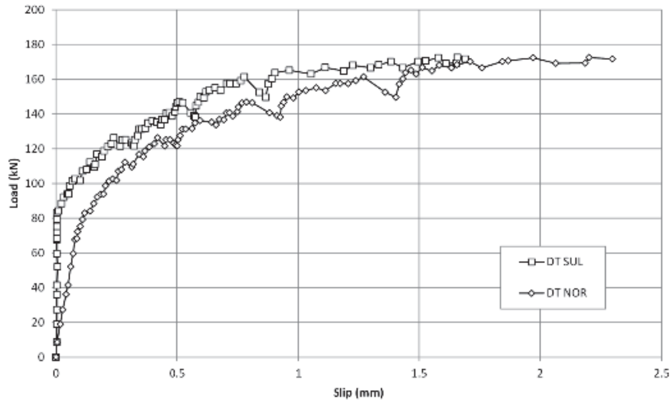

Figure 8
Load (kN) × slip (mm) on steel – concrete interface, at the ends of Specimen 01

section. These curves are shown, respectively, in Figures 7, 8 and 9 for Specimen 01.

3.3 Discussion of the results

To evaluate the degree of interaction of the beams, calculations were done for the specimens with full interaction and for the steel profile performing as an isolated beam. The calculated ultimate bending moment obtained for these two conditions were 10037 kN.cm and 3891 kN.cm respectively. The average ultimate bending moment obtained in the tests was 9245 kN.cm (this value is between the two calculated values). Therefore, the specimens had partial-interaction composite behavior.
When the applied load was between 80 and 100 kN, slip rate increased, and the load × slip curves (Figure 8), as well as load × strain (Figure 9) and load × deflection (Figure 7) curves, started bending. This indicates that at this load level the connectors started to show decreasing stiffness and partial-interaction mode begun.
The load × strain curve (Figure 9) show that the strains at

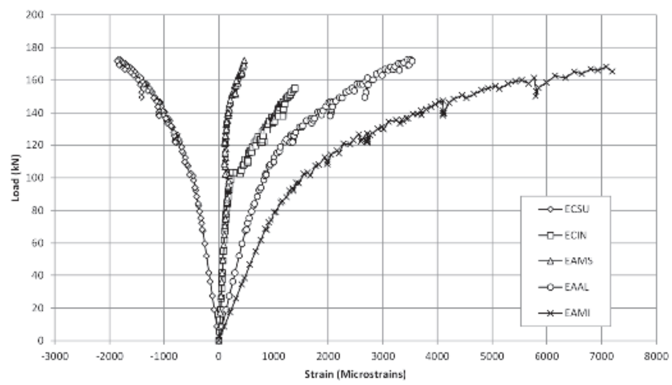

Figure 9
Load (kN) × strain (10^{-6}) at various points of the mid-section of Specimen 01

the bottom face of the slab and on the upper flange of the steel profile (ECIN and EAMS) have almost identical trajectories until the load reaches the value of approximately 100 kN. After reaching this load, the strain curves started diverging. This indicates that after load reached this value, relative slip between the bottom face of the slab and the upper flange of the profile increased, as these surfaces started deforming at different rates, with concrete showing larger deformations than steel profile.

By the end of the tests, slip increased drastically, showing well-defined plateaus on load × slip curve (Figure 8), indicating that the connectors had reached their ultimate strength. The end of the tests followed shortly after, with high level of cracking along the connectors' line and local failure of concrete next to the connectors.

4. Behavior of the connectors

According to Brazilian Standard ABNT NBR 8800:2008[8], the horizontal shear force at the interface of the composite section (F_{hd}) is equal to the compressive force on the slab when neutral axis is in the steel profile or equal to the tensile force on the steel profile when neutral axis is in the slab. This force is resisted by the connectors located between the maximum moment section and closest inflection point; in this case, by the two connectors at the ends of the beam, because the contribution of the central connector is negligible, since the shear flow in the central portion of the span is zero or very small.

