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EXPERIMENTAL INVESTIGATION OF NEW THIN-WALLED PERFOBOND SHEAR CONNECTORS

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Abstract. Composite steel and concrete constructions is being frequently specified in order to guarantee optimized material consumption, higher resistance capacity and increase of flexural stiffness for structural components. Several scientific researches seek alternatives which enables an efficient interaction between steel and concrete and, at same time, make possible the exploitation of the best characteristics associated with both materials. In this context, the main objective of this study is to present a new alternative for shear connectors, conceived to ensure composite action in an innovative structural system, composed by a thin-walled steel truss and one-way concrete slab, in process of development aided by experimental and computational investigation. The solution that would be presented consists of a Thin-Walled Perfobond (TWP) ribs, which is made up by two plates with 1,25 mm nominal thickness each and attached to the referred truss by self-drilling screws. Although the perfobond is very usual for conventional steel beams (generally composed by compact sections), its application on cold-formed profiles has yet to be exploited. It is important to point out that the shear connectors were conceived to provide a restraint mechanism for vertical and horizontal relative displacement between steel beam and concrete slab. A set of results are also presented related to preliminary experimental push-out tests, according to recommedations estabilished by Eurocode 4, performed in Structures Laboratory (LABEST) of Federal University of Rio de Janeiro (COPPE/UFRJ) involving perfobond shear connectors. The results indicate important aspects regarding structural behavior of the referred component in terms of resistance capacity and relative displacements between steel and concrete.

Keywords: *Perfobond, Thin-walled Perfobond, Composite structural systems, Push-out tests, Shear connector*

1 INTRODUCTION

Association of different structural materials is being increasingly used in order to provide benefits to constructions, such as optimized material consumption, higher resistance capacity, increase of flexural stiffness for structural components and reduction of costs. Over past decades, a particular solution, involving composite steel and concrete structures, is being reported: (a) composite beams, (b) tubular columns filled with concrete and (c) steel decks represent some examples.

Composite beams, generally composed by association between steel profiles and concrete slabs, enables specification of lighter sections and reduced height floor. The interaction of both material is provided by shear connectors, which has to present sufficient resistance capacity and adequate stiffness, as recommended by Eurocode 4.

In almost all cases, constructions frequently use conventional steel sections, characterized by more compact dimensions when compared with cold-formed profiles. In recent years, on the other hand, a series of studies demonstrates special interesting on new composite structural systems composed by thin-walled members. Specifications of such components generally is advantageous because of their elevated structural efficiency and also easy fabrication process compared to other types of steel sections.

Although the benefits of composite action between cold-formed steel members and concrete slabs are known, use of such advantages in design procedures is not quite simple and demands more information, especially about shear connector behavior. In comparison with steel conventional beams, there are few publications about ultimate load, load-displacements relationship and analytical expressions for shear connectors.

In this paper, the main objective consists of study a new alternative for perfobond shear connector, conceived to ensure composite action in an innovative structural system, composed by a thin-walled steel truss and one-way slab. It is important to highlight that, in all cases, the connection between perfobond ribs and the chord is made of self-drilling screws.

In this context, experimental analysis of 4 push-out tests was performed in Structures Laboratory of Federal University of Rio de Janeiro, according to recommendations established by Eurocode 4, and some preliminary results would be presented

The results indicate important aspects regarding structural behavior of the referred solution in terms of ultimate load and relative displacements between steel and concrete. Furthermore, in this present work, experimental data would be compared with analytical expression proposed by Oguejiofor & Hosain (1997) after computational investigation of push-out tests.

2 STUDIES OF PERFOBOND SHEAR CONNECTORS

Over past decades, many analytical and experimental researches involving shear connectors are reported, in order to guarantee structural efficacy for composite steel-to-concrete systems. In this context, as mentioned by KIM et al (2011), perfobond shear connector, which is composed by a perforated plate welded to steel beams, has very distinct qualities compared to other types (stud bolts, for example), such as: (a) better structural safety, (b) superior quality of welded connection, (c) constructability, (d) cost-effectiveness.

Perfobond ribs were first conceived in Germany, by Leonhardt, Andra and Partners of Stuttgart firm, as a solution for the Third Caroni Bridge in Venezuela (Valente & Cruz, 2004). Specially because of easy installation process, higher fatigue strength and improved shear capacity, perfobond connectors are widely used in bridges and composite steel-to-concrete floor systems.

In Figure 1, the main characteristics associated to the behavior of perfobond connector can be seen, including three mechanisms of shear resistance. The ultimate capacity, in terms of vertical and horizontal loads, is given as a function of the presence of concrete dowels inside perforations, transverse bars and contact surface between steel plate and concrete slab.

As stated by several authors (Vianna et al, 2013; Ahn et al, 2010), the contribution of both concrete dowels and steel bars provides higher resistance capacity, especially against vertical relative displacements, as well as improvements regarding ductility of the connection.

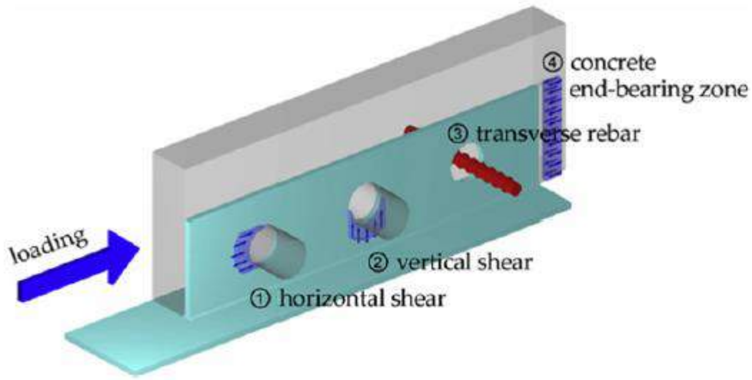


Figure 1 – Simplified force distribution acting on Perfobond shear connectors
(Ahn et al, 2010)

Another important aspect relative to the behavior of perfobond refers to slip capacity, as recommended by Eurocode 4 (EN 1994-1-1:2004). Although this type of shear connector usually presents elevated ultimate loads and stiffness, other characteristics are required in order to avoid brittle fracture in the contact surface between steel and concrete.

In this context, Eurocode 4 (EN 1994-1-1:2004) establishes a minimum slip capacity to guarantee adequate ductility, which prevent sudden collapse and enables better force redistribution. For this reason, experimental investigations must be able to measure and record the relative displacements between steel beams and concrete slabs in horizontal/vertical directions, during the push-out tests.

In Figure 2, different types of traditional solutions for shear connectors can be observed (Mazoz et al, 2013). Note that, in almost all cases, referred components are welded on top of compact steel section in order to enable interaction between steel beams and concrete slabs. Furthermore, all alternatives are capable to provide stiffness and adequate resistant capacity to transfer forces acting in horizontal and vertical directions (uplifting effect).

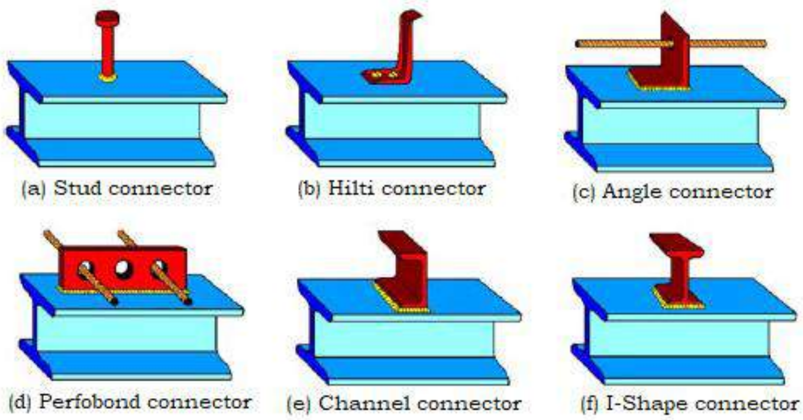


Figure 2 – Different types of traditional shear connectors
(Adapted from Mazoz et al, 2013)

3 ANALYTICAL EQUATIONS FOR PREDICTING ULTIMATE LOAD

Several studies regarding the behavior of perfobond shear connectors and analytical design equations have been reported, as a result of experimental push-out tests and computational analyses through Finite Element Method (FEM).

