Out-of-plane shear strength of steel-concrete sandwich panels

Juan Sagaseta¹, Phil Francis^{1, 2}

¹University of Surrey, UK

²Steel Construction Institute, UK

Abstract

Steel-concrete-steel (SCS) sandwich panels consist of two steel plates connected with tie bars filled with concrete; composite action is achieved using headed studs in the plates. This form of composite construction has recently regained interest in the construction industry as it allows modular construction and decongestion of reinforcement which is particularly useful in large infrastructure such as tunnels, wind turbines and nuclear energy facilities. This paper investigates the out-of-plane shear resistance of SCS panels without shear reinforcement. In practice, the spacing between tie bars acting as shear reinforcement can be significant and the shear resistance is governed in some cases by that of a member without shear reinforcement. A qualitative comparison of the shear transfer actions is presented between SCS and conventional reinforced concrete (RC) members without shear reinforcement. Existing design formulae for shear in RC are applied to existing experimental data of SCS panels. This study shows that the shear resistance models for RC give conservative predictions of strength of SCS slender panels with low or medium levels of shear connection at the interface between the concrete and the steel. This inbuilt conservatism is due to the bond-slip of the interface resulting into a concentration of the flexural cracks towards mid-span which allows the development of full arching action (shift of Kani's valley). A strut-and-tie model is presented for SCS which provides more accurate predictions of strength in such cases and also in other cases such as short-span members (discontinuity region).

Keywords

steel-concrete sandwich panels, shear design, bond-slip, Kani's valley.

1 Introduction

Steel-concrete-steel (SCS) sandwich panels consist of two steel plates filled with concrete as shown in Fig. 1. The steel plates are connected with each other with tie bars to hold the plates together during concrete pouring, which then removes the need for temporary propping to support the fresh concrete pressures. The steel plates are connected to the concrete through headed studs to enable composite action. The two steel plates act as reinforcement and formwork which reduces on-site time of construction compared to conventional RC structures (IRF, 2013; Sener, Varma, 2014). This form of construction has recently regained interest in the construction industry, especially for large infrastructure facilities in transport and energy sectors (e.g. tunnels, wind turbines and nuclear energy facilities as shown in Fig. 1). The first developments of SCS construction took place in the 1980s when double skin composite structures were conceived for submerged tube tunnels (Oduyemi, Wright, 1989; Wright et al., 1989). Since then, different composite systems have been developed and used in industry although some aspects of the structural behaviour are still not well understood. This paper focuses on SCS panels with short headed studs and welded tie bars.

Figure 1: Example of steel-concrete modular construction using SCS walls (IRF, 2013).

The development of design codes for steel-concrete (SC) modular construction is still in its infancy. Japan, South Korea and most recently the USA have produced preliminary design codes (JEAC 4618, 2009; KEPIC-SNG, 2010; AISC N690S1-15, 2015). In Europe, the only available background documents are provided by the Steel Construction Institute (SCI) in the UK, including the guideline for SCS panels (Narayanan et al., 1994) and the Bi-steel manual (Bowerman et al., 2003). A European Consortium with contributors from industry is currently leading a project on the development of design rules for SC structures used in Nuclear Power Plants which will follow a similar format to Eurocodes (SCIENCE, FP7 2013-2017). The authors have contributed to this project which includes a review of the out-of-plane shear resistance of SCS members.

Existing experimental evidence of SCS beam tests shows a large variety of failure modes (Oduyemi, Wright, 1989; Roberts et al., 1996; Takeuchi et al., 1999; Chu et al. 2013; KEPRI, 2006; Sener, Varma, 2014; Ludovic, 2015). Some of these failure modes (Fig. 2) were identified in early experimental work by Oduyemi et al. (1989); namely buckling of the steel plate in compression, crushing of the concrete and yielding of the plate in tension, de-bonding of the tension plate due to slip (interface- shear) and shear failure of the concrete (out-of-plane or beam shear). Some of these failure modes are similar to those observed in RC members failing in bending and shear. SCS members are generally more flexible in bending compared to RC members due to limited tension stiffening and potential bond-slip of the interface between the steel plate and the concrete. These differences are further discussed in this paper with regards to the out-of-plane shear resistance of members without shear reinforcement.