Since neutral axis is in the slab in this case, the resultant tension force on the steel profile (F_{hd}) and the average slip between steel and concrete at the ends of the beam were obtained at different load levels from strain gauges (EAMI, EAAL and EAMS) and displacement transducers (DT SUL and DT NOR). Dividing the resultant tension force and the average slip values by the number of engaged connectors (two), the force × slip curves for the connector were obtained.

Table 5 shows the values of force and slip that were used to describe the connector's force × slip curves, as well as the strain and displacement values from which they were calculated.

Figure 10 shows the strain and stress distribution given by strain gauges at a certain load and the resultant forces obtained. Figure 11 shows the force × slip curves.

Table 5
Obtaining shear force and slip of the connector at various load levels

	Load (kN)	Strain			Strain			Resultant force on profile F_{hd} (kN)	Shear force per connector (kN)	Slip (mm)		Average slip (mm)
		ε_{ami}	ε_{aal}	ε_{ams}	σ_{ami}	σ_{aal}	σ_{ams}			SUL	NOR	
Specimen 01	0.0	0.000000	0.000000	0.000000	0.00	0.00	0.00	0.0	0.0	0.000	0.000	0.000
	39.2	0.000469	0.000272	0.000058	93.81	54.35	11.59	88.1	44.0	0.004	0.047	0.025
	58.8	0.000720	0.000416	0.000091	143.94	83.28	18.15	136.0	68.0	0.005	0.068	0.037
	78.5	0.001002	0.000574	0.000126	200.32	114.89	25.11	188.8	94.4	0.005	0.102	0.053
	103.0	0.001554	0.000860	0.000186	310.86	172.02	37.19	288.8	144.4	0.072	0.212	0.142
	103.0	0.001681	0.000865	0.000115	336.15	172.92	23.03	295.8	147.9	0.102	0.251	0.177
	117.7	0.002169	0.001105	0.000126	369.00	220.92	25.25	352.5	176.3	0.176	0.344	0.260
	137.3	0.003272	0.001670	0.000161	369.00	334.02	32.26	406.4	203.2	0.386	0.591	0.488
	156.9	0.005010	0.002459	0.000286	369.00	369.00	57.27	452.4	226.2	0.657	1.078	0.868
	167.7	0.007209	0.003171	0.000434	369.00	369.00	86.71	483.2	241.6	1.269	1.613	1.441
Specimen 02	0.0	0.000000	0.000000	0.000000	0.00	0.00	0.00	0.0	0.0	0.000	0.000	0.000
	39.2	0.000509	0.000300	0.000066	101.88	60.10	13.26	97.1	48.5	0.009	0.012	0.010
	58.8	0.000745	0.000447	0.000099	148.91	89.36	19.70	142.8	71.4	0.015	0.016	0.015
	78.5	0.001052	0.000630	0.000142	210.42	126.02	28.43	202.1	101.0	0.027	0.030	0.028
	98.1	0.001465	0.000880	0.000199	293.09	176.10	39.74	281.8	140.9	0.071	0.074	0.073
	117.7	0.002085	0.001225	0.000227	369.00	245.09	45.38	373.3	186.7	0.154	0.147	0.151
	137.3	0.003421	0.001927	0.000345	369.00	369.00	69.09	441.2	220.6	0.310	0.335	0.323
	156.9	0.005362	0.002808	0.000589	369.00	369.00	117.72	494.0	247.0	0.445	0.680	0.563
	164.8	0.006953	0.003399	0.000750	369.00	369.00	149.97	519.5	259.8	0.606	0.949	0.778
Specimen 03	0.0	0.000000	0.000000	0.000000	0.00	0.00	0.00	0.0	0.0	0.000	0.000	0.000
	39.2	0.000541	0.000288	0.000054	108.26	57.69	10.85	98.3	49.1	0.007	0.026	0.017
	58.8	0.000815	0.000438	0.000085	163.07	87.67	17.07	148.8	74.4	0.011	0.030	0.020
	78.5	0.001195	0.000615	0.000124	239.02	122.98	24.85	215.4	107.7	0.017	0.036	0.026
	98.1	0.001810	0.000887	0.000188	362.00	177.48	37.69	322.0	161.0	0.059	0.094	0.077
	117.7	0.002657	0.001252	0.000248	369.00	250.31	49.63	385.9	193.0	0.132	0.172	0.152
	137.3	0.004696	0.001998	0.000348	369.00	369.00	69.53	444.5	222.3	0.329	0.322	0.325
	151.0	0.007070	0.002911	0.000462	369.00	369.00	92.39	482.0	241.0	0.747	0.523	0.635
					Average max shear force per connector (kN)				**247.5**			

