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Push tests on demountable shear connectors in profiled composite slabs

*Jie Yang[1)], Dennis Lam[2)], Xianghe Dai[3)], Therese Sheehan[4)], Kan Zhou[5)], Yong C. Wang[6)] and Razvan M. Sencu[7)]

[1) - 5)] *School of Engineering, Faculty of Engineering and Informatics, University of Bradford, Bradford, UK*
[6), 7)] *School of Mechanical, Aerospace and Civil Engineering, University of Manchester, Manchester, UK*

[1)] *j.yang17@bradford.ac.uk*

ABSTRACT

This paper presents thirty-two push tests to investigate the behaviour of demountable shear connectors in profiled composite slabs. Demountable shear studs (19mm and 22mm in diameter) and Gr. 8.8 M20 bolts were used as the shear connectors. Important individual parameters such as concrete strength, embedment height of the shear connectors, reinforcement pattern and deck profile were investigated. In particular, separable slab segments with a cast in edge-trim of the standard push test specimen were fabricated, intended to facilitate the reuse of the composite slab and steel beam after the first cycle of use without the need of cutting along the centre-line of the beam. The contribution of edge trims to the behaviour of the shear connection is highlighted. The experimental results showed that the shear capacity and behaviour of the demountable connectors in separate slabs and continuous slab were similar and could both fulfil the 6mm minimum ductility requirement stated in Eurocode 4 for welded studs when proper embedment height of connector was ensured. The results on the shear resistance of these form of shear connectors were compared against existing design equations.

[1)] Post-Doctoral Researcher
[2)] Professor
[3)] Senior lecturer
[4)] Lecturer
[5)] Post-Doctoral Researcher
[6)] Professor
[7)] Post-Doctoral Researcher

1. INTRODUCTION

A large part of the carbon footprint associated to building constructions is embodied in the materials used for construction. Steel and concrete has been the most commonly used building materials in construction today. Reuse of steel and concrete that are energy-intensive is then identified as an effective method to reduce the energy and carbon impact of construction industry.

For steel framed structures using steel-concrete composite floor systems, which have been popular in the UK, there is a main technical obstacle to reuse the steel beams and composite slabs. The composite action between the steel beam and slab is conventionally established through shear connectors that are welded to the flange areas of the steel beam and embedded in concrete, which causes difficulty to detach the beam from the slab at the end of the life of the structure and results in steel recycling and concrete demolition or landfilling.

One practical and sustainable solution is to make the conventional welded shear studs demountable, to facilitate reuse of the steel beams and slabs in their following life cycles. The environmental impact that is associated to the structural systems can thus be significantly reduced by repeated use of the structural components. Behaviour of the demountable shear studs is then a major concern. Lam and Saveri (2012) and Dai, et al (2015) explored the behaviour of the standard 19mm headed shear studs with a 16mm threaded end; and with or without a collar step, a M16 Gr 8.8 nut is used to fasten the connector to the steel beam. A series of the push off tests have been carried out to assess its potential and suitability. Test results shown that these shear connectors can be easily demounted after test and process high ductility in comparison with the welded shear connectors but relatively lower initial stiffness. Numerical modelling results showed that the lower initial stiffness of the demountable shear connectors did not affect the initial stiffness of the composite beam analysed (Lam and Dai 2013).

The use of bolts as demountable shear connectors has been explored recently. Pavlović et al (2013) carried out push tests for prefabricated solid slabs using four Gr. 8.8 M16 bolts in each specimen with embedded nuts. It was found that bolted shear connectors with a single embedded nut achieved approximately 95% of the shear resistance of the welded headed studs shear connectors. Pathirana et al. (2012) and Mirza et al. (2010) used blind bolts as demountable studs. It was found that they behaved in a very similar way to welded headed studs in terms of stiffness and strength but it had a relatively brittle behaviour. Other types of configurations, e.g. high-strength friction-grip bolts in geopolymer concrete, epoxy resin injection bolts, friction bolts with cast in cylinders, and Hollo-Bolts, etc. are also explored by Ataei et al (2016), Kozma et al (2019) and Tan et al (2019).

The behaviour of a shear connector in profiled composite slabs is more complex than in a solid slab. It can be influenced by such as the direction of the trough of a profiled sheeting relative to the span of the beam, the breadth and depth of sheeting (Mirza and Uy 2010), the type, diameter and embedment height of the shear connectors, the number of connectors in one trough and the spacing, reinforcement patterns (Nellinger et al 2017) and whether or not the connectors are placed centrally within a trough, etc. When modern forms of profiled sheeting are used, the shear

resistance of the connections is usually reduced compared to those in solid slabs. Rehman et al (2016) conducted twelve full-scale push tests on demountable shear connectors in profiled slabs considering different concrete strengths, numbers of connectors per trough and different connector diameters. A combined Eurocode 3 and 4 methods was found to provide a safe prediction of shear resistance for specimens with single and pairs of demountable connectors per trough but one row of shear connectors was considered in the study.

