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Experimental study on demountable shear connectors in profiled composite slabs

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Abstract

This paper presents an experimental study on demountable shear connectors in profiled composite slabs. Overall, three groups of push-off tests were conducted to assess the shear capacity, stiffness and ductility of the shear connectors. In all the specimens, a pair of shear studs were used per trough and were bolted to each side of the flange of a loading beam. Different concrete strength, embedment height of the shear studs and reinforcement cage were considered. Particularly, a joint was made between the pair studs in two groups of specimens when casting and formed two completely separate slabs per half specimen, to evaluate the load transfer between the pair studs. The experimental results showed that the shear capacity and behavior of the demountable connectors in separate slabs and continuous slab were both similar to the welded connectors and could fulfill the 6mm minimum ductility requirement stated in Eurocode 4 if proper embedment height of connector was used. The shear capacities of the tested specimens were compared against the calculated results obtained from the equations used for welded shear connectors in Eurocode 4 and bolted connections in Eurocode 3. Generally, the Eurocodes prediction underestimated the shear capacities of the push-off specimens.

Keywords: *Demountable shear connectors; profiled composite slabs; discontinuous slabs; push-off tests; shear capacity; ductility*

1. Introduction

Steel framed structures using composite floors have been commonly used in the UK [1]. The composite action between the steel beam flange and floor slabs is normally established through the use of shear connectors. These connectors are traditionally welded to the steel beam flange and embedded in the slabs, which causes difficulty in reusing the steel and the slabs at the end of the life of the structure. One solution is to make the structure demountable, i.e. to detach the floor slabs from the beams using demountable shear connectors. The main benefit of reusing is the reduction of the carbon footprint and cost caused by production of the steel and cement used in construction.

Profiled slabs have attracted more attention as the amount of concrete used is reduced compared to traditionally solid slabs. Knowledge of the performance of welded stud shear connectors in profiled slabs has been established as well as that of bolted connectors in solid slabs. To determine the shear capacity of the connectors between profiled slabs and beams, Lam *et al* [2] conducted twelve full-scale push-off tests on welded headed studs in precast concrete hollow core slabs. The effects of the size of the gap between the floor slab units, the amount of steel placed across the units and the strength of concrete were examined. It was found that the capacity of the shear connectors were reduced compared with that in a solid slab. Mirza and Uy [3] carried out both push-off tests and numerical simulation on solid and profiled slabs with welded shear connectors and different strain regimes were imposed on the concrete element. They concluded that the strength and the load-slip behaviour of composite steel-concrete beams were greatly influenced by the strain regimes existent in the concrete element. Furthermore, it was found that the shear capacities of the connectors depended significantly on the width and rib types of profiled steel decks based on numerical results from their parametric study. Nellinger *et al* [4] tested 20 push-off tests on welded shear stud connectors with profiled slabs using 58 mm and 80 mm deep steel decks. The effect of stud diameter, number of studs in each rib, a second layer of reinforcement and the welding procedure were examined. Pavlović *et al* [5] carried out push-off tests 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, while the specimen with concrete grade of C35/45 showed brittle behaviour. Pathirana et al. [6] and Mirza et al. [7] 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. Dai *et al* [8] developed a group of demountable shear connectors using headed studs and tested them in solid slabs. Their potential and suitability in terms of replacing welded shear studs were assessed. It was found that the demountable connectors could be easily demounted after testing and had similar capacity and behaviour to those of welded shear connectors. A parametric study was then carried out to understand the effect of concrete grades and stud collar sizes on the shear behaviour of the demountable shear connections. Rehman *et al* [9] conducted twelve full-scale push-off tests on demountable shear connectors in profiled slabs considering different concrete strengths, numbers of connectors per trough and different connector diameters. A reinforcement cage was introduced to prevent pre-mature toe failure of the composite slabs. It was found that similar shear behaviour was obtained compared to that of the welded shear studs and the specimens fulfilled the minimum ductility requirement of 6mm required by Eurocode 4 for welded shear connectors. 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.