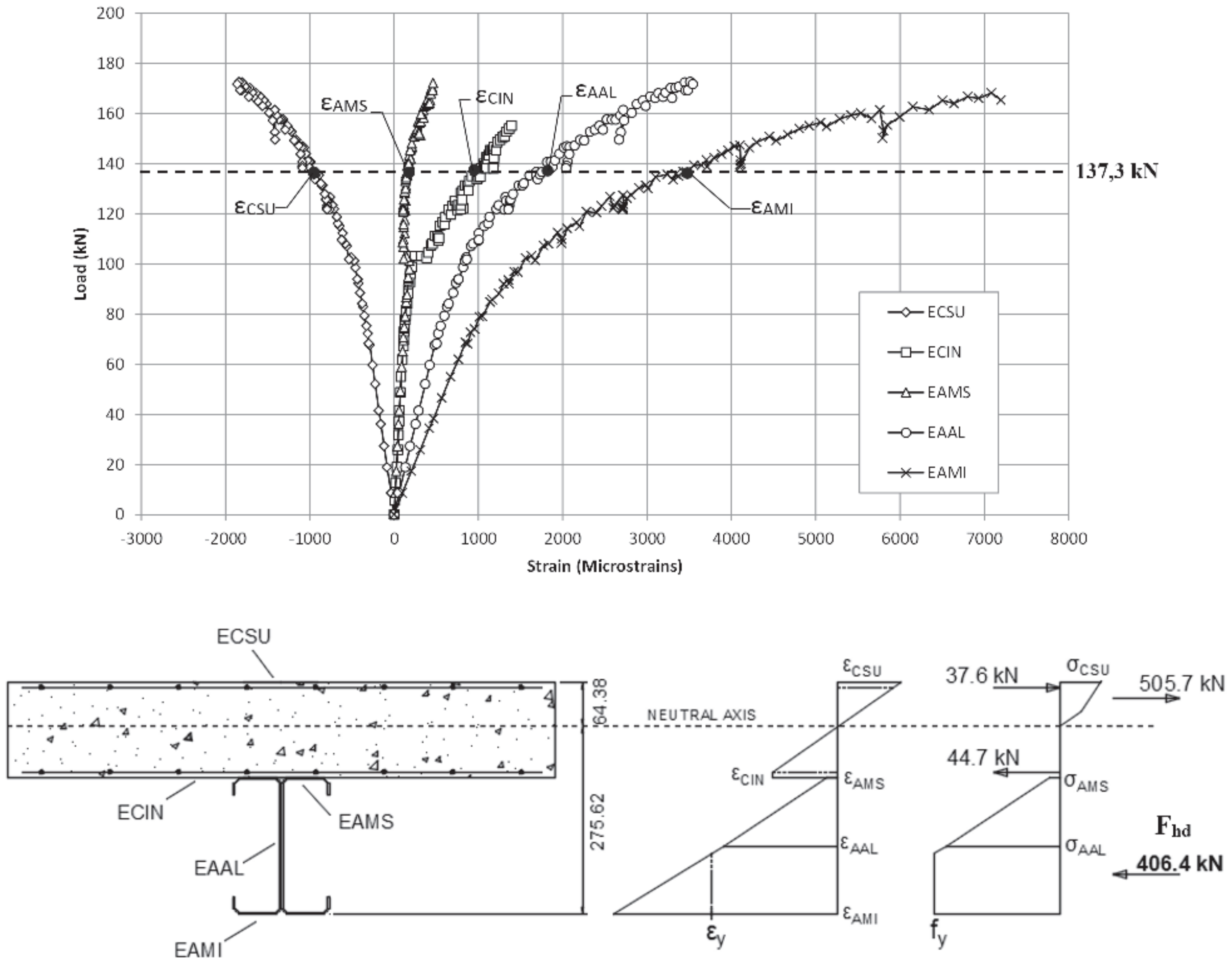

Figure 10
Resultant force F_{hd} for applied load of 137.3 kN on Specimen 01 obtained from strain gauges

