International Journal of Physical Sciences Vol. 7(22), pp. 2876-2890, 9 June, 2012
Available online at http://www.academicjournals.org/IJPS
DOI: 10.5897/IJPSX11.004
ISSN 1992 - 1950 ©2012 Academic Journals

Review

Various types of shear connectors in composite structures: A review

Ali Shariati, N. H. RamliSulong, MeldiSuhatril and Mahdi Shariati*

Department of Civil Engineering, University of Malaya, 50603 Kuala Lumpur, Malaysia.

Accepted 17 January, 2012

In this paper, an attempt has been made to review various types of shear connector in composite structures. This review tries to identify the shear connectors that are most relevant to composite structures and reviews representative journal publications that are related to this topic. It attempts to cover all types of shear connector. The article concludes with a discussion of recent applications of shear connectors in composite structures. Comparative studies, which have been conducted by several researchers, were covered to address the applicability and the efficiency of various shear connectors. The representative shear connectors for stud connectors as commonly used shear connectors in composite structures were discussed and a summary of their behaviour was included.

Key words: Shear connector, composite beam, headed studs, perfobond ribs, T-rib connector, oscillating perfobondstrips, waveform strips, channel connector, pyramidal shear connectors, non-welded connectors, INSA HILTI shear connector, rectangular-shaped collar connectors.

INTRODUCTION

Composite structures using normal weight concrete have been used since 1920 and there has been considerable use of composite structures for bridge construction from 1950 as a result of the research (Viest et al., 1997; Viest, 1956a, b; Hegger et al., 2005). Its main development in building structures in the last decade was an outcome of the basic design provisions introduced in the 1961 American Institute of Steel Construction (AISC) specification. The growth of these provisions were found in studies by Slutter and Dristroll (1965).

Shear connectors between concrete slabs and steel beams in composite construction can play an important role in the seismic response of a structure. They provide the necessary shear connection for composite action in flexure, and can be used to distribute the large horizontal inertial forces in the slab to the main lateral load resisting elements of the structure (Figure 1). During an earthquake, such shear connectors are subjected to reverse cyclic loading (Hawkins and Mitchell, 1984). This component enables the development of a composite action by assuring the shear transfer between the steel profile and the concrete deck (Vianna et al., 2009).

Floor systems that consist of a non-composite concrete slab over steel girders were often used in the construction of bridges before the 1970s. However, the current load requirements could not be fulfilled through the construction of bridges with such a system, thus, requiring many existing bridges to be replaced or strengthened. Connecting the existing concrete slab and steel girders is a potentially economic way to strengthen these floor systems as it allows for composite action to be developed. As opposed to the original non-composite condition, composite action allows the existing steel girder and concrete slab to act together more efficiently. In non-composite girders, the steel girders and the concrete slab act separately in flexure. Hence, by using shear connectors to connect the two structural components, the load-carrying capacity of the girders could be increased by more than 50% as compared to that of non-composite girders. By connecting the existing steel girder to the existing concrete slab, it allows the transfer of shear forces at the steel concrete interface, thus, enabling the benefits of composite actions to be achieved. Prior to casting the concrete slab, shear connectors are welded to the top of the steel girder in order to develop composite action in the construction of

*Corresponding author. E-mail: shariati@siswa.um.edu.my.

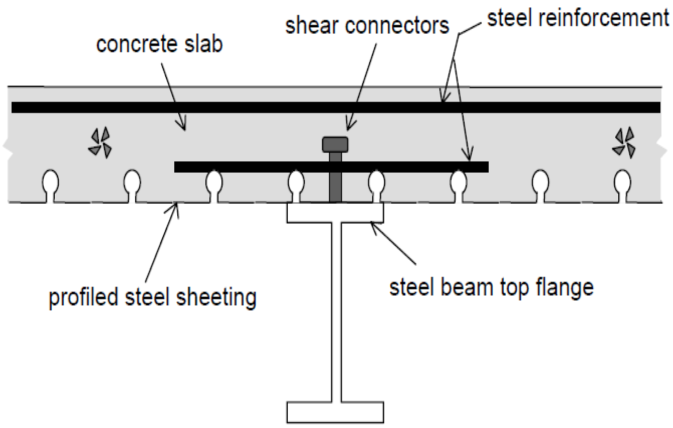

Figure 1. Shear connectors between concrete slabs and steel beams in a composite beam.

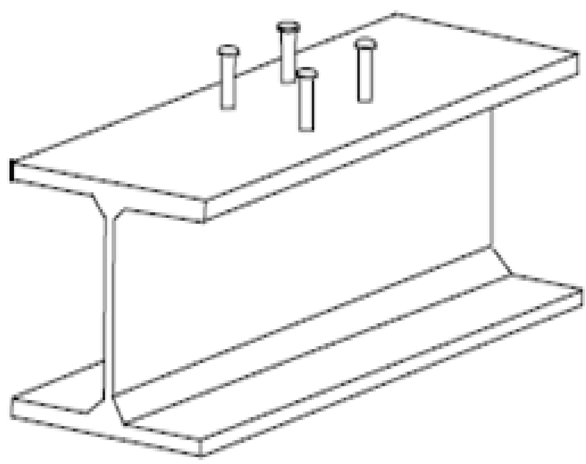

Figure 2. Head stud shear connector.

new bridges (Kwon et al., 2009).

The design of shear connectors is a vital aspect in the design of composite beams. Shear connectors are of many types, and according to the distribution of shear forces and functional dependency between strength and deformation, they are often categorized as rigid or flexible. For rigid shear connectors, shear forces are resisted through the front side by shearing, and in the proximity of ultimate strength its deformation is insignificant. Stronger concentrated stress in the surrounding concrete is produced by this type of connector, which results in either failure of the concrete or failure of the weld. Whereas, for flexible shear connectors, shear forces are resisted by bending, tension or shearing at the root, at the connection point of the steel beam, a point where upon reaching the ultimate strength values, such connectors are subjected to plastic deformation. Flexible shear connectors are more ductile and are not as prompt in terms of the manner of failure. Even with a lot of movement between the concrete slab and the steel beam, the shearing strength is maintained by the shear connector. Apart from depending on the strength of the shear connector itself, the shear strength and stiffness of the connection is also dependent on the resistance of the concrete slab against longitudinal cracking caused by high concentration of the shear force at each connector. Concrete resistance is a function of its splitting strength, which is directly related to the nature of concrete construction around the connector.

The experimental push-out tests provide current knowledge of the load-slip behaviour of the shear connectors in composite beams. Numerous researches on push-out tests, also called composite beam tests, have been conducted.

In order to ascertain the behaviour of different types of shear connectors, experimental tests have been done extensively. In this paper, an attempt has been made to review the different types of shear connector that can be found in composite structures.

HEADED STUDS

To resist horizontal shear and vertical uplift forces in composite steel-concrete structures, the most commonly used type of shear connector is the head stud. Also referred to as the Nelson stud (Figure 2), this type of connector contributes to the shear transfer and prevents uplift, as it is designed to work as an arc welding electrode, and, simultaneously, after the welding, acts as the resisting connector with a suitable head. As a result of the high degree of automation in the workshop or on site, this type of connector is commonly used worldwide. However, in structures submitted to fatigue, the use of this type of connector has some restrictions, the requirement for specific welding equipment and a high power generator on site for its use limits the utilization of such connectors. Another drawback is that the strength for concrete grades higher than C30/37 is normally governed by the strength of the steel cross section of the stud. Hence, higher concrete grades will not be advantageous for this connector device. Furthermore, it is practically impossible to automate the welding of headed studs (Zingoni, 2001).

Much research has been carried out on headed stud shear connectors and various equations have been proposed to estimate the strength of studs (Viest, 1956a; Ollgaard et al., 1971; Gelfi and Marini, 2002; Lee et al., 2005). Viest carried out the initial studies on stud shear connectors, where full-scale push out specimens were tested with various sizes and spacing of the studs. The push-out and composite beam tests were used in studies on stud shear connectors to evaluate shear capacities. In order to investigate the behaviour of headed shear stud connectors in solid slabs, an accurate nonlinear finite element model were developed by Ellobody (2002) and Lam and Ellobody (2005). Validation against test results and comparison with data specified in the current codes of practice, such as BS5950 (Standard, 1994) and AISC (AISC, 2005a), were carried out using the effective numerical model (Lam and El-Lobody, 2001). The results of the experiment conducted by these authors are comparable with the results obtained from the finite element analysis. The finite element model offered accurate predictions on the capacity of the shear connection, the load slip behaviour of the headed studs and the failure modes.

Ellobody (2002) conducted another finite element model by considering the linear and non-linear behaviour of the materials in order to simulate the structural behaviour of headed stud shear connectors. The use of the model in examining variations in concrete strength and shear stud diameter in parametric studies are also presented. Consequently, it was mentioned that the finite element results suggested that BS5950 (Standard, 1994) may overestimate the headed stud's shear capacity.

The experimental tests to assess the behaviour of the shear connection between the steel and lightweight concrete that were carried out at the University of Minho were described in another work (Valente and Cruz, 2004b). The behaviour of stud shear connectors embedded in engineered cementitious composites (ECC) was investigated (Li et al., 2006), while to examine the capacity of large stud shear connectors embedded in a solid slab, an accurate non-linear finite element model of the push-out specimen was performed (Nguyen and Kim, 2009).

The AISC (2005b), CSA (2001) and Eurocode (2004) standards currently provide design equations for the calculation of the resistance of a stud shear connector. The investigation of the shear capacity studs has been conducted thoroughly and tabulated values can be found in BS 5950: Part 3 (BSI, 1990) and BS 5400: Part 5 (BSI, 1983) as well.