Figure 2: Modes of failure in SCS members.

SCS panels are generally reinforced in shear with tie bars connecting the steel plates acting as transverse (out-of-plane) reinforcement. The number of tie bars and spacing is governed in design by the pressure of the fresh concrete applied to the formwork during construction stages; this has been demonstrated during the design of a reference SC building in the SCIENCE project (Tusher, 2017). These design considerations lead to typical tie bar spacings between 400 mm and 800 mm with bar diameters ranging from 8 mm to 25 mm. It is worth noting that the tie bar spacing in many tests in the literature can be larger than the recommended maximum spacing for shear reinforcement of $0.75d$ (Eurocode 2, 2004). In addition, the shear reinforcement ratio can be low in many cases, meaning the shear strength is governed by the shear resistance of the member without shear reinforcement ($V_{R,c}$). Hence, the study of the out-of-plane shear resistance of SCS members without shear reinforcement can be relevant although the lack of tests compared to shear reinforced cases might indicate otherwise. This paper focuses mainly on $V_{R,c}$ of SCS panels without tie bars, in which the shear resistance is assessed using similar formulae applied to RC members. It is shown in this work that bond-slip at the interface can have a positive effect on shear strength.

2 Out-of-plane shear in SCS without shear reinforcement

2.1 Shear-transfer actions

The shear behaviour of SCS members without transverse reinforcement can be investigated by looking at the shear-transfer actions similarly as in RC members (Fernández Ruiz et al., 2015). Figure 3 shows the shear failure of a SCS beam tested by Varma et al. (2011); the mode of failure is similar to that observed in RC members, with the sudden development of a diagonal shear crack from an existing flexural crack. The shear-resisting actions have been traditionally classified by researchers as beam or arching action depending on whether the inner lever arm is constant or not respectively. Beam shear-transfer actions rely on the variation of the forces in the tension chord and they include cantilever action (described by Kani's tooth model 1964), aggregate interlock and dowel action. Aggregate interlock action along the critical crack in SCS panels will be similar to RC members whereas dowel and cantilever actions can be different due to the presence of shear studs in the steel plates.

Figure 3: Diagonal shear failure of simply supported SCS beam test (Varma et al., 2011).

Dowel action in SCS members is clearly different to RC members where the flexural reinforcement is embedded in the concrete. Tests show that the delamination crack in SCS beams failing in shear can develop at a nearly horizontal surface near the head of the shear studs at the tensile plate (Fig. 3). The contribution of dowel action in the tests will depend therefore on the number of studs, type and layout. The influence of these parameters explains the apparently contradicting views on dowel action reported in the literature of SCS. For

example, Oduyemi et al. (1989) who used long but fewer studs in their beam tests reported that dowel action was negligible and even lower than RC members whereas Leng & Song (2016), who used a large number of studs in their tests, proposed a model based on dowel action that predicts the strength of their tests satisfactorily. Regarding cantilever action, it can be envisaged that the presence of shear studs on the compression zone would be beneficial towards shear strength. This could be justified on the studs providing confinement in the compression region and potentially enhance the development of arching action above the diagonal crack. Similarly to dowel action, the shear at the compression head would be influenced by the number, type and layout of the studs. Considering that current design formulae for shear of SCS members do not take into account the shear studs explicitly, the phenomena discussed above will be a source of scatter and uncertainty in the predictions of test results. Therefore, in this paper only tests with similar type of studs and layout are investigated; only the variation in the number of studs is analysed in section 3.

If beam shear-transfer actions fail to resist the applied shear, arching action will develop when this is feasible. In such cases, the force in the tension chord is constant and the load is transferred to the support through a direct strut (full-arching action) or through an indirect strut or elbow-shaped strut (Muttoni, Fernández Ruiz, 2008). Arching action governs in cases such as short-span beams (shear span-to-effective depth ratio a/d between 1 and 2) whereas in slender beams its contribution is much more restricted (Muttoni, Fernández Ruiz, 2008; Sagaseta, Vollum, 2010). This effect is well known and explained by Kani's valley indicating that the shear strength is highly influenced by the relative position of the flexural cracks with respect to the direct strut. The position of the flexural cracks is mainly governed by the slenderness of the member. In SCS members there are other factors affecting the location of the flexural cracks such as the bond characteristics between the flexural reinforcement and the concrete and also the presence of shear studs which can induce the development of flexural cracks where the shear studs are placed. The influence of flexural cracks in SCS is further discussed in section 3.

2.2 Shear resistance of SCS members according to RC shear models

As part of the SCIENCE project, a database of 175 SCS beam tests was gathered from the literature including tests carried out in the project; the tests are disseminated along many publications (Oduyemi, Wright, 1989; Roberts et al., 1996; Takeuchi et al., 1999; Chu et al. 2013; KEPRI, 2006; Foundoukos et al. 2008; Sohel et al. 2011; Sener, Varma, 2014; Ludovic, 2015; Sener et al. 2015; Tan et al. 2015; Yang et al. 2016). From this database, only 25 tests correspond to slender beams failing in shear; short-span beam tests and tests failing in bending and interfacial-shear were not considered in subsequent studies. Only 10 tests from the 25 slender beam group correspond to specimens without shear reinforcement. Furthermore, 5 of those tests were not considered in this paper due to having either rather long studs (reaching mid-height or larger) or having some tie bars outside the critical zone which could have some additional contribution to the strength. The remaining five tests (summarized in Table 1) are believed to have similar characteristics to establish a reasonable comparison and to enable a better understanding of the different physical phenomena underpinning the shear strength. A parallel study, including members with shear reinforcement, was carried out by Francis et al. (2017) using the same database focusing on the calibration of the partial factors used in Eurocodes (EN 1990:2002; EN 1992-1-1:2004).

Table 1. SCS beam test without shear reinforcement considered in the analysis

Test	f_c MPa	f_y MPa	h mm	d mm	b mm	t mm	ρ %	a/d	η_{sc}	V_{test} kN
E1 (KEPRI, 2006)	39.0	288	150	147	150	6.0	4.08	3.74	0.96	39
SP1-1 (Varma et al. 2011)	42.1	448	457	454	306	6.4	1.41	3.21	1.13	206
SP1-3 (Varma et al. 2011)	42.1	448	457	452	306	9.5	2.10	3.22	0.76	224
SP1-4 (Varma et al. 2011)	42.1	448	457	454	306	6.4	1.41	2.50	0.79	230
JZ3.0-N (Leng, Song, 2016)	25.6	350	300	297	300	6.0	2.02	3.03	1.65	100

Note: aggregate size d_g was 10 mm, 19 mm and 25 mm for tests E1, SP and JZ3.0-N respectively.

The tests shown in Table 1 were analysed using three different shear resistance models widely accepted in RC design; viz. Eurocode 2 (EN 1992-1-1:2004), fib Model Code for Concrete Structures 2010 (2013) and the Critical Shear Crack Theory (CSCT). The first approach is empirical and it is based on the work from Zsutty (1968) whereas the other two are mechanical models considering the strain in the concrete, (Vecchio, Collins, 1986; Muttoni, Fernández Ruiz, 2008). The models and the application to SCS are described below.

In Eurocode 2,

$$V_{R,c} = 0.18k(100\rho f_c)^{1/3}bd \quad (1)$$

In fib MC2010, levels of approximation (LoA) I and II

$$V_{R,c} = k_v(f_c)^{1/2}zb \quad (2)$$

$$k_v(\text{LoA I}) = \frac{180}{1000 + 1.25z} \quad (2b)$$

$$k_v(\text{LoA II}) = \frac{0.4}{1 + 1500\varepsilon_x} \cdot \frac{1300}{1000 + k_{dg}z} \quad (2c)$$

In the CSCT,

$$V_{R,c} = \frac{(1/3)}{1 + 120 \frac{\varepsilon d}{16 + d_g}} (f_c)^{1/2}zb \quad (3)$$

The partial factors of safety were taken as 1 in all cases. The inner level arm (d) was measured from the top of the member to the centroid of the tensile plate; the level arm (z) was taken as $0.9d$. The flexural reinforcement ratio (ρ) is obtained as the thickness of the tensile plate over the inner lever arm (t/d) as in shear formulae in the Bi-steel manual (Bowerman et al., 2003). Size effect is considered by the three approaches; Eq. (1) uses coefficient $k = 1 + \sqrt{200/d} \leq 2$ (d in mm) whereas in Eq. (2) this is part of the reduction coefficient k_v . In the case of the CSCT, size effect is coupled with the strain effect (Fernández Ruiz et al. 2015). Equations (2) and (3) consider the reduction in strength due to the use of small size aggregates or different types of concretes such as high-strength concrete which could be the case in SC construction. This is taken into account using parameter d_g (size of the maximum aggregate) and coefficient $k_{dg} = 32/(16 + d_g) \geq 0.75$. Equations (2c) and (3) are a function of a reference strain at a given control section and depth; in Eq. (2c) ε_x is taken at a cross section at a distance d from the applied load at mid-depth whereas in Eq. (3) ε is taken at a distance $d/2$ from the applied load and $0.6d$ from the top (compression fibre). These strains are given by Equations (4) and (5) in MC2010 and Muttoni & Fernández Ruiz (2008) respectively.

$$\varepsilon_x = \frac{1}{2E_s A_s} \left(\frac{M_{Ed}}{z} + V_{Ed} \right) \quad (4)$$

$$\varepsilon = \frac{M_{Ed}}{bd\rho E_s (d - c/3)} \cdot \frac{0.6d - c}{d - c} \quad (5)$$

where $c = d\rho n(\sqrt{1 + (2/\rho n)} - 1)$ and $n = E_s/E_c$. M_{Ed} and V_{Ed} are the applied design bending moment and shear respectively at the shear control sections. These strain equations were derived from cross-sectional analyses assuming that plane sections remain plane with different considerations. In Eq. (4), ε_x is equal to half the strain in the tension chord (i.e. compression chord strain equal to zero) whereas in Eq. (5) it is assumed that the moment does not cause yielding of the reinforcement and therefore concrete behaves linearly in compression with zero stresses in tension. The effect of shear on strain is considered in Eq. (4) adopting a free-body diagram with an assumed inclination of the diagonal strut of $\sim 25^\circ$ (i.e. $\cot\theta=2$) to obtain a simplified conservative expression. In Eq. (5), ε is obtained from bending considerations only, however the strain is assessed at a shear control section closer to the load ($0.5d$). This control section was proposed to consider the increase of strains at the critical crack due to shear and due to the loss of bond along the delamination crack at the level of the flexural reinforcement (Fernández Ruiz et al., 2015). In both expressions, the influence of the compression plate on the strain is neglected. It is important to note that the strain relationships (4) and (5) assume a perfect bond between the steel and the concrete. Moreover, Eq. (1) was derived empirically using tests with embedded reinforcement with good bond characteristics. This suggests that any effects related to a reduction in bond in the SCS panels will not be captured by the three resistance models (this is further discussed in section 3.1).

Figure 4 shows the ratio $V_{test}/V_{R,c}$ between the experimental and shear resistance predicted using Equations (1), (2) and (3) for different values of a/d . Figure 4 shows that the strength predictions using the three models are very similar, giving a similar average $V_{test}/V_{R,c}$ equal to 1.19, 1.21 and 1.09 for equations (1), (2c) and (3) respectively with very similar scatter (COV around 15% in the three cases). The amount of data and the range of the different test parameters are not sufficient to capture significant differences between the models, although some differences are expected due to the different considerations made regarding strain and ρ . Equation (2b), corresponding to fib MC2010 level of approximation LoA I, gives conservative estimates as expected. In all cases, the average value of $V_{test}/V_{R,c}$ is slightly higher (around 10%) in SCS panels compared to RC tests. Studies using these formulae in RC members (Walraven, 2002; Muttoni, Fernández Ruiz, 2008; Sigrist et al. 2013) provide an average value of $V_{test}/V_{R,c}$ equal to 1.02, 1.15 and 0.99 for Equations (1), (2c) and (3) respectively with COV of around 10% in the three cases. The reasons behind the conservative predictions of the SCS tests are discussed in section 3 in terms of the influence of bond-slip.

The slight conservatism in the SCS predictions does not pose a high level of concern. It does however raise the need to carry out further research and testing; in view of Fig. 4, further tests with $a/d > 3.5$ should be performed. Mechanical models could be developed in which the shear-transfer actions were modelled explicitly or perhaps adopting a shear strain-based approach with bond-slip considerations. Lacking a better model, a reasonable preliminary solution for design is to calibrate the partial factors in the shear resistance models used in RC. This can be done with a reliability analysis as suggested in Annex D (EN 1990:2002) which takes into account penalization factors due to the limited experimental data available and the known limitations and uncertainties of the resistance functions (Francis et al. 2017).

Figure 4: Shear strength predictions of SCS slender beam tests using RC shear models.

3 Influence of the shear connection ratio

In order to investigate the reasons behind the conservatism in the predictions, $V_{test}/V_{R,c}$ was compared against different parameters. Figure 4 shows the results for slender beams with a/d ratios ranging from 3.7 to 2.5; a very small increase of $V_{test}/V_{R,c}$ is observed as a/d reduces. This increase becomes significant for a/d lower than around 2.5 due to arching action similarly to that in RC short-span beams (Clark, 1951; Swamy et al. 1970; Collins, Mitchell, 1991). Varma et al. (2011) suggested that SCS tests with a/d of 2.5 might be influenced by arching action and recommended using larger slenderness of around 3 to obtain a lower-bound strength in testing. This paper shows that this is true only for certain levels of shear connection at the interface between the steel and the concrete. Whilst the scatter shown in Fig. 4 is not significant, it was found that it can be explained due to the difference in the degree of shear connection in the tests. The degree of shear connection, defined by Eq. (6), is an indicator of the relative stiffness of the shear connection to the capacity of the plate. Beams with a shear connection ratio greater than 1 will fail in the plate before the studs become overloaded and ‘full shear connection’ is achieved.

$$\eta_{sc} = \frac{n_{studs} P_{Rd}}{N_{p,Rd}} \quad (6)$$

In Eq. (6) n is the number of studs in the ‘bending span’, P_{Rd} is the resistance of an individual shear connector and $N_{p,Rd}$ is the force that can be carried in the plate (i.e. $f_y t$). The ‘bending span’ is defined as the distance between the critical cross-section and the nearest point of inflection. This definition can be problematic in statically indeterminate cases; rules are summarized in the SCIENCE design guide (2017). In this study the beams in the sample are all statically determinate and therefore η_{sc} can be easily determined.

Figure 5 suggest that there is a decreasing trend of the $V_{test}/V_{R,c}$ ratio with increasing the shear connection ratio; tests with a very stiff shear connection have $V_{test}/V_{R,c}$ near 1 which seems reasonable as the behaviour of RC and SCS is similar. Slender beam tests with medium and low levels of shear connection shown in Fig. 5 had a higher $V_{test}/V_{R,c}$ than test with almost full shear connection. These results might seem odd at first instance as the trend is opposite to tests with shear reinforcement in which lower bond results in a reduced shear strength as shown experimentally and analytically by Hong et al. (2010) and Qin et al. (2016).

Figure 5: Influence of shear connection on strength and relative position of critical crack.

The enhanced strength of the tests without shear reinforcement with low η_{sc} can be easily explained by looking at the crack pattern (Fig. 5) which shows that as η_{sc} reduces the direct strut between the load and the support can develop fully. Figure 5 shows that as the level of shear connection increases, the flexural cracks form at a further distance from the loading point and they start intercepting the direct strut which results in a reduction in strength (right hand side of Kani's valley). A similar experimental observation of enhanced strength in RC beams with low bond flexural reinforcement was made by Leonhard & Walther (1962) as pointed out by Muttoni & Fernández Ruiz (2008). It can be concluded that SCS tests with partial or low shear connection ratio can result in a shift of Kani's valley towards higher values of a/d resulting in arching action in tests with relatively large a/d ratio (larger than 3) assuming that the members are fully anchored at the ends.

4 Strut-and-tie modelling of SCS panels

Tests on the left hand side of Kani's valley (direct strut can develop) can be analysed using plasticity based approaches such as the strut-and-tie method (STM). A STM is presented here for SCS panels without shear reinforcement and full-arching action. The model is then used to reanalyse the five tests shown in Table 1. The proposed approach (Fig. 6) is based on the model developed by Sagaseta & Vullum (2010) for RC members. According to this model, the failure load V_R and inclination of the strut θ can be obtained from equations (7) and (8)

$$V_R = 2(l_b \sin^2 \theta + x_c \sin 2\theta) \cdot b(0.6v f_c) \quad (7)$$

$$V_R = 4 \tan \theta [h - (x_c/2) - (a) \tan \theta] \cdot b(v f_c) \quad (8)$$

where l_b and l_t are the length of the support and loading plates respectively, x_c is half the length of side the node at the bottom (vertical side), b is the width of the specimen and h is the total height (Fig. 6). The model assumes that failure is governed by crushing of the direct strut; stress limit from Eurocode 2 is used which is equal to $0.6v f_c$ with $v = (1 - f_c/250)$. More refined predictions of strength could be obtained using a limit on the concrete strength in the strut based on the strain in the bottom tie using Collins & Mitchell (1991) formula.

Figure 6: Geometry of proposed STM for SCS panels without shear reinforcement with full-arching action.

In Eq. (7) the strut width is calculated from a simplified nodal geometry assuming that full anchorage can be developed at the back of the node and that nodal stresses are not critical. In calculating distance x_c in SCS members, the contribution of the shear studs in developing bond stresses can be considered approximately by calculating the centre of gravity of the steel plate and studs according to Eq. (9)

$$x_c = \frac{4t^2 s_x s_y + \pi \phi_{\text{stud}}^2 (h_{\text{stud}}^2 + 2h_{\text{stud}}t)}{8ts_x s_y + 2\pi \phi_{\text{stud}}^2 h_{\text{stud}}} \quad (9)$$

where t is the thickness of the plate, s_x and s_y are the stud spacing in the longitudinal and transverse directions, h_{stud} and ϕ_{stud} are the height and diameter of the stud respectively. This simplified approach does not consider the nonlinearity of the bearing stresses in the concrete around the shank of the stud connector. As shown by Johnson (1975) studs act as cantilevers with a high concentration of stresses at the base. This effect would result in an overestimation of distance x_c in Eq. (9), however this was not the case as the equation also assumes constant friction along the plate. Both effects seem to counterbalance as Eq. (9) is shown to give reasonable results. The small eccentricity developed in the assumed tension chord with respect to the plate carrying the tension is balanced by the pull-out forces from the studs at the interface during the bond-slip process (Fig. 2).

The STM approach was validated against 5 short-span SCS beam tests with a/d lower than 2 tested by Taekuchi et al. (1999). The results are summarized in Fig. 7. The average value of V_{test}/V_R using the STM for $a/d < 2$ was 1.12 with COV equal to 11%. Using Eurocode 2 in this case requires the use of the shear enhancement factor ($\beta = 2d/a$) in Eq. (1). Using this approach resulted in a $V_{\text{test}}/(\beta V_{R,c})$ ratio equal to 1.29 with COV equal to 29%.