In this context, Oguejiofor & Hosain (1994) proposed a preliminary analytical equation for perfobond rib, based on experimental results, as a function of the concrete strength, presence of transversal bars and splitting resistance of concrete slab (Eq.1).

$$Q_{rib} = 2{,}871 \cdot n \cdot d^2 \cdot \sqrt{f_c} + 1{,}233 \cdot A_{tr} \cdot f_y + 0{,}590 \cdot A_{cc} \cdot \sqrt{f_c} \tag{1}$$

where,

Q_{rib} is the ultimate load per rib, in N;

n represents the number of holes;

d is the hole diameter, in millimeters;

f_c is the compressive concrete strength, in MPa;

A_{tr} is the area of transversal bars, in mm^2;

f_y represents yield strength of transversal bars, in MPa;

A_{cc} is the area of concrete subject to shear loads, in mm^2;

According to Kim et al (2013), the equation proposed by Oguejiofor & Hosain (1994) overestimates the bearing concrete resistance as well as the contribution of transversal bars inside the holes. After computational investigations, Oguejiofor & Hosain (1997) estabilished a new formulation to evaluate perfobond shear capacity, as follows:

$$Q_{rib} = 3{,}310 \cdot n \cdot d^2 \cdot \sqrt{f_c} + 0{,}91 \cdot A_{tr} \cdot f_y + 4{,}500 \cdot h \cdot t \cdot f_c \qquad (2)$$

where,

h is the height of the shear connector, in milimeters;

t represents the thickness of the shear connector, in milimeters;

Based on studies performed by Oguejiofor & Hosain (1997), Ahn et al (2010) developed an analytical investigation of push-out tests, depending on concrete compressive strength and shear connector arrangement. As result of the referred experimental program, the shear capacity per rib can be expressed by:

$$Q_{rib} = 2{,}980 \cdot n \cdot d^2 \cdot \sqrt{f_c} + 1{,}21 \cdot A_{tr} \cdot f_y + 3{,}140 \cdot h \cdot t \cdot f_c \qquad (3)$$

Note that Eq. 3 is very similar to Eq. 2 and can be written as a function of concrete strength, presence of transversal bars and holes on the connector surface. In the present work, the obtained results are compared with the equations 2 and 3 and preliminary conclusions are addressed.

4 EXPERIMENTAL TESTS OF THIN-WALLED PERFOBOND SHEAR CONNECTOR

4.1 General Provisions

The characteristics associated with an innovative thin-walled shear connector will be presented, including type and yield strength of steel components, dimensions of perfobond ribs and self-drilling screws arrangements.

The obtained results of experimental investigation are considered a preliminary study to evaluate the behavior of 1,25 mm thick cold-formed perfobond shear connector. The results indicate the connection is suitable for steel framing structural systems.

The following aspects should be highlighted: (a) traditional perfobond ribs are composed by thicker plates if compared to thin-walled perfobond shear connector adopted in the present research, (b) lack of information regarding shear resistance and slip capacity of perfobond ribs bolted with self-drilling screws and (c) shear connectors specially designed for provide composite behavior of cold-formed steel trusses and one-way slabs.

As previously mentioned, traditional solutions of perfobond shear connectors involves more compact perforated plates in comparison to thin-walled alternative predicted in this work. Experimental tests performed by Vianna et al (2013), for example, included eight specimens composed by 12,5 mm thick ribs, welded to W-shape beams.

In push-out tests studied by Zheng et al (2016), perfobond ribs consisted of 20,0 mm nominal thickness plate. In experimental tests performed by Cândido-Martins et al (2010), eight specimens with 15,0 mm thick ribs were investigated. Note that, in almost all researches involving perforated shear connectors, the behavior in terms of resistance and slip capacity is well-known for thick plates, generally more suitable for medium and long-span composite beams

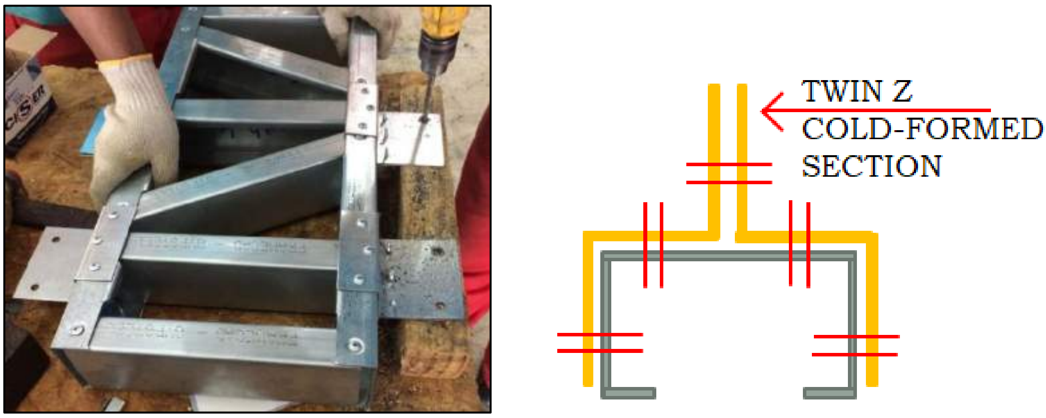


Figure 4 – Thin-walled perfobond shear ribs in process of fabrication

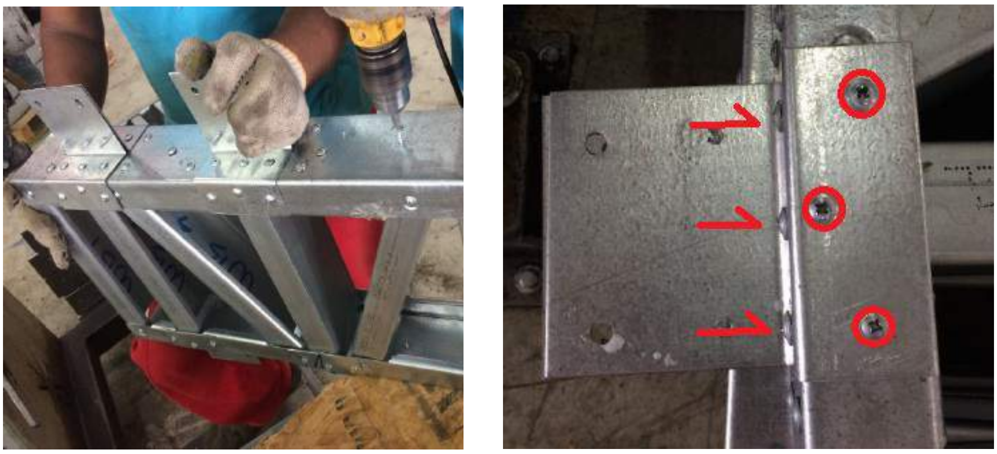


Figure 5 – Detailing of self-drilling screws adopted in push-out specimens

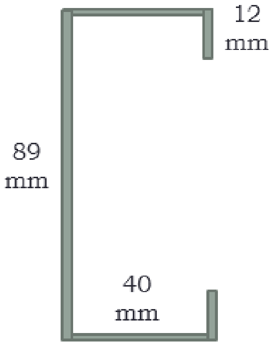
In Figure 4, it is possible to observe the process of fabrication of thin-walled perfobond ribs, composed by twin 1,25 mm plate thickness Z cold-formed section. The connection between shear connector and the chord of the truss is provided by twelve 4,8 mm self-drilling screws. Furthermore, three self-drilling screws were applied to permits connection between the twin Z CFS, in order to provide stiffness improvement, enabling these components to work as a single structure.