Knowledge of the performance of welded stud shear connectors in both solid and profiled slabs has been well established, however the behaviour of bolted/demountable connectors remains unfamiliar to structural engineers especially those with the use of profiled sheeting, and are not explicitly covered by common codes of practices.

The main objective of this paper is to explore the shear behaviour of different forms of demountable shear connectors placed centrally in profiled composite slabs with rib running transversely to the span of the beam. Two rows of shear connectors were considered as specified in EN1994-1-1 (CEN 2004). Machined traditional headed studs (19mm and 22mm in diameter) and Gr. 8.8 M20 bolts were tested and compared. Important individual parameters such as concrete strength, embedment height of the shear connectors, reinforcement pattern and deck profile were varied. In particular, two separate slab segments per half of the standard push specimen were fabricated, intended to facilitate reuse of the composite floor and steel beam after the first service life with no cut effort made at the centre-line of the beam. The contribution of edge trims used as formwork to the shear behaviour of the connection is highlighted. The results on the shear resistance of the tested shear connectors are compared against existing design equations.

2. EXPERIMENTAL PROGRAMME

To investigate the shear capacity, stiffness and ductility of the demountable shear connectors, thirty-two push tests were conducted in the Heavy Structures Laboratory, University of Bradford. Three forms of shear connection were used, as shown in Fig.1, i.e. machined shear studs with collar, machined shear studs with embedded single nut and Gr. 8.8 bolts with embedded single nut. Modern profiled sheeting with nominal height of 60 mm (SMD TR60) and 80 mm (TATA CF80) were considered. Concrete strength ranged from 24.2 N/mm^2 to 55.3 N/mm^2 (average concrete strength tested from cubes). Reinforcement patterns and the embedment height of the shear connectors (100 mm and 120 mm) were also varied. Specimens' details, material properties, test setup, instrumentation and testing procedures are presented in the following sub-sections.

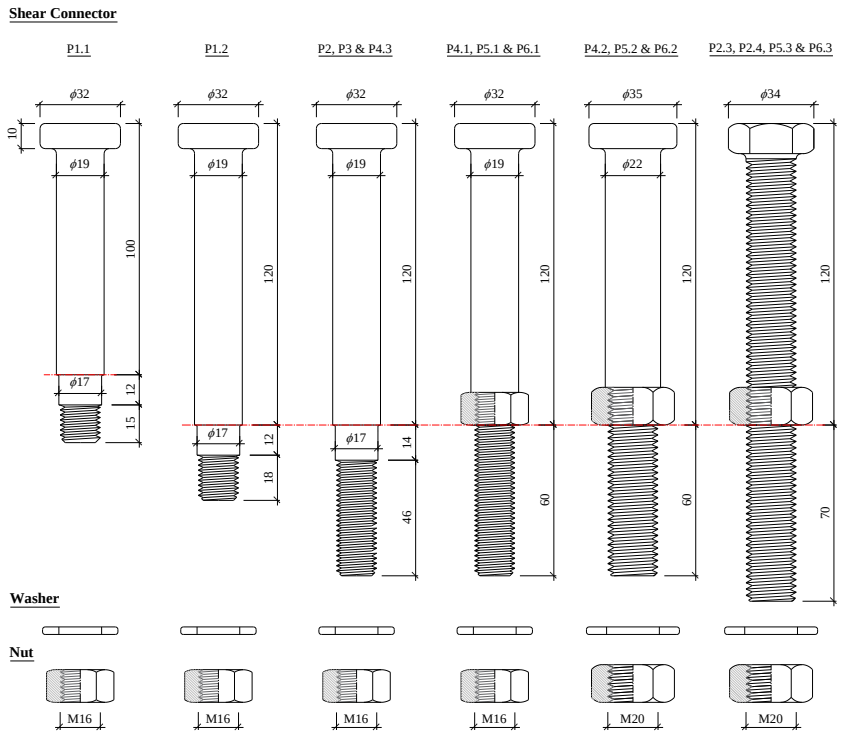

Fig. 1 Type and dimensions of shear connectors (machined headed studs and Gr. 8.8 bolt; unit: mm)

2.1 Specimen details and material properties

The push test specimens with profiled slabs designed in this paper was based on the standard specimen given in EN1994-1-1. Pair and two rows of shear connectors were considered per slab and overall eight connectors were used per specimen. The connectors were located 50 mm off the central line of the beam flange and at 100 mm transverse spacing. The beam section used was 254×254×73 UC and 900 mm in length. The depth and height of the slabs was 150 mm and 900 mm (designed based on the deck profiles and rows of shear connectors considered, respectively. The overall width of the specimens was 610 mm.

To facilitate disassembly and reuse of the steel beam and composite slabs, separable slab segments were investigated. Two fabrication techniques were employed: 1) using a steel plate placed in between the pair of shear connectors to separate the slabs in timber moulds during concreting and removing it after concrete has hardened; 2) using double edge trims (130mm or 150mm in height) embedded in the concrete at the central line of the pair connectors.

Rebar cages were adopted at the toe of all the push specimens to prevent premature concrete failure of the slabs. The push test specimens were prepared in eight batches (Series P1 to P6, P2.3 and P2.4) and were cast in the horizontal position and air-cured; a summary of the specimens' details and concrete compressive strength is illustrated in Table 1. A duplicate specimen was prepared for each configuration excluding the Series P1. Reinforcement patterns and geometry of typical specimens are shown in Figure 2.

Table 1 Summary of specimen details

		Shear Connector					**Concrete**	**Deck**	**Edge trim**	**Reinforcement**
		h_{sc}	d_s	d_c	n_r	f_u	f_{cu}			
Series	**No.**	mm	mm	mm	-	N/mm^2	N/mm^2			
P1.1	1	100	19	17	2	523.2	48.5	TR60	No	R_0: two layers
P1.2	1	120	19	17	2	523.2	48.5	TR60	No	R_0: two layers
P2.1	2	120	19	17	2	523.2	44.6	TR60	No	R_1: R_0+extra
P2.2	2	120	19	17	1	523.2	44.6	TR60	No	R_1: R_0+extra
P3.1	2	120	19	17	2	523.2	25.4	TR60	No	R_1: R_0+extra
P3.2	2	120	19	17	1	523.2	25.4	TR60	No	R_1: R_0+extra
P4.1	2	120	19	16	1	523.2	24.2	TR60	Yes	R_2: A193+Ubar
P4.2	2	120	22	20	1	521.9	24.2	TR60	Yes	R_2: A193+Ubar
P4.3	2	120	19	17	1	523.2	24.2	TR60	Yes	R_2: A193+Ubar
P5.1	2	120	19	16	1	523.2	44.3	TR60	Yes	R_3: A193 mesh
P5.2	2	120	22	20	1	521.9	44.3	TR60	Yes	R_3: A193 mesh
P5.3	2	120	20	20	1	804.8	44.3	TR60	Yes	R_3: A193 mesh
P6.1	2	120	19	16	1	523.2	55.3	TR60	Yes	R_2: A193+Ubar
P6.2	2	120	22	20	1	521.9	55.3	TR60	Yes	R_2: A193+Ubar
P6.3	2	120	20	20	1	804.8	55.3	TR60	Yes	R_2: A193+Ubar
*P2.3	2	120	20	20	1	804.8	47.6	CF80	Yes	R_2: A193+Ubar
*P2.4	2	120	20	20	1	804.8	45.3	CF80	Yes	R_3: A193 mesh

* Push tests reported by Dai et al (2018).

Symbols in Table 1:

R_0	Two layers of rebar reinforcement (φ10mm)
R_1	Two layers of rebar reinforcement plus extra reinforcement around the connectors (φ10mm)
R_2	One layer of A193 (φ7mm) reinforcement mesh plus one U-bar per connector (φ10*mm*)
R_3	One layer of A193 (φ7mm) reinforcement mesh
d_c	Mean diameter of the collar of a shear connector (in contact with the steel beam element; mm)
d_s	Mean diameter of the shank of a shear connector (embeded in concrete; mm)
f_{cu}	Mean value of the compressive strength of concrete (cubes; N/mm^2)
f_u	Mean value of the ultimate strength of a shear connector (N/mm^2)
h_{sc}	Overal nominal height of a shear connector embeded in concrete (mm)
n_r	Number of shear connectors per rib (e.g. n_r=1 for seperable slab segments)

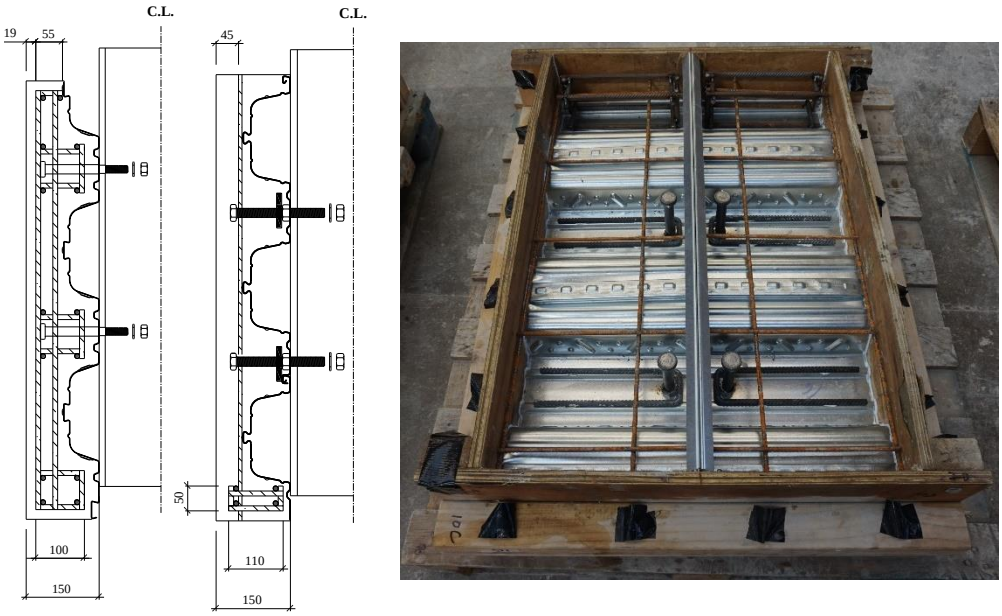

Fig. 2 Reinforcement patterns and edge trims in the timber moulds before concreting

2.2 Test setup and Instrumentation

Fig. 3 illustrates the test setup and instrumentation. A 100-tonne actuator was employed to exert the compressive load on the specimens.

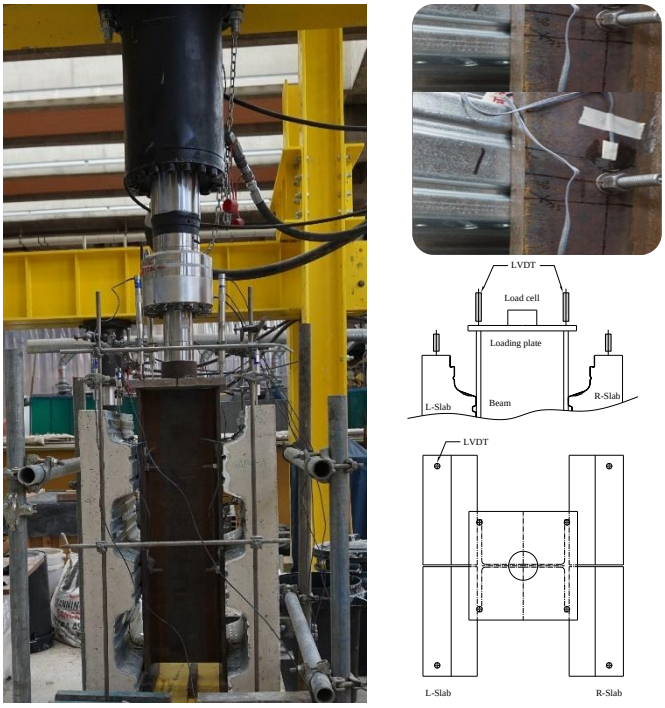

Fig. 3 Push test setup and instrumentation

A loading plate was placed on the top end of the beam section. Four of the eight LVDTs adopted were put on each corner of this plate to measure the movement of the beam during the experiments. The other four LVDTs were placed on the profiled slabs with two on each slab, to measure the displacement of the slabs. The relative slip between the slabs and the beam section was then obtained as the mean difference of this two set of LVDTs measurements. In total eight strain gauges were attached to the steel beam flanges 30mm above bolt hole centres, to monitor the load transfer/distribution among shear connectors during tests.

2.3 Testing procedure

During the experiments, compressive load at steel section was applied by the minimum interval of a certain load, e.g. 24 kN, or 0.5 mm of slip; at each interval, load increment was only applied after the load settled. Subsequent load increments were imposed such that failure did not occur in less than 15 minutes.

When duplicate specimens were prepared, the first specimen was tested to obtain the failure load and the second specimen was first applied in increments up to 40% of the failure load and then cycled between 5% and 40% of the failure load from 2 times up to 25 times. The tests were terminated after either concrete failure or shear failure of the connectors were observed and sufficient slip measurements were collected.

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

Experimental results on a total of thirty-two push tests were analysed and summarized in Table 2. The ultimate load P_{ult}, the load at 6mm slip P_{6mm}, the slip at the ultimate load δ_u, and modes of failure for each configuration are listed, where P_{ult}, P_{6mm} and δ_u are given as the mean value of duplicated specimens (for Series P2 to P6). Three modes of failure were observed from the push tests conducted, i.e. concrete cone failure, concrete crushing near the vicinity of the shear connectors and shearing or fracture of the connectors. In the cases when the δ_u was smaller than 6 mm, the ratio between the resistance at 6mm, P_{6mm}, and the ultimate resistance P_{ult} ranged from 0.90 to 0.95, which indicated a ductile manner of the behaviour of the shear connectors (excluding P1.1). The ranging values of the other results are summarized as follows.

Currently there is no design rules specified for demountable shear connectors. The methods available for welded headed studs in EN1994-1-1 (CEN 2004) and bolted connections in EN1993-1-1 (CEN 2005) are used herein to predict the shear resistance of the demountable connectors. The ratio between the tested values to the Eurocodes predictions are also given in Table 2. Equations in EN1994-1-1 are given as follows,

$$P_{Rd,S} = 0.8 f_u \pi {d_c}^2/4 \tag{1}$$

$$P_{Rd,C} = 0.29 \alpha {d_s}^2 \sqrt{f_{ck} E_{cm}} \tag{2}$$

where P_{Rd} is the characteristic resistance of a shear connector; $\alpha = 1$ (for $h_{sc}/d_s > 4$); f_{ck} is the characteristic cylinder strength of the concrete; E_{cm} is the secant modulus of elasticity of the concrete.

The resistance is taken as the smaller of Eq. (1) and Eq. (2), with a reduction factor K_t to consider the effect of profiled sheeting. K_t can be calculated by

$$K_t = \frac{0.7}{\sqrt{n_r}} \frac{b_o}{h_p} (\frac{h_{sc}}{h_p} - 1) \quad (3)$$

where n_r is the number of shear connectors in one rib; The reduction factor K_t should not exceed the appropriate value $K_{t,max}$ of 0.75 for n_r=1 and 0.60 for n_r=2.

Equation from EN1993-1-1 is given as follows,

$$F_{V,Rd} = \alpha_V f_u A \quad (4)$$

where $F_{V,Rd}$ is the shear resistance of a shear connector; α_V=0.6 in this paper; *A* is the tensile stress area of the bolt where the shear plane passes through the threaded portion of the shear connector and is the gross cross section of the bolt where the shear plane passes through the unthreaded portion of the shear connector.

Table 2 Experimental results from push tests

Series	P_{ult} (kN /connector)	δ_u (mm)	P_{6mm} (kN /connector)	Mode of failure	P_{test}/P_{EC4+3}
P1.1	34.0	2.7	21.3	C	0.60
P1.2	56.2	7.3	55	C	0.99
P2.1	65.8	17.7	57.2	CS	1.15
P2.2	68.2	18.9	54.5	CS	0.96
P3.1	68.4	29.8	53.0	CS	1.39
P3.2	71.7	24.8	51.2	CS	1.17
P4.1	52.8	8.9	47.3	CS	1.07
P4.2	58.4	11.0	51.7	C	0.88
P4.3	56.6	11.1	53.5	C	1.18
P5.1	49.2	5.3	46.5	CS	1.00
P5.2	60.8	6.9	59.1	C	0.79
P5.3	61.4	14.0	57.8	C	0.64
P6.1	55.8	5.1	52.5	CS	1.13
P6.2	58.1	4.3	53.0	C	0.76
P6.3	70.5	4.5	63.5	C	0.62
P2.3	68.9	8.8	63.1	C	0.88
P2.4	53.3	12.1	43.9	C	0.70

Note: C-concrete failure, S-Connector shear failure, CS-combined.

The ratio between the tested values to the Eurocodes predictions ranged from 0.60 to 1.39. In the cases where fracture of the shear connectors govern the failure of the specimens, e.g. P2, P4.1, P5.1 and P6.1, the predictions in shear resistance from Eurocodes agree well the tested results, , whist in general the predictions overestimated the concrete resistance. The equations do not take reinforcement patterns into account, thus estimations based on them cannot well represent the experimental values.

Typical load vs. slip curves and modes of failure are illustrated in Figs. 4-8, where the load/connector was calculated as the total load divided by 8 shear connectors. The curves shown in Figs. 5-6 are given as the linked peak values at each loading step to

provide a better depiction, omitting the downwards strikes (small load decrease caused by minor concrete cracking, see Figs. 7-8).

When a smaller embedment height of the shear connectors was adopted, i.e. 100 mm, approximately 5 times of the shank diameter, a semi-ductile behaviour of the specimens was obtained, as illustrated in Fig. 4. A brittle concrete shear failure occurred in the elastic range and formed a conical shape whilst little shear deformation was observed in the demountable shear connectors after test. The ultimate resistance was attained prior to the 6mm slip capacity limit specified in EN1994-1-1 for ductile welded shear connectors. In contrast, when the embedment height was increased to 120 mm, approximately 6 times of the shank diameter, a ductile behaviour can be achieved, i.e. the slip capacity was higher than 6 mm.

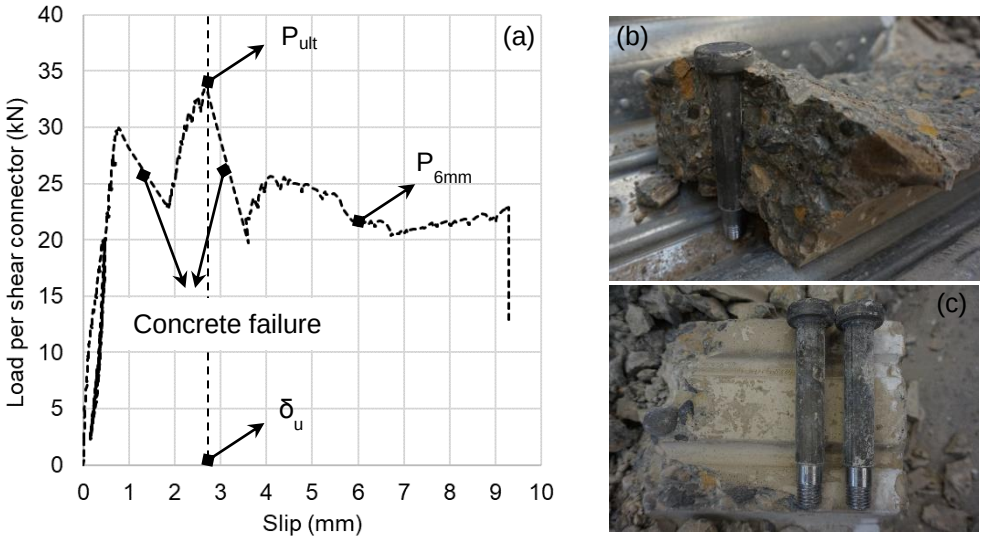

Fig. 4 Results from P1.1: (a) load vs. slip curve (b) concrete cone failure (c) shear connector, 100mm embedment height, 19mm shank diameter, and 17mm step collar

To ensure a ductile behaviour of the shear connectors, embedment height of 120 mm was adopted for the rest of the specimens. If extra reinforcement was used around the shear connectors (see Fig. 2), better confinement to the concrete cone can be achieved, as demonstrated in Fig. 5, cone failure of the concrete can be prevented and the failure was altered to concrete crushing. The concrete compressive strength of Series P3 was only approximate half of those for Series P1, however, full shear strength of the shear connectors was developed and a significant increase in the slip capacity was achieved. Two plastic hinges were developed in the shank and collar parts of the shear connectors in contrast with the relatively un-deformed shape of the connectors obtained from Series P1.

A similar load vs. slip behaviour can be observed between the specimens with continuous slabs and those with separable slabs, indicating the feasibility of adopting separable slabs to facilitate reuse of both the steel beam and the composite slabs after

their first service life. One of the possible reasons for the difference in initial slips could be the hole clearance. The hole diameter in the beam flange was 1 mm larger than the diameter of the stud collar, which can result in a maximum of 2 mm slippage.

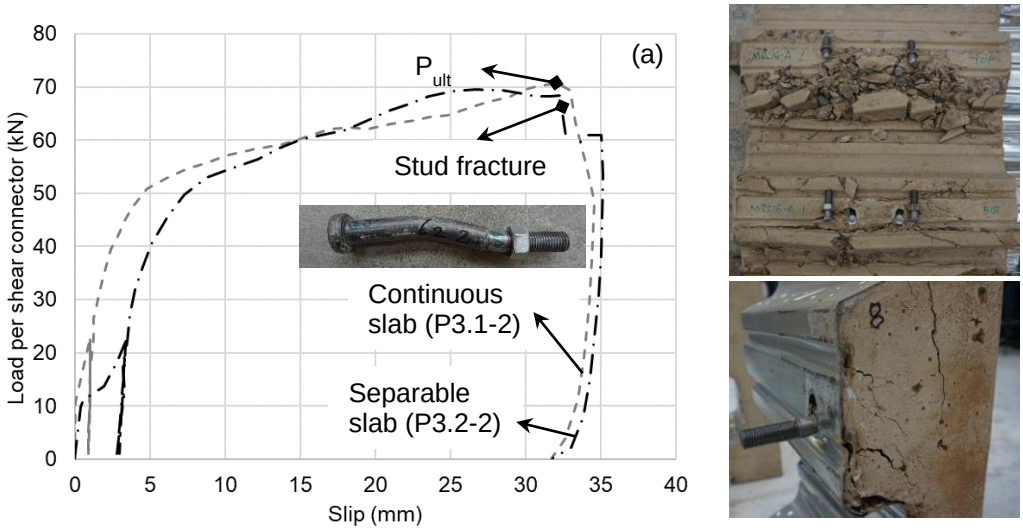

Fig. 5 Results from P3.1-2 and P3.2-2: (a) load vs. slip curves and a shear connector with two plastic hinges developed (b) concrete crushing and fracture of 4 studs from P3.1-2 (c) concrete cracking, deck hole elongation and stud shearing from P3.2-2, 120mm embedment height, 19mm shank diameter, and 17mm step collar

The effect of single embedded nut on the behaviour of the shear connectors can be explored from the results shown in Fig. 6. Comparisons were made among the 19mm headed stud with machined 16mm threads (specimen P4.1-1), 19mm stud with a 17mm step collar and 16mm threads (specimen P4.3-1), and 22mm stud with machined 20mm threads (specimen P4.2-1). The first and the third type of shear connection had one nut per connector embedded in concrete.

The shear connectors without the single embedded nut (collar stud, specimen P4.3-1) experienced a stiffness reduction after the almost identical linear segment of the load-slip curves compared to specimens P4.1-1 and P4.2-1, as illustrated in Fig. 6 (a). However, predominant mode of failure of the specimen P4.3-1 was concrete cone failure, as given in Fig. 6 (b), same to the specimen P4.2-1, therefore similar load-slip relationships were obtained afterwards.

Specimen P4.1-1 experienced stud fracture at a later stage of the test. For a same configuration but higher concrete compressive strength, i.e. specimen P6.1-1, the stud fracture occurred at an earlier stage (refers to Table 2). Fractured and/or deformed shear connectors are shown in Fig. 6 (c), in contract to the well reinforced shear connection discussed earlier, only one plastic hinge was developed in the shear connectors, in the cases with one layer of A193 reinforcing mesh. Local buckling of the double edge trims placed at the centreline of the steel beam was observed during the test, as illustrated in Fig. 6 (d). The trims contributed to the delay in concrete cracking

near the crest of the profiled sheeting and resulted in a relatively gradual decrease in load after the peaks.

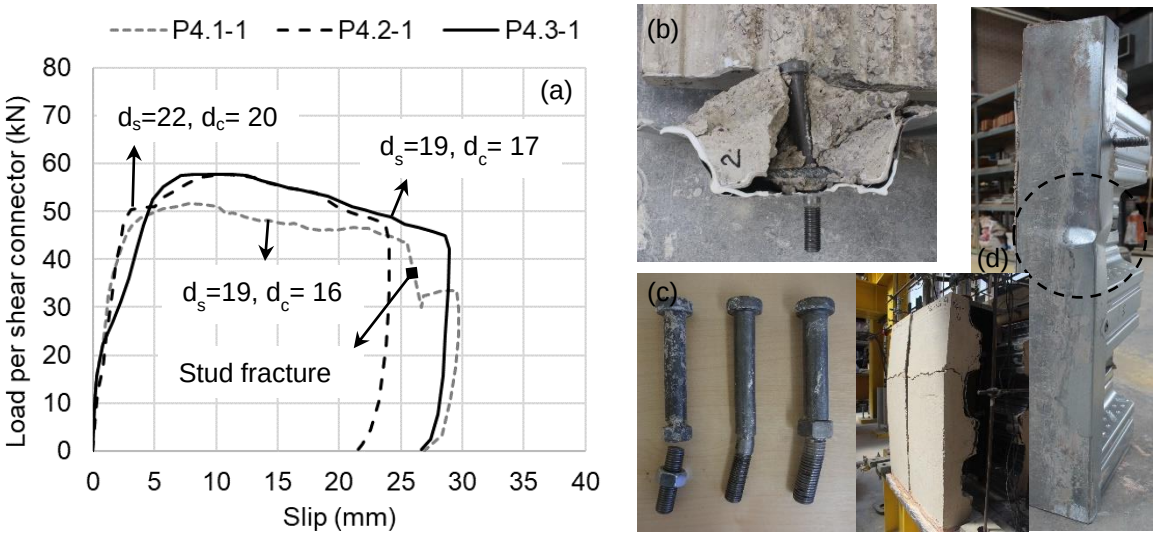

Fig. 6 Results from P4.1-1, P4.2-1 and P4.3-1: (a) load vs. slip curves (b) concrete cone (c) shear connectors after test (d) local buckling of the edge trim and transverse concrete cracking in slabs

Different strength of the shear connectors were compared in Fig. 7. The specimens with Gr. 8.8 bolts and machined 22mm headed studs experienced almost identical stiffness at the elastic range. Both specimens attained the ultimate load at an early stage of the tests and prior to the 6 mm slip limit. The Gr. 8.8 bolts contributed to overall higher resistance but similar slip capacity compared to the 22mm studs. Concrete cone failure was predominant and the studs with lower strength has more shear deformation.

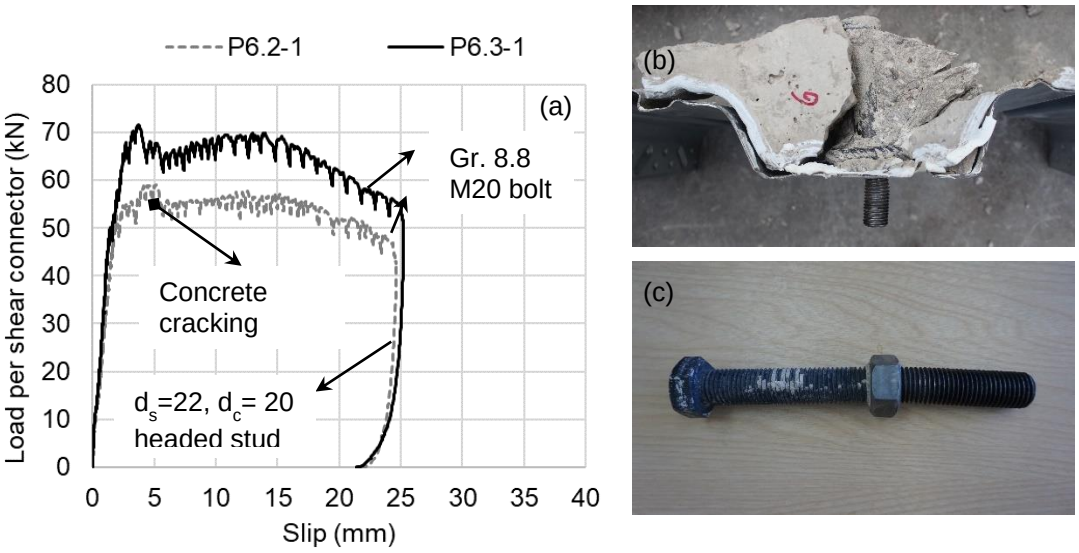

Fig. 7 Results from P6.2-1 and P6.3-1: (a) load vs. slip curves (b) concrete cone with deformed headed stud (c) deformed bolt after test

As shown in Fig. 8 (a), the profiles of the decking affected the initial stiffness, resistance and slip capacity of the shear connection, owing to the fact that the amount of concrete around the shear connectors was different. The existence of U-bars at the vicinity of the shear connectors contributed to the confinement to the concrete, and thus enhanced both the stiffness and resistance of the shear connection, which can also result in earlier fracture of the shear connectors, as illustrated in Fig. 8 (b).

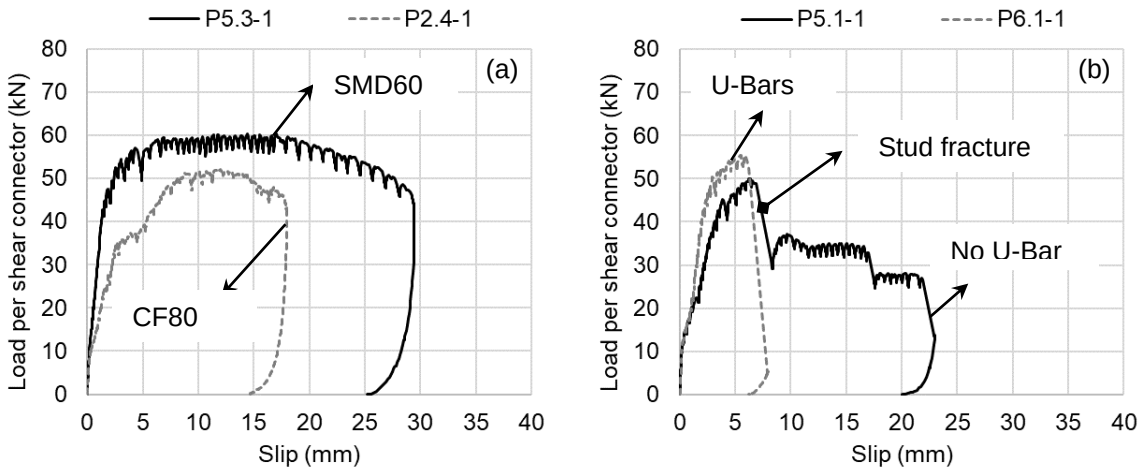

Fig. 8 Comparisons of load vs. slip curves: (a) P5.3-1 and P2.4-1 (b) P5.1-1 and P6.1-1

4. CONCLUSIONS

A summary of thirty-two push tests on demountable shear connectors, i.e. machined headed studs and Gr. 8.8 bolts was presented in this paper. Important parameters such as strength, embedment height and type of shear connectors, concrete strength, deck profiles, and reinforcement patterns were investigated. In particular, behaviour of separable slab segments with cast in edge trims were considered, to facilitate and promote the reuse of the composite slab and steel beam after the first service life without the need of cutting along the centre-line of the beam. The experimental results showed that the shear capacity and behaviour of the demountable connectors in separate slabs and continuous slab were similar and could both fulfil the 6mm minimum ductility requirement stated in Eurocode 4 for welded studs when proper embedment height of connector was ensured. Typical modes of failure obtained were concrete cone failure, concrete crushing and fracture/shear failure of the shear connectors. The results on the shear resistance of these form of shear connectors were compared against existing design equations.

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