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Review of stud shear resistance prediction in steel-concrete composite beams

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Abstract. In steel-concrete composite beams, longitudinal shear forces are transferred across steel flange-concrete slab interface by means of shear connectors. The connector behavior is highly non-linear and involves several complex mechanisms. The design resistance and stiffness of composite beams depends on the shear connection behavior and the accuracy in the connector resistance prediction is essential. However determining the stud shear resistance is not an easy process: analytical methods do not give an adequate response to this problem and it is therefore necessary to use experimental methods. This paper present a summary of the main procedures to predict the resistance of the stud shear connectors embedded in solid slab, and stud shear connectors in composite slab using profiled steel sheeting with rib perpendicular to steel beam. A large number of experimental studies on the behavior of stud shear connectors and reported in the literature are also summarized. A comparison of the stud shear resistance prediction using six reference codes (AISC, AASHTO, Eurocode-4, GB50017, JSCE and AS-2327.1) and other procedures reported in the literature against experimental results is presented. From this exercise, it is concluded that there are still inaccuracies in the prediction of stud shear resistance in all analysed procedures and that improvements are needed.

Keywords: stud shear connectors; stud shear resistance; composite beams; profiled steel sheeting; push-out test

1. Introduction

In composite beam design, headed stud shear connectors are used to transfer longitudinal shear forces across the steel-concrete interface. Headed steel studs welded to the flange of the steel beam and embedded in concrete solid slab (SSL), or in composite slab using profiled steel sheeting (PSS), have been the most common procedure to transfer longitudinal shear forces between the steel beam and concrete slab in composite beams (An and Cederwall 1996, Lee *et al*. 2005, Nguyen and Kim 2009a, Pallarés and Hajjar 2010, Rambo-Roddenberry 2002, Shim 2004, Shim *et al*. 2004, Xue *et al*. 2008, 2012).

Predicting the stud shear resistance (SSR) is essential in composite section design. However, the analysis of the reactions of stud shear connectors embedded in concrete is very complex due to the several mechanisms involved. Near failure the studs that are under the combined effects of shear, bending and tension, while the surrounding concrete is subjected to combined tension and compression, both materials undergoing inelastic deformations (Hawkins and Mitchell 1984).

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Early SSR prediction equations, developed between the 1950s and the 1970s, were for composite beams with concrete SSL. These equations were conceived based on push-out test results.

The use of PSS in composite construction has been common since the1960s (Rambo-Roddenberry 2002). The PSS is an advantageous technology because it acts as formwork and works as the tensile reinforcement for the concrete slab. In the 1970s, the prediction equations developed for SSR in SSL were modified to estimate the SSR when PSS are used. Such equations were based upon full-scale beam tests (Grant *et al*. 1977).

The main objectives of this paper are to summarize, the several procedures for SSR prediction developed throughout history (Bonilla *et al*. 2012, 2015, Driscoll and Slutter 1961, Fisher 1970, Goble 1968, Grant *et al*. 1977, Hawkins and Mitchell 1984, Jayas and Hosain 1988, Lloyd and Wright 1990, Mottram and Johnson 1990, Nguyen and Kim 2009a, Oehlers and Johnson 1987, Ollgaard *et al*. 1971, Rambo-Roddenberry 2002, Slutter and Driscoll 1965, Viest 1956, Xue *et al*. 2008), and to illustrate the accuracy in terms of SSR prediction of six international reference codes (AASHTO 2014, AISC 2010, AS-2327.1 2003, Eurocode-4 2004, GB50017 2003, JSCE 2007). The evolution of several prediction methods and the development of numerous experimental studies of many authors are presented chronologically. Throughout the paper, the SSR prediction procedures for composite beams using SSL are presented first and then followed by those developed for composite beams using PSS.

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The validity of the SSR prediction using several procedures (AASHTO 2014, AISC 2010, AS-2327.1 2003, Bonilla *et al*. 2015, Eurocode-4 2004, GB50017 2003, JSCE 2007, Rambo-Roddenberry 2002) is compared against experimental results (An and Cederwall 1996, Dai *et al*. 2015, Diaz *et al*. 1998, Jayas and Hosain 1988, Johnson and Yuan 1997, Lam and Ellobody 2005, Lyons *et al*. 1994, Ollgaard *et al*. 1971, Rambo-Roddenberry 2002, Rambo-Roddenberry *et al*. 2002, Robinson 1988, Shim 2004, Sublett *et al*. 1992, Xue *et al*. 2008, 2012). In the verification of the SSR prediction in composite beams with SSL, commercial studs with diameters of 13, 16, 19, 22, 25, 27 and 30 mm were considered. Analyses referring to studs with diameters greater than 22 mm are also included due to their advantages (Lee *et al*. 2005, Nguyen and Kim 2009a, Shim 2004, Shim *et al*. 2004) in composite bridges. However, for the verification of the SSR prediction using PSS are used only for stud commercial diameters of 10, 13, 16 and 19 mm.

Along the paper, graphics and tables compare the different predictions, show to the reader the tendency of the codes AISC (2010), AASHTO (2014), Eurocode-4 (2004), AS-2327.1 (2003), GB50017 (2003) and JSCE (2007) with respect to the real experimental values of stud shear resistance embedded in solid slabs or in composite slabs using profiles steel sheeting.

2. Evolution of prediction procedures of stud shear resistance in composite beams

This review includes the shear resistance prediction equations developed for welded headed stud in concrete SSL and the more recent research on welded headed studs in concrete slabs with PSS with rib perpendicular to steel beam.

2.1 Stud shear resistance prediction in composite beams with solid slab

The prediction equations of SSR have predominantly been derived from empirical studies. The push-out test is the most common way used to evaluate SSR and their behavior. The first push-out test for studying the behavior of the headed studs was conducted by Viest (1956) at the University of Illinois. Viest (1956) observed three types of failure: stud shearing, concrete failure and mixed failures (failure of both materials) and proposed one of the first equations to predict the SSR in composite beams with solid slab (see Table 1). Later, Driscoll and Slutter (1961) proposed a modification of Viest's equation. They observed that the height to diameter ratio (h/d) for studs embedded in normal weight concrete should be equal or larger than 4.2 if the full shear strength of the anchor had to be developed (see Table 1). Chinn (1965) performed ten solid slab push-out tests and two beam tests using lightweight and normal weight concrete. Several stud diameters were used, with stud lengths approximately four times the diameter. Slutter and Driscoll (1965) studied the ultimate design resistance of composite beams. Davies (1967) developed twenty half-scale solid slab push-out tests, where the number, spacing, and pattern of welded studs were variables. The authors found that the SSR increased linearly with the longitudinal stud spacing. Mainstone and Menzies (1967) performed eighty-three push-out specimens to analyze the behavior of headed anchors under both static and fatigue loads. Goble (1968) investigated the behavior of thin flange push-out specimens and proposed equations for SSR prediction (see Table 1).

Ollgaard *et al*. (1971) performed 48 solid slab push-out tests using normal weight concrete and lightweight concrete. Stud diameters tested were 16 mm and 19 mm. Variables considered were: the concrete compressive strength, splitting strength, modulus of elasticity, density and aggregate type, and connector diameter and number per slab. The stud tensile strength, slab reinforcement, and specimen geometry were constant for all tests. The authors observed that the SSR is significantly influenced by: the stud cross-sectional, the concrete compressive strength and modulus of elasticity. Based on this research the authors (Ollgaard *et al*. 1971) proposed an equation to SSR prediction. It should be highlighted that this equation was the first adopted by AISC (1993) to SSR prediction (see Table 1). Oehlers and Coughlan (1986) analyzed 116 push-out tests to observe the stiffness behavior of shear stud connections in composite beams. Oehlers and Johnson (1987) analyzed several push-out test results and proposed an equation to predict the SSR in composite beams (see Table 1).

An and Cederwall (1996) carried out eight push-out test specimens using high strength and normal strength concrete. It was found that the existing European design code (Eurocode-4 1993) was not adequate to estimate the shear strength of studs embedded in high strength concrete.

Rambo-Roddenberry (2002) and Rambo-Roddenberry *et al*. (2002) developed a total of 24 solid slab push-out specimens. The steel/concrete interface, as well as the effect of the transverse load, applied to the specimens, was investigated. As a result it was observed that the transverse load applied to the specimen increases the apparent resistance of the stud.