In Figure 5, note that each part of the perfobond ribs are connected to the chord with six self-drilling screws (three attached to the flange and three attached to the web of chord section). The main idea is to avoid loss of effectiveness of the chord if all segments of the cross-section were not conveniently connected.

All components, including diagonals, chords and ribs, were fabricated with 1,25 mm thick ZAR-345 steel. In terms of cross-section, all the members of the truss were built with stiffened cold-formed U 89x40x12x1,25 mm profiles, produced in an automated process of fabrication.

GypSteel Group (www.gypsteel.com.br) manufactured all the steel cold-formed components applied in the present research. Table 1 shows the geometrical and material properties of typical CFS of the chords, diagonal and post members of the trussed beam.

Table 1 – Geometrical and material properties of typical section fabricated by GypSteel



GEOMETRICAL AND MATERIAL PROPERTIES	
Type of steel	ZAR-345
Yield strenght (f_{yk})	345 MPa
Ultimate strenght (f_{uk})	485 MPa
Total area of cross-section (A_s)	2,41 cm^2
Thickness (t)	1,25 mm
Plastification force (N_{pl})	83,2 kN

4.2 Composite steel trusses and concrete slabs

As mentioned before, the studies regarding the behavior of thin-walled perfobond shear connectors consist of a fundamental requirement to enable composite action between cold-formed steel trusses and one-way concrete slabs. Fig. 6 shows the manufacturing facilities of GypSteel Group.

In the following, preliminary details of the innovative composite structural system are presented, including (a) the required ultimate shear capacity, based upon force equilibrium conditions of the cross-section, (b) expected live and dead loads as a function of the floor system span and (c) usual span between trusses.

Figure 6 – Steel cold-formed manufacturing facilities and layout of GypSteel Group headquarters

During the development of the engineering design of the composite floor system, GypSteel Group team described that, based upon practical procedures, steel framing constructions usually do not take into account any kind of interaction between steel and concrete. The reason is generally associated to the lack of information and reliable experimental data regarding the composite behavior of cold-formed steel trusses combined with concrete slabs. Therefore, when concrete slabs are considered, the steel trusses are designed to fully support dead and live loads without any contribution of composite action, conducting to low competitive construction system.

In this context, the innovative cold-formed structural floor system was conceived to provide (i) optimized material consumption, (ii) improved resistance and flexural stiffness, (iii) advantages in manufacturing, transportation and erection and (iv) economic competitiveness.

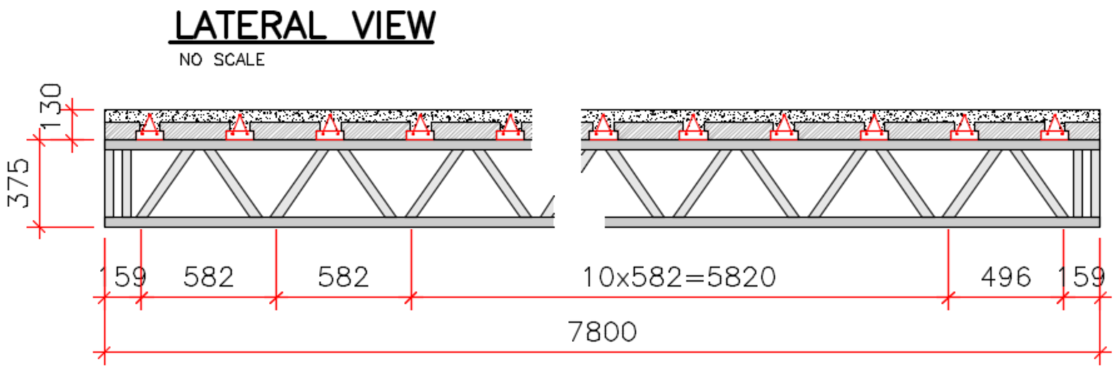


Figure 7 – Lateral view of the composite truss

Figure 7 illustrates the lateral view of the steel truss. The referred structural member consists of a “Warren” type truss configuration with alternated compression and tension diagonals. Furthermore, double vertical posts in both ends are included to provide reinforcement in the support region.

Another important aspect related to the composite system refers to the distance between supports, support end conditions and spacing between trusses. It was taken 7,5m clear span, simply supported ends condition and 600 mm is the distance between beams (typically adopted in steel framing construction).

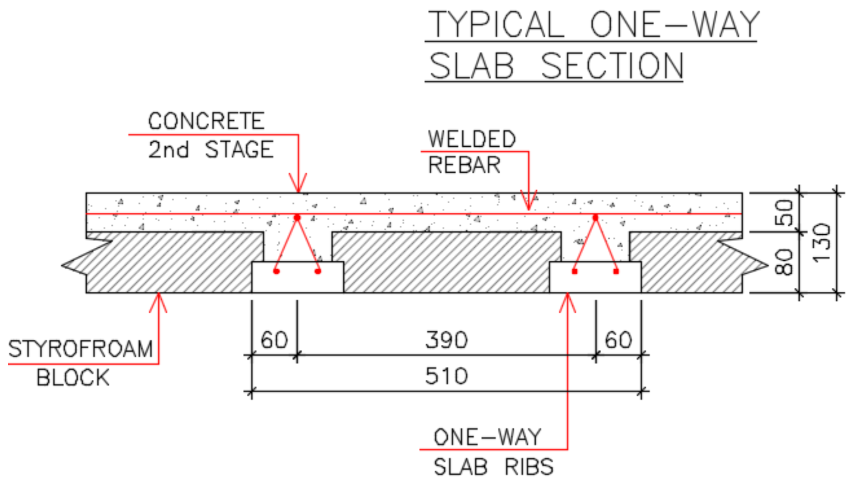


Figure 8 – One-way slab ribs positioned on top chord of the truss

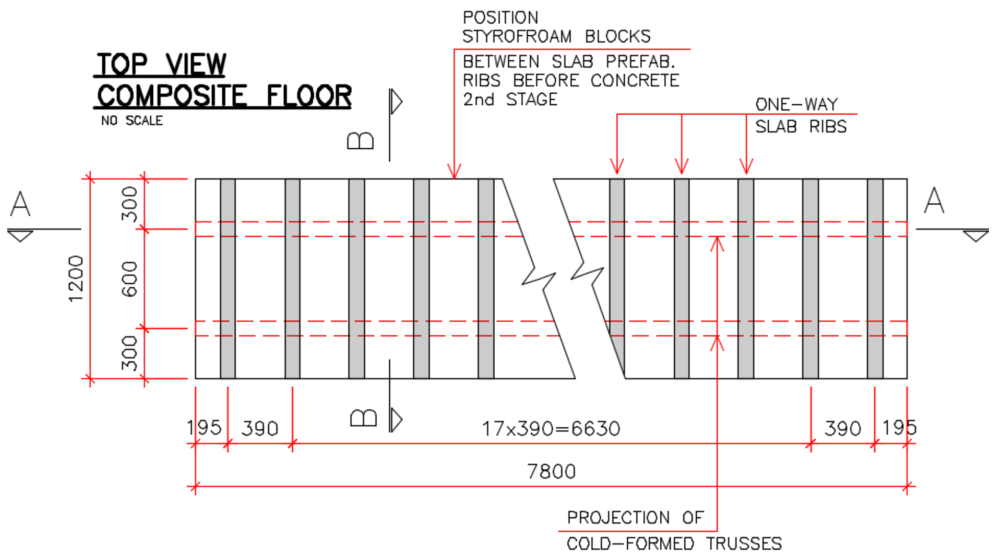


Figure 9 – Top view of composite floor (expected full-scale test arrangement)