The stud's root is provided to transmit the horizontal shear force acting at the steel-concrete interface, while the head is provided for preventing uplift of the slab. The cross-sectional area of a stud connector is directly proportional to its shear strength and its ultimate shear strength is influenced largely by the concrete's compressive strength and modulus of elasticity.

The stud connector capacity may be assumed to be the failure load divided by the number of studs (Kim et al., 2001). The stud connection capacity can be calculated from a slightly modified version of the equations in the Eurocode 4 (BSI, 1992), AISC (Highway) and CSA codes:

$$Q_k = Kr\phi_s A_s (f_c E_c)^{0.5} \leq \phi_s A_s f_u$$

where ϕ_s is the performance factor of the stud (engaged as 0.8); A_s is the cross-sectional area of the stud; f_c is the compression of the concrete; E_c is the modulus of elasticity for the concrete; f_u is the ultimate tensile strength of the stud; K and r are the empirical factors of connection resistance and the reduction factor for slabs including profiled steel sheeting, respectively.

The factor K is 0.46 in Eurocode 4 (BSI, 1992) and 0.5 in the American code (Highway). The factor r depends on the shape and orientation of the connector as well as the geometry of the steel deck.

The following equations in Eurocode 4 (ENV 1994-2, 1997) state the design strength of the stud shear connectors while welded automatically (Units are N and mm):

$$P_{Rd} = 0.8\ f_u(\frac{\pi d^2}{4})/\gamma_v)$$

$$P_{Rd} = 0.29\ \alpha d^2 \sqrt{f_{ck} E_{cm}}/\gamma_v$$

where d is the diameter of the stud; f_u is the ultimate strength of the steel; f_{ck} is the compressive strength of the concrete; E_{cm} is the elastic modulus of the concrete; the safety factor is 1.25.

The modulus of elasticity together with the compressive strength of the concrete are defined as critical variables that control the capacity of the stud shear connector. These two factors achieve the ultimate strength of weld as follows. The tensile strength of the connector material is the limitation of this equation (Kwon et al., 2009).

$$Q_u = 0.5 A_{sc} \sqrt{f_{ck}{}' E_c} \leq A_{sc} F_u$$

where Q_u is the connection capacity of the stud (N); A_{sc} is the cross section (mm^2); $f_{ck}{}'$is the compressive strength of the concrete (MPa); E_c is the elastic modulus of the concrete (MPa); F_u is the tensile strength of the stud (MPa).

According to previous researchers, there are several parameters that influence stud connectors. Among the most important are the shank diameter, the height of the stud and its tensile strength, as well as the compressive strength and modulus of elasticity of the concrete and direction of concrete casting. While evaluating the structural performance of the shear connection of the stud in precast deck bridges, the bedding height and the material properties of the filling material must also be taken into account (Shim et al., 2000, 2001). The behaviour of shear connections in composite beams with a full-depth precast slab was investigated in a study (Shim et al., 2001). Shim et al. (2001) also conducted a study on the design of connections in concrete and composite steel bridges with precast decks. In order to investigate the static and fatigue behaviour of large stud shear connectors for steel-concrete composite bridges, the push-out tests were performed by Shim et al. (2004) and Lee and Han (1998).

Due to the small space on the top flange, a dense distribution of shear connectors might create safety concerns for field workers. Thus, in composite bridges, various advantages and convenience can be obtained from the use of large studs, which are larger than 25 mm in diameter (Lee et al., 2005). Several experiments have been conducted on studs of more than 25 mm. A number of push-out tests on 25 mm studs were conducted by Hanswille (Sedlacek et al., 2003), and push-off tests on 31.8 mm studs were carried out by Badie et al. (2002), which showed that the equation given in the AASHTO LRFD bridge design specification can be safely used in determining the ultimate strength. An investigation on the stud shear connection in high strength concrete was performed by (Li and Krister, 1996; An and Cederwall, 1996) while a study on the use of shear stud strength at early concrete ages was conducted by Topkaya et al. (2004).

As stated in Eurocode 4 (Eurocode, 2004), in order to ensure the ductile behaviour of the composite girder, a slip capacity of at least 6 mm is deemed to be sufficient. However, there is no indication on the required slip capacity for shear connectors indicated in the current US design provisions for buildings and bridges (AISC, 2005a, Highway). The behaviour of welded shear studs under fatigue loading was examined by Slutter et al. (1967). In their study, the consequences of stress range, minimum stress, and load reversal on the fatigue life of welded shear studs were inspected.

ASTM A325 high-strength bolts were used as shear connectors for composite construction in previous research on post-installed shear connectors conducted (Dedic and Klaiber, 1984). Hungerford (2004), Schaap (2004) and Kayir (2006) examined the structural performance and constructability of eleven types of 19 mm diameter post-installed shear connectors. The test, which was conducted under high-cycle fatigue loading, only involved a limited number of post-installed shear connectors due to the time and cost considerations (Kwon et al., 2009). According to Kayir (2006), the fatigue strength of the post-installed shear connectors that do not require welding is significantly higher than that of welded shear studs. Kwon et al. (2009) reported the details of the complete study while Kwon et al. (2008) described the details of the application of this technique for the strengthening of an actual bridge.

Based on the reviewed papers on stud shear connectors some advantages and disadvantages of this type of shear connector can be summarized.

Disadvantages of stud connectors

Generally, this kind of connector presents non-ductile behaviour and it cannot undergo the large interfacial slip produced by the applied loads; severe crushing of the concrete occurs at the front of the connector's root seriously decreasing the modulus of the concrete. Breakdown of the shear connection can occur either by the stud shearing failure or by the crushing of concrete. The stress developed by the applied load on the shattering restraint of the lower surface of the concrete slab in contact with the steel flanges and the limitation of the concrete expansion due to the transverse reinforcement determine the strength of the connector. Thus, the performance of connectors can be improved by increasing the concrete strength or by reducing the bearing stress transmitted to the concrete at the root of the stud (Matus and Jullien, 1996). The occurrence of a certain amount of slip is required before composite action can be established for this type of shear connector.

Advantages of stud connectors

The advantages of headed stud connectors can be summarized as follows: fast welding, good anchor in concrete, the arrangement of reinforcement through the slab is easy, production of large scale size is easy, the standard dimensioned head is a resistance factor for slab uplift and they are practical for use in steel deckslabs. Four portions that are considered for load bearing of studs as suggested by Lungershausen (1988) include the concrete behind the weld collar, bending and shearing load-bearing capacity in the lower area of the connector shaft, tensile force in the connector shaft and the friction forces in the composite interface. There are almost no tensile forces acting on the shank in high strength concrete (Hegger et al., 2001).

PERFOBOND RIBS

In the late 1980s, the office of Leonhardt, Andrå and Partners developed a new type of connector called the perfobondrib (Leonhardt et al., 1987a), which was introduced in recognition of the unsatisfactory behaviour of shear studs resulting from fatigue problems caused by live loads on composite bridges. Developed in Germany, this connector includes a welded steel plate, with a number of holes (Figure 3) (Ahn et al., 2010). The flow of concrete through the rib holes formed dowels that provide resistance in both the vertical and horizontal directions. This shear connector is a viable alternative to the headed stud connector, as signified in the experimental studies conducted previously (Ahn et al., 2010) and recently (Kisa 2011 and Jumaat et al., 2011). This connector was initially used in building structures (Ferreira et al., 1998). The fact that it not only ensures the concrete steel bond, but also enables a better anchorage of the internal columns hogging moment has encouraged its adoption.

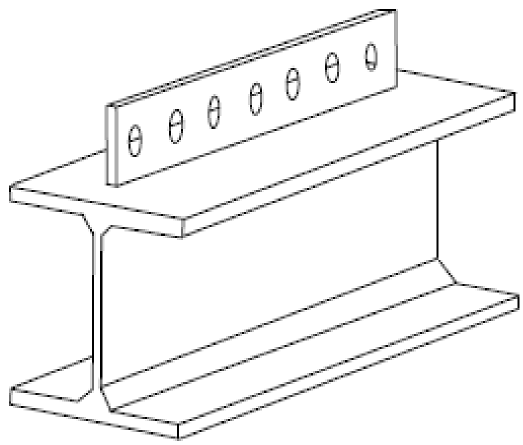

Figure 3. Perfobond ribs shear connector.

By passing these through the perfobond web holes or simply by being superimposed to the transverse reinforcing bars that are generally used on them will allow these bars to be anchored. A study done by Zellner (1987) indicated that a one metre length of perfobond connector is comparable to eighteen 22 mm diameter studs disposed in two lines or twenty four 19 mm diameter studs disposed in three lines.

Push-out and composite beam tests, as well as numerical simulations have been used in conducting studies on the shear capacity and behaviour of the perfobond rib (Veldanda and Hosain, 1992; Oguejiofor, 1994; Oguejiofor and Hosain, 1997; Hosaka et al., 2000; Ushijima et al., 2001; Medberry and Shahrooz, 2002; Valente and Cruz, 2004a; Nishido, 2005; Kim and Jeong, 2006; SAL and Ferreira, 2007; de Andrade, 2007; Vellasco et al., 2007; Al-Darzi et al., 2007a; Vianna, 2008; Jeong et al., 2009; Vianna et al., 2009; Candido-Martins et al., 2010). Equations for predicting the shear capacity of perfobond-rib shear connectors, as derived from these studies, were proposed. Through many researches, the shear-capacity equations of perfobond ribs are expressed.