Shim (2004) performed twelve push-out test specimens using stud shear connectors with diameter of 25, 27 and 30 mm respectively. The author concluded that the design shear resistance in Eurocode-4 (2004) gives conservatives values for large studs. Shim *et al*. (2004) using push-out tests investigated the shear stiffness of large stud connectors in an elastic range and were proposed tri-linear load slip curves. Lee *et al*. (2005) developed several push-out tests and beam tests using large studs with diameters of 25, 27 and 30 mm to investigate their static and fatigue behavior. This study showed that the design shear resistance in Eurocode-4 (2004) could be extended to cover the use of studs up to 30 mm diameter; however, in AASHTO (2014), the safety factor has to be increased. The fatigue strength of large studs was a little lower than that of normal studs, and the fatigue design code for large studs need to be improved conservatively.

Xue *et al*. (2008) performed 30 push-out tests to investigate the effects of stud diameter and height, concrete

Table 1 Equations for SSR prediction in composite beams with SSL

Source	Equations [(1),(2)]
Viest (1956)	$Q_{sc-sol} = 5.25 \cdot d^2 \cdot \bar{f}_c' \cdot \sqrt{4000/\bar{f}_c'}$; *for d <1in* $Q_{sc-sol} = 5 \cdot d \cdot \bar{f}_c' \cdot \sqrt{4000/\bar{f}_c'}$; *for d >1in*
Driscoll and Slutter (1961)	$Q_{sc-sol} = 932 \cdot d^2 \cdot \sqrt{\bar{f}_c'/A_{sc}}$; $for\ h/d < 4.2$ $Q_{sc-sol} = 222 \cdot h \cdot d \cdot \sqrt{\bar{f}_c'/A_{sc}}$; $for\ h/d < 4.2$
Chinn (1965)	$Q_{sc-sol} = 39.22 \cdot d^{1.766}$
Slutter and Driscoll (1965)	$Q_{sc-sol} = 930 \cdot d^2 \cdot \sqrt{\bar{f}_c'}$; $for\ f_c' < 4000\ psi$
Goble (1968)	$Q_{sc-sol} = 882 \cdot d^2 \cdot \sqrt{\bar{f}_c'}$; $for\ stud\ shear\ failure\ mode$ (Eq. (a)) $Q_{sc-sol} = 4.70 \cdot t_f \cdot d^2 \cdot \bar{f}_u$; $for\ pull\ out\ failure\ mode$ (Eq. (b))
Ollgaard *et al*. (1971)	$Q_{sc-sol} = 1.106 \cdot A_{sc} \cdot \bar{f}_c'^{\,0.3} \cdot \bar{E}_c^{0.44}$ $(initial\ expression)$ $Q_{sc-sol} = 0.5 \cdot A_{sc} \cdot \sqrt{\bar{f}_c' \cdot \bar{E}_c}$ $(final\ expression)$
Oehlers and Johnson (1987)	$Q_{sc-sol} = K \cdot A_{sc} \cdot \left(\frac{\bar{E}_c}{\bar{E}_s}\right)^{0.4} \cdot \bar{f}_c'^{\,0.35} \cdot \bar{f}_u^{0.65}\ where\ K = 4.1 - n^{-1/2}$
Xue *et al*. (2008)	$Q_{sc-sol} = 3 \cdot \lambda \cdot A_{sc} \cdot \bar{f}_u \cdot \left(\frac{\bar{E}_c}{\bar{E}_s}\right)^{0.4} \left(\frac{\bar{f}_{cu}}{\bar{f}_u}\right)^{0.2}$; λ $= \begin{cases} 6 - \left(h/1.05 \cdot d\right) for \left(h/d \le 5\right) \\ 1\ for \left(5 < h/d < 7\right) \\ \left(h/d\right) - 6\ for \left(h/d \ge 7\right) \end{cases}$
Pallarés and Hajjar (2010)	$Q_{sc-sol} = 17 \cdot A_{sc} \cdot \bar{f}_c'^{\,0.45} \cdot \bar{E}_c^{0.04}$ $Q_{sc-sol} = 6.2 \cdot A_{sc} \cdot (\bar{f}_c' \cdot \bar{E}_c)^{0.2}$ *in all cases should be* $Q_{sc-sol} \le 0.65 \cdot A_{sc} \cdot \bar{F}_u$ $Q_{sc-sol} = 18 \cdot A_{sc} \cdot \bar{f}_c'^{\,0.5} \cdot h^{0.2}$ $Q_{sc-sol} = 9 \cdot \lambda \cdot \bar{f}_c'^{\,0.5} \cdot d^{1.4} \cdot h^{0.6}$

[(1)] Units: pounds, inches (Viest 1956); kips, inches (Chinn 1965, Driscoll and Slutter 1961, Ollgaard *et al*. 1971, Pallarés and Hajjar 2010, Slutter and Driscoll 1965); N, mm (Oehlers and Johnson 1987) (Xue *et al*. 2008); Ec. a - kips, psi, inches (Goble 1968), Ec. b – kips, ksi, inches (Goble 1968).

[(2)] In the equations of this table, the symbols are as defined in the Notation - beginning of the paper. For material properties (ex. $\bar{f}_c'$, $\bar{f}_{cu}$), values $\overline{(.)}$ are equivalent but experimental averages.

strength, stud welding technique, transverse reinforcement and steel beam type on SSR and proposed an equation for SSR prediction (see Table 1).

Nguyen and Kim (2009b) analyzed 32 push-out specimens with different stud diameters and concrete strength using FE analyses. These results were compared with the results obtained according to the design rules specified in Eurocode-4 (2004) and AASHTO (2014). The comparison showed that the AASHTO (2014) over-estimated the resistance of stud shear connectors. The Eurocode-4 (2004) was generally conservative for stud diameters of 22 and 25 mm, and less conservative for stud diameters of 27 mm. The Eurocode-4 (2004) overestimated the resistance of stud diameter of 30 mm.

Pallarés and Hajjar (2010) carried out an extensive review of 391 test results available in the literature and proposed four equations for the stud shear resistance prediction when the connector is embedded in solid slab (see Table 1).

Xu *et al*. (2012) performed several push-out tests to study the behavior of stud shear connectors placed in groups. They also made a parametric study using FE analysis. The authors found, using AASHTO (2014), Eurocode-4 (2004), Japanese Code (JSCE 2007) and Chinese Code (GB50017 2003), that the stud shear resistance of stud groups, in many cases, was underestimated or overestimated. Xue *et al*. (2012) developed push-out tests to investigate the different behavior between single-stud and multi-stud connectors, using stud diameter of 22 mm. The results showed that the prediction of SSR based on Eurocode-4 (2004) matched well the multi-stud test results. Nevertheless the predictions based on AASHTO (2014) and Chinese code (GB50017 2003) agreed well with the single-stud test results.

In the past 15 years, different researchers have developed FE analyzes to model composite structures. In the study of stud connector behavior, the works of Alenezi *et al*. (2015), Ban *et al*. (2016), Dai *et al*. (2015), Ellobody and Lam (2002), Ellobody and Young (2006), Lam and Ellobody (2005), Lu *et al*. (2012), Mirza (2008), Nguyen and Kim (2009a), Pavlović *et al*. (2013), Qureshi *et al*. (2011a, b), Qureshi and Lam (2012), Xu *et al*. (2012), and Xu and Sugiura (2013), among others, are important references.

Bonilla *et al*. (2012) based on results derived from numerical models, developed an equation to predict the SSR. The new proposed equation (see Eq. (1)) is similar to Ollgaard's formula (Ollgaard *et al*. 1971) but considered for the SSR the *h*/*d* ratio and the longitudinal spacing of the studs.

$$Q_{sc-sol} = \alpha \cdot \beta \cdot \gamma \cdot A_{sc} \cdot \sqrt{\bar{f}_c' \cdot \bar{E}_c} \leq 0.8 \cdot A_{sc} \cdot \bar{F}_u \quad (1)$$

where α take values between 0.72 and 1.00, and this coefficient is function of *h*/*d* ratio (in the range of 2.6 to 4.4), $\bar{f}_c'$ (in the range of 20 to 40 MPa) and *d* (Bonilla *et al*. 2012); β adopts 0.37 and 0.32 for $d \leq 19\,mm$ and $19 < d \leq 25\,mm$ respectively, and γ is function of the longitudinal stud spacing (in the range of 5*d* to 25*d*), $\bar{f}_c'$ (in the range of 20 to 40 MPa) and *d*, and this coefficient take values between 0.54 and 1.00 (Bonilla *et al*. 2012).

Unlike the previous procedures and the procedures adopted by AISC (2010) and Eurocode-4 (2004), Eq. (1) (introducing the coefficient γ) reduces the SSR due to the proximity of the connectors in the longitudinal direction of the steel beam. In addition, Eq. (1) considers the variation of SSR when the *h*/*d* ratio varies introducing the coefficient α. Unlike Eurocode-4 (2004) the coefficient α is function of concrete strength and stud diameter. Su *et al*. (2014) developed an investigation of the static behavior of grouped stud shear connectors in high-strength concrete. The authors performed five push-out test specimens with different arrangements of the studs. It was observed that the load-slip equation given by Lorenc and Kubica (2006), and Xue *et al*. (2008, 2012), based on a single stud shear connector in normal strength concrete, do not apply to grouped stud shear connectors in high-strength concrete. A new equation was proposed based on the test results. Su *et al*. (2014) compared the SSR from test results with international codes AASHTO (2014), Eurocode-4 (2004), and GB50017 (2003) and concluded that these codes are conservative when high strength concrete is used.

Han *et al*. (2015) developed eighteen push-out tests to evaluate the load-slip behavior, SSR and ultimate slip of shear studs embedded in elastic concrete, using different rubber contents. Test results show that the ductility of stud improves significantly with the increasing rubber content. It is concluded that the equations provided by Eurocode-4 (2004) and GB50017 (2003) can still apply to shear studs embedded in elastic concrete.

Kim *et al*. (2015) investigated the stud connector behavior embedded in an ultrahigh-performance concrete deck through 15 push-out tests. The SSR and relative slips were measured. It was observed that the SSR is greater than that obtained by the AASHTO (2014) by a margin of 2% to 13%. The comparison with the Eurocode-4 (2004) provides a margin of 27% to 42%.

Hicks and Uy (2015), in their paper, provide an overview of the new Australian composite design standard for buildings: AS/NZS 2327 (AS/NZS 2327 201X), which is under preparation, and is the first joint Australian and New Zealand design standard for composite buildings. For SSR prediction embedded in SSL, Eqs. (2)-(3) that will be used by the code AS/NZS 2327 are presented as follows. For design purposes they should be utilized considering the smaller value of the two equations:

$$Q_{sc-sol} = \emptyset \cdot 0.70 \cdot d^2 \cdot F_u \quad (2)$$

$$Q_{sc-sol} = \emptyset \cdot 0.29 \cdot d^2 \cdot \sqrt{f_c' \cdot E_c} \quad (3)$$

where $\emptyset$ is the capacity reduction factor taken as 0.8. The diameter of stud shank connector should be between 16 and 25 mm. The ultimate tensile strength of the stud material (F_u) should be not greater than 500 MPa. The concrete strength should be between 16 and 80 MPa.

de Lima Araújo *et al*. (2016) performed twenty push-out tests to analyze the shear resistance of headed stud connectors associated with precast hollow-core slabs with a structural concrete topping. An equation for the SSR prediction in composite precast hollow-core slab beams was presented.

Shen and Chung (2017) developed 11 standard push-out tests and 11 modified push-out tests using solid slab and composite slab in order to investigate stud shear connectors behavior under shear forces and under combined forces of shear and tension. Three distinctive failure modes were observed: shank shearing, concrete conical failure and stud bending with concrete conical failure. When the connection is under tension force, it is concluded that the SSR should be reduced with a factor of 0.84 for the case of SSL, and a factor of 0.75 for the case of a composite slab using PSS.

Qi *et al*. (2017) developed six push-out test specimens with the same geometric dimensions to investigate the effect of the damage degree and location on the static behavior and shear resistance of stud shear connectors. A theoretical formula with a reduction factor K was proposed to consider the reduction of the SSR due to the presence of initial damage of stud due to corrosion, fatigue, unexpected overloading, weld defect or other factors. This coefficient was applied using AASHTO (2014), Eurocode-4 (2004), GB50017 (2003) and compared with the test results found in the literature. It was found that the proposed method produced good predictions of the SSR with initial damage.

2.2 Stud shear resistance prediction in composite beams using profiled steel sheeting

Table 2 Equations for SSR prediction in composite beams using PSS

Source	Equations [(1),(2),(3)]
Fisher (1970)	$Q_{sc-rib} = 0.36 \cdot \frac{w_r}{h_r} \cdot Q_{sc-sol}$
Grant *et al*. (1977)	$Q_{sc-rib} = \frac{0.85}{\sqrt{n_r}}\left(\frac{h-h_r}{h_r}\right)\left(\frac{w_r}{h_r}\right) \cdot Q_{sc-sol} \leq Q_{sc-sol}$
Hawkins and Mitchell (1984)	$Q_{sc-rib} = 5.4 \cdot \sqrt{f_c'} \cdot A_c$
Jayas and Hosain (1988)	$Q_{sc-rib} = 0.35 \cdot \lambda \cdot \sqrt{f_c'} \cdot A_c \leq Q_{sc-sol} \quad for\ h_r = 76\,mm$ $Q_{sc-rib} = 0.61 \cdot \lambda \cdot \sqrt{f_c'} \cdot A_c \leq Q_{sc-sol} \quad for\ h_r = 38\,mm$ λ = 1.0 for normal density concrete, or 0.85 for semi-low density concrete, or 0.75 for structural low density concrete.
Mottram and Johnson (1990)	$Q_{sc-rib} = SRF \cdot Q_{sc-sol} \quad where \quad SRF = \frac{0.75 \cdot r}{\sqrt{n_r}}\left(\frac{h}{h-h_r}\right) \leq 1.0$ For central or strong position studs (see Fig. 1), r is the lesser of w_r/h_r and 2.0, for weak position studs (see Fig. 1), r is the least of w_r/h_r, $(e/h_r)+1$, and 2.0.
Lloyd and Wright (1990)	$Q_{sc-rib} = \left(A_c \cdot \sqrt{f_c'}\right)^{0.34}$

(1) Q_{sc-sol} : SSR from Ollgaard *et al*. (1971)

(2) Units: psi, inches (Hawkins and Mitchell 1984, Lloyd and Wright 1990); N, MPa, mm (Jayas and Hosain 1988);

(3) For a better understanding, the same notation for each variable was used in the equations in this table.

Fisher (1970) proposed an equation to SSR prediction with PSS and made recommendations regarding the design of composite beams with PSS (see Table 2). Seven years later, Grant *et al*. (1977) used composite beam tests instead of push-out tests to develop equations which predict the SSR using PSS. A modification to the equation developed by Fisher (1970) was made to include the height of the stud shear connector. Years later, several researchers (Easterling *et al*. 1993, Hawkins and Mitchell 1984, Jayas and Hosain 1988, 1989, Lloyd and Wright 1990, Lyons *et al*. 1994, Mottram and Johnson 1990, Rambo-Roddenberry 2002, Robinson 1988) have shown that the procedure developed by Grant *et al*. (1977) (see Table 2) is unconservative. Johnson and Oehlers (1981) performed four composite T-beam tests and 101 push-out tests. Hawkins and Mitchell (1984) performed push-out tests under reversed cyclic loading and under monotonic loading to study the seismic response of shear connectors with PSS. Four failure modes (stud shearing, concrete pull-out, rib shearing, and rib punching) were observed by the authors. They proposed an equation to predict the resistance of the connectors when concrete pull-out failure occurs (see Table 2).

Elkelish and Robinson (1986) developed composite beam tests using PSS as well as finite element analysis. Twenty-four simply supported beams were used in the analysis. Six experimental beams were used to verify the analysis method. Robinson (1988) performed forty nine push-out tests and two composite beam tests with PSS. The push-out tests had only one or one pair of studs per half specimen. All push-out specimens failed at a load lower than that predicted by Grant *et al*. (1977). Jayas and Hosain (1988) performed eighteen push-out tests and four pull-out tests. Five of the push-out specimens had PSS with the rib parallel to the steel beam, and eight had the rib perpendicular to the steel beam. The authors proposed equations to predict SSR (see Table 2). Jayas and Hosain (1989) performed four more composite beam tests and two push-out tests with PSS with rib perpendicular to the beam. Oehlers (1989) discussed the effect of slab splitting on stud strength. Several push-out tests were performed to determine the splitting strengths of concrete slabs.