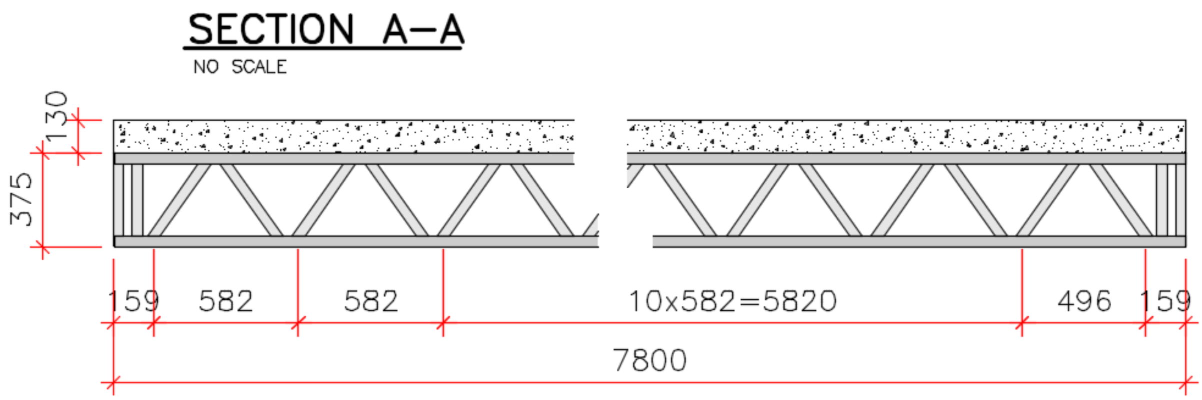


Figure 10 – Longitudinal section A-A (aligned with steel truss)

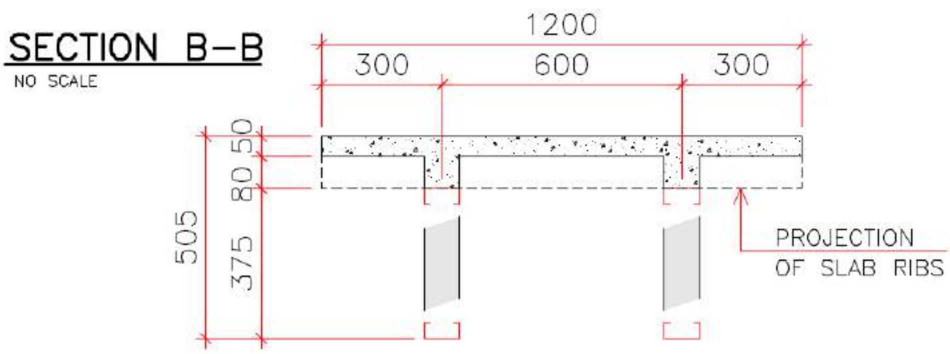
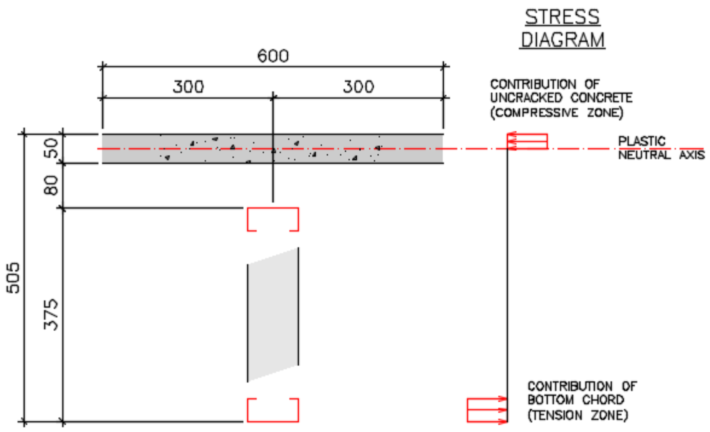


Figure 11 – Transversal section B-B (between slab ribs)

A typical one-way slab detail and arrangement of prefabricated slab ribs are presented in Figure 8, 9, 10 and 11. In first step, steel trusses, prefabricated slab ribs styrofroam blocks and welded rebar mesh are positioned over supports located at the beam ends. Furthermore, a second step of concrete pouring is applied until the total height of slab reaches 130 mm.

Once preliminary detailing of thin-walled composite system is defined, it is possible to evaluate an important requirement for adequate interaction between steel trusses and concrete one-way slabs: thin-walled perfobond shear capacity.

As recommended in technical literature, composite trusses must be designed considering full interaction with concrete slabs, which means that no slippage between both materials is allowed and cross-section resistance capacity is given as a function of full plastification of tension and compressive zones. Generally, the contribution of top chord is neglected in the design.



$$\sum F_x = 0$$

$$R_{cd} = R_{sd}$$

$$0{,}85 \cdot f_{cd} \cdot b \cdot t_c = A_s \cdot f_{sd}$$

$$t_c = \frac{A_s \cdot f_{sd}}{0{,}85 \cdot f_{cd} \cdot b}$$

Figure 12 – Force equilibrium analysis and resistant mechanisms of cross-section

According to Eurocode 4 (EN-1994-1-1:2004), resistant section of slab can be (conservatively) evaluated by the concrete compressive zone above styrofroam blocks (50mm thick as shown in Fig. 8). The constitutive model adopted to obtain the ultimate bending moment capacity of the composite system is a rigid-plastic model. The contribution of bottom chord, on the other hand, is based on elastic-perfectly plastic constitutive model.

Figure 12 shows the adopted resistant section model of the thin-walled composite system, composed by a rectangular concrete slab with dimensions of 50x600 mm assuming the effective width is equal to the trusses transversal spacing. In order to determinate the depth of the plastic neutral axis, it is needed to verify the maximum compressive force associated to the concrete and tension force related to bottom chord of the truss, as can be observed in Equations 4 and 5.

$$R_{cd} = b_{ef} \cdot t_c \cdot f_{ck}/\gamma_c \tag{4}$$

$$R_{sd} = A_g \cdot f_y/\gamma_{\alpha 1} \tag{5}$$

where

R_{cd} and R_{sd} represents maximum compressive and tension forces provided by concrete slab and bottom steel chord, respectively.

b_{ef} represents effective width of concrete slab, which can be obtained as a function of clear span and spacing between trusses (shear lag effect).

A_g is the gross area of bottom chord.

t is the thickness of concrete slab.

γ_c and γ_s are the resistance factors for concrete bearing and yielding of steel members, respectively.

Taking into consideration the force equilibrium analysis and assuming compressive concrete strength of 35 MPa and steel yield strength of 345 MPa, it is possible to compute the plastic neutral axis position, located 5 mm below the top surface of the slab. Furthermore, the required shear connectors resistance capacity, given as a function of the minimum value between $\mathrm{R_{cd}}$ and $\mathrm{R_{sd}}$, is approximately 76 kN.

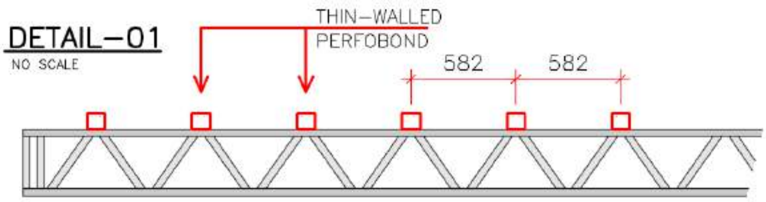


Figure 13 – Arrangement of thin-walled perfobond connectors

Figure 13 illustrates six thin-walled perfobond connectors placed between end support and the mid-section of the composite truss. It is clear that the required ultimate capacity of a single shear connector is 12,7 kN. The present results are an important issue observed during the design development, very much useful for the definition of the innovative cold-formed perfobond ribs.

4.3 Fabrication of push-out specimens

The fabrication of four push-out test specimens was carried-out in GypSteel Group headquarters and included: (a) automated cold-forming of stiffened U 89x40x12x1,25 mm sections, (b) assembling of steel members with 4,8 mm diameter self-drilling screws, (c) preparation of rebars and form-work and (d) pouring concrete. Push-out experimental program was fully conceived based on the Eurocode 4 recommendations (EN-1994-1-1:2002).

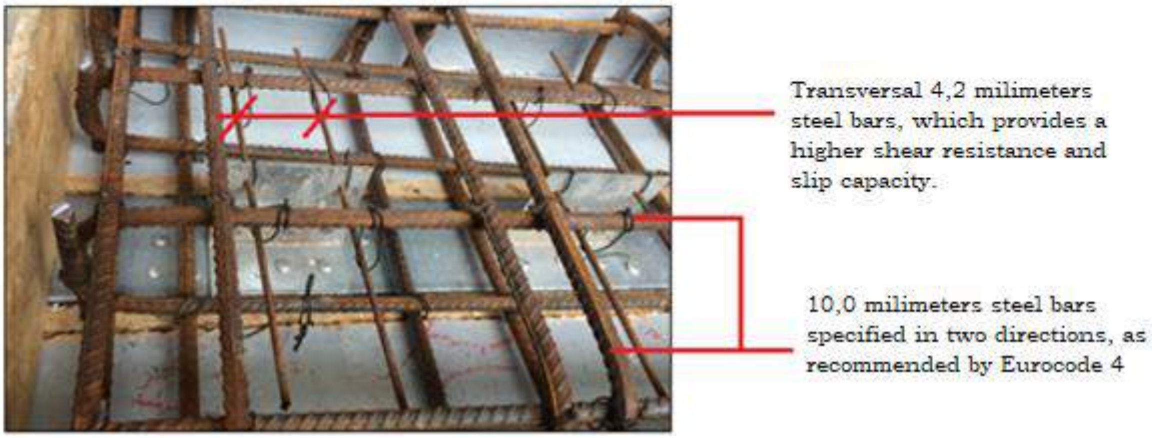


Figure 14 – Preparation of rebars and form-work

Figure 15 – Pouring concrete 1st step

As can be seen in Figure 14, 15 and 16, all the push-out specimens were prepared in horizontal position, in similar constructional condition to the composite steel floor system. In addition, as established by Eurocode 4, grease was applied on form-work and chord surfaces in order to avoid chemical bond with concrete.

In Figure 14, it can be observed the presence of two thin-walled perfobond shear connectors and four 4,2 milimeters rebars, inside each slab of the prototype. Additionally, there are 10,0 milimeters steel bars, specified to avoid the concrete shrinkage.

Figure 16 – Pouring concrete 2nd step and application of grease

Figures 15 and 16 show first and second concrete steps, casted in horizontal position and subject to air-cured process. The first slabs were poured seven days earlier than the other half and the concrete compressive standard tests were performed to evaluate its mechanical strength (150 mm diameter and 300 mm height cylinder specimens). The average concrete strength, obtained in the date of push-out tests, demonstrates a value of 39 MPa.

4.4 Experimental push-out tests

The behavior of Z thin-walled perfobond shear connectors, attached to chord member by self-drilling screws, were observed through the experimental tests of four push-out specimens. The obtained results indicate preliminary important conclusions about the applicability of such components in cold-formed structural systems.

All the tests were performed in the Structures and Material Laboratory of the Federal University of Rio de Janeiro and followed recommendations established by the Eurocode 4. Figure 17 presents the experimental arrangement, including instrumentation for data acquisition.

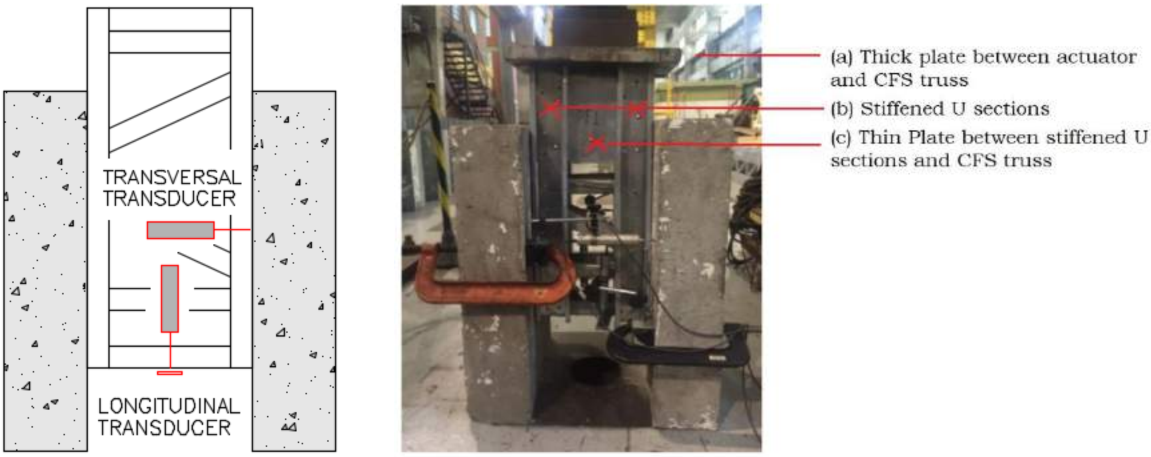


Figure 17 – Arrangement adopted during experimental investigations of prototype PF-1

The first push-out test included four displacement transducers measuring the relative displacement between the steel truss and the concrete slabs: two for longitudinal and two for transversal displacements. Figure 17 demonstrates important details regarding reinforcements located at the top of the truss, region of load application, including a (a) thick steel plate at the top of truss, (b) stiffened U sections and (c) thin plates between stiffened U sections and CFS truss.

A thick steel plate is placed between hydraulic actuator (250 kN capacity) and the CFS truss, in order to provide uniform distribution of forces. Additionally, the specimen shown in Figure 17 and Figure 18, called PF-1, was reinforced with two stiffened U profiles and a thin plate, positioned in both sides of the truss, to avoid local buckling effect.

The first push-out test was performed almost sixty days after concrete pouring. The test procedure was divided in two steps: (i) application of 25 cycles of loading/unloading of the prototype, ranging from 5 up to 40% of the expected ultimate load, and (ii) application of the compression until failure load. All the loading steps were programed with a constant rate of 0,005 mm/s (slip control mode).

