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Dennis Lam (corresponding author)
Xianghe Dai (co-author 1)
Ashraf Ashour (co-author 2)
Naveed Rahman (co-author 3)

Recent Research on Composite Beams with Demountable Shear Connectors

This paper presents experimental and numerical investigation on an innovative composite floor system with deconstructability. In this system, a composite slab formed with metal profiled decking is connected to a steel beam using demountable shear connectors. A series of push tests was conducted to investigate the behaviour of this form of shear connectors. In addition to the push tests, a full-scale composite beam was tested to failure in the laboratory under a number of cycles of monotonic loading. For direct comparison, a similar composite beam test was conducted using same section size, concrete strength, but using the conventional welded headed stud connectors. Test results showed that the behaviour of the composite beam with demountable shear connectors is comparable with the specimen with welded shear connectors. After the test was terminated, the demountable shear connectors were unfastened and the composite floor can be easily lifted off from the steel beam. Test result showed that these demountable shear connectors possess high ductility in comparison with the equivalent welded shear connectors. Simple design rules currently use in Eurocode 4 for the welded shear connections and Eurocode 3 for bolts are proposed to predict the shear capacity of this form of demountable shear connectors.

Keywords: Composite Structures; Circular Economy; Ductility Characteristics; Demountable Shear Connectors; Metal Profiled Decking; Eurocode 4.

1 Introduction

Our cities are growing and the challenge of building social and economic prosperity on a sustainable basis has not been so critical since World War II. Our global population is projected to reach 9.6 billion by 2050, with half the population already living in urban areas. Countries all over the world need to be making a term investment in their building assets, to provide the foundations for the economic and social prosperity.

The winners for the future will be those who consider the full life-cycle of their assets, with a strategic plan in place for resilient infrastructure that are designed for the circular economy. We need a new way of thinking and operating if we are to respond to the multiple challenges represented by our changing world. It can be hard to think beyond the world we are building and living in now, but it is essential that we do so, if we are going to create a more sustainable built environment. Embedding circular economy and whole lifecycle thinking is key to creating a more sustainable built environment by improving the design and performance throughout the life cycle of products and services. Our ability to embrace and harness the benefits of emerging technologies such as building information modelling (BIM) and 3D printing will be important to meet the challenge ahead.

Currently, there is a lack of systems approach to maximize the values of building components and materials throughout each stage of the building's life. There is a failure of integrated thinking of the future value of building components and materials among the stakeholders and a lack of manufacturing innovations to allow reuse and repair of building components that create better value of recycled materials, and an absence of logistics system to enable reuse. Therefore, a system re-design which connects all stages and players in the value chain from feedstock providers, manufacturers, architects and designers, construction teams, demolition and waste management companies and clients, as well as those who finance and regulate the sector by organising the building chain in a circular way while fulfilling the growth ambition is badly needed.

To cater for these needs, a four-way strategy that improves the circularity of the construction sector is proposed: (1) Smart design: commit to smart design of buildings in order to make them more suitable for repurposing and for the reuse of materials. (2) Dismantling and separation: efficient dismantling and separation of waste streams enables high value reuse. (3) High-value recycling: high value recovery and reuse of materials and components. (4) Marketplace and resource bank: exchanging commodities between market players. The roadmap and action agenda present a large number of short and long term actions that can contribute to transforming the construction chain. For new build, smart design of buildings is important in the transition of construction circular economy. [1] This has led to research on the reuse of steel beams in composite construction. Steel-concrete composite beams are the most cost effective construction system for multi-storey steel frame buildings owing to the composite action between steel beams and composite slabs. In the current construction practice, composite action is achieved by shear studs welded through the profiled sheeting to the top flange of the steel beam and embedded in the concrete slab which make dismantling, alteration and deconstruction of the composite structures almost impossible when composite structures reach the end of their design life. In the current practice, steel beams have to go through a recycling process and cannot be re-used. Although steel is 100% recyclable, however, the recycling process requires a significant amount of energy and produces carbon emission into the environment.

In this research, a new form of shear connectors, which are demountable, were used as an alternative to welded connectors in composite beams. The demountable shear connectors allow the steel beam to be reused at the end of the structural design life, bolted shear connectors have been rarely used in construction, apart from rehabilitation work. Unlike welded shear connections, the demountable shear connectors are easy to dismantle, enabling the steel beam to be reused without a recycling process. In addition, demountable shear connectors can be easily installed on site into the predrilled flange of the steel beam and steel profiled decking. Due to the lack of information about the behaviour of demountable shear connector, they have not been widely adopted in construction practice and no design guidance is currently available.

Eurocode 4 [2] provides a simple procedure for push-off test and equations to predict the shear capacity of shear studs in composite beams. However, the push-off test details provided in Eurocode 4 are for welded shear connectors in solid concrete slabs. Mottram and Johnson [3] suggested a geometric adjustment to the standard push-off test for welded headed stud connectors in metal decking slabs. Pavlovic *et al*. [4] studied the M16 Gr8.8 bolted shear connector through push-off tests in solid slabs and compared the experimental results with welded headed shear studs in solid slabs. It was found that the Gr8.8 bolted shear connectors with a single embedded nut achieved about 95% of the shear resistance under static loads, but the stiffness reduced by 50% as compared with the welded headed stud. However, their research was only focused on solid slabs with high strength bolts. This is quite different from this research as demountable headed shear connectors are used with cast in-situ metal deck composite slabs. Dai *et al.* [5] performed a series of push-off tests using demountable shear connectors with solid slabs and concluded that the demountable shear connector has a slip of up to 20mm before failure and the shear capacity was about 84% of the welded shear connector at the slip of 6 mm.

Dallam [6] and Marshall *et al.* [7] investigated the high strength friction-grip (HSFG) bolts. But their main purpose was to investigate the pre-tension behaviour of the high friction grip bolts. Dedic and Klaiber [8] and Kwon *et al.* [9] investigated the shear capacity and performance of post installed bolted shear connectors under fatigue loading. The focus of their research was to strengthen the existing non-composite buildings using high strength friction grip bolts as shear connectors. Pathirana *et al.* [10] and Mirza *et al.* [11] carried out research on demountable studs using blind bolts. It was found that blind bolts behaved very similar to welded headed studs in terms of stiