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EXPERIMENTAL INVESTIGATION INTO STUD SHEAR CONNECTIONS UNDER COMBINED SHEAR AND TENSION FORCES

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ABSTRACT

In order to investigate structural behaviour of stud shear connections with both solid and composite slabs under combined shear and tension forces, a systematic experimental investigation with a total of six test series are conducted. These include 11 standard push-out tests where the shear connections are under shear forces, and 11 modified push-out tests where the shear connections are under combined shear and tension forces. It should be noted that the testing method of the standard push-out tests recommended in EN 1994-1-1 is adopted, and headed shear studs are installed in either “favourable” or “unfavourable” positions of decking troughs. Test results of all the 22 push-out tests are fully presented, and these include typical modes of failure, measured load-slippage curves as well as shear resistances of the test specimens. It is found that the shear resistance of the shear connection should be reduced with a factor of 0.84 for the case of a solid concrete slab, and a factor of 0.75 for the case of a composite slab, provided that the tension force T_n is smaller than or equal to 0.267 Q_m where Q_m is the shear resistance of the shear connection. These data will be adopted in subsequent numerical investigations for calibration of advanced finite element models. These models will then be used to perform parametric studies on stud shear connections with a wide range of geometrical configurations and loading conditions to provide additional data for formulation of design rules.

Key words: Stud shear connections; Shear resistances; Push-out tests; Combined shear and tension forces; Load-slippage curves.

1. INTRODUCTION

In steel-concrete composite structures, effective shear connections are essential in all composite members in order to achieve good structural behaviour in resisting applied loads as well as in deforming consistently within the members. Stud shear connections are widely used in building structures in many parts of the world owing to simple installation. Among a number of different diameters, headed shear studs with a diameter of 19 mm and a height of 100 mm are widely used while the tensile strengths of the stud steel materials at 450 N/mm^2 are commonly specified. Owing to full development of a dowel action in the shear connections with solid concrete slabs, a typical value of the shear resistance at about 110 to 135 kN per stud is readily achieved.

In general, a stud shear connection should be stiff, strong and ductile, and as shown in Fig. 1, the load-slippage curve of such an effective shear connection may be described as follows:

a) In the initial part of the curve, the applied force is equal to at least half of the shear resistance, Q_m , i.e. 0.5 Q_m , at a small slippage, s , equal to 0.5 mm.
b) Along the loading part of the curve, the shear resistance, Q_m , is fully mobilized before or at a slippage of s_m , which is commonly taken as 6 mm.
c) Along the unloading part of the curve, the reduced shear resistance at a slip of s_u , which is commonly taken as 8 mm, should not be smaller than 0.8 Q_m.

Push-out tests have been widely employed to obtain the shear resistances of these stud shear connections. Based on various experimental investigations (Oehlers & Johnson, 1987; Mottram & Johnson, 1989; Yuan & Johnson, 1998; Oehlers & Bradford, 1995), the deformation characteristics of stud shear connections are generally considered to depend on:

a) compressive and tensile strengths of the concrete as well as sizes of aggregates;
b) yield and tensile strengths of headed studs as well as their shapes and sizes;
c) yield and tensile strengths of profiled steel deckings as well as their cross-sectional shapes and dimensions, if present;
d) dimensions of longitudinal stiffeners in the troughs of the deckings, if present;
e) number of headed studs per trough as well as their positions and spacings;
f) spanning direction of the decking, if present;
g) welding quality of headed studs, and dimensions of welding collars at stud roots;
h) sizes and arrangement of steel reinforcements within the concrete slab in the vicinity of the studs;
i) orientation of steel-concrete interfaces during concreting (for preparation of test specimens);
j) friction along the steel-concrete interfaces, and
k) tilting and initial bedding of the test specimen in a push-out test.

Owing to a large number of factors which may affect deformation characteristics of the stud shear connections, significant variations in the test results obtained from push-out tests are often encountered. Hence, design rules on the shear resistances of these stud shear connections have been developed according to test results of a large number of push-out tests covering a wide range of material specifications and geometrical configurations, in particular for stud shear connections with composite slabs using profiled steel deckings.

1.1 Design Rules for Shear Resistances

a) Stud shear connections under shear forces

According to the literature, a number of design methods for shear resistances of the stud shear connections with solid concrete slabs adopted in many modern design codes are based on the research work carried out by Ollgaard *et al*. (1971):

$$Q_m = \min\left(c_1 d^2 \sqrt{f_{cy} E_{cm}}, c_2 A_{sc} f_u\right) \quad (1)$$

where

- d is the diameter of the stud shank (in mm);
- A_{sc} is the cross-sectional area of the headed shear stud (in mm^2);
- f_u is the tensile strength of the shear stud (in N/mm^2);
- f_{cy} is the cylinder strength of the concrete (in N/mm^2); and
- E_{cm} is the mean elastic modulus of the concrete (in N/mm^2).

$c_1 = 0.29$; $c_2 = 0.80$ EN 1994 (2a)

$c_1 = 0.39$; $c_2 = 1.00$ AISC (2b)

It should be noted that the first part of Eqn. (1) relates to conical concrete failure while the second part of Eqn. (1) relates to fracture of a stud shank at its root under shear force. It has been adopted in many design codes, such as EN 1994 and AISC, each with different values of c_1 and c_2 as given in Eqn. (2). Both $\boldsymbol{c_1}$ and $\boldsymbol{c_2}$ are calibration factors, and their values are based on various reliability analyses of test results reported in the literature. In EN 1994-1-1, c_1 and c_2 are adopted to be 0.29 and 0.80 respectively so that the shear resistance Q_m corresponds to a characteristic shear resistance with a lower 5% fractile among all the relevant test results. However, $\boldsymbol{c_1}$ and $\boldsymbol{c_2}$ are adopted to be 0.39 and 1.00 respectively in AISC instead in order to give an average value of the shear resistance Q_m , i.e. a characteristic resistance with 50% fractile among all the relevant test results.

Pallares and Hajjar (2010) reviewed 391 push-out tests with solid concrete slabs. After data analysis, four formulas are proposed to predict the shear resistance of stud shear connections under conical concrete failure while only one formula, i.e. $0.65A_{sc}f_u$, is provided for fracture of stud shank.

In addition, the following design expression is also available in the literature to evaluate the shear resistance of stud shear connections with solid concrete slabs:

$$Q_{m,Oehlers} = 5.0\left({f_{cu}}/{f_u} \right)^{0.35} \left({E_{cm}}/{E_s} \right)^{0.40} A_{sc} f_u \quad (3)$$

Eqn. (3) is based on an analysis on 110 push-out tests reported by Oehlers et al. (1987), and this is the basis of tabulated shear resistances of stud shear connections given in BS5950-3 (BSI, 1990). Compared with Eqn. (1), this expression reflects the effects of material properties of both the concrete and the stud steel in a rational manner although it is not able to identify explicitly the critical failure modes in the stud shear connections with different material specifications and geometrical configurations.

b) Shape factor for shear connections with composite slabs

According to EN 1994-1-1 (BSI, 2004), a shape factor, k_d , should be applied to the shear resistance of the stud shear connection obtained from Eqn. (1) when a composite slab is present instead of a solid concrete slab. Such a reduction is found to be necessary to allow for the presence of the profiled steel decking, or more accurately, to allow for a reduced amount of concrete surrounding headed stud(s) in a decking trough. The shape factor, k_d , is given by:

$$k_d = \frac{0.7}{\sqrt{n_r}} \frac{b_o}{h_p} \left(\frac{h}{h_p} - 1 \right) \quad (5)$$

where

n_r	is the number of headed stud(s) per trough;
h	is the height of the headed stud respectively; and
h_p and b_o	are the depth and the average width of the decking trough respectively.

However, there is little information on the limits of these dimensions as well as the geometrical configurations of the profiled steel deckings for which Eqn.(5) is applicable.

For many modern profiled steel deckings which are designed to span in the range of 3.6 to 4.5 m, there is often a longitudinal central stiffener, with a typical height of 5 to 8 mm, present at the bottom of each of the decking troughs. These stiffeners are provided intentionally to increase the hogging moment resistances of the decking under compression over internal supports. This requires the headed studs to be located off-center. However, no specific guidance on the design using off-centre headed studs is given in EN 1994-1-1 although some detailing rules in BS 5950-3 may be adopted. As shown in Fig. 2, an important dimension, e, is established as the mid-height distance from the centreline of the headed stud to the web of the decking trough. Hence, for any headed stud installed in either the "favourable" or the "unfavourable" position of the decking trough, the value of k_d given by Eqn.(2) above should be determined with b_o being taken as 2 e according to BS 5950-3.

c) Stud shear connections under combined shear and tension forces

According to Clause 6.6.3.2 of EN 1994-1-1, the effect of tension forces on stud shear connections with solid concrete slabs may be neglected when determining their shear resistances if the tension forces are smaller than 10% of the shear forces. It should be noted that a tension force is always induced in the shank of the headed stud in the shear connection even in a standard push-out test owing to dowel action. Though the magnitude of the tension force is somehow difficult to be quantified, such an effect has been implicitly incorporated in the design.

According to the literature, the shear resistance of a stud shear connection ~~with a solid concrete slab~~ under a large tension force is given by:

$$(Q_n / Q_m)^p + (T_n / T_m)^p = 1 \qquad (4)$$

where

Q_m	is the shear resistance of the shear connection under shear force only;
T_m	is the pull-out resistance of the shear connection under tension force only;
Q_n and T_n	are the co-existing shear and tension forces acting at the connection at failure; and
p	is a parameter with different values adopted in different design codes.

In Clause 5.3.3.6 of BS 5400-5 (BSI, 2005), Eqn. (4) is adopted, and the value of p is taken to be 2.0. As Q_n is determined by means of the von Mises criterion for stud steel yielding under combined shear and tension forces, T_m is taken as 1.73 Q_m for simplicity. However, this formula should be used with caution as tensile failure of the shear connection is assumed to take place only in the stud shank, and any failure in the concrete has not been incorporated.

In ACI 318 (2008), Eqn. (4) is also adopted, but the value of p is taken to be 5/3 according to McMackin et al. (1973). In general, T_m is conservatively assumed to be equal to 0.85 Q_m (Johnson, 1994). Hence, whenever the applied tension force, T_n , in the stud shear connection is considered to be large, the shear resistance of the shear connection should be reduced according to Eqn.(4).

In the recent years, a number of experimental investigations (Sarri et al.; 2004; Wu, 2006; ~~Pallares & Hajjar, 2010;~~ Lin et al. 2014) have been reported in which constant tension forces were first applied to the stud shear connections, and shear forces were then applied steadily until failure. ~~A s~~Series ~~large amount of~~ of test data have been collected, and different versions of Eqn.(4) with both linear and non-linear interaction curves have been proposed (McMackin et al. 1973~~Lin et la., 2014~~; Mirza & Uy, 2010) with different parameters for practical design of stud shear connections with solid concrete slabs and composite slabs. Two interaction curves are plotted in Figure 9 for shear connection under combined shear and tension forces. However, few test data on stud shear connections with composite slabs are available. It should be noted that the parameter "p" in Eqn.(4) was recommended to be 5/3 for solid concrete slabs and 3/2 for composite slabs.

1.2 Recent Concern on Structural Behaviour of Stud Shear Connections

While it is well known that when a stud shear connection slips, its shear resistance is mobilized under development of a dowel action in the headed stud, and hence, a tension force is developed in the shank of the headed stud. This tension force is difficult to be measured accurately in experiments, but its effect on the shear resistance of the stud shear connection has been accounted for implicitly during formulation of the design rules according to the test results of standard push-out tests.

However, according to the numerical investigations into structural behaviour of stud shear connections in composite beams with large web openings reported by Wang and Chung (2008) and Lawson et al. (2013), large tension forces are found in the shanks of the headed studs located in the proximity of the web openings. The maximum tension forces in the shanks of those critical studs are found to be equal to about 30% of their basic shear resistances. Hence, it is highly desirable to develop an effective analysis and design method to assess structural adequacy of these stud shear connections under combined shear and tension forces.

In order to examine structural behaviour of stud shear connections under combined shear and tension forces, a new type of push-out test is proposed (Shen, 2013), and it is referred as a modified push-out test as shown in Fig. 3. In general, the modified push-out test follows the established standard push-out test recommended in EN 1994-1-1 except the concrete-steel interface is inclined to an angle of about 15^o from the vertical. Under an applied force, P , a significant tension force, i.e. P x sin 15^o or 0.267 P, is readily induced to the shear connections. Hence, the proposed test set-up provides a simple and steady application of proportional shear and tension forces onto the test specimens.

It is worthwhile to note that a number of load deformation curves of full-scale long span beam tests were compared with those of the companion push-out tests by Ranzi et al. (2006) and Hicks (2007). A significant discrepancy in the deformation characteristics of these shear connections in push-out tests is found, when compared with those in the beam tests. Such discrepancies are believed to be even more complicated in composite beams with profiled steel deckings of different geometrical configurations as well as with headed studs installed in different positions of the decking troughs. Hence, it is highly desirable to qualify and quantify deformation characteristics of the stud shear connections through a systematic experimental and numerical investigation.

2. OBJECTIVES AND SCOPE OF WORK

In order to examine structural behaviour of stud shear connections under combined shear and tension forces, a research and development programme was undertaken, and both experimental and numerical investigations have been conducted. The present paper describes details of push-out tests as well as key test results and data analyses.

The scope of work of the experimental investigation includes:

a) Push-out tests

A total of six series of push-out tests with a total of 22 specimens are conducted. These include 11 standard push-out tests where the shear connections are under shear forces, and 11 modified push-out tests where the shear connections are under combined shear and tension forces.

It should be noted that the standard push-out tests are conducted to provide reference test data on the stud shear connections with both solid and composite slabs under shear forces, and they allow direct and effective comparison with those test data of the stud shear connections under combined shear and tension forces. Complementary materials tests have also been carried out to provide basic data on various mechanical properties of concrete, headed studs, profiled steel deckings and steel reinforcements.

b) Comparison with measured and design shear resistances and design parameters

In each test series with a specific geometrical configuration and loading condition, the measured load-slippage curve of the test specimen with the minimum shear resistance is selected to be normalized according to measured material strengths to provide a representative load-deformation characteristic of the shear connection. Hence, direct comparison on the shear resistances of various test series allows determination of the values of i) the shape factor, k_d , for the cases of headed studs installed in either "favourable" or "unfavourable" positions of decking troughs, and ii) the reduction factor, k_t , for the presence of high tension forces in the shear connections. Comparison of these values with design values according to codified design rules is also presented to demonstrate adequacy of these design rules.

The areas of interest of the experimental investigation include:

i) structural behaviour of shear connections under shear forces, and those of shear connections under combined shear and tension forces;
ii) typical failure modes of the shear connections;
iii) deformation characteristics of the shear connections with composite slabs having headed studs installed in “favourable” and “unfavourable” positions of decking troughs; and
iv) shear resistances of the shear connections with solid concrete slabs and composite slabs having different geometrical configurations and loading conditions, and hence, the values of both k_d and k_t .

These data will also be adopted in calibration of advanced finite element models during subsequent numerical investigations. These models will then be used to perform parametric studies on stud shear connections with a wide range of geometrical configurations and loading conditions to provide data for formulation of design rules. Details of calibration of these finite element models and subsequent parametric studies will be reported separately.

3. EXPERIMENTAL INVESTIGATION

In order to investigate structural behaviour of stud shear connections with both solid and composite slabs under combined shear and tension forces, six series of standard push-out tests with a total of 22 specimens are conducted. These include 11 standard push-out tests where the shear connections are under shear forces, and 11 modified push-out tests where the shear connections are under combined shear and tension forces. It should be noted that the testing method of the standard push-out tests recommended in EN 1994-1-1 is adopted in the present investigation.

3.1 Test programme

Table 1 summarizes the test programme of the experimental investigation. As shown in Fig. 3, all of the tests are grouped into six test series according to the following designation:

First label:	S or M		
	where	S	denotes a standard push-out test
		M	denotes a modified push-out test
Second label:	S or C		
	where	S	denote a solid slab
		C	denotes a composite slab
Third label:	F or U	(for stud shear connections with composite slabs only)	
	where	F	denotes all headed studs installed in the “favourable” position of the decking trough

U denotes all headed studs installed in the "unfavourable" position of the decking trough

Fourth label: r denotes the presence of a longitudinal central stiffener with a height of 10 mm at the bottom of the decking trough of profiled steel deckings in composite slabs

Hence, in Test Series SS, all the stud shear connections with solid slabs are tested under standard push-out tests while in Test Series MCFr, all the stud shear connections with composite slabs are tested under modified push-out tests having all the headed stubs installed in the favourable positions of the decking troughs of the profiled steel deckings.
In all test specimens, the concrete slabs are 600 mm wide and 125 mm thick. Headed studs with a diameter of 19 mm and a height of 100 mm are used, and the tensile strength of the stud steel materials is 500 N/mm^2. For those test specimens with composite slabs, the profiled steel deckings is 1.0 mm thick, and the yield strength is 280 N/mm^2.

It should be noted that:

a) *Test Series SS with solid slabs*
In this series of standard push-out tests on stud shear connections with solid concrete slabs, four pairs of headed studs are installed in each test specimen, and they are fully embedded into the concrete. The shear connections are primarily under shear forces. Hence, both the measured load-slippage curves and the measured shear resistances of the test specimens are readily taken as the basic data of the shear connections under shear forces for subsequent comparison.

b) *Test Series SCFr and SCUr with composite slabs*
In these two test series of standard push-out tests on shear connections with composite slabs, four pairs of headed shear studs are installed in each test specimen, and they are installed in either the "favourable" and the "unfavourable" positions in the decking troughs respectively, owing to presence of longitudinal central stiffeners. The stud shear connections are primarily under shear forces. Hence, the differences in the measured load-slippage curves as well as the measured shear resistances of the shear connections, when compared with those obtained in Test Series SS, will provide a direct measure on structural implication of the presence of deckings, or more specifically, the absence of concrete at regular intervals within the trough levels of the deckings in the composite slabs.

c) *Test Series MS with solid slabs*
In this series of modified push-out tests on stud shear connections with solid concrete slabs, four pairs of headed studs are installed in each test specimen, and they are fully embedded

into the concrete. The shear connections are under co-existing shear and tension forces. Hence, both the measured load-slippage curves and the measured shear resistances of the test specimens are readily adopted as reference data of the shear connections under combined shear and tension forces for subsequent comparison.

d) *Test Series MCFr and MCUr with composite slabs*

In these two test series of modified push-out tests on shear connections with composite slabs, the test specimens are similar to those in Test Series SCFr and SCUr, except that all the shear connections are under co-existing shear and tension forces. Hence, both the measured load-slippage curves and the measured shear resistances of the shear connections of these test series will be compared directly with those obtained from other test series.

3.2 Preparation of test specimens

Test specimens of the push-out tests are prepared with the following two different procedures:

- Procedure M1

  In this preparation procedure, all the headed studs are welded onto both flanges of H-sections. Then, solid concrete slabs are cast horizontally on one side of the H-sections with timber formwork. After the concrete has gained sufficient strength, the H-sections are then turned up-side down to cast the other solid concrete slabs with timber formwork.

  Although this procedure is simple, different batches of concrete are used, and hence, the ages of the concrete slabs at both sides of the H-sections differ by 3 to 7 days. Hence, the concrete strengths of these two slabs are expected to have some difference. Despite both the dimensions and the alignment of the two solid concrete slabs have been carefully controlled to facilitate subsequent load application on the test specimens, additional control is needed to achieve a high level of flatness at the bottom of the test specimens for effective load application. Only the test specimens of the first two series, namely Series SS and MS, are prepared using this procedure.

- Procedure M2

  In order to improve quality of the test specimens, a different preparation procedure is adopted in which H-sections are first split along their longitudinal axes to form two T-sections. Then, all the headed studs are welded onto the top surfaces of the flanges of the T-sections. Concreting of the slabs of both T-sections are cast horizontally with timber formwork at the same time, and hence, the same batch of concrete is used. After the concrete has gained sufficient strength, the webs of the two T-sections are bolted up with cover plates to form the test specimens. As the two slabs are aligned carefully during bolting up of the webs of the T-sections, the flatness of the test specimens at their bases is readily improved. This procedure is adopted for the test specimens of all the other test series.

3.3 Test set-up and instrumentation

Fig. 4 illustrates typical test set-up of the push-out tests. The compression machine has a loading capacity of 1,000 tons, and the loading is applied through displacement control. In general, a total of 8 transducers are employed to measure relative slippages of the connections at the positions of the first as well as the second rows of headed studs. Another two transducers are also employed in each test to monitor any lateral movement of the test specimens. It should be noted that in each test, both the applied force and the slippages of the stud shear connection are measured and recorded continuously during the entire deformation range.

3.4 Loading procedures

According to the test procedures given in Appendix B of EN 1994-1-1, a vertical load up to 40% of the expected failure load of the test specimen is applied in the initial stage of load application in each test. The vertical load is applied in a cyclic manner between 5% and 40% of the expected failure load 25 times at a loading rate of 240 kN/min, and the load is applied through load control. After the cyclic loadings, the loading is applied through displacement control. Hence, a displacement at the loaded point is imposed on the test specimen at a rate of 0.1 to 0.2 mm/min, so that failure will not occur in less than 15 minutes after commencement of the displacement control.

The test will be terminated whenever a failure occurs, or there is a 20% reduction in the applied force when compared with its maximum value. Alternatively, the connections are considered to have been failed when their slippage exceeds 20 mm.

4. Test Results

All the standard and the modified push-out tests together with various materials tests on the concrete, the headed shear studs, the profiled steel deckings and the steel reinforcements have been conducted successfully. Test results such as the failure modes of the test specimens, their load-slippage curves, and their maximum applied loads are presented in the following sections. It is necessary to have the measured shear resistances of the shear connections normalized according to measured material strengths in order to allow direct comparison.

4.1 Failure modes

Generally, there were three distinctive failure modes observed among all the tests: i) stud shear fracture, ii) concrete conical failure, and iii) stud bending with concrete conical failure, as shown in Fig. 5. It should be noted that:

a) Stud fracture (SF)

This failure mode always occurs in those stud shear connections with solid concrete slabs, and it is also called "shank shearing" (Johnson & Yuan, 1998) as the headed studs are fractured in their shanks just above the weld collars. After failure, most of the parts of the headed studs remain embedded in the concrete slabs. The concrete in the vicinity of the stud shanks always undergoes significant local crushing while a separation between the stud shanks and the concrete is often observed at stud roots. Generally, the corresponding load-slippage curves often exhibit a high degree of ductility, and the slippage capacities in all these test specimens are larger than 6 mm.

b) Concrete conical failure (CCF)

For shear connections with composite slabs, this failure mode always occurs when headed studs are installed at the “favourable” positions of the decking troughs. A cone-shaped portion of the concrete is often carried away by the headed studs at failure, and the size of the concrete cone depends largely on the geometrical dimensions of the profiled steel decking. In the present study, the width and the height of the concrete cone are found typically to be 600 and 220 mm respectively. Moreover, a surface crack is always observed at the outer surfaces of the solid concrete slabs, and it is located at the mid-height of the concrete slab between the two rows of the headed studs.

c) Stud bending with concrete conical failure (SB/CCF)

For shear connections with composite slabs, this failure mode always occurs when headed studs are installed at the “unfavourable” positions of the decking troughs. Owing to significant double curvature bending of headed studs, large slippage in the shear connections is apparent together with gross distortion in the profiled steel deckings. Nevertheless, a cone-shaped portion of the concrete is often carried away by the headed studs at failure, similar to the failure mode of concrete conical failure. Moreover, a surface crack is always observed at the outer surfaces of the concrete slabs, and it is located at the mid-height of the concrete slab between the two rows of the headed studs.

4.2 Deformation characteristics

All the measured load-slippage curves of the standard push-out tests and the modified push-out tests are plotted in Fig. 6 and Fig. 7 respectively, and the same scales are adopted in all the graphs in order to illustrate different degrees of ductility exhibited among the test specimens. It should be noted that:

i) In Test Series SS, all test shear connections are stiff and strong, and their load-slippage curves exhibit a highly ductile behaviour. Similar deformation characteristics are also exhibited in the shear connections in Test Series MS. All of them are able to provide a high level of shear resistance at a slippage at 6 mm. Nevertheless, the curves drop suddenly at large slippages because of stud fracture.

ii) In both Test Series SCFr and MCFr, the shear resistances of the shear connections are significantly reduced, when compared with those of the shear connections in Test Series SS and MS. More importantly, the load-slippage curves exhibit a rather brittle behaviour that their shear resistances are readily mobilized at a slippage of merely 1 to 2 mm. Then, the curves drop sharply because of concrete conical failure with limited deformations in the headed studs.

iii) However, the deformation characteristics of the shear connections in Test Series SCUr and MCUr are very different to those of the shear connections in Test Series SCFr and MCFr. Although their shear resistances are similar, they exhibit a highly ductile behaviour through significant double curvature bending in the headed studs. It should be noted that these connections exhibit only a small reduction in their resistances at large slippages. Then, the curves drop eventually because of occurrence of brittle concrete conical failure.

Key test results of the push-out tests are summarized in Table 2. It should be noted that:

i) Both the shear resistance, Q_m , and the corresponding slippage, s_m , are the two most important results of the tests.
ii) The slippage capacity, s_u , is a measure of ductility of the shear connection, and it is taken as the slippage at which the applied shear force is decreased from its maximum value by 20% during unloading.
iii) The stiffness of the shear connection, K_s , is a linear elastic response of the shear connection under the applied shear force, and it is taken as the slope of the load-slippage curve during the 25^{th} loading cycle.
iv) The applied shear force at a slippage of 0.5 mm is denoted as Q_0 , and its magnitude is also a measure of the initial stiffness of the shear connection. As all the values of Q_o are larger than 0.5 x Q_m , all the shear connections are considered to perform satisfactorily.

In addition, according to the deformation characteristics of all the test series, the shear connections in Test Series SCFr are stronger than those in Test Series SCUr, but the deformation ductility is greatly reduced. Consequently, the installation position of those shear studs in the shear connections in Test Series SCFr is not necessarily “favourable” as the shear connections are only marginally stronger but highly brittle, when compared with that in the shear connections in Test Series SCUr. Similarly, the installation position of those shear studs in the shear connections in Test Series SCUr is not necessarily “unfavourable” as the shear connections are only marginally weaker but highly ductile, when compared with that in the shear connections in Test Series SCFr. Similar conclusions are readily drawn in comparing those shear connections between Test Series MCFr and MCUr.

5 Comparison between measured and design shear resistances and design parameters

As shown in Fig.~~ure~~ 8, there are significant variations in the deformation characteristics of those shear connections even with nominally identical material and geometrical specifications. This is mainly due to variations in the material strengths of both the concrete and the shear studs in the test specimens. In order to allow direct comparison on the load-slippage curves of the shear connections, it is essential to select a representative load-slippage curve in each test series, and then modify the shear resistance of the representative curve according to material strengths for direct comparison of the test results among all test series. Owing to the limited number of tests within each test series, it is decided to select the load-slippage curve with the lowest shear resistance to be the reference curve for normalization.

In each test series, both the material strengths of the concrete and the shear studs are larger than the respective design values of the materials: i) a compressive cube strength at 35 N/mm^2 for the concrete, and ii) a tensile strength at 500 N/mm^2 for the stud steel. Hence, it is essential to reduce the measured shear resistance of the shear connection according to the ratio of design strength to measured strength of either the concrete or the shear stud, depending on the relevant failure mode of the shear connections in the series. This process is referred as 'normalization' to design values, and Table 3 summarizes the normalized shear resistances of the shear connections.

As suggested by Ollgaard et la (1971), the following expression is adopted to describe the normalized load-slippage curves of the shear connections in various test series:

$$Q = Q_m \left(1 - e^{-\alpha s} \right)^{\beta} \quad (6)$$

where

Q	is the shear force of the shear connection;
Q_m	is the normalized shear resistance of the shear connection; and
α and β	are parameters selected to describe the load-slippage curve of the shear connection; and
s	is the slippage of the shear connection.

The values of both α and β for the shear connections in various test series are presented in Table 4. It should be noted that for each shear connection, Eqn. (1) is applicable only when the slippage, s, is smaller than or equal to s_m , i.e. the shear resistance of the shear connection increases non-linearly according to Eqn. (1), and the shear resistance Q_m will be readily mobilized when the slippage s reaches s_m . When the slippage exceeds s_m , a linear reduction to 80% of the shear resistance at a slippage of s_u is assumed conservatively. For any slippage which is larger than s_u , the shear connection is considered to have failed. The values of s_m and s_u are also presented in Table 4.

All the normalized load-slippage curves of various test series described in accordance with Eqn. (1) are plotted as the “curves in black” onto the same graphs in Fig. 8 for both shear connections under standard and modified push-out tests for direct comparison. In addition, all the load-slippage curves in various test series after normalization are also plotted onto the graphs as the “curves in grey” in Fig. 8 to demonstrate suitability of those “curves in black”, that is, being conservative and yet efficient.

5.1 Shear resistances of stud shear connections with solid concrete slabs

Table 5 presents the normalized value of the shear resistance of the shear connection with a solid concrete slab under shear force, $Q_{m,SS}$. The shear resistances according to Eqn. (1), (2a) & (2b), and (3) have also been evaluated, and they are also summarized in Table 5 for direct comparison. Hence, the normalized value of the shear resistance $Q_{m,SS}$ is found to be 121.2 kN while design values at 144.0, 115.2 and 118.7 kN according to AISC, EN 1994, and Oehlers & Johnson (1987), respectively are obtained. In general, the measured value agrees well with the design values, and EN 1994 gives the lowest value among all three. Hence, the test results of Test Series SS are considered to be in order, and the normalized value is readily adopted as a reference value for subsequent data analysis.

5.2 Shape factor for shear connections with composite slabs

The presence of profiled steel decking affects significantly structural behaviour of the stud shear connections with composite slabs. These stud shear connections are often unable to develop their full resistances compared with those solid concrete slabs. In general, a shape factor, k_d , is adopted in various design codes to allow for such an effect. Table 6 summarizes the normalized shear resistances of the shear connections with composite slabs under shear force, k_d $Q_{m,SS}$, i.e. in Test Series SS, SCFr and SCUr together with the corresponding design values to EN 1994. Comparison on the values of k_d obtained from tests and design rules to EN 1994 is also provided.

It should be noted that the shape factors, k_d , are found to be 0.49 and 0.41 for the cases of headed studs installed at the “favourable” and the “unfavourable” positions respectively according to test results. The corresponding values of the shape factor, k_d , according to EN 1994 are found to be 0.70 and 0.28 respectively. Hence, the design rules are shown to be unconservative for the case of “favourable” position, and yet very conservative for the case of “unfavourable” position. Such a large discrepancy in the value of k_d for both positons may be a result of the presence of a large longitudinal central stiffener (with a height, h_s = 10 mm) in a relatively narrow trough (with an average trough width, b_o = 110 mm) in the profiled steel decking.

5.3 Reduction factor for the presence of large tension forces

The presence of large tension forces significantly affects structural behaviour of stud shear connections in the modified push-out tests. These stud shear connections are often unable to develop their full resistances when compared with stud shear connections in the standard push-out tests.

In order to make a comparison with the test results and the interaction formulas proposed by other researchers, the interactive reduction factors are presented in Table 7. It should be noted that the pull-out resistance of shear connection, T_m , ~~is not obtained in this experimental work, it~~ is assumed to be 92kN according to McMackin et al (1973), and the applied tension force T_n is equal to 0.267 Q_n due to the inclined interface in the modified push-out tests. A comparison between the test results and the interaction formulas are presented in Figure 9. It is found that both the two interaction formulas are not able to predict the shear resistance of shear connections under ~~combined shear and~~ tension forces in this paper. This discrepancy is ~~y is~~ due to the material properties ~~of the concrete and steel~~, the geometry of the profiled steel decking, and the positions of stud shear connections.

Hence, a series of ~~The~~ reduction factors, k_t , presented in Table 8 are proposed in this paper to predict the ~~the~~ shear resistances of various stud shear connections ~~, Q_m ,~~ under a large tension force which is smaller than or equal to 0.267 of the applied shear force ~~, T_n , are presented in Table 7~~. It is found ~~should be noted~~ that ~~, provided T_n is smaller than or equal to 0.267 Q_m~~:

a) for shear connections with solid concrete slabs, the value of k_t should be taken as 0.84, and hence, the reduced shear resistance of the shear connection becomes k_t $Q_{m,\,SS}$, i.e. 0.84 $Q_{m,SS}$;

b) for shear connections with composite slabs, the values of k_t should be taken as 0.37 and 0.32 for the cases of headed studs installed at the “favourable” and the “unfavourable” positions respectively, when compared with the shear resistances of the shear connections with solid concrete slabs, i.e. 0.37 $Q_{m,SS}$ or 0.32 $Q_{m,SS}$;

c) alternatively, the value of k_t may be taken as 0.755 and 0.780 for headed studs being installed in the “favourable” and the “unfavourable” positions respectively, when compared with the shear resistances of the shear connections with composite slabs, k_d $Q_{m,SS}$, i.e. 0.755 k_d $Q_{m,SS}$ and 0.780 k_d $Q_{m,SS}$ respectively; and

d) as a whole, the value of k_t may be taken as 0.84 for shear connections with solid concrete slabs, and as 0.75 conservatively for shear connections with composite slabs for both the cases of headed studs installed at the “favourable” and “unfavourable” positions.

~~d)~~

~~e)~~

6 Conclusions

In order to investigate structural behaviour of stud shear connections with both solid and composite slabs under shear forces, and under combined shear and tension forces, a total of six test series with 22 test specimens are conducted. These include 11 standard push-out tests where the shear connections are under shear forces, and 11 modified push-out tests where the shear connections are under combined shear and tension forces. It is shown that the shear resistances of the stud shear connections with different geometrical configurations and loading conditions have been obtained successfully. The following key findings are established:

a) Among all the 22 push-out tests, three distinct failure modes have been identified:
   i) fracture of studs in shear after large slippages in Test Series SS and MS;
   ii) highly brittle concrete conical failure with small slippages in Test Series SCFr and MCFr; and
   iii) bending of studs with large slippages followed by a brittle concrete conical failure in Test Series SCUr and MCUr.

b) Based on the deformation characteristics of the shear connections in Test Series SCFr and MCFr, the installation position of those shear studs in the shear connections is not necessarily "favourable". Similarly, the installation position of those shear studs in the shear connections in Test Series SCUr and MCUr is neither necessarily "unfavourable".

c) All the measured shear resistances have been normalized according to measured materials strengths. In general, the normalized shear resistance of the shear connection with a solid concrete slab under a shear force, $Q_{m,SS}$, from Test Series SS agrees well with the design (back analysis) values of three different codified design rules. Hence, $Q_{m,SS}$ is readily adopted as a reference value for subsequent data analysis with Q_m of all the other shear connections to determine the values of i) shape factor k_d which allows for the presence of profiled steel decking in composite slabs, and ii) reduction factor k_t which allows for the presence of tension forces.

d) Based on the test results of all the stud shear connections with various geometrical configurations and loading condition covered in the present investigation, it is found that the reduced shear resistance of the shear connection in the presence of a tension force is given by 0.84 $Q_{m,SS}$ for shear connections with solid concrete slabs, and by 0.75 k_d $Q_{m,SS}$ for shear connections with composite slabs. Consequently, the values of k_t are 0.84 and 0.75 for shear connections with solid concrete slabs and with composite slabs respectively.

e) The design rules for k_d given in EN 1994-1-1 and BS5950-3 are shown to be unconservative for the case of headed studs installed in the "favourable" position of the decking troughs, and yet very conservative for the case of headed studs installed in the “unfavourable” position of the decking troughs. This may be a result of adopting a profiled steel decking in the present study with various geometrical dimensions outside the dimensional ranges adopted during development of the design rules.

These data will be adopted in subsequent numerical investigations for calibration of advanced finite element models. These models will then be used to perform parametric studies on stud shear connections with a wide range of geometrical configurations and loading conditions to provide additional data for formulation of design rules. The numerical investigation will be reported separately.

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EXPERIMENTAL INVESTIGATION INTO STUD SHEAR CONNECTIONS UNDER COMBINED SHEAR AND TENSION FORCES

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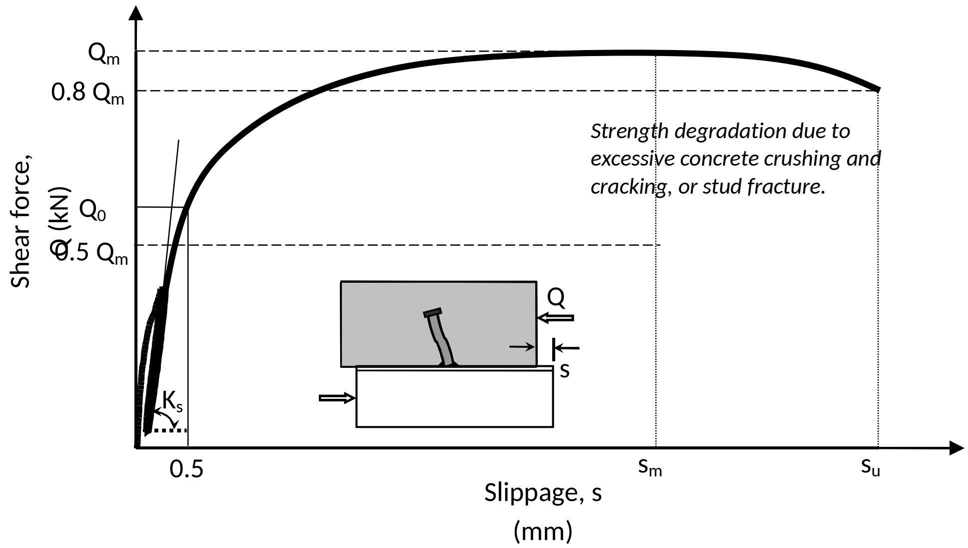


Figure 1: Typical load-slippage curve of a shear connection with a solid concrete slab

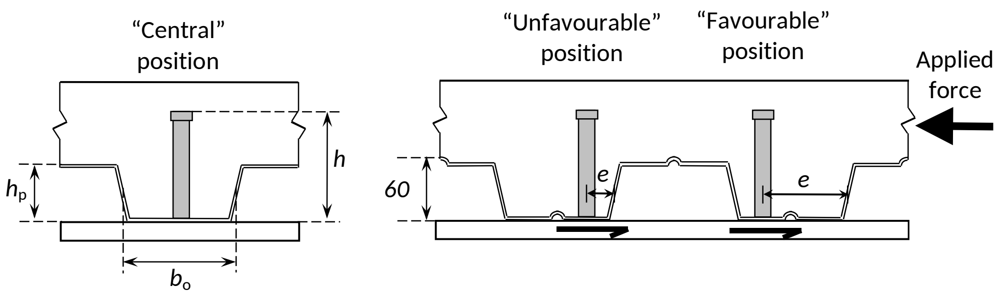


Figure 2: Typical geometrical configuration of a stud shear connection with a composite slab

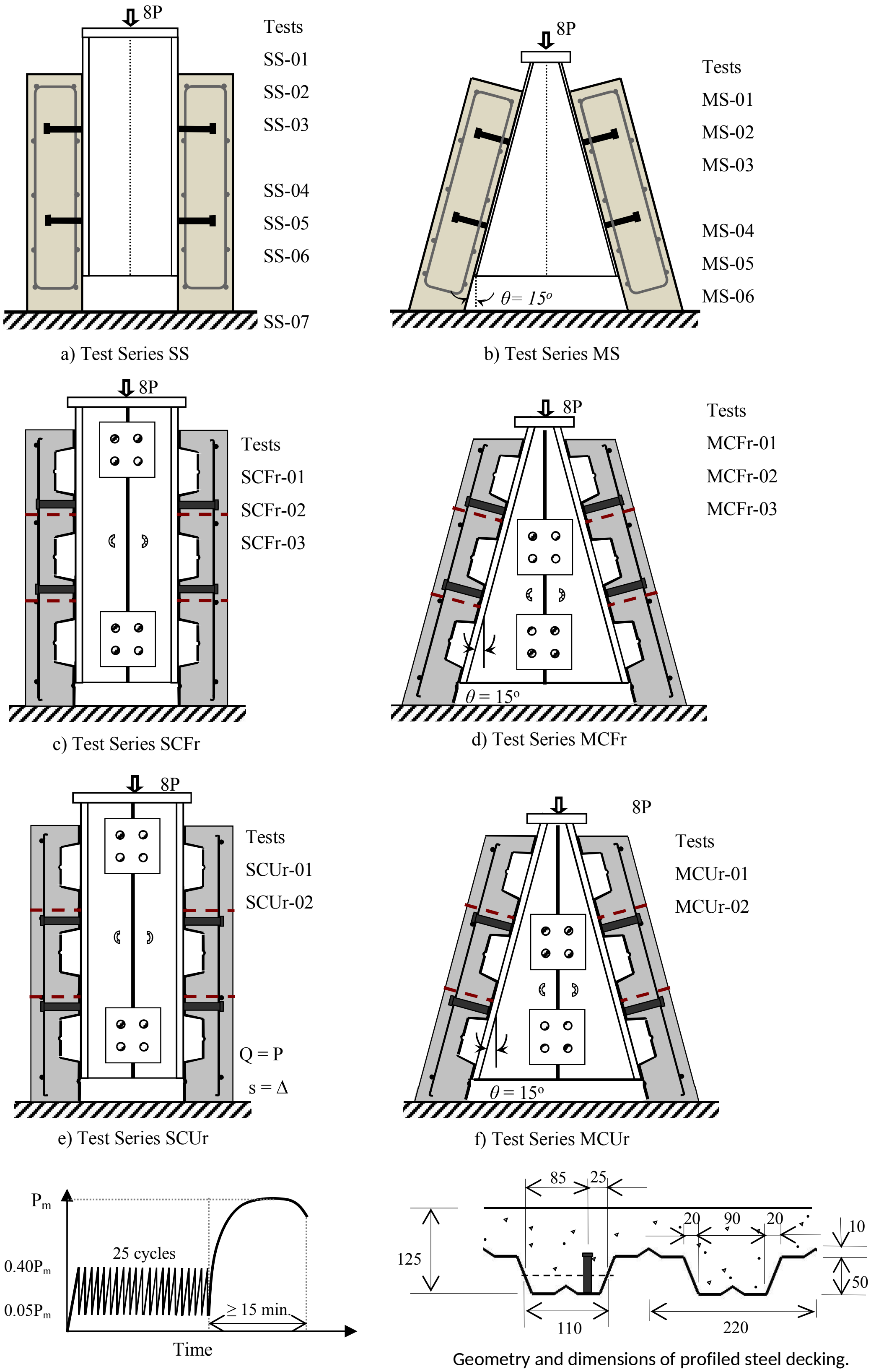
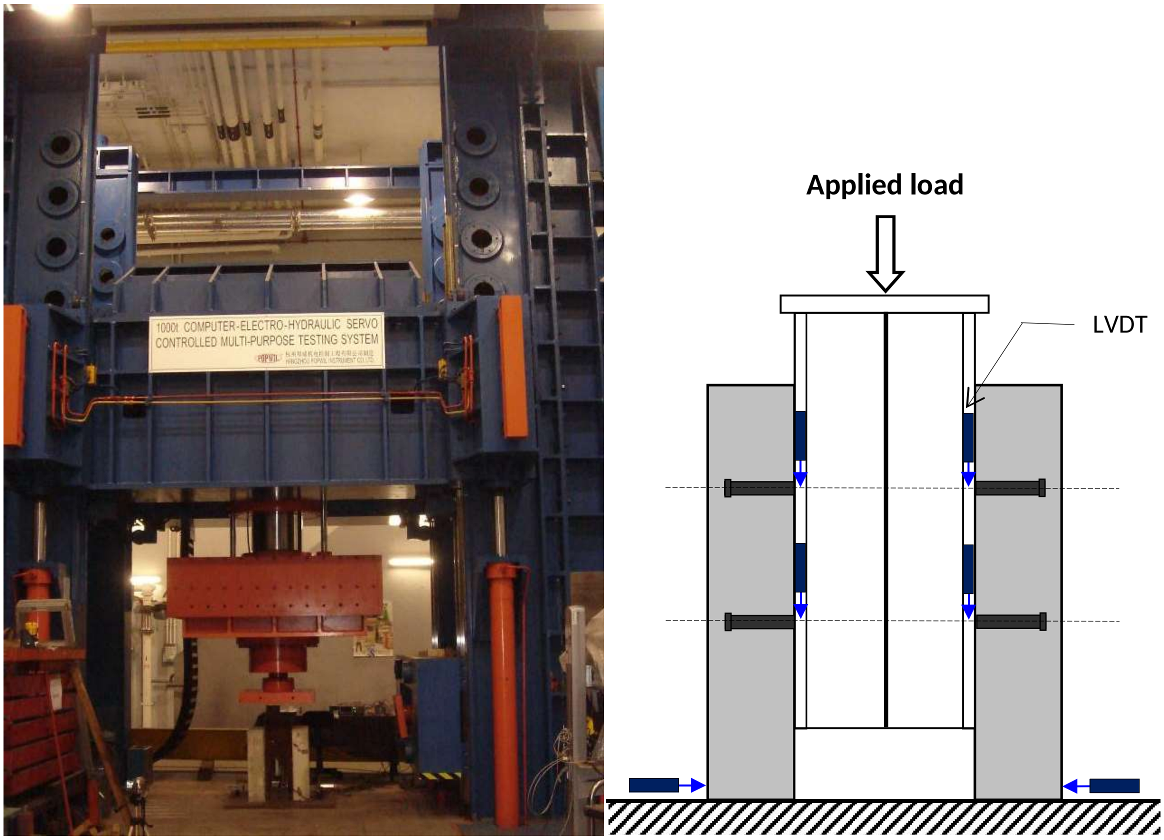
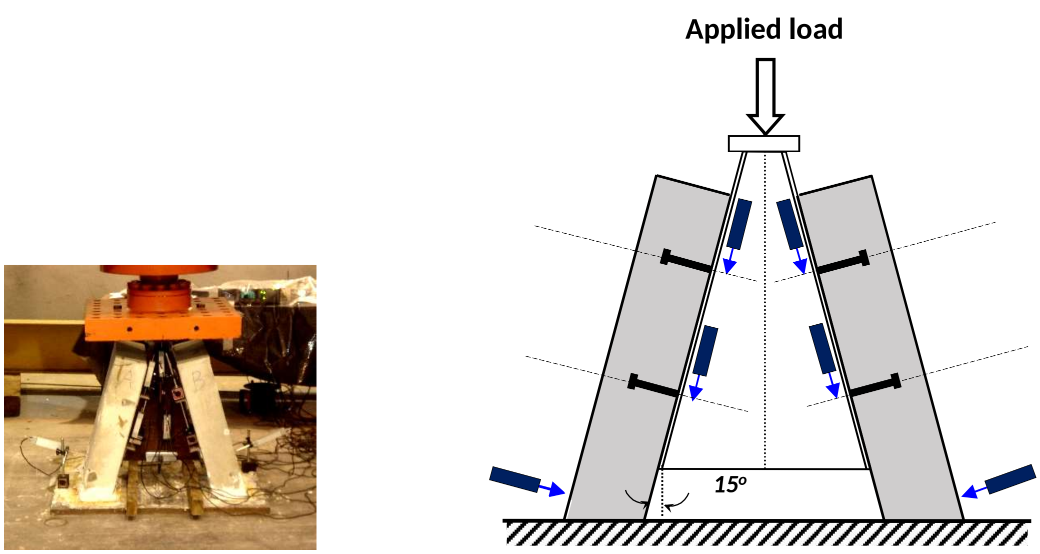


Figure 3: Push-out test specimens



i) Standard push-up tests



ii) Modified push-up tests

Figure 4：Typical test set-up and instrumentation of push-out tests

a) Stud fracture with concrete local crushing in Test Series SS

Failure mode: SF

b) Concrete conical failure in Test Series SCFr

Failure mode: CCF

c) Double curvature bending of headed studs with concrete conical failure in Test Series MCUr

Failure mode: SB/CCF

Figure 5: Typical failure modes of shear connections

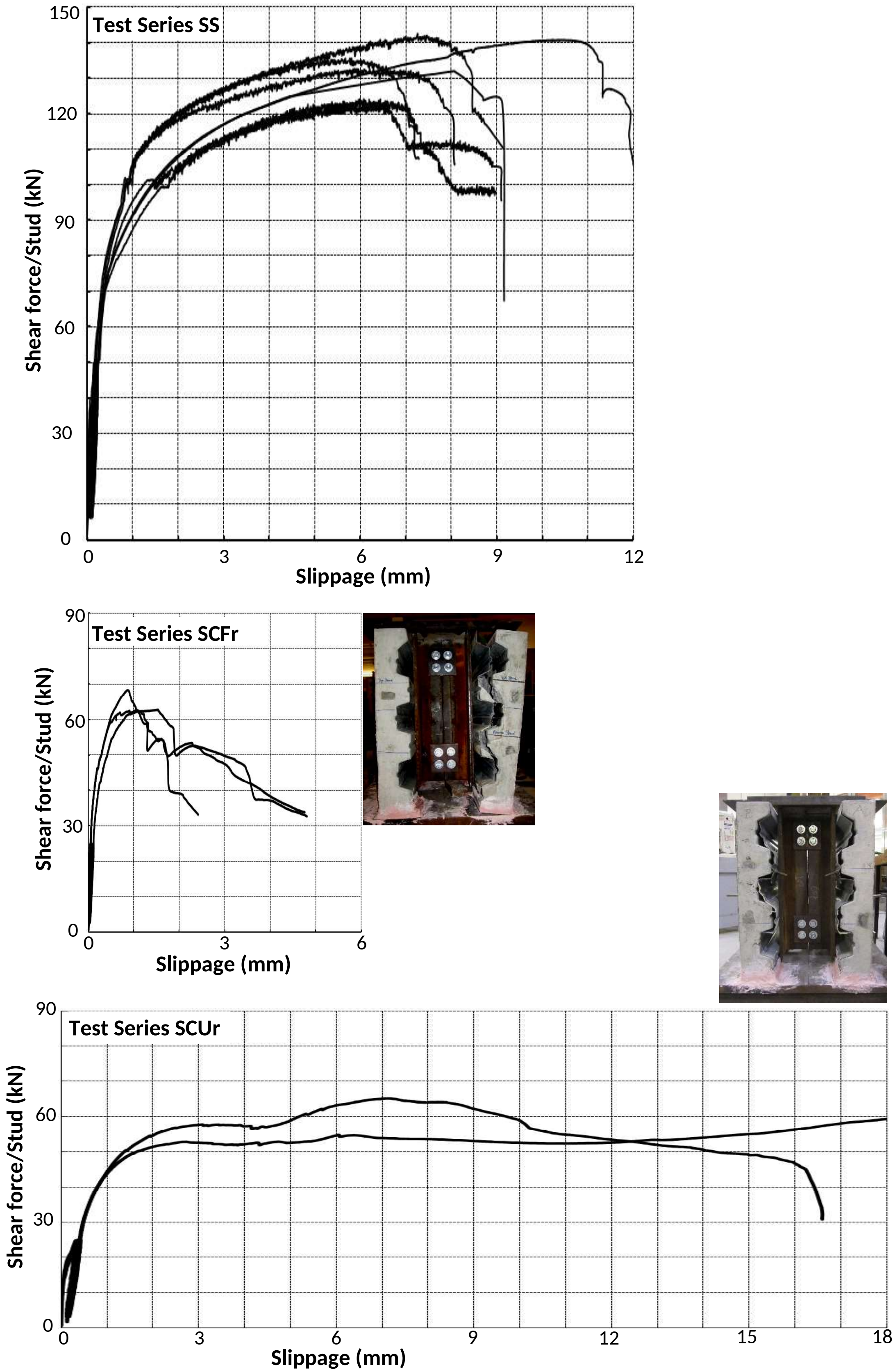


Figure 6: Measured load-slippage curves of standard push-out tests

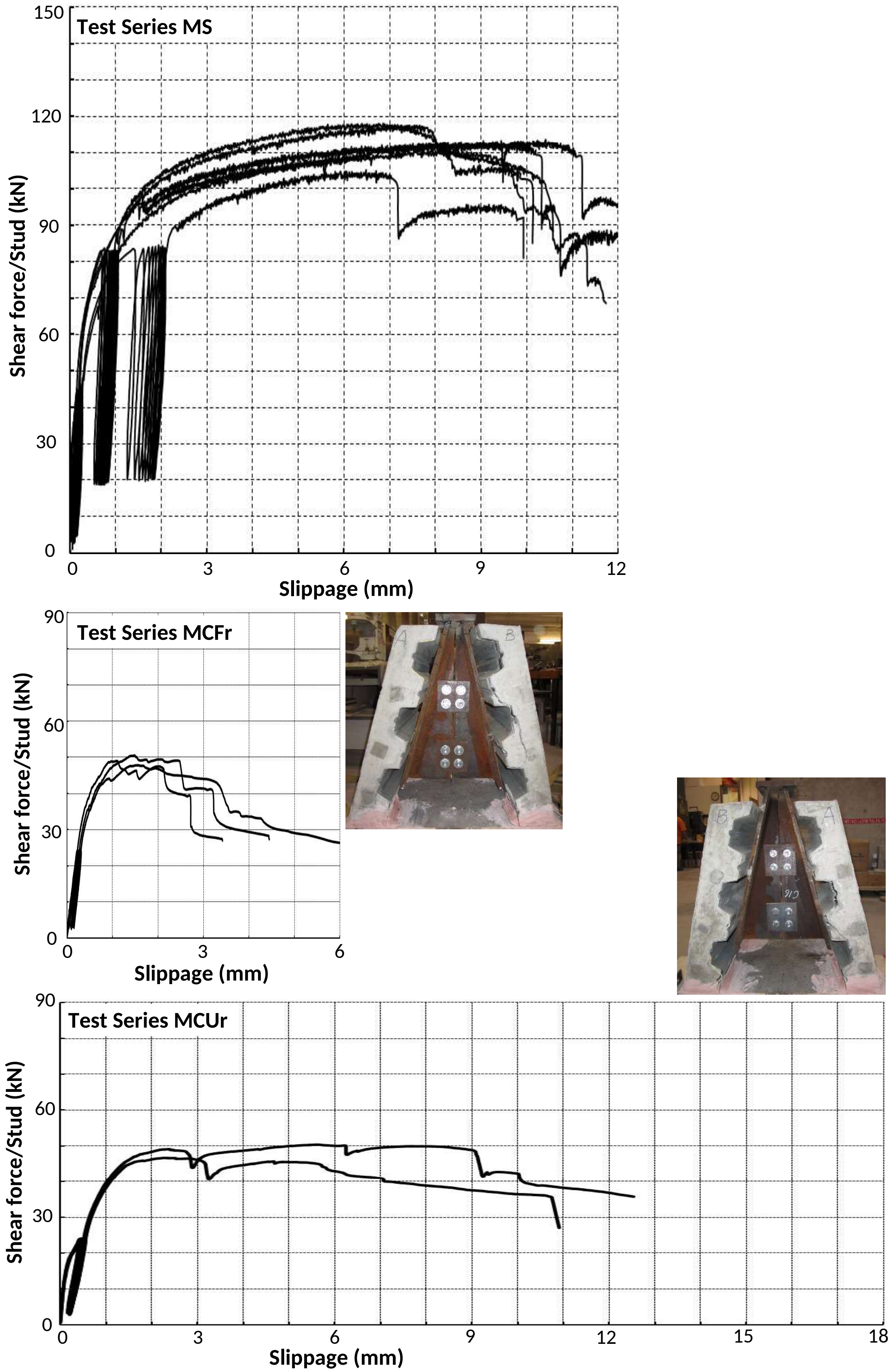


Figure 7: Measured load-slippage curves of modified push-out tests

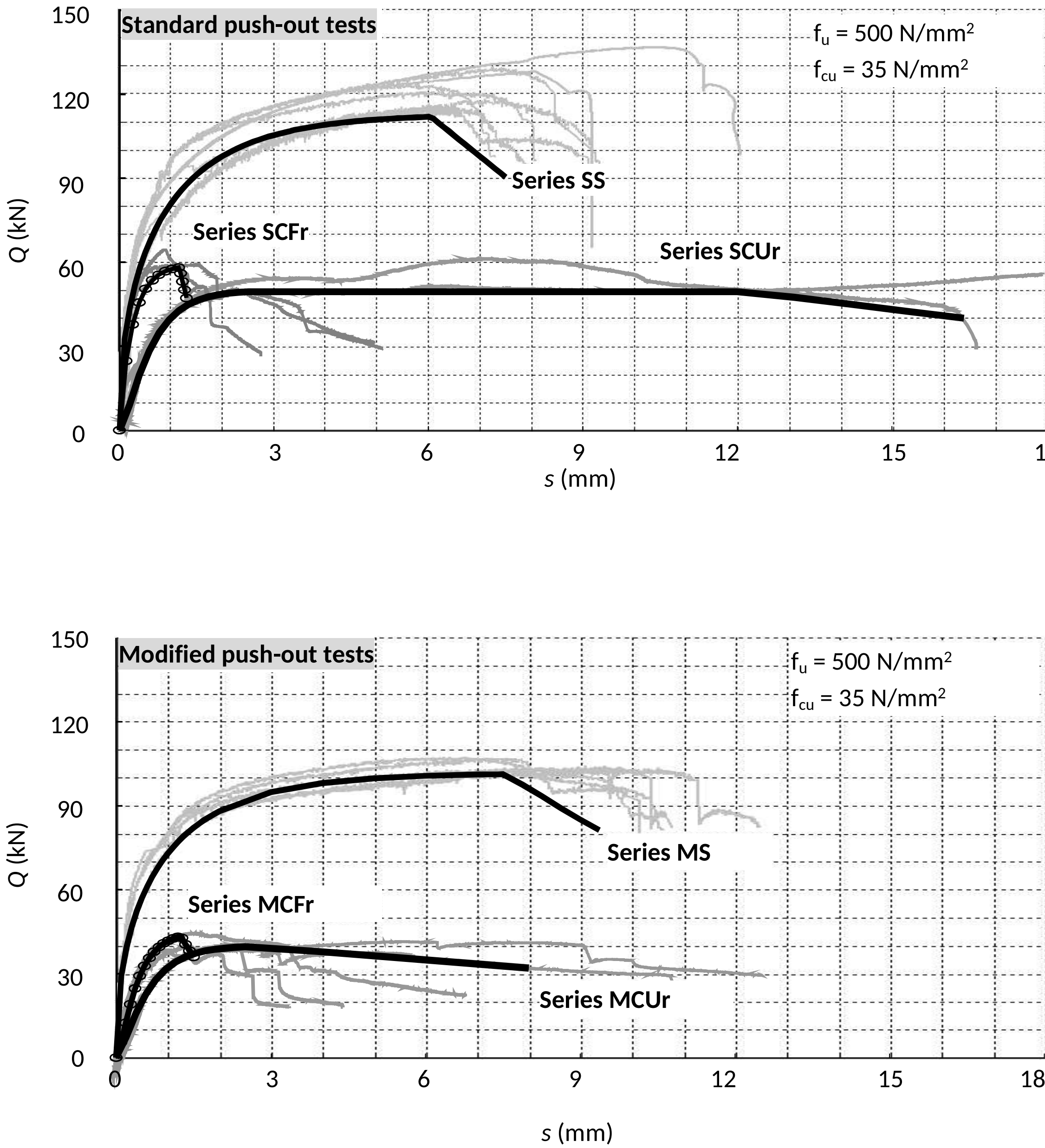


Figure 8: Normalized load-slippage curves

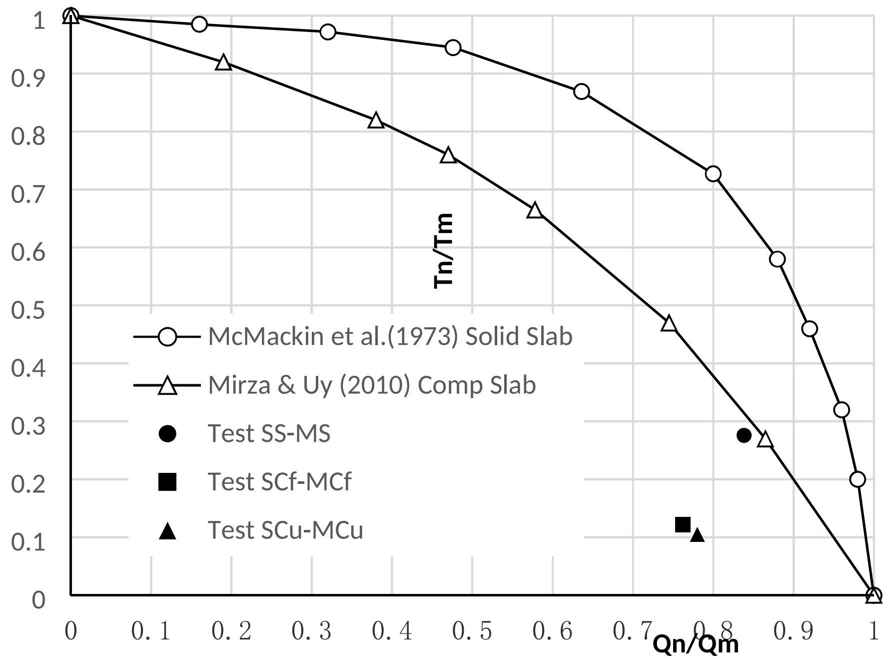


Figure 9: Interaction curves for shear resistance of shear connections under tension forces

EXPERIMENTAL INVESTIGATION INTO STUD SHEAR CONNECTIONS UNDER COMBINED SHEAR AND TENSION FORCES

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Table 1: Test programme

Test Series	**Test Designation**	**Specimen Type**	**Slab Type**	**Stud Position**	**Loading Conditions**
SS	SS-01 SS-04 SS-02 SS-05 SS-03 SS-06	Standard	Solid	-	Shear force
SCFr	SCFr-01 SCFr-02 SCFr-03	Standard	Composite	Favourable	Shear force
SCUr	SCUr-01 SCUr-02	Standard	Composite	Unfavourable	Shear force
MS	MS-01 MS-04 MS-02 MS-05 MS-03 MS-06	Modified	Solid	-	Combined shear and tension forces
MCFr	MCFr-01 MCFr-02 MCFr-03	Modified	Composite	Favourable	Combined shear and tension forces
MCUr	MCUr-01 MCUr-02	Modified	Composite	Unfavourable	Combined shear and tension forces

Table 2: Key test results of push-out tests

Test Designation	Q_m (kN)	s_m (mm)	s_u (mm)	Failure Mode	Minimum Q_m (kN)	K_s (kN/mm)	Q_o (kN)	Q_0 / Q_m
SS-01	125.7	7.0	9.7	SF	123.0	295	62.5	0.50
SS-02	124.2	6.1	9.0			306	70.0	0.56
SS-03	123.0	6.4	9.1			298	76.5	0.62
SS-04	142.6	7.2	8.6			279	83.5	0.59
SS-05	135.6	6.1	7.5			300	84.6	0.62
SS-06	132.7	6.3	8.5			302	83.8	0.63
SCFr-01	63.0	0.9	1.6	CCF	62.6	301 fs	59.8	0.95
SCFr-02	68.3	0.9	1.3			327	61.5	0.90
SCFr-03	62.7	1.5	3.0			286	54.5	0.87
SCUr-07	52.8	2.8	30.0	SB/CCF	53.0	86	30.1	0.57
SCUr-08	57.6	3.1	16.1			76	30.0	0.52
MS-01	110.7	7.0	9.7	SF	103.2	106	90.8	0.82
MS-02	113.3	6.1	9.0			120	81.6	0.72
MS-03	113.6	6.4	9.1			124	84.1	0.74
MS-04	104.9	7.2	8.6			160	83.9	0.80
MS-05	118.0	6.1	7.5			157	86.1	0.73
MS-06	117.6	6.3	8.5			199	94.1	0.80
MCFr-01	49.0	1.1	2.6	CCF	48.0	106	40.2	0.82
MCFr-02	50.0	1.5	3.3			120	36.0	0.72
MCFr-03	47.8	1.6	3.6			124	35.4	0.74
MCUr-07	48.9	2.4	10.2	SB/CCF	46.7	64	29.8	0.61
MCUr-08	46.6	2.5	9.1			58	23.8	0.51

Note: The value of Q_0 / Q_m should be larger than 0.5 for satisfactory performance of the shear connection.

Table 3: Normalized shear resistances of various shear connections

Test Series	Minimum Q_m (kN)	Failure mode	Normalization factor		Normalized Q_m (kN)
SS	123.0	SF	500 / 507.8	= 0.985	121.2
SCFr	62.6	CCF	35 / 37.2	= 0.941	58.9
SCUr	53.0	SB/CCF	35 / 37.4	= 0.936	49.6
MS	103.2	SF	500 / 507.8	= 0.985	101.6
MCFr	48.0	CCF	35 / 37.4	= 0.936	44.9
MCUr	46.7	SB/CCF	35 / 42.2	=0.830	38.7

Notes:

Reference material strengths:

a) Concrete cylinder compressive strength 35 N/mm^2

b) Stud tensile strength 500 N/mm^2

Table 4 Normalized load-slippage curves

Test Series	Q_m (kN)	α	β	s_m (mm)	s_u (mm)
SS	121.2	0.65	0.45	6.0	7.5
SCFr	58.9	3.50	0.85	1.2	1.5
SCUr	49.6	2.00	1.50	2.5	16.5
MS	101.6	0.65	0.45	7.5	9.0
MCFr	44.9	3.00	1.50	1.2	1.5
MCUr	38.7	2.00	1.50	2.5	8.0

Note:

$$Q = Q_m (1 - e^{-\alpha s})^{\beta}$$

Table 5: Comparison of shear resistances of shear connections with solid concrete slabs

Test Series	Measured resistance, Q_n (kN)	Design resistance, Q_n (kN)			
		AISC Eqn. (1) & (2a)	EN 1994 Eqn (1) & (2b)	Pallares and Hajjar (2010)	Eqn. (3)
SS	121.2	144.0	115.2	92.1	118.7

Notes:

a) All shear connections fail with stud fracture.

b) The following parameters are adopted:

d = 19 mm

f_{cy} = 27.9 N/mm^2

E_{cm} = 24.1 kN/m^2

f_u = 500 N/mm^2

E_s = 205 kN/mm^2

Table 6: Comparison of shear resistances of shear connections with composite slabs

Test Series	Shear resistance, Q_m (kN)		Reduction factor	
	Normalized value	EN 1994	Test result	EN 1994
SS	121.2	115.2	---	---
SCFr	58.9	80.6	0.49	0.70
SCUr	49.6	32.3	0.41	0.28

Table 7: Interactive reduction factors on shear resistances of shear connections

Test Series Standard - Modified	T_m (kN)	Q_m (kN)	T_n (kN)	T_n/T_m	Q_n (kN)	Q_n/Q_m
SS - MS	121.2	92	101.6	**0.84**	25	**0.28**
SCf - MCf	58.9	92	44.9	**0.76**	11	**0.12**
SCu - MCu	49.6	92	38.7	**0.78**	10	**0.11**

Table 8: Reduction factors on shear resistances of shear connections

a) Effect of profiled steel decking

Test Series	Q_m (kN)	Failure Mode	k_d	
			Test	Design
SS	121.2	SF	-	-
SCFr	58.9	CCF	0.49	0.70
SCUr	49.6	SB/CCF	0.41	0.28

b) Effect of high tension forces

Test Series	Q_m (kN)	Failure Mode	k_d x k_t
			Test
MS **SS**	101.6 121.2	SF SF	0.84
MCFr **SS**	44.9 121.2	CCF SF	0.37 or 0.49 × 0.755
MCUr **SS**	38.7 121.2	SB/CCF SF	0.32 or 0.41 × 0.780