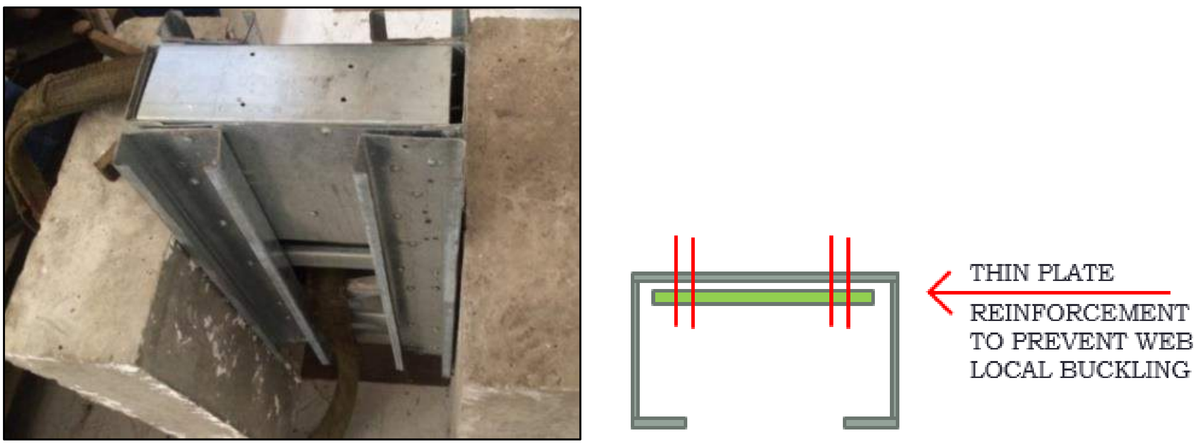


Figure 18 –Top face of push-out specimen

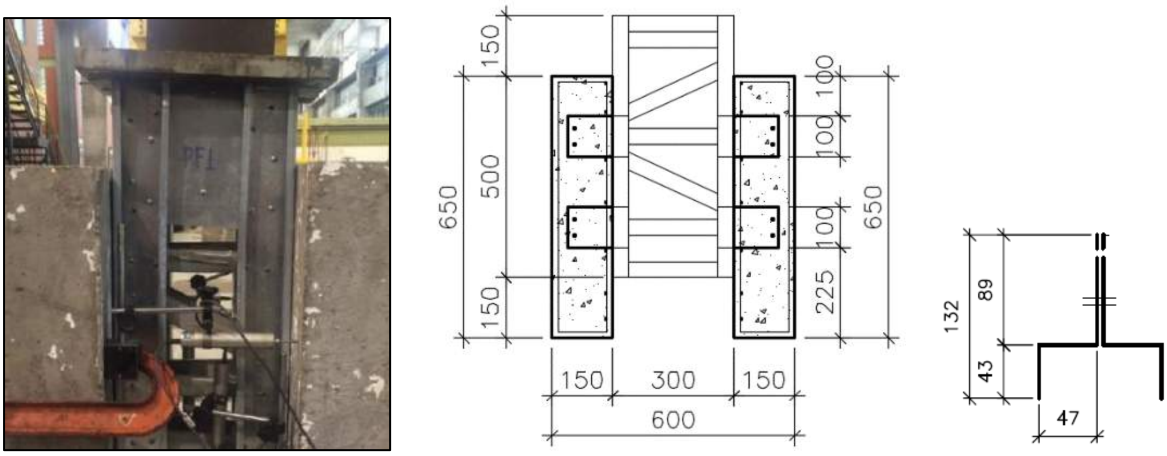


Figure 19 – Push-out arrangement of prototype PF-1

According to Eurocode 4 provisions, during the second step of loading the relative displacement between steel and concrete slabs is recorded, at least, until 80% of the ultimate (maximum) load after the push-out prototype collapses. In this condition, it is possible to verify the slip capacity of thin-walled perfobond ribs.

The obtained collapse load was 188 kN, taking into account the contribution of four thin-walled perfobond ribs (two at each chord). However, the observed failure mode was not associated to the shear connector, but occurred due to web local buckling of the chord.

Figure 20 shows the collapsed zone, located at the top of the prototype. The adopted reinforcements were not able to prevent local buckling of the CFS members. Even though, the mode of collapse do not correspond to shear connectors, important preliminary conclusions can be made.

The obtained value of 188 kN indicates that perfobond rib shear capacity is, approximately, almost four times higher than the required strength (as previously mentioned in section 4.2). The experimental strength of a single shear connector could be taken as 47 kN. Hence, it is evident that, in terms of resistance capacity, the thin-walled perfobond is (by far) adequate to provide composite behavior for the innovative steel truss and concrete one-way slab structural system.

Figure 22 shows the experimental results of relative displacements in longitudinal direction. The observed collapse load is 188 kN and the relative displacements, when push-out specimen was subject to 80% of its failure load, is equal to 10 mm.

In Figure 23, the results acquired by two transducers oriented in horizontal (transversal) direction show small relative displacements between the truss and slabs (uplifting effect), as expected.

Although the magnitude of slippage between steel and concrete might suggest a ductile behavior of the connection, it is important to emphasize that the results were affected by visible rotations of CFS diagonals (Figure 21) after peak load. In order to avoid such problem in other push-out tests, a new reinforcement was introduced in the other push-out specimens, as can be seen subsequently.

Figure 20 – Deformed configuration of specimen PF-1

Figure 21 – Visible rotations associated with diagonals, vertical and top of the prototype

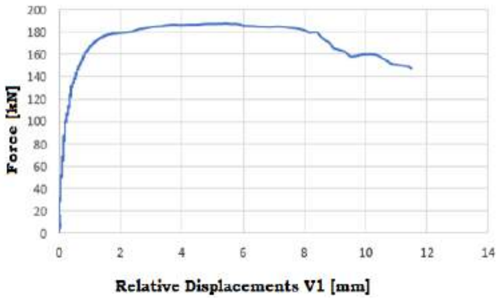


Figure 22 – Relative displacements in longitudinal direction PF1

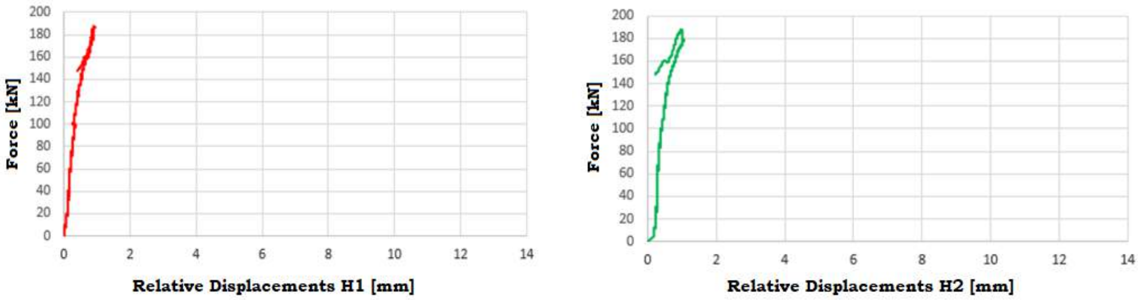


Figure 23 – Relative displacements in transversal direction PF1

The second push-out specimen, called PF2, was investigated after some adjusts to prevent local buckling at the top of the truss. In Figure 24, it is possible to observe that two reinforcements were adopted: (a) thin plate and (b) stiffened U sections.

Note that, in first place, the objective was to avoid local buckling at the web of the chord and try to see a collapse associated with the thin-walled shear connectors. Additionally, stiffened U sections were attached to the diagonals of the truss in order to prevent mistakes about results provided by displacement transducers.

After experimental investigation, the peak load was 194 kN and, therefore, thin-walled perfobond ribs were subjected to 48,5 kN. Similarly to PF1, it was observed that the mode of collapse was associated to a not desirable local buckling of the truss.

In Figure 25, the obtained data of vertical relative displacements are presented. The presence of stiffened U profiles avoided rotations of the diagonals and resulted in a more realistic behavior of slippage between steel truss and concrete slabs. The referred displacements observed, related to the peak load, was 2 milimeters (average value) and, therefore, considerably smaller than those presented by PF1.

In Figure 26, relative displacements in transversal directions is shown. The results, as expected, indicated small effects. During the experimental investigation, including at a ultimate load level, it is evident a similar behavior related to both sensors (H1 and H2), resulting in a slippage of 0,4 milimeters.