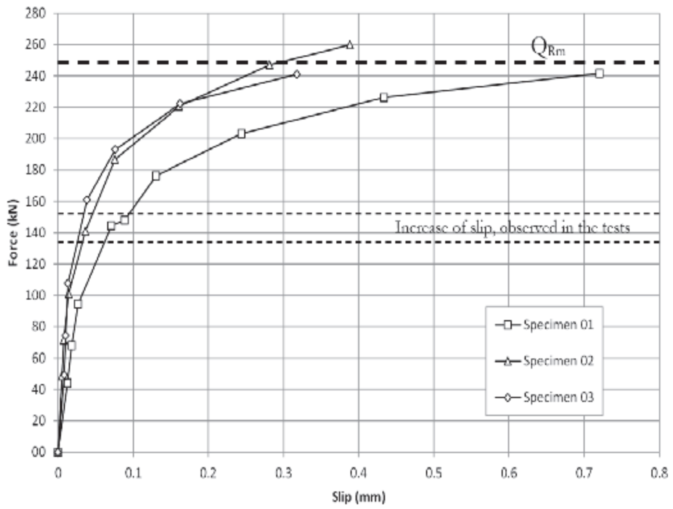

Figure 11
Force (kN) × slip (mm) curves of the connector

The force × slip curves of the three specimens converge to a maximum average strength (Q_{Rm}) of 247 kN.
The strength of the tested connector was calculated using the analytical models for *Perfobond* proposed by Oguejiofor & Hosain[4], Medberry & Shahrooz[5] and Al-Darzi *et al.*[6]. Predictions of these models are compared with experimental average strength at Table 6.
Even though the average of the three predictions are close to the experimental strength, none of the individual predictions are close enough to the experimental strength, with the models of Medberry & Shahrooz[5] and Al-Darzi *et al.*[6] resulting in predictions 20 and 17% above experimental strength respectively, and the quite conservative model of Oguejiofor & Hosain[4] predicting strength 31% below.
A possible explanation for the unsatisfactory approximation of the predictions is the fact that the thickness of the plate, the diameter of the holes and the height of the tested connector are beyond the applicability limits of the analytical models developed for *Perfobond*.
With the connector's strength value (Q_{Rm}) of 247 kN, theoretical ultimate bending moment of the composite beam in

Table 6
Comparison of the experimental and predicted result of Q_R

Experimental strength	Analytical predictions			Average analytical prediction
	Oguejiofor & Hosain[4]	Medberry & Shahrooz[5]	Al-Darzi *et al.*[6]	
247 kN	171 kN	296 kN	289 kN	252 kN
Difference	31%	20%	17%	2%

partial-interaction mode was calculated and compared to the average experimental ultimate bending moment shown in Table 4. The interaction degree for such Q_{Rm} value was found to be 0.78, which resulted in a theoretical ultimate bending moment of 9322 kN.cm. This value is only 1% different from average experimental ultimate bending moment.

5. Conclusions

The test results show that the proposed solution is viable, since the tested beams showed a satisfactory composite behavior, resisting 238% higher average moment (9245 kN.cm) than the bending strength moment of the isolated steel beam (3891 kN.cm). And the tested connector showed strength compatible with theoretical predictions, and comparable with strength of other commonly used connectors.
Among the analytical models used in this study, only the model of Oguejiofor & Hosain[4] was appropriate, since the other models predicted strength values higher than the obtained in tests. However, the model of Oguejiofor & Hosain[4] was very conservative, perhaps because it was developed for *Perfobond*, which usually has different dimensions than the connector dimensions used in the present work. Therefore, the development of an analytical model that can be applied for a wider range of geometry dimensions is desirable.

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