The main objective of this paper is to fill the research gap on demountable shear connectors (machined from headed shear studs) in a profiled metal deck (60 mm deep) using two rows of connectors with a pair of studs in each row/rib and provide a better understanding on this form of demountable shear connector. Parameters of concrete strength, embedment height of the shear studs and reinforcement cage were considered. Additionally, behaviour of continuous and discontinuous slabs were examined and compared.

2. Experimental programme

To assess the shear capacity, stiffness and ductility of the demountable shear connectors, three groups of push-off tests were conducted in the Heavy Structures Laboratory of University of Bradford. Considered parameters were concrete strength, embedment height of the shear studs and reinforcement cage. Specimens details, instrumentation and testing procedures are presented in the following sub-sections.

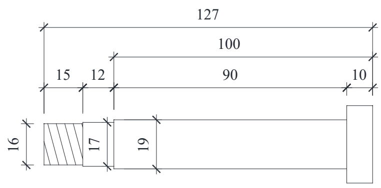
2.1. Specimen details

To achieve demountability of the shear connectors, T.W. Nelson studs (19 mm in diameter) were adopted and were machined according to the design drawings given in Fig. 1, where h_{sc} denotes the height of the studs embedded in concrete. TR60 steel deck, illustrated in Fig. 2, from SMD manufacturer was used to form the profiled composite slabs. Overall, three groups of push-off tests were tested in this paper, namely PUSH-1, PUSH-2 and PUSH-3. A push-off specimen comprises of one beam section (254×254×73UC, length 900 mm), two profiled slabs and eight headed shear studs (two on each trough). In particular, a joint was made by using a 4 mm thick steel plate between the pair of studs in two groups of specimens when casting and formed two completely separate slabs per half specimen, to evaluate the load transfer between the pair of studs.

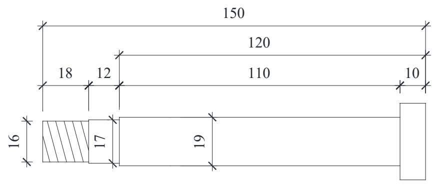
The height of the profiled slabs was 900 mm based on the deck profile for specimens with two rows of connectors. The width and depth of the slabs and the transverse spacing of the shear studs was 610 mm, 150 mm and 100 mm, respectively, similar to the dimensions stated in Eurocode 4 [10]. The specimens were grouped by the batch of concrete casted. The average test-date concrete cube strength was 48.5 MPa for PUSH-1 specimens, 44.6 MPa for PUSH-2 specimens and 25.4 MPa for PUSH-3 specimens.

The diameter of the machined collar part of the shear studs was 17 mm with no clearance in the steel deck and a 1 mm hole clearance in the steel beam flange. The length of the collar (12 mm in this study) was based on the flange thickness of the steel section. A torque of 120 N.m was applied to each of the 19mm Nelson shear studs when assembling the push-off specimens before experiments. The steel section and shear studs were initially in contact when assembling the specimens to make sure that the load can be applied evenly to each of the studs in the beginning of the experiments.

The design drawings of reinforcement cages are illustrated in Fig. 3, where the first drawing is for PUSH-1 while the second one (extra reinforcement around the studs) is for PUSH-2 and 3. φ10 rebar with a yield strength of 500 MPa was used for the reinforment cages.



(a) h_{sc} = 100mm



(b) h_{sc} = 120mm

Fig. 1. Dimensions of machined studs.

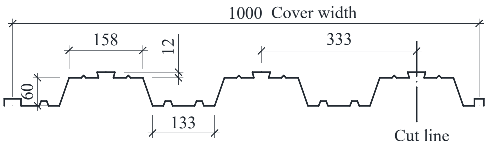


Fig. 2. Steel deck profile (TR60).

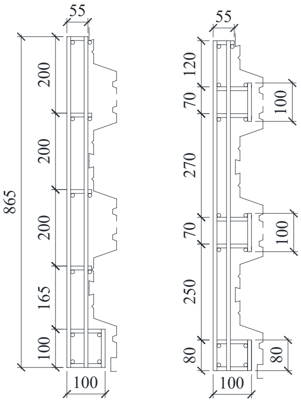


Fig. 3. Design of reinforcement cages (side view).

2.2. Test setup and Instrumentation

Fig. 4 shows the push-off test setup. A 100-tonne actuator was employed to exert the compressive load on the specimens. 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. Overall, eight strain gauges were attached to the beam flange near the shear studs to monitor the load distribution among the studs during tests. Fig. 5 shows the positions of the LVDTs and strain gauges.

Fig. 4. Push-off test setup.

Fig. 5. LVDTs and Strain gauges.

2.3. Testing procedures

During the experiments, compressive load at steel section was applied by the minimum interval of 24 kN or 0.5 mm of slip; at each interval, load increment was only applied after the load settled. Loading rate was 0.2 mm/sec. In some cases, cycle loadings were applied between 5% and 40% of the estimated maximum failure load and then the specimens were loaded up to failure.

3. Experimental results

3.1. Group PUSH-1

In group PUSH-1, the embedment height in the profiled slabs of the shear studs were examined. The considered height was 100 mm and 120 mm, and the two specimens in this group were thus named as H100 and H120, respectively. The load (per stud) vs. slip curves and failures of PUSH-1 were given in Fig. 6 and 7, respectively.

For specimen H100, two cycle loadings were applied between 20 kN and 160 kN (20 kN/stud). A brittle concrete failure was observed when the applied load reached 30 kN/stud (first peak load), and the load was dropped off. After that, a second peak load (which also was the maximum load at 34 kN/stud and 2.7 mm slip) was obtained following by brittle concrete failure and a larger load drop-off. The application of further displacement loading led to a third peak load and then a slow and ductile load decrease. The load at 6 mm slip was 21.3 kN/stud, which was 62.6% of the maximum, which revealed this specimen performed a semi-ductile behavior, nevertheless it did not meet the 6 mm ductility requirement specified in the Eurocode 4 for welded shear connectors.

For specimen H120, two loading cycles were also applied between 20 kN and 160 kN (20 kN/stud). When the applied load reached 40 kN (71.2% of the actual maximum failure load), another 6 loading cycles were applied due to equipment fault, during which concrete failure was observed in the bottom rib of R-slab (Fig. 7, R-slab). With further displacements, the load increased until final concrete failure in both ribs of L-slab occurred. This specimen met the 6 mm ductility requirement with a load value of 55.0 kN/stud, 58.2% higher than that of H100. A maximum load of 56.2 kN/stud was obtained at slip of 7.3 mm.

Thus, the embedment height of the shear connectors in profiled slabs had a large influence on the load capacity of the specimen but did not affect the mode of failure. There was no obvious deformation in shear studs as shown in Fig. 7 in both tests, while a obvious cone failure of concrete was observed formed from the vicinity of the head of the connector and cracked through the depth of the concrete in a 45 degrees direction. The cracks propagated transversely across the rib and caused further rib failure at a very late stage of the tests. There was no obvious deformation in the holes of the profiled steel deck.

From this group of tests, it was concluded that 120 mm embedment height was as a better option and thus was adopted in the following tests, and extra reinforcement was added around the studs to provide better confinement to the concrete cone.

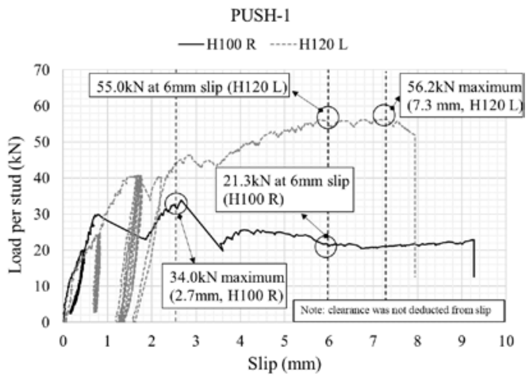


Fig. 6. Comparison of load vs. slip curves (PUSH-1).

Fig. 7. PUSH-1 after test.

3.2. Group PUSH-2

In group PUSH-2, the behaviour of continuous slabs (MR) and discontinuous slabs (GAP4) were examined. Additionally, the enhancement of extra reinforcement around the shear connectors to load capacity and slip behaviour of the specimens were addressed. Two identical specimens were tested for each type. The load (per stud) vs. slip curves and failures of PUSH-2 were given in Fig. 8 and 9, respectively.

For PUSH-2 specimens, five loading cycles were applied between 20 kN and 200 kN (25 kN/stud). During the tests, small concrete cracks occurred around the studs causing the shot load drop-off lines in the curves. Larger load drop off occurred suddenly at the later stage of the tests caused by fracture of two of the shear studs in each test. After this, displacement was applied further to capture a full load-slip profile of the tests. Another two studs fractured finally and then unloading was applied. All of the specimens in this group met the 6 mm ductility requirement. The load at 6 mm slip was 57.8 kN and 53.4 kN for MR and GAP4 specimens, respectively. The maximum loads were 66.5 kN and 68.4 kN at slips of 17.9 mm and 19.2 mm for MR and GAP4 specimens, respectively. The GAP4 specimens (discontinuous slabs) had a similar load-slip behaviour compared to the MR specimens (continuous slabs). They had a relatively lower initial stiffness and load level at the 6 mm slip but a higher maximum load and slip capacity. Ongoing work on discontinuous slabs with edge trims showed better performance compared to continuous slabs.

The extra reinforcement around the shear studs had a great influence on the capacity, ductility and mode of failure of the specimens tested. The reason might be that it improved the embedment condition of the shear studs and provided better confinement to the concrete cone as it overlapped with the failure surface of the concrete cone and prevented pull-out of the studs, and thus contributed to the failure load and ductility increase by altering the mode of failure from brittle concrete failure to stud failure. The stud finally fractured at the collar part as can be seen in Fig. 9. Rib failure in the slabs was prevented. The elongation of the holes in the profiled deck was observed after the slabs were dismantled from the beam section and the decks were removed from the specimens.

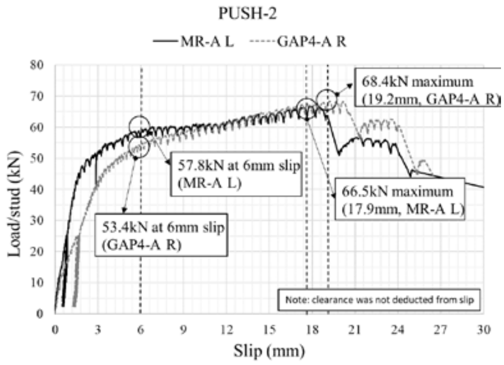


Fig. 8. Comparison of load vs. slip curves (PUSH-2).

Fig. 9. PUSH-2 after test.

3.3. Group PUSH-3

In group PUSH-3, continuous slabs (MRC16) and discontinuous slabs (GAP4C16) were examined with lower concrete grade. The reinforcement cages used were kept as the same with those in group PUSH-2 specimens. Two identical specimens were tested for each type. The load (per stud) vs. slip curves and failures of PUSH-3 at single loading regime were given in Fig. 10 and 11, respectively. In Fig. 11, the photo for the half specimen was taken from nearside of the stud. Table 1 summarizes the load at 6 mm slip, maximum load and the slip at the maximum load for all the specimens mentioned.

For PUSH-3 specimens, similar load-slip behaviour was observed compared to PUSH-2 specimens. Small cracks led to drop-off of the load during the tests, and stud fracture occurred in a later stage. In contrast, more severe concrete crushing underneath the studs was observed as shown in Fig. 11. As can be seen from the curves, all the specimens in this group met the 6 mm ductility requirement as well. The load at 6 mm slip was 54.2 kN and 56.3 kN for MRC16 and GAP4C16 specimens, respectively. The maximum load was 66.0 kN and 73.7 kN at slip of 28.5 mm and 23.6 mm for MRC16 and GAP4C16 specimens, respectively. The GAP4C16 specimens (discontinuous slabs) had a higher load at 6 mm slip and a higher maximum load compared to the MRC16 specimens (continuous slabs). Initial stiffness was almost identical in both cases. But the slip capacity of the GAP4C16 specimen was lower than that of the MRC16 specimen.

It was found that a lower concrete strength did not lead to lower strength and ductility of the specimens. The reason might be that the concrete strength was fully developed and the extra reinforcement around the shear studs provided superior confinement to the concrete cone.

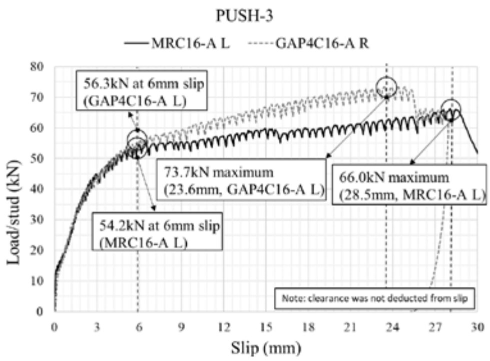


Fig. 10. Comparison of load vs. slip curves (PUSH-3).

Fig. 11. PUSH-3 after test.

Table 1. Summary of tested results.

ID	Load at 6mm slip (kN/stud)	Max. load (kN/stud)	Slip at max. load (mm)
H100	21.3	34.0	2.7
H120	53.0	56.2	7.3
MR	57.8	66.5	17.9
GAP4	53.4	68.4	19.2
MRC16	54.2	66.0	28.5
GAP4C16	56.3	73.7	23.6

Note: MR, GAP4, MRC16 and GAP4C16 have duplicate speimens.

4. Design equations in Eurocodes

Currently there is no design rule for demountable shear connectors. The methods available for welded headed studs in Eurocode 4 and bolted connections in Eurocode 3 are therefore used to predict the shear resistance of the demountable connectors tested in this paper. Equations in Eurocode 4 are given as follows,

$$P_{Rd,S} = 0.8 f_u \pi d^2/4 \quad (1)$$

$$P_{Rd,C} = 0.29\alpha d^2\sqrt{f_{ck}E_{cm}} \quad (2)$$

where

P_{Rd} is the characteristic resistance of a shear connector;

$\alpha = 1$ (for $h_{sc}/d > 4$);

d is the diameter of the shank of the stud (17 mm in this paper);

f_u is the ultimate tensile strength of the stud but not greater than 500 N/mm^2;

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 (1) and (2), with a reduction factor of

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

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 Eurocode 3 [11] 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 and f_u = 505 N/mm^2 in this paper;

Comparison of the resistance predictions using the method provided in Eurocodes 4 and 3 and the tested results is given in Table 2. Basically, the Eurocodes underestimated the resistance with exception of the specimen with 100 mm stud embedment (H100).

Table 2. Comparisons of shear capacity between test and equation results.

ID	Experiment P_{Test} (kN/stud)	Combined EC4+3 P_{Rd} (kN/stud)	P_{Test}/P_{Rd}
H100	34.0	54.5	0.62
H120	56.2	54.5	1.03
MR	66.5	54.5	1.22
GAP4	68.4	68.1	1.00
MRC16	66.0	39.3	1.68
GAP4C16	73.7	49.1	1.50
		Average	1.18
		Standard Deviation	0.35
		Coefficient of Variation	0.293

5. Conclusions

Three groups of push-off tests have been carried out to assess the shear capacity, stiffness and ductility of the demountable shear connectors in profiled composite slabs. Concrete strength, embedment height of the shear studs and reinforcement were examined. Particularly, discontinuous slabs were formed and tested in comparison with continuous slabs. The following conclusions may be drawn based on the experiments presented in this paper:

(1) The shear capacity and behavior of the demountable connectors in discontinuous slabs and continuous slabs were both similar to the welded connectors and fulfilled the 6mm ductility requirement if

the proper embedment height of the connector was used; Continuous slabs had a lower maximum strength compared to discontinuous slabs while their strength at 6mm slip was similar;

(2) The height of the stud above the deck was important, it would affect the behaviour of the shear studs. 100 mm height was not sufficient, concrete rib failure occurred earlier at low slip compared to 120 mm height;

(3) The mode of failure could be altered from concrete cone failure to stud fracture by improving the concrete confinement. The use of a modified reinforcement cage increased both the load and slip capacity of the specimens; A lower concrete strength did not lead to a lower load capacity and ductility as superior confinement to the concrete cone was supplied by extra reinforcement around the headed studs.

(4) Generally, the Eurocodes prediction underestimated the shear capacities of the push-off specimens with the exception of the specimen with a stud embedment height of 100 mm;

The presented experimental work in this paper will contribute to the development of design rules for demountable shear connectors and validation of numerical simulation.

Acknowledgement

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