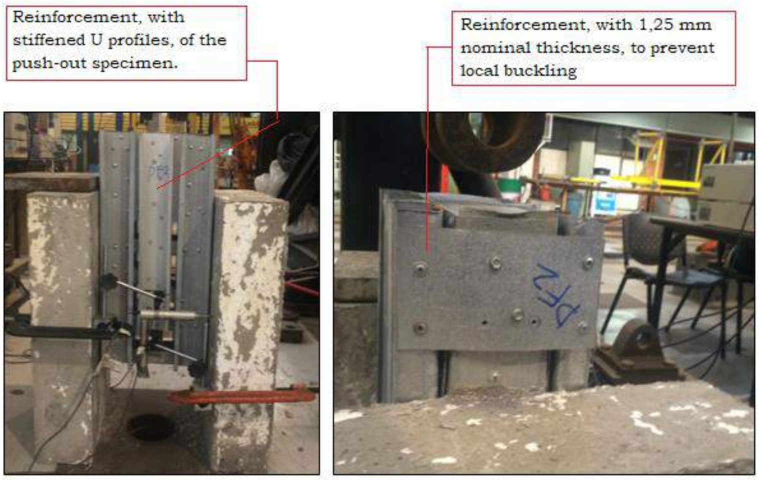


Figure 24 – Adjusts adopted to prevent local buckling at the top face of the prototype

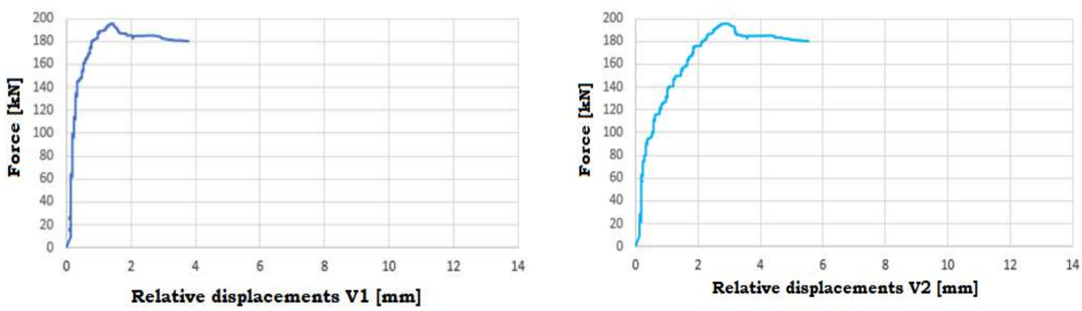


Figure 25 – Relative displacements in longitudinal direction PF2

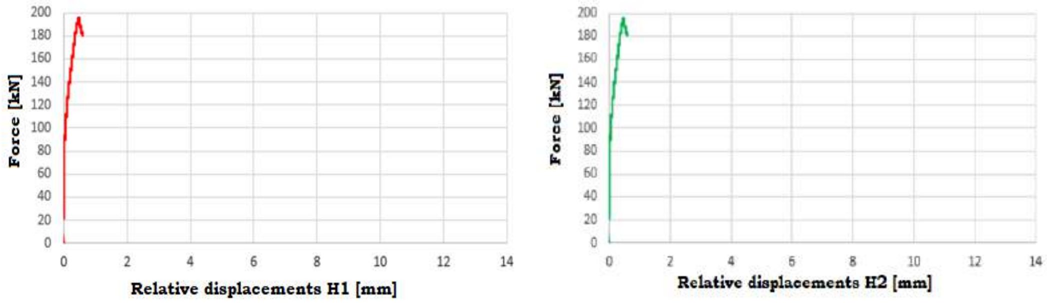


Figure 26 – Relative displacements in transversal direction PF2

Once previous results indicated modes of failure associated to local buckling and also ineffectiveness of adopted reinforcements, a last attempting to induce collapse to thin-walled perfobond ribs was performed in specimens PF3 and PF4.

As can be seen in Figure 27, the modifications adopted in push-out specimens PF3 and PF4 is characterized by a “cutoff” in the middle of the truss, conceived to eliminate the contribution of two thin-walled perfobond connector and prevent local buckling at the top of the prototype.

Figure 27 – Adaptations performed in prototypes PF3 and PF4

The adaptations indicated additional information about ultimate shear capacity of referred thin-walled perfobond connector. The results presented in Figure 28 showed a peak load of 120 kN and, therefore, 60 kN per perfobond rib, which is higher than shear resistance predicted by analytical solutions (Equations 2 and 3).

It is important to point out, however, that local buckling of the chord affected further conclusions about thin-walled perfobond ultimate capacity. After this preliminary study regarding a new perfobond solution, it is advised, for future works, that a different arrangement be adopted. In this context, if an experimental analysis of real thin-walled perfobond behavior is desirable, it is recommended a more compact steel beam (double stiffened U section, for example).

As can be seen in Figures 28, 29, 30 and 31, the modifications adopted in PF3 and PF4 produced a negative effect in terms of force-displacements investigation. The referred “cutoff” in the middle of the truss generated a high slippage between steel beam and concrete slab, affecting better interpretation of experimental results.

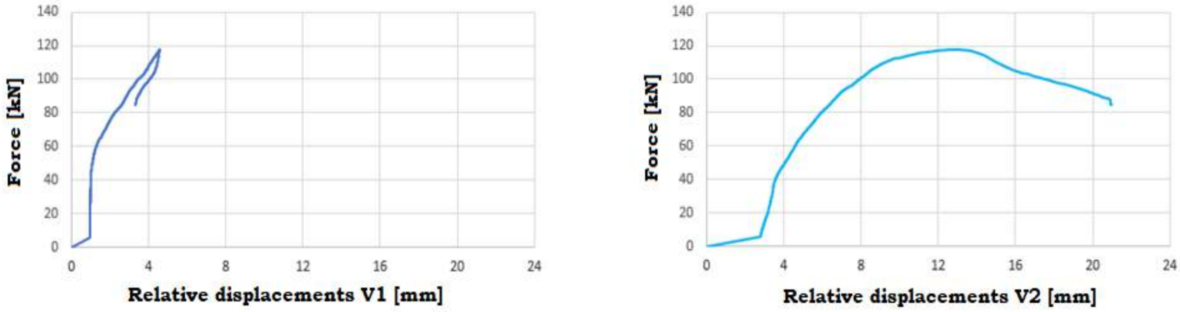


Figure 28 - Relative displacements in longitudinal direction PF3

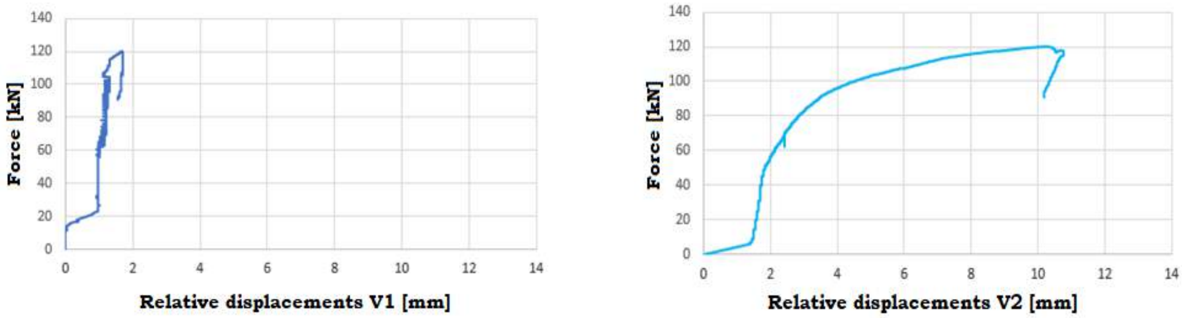


Figure 29 - Relative displacements in longitudinal direction PF4

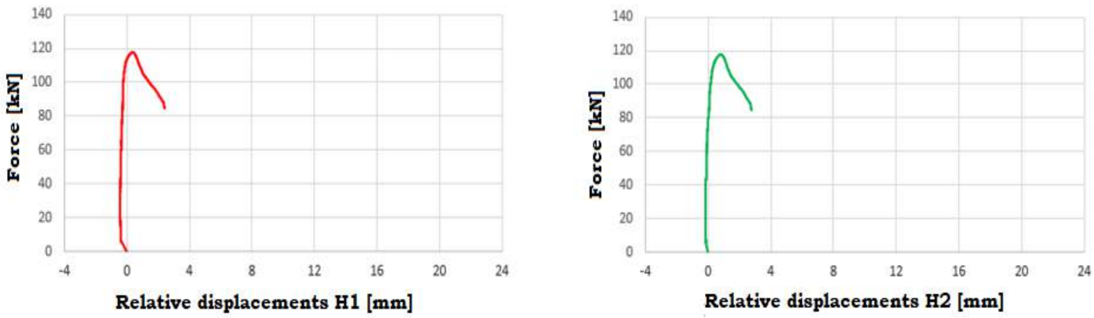


Figure 30 – Relative displacements in transversal direction PF3

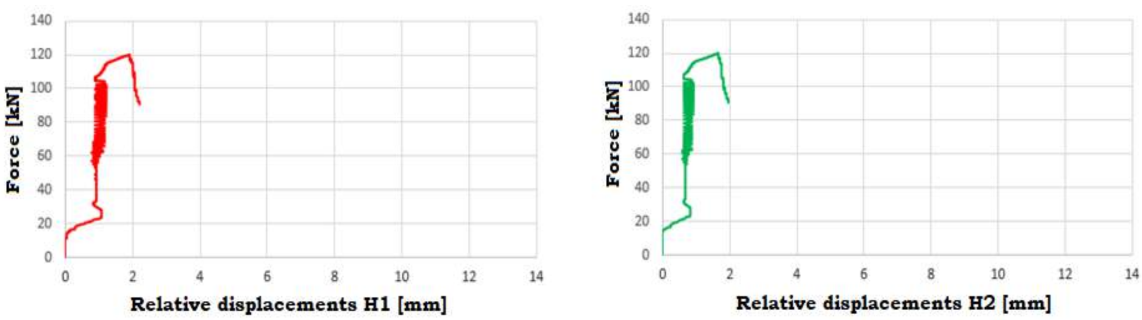


Figure 31 – Relative displacements in transversal direction PF4

Table 2 – Properties of perfobond, concrete and transversal bars adopted in push-out tests

number of holes	Diameter of holes [mm]	Height of ribs [mm]	Thickness of ribs [mm]	Average Concrete Strenght [MPa]	Area of transversal Reinforc. [mm2]
2	8,0	89,0	2,5	39,0	27,7

Table 3 – Comparison between experimental peak load and analytical solutions

Prototype	Peak Load [kN]	Number of Perfob. Ribs	Force (N) Per Rib [kN]	Equation 2		Equation 3	
				N_2 [kN]	N/N_2	N_3 [kN]	N/N_3
PF-1	188,0	4,0	47,0	56,8	83%	49,7	94%
PF-2	194,0	4,0	48,5	56,8	85%	49,7	97%
PF-3	120,0	2,0	60,0	56,8	106%	49,7	121%
PF-4	120,0	2,0	60,0	56,8	106%	49,7	121%

In Table 2 and 3, the properties of all push-out members and a comparison between experimental and analytical results are presented, respectively. Note that, in Table 2, the characteristics of the connector take into account only the vertical segments of twin Z cold-formed section. As a suggestion for future studies, concerning a parametrical evaluation of TWP, the contribution of all segments in contact with concrete (“T” section) should be exploited.

In Table 3, it is observed that the analytical formulations and experimental results (force per rib) are good related. Although the TWP ultimate load was not reached, the equations 2 and 3 are adequate, at least, to provide a preliminary estimation of shear capacity.

5 CONCLUSIONS

In this work, a new thin-walled perfobond connector and an innovative composite structural system were presented. The main objective was to present innovative concept for shear connector, which is composed by twin Z cold-formed sections, connected to the truss by self-drilling screws and designed to ensure composite behavior between thin-walled steel truss and one-way concrete slab.

Furthermore, preliminary conclusions, obtained from a series of push-out experimental tests performed in the Structures Laboratory (LABEST) of the Federal University of Rio de Janeiro (COPPE/UFRJ), were reported in this study, regarding shear resistance and slip capacity of the thin-walled perfobond shear connector.

The obtained results show important issues concerning the behavior of 1,25mm thick perfobond shear connector: (a) the referred connection is suitable for steel floor structural systems and (b) prediction of perfobond shear ultimate capacity by traditional formulations provides a reliable estimation for design procedures.

The experimental investigations of specimens PF1 and PF-2 showed a peak load of approximately 190 kN (47,5 kN per rib). On the other hand, prototypes PF-3 and PF-4 presented ultimate load of 120 kN (60 kN per rib). In all tests, the failure mode was associated to the local buckling of the truss, not affecting the shear connectors structural integrity.

Although the collapse of the shear connector was not observed, it is clear that the peak load, acting on each perfobond rib, is considerably higher than the required resistance capacity (12,7 kN per rib) and, therefore, suitable to ensure composite action between steel cold-formed trusses and one-way slabs. It is important to emphasize that the referred shear capacity is associated to compressive concrete strength of 35 MPa, steel yield strength of 345 MPa and the presence of six thin-walled perfobond.

Based on the preliminary experimental program, it seems that the traditional solutions (Equations 2 and 3) are applicable to the prediction of ultimate capacity of thin-walled perfobond ribs, connected by self-drilling screws. As can be seen in Table 3, experimental peak load results are well related to those predicted by referred analytical formulations.

Complimentary pushout tests are under way, with additional reinforcement of the steel trussed beam in order to prevent premature damage of the structure, enabling identification of the shear connector loading and deformation capacity.

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REFERENCES

AHN, J.H; LEE, C.G; WON, J.H; KIM, S.H. Shear resistance of the perfobond-rib shear connector depending on concrete strength and rib arrangement. Journal of Constructional Steel Research 66, 1295-1307, 2010.

CÂNDIDO MARTINS, J.P.S; COSTA NEVES, L.F; VELLASCO, P.C.G.S. Experimental evaluation of the structural response of Perfobond shear connectors. Engineering Structures 32, 1976-1985, 2010.

CEN, EN 1994-1-1. Eurocode 4: Design of Composite Steel and Concrete Structures, Part 1-1: General Rules and Rules for Buildings, Brussels, 2004.

MAZOZ, A; BENANANE, A; TITOUM, M. Push-out tests on a new shear connector of I-Shape. International Journal of Steel Structures, Vol 13, No 3, 519-528, 2013.

OGUEJIOFOR, E.C; HOSAIN, M.U. Numerical analysis of push-out specimens with perfobond ribs connectors. Computers and Structures, Vol 62, No 4, 617-624, 1997.

KIM, S.H; CHOI, K.T; PARK, S.J; PARK, S.M; JUNG, C.Y. Experimental shear resistance evaluation of Y-type perfobond rib shear connector. Journal of Constructional Steel Research 82, 1-18, 2013.

KIM, Y.H; CHOI, H; LEE, S; YOON, S.J. Experimental and analytical investigations on the Hat Shaped shear connector in the steel-concrete composite flexural member. International Journal of Steel Structures. Vol 11, No 1, 99-107, 2011.

VALENTE, I; CRUZ, P.J.S. Experimental analysis of Perfobond shear connection between steel and lightweight concrete. Journal of Constructional Steel Research 60, 465-479, 2004.

VIANNA, J.C; ANDRADE, S.A.L; VELLASCO, P.C.G; COSTA-NEVES, L.F. Experimental study of Perfobond shear connectors in composite construction. Journal of Constructional Steel Research 81, 62-75, 2013.

ZHENG, S; LIU, Y; YODA, T; LIN, W. Parametric study on shear capacity of circular-hole and long-hole perfobond shear connector. Journal of Constructional Steel Research 117, 64-80, 2016.