Based on the work of Oguejiofor and Hosain (1997) (Verissimo, 2007; Vianna et al., 2009), a modified shear-capacity equation was proposed by Veríssimo et al. (2006b):

$$Q = 4.04\frac{n_{sc}}{b}h_{sc}t_{sc}f_{ck} + 2.37nd^2\sqrt{f_{ck}} + 0.16A_{cc}\sqrt{f_{ck}} + 31.85 \times 10^6(A_{tr}/A_{cc})$$

where Oguejiofor and Hosain (1997) also provided a prediction for the capacity of the perfobond rib connector:

$$q_u = 4.50htt_{sc}f_c' + 3.31\ nd^2\sqrt{f_c'} + 0.91A_{tr}f_{yr}$$

The following equation, which is used to quantify the resistance capacity of the shear connection, was established (Oguejiofor and Hosain, 1997) based on a regression analysis of the results of normal weight concrete specimens, with different connector geometries and reinforcement distribution. Three essential parameters were contributed from this expression: the concrete dowels passing through the perfobondrib holes, the concrete slab subjected to shear and the transversal reinforcement:

$$q_u = 0.590tA_c\sqrt{f_c'} + 1.233\ A_{tr}f_y + 2.871\ nd^2\sqrt{f_c'}$$

where f_c' is the compressive strength of concrete; f_y is the yield strength of steel; A_c is the shear area of concrete; A_{tr} is the area of transversal reinforcement that passes through the holes; *d* is the diameter of the perfobond rib holes; *n* is the quantity of perfobondrib holes.

Subsequent to the proposal of the above equation, more tests were conducted by the same author and a new expression was established to allow for better quantifying of the shear connection resistance capacity based on the results of his experiment. Since it refers to the local resistance under the perfobond connector, the first part of this new equation differs from the first part of the above equation,

$$q_u = 4.50htf_c' + 3.31\ nd^2\sqrt{f_c'} + 0.91A_{tr}f_{yr}$$

where *h* is the height of the rib; *t* is the thickness; A_{tr} is the total area of the transversal reinforcement. In another research, a comparative study was presented between the predicted and ultimate load of 16 tests performed with normal weight concrete specimens (Oguejiofor and Hosain, 1994; Oguejiofor and Hosain, 1997; Chatterjee and Kumar, 2009; Dogan and Roberts, 2010a, b). A more general equation for the calculation of this strength was proposed based on the push-out test results (Medberry and Shahrooz, 2002):

$$q_u = 0.747\ bh\sqrt{f_{ck}} + 0.413b_fL_c + 0.9\ A_{tr}f_y + 1.66n\pi\left(\frac{D}{2}\right)^2$$

where b is the thickness of the slab (mm); h is the length of the slab in front of the connector (mm); bf is the width of the steel section flange (mm); Lc is the contact length between the concrete and the flange of the steel section (mm).

For the evaluation of this strength, the following equation was proposed by Veríssimo et al. (2006a), which was derived based on the proposed equation (Oguejiofor and Hosain, 1994) and was also supported by push-out tests:

$$q_u = 4.04\frac{n_{sc}}{b}h_{sc}t_{sc}f_{ck} + 2.37nd^2\sqrt{f_{ck}} + 0.16A_{cc}\sqrt{f_{ck}} + 31.85 \times 10^6(A_{tr}/A_{cc})$$

where A_{cc} is the longitudinal concrete shear area per connector (mm^2).

The model expressed by the following equation was proposed by Al-Darzi et al. (2007b):

$$q_u = 255.31 + 7.62 \times 10^{-4} h_{sc} t_{sc} f_{ck} - 7.59 \times 10^{7} A_{tr} f_y + 2.53 \times 10^{-3} A_{sc} \sqrt{f_{ck}}$$

where A_{sc} is the area of concrete present at the connector holes.

In order to predict the contribution of individual holes to the perfobond connector resistance, (Ushijima et al. 2001) proposed an alternative formula:

$$q_{uho} = 3.38 D^2 \sqrt{\frac{t_{sc}}{D} f_{ck}} - 39$$

where D is the diameter of the hole in the shear connector (mm); t_{sc} is the thickness of the perfobond connector (mm); f_{ck} is the compressive strength of concrete in the cylinder (MPa).

Finally, based on a study conducted by Marecek et al. (2004) on two perforated connectors that were placed side by side, it was found that the overall resistance is less than the sum of the resistances of the individual connectors due to the interaction between the two connectors. They proposed the following equations in their study to predict this overall resistance (P_{double}) from the resistance of each individual connector (P_{rk}) that was placed alongside an axial spacing b (mm):

$$P_{Double} = k_d P_{RK}$$

$$K_d = 1.66 + \frac{b - 100}{14000} \leq 1.85$$

Although, specifying some reinforcing bars in the hogging moment region is common in order to avoid concrete cracking, the design of the connection does not normally take into consideration the extra resistance provided by its use (Wf et al., 1993). This connector aims to transfer the forces of the reinforcing bar directly to the column flange from the hogging moment region. The seated and double web angles are the other elements that exist in the internal and external connections (Vellasco et al., 2007). An undercut allows the achievement of the resistance to uplift. The placing of reinforcement is facilitated by perforated plates with open apertures. Machacek and Studnicka (2002) and Marecek et al. (2005) described the tests performed with rib connectors in which the O-form and C-form apertures were combined. Veríssimo (2004) described the advantages and the behaviour of the S-form perforated connector, and he also quantified its load and deformation capacity. As stated by Kraus and Wurzer (1997), the extreme local compression acting at the contact surfaces of the connector openings transmits the shear force from the steel strip to the concrete slab. This results in failure in the concrete. Following the failure of the concrete dowels, the transversal reinforcement pressed the concrete friction at the cracked concrete surfaces against each other, thus, allowing the connection to retain considerable shear strength (Zellner, 1987).

Oguejiofor (1994) presented the perfobond rib shear connectors' behaviour in composite floor systems with solid concrete slabs. Tests using perfobond connectors, shear studs, oscillating perfobondconnectors and T-connectors were performed (Galjaard and Walraven, 2000) with both normal weight and lightweight concrete. A test on normal shear studs and a number of modified shear studs, T-connectors and T-bulb connectors on normal weight concrete was conducted (Hegger et al., 2001). In addition, an extensive experimental study was carried out with different perfobond connector geometries on normal weight concrete (Oguejiofor and Hosain, 1997, 1994). There were also some tests regarding the use of perfobond connectors on normal weight concrete for building structures conducted (Ferreira et al., 1998), while several other tests with a modified perfobond connector were done (Machacek and Studnicka, 2002) using both normal weight and lightweight concrete.

Experimental studies on this particular type of shear connector have been conducted by quite a number of authors (Veldanda and Hosain, 1992; Oguejiofor and Hosain, 1992, 1994, 1997; Ferreira et al., 1998; Valente and Cruz, 2004a; Marecek et al., 2005; Kim and Jeong, 2006; Takami et al., 2005; Nishido, 2005), Iwasaki et al., 2005; Fukada et al., 2005; Cndido-Martins et al., 2010). Oguejiofor and Hosain (1992, 1994) made some early proposals of perfobond shear strength (Equation 1), taking into account the involvement of three fundamental parameters: concrete slab in shear, the concrete dowels formed in the perfobond holes as well as transverse reinforcement.

Using the same shear connection details as the perfobond ribs, the flexural and push-out tests were carried out and the longitudinal shear resistances were evaluated accordingly (Jeong et al., 2009). The results obtained from eighteen push-out tests on T-perfobond shear connectors were presented (Vianna, 2008). Studies on the behaviour of the perfobond connector have been carried out recently by several authors, mostly from push-out tests. Reference is made, among others, to the studies of Iwasaki et al. (2005), Medberry and Shahrooz (2002), Oguejiofor and Hosain (1992), Valente and Cruz (2009), Ushijima et al. (2001) and Hosaka et al. (2000). The conclusion drawn from the studies conducted by these authors is that several geometrical properties, such as the height, length and thickness of the plate, the number of holes, the concrete compressive strength, and the percentage of transverse reinforcement provided in

Figure 4. T-RIB shear connector.

the concrete slab influence the structural response of the perfobond connector.

The use of the perfobond geometry for thinner slabs, usually used in residential buildings, was adapted by Ferreira (Vellasco et al., 2007) in which the contributions to the overall shear connector strength from the concrete cylinders formed through the shear connector holes and from the reinforcement bars in shear were isolated. The studies done by Zellner (1987), Machacek and Studnicka (2002), Vellasco et al. (2007), Fink and Petraschek (2007), Gundel and Hauke (2007), Hechler et al. (2008), Hegger and Rauscher (2007), and Veríssimo et al. (2006) can be referred to.

The description and analysis on the properties of steel to concrete connection were made possible through the experimental works with perfobondand rib connectors, as carried out by Oguejiofor and Hosain (1994), Machacek and Studnicka (2002), Medberry and Shahrooz (2002), Vianna et al. (2008), Ferreira (2000), Galjaard and Walraven (2001), and Poot and Eligehausen (2001) as well as the studies developed by Hegger et al. (2001) and Galjaard and Walraven (2000) with headed studs and T-connectors.

The principal disadvantage of this type of shear connector lies with the placement of the transversal bottom slab reinforcement, which is often very difficult. Such connectors have high fatigue resistance, a high shear resistance capacity, and are easy to install due to the shape of the ribs (Leonhardt et al., 1987b). Consequently, by using concrete dowels, a concrete end-bearing zone, and transverse rebars in the rib holes, this connector resists vertical uplift forces and horizontal shear at the steel concrete interface (Leonhardt et al., 1987a). The fact that it also enables a better anchorage of the internal columns hogging moment reinforcing bars apart from ensuring the concrete steel bond is the determining factor for the adoption of such connectors. By passing the reinforcing bars through the perfobond web holes, anchorage on such bars can be developed. Similarly, simply by superimposing these bars to the transverse reinforcing bars that are generally used on them could also provide anchorage. Issues that involve particular structural, technological or economical needs of specific projects are interrelated with the motivation of developing new products for the shear transfer in composite structures.

T-RIB CONNECTOR

In the scope of a study on perfobond connectors, Vianna et al. (2009) presented an alternative connector for headed studs, called the T-perfobond (Figure 4). The author also provided a comparative study between the behaviour of these connectors and a limited number of T-perfobond connectors. By adding a flange to the plate, which acts as a block, the derivation of this connector from the perfobond connector was created. The need to combine the large strength of a block type connector with some ductility and uplift resistance arising from the holes at the perfobond connector web is a motivating factor for the development of this T-perfobond connector.

In order to prevent a premature loss of stiffness in the connection, the T-rib connector detail should minimize the prying action effect (Ferreira, 2000). As leftover rolled sections can be used to produce the T-rib connectors, it could reduce cost and minimize welding work. The four steps involved in the fabrication process of the T-rib connectors: (i) initial profile, (ii) web holes, (iii) flange holes, (iv) opposite flange saw cut are as shown in Figure 4.

For similar longitudinal plate geometries, the resistance and stiffness of T-perfobond connectors are higher than that of perfobond connectors. In addition to this advantage, the use of T-perfobondc onnectors offers benefits in terms of saving material and labour, as they are produced with ordinary laminated I or H sections.

OSCILLATING PERFOBONDSTRIPS

As compared to the headed studs and T-shape connectors, this type of connector has larger load capacity. However, due to the fast drop of the load capacity after the peak, the performance of this connector in the case of ordinary strength and normal weight concrete is rather disappointing. Nonetheless, the absence of such behaviour when they are in use in lightweight concrete, concrete with fibres or high strength concrete allows the oscillating perfobond strips connectors (Figure 5) to perform well (Rodera, 2008).

The difference in the failure modes for lower and higher

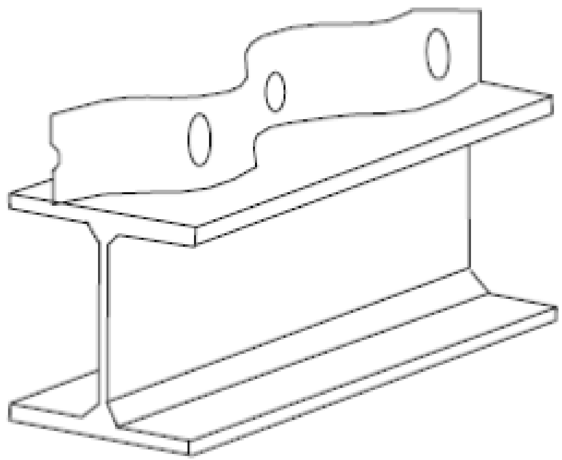

Figure 5. Oscillating-perfobondstrip shear connector.

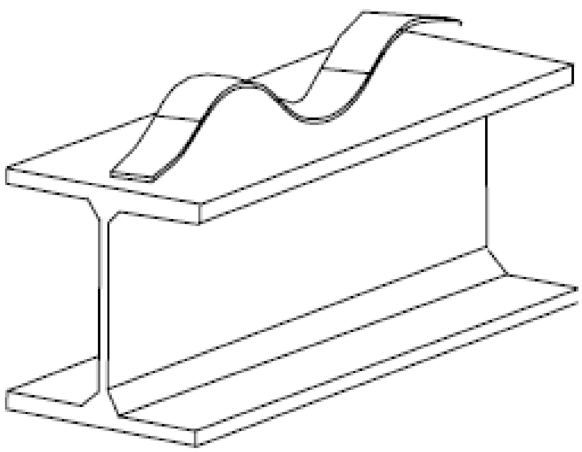

Figure 6. Waveform-strip shear connector.

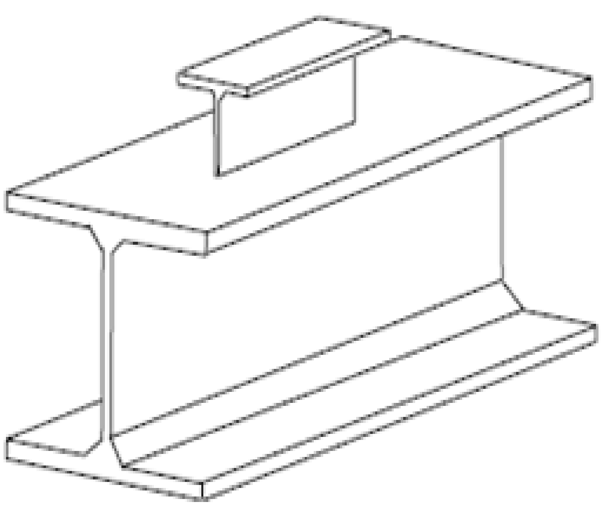

Figure 7. T shear connector.

concrete strength for oscillating perfobond strip connectors should be taken into consideration. The addition of steel fibres to the concrete reported a very positive effect.

WAVEFORM STRIPS

The objective of the curved form is to improve the transfer of force between the steel and the surrounding concrete as opposed to a straight connector. It is however recognized that it would be more difficult to weld using conventional automated welding equipment. The strips are welded to the HE-section with two fillet welds of 5 mm waveform strip with a width of 50 mm, a thickness of 6 mm and bend in 2 waves with amplitude 110 mm; Figure 6. Although the strip is meant to be welded using point weld equipment, such equipment with sufficient capacity is very scarce, and it is even doubtful whether the connector could be successfully welded using this equipment (Galjaard et al., 2001).

T-CONNECTORS

This connector is a section of a standard T-section welded to the H or I section with two fillet welds (Figure 7).T-connectors evolved from the observation by Oguejiofor (1997) that a large part of the bearing capacity of a perfobond strip was the result of the direct bearing of the concrete at the front end of the (discontinuous) perfobond strip. Therefore, a T section, which has a larger cross section than a single strip, and by its shape could prevent vertical separation between the steel-section and the concrete, seemed a good alternative.

The behaviour of the T-connector is very favourable. The beating stress on the front of the T is very high, as a result of the relatively small area. Local concrete crushing occurs, which results in a quasi-plastic performance (Zingoni, 2001).The load capacity for T-connectors is similar to that of the oscillating perfobondstrip, however, the ductility of these connectors is much larger (Rodera, 2008). When used in concrete with fibres, lightweight concrete or a higher strength concrete, there is a notable increase in the load capacity and ductility of this type of connector. In the case of the T-shape connectors, the strength of the connector itself is vital and the concrete is no longer decisive. Disregarding the perfobond strip, the resistance characteristic of the T-shape connectors is considered the highest and its failure mode varies according to different concrete strengths.

CHANNEL CONNECTOR

Channel connectors might not need inspection procedures, such as bending test of headed studs, due to

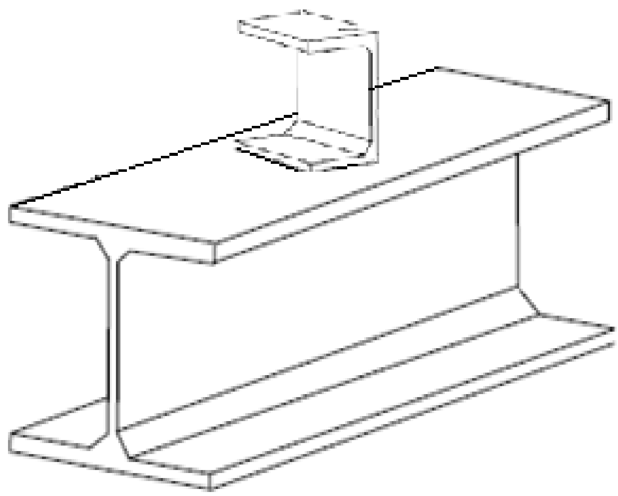

Figure 8. Channel shear connector.

the highly reliable conventional welding system used in the welding of these connectors. The load carrying capacity of a channel shear connector is higher than that of a stud shear connector. This enables replacement of a large number of headed studs with a few channel connectors (Maleki and Bagheri, 2008a). Viest et al. (1952) reported on the test results of full size and push-out specimens. The focus of this preliminary study was to understand the channel shear connectors' behaviour and to evaluate whether the use of channels as shear connectors is feasible (Figure 8). Slutter and Driscoll (1962) reported the results of another experimental study carried out at the Lehigh University concerning shear connectors. The results of 41 push-out specimens tested by the mentioned research constituted the basis from which the equations included in the American Institute of Steel Construction specification (AISC, 2005) (AISC, 2005b) and the Canadian standard (CSA, 2001) for the strength of channel shear connectors embedded in a solid concrete slab were derived.

In order to assess the accuracy of the design code equations for the strength of channel shear connectors, an experimental study using specimens with different channel sizes and lengths under monotonic loading was conducted (Pashan, 2006).Several researchers (Maleki and Bagheri, 2008a, b; Güney and Kuruşçu, 2011; Jumaat et al., 2011; Shariati et al., 2010 a, b; 2011 a, b) presented the behaviour of channel shear connectors embedded in a solid concrete material slab based on an experimental study conducted under monotonic and low-cycle fatigue loading and proposed an effective numerical model using the finite element method to simulate the push-out test of channel shear connectors.