Mottram and Johnson (1990) performed thirty-five push-out tests using PSS with rib perpendicular to steel beam. The tests showed that the resistance per stud for two studs per rib was less than for one stud per rib and the resistance per stud for two studs placed diagonally was less than for the unfavorable position (see Fig. 1). The authors (Mottram and Johnson 1990) recommend that off center studs be placed on the favorable position side away from the mid-span of the beam. The tests showed that the SSR in unfavorable position was 35% lower than the SSR in favorable position (see Fig. 1). The authors proposed an equation to predict SSR using PSS (see Table 2). Finally, the authors proposed a resistance reduction factor, SRF,

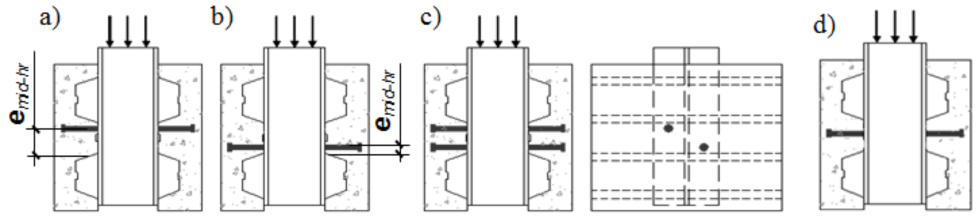

Fig. 1 Frontal and lateral views of composite specimen, representing the stud positions in the deck rib: (a) Favorable or Strong position (SP), (b) Unfavorable or Weak position (WP), (c) Staggered or Zigzag position (StP) and (d) (MP) Middle position

which must be multiplied by the equation for solid slab proposed by Ollgaard *et al*. (1971). Unlike Grant's equation, this equation takes into account the position of the studs in the rib. Lloyd and Wright (1990) carried out forty two push-out tests using PSS. From this study, a procedure to predict the SSR following a similar approach from Hawkins and Mitchell (1984) was developed (see Table 2). In this case, the equation adopted for calculating A_c (area of concrete pull-out failure surface) is different from the Hawkins and Mitchell's procedure (Hawkins and Mitchell 1984).

Wright and Francis (1990) developed four full-scale tests of composite beams with concrete slab using PSS and three push-out test specimens. All beams were designed for partial shear connection. Concrete shear failure around the studs was observed in all beams and push-out specimens. Kitoh and Sonoda (1990) performed composite beam tests. Three-dimensional finite element analyses were also performed. Stud forces were measured during the tests. The tensile forces in the studs were about 10% of the shearing forces in the elastic region and increased between 30% and 50% of the shearing forces at ultimate resistance.

Sublett *et al*. (1992) performed 36 push-out tests using PSS to determine the SSR. They observed that studs in the strong position exhibited a larger stiffness than those in the weak position and that strength of concrete influences significantly the stud shear resistance. In the same year, Lawson (1992) proposed a new procedure for calculating the resistance reduction factor for shear connectors welded through PSS.

Johnson and Yuan (1997) analyzed the results of more than 300 push-out tests using PSS and proposed equations based on the seven failure modes observed. Five modes of failure were considered for PSS with ribs perpendicular to steel beam (shank shearing, rib punching, rib punching with shank shearing, rib punching with concrete pull-out, and concrete pull-out). For each failure mode, theoretical models were developed in which the position of the stud within the rib of the PSS is considered.

Rambo-Roddenberry (2002) performed 93 push-out tests using PSS. The effect of stud diameter, concrete strength, PSS height, transverse load, stud position, number of studs inside the rib and PSS thickness, among others factors on SSR, were studied. The transverse load was varied from 5% to 20% of the vertical load. The study showed that the SSR increased apparently with the transverse load. Based on the above experimental studies, it was proposed a method to predict the SSR using PSS with rib perpendicular to the steel beam. This procedure consists of:

(1) For headed shear studs in PSS of 51 mm and 76 mm in height where the relation between the connector diameter (d) and the I-beam flange thickness (t_f) is (d/t_f) ≤ 2.7, the following equation was proposed

$$Q_{sc} = R_p \cdot R_n \cdot R_d \cdot A_{sc} \cdot \bar{F}_u \tag{4}$$

In Eq. (4), R_p is the reduction coefficient that considers the stud position inside the steel deck rib, defined for three conditions: R_p = 0.68 for the strong or favorable position (SP) [$e_{\text{mid-hr}} \geq$ 56 mm (2.2 in)]; R_p = 0.48 for the unfavorable or weak position (WP) ($e_{\text{mid-hr}} <$ 56 mm) and R_p = 0.52 for headed shear studs in staggered position - where $e_{\text{mid-hr}}$ is the average distance from the stud to the PSS. R_n is a reduction coefficient that considers the number of connectors inside the PSS ribs and is defined for two cases: R_n = 1.0 for one stud per rib or for studs in staggered position (StP - Fig. 1). R_n = 0.85 for two studs per rib. R_d is a coefficient that takes into consideration the thickness of the PSS and takes the following values: R_d = 1.0 for the SP (strong position - Fig. 1) of studs and all sheet thicknesses (or sheet number or gauge as known in practice). R_d = 0.88 for studs placed in WP (weak position - Fig. 1) inside the ribs and profiled sheet # 22, R_d = 1.0, 1.05 and 1.11, for sheet gauges, # 20, # 18 and # 16, respectively.

(2) For headed studs in PSS with heights of 25 mm and 38 mm and where the relation (d/t_f) ≤ 2.7, the following equation was proposed:

$$Q_{sc} = R_n \cdot 3.08 \cdot e^{0.048 \cdot A_{SC} \cdot \bar{F}_U} \tag{5}$$

In Eq. (5), R_n takes the following values 1.0 or 0.85 for one or two headed studs per rib respectively. When (d/t_f) ≤ 2.7 is not satisfied, the values obtained in Eqs. (4)-(5) should be multiplied by the factor (1.5·[(d/t_f)-2.7]).

Qureshi *et al*. (2011b) developed advanced numerical models using finite element method to study the behavior of stud shear connectors in composite beams with PSS oriented perpendicular to the beam axis. It was observed that SSR of connector pairs placed in SP was 94 % of the resistance of a single shear stud on average when the transverse spacing between studs was 200 mm or more. The resistance of StP of studs was only 86 % of the resistance of a single stud, and the resistance of double shear studs in SP

Table 3 Shear resistance coefficients (Bonilla *et al*. 2015)

Coefficients	Criterion		Deck height (h_r)		
			$h_r \leq 38$ mm	$38 < h_r \leq 60$ mm	$60 < h_r \leq 80$ mm
α_1	1SP ($e_{mid\text{-}hr} \geq 56$ mm)		-	0.36	0.33
	1StP		-	0.30	0.28
	1WP ($e_{mid\text{-}hr} < 56$ mm)		-	0.27	0.25
			$h_r \leq 80$ mm		
α_2	Single Stud in SP, WP or StP		1.00		
	Pairs of Studs in SP or WP		0.87		
α_3			$h_r \leq 38$ mm	$38 < h_r \leq 80$ mm	
	SP, StP - any Gauge		-	1.00	
	In WP	Gauge 22	-	0.88	
		Gauge 20	-	1.00	
		Gauge 18	-	1.05	
		Gauge 16	-	1.11	

was higher than that of the StP.

Bonilla *et al*. (2015) based on numerical studies, developed a procedure to predict the SSR using PSS with ribs perpendicular to steel beam. For PSS with h_r = 25 mm or h_r = 38 mm height, Eq. (6) was suggested

$$Q_{sc} = \alpha_2 \cdot 3.08 \cdot e^{0.048 \cdot A_{SC} \cdot \bar{F}_u} \leq 0.8 \cdot A_{sc} \cdot \bar{F}_u \tag{6}$$

For steel decks with 38 mm < $h_r \leq 80$ mm height, Eq. (7) was suggested

$$Q_{sc} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot A_{sc} \cdot \sqrt{\bar{f}_c' \cdot \bar{E}_c} \leq 0.8 \cdot A_{sc} \cdot \bar{F}_u \tag{7}$$

In Eq. (7), the coefficient α_1 considers the stud position. In Eqs. (6)-(7) the coefficients α_2 takes into account the number of connectors inside the rib of the PSS. In Eq. (7) the coefficient α_3 takes into account the thickness of the PSS (see Table 3). It should be noted that Eqs. (6)-(7), as well as above Eqs. (1), (4) and (5), were obtained by regression analysis using experimental data. Therefore, the properties of the materials in these equations are referred to average experimental values.

Nellinger (2015) developed 33 push-out tests where the transverse loading, the shape of the PSS, the number of studs per rib, the number of reinforcement layers, the welding method and the concrete strength were investigated. Taking into account the small number of tests, the author state that it is not possible concluded about the influence of the degree of transverse loading on the resistance and displacement capacity of the shear connection. However, it can be concluded that transverse loading tends to improve the SSR.

3. Procedures adopted in different codes for stud shear resistance prediction

The codes considered in this paper (AASHTO 2014, AISC 2010, AS-2327.1 2003, Eurocode-4 2004, GB50017 2003, JSCE 2007), are good references as they are acknowledged all over the world, and a great number of other codes have them as an orientation. The equations established by the above codes to predict the SSR are considered in this analysis without the partial safety factor, that is to say, the SSR prediction value is unfactored.