Figure 7 shows that the STM also gives reasonable predictions of strength of the slender beams shown in Table 1 (average V_{test}/V_R equal to 0.97 with COV 17% for $a/d > 2$). The results are reasonable and conservative in the tests with low shear connection ratio. Figure 7 shows that STM predictions have a larger scatter for $a/d > 2$ which is due to the influence of η_{sc} and a/d . The three points below the line $V_{\text{test}}/V_R = 1$ in Fig. 7 have $a/d > 2.5$ and a relatively large shear connection ratio ($\eta_{sc} > 0.8$). In this case the STM is expected to give unsafe results in many cases as the flexural crack can intercept the strut as shown in Fig. 7.

Figure 7: Predictions of shear strength according to STM and influence of a/d and η_{sc} .

It can be concluded that the application of STM in SCS members is feasible and provides reasonable predictions of strength of discontinuity regions (D-regions) such as short-span beams. Moreover, the suitability of the STM in the case of slender beams with a low shear connection confirms the shift in Kani's valley in such cases. In design of slender members, the use of STM would not be recommended as the gain in accuracy in cases with partial shear connection would not compensate the higher complexity in the analysis. Whilst partial shear connection is predominant in tests in the literature, it is questionable whether this would be the case in practice where engineers often design SCS members with full shear connection.

5 Conclusions

Steel-concrete sandwich panel construction offers significant advantages in design of large infrastructure. The design of out-of-plane shear for members without shear reinforcement is investigated and a comparison between SCS and RC members is made. The following conclusions can be drawn from this paper

- 1) Similar shear-transfer actions can be identified in SCS members as in RC although their contribution to the shear resistance can be different due to the influence of the shear studs in the tension and compression chords. Dowel action and shear transfer at the compression head can be different whereas aggregate interlock and arching action can be similar to RC members.
- 2) The three resistance models investigated for shear in RC (Eurocode 2, fib MC2010, CSCT) provided similar predictions of strength of SCS slender beams without shear reinforcement. The accuracy in the predictions was similar to RC members with a slight level of conservatism (average $V_{\text{test}}/V_{R,c}$ was around 10% higher in SCS). Such models can be implemented in future codes after a recalibration of the partial factors for safety.
- 3) It is shown that the inbuilt conservatism in the three resistance models is due to the relative position of the flexural cracks in many tests which are localized nearer the loading point compared to RC tests. This crack pattern allowed the development of a direct strut (even in slender beams) since the longitudinal reinforcement was sufficiently anchored at the ends which resulted in a significant increase in strength. This shear enhancement can be viewed as a shift of Kani's valley.
- 4) The bond-slip and the shear connection ratio of the interface between the steel and the concrete are shown to play a major role in the shift of Kani's valley. This justifies that for slenderness a/d of around 3 the shear strength increased as the level of shear connection reduced which is opposite to that observed in shear reinforced beams.
- 5) It is shown that the strut-and-tie method presented, using similar principles as in RC design, can be applied to D-regions in SCS members such as short-span beams. The STM model presented provides reasonable predictions of beam tests with a/d from 0.5 to 2 (normal range of application) and varying shear connection ratios.
- 6) The proposed STM provided more accurate predictions of slender beams ($a/d \approx 3 - 4$) with low shear connection ratios ($\eta_{sc} < 0.8$) than the three shear resistance models investigated due to the shift in Kani's valley. However, for practical purposes in shear design of SCS, where the level of shear connection is uncertain, it is recommended to use the proposed STM only in D-regions and members with $a/d < 2$.

6 Acknowledgements

The research leading to these results has received funding from the European Community's Research Fund for Coal and Steel (RFCS) under Grant Agreement no. RFSR-CT-2013-00017, Steel Composite for Industrial Energy and Nuclear Construction Efficiency (SCIENCE). The authors would like to acknowledge the contributions of collaborators in this project.

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