A test was carried out on push-out specimens made of plain concrete, reinforced concrete (RC), fibre reinforced concrete (FRC) and engineered cementitious composite (ECC). Based on the results, the reversed cyclic shear strength of most specimens is lower than their monotonic strength by about 10 to 23%. The results also indicated that the shear strength and load-displacement behaviour of the specimens is slightly affected by the use the polypropylene fibres (FRC specimens). However, a considerable increase in ultimate strength and ductility of channel shear connectors was achieved by the use of the polyvinyl alcohol fibres (ECC specimens) (Maleki and Bagheri, 2008a).

A validation against experimental test results and a comparison with data given in North American design codes was carried out for additional research on the shear capacity of channel shear connectors embedded in a solid reinforced concrete slab under monotonic loading using the finite element model. To investigate the variations in concrete strength, channel dimensions and the orientation of the channel, parametric studies using this nonlinear model were performed. It was found that to determine the ultimate strength of channel shear connectors, the significant parameters include the strength of concrete, the web and flange thicknesses of the channel and the length of the channel, whereas the height of the channel section was regarded otherwise. Moreover, a change in the stiffness and the ultimate strength of the shear connector can be caused by changing the orientation of the channel (Maleki and Bagheri, 2008b).

In another research, Maleki and Mahoutian (2009) investigated the capacity of channel shear connectors embedded in normal and polypropylene concrete both experimentally and analytically. Before a prediction for shear capacity of channel connectors in polypropylene concrete could be reached, an extensive parametric study was performed. An equation was suggested for the shear capacity of these connectors when used in polypropylene concrete, which is to be included in design codes.

Viest et al. (1952), Pashan (2006), Ollgaard et al. (1971), Viest (1960) and Johnson (1970) reported on a literature review of composite beam research from 1920 to 1958 and 1960 to1970 for stud and channel shear connectors that are embedded in normal concrete.

The results of the push-out test showed that the strength of the composite system can be affected by other factors apart from the concrete strength, which include flange thickness, web thickness and channel length. Several equations for obtaining the channel shear connector capacity were proposed based on these investigations. Years later, building codes adopted some of these equations. The current Canadian code (NBC, 2005), for instance, suggests the use of the following equation for the calculation of the strength of a channel shear connector embedded in a solid concrete slab.

$$Q_n = 36.5(t_f + 0.5t_w)L_c\sqrt{f_c} \qquad (1)$$

Where:

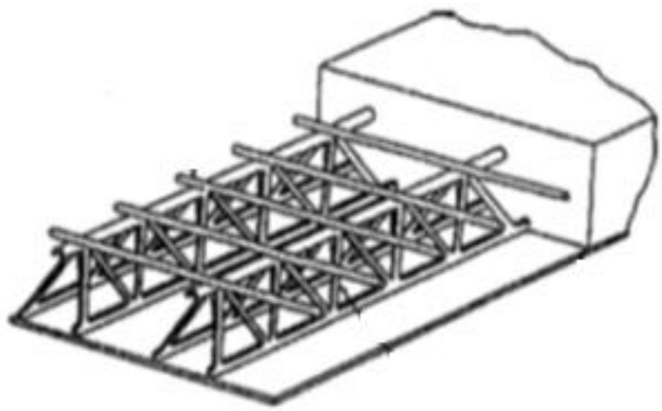

Figure 9. Pyramidal shear connector.

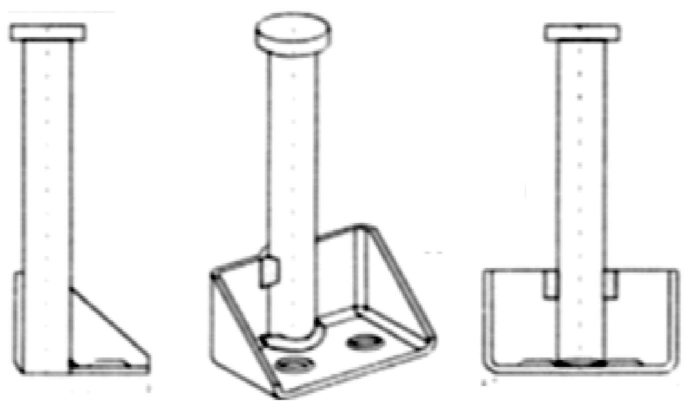

Figure 10. Non-welded shear connector.

Qn is the nominal strength of one channel shear connector (N); tf is the flange thickness of the channel shear connector (mm); tw is the web thickness of the channel shear connector (mm); Lc is the length of the channel shear connector (mm); fc is the specified compressive strength of the concrete (MPa).

For the capacity of channel connectors embedded in polypropylene concrete, the following revision to the Canadian code equation was suggested by Maleki and Mahoutian (2009).

$$\mathbf{Q_n = 27.2\ (t_f + 0.5t_w)L_c\sqrt{f_c}}$$

WhereQn, is in Newtons, tf,tw, Lc are in mm, and fc is in MPa units.

PYRAMIDAL SHEAR CONNECTORS

Sufficient bending strength and flexural rigidity for loads during and after construction is expected from a steel plate-concrete composite slab with pyramidal shear connectors (Figure 9). A TSC composite slab, which is composed of a bottom steel deck and concrete through pyramidal shear connectors could also be one of them (Lee and Han, 1998). The fatigue problem should play a significant role in design when such a TSC composite slab is applied to a bridge deck subjected to traffic loads. In particular, the fatigue strength of the thin bottom plate may be reduced through the welding of shear connectors (Matsui, 1984).

NON-WELDED CONNECTORS

A new non-welded shear connector, which is fixed by fastening pins using a powder-actuated tool, was developed following the difficulties of welding shear studs through profiled sheeting on site. Composite beams and push-out specimens, with and without profiled sheeting, were used in testing non-welded connectors (Crisinel, 1990).

Cold-formed from mild steel, this new connector (Figure 10) is L-shaped and two hardened steel fastening pins, which are driven through the connector and into the flange of the steel beam using a powder-actuated tool were used to fix the foot of the connector to the flange of the steel beam (Crisinel, 1990).

The behaviour of these connectors is ductile and resembles that of the stud shear connector, as found in the studies conducted on them based on push-out tests and beam tests with and without profiled sheeting. Providing the connectors are positioned correctly, the strength reduction of the non-welded connectors caused by the presence of profiled sheeting can be estimated with the same formula that has been developed for the shear studs (Crisinel, 1990).

This device has been specially designed to diminish and redistribute the bearing stresses transmitted to the concrete and to sustain non-linear deformations without inducing heavy damage on the steel-concrete connection.

INSA HILTI SHEAR CONNECTOR

With the principal objective of breaking the speed barrier by using modern fixing techniques, such as the multi-purpose Hilti cartridge-operated gun, the author reports on the development of a new shear connector for wood concrete mixed structures (Mungwa et al., 1999). This shear connector possesses higher rigidity, ductility and ultimate strength,as indicated in the test results. The INSA Hilti shear connector (Figure 11) is the innovative feature of the new shear connector. When subjected to alternating loads, the mechanism of shear transfer at the wood concrete interface for traditional dowel-type connectors, such as nails and screws, are notorious for shearing the wood along the grain fibres. Whereas failure of the wood in front of the connector causes the failure of

Figure 11. INSA HILTI shear connector.

non-dowel-type connectors, such as rigid ring connectors. The failure pattern is very much ductile despite the fact that the new connector is tubular. The connector was subjected to a push-out test, which is normally referred to as a local test (Mungwa et al., 1999).

Load-displacement curves are drawn from some results of the related experimental campaign carried out in reference to the steel connectors. Corresponding to the extraction of the screws from the beams, the tests were performed up to collapse. Three different types of shear connectors developed by the Hilti Company are presented (Mungwa et al., 1999) in Figure 11; the results of the push out tests carried out are also shown. The three types of shear connector are: (A) Hilti HVB connector, a flexible connector, which is commonly used to connect concrete slabs to steel beams; (B) Hilti Tubular connector, a tubular connector with a drilled head, where the holes are filled by mortar in the timber-concrete connection systems; and (C) INSA-Hilti connector, a hollow cylinder made of galvanized heat-treated anti-corrosion steel with varying cross-section size and wall thickness in the part which penetrates into the wooden matrix. The largest shear capacity is exhibited by the type B connection while the largest ductility is displayed by the type C connection, as shown in Figure 11.The notched shear key/anchor connector, which was initially developed by Natterer et al. (1996), is a connection system that can be inserted within the group (c) (Natterer et al., 1996). It consists of a steel dowel glued into a tapped pilot hole in the wood by an adhesive (Gutkowski et al., 2008). The HBV-system is another connection type, which belongs to group (d) or group (b), if it is continuous or discontinuous, respectively (Bathon and Graf, 2000). It consists of a steel mesh glued to the timber beam on one half and immersed into the concrete slab on the other one (Figure 11). This system provides a connection between the timbers and concrete that is stiff in the elastic range and ductile in the plastic one. In Figure 11 the HBV system is compared to other types of shear connector in terms of load-displacement curves rationally caused by the perforations for the installation of traditional connectors, an innovative type of connection, which basically consists of steel collars, surrounding the wooden stock, equipped with connectors (Mazzolani et al., 2005) is conceived. A high level of reversibility requirements is satisfied by this kind of device.

RECTANGULAR-SHAPED COLLAR CONNECTORS

This connection device consists of a collar composed of two or more parts, astride the timber beam, bolted together at adjacent wings (Figure 12). At the collar-beam interface, a rubber layer is interposed. The superior wings of the collar or a steel stud, purposely welded to the collar in the upper part, which are immersed in the concrete cast, guarantees the slipping action transmission. The force-slip relationship is used to discuss the connection behaviour (Faggiano et al., 2009).With the collaboration of the DIST (Department of Structural Engineering) of the University of Naples “Federico II” (Professor F. M. Mazzolani coordinator), the use of collar connectors at the DECIVIL (Department of Civil Engineering and Architecture) of the Superior Technical Institute in Lisbon (Esposito 2006), is developing. In the framework of the international research project PROHITECH (Earthquake Protection of Historical Buildings by Reversible Mixed Technology), research activities have already started.

CONCLUSION

An attempt has been made to review various types of shear connector in composite structures. The review concludes with a discussion of recent applications of shear connectors in composite structures. Despite being commonly used to transfer longitudinal shear forces across the steel concrete interface, the headed stud shear connectors have some disadvantages and difficulties to be used in composite beams. Other alternatives

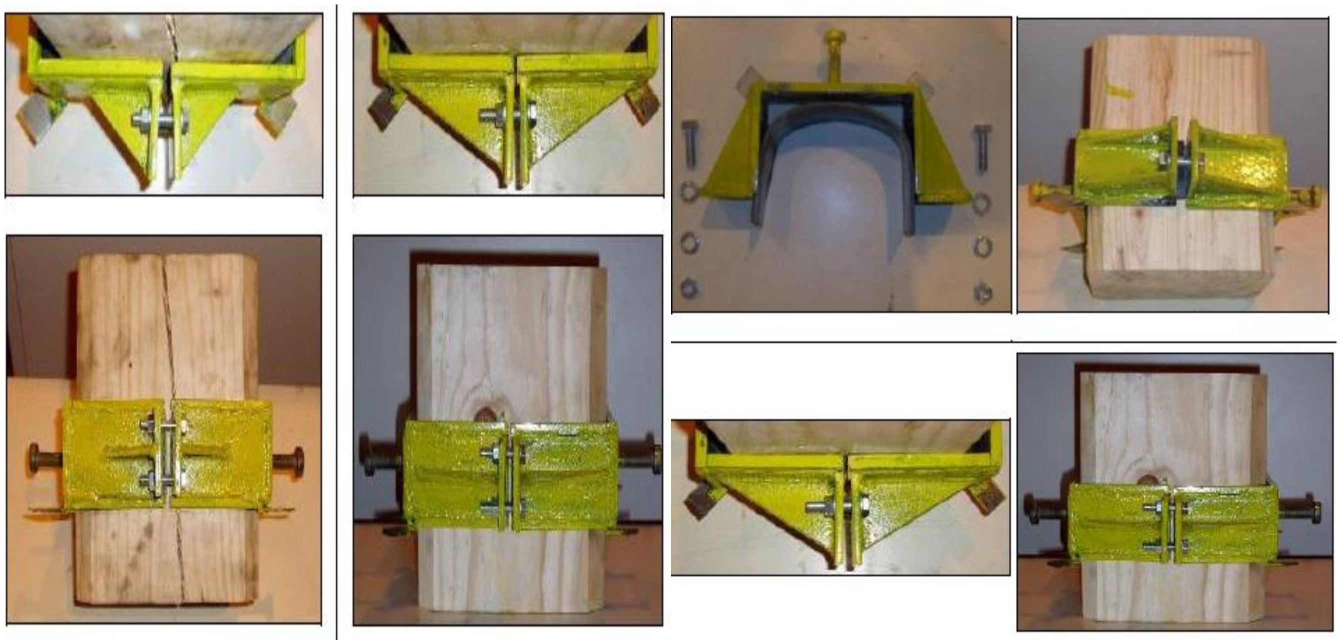

Figure 12. Rectangular-shaped collar shear connector.

to stud shear connectors are presented and discussed.

The following general conclusions can be drawn with respect to the connector devices:

1) In order to eliminate some of the problems and difficulties associated with standard shear studs, the perfobond shear connector has been developed. To further examine the behaviour of composite members utilising the perfobond shear connectors, a coordinated experimental and analytical study was carried out. This study also serves as a means to develop a reasonably simple, yet reliable design equation. Other types of profobond shear connectors are oscillating perfobond strips waveform.
2) To combine the large strength of a block type connector with some ductility and uplift resistance arising from the holes at the perfobond connector web, T-perfobond connectors were introduced. For similar longitudinal plate geometries, the resistance and stiffness of this type of connector are generally higher than that of the perfobond connectors. Moreover, the production of these connectors with ordinary laminated I or H sections minimizes labour and material usage, thus, offering a competitive advantage.
3) The load capacity of oscillating perfobond strip connectors when compared to that of the headed studs and T-shape connectors is generally larger. However, due to the fast drop in the load capacity after the peak, it portrays unsatisfactory performance when used in the case of ordinary strength and normal weight concrete.
4) The objective of the curved form is to improve the transfer of force between the steel and surrounding concrete so that it is better than a straight connector. However, it is recognized that it would be more difficult to weld using conventional automated welding equipment. The strips are welded to the HE-section with two fillet welds of 5 mm. The waveform strips havea width of 50 mm, a thickness of 6 mm and bend in 2 waves with amplitude 110 mm. The strip is meant to be welded using point weld equipment, however, such equipment with sufficient capacity is very scarce, and it is doubtful whether the connector could be successfully welded using this equipment (Galjaard et al., 2001).
5) The behaviour of the T-connector is very favourable. The beating stress on the front of the T is very high, as a result of the relatively small area. Local concrete crushing occurs, which results in aquasi-plastic performance. This connector is a section of a standard T-section welded to the H or I section with two fillet welds. T-connectors evolved from the observation by Oguejiofor (1997) that a large part of the bearing capacity of a perfobond strip was the result of the direct bearing of the concrete at the front end of the (discontinuous) perfobond strip. Therefore, a T section, which has a larger cross section than a single strip, and by its shape could prevent vertical separation between the steel-section and concrete, seemed a good alternative.
6) The behaviour of the T-connector is very favourable. The beating stress on the front of the T is very high, as a result of the relatively small area. Local concrete crushing occurs, which results in a quasi-plastic performance (Zingoni, 2001).The load capacity for T-connectors are similar to that of the oscillating perfobond strip; however,

the ductility of these connectors is much larger. When used in concrete with fibres, lightweight concrete or a higher strength concrete, there is a notable increase in the load capacity and ductility of this type of connector. In the case of the T-shape connectors, the strength of the connector itself is vital and the concrete is no longer decisive. Disregarding the perfobond strip, the resistance characteristic of the T-shape connectors is considered the highest and its failure mode varies according to different concrete strengths.

7) Channel connectors might not need inspection procedures, such as bending test of headed studs due to the highly reliable conventional welding system used in the welding of these connectors. The load carrying capacity of a channel shear connector is higher than that of a stud shear connector. This enables replacement of a large number of headed studs with a few channel connectors.

8) Cold-formed from mild steel is L-shaped with two hardened steel fastening pins, which are driven through the connector and into the flange of the steel beam using a powder-actuated tool were used to fix the foot of the connector to the flange of the steel beam (Crisinel, 1990).

9) INSA Hilti shear connector with the principal objective of breaking the speed barrier by using modern fixing techniques, such as the multi-purpose Hilti cartridge-operated gun. This shear connector possesses higher rigidity, ductility and ultimate strength. When subjected to alternating loads, the mechanism of shear transfer at the wood concrete interface traditional dowel-type connectors, such as nails and screws, are notorious for shearing wood along the grain fibres.

10) Rectangular-shaped collar connector consists of a collar composed by two or more parts, astride the timber beam, bolted together at adjacent wings. At the collar-beam interface, a rubber layer is interposed. The superior wings of the collar or a steel stud, purposely welded to the collar in the upper part, which are immersed in the concrete cast guarantees the slipping action transmission.

11) Pyramidal shear connector which is a welding shear connector may reduce the fatigue strength of the thin bottom plate.

REFERENCES

Ahn J, Lee C, Won J, Kim S (2010). Shear resistance of the perfobond-rib shear connector depending on concrete strength and rib arrangement. J. Constr. Steel Res., pp. 2382-2397.

American Institute of Steel Construction (AISC) (2005a). Specification for structural steel buildings. American Institute of Steel Construction. Inc., Chicago (il) ANSI/AISC 360-05.

American Institute of Steel Construction (AISC) (2005b). Aisc 360-05, specification for structural steel buildings. American Institute of Steel Construction. Inc: Chicago, Il.

Al-Darzi S, Chen A, Liu Y (2007a). Finite element simulation and parametric studies of perfobond rib connector. Am. J. Appl. Sci., 4: 122-127.

Al-Darzi S, Chen A, Liu Y (2007b). Development of new hole shape of perfobond shear connectors, parametric study. Proceedings of the 2nd international symposium on connection between steel and concrete. University of Stuttgart, Germany, pp. 1401-1413.

An I, Cederwall K (1996). Push-out tests on studs in high strength and normal strength concrete. J Constr. Steel Res., 36: 15-29.

Badie S, Tadros, M, Kakish, H, Splittgerber D, Baishya M (2002). Large shear studs for composite action in steel bridge girders. J. Bridge Eng., 7: 195-203.

Bathon L, Graf M (2000). A Continuous Wood-Concrete-Composite System. In: Proc. of the 6th World Conference on Timber Engineering, Whistler, B.C.

Bsi B (1983). British Standards Institution, BS 5400: Part 5 - Steel, concrete and composite bridges. Code of practice for design of composite bridges, BS 5400-9.1.

BSI B (1990). British Standards Institution, BS 5950: Part 3: Section 3.1 - Structural use of steelwork in building. Code of practice for design of simple and continuous composite beams, BS 5950-3.1.

BSI DE, Eurocode 4 (1992). Cen4: Design of composite steel and concrete structures. Part 1.1: General rules and rules for buildings. British Standards Institution, London, p. 226.

Candido-Martins J, Costa-Neves L, Vellasco P (2010). Experimental evaluation of the structural response of perfobond shear connectors. Eng. Struct., 32(8): 1976-1985.

Chatterjee S, Kumar K (2009).Effective electronic waste management and recycling process involving formal and non-formal sectors. Int. J. Phys. Sci., 4: 893-905.

Crisinel M (1990). Partial-interaction analysis of composite beams with profiled sheeting and non-welded shear connectors. J. Constr. Steel Res., 15: 65-98.

CSA (Canadian standard association) (2001). Can/csa-s16-01, limit states design of steel structures. Rexdale (ontario). Canadian standard association.

De Andrade S (2007). Semi-rigid composite frames with perfobond and t-rib connectors part 2: design models assessment. J. Constr. Steel Res., 63: 280-292.

Dedic D, Klaiber F (1984). High-strength bolts as shear connectors in rehabilitation work. Concrete Int., 6: 41-46.

Dogan O, Roberts T (2010a). Comparing experimental deformations of steel-concrete-steel sandwich beams with full and partial interaction theories. Int. J. Phys Sci., 5(10): 1544-1557

Dogan O, Roberts T (2010b). Comparison of experimental internal forces with full and partial interaction theories in steel-concrete-steel sandwich beams. Int. J. Phys. Sci., 5: 2322-2334.

Ellobody E (2002). Finite element modeling of shear connection for steel concrete composite girders. Ph.d. Thesis. Leeds: School of Civil Engineering, The University of Leeds.

Env 1994-2, E (1997). Cen4: design of composite steel and concrete structures. Part 2, Bridges. Cen. Brussels.

Esposito V (2006). Degree thesis: Numerical-experimental analyses of steel connectors for composite wood-steelconcrete floors (Analisi numerico-sperimentali di un sistema di connessione in acciaio per solai composti legnoacciaio-calcestruzzo). Italy, 2006.

Eurocode (2004). CEN4: Design of composite steel and concrete structures. Part 1.1: General rules and rules for buildings. Comité Européen de Normalisation: Bruxelles.

Faggiano B, Marzo A, Mazzolani FM, Calado lM (2009). Analysis of rectangular-shaped collar connectors for Composite timber-steel-concrete floors: push-out tests. J. Civil Eng. Manage., 15: 47-58.

Ferreira I (2000). Semi-rigid systems for composite building construction. Ph.d. Thesis. Brazil: Pontifícia Universidade Católica do Rio de Janeiro.

Ferreira I, De Andrade S, Vellasco P (1998). A design model for bolted composite semi-rigid connections. Stability and ductility of steel structures, p. 293.

Fink J Petraschek T (2007). Novel shear connectors. Experimental investigations and numerical comparisons, pp. 1327-1336.

Fukada K, Fujii K, Iwasaki H, Toyota T (2005). Slip behavior of perforated rib shear connector in composite girders with thick concrete slab. In, 2005. Taylor & Francis, p. 81.

Galjaard H, Walraven J (2000). Behaviour of shear connector devices for lightweight steel-concrete composite structures: Results, observations and comparisons of static tests. In: second international symposium on structural lightweight aggregate concrete, 2000 Kristiansand, Norway, pp. 221-230.

Galjaard H, Walraven J, Eligehausen R (2001). Static tests on various types of shear connectors for composite structures, pp. 1313-1322.
Gelfi P, Marini A (2002). Stud shear connection design for composite concrete slab and wood beams. J. Struct. Eng., 128: 1544.
Gundel M, Hauke B (2007). Fatigue resistance of aperture plate shear connectors. pp. 1349-1358.
Güney D, Kuruşçu A (2011). Optimization of the configuration of infill walls in order to increase seismic resistance of building structures. Int. J. Phys. Sci., 6(4): 698-706.
Gutkowski R, Brown K, Shigidi A Natterer J (2008). Laboratory tests of composite wood-concrete beams. Construction and Building Materials, 22: 1059-1066.
Hamidian M, Shariati M, Arabnejad KM, Shariati A (2011). Assessment of high strength and light weight aggregate concrete properties using ultrasonic pulse velocity technique. Int. J. Phys. Sci., 6: 5261-5266.
Hawkins N, Mitchell D (1984). Seismic response of composite shear connections. J. Struct. Eng., 110: 2120-2136.
Hechler O, Berthellemy J, Lorenc W, Seidl G, Viefhues E (2008). Continuous shear connectors in bridge construction. Tagungsband composite construction ccvi, Copper Mountain, co, USA.
Hegger ISR (2007). Innovative shear connection for composite beams with uhpc. 2nd international symposium on connections between steel and concrete, pp. 1295-1204.
Hegger J, Rauscher S, Goralski C, Russell H (2005). Push-out tests on headed studs in high-strength concrete. Special Publication, 228: 769-786.
Hegger J, Sedlacek G, Döinghaus P, Trumpf H (2001). Studies on the ductility of shear connectors when using high-strength concrete, pp. 1024-1045.
Highway Bridge Design, Load and resistance factor design (lrfd) for highway bridge substructures.
Hosaka T, Mitsuki K, Hiragi H, Ushijima Y, Tachibana Y, Watanabe H (2000). An experimental study on shear characteristics of perfobond strip and it's rational strength equations. J. Struct. Eng., 46: 1593-1604.
Hungerford B (2004). Methods to develop composite action in non-composite bridge floor systems: part ii. Ms thesis, department of civil, architectural and environmental engineering, university of texas at austin, united state. PH.D Thesis.
Iwasaki H, Fujii K, Fukada K, Toyota T HN (2005). A consideration on slip test methods for perforated rib shear connector focusing on concrete confinements. First Int. Conf. on advances in experimental structural engineering, pp. 871-876.
Jeong Y, Kim H, Koo H (2009). Longitudinal shear resistance of steel-concrete composite slabs with perfobond shear connectors. J. Constr. Steel Res., 65: 81-88.
Johnson R (1970). Research on steel-concrete composite beams. J. Struct. Div., 96: 445-459.
Jumaat M, Rahman M, Alam M, Rahman M (2011). Premature failures in plate bonded strengthened rc beams with an emphasis on premature shear: a review. Int. J. Phys. Sci., 6: 156-168.
Kayir H (2006). Methods to develop composite action in non-composite bridge floor systems: fatigue behavior of post-installed shear connectors. Ms thesis, department of civil, architectural and environmental engineering, University of Texas at Austin, United State. fsel.engr.utexas.edu/publications/browse.
Kim B, Wright, H Cairns R (2001). The behaviour of through-deck welded shear connectors: an experimental and numerical study. J. Constr. Steel Res., 57: 1359-1380.
Kim H, Jeong Y (2006). Experimental investigation on behaviour of steel-concrete composite bridge decks with perfobond ribs. J Constr. Steel Res., 62: 463-471.
Kisa M (2011). Vibration and stability of multi-cracked beams under compressive axial loading. Int. J. Phys. Sci., 6(11): 2681-2696.
Kraus D, Wurzer O (1997). Nonlinear finite-element analysis of concrete dowels. Comput. Struct., 64: 1271-1279.
Kwon G (2008). Strengthening existing steel bridge girders by the use of post-installed shear connectors. p. 261.
Kwon G, Engelhardt R, Klingner R (2009). Behavior of post-installed shear connectors under static and fatigue loading. J. Constr. Steel Res., p.65.
Lam D El-Lobody E (2001). Finite element modelling of headed stud shear connectors in steel-concrete composite beam. Elsevier science Ltd, p. 401.
Lam D, Ellobody E (2005). Behavior of headed stud shear connectors in composite beam. J. Struct. Eng., 131: 96.
Lee K, Han J (1998). Fatigue behavior of composite beams with pyramidal shear connectors under repeated loading. KSCE J. Civil Eng., 2: 119-128.
Lee P, Shim C, Chang S (2005). Static and fatigue behavior of large stud shear connectors for steel-concrete composite bridges. J. Constr. Steel Res., 61: 1270-1285.
Leonhardt, E, Andra H, Harre W (1987a). New Improved Shear Conector with High Fatigue Strength for Composite Structure. New beneficial agents for composite steel-composite structures with high Danerfestigkeit) Concrete and Stahlbetoubau, 12: 325-331.
Leonhardt F, Andrä, W, Andrä, H, Harre W (1987b). New and beneficial means of composite steel-composite structures with high fatigue strength. Concrete and reinforced concrete, 82: 325-331.
Li L, Guo Y, Liu F, Bungey J (2006). An experimental and numerical study of the effect of thickness and length of CFRP on performance of repaired reinforced concrete beams. Constr. Build. Mater., 20: 901-909.
lungershausen H (1988). Zur Schubtragfahigkeit von Kopfbolzendubeln. Technisch-wissenscha/tliche Mitteilung, 88: 7.
Machacek J, Studnicka J (2002). Perforated shear connectors. Steel Compos. Struct., 2: 51-66.
Maleki S, Bagheri S (2008a). Behavior of channel shear connectors, Part I: Experimental study. J. Constr. Steel Res., 64 1333-1340.
Maleki S, Bagheri S (2008b). Behavior of channel shear connectors, Part II: Analytical study. J. Constr. Steel Res., 64 1341-1348.
Maleki S, Mahoutian M (2009). Experimental and analytical study on channel shear connectors in fiber-reinforced concrete. J. Constr. Steel Res., 65: 1787-1793.
Marecek J, Samec JJS (2004). Numerical analysis of Perfobond shear connector. The 4th international conference on advanced engineering design. www.conferencealerts.com/seeconf.mv?q=ca1888ih.
Marecek J, Samec JS (2005). Perfobond shear connector behaviour In: Eurosteel, In: Eurosteel, 4th European conference on steel and composite structures, v. B. 2005. p. 4th European conference on steel and composite 4.3.1-8.
Matsui S (1984). A Study on Fatigue and Design Method of Concrete Slabs of the Road Bridges. The thesis of Osaka University.
Matus R, Jullien J (1996). An experimental and analytical study of a new shear connector for composite steel-concrete beams. In, 1996. Elsevier Science, p. 413.
Mazzolani F, Faggiano B, Marzo A (2005). Retrofitting of complex wooden structures by means of mixed reversible technologies: a study case. Proceedings of the Cost C, 12: 19-22.
Medberry S, Shahrooz B (2002). Perfobond shear connector for composite construction. Eng. J., 39: 2-12.
Mungwa JJ, Foudjet A, Hentges G (1999). Experimental study of a composite wood-concrete beam with the INSA-Hilti new flexible shear connector. Constr. Build. Mater., 13: 371-382.
Natterer J, Hamm J, Favre P (1996). Composite wood-concrete floors for multi-story buildings. pp. 3431-3435.
NBC 2005. National Building Code of Canada.
Nguyen H, Kim S (2009). Finite element modeling of push-out tests for large stud shear connectors. J. Constr. Steel Res., 65: 1909-1920.
Nishido T (2005). Experimental study on rigidly connected steel pipe in girder and concrete pier with perfobond rib shear connectors. Eurosteel (2005). 4th Europ. Conf. Steel Compos. Struct., 4.(2): 7-14.
Oguejiofor E, Hosain M (1992). Behaviour of perfobond rib shear connectors in composite beams: full-size tests. Can. J. Civil Eng., 19: 224-235.
Oguejiofor E, Hosain M (1994). A parametric study of perfobond rib shear connectors. Canadian J. Civil Eng., 21: 614-625.
Oguejiofor E, Hosain M (1997). Numerical analysis of push-out specimens with perfobond rib connectors. Comput. Struct., 62: 617-624.
Oguejiofor EC (1994). Perfobond rie shear connectors for composite beams. Doctor of philosophy dissertation submitted to univeirsity of Saskatchewan.

Ollgaard J, Slutter R, Fisher J (1971). Shear strength of stud connectors in lightweight and normal-weight concrete. AISC Eng. J., 8: 55-64.

Pashan A (2006). Behaviour of channel shear connectors: push-out tests. MS Thesis, Department of Civil Engineering, University of Saskatchewan, Canada.

Poot S, Eligehausen R (2001). Perfo-bond connection and tests. p. 1095-1104.

Rodera GA (2008). Design of composite beams using light steel sections. pdfgemi.com/book/steel-beam-design-examples-pdf.html.

Sal R, Ferreira D (2007). Semi-rigid composite frames with perfobond and T-rib connectors Part 1: Full scale tests. J. Constr. Steel Res., 63: 26.

Schaap B (2004). Methods to Develop Composite Action in Non-Composite Bridge Floor Systems: Part I. MS Thesis, Department of Civil, Architectural and Environmental Engineering, University of Texas at Austin, United State. pp. 257-273.

Sedlacek G, Hoffmeister B, Trumpf H, Kühn B, Stötzel J, Hechler O, Stangenberg H, Bitar D, Galea Y, Lukic M (2003). Composite bridge design for small and medium spans. EUR, 1-525.

Shariati M, Ramli SNH, Arabnejad KMM (2011a). Experimental assessment of channel shear connectors under static and fully reversed cyclic loading in high strength concrete. Materials & Design, doi: 10.1016/j.matdes.2011.08.008.

Shariati M, Ramli SNH, Maleki S Arabnejad KMM (2010a). Experimental and analytical study on channel shear connectors in light weight aggregate concrete. In: Steel & Composite Structures-Proceedings of the 4th Int. Conf., Sydney, Australia. pp. 293-298.

Shariati M, Ramli SNH, Sinaei H, Arabnejad KMM, Shafigh P (2010b). Behavior of Channel Shear Connectors in Normal and Light Weight Aggregate Concrete (Experimental and Analytical Study). Adv. Mater. Res., 168-170: 2303-2307.

Shariati M, Sulong NHR, Kh MMA, Mahoutian M (2011b). Shear resistance of channel shear connectors in plain, reinforced and lightweight concrete. Sci. Res. Essays, 6: 977-983.

Shim C, Kim J, Chang S, Chung C (2000). The behaviour of shear connections in a composite beam with a full-depth precast slab. Proceedings of the Institution of Civil Engineers-Structures and Buildings, 140: 101-110.

Shim C, Lee P, Chang S (2001). Design of shear connection in composite steel and concrete bridges with precast decks. J. Constr. Steel Res., 57: 203-219.

Shim C, Lee P, Yoon T, (2004). Static behavior of large stud shear connectors. Eng. Struct., 26: 1853-1860.

Slutter R, Driscoll G (1962). Test results and design recommendations for composite beams. Lehigh University, Fritz Engineering Laboratory, Report, p. 279.

Slutter R Driscoll, GC (1965). Flexural strength of steel-concrete composite beams. J. Struct. Eng., ASCE, pp. 71-99.

Slutter R, Kim S, Fisher J (1967). Tests of lightweight concrete composite beams and pushout specimens with cellular steel deck. Report 200.67. 438.1, Fritz Laboratory, Lehigh University, United States.

Standard B (1994). Structural use of steelwork in building. BS5950 Part4: Code of practice for design of composite slabs with profiled steel sheeting. www.scribd.com/k2v1n5/d/57264633-5950-9-1994.

Takami K, Ishii N, Hamada S (2005). Shear properties of bonded T-shaped shear connector. Eurosteel 2005, 4th European Conference on steel 7 Compost. Struct., 4(2): 1-6.

Topkaya C, Yura J, Williamson E (2004). Composite shear stud strength at early concrete ages. J. Struct. Eng., 130: 952.

Ushijima Y, Hosaka T, Mitsuki, K, Watanabe, H, Tchibana Y, Hiragi, H. (2001). An experimental study on shear characteristics of perfobond strip and its rational strength equations. In, 2001. pp. 1066-1075.

Valente I, Cruz P (2004a). Experimental analysis of Perfobond shear connection between steel and lightweight concrete. J. Constr. Steel Res., 60: 465-479.

Valente M, Cruz P (2004b). Experimental studies on shear connection between steel and lightweight concrete. www.citeulike.org/journal/els-0143974x/page/9.

Valente I, Cruz P (2009). Experimental analysis of shear connection between steel and lightweight concrete. J Constr. Steel Res., 65: 1954-1963.

Veldanda M, Hosain M (1992). Behavior of perfobond rib shear connectors: push-out tests. Can. J. Civil Eng., p.19.

Vellasco P, De Andrade S, Ferreira l, De Lima L (2007). Semi-rigid composite frames with perfobond and T-rib connectors Part 1: Full scale tests. J. Constr. Steel Res., 63: 263-279.

Veríssimo GS, Paes JLR, Valente MI, Cruz PJS, Fakury RH (2006). Design and experimental analysis of a new shear connector for steel and concrete composite structures. 3rd International Conference on Bridge Maintenance, Safety and Management.

Veríssimo G (2004). Development of a shear connector plate gear for composite structures of steel and concrete and study their behavior. Belo Horizonte, Minas Gerais FederalUniversity-Graduate Program in Structural Engineering.

Veríssimo G, Paes, J, Valente M, Cruz, P, Fakury R (2006a). Design and experimental analysis of a new shear connector for steel and concrete composite structures. en.scientificcommons.org/paulo_j_s_cruz.

Veríssimo G, Paes J, Valente, MI, Cruz, PJS, Fakury R (2006b.) Design and experimental analysis of a new shear connector for steel and concrete composite structures.

Verissimo GS (2007). Development of a shear connector plate gear for composite structures of steel and concrete and study their behavior. Ph.D. thesis. Universidade Federal de Minas Gerais Belo Horizonte; Portugal.

Vianna J (2008). Structural behaviour of T-Perfobond shear connectors in composite girders: An experimental approach. Eng. Struct., 30: 2381-2391.

Vianna JDC, Costa-Neves lF, Da S, Vellasco, PCG, De Andrade SAL (2009). Experimental assessment of Perfobond and T-Perfobond shear connectors' structural response. J. Constr. Steel Res., 65: 408-421.

Viest I (1956a). Investigation of stud shear connectors for composite concrete and steel t-beams. In, 1956a. ACI.

Viest I (956b). Test of Stud Shear Connectors Parts I, II, III y IV. Test Data, Nelson Stud Welding, Lorain, Ohio.

Viest I (1960). Review of research on composite steel-concrete beams. J. Struct. Division, 86: 1-21.

Viest J, Colaco J, Furlong R, Griffis l, Leon R, Wyllie l (1997). Composite construction design for buildings. In, 1997. ASCE.

Viest I, Siess, C, Appleton J, Newmark N (1952). Full-scale tests of channel shear connectors and composite t-beams. Bulletin Series, p. 405.

Wai-Fah C, Robert FL, Ben K (1993). Semi-rigid connections in steel frames. Council on Tall Buildings and Urban Habitat. McGraw-Hill. pp. 1-318

Zellner W (1987). Recent designs of composite bridges and a new type of shear connectors. In, 1987. ASCE, pp. 240-252.

Zingoni A (2001). Behaviour of Different Types Of Shear Connectors For Steel-Concrete Structures. In, 2001. Elsevier Science Ltd, p. 385.