The AISC (2010) equation for the prediction of nominal shear resistance of the studs embedded in solid slab or in composite slab is given as (8)

$$Q_{sc-AISC} = 0.5 \cdot A_{sc} \cdot \sqrt{f_c' \cdot E_c} \leq R_g \cdot R_p \cdot A_{sc} \cdot F_u \tag{8}$$

In the AISC (2010), in order to design the composite concrete-steel section the correct use of the inequality in Eq. (8) is required. When stud connectors are embedded in composite slab using PSS with rib perpendicular to steel beam, it was observed (Bonilla *et al*. 2015) that the second part of the inequality prevails. In Eq. (8), R_g and R_p coefficients take into account, respectively, the number of studs inside PSS ribs, and the shear connector position inside the rib of PSS (SP, StP, WP – Fig. 1). Such positions influence the shear resistance of the stud connectors (Johnson and Yuan 1997, 1998, Lyons *et al*. 1994, Rambo-Roddenberry 2002, Rambo-Roddenberry *et al*. 2002).

In AASHTO (2014), the nominal shear resistance of one stud shear connector embedded in concrete SSL is taken as (9)

$$Q_{sc-\mathrm{AASHTO}} = 0.5 \cdot A_{sc} \cdot \sqrt{f_c' \cdot E_c} \leq A_{sc} \cdot F_u \tag{9}$$

Eurocode-4 (2004) defines Eq. (10) for calculating the shear resistance for headed stud connectors in the composite section made up of solid concrete slabs.

$$Q_{sc-EC-4} = 0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}} \leq 0.8 \cdot A_{sc} \cdot f_u \tag{10}$$

In Eq. (10) the value of α is determined from the following Eq. (11)

Table 4 Experimental studies of push-out test with SSL for the comparison with prediction shear resistance

Source	Series(*)	d (mm)	$\bar{f}_c'$ (MPa)
Ollgaard *et al*. (1971)	A, LA, SA, B, LB, SB, 2B, C-, C, D-, D, E-, E, LE, SE, 2E	16, 19	18.41 - 35.03
An and Cederwall (1996)	NSC11, NSC12, NSC21, NSC22, HSC11, HSC12, HSC21, HSC22	19	30.77 – 91.24
Dai *et al*. (2015)	Test 1, Test 2, Test 3, Test 4, Test 5, Test 6, Test 7	19	15.88 – 49.1
Rambo-Roddenberry (2002)	S1, S2, S3, S4, S5, S6, S7, S8	19	23.68 – 33.64
Lam and Ellobody (2005)	SP-1, SP-2, SP-3, SP-4	19	16 - 40
Xue *et al*. (2008)	STUD1-3, STUD4-6, STUD7-9, STUD10-12, STUD13-15, STUD16-17, STUD18, STUD20, STUD21, STUD23-24, STUD26	13, 16, 19	30.5 – 38.9
Xue *et al*. (2012)	SD-1, MD-1, MD-2, MD-3, MD-4	22	55.7
Shim (2004)	ST25-A1, ST25-A2, ST25-A3, ST25-B1, ST25-B2, ST25-B3, ST27-A1, ST27-A2, ST27-A3, ST27-C1, ST27-C2, ST27-C3, ST30-A1, ST30-A2, ST30-A3, ST30-C1, ST30-C2, ST30-C3	25, 27, 30	35.3 – 64.5

Note: (*) = Nomenclature of specimens can be found in references (An and Cederwall 1996, Dai *et al*. 2015, Lam and Ellobody 2005, Ollgaard *et al*. 1971, Rambo-Roddenberry 2002, Shim 2004, Xue *et al*. 2012, 2008).

In the series S1 - S7 of Rambo-Roddenberry (2002), there is an applying transverse load on the solid slab.

$\bar{f}_c'$ is the average of the experimental value in the range given.

$$\alpha = 0.2 \cdot \left(\frac{h}{d} + 1\right) \; for \; 3 \leq \frac{h}{d} \leq 4 \quad \text{and} \quad \alpha = 1 \; for \; \frac{h}{d} > 4 \tag{11}$$

For composite section made of concrete slab and PSS orientated perpendicular to the I-beam axis, it is necessary to reduce the value obtained in Eq. (10) by multiplying this value by the coefficient k_t obtained from the following Eq. (12).

$$k_t = \frac{0.7}{\sqrt{n_r}} \cdot \frac{w_r}{h_r} \cdot \left(\frac{h}{h_r} - 1\right) \tag{12}$$

In Eq. (10), the shear connector position inside the rib of PSS is not considered. However, based on several researches (Lyons *et al*. 1994, Rambo-Roddenberry 2002, Rambo-Roddenberry *et al*. 2002), the stud position greatly influences the shear resistance of stud connectors. Consequently, one important parameter that might be considered in the equation for evaluating the shear resistance is the stud position inside the rib.

In GB50017 (2003) the stud shear resistance embedded in concrete SSL is taken as in Eq. (13)

$$Q_{sc-GB50017} = 0.43 \cdot A_{sc} \cdot \sqrt{f_c' \cdot E_c} \leq 0.7 \cdot A_{sc} \cdot \gamma \cdot F_u \tag{13}$$

In JSCE (2007) the SSR embedded in concrete SSL shall be determined as the lesser value from the following two equations

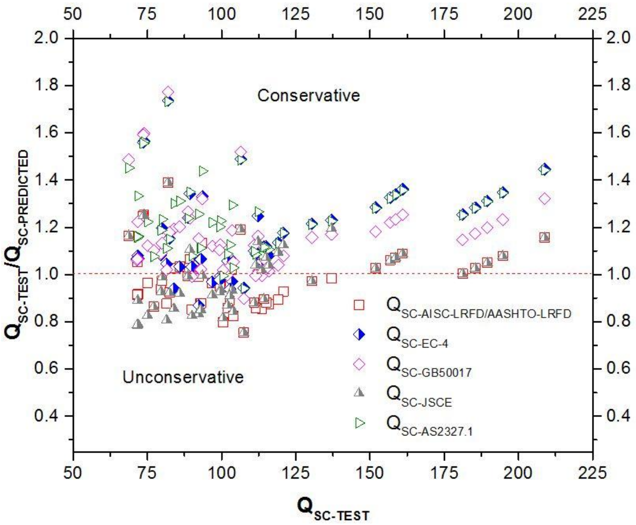

Fig. 2 Prediction of the SSR embedded in concrete SSL without transverse load using stud with d = 13, 16, 19 and 22 mm

$$Q_{sc-JSCE} = 31 \cdot A_{sc} \cdot \sqrt{\left(h/d\right) \cdot f_c'} + 10000 \tag{14}$$

$$Q_{sc-JSCE} = A_{sc} \cdot F_u \tag{15}$$

In AS-2327.1 (2003) the nominal shear resistance for stud connectors embedded in concrete SSL is taken as lesser value from the following two equations:

$$Q_{sc-AS} = 0.31 \cdot d^2 \cdot \sqrt{f_c' \cdot E_c} \tag{16}$$

$$Q_{sc-AS} = 0.63 \cdot d^2 \cdot F_u \tag{17}$$

4. Verification of the accuracy of the stud shear resistance prediction

This section presents the verification of the SSR prediction accuracy for different methods, without considering partial safety factors. The prediction of SSR in SSL and the prediction of SSR using PSS are verified. The unfactored SSR calculated using AISC (2010), AASHTO (2014), Eurocode-4 (2004), GB50017 (2003), JSCE (2007) and AS-2327.1 (2003), are verified against the test results carried out by various researchers such as Ollgaard *et al*. (1971), An and Cederwall (1996), Dai *et al*. (2015), Rambo-Roddenberry (2002), Lam and Ellobody (2005), Xue *et al*. (2008), Xue *et al*. (2012) and Shim (2004). Consequently, it should be noted that in this work, the analysis of the safety in the design of the codes analyzed is not performed. Some test parameters from these experimental studies are shown in Table 4. It should be noted that the verification of the codes and procedures analyzed in this paper has been carried out separately for the push-out tests with transverse load and without transverse load.

For push-out tests without transverse load, one can observed in Fig. 2 the trend to conservative values of SSR calculated by Eurocode-4 (2004), GB50017 (2003) and AS-2327.1 (2003) can be noticed for stud shear connector diameters of 13, 16, 19 and 22 mm. For these stud diameters, the AISC (2010), AASHTO (2014) and JSCE (2007) are offering better results, that is, less conservative values.

For stud connector diameters of 25, 27 and 30 mm the prediction by AISC (2010), AASHTO (2014) and JSCE (2007) give excessively unconservative results in some cases. However, Eurocode-4 (2004), GB50017 (2003) and AS-2327.1 (2003) are offering more conservative results (see Fig. 3). It might be noticed here that the terms "conservative" and "unconservative" does not mean safe and unsafe resistance design, respectively. The definition of safe or unsafe is associated to the design coefficients used hpgby each particular Standard.

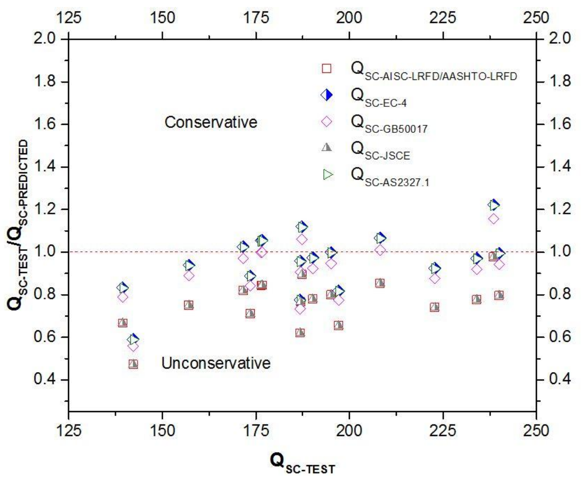

Fig. 3 Prediction of the SSR embedded in concrete SSL without transverse load using stud with d = 25, 27 and 30 mm

Table 5 shows the statistical data where the unfactored SSR ($Q_{sc–AISC}$, $Q_{sc–AASHTO}$, $Q_{sc–EC-4}$, $Q_{sc–GB}$, $Q_{sc–JSCE}$ and $Q_{sc–AS}$) is compared with data from experimental tests. The mean value ratios of $Q_{sc\text{-}test}/Q_{sc–AISC/AASHTO}$, $Q_{sc\text{-}test}/Q_{sc–EC-4}$, $Q_{sc\text{-}test}/Q_{sc–GB}$, $Q_{sc\text{-}test}/Q_{sc–JSCE}$ and $Q_{sc\text{-}test}/Q_{sc–AS}$, for stud diameters of 13, 16, 19 and 22 mm are 0.98, 1.17, 1.17, 0.98 and 1.23 respectively, with the corresponding coefficients of variation (COV) of 0.12, 0.16, 0.14, 0.14 and 0.12, respectively. One should note that R^2 coefficient takes a value of 0.864 for the prediction using AISC (2010) and AASHTO (2014) for stud diameters of 13, 16, 19 and 22. For the same diameters the R^2 coefficient takes a value of 0.838 for the prediction using JSCE (2007). However, for stud diameters of 25, 27 and 30 mm, the mean values of $Q_{sc\text{-}test}/Q_{sc–AISC/AASHTO}$, $Q_{sc\text{-}test}/Q_{sc–EC-4}$, $Q_{sc\text{-}test}/Q_{sc–GB}$, $Q_{sc\text{-}test}/Q_{sc–JSCE}$ and $Q_{sc\text{-}test}/Q_{sc–AS}$ ratios are equal to 0.76, 0.95, 0.91, 0.76 and 0.95, respectively. The COV for $Q_{sc\text{-}test}/Q_{sc–AISC/AASHTO}$, $Q_{sc\text{-}test}/Q_{sc–EC-4}$, $Q_{sc\text{-}test}/Q_{sc–GB}$, $Q_{sc\text{-}test}/Q_{sc–JSCE}$ and $Q_{sc\text{-}test}/Q_{sc–AS}$ ratios are, approximately, equal to 0.15 in all cases. Table 5 summarizes the others statistical measures that show the prediction behavior for the procedures analyzed.

Some experimental results applying a transverse load in the push-out tests are also utilized in the verification of the codes analyzed in this work. Table 6 compares the experimental results developed by Rambo-Roddenberry (2002) against the SSR prediction obtained from the codes mentioned above. The mean value ratios of $Q_{sc\text{-}test}/Q_{sc–AISC/AASHTO}$, $Q_{sc\text{-}test}/Q_{sc–EC-4}$, $Q_{sc\text{-}test}/Q_{sc–GB}$, $Q_{sc\text{-}test}/Q_{sc–JSCE}$ and $Q_{sc\text{-}test}/Q_{sc–AS}$, for stud diameters of 19 mm are 1.03, 1.25, 1.21, 1.00 and 1.30, respectively. The predictions given by the codes AS-2327.1 (2003), Eurocode-4 (2004) and GB50017 (2003) tend to give conservative SSR values. It should be taken into consideration that the number of push-out tests summarized in Table 6 is small.

For stud shear connectors placed in composite beams using PSS, the prediction procedures are verified against the test results carried out by several researchers such as Diaz *et al*. (1998), Rambo-Roddenberry (2002), Lyons *et al*. (1994), Sublett *et al*. (1992), Jayas and Hosain (1988), Robinson (1988) and Johnson and Yuan (1997) (see Table 7). The SSR prediction when the push-out test is under transverse load is analyzed firstly. The SSR prediction obtained according to AISC (2010) show a trend towards unconservative values (see Fig. 4). However, the prediction according to Eurocode-4 (2004) presents very scattered values unconservatives in some cases but very conservative in other cases (see Fig. 5).

The procedures developed by Rambo-Roddenberry (2002) and Bonilla *et al*. (2015) show a trend to be more conservative and less scattered than AISC (2010) and Eurocode-4 (2004) respectively (see Fig. 6-7). In Figs. 4, 5, 6 and 7, the symbol (X)Y(Z)h_r(K) means X = number of studs, Y = stud position, Z = stud diameter, K = nominal rib height.

Table 8 shows the statistical data with the results of the comparison between the unfactored SSR in composite slab

Table 5 Statistical data from results using different procedures for SSR prediction in SSL without transverse load

	d = 13, 16,19 and 22 (mm)					d = 25, 27 and 30 (mm)				
Statistical parameters	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}AISC/AASHTO}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}EC\text{-}4}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}GB}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}JSCE}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}AS}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}ISC/AASHTO}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}EC\text{-}4}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}GB}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}SCE}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}AS}$
Mean	0.98	1.17	1.17	0.98	1.23	0.76	0.95	0.91	0.76	0.95
Maximum value	1.39	1.74	1.77	1.39	1.73	0.98	1.22	1.16	0.98	1.22
Minimum value	0.75	0.87	0.89	0.75	0.94	0.47	0.59	0.56	0.47	0.59
Coefficient of variation	0.12	0.16	0.14	0.14	0.12	0.15	0.15	0.15	0.15	0.15
$Q_{sc\text{-}test}$ / $Q_{sc\text{-}PREDICTED}$ < 0.8 (%)	3.8	0.0	0.0	7.5	0.0	66.7	11.1	22.2	66.7	11.1
0.8 < $Q_{sc\text{-}test}$/ $Q_{sc\text{-}PREDICTED}$ < 1 (%)	58.4	15.1	9.4	49.1	3.8	33.3	55.5	55.6	33.3	55.6
1 < $Q_{sc\text{-}test}$ / $Q_{sc\text{-}PREDICTED}$ < 1.2 (%)	32.1	47.2	62.3	37.7	39.6	0.0	27.9	22.2	0.0	27.8
$Q_{sc\text{-}test}$ / $Q_{sc\text{-}PREDICTED}$ > 1.2 (%)	5.7	37.7	28.3	5.7	56.6	0.0	5.5	0	0.0	5.5
R^2 of the prediction:	*0.864*	*0.583*	*0.717*	*0.838*	*0.53*	*0.03*	*0.09*	*0.05*	*0.03*	*0.09*

Table 6 Statistical data from results generated using different procedures for SSR prediction in SSL with transverse load

Source	Series	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}AISC/AASHTO}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}EC\text{-}4}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}GB}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}JSCE}$	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}AS}$
Rambo-Roddenberry (2002)	S1	1.16	1.37	1.34	1.09	1.46
	S2	1.15	1.36	1.33	1.08	1.45
	S3	1.11	1.32	1.29	1.05	1.41
	S4	1.09	1.30	1.27	1.03	1.39
	S5	0.83	1.04	0.99	0.84	1.03
	S6	0.86	1.08	1.03	0.88	1.08
	S7	1.02	1.28	1.22	1.02	1.28
Mean		1.03	1.25	1.21	1.00	1.30
Maximum value		1.16	1.37	1.34	1.09	1.46
Minimum value		0.83	1.04	0.99	0.84	1.03
Coefficient of variation		0.13	0.11	0.12	0.10	0.14
$Q_{sc\text{-}test}$ / $Q_{sc\text{-}PREDICTED}$ < 0.8 (%)		0.0	0.0	0.0	0.0	0.0
0.8 < $Q_{sc\text{-}test}$ / $Q_{sc\text{-}PREDICTED}$ < 1 (%)		28.6	0.0	14.3	28.6	0.0
1 < $Q_{sc\text{-}test}$ / $Q_{sc\text{-}PREDICTED}$ < 1.2 (%)		71.4	28.6	14.3	71.4	28.6
$Q_{sc\text{-}test}$ / $Q_{sc\text{-}PREDICTED}$ > 1.2 (%)		0.0	71.4	71.4	0.0	71.4
R^2 *of the prediction by procedures*:		*0.1*	*0.03*	*0.05*	*0.12*	*0.01*

using PSS ($Q_{sc\text{–}AISC}$, $Q_{sc\text{–}EC\text{-}4}$, $Q_{sc\text{-}Rambo\text{-}R. (2002)}$ and $Q_{sc\text{–}Bonilla\ et\ al. (2015)}$) and experimental tests. The mean values of $Q_{sc\text{-}test}$ /$Q_{sc\text{-}AISC\text{-}LRFD}$, $Q_{sc\text{-}test}$ /$Q_{sc\text{-}EC\text{-}4}$, $Q_{sc\text{-}test}$ /$Q_{sc\text{-}Rambo\text{-}R. (2002)}$ and $Q_{sc\text{-}test}$ /$Q_{sc\text{–}Bonilla\ et\ al. (2015)}$ ratios are 0.91, 1.18, 1.04 and 1.08, respectively, with their corresponding COV (coefficient of variation) of 0.17, 0.22, 0.16 and 0.13, respectively. One should note that the procedure developed by Bonilla *et al*. (Bonilla *et al*. 2015) shows the lowest COV value equal to 0.13. The $Q_{sc\text{-}test}$/$Q_{sc\text{-}PREDICTED}$ < 0.8 and 0.8 < $Q_{sc\text{-}test}$/ $Q_{sc\text{-}PREDICTED}$ < 1 ratios, for the different procedures analyzed, show that the prediction by AISC (2010) was more unconservative than the prediction by Eurocode-4 (2004), Rambo-Roddenberry (2002) and Bonilla *et al*. (2015). However, the $Q_{sc\text{-}test}$/$Q_{sc\text{–}PREDICTED}$ > 1.2 ratios indicates that Eurocode-4 (2004) is excessively conservative. One should note that, the methods of Rambo-Roddenberry (2002) and Bonilla *et al*. (2015) offer the higher R^2 coefficients values equal to 0.871 and 0.873 respectively (see Table 8).

Table 9 summarizes several results of the comparison between unfactored SSR prediction, according to the procedures above mentioned, and push-out tests without transverse load. The mean values of $Q_{sc\text{-}test}$ /$Q_{sc\text{-}AISC\text{-}LRFD}$, $Q_{sc\text{-}test}$ /$Q_{sc\text{-}EC\text{-}4}$, $Q_{sc\text{-}test}$ /$Q_{sc\text{-}Rambo\text{-}R. (2002)}$ and $Q_{sc\text{-}test}$ /$Q_{sc\text{–}Bonilla}$

Table 7 Experimental studies of push-out tests using PSS for the comparison with design shear resistance

h_r	Source	Series[(*)]	d (mm)	Stud position
25 mm	Diaz *et al*. (1998)	ST2, ST3, ST5, ST6	10, 13	MP, 2 MP
51 mm	Rambo-Roddenberry (2002)	D1, D2, D3, D4, D5, D6, D7, D8, D9, D10, D11, D12, D13, D14, D15, D16, D18, D26, D27, D28, D29	10, 13, 16, 19	SP, 2 SP, WP
	Lyons *et al*. (1994)	S1, S2, S3, S4, S10, S14, S17, S19, S21, S27, S28, S29	19	SP, 2 SP, WP, StP
	Sublett *et al*. (1992)	S3, S4, S14	19	SP, WP
76 mm	Rambo-Roddenberry (2002)	D20, D22	10	SP, WP
	Lyons *et al*. (1994)	S6, S7, S8, S13, S14, S15	19	SP, StP
	Sublett *et al*. (1992)	S1, S2, S13, S15, S16	19	SP, WP
	Jayas and Hosain (1988)	S56-7JDT-7, S57-8JDT-8	19	SP, 2 SP
	Robinson (1988)	QI, QII, TII, TVIII	19	SP, 2 SP, 2 WP
80 mm	Johnson and Yuan (1997)	G1F, G3FL, G5U	19	SP, WP

Note: (*) = Nomenclature of specimens can be found in references (Diaz *et al*. 1998, Jayas and Hosain 1988, Johnson and Yuan 1997, Lyons *et al*. 1994, Rambo-Roddenberry 2002, Robinson 1988, Sublett *et al*. 1992).

In the push-out test developed by Sublett *et al*. (1992), Lyons *et al*. (1994), Diaz *et al*. (1998) and Rambo-Roddenberry (2002) is applying a transverse load on the composite slab.

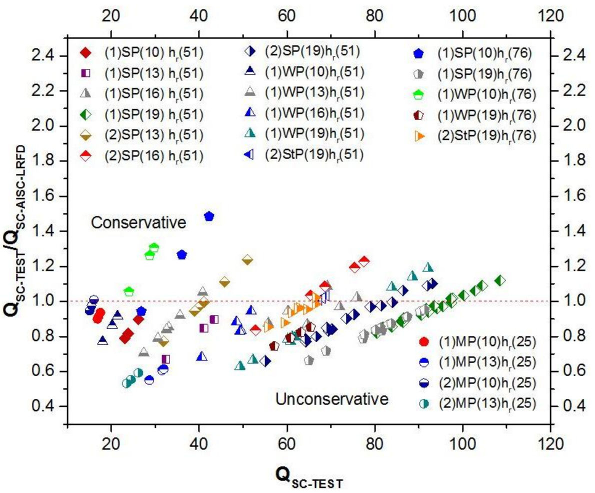

Fig. 4 Prediction of the SSR in composite beams using PSS under transverse load according to AISC (2010)

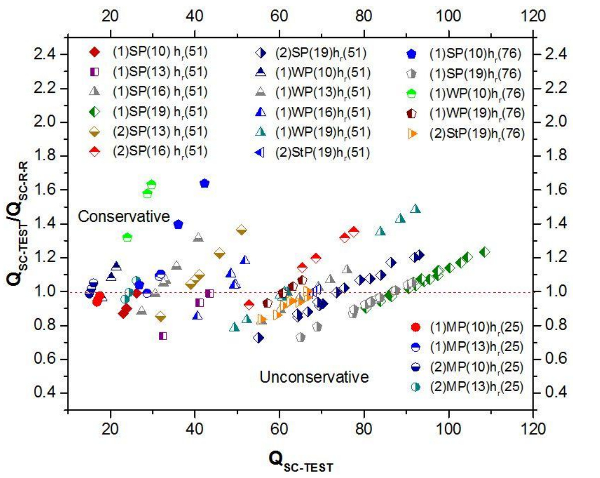

Fig. 6 Prediction of the SSR in composite beams using PSS under transverse load according to Rambo-Roddenberry (2002)

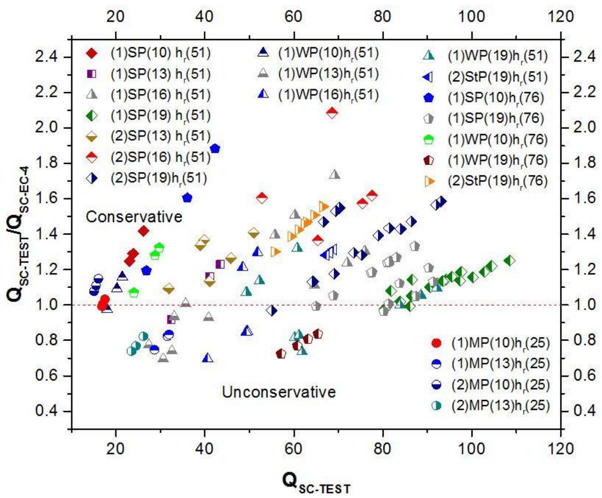

Fig. 5 Prediction of the SSR in composite beams using PSS under transverse load according to Eurocode-4 (2004)

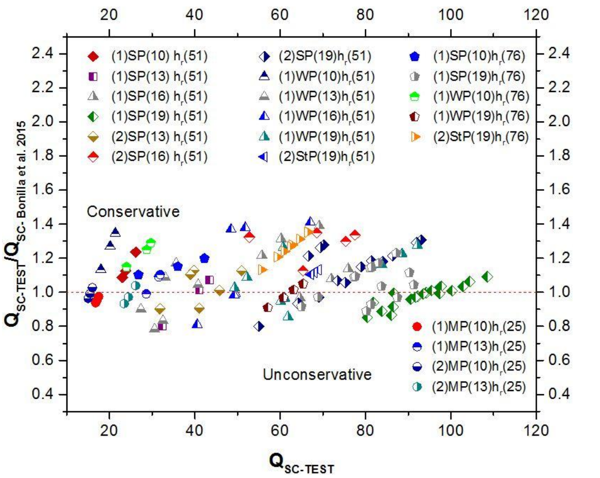

Fig. 7 Prediction of the SSR in composite beams using PSS under transverse load according to Bonilla *et al*. (2015)

Table 8 Statistical data from the prediction results generated using different procedures for the push-out test under transverse load

Statistical parameters	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}AISC}$	$Q_{sc\text{- }test}$ / $Q_{sc\text{-}EC\text{-}4}$	$Q_{sc\text{- }test}$ / $Q_{sc\text{-}Rambo\text{-}R.\ (2002)}$	$Q_{sc\text{- }test}$ / $Q_{sc\text{-}Bonilla\ et\ al.\ (2015)}$
Mean	0.91	1.18	1.04	1.08
Maximum value	1.49	2.09	1.64	1.41
Minimum value	0.53	0.69	0.73	0.78
Coefficient of variation	0.17	0.22	0.16	0.13
$Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ < 0.8 (%)	21.7	7.8	3.9	1.5
0.8 < $Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ < 1 (%)	53.5	15.5	45.7	32.6
1 < $Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ < 1.2 (%)	20.2	33.3	37.2	41.9
$Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ > 1.2 (%)	4.6	43.4	13.2	24.0
R^2 of the prediction by procedures:	*0.779*	*0.650*	*0.871*	*0.873*

Table 9 Statistical data from the prediction results generated using different procedures for the push-out test without transverse load

Source	Series	$Q_{sc\text{-}test}$ / $Q_{sc\text{-}AISC}$	$Q_{sc\text{- }test}$ / $Q_{sc\text{-}EC\text{-}4}$	$Q_{sc\text{- }test}$ / $Q_{sc\text{-}Rambo\text{-}R.\ (2002)}$	$Q_{sc\text{- }test}$ / $Q_{sc\text{-}Bonilla\ et\ al.\ (2015)}$
Jayas and Hosain (1988)	S56-7JDT-7	0.57	0.90	0.62	0.73
	S57-8JDT-8	0.78	1.03	0.86	1.04
Robinson (1988)	QI	0.83	1.40	0.94	1.14
	QII	0.66	1.30	0.72	0.84
	TII	0.79	1.57	0.87	1.02
	TVIII	0.73	1.16	0.92	1.04
Johnson and Yuan (1997)	G1F	0.91	1.27	1.00	0.95
	G3FL	0.86	1.36	0.95	1.02
	G5U	0.86	0.96	1.02	0.93
Mean		0.78	1.22	0.88	0.97
Maximum value		0.91	1.57	1.02	1.14
Minimum value		0.57	0.90	0.62	0.73
Coefficient of variation		0.14	0.18	0.15	0.13
$Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ < 0.8 (%)		55.6	0.0	22.2	11.1
0.8 < $Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ < 1 (%)		44.4	22.2	55.6	33.3
1 < $Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ < 1.2 (%)		0.0	22.2	22.2	55.6
$Q_{sc\text{-}test}$ / $Q_{sc\text{- }PREDICTED}$ > 1.2 (%)		0.0	55.6	0.0	0.0
R^2 of the prediction by procedures:		0.03	0.15	0.07	0.74

et al. (2015) ratios are 0.78, 1.22, 0.88 and 0.97, respectively.

The COV for $Q_{sc\text{-}test}$ /$Q_{sc\text{-}AISC\text{-}LRFD}$, $Q_{sc\text{-}test}$ /$Q_{sc\text{-}EC\text{-}4}$, $Q_{sc\text{-}test}$ /$Q_{sc\text{-}Rambo\text{-}R.\ (2002)}$ and $Q_{sc\text{-}test}$ /$Q_{sc\text{–}Bonilla\ et\ al.\ (2015)}$ ratios are respectively equal to 0.14, 0.18, 0.15 and 0.13. It is observed that the prediction according to AISC (2010) and Rambo-Roddenberry (2002) tends to overestimate the SSR values. Nevertheless, the tendency of the Eurocode-4 (2004) is to underestimate those values. In this case, apparently, Bonilla's *et al*. (2015) ethod provides a better SSR prediction with an R^2 coefficient equal to 0.74.

5. Conclusions

This paper present the main procedures to predict the SSR embedded in SSL, and in composite slab using PSS with rib perpendicular to steel beam. A large number of experimental studies reported in the literature are summarized. The equation developed by Ollgaard *et al*. (1971) has been the fundamental basis of the current SSR estimation methods. The SSR prediction methods of the six codes (AASHTO 2014, AISC 2010, AS-2327.1 2003, Eurocode-4 2004, GB50017 2003, JSCE 2007) do not take into account the effect of the longitudinal and transverse spacing of the stud connectors when they are installed in solid slabs.

When the stud connectors are installed in SSL without transverse load in the push-out tests, the codes AISC (2010), AASHTO (2014) and JSCE (2007) offer suitable SSR results fundamentally for commercial stud diameters between 13 and 22 mm. However, for studs diameters of 25, 27 and 30 mm, the mentioned codes prediction give unconservative results. The estimation by Eurocode-4 (2004), GB50017 (2003) and AS-2327.1 (2003) offers very conservative SSR values for stud diameters between 13 and 22 mm. Nevertheless, for stud diameters of 25, 27 and 30 mm, Eurocode-4 (2004), GB50017 (2003) and AS-2327.1 (2003) lead to less conservative results.

For stud shear connectors with PSS and transverse load in the push-out tests, the SSR prediction by AISC (2010) gives unconservative values. Whereas Eurocode-4 (2004) offers very scattered values. In some cases, Eurocode-4 (2004) results are unconservative and in others the results are excessively conservative. The Rambo-Roddenberry (2002) method and Bonilla *et al*. (2015) method gives more conservatives results than AISC (2010) and Eurocode-4 (2004), and less scattered prediction than Eurocode-4 (2004).

There are few experimental studies in the two following conditions: (1) push-out test with applied transverse load in the SSL, and (2) push-out test using PSS without transverse load. In the first condition, the codes AS-2327.1 (2003), Eurocode-4 (2004), and GB50017 (2003) tend to underestimate the SSR. In the second condition, the prediction according to AISC (2010) and Rambo-Roddenberry (2002) tend to overestimate the SSR value. On the contrary, the tendency of the Eurocode-4 (2004) is to underestimate the SSR and Bonilla's *et al*. (2015) method, apparently, provides a better SSR prediction.

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Notation

The following symbols are used in this paper:

A_c Area of concrete pull-out failure surface.
A_{sc} Cross-sectional area of headed stud shear connector.
d Diameter of headed stud shear connector.
$e_{mid\text{-}hr}$ Distance from center of stud to mid-height of deck web on loaded side.
E_c Initial Young's modulus of concrete.
E_{cm} Secant modulus of elasticity of concrete tabulated in the Eurocode-4 (2004).
E_s Initial Young's modulus of headed stud shear connector.
f'_c Specified minimum compressive strength of concrete.
f_{ck} Characteristic value of the cylinder compressive strength of concrete.
f_{cu} Cube strength of concrete.
f_u Specified ultimate tensile strength.
F_u Specified ultimate tensile strength of the material of the stud.
h Height of headed stud shear connector.
h_r Nominal rib height.
n Number of studs subjected to similar displacements.
n_r Number of shear connectors in one rib of the profiled steel sheeting.
t_f Flange thickness of the steel beam.
w_r Average width of deck rib.
$Q_{sc-AISC}$, $Q_{sc-AASHTO}$, $Q_{sc-EC\text{-}4}$, Q_{sc-GB}, $Q_{sc-JSCE}$ and Q_{sc-AS} are the unfactored resistance calculated using the AISC (2010), AASHTO (2014), Eurocode-4 (2004), GB50017 (2003), JSCE (2007) and AS-2327.1 (2003) respectively.
$Q_{SC\text{-}PREDICTED}$ Unfactored predicted value from the Standard.
Q_{sc-rib} Unfactored resistance of stud shear connector embedded in composite slab using PSS.
Q_{sc-sol} Unfactored resistance of stud shear connector embedded in solid slab.
$Q_{sc\text{ - }test}$ Resistance of shear connection per stud obtained from push-out tests.
R_d Coefficient that takes into consideration the thickness of the PSS.
R_g, R_n Reduction factor associated with the number of studs inside the PSS rib.
R_p Reduction factor associated with the position of studs in the PSS rib.
γ Ratio of the minimum tensile strength to yielding strength of the stud.
$\overline{(.)}$ Throughout the entire article, the dash is referred to the average value of the materials.

The following abbreviations are used in this paper:

SSL Solid slab.
PSS Profiled steel sheeting.
SSR Stud shear resistance.
COV Coefficient of variation.
SP Connector in favorable or strong position.
WP Connector in unfavorable or weak position.
MP Connector in middle position.
StP Connector in staggered or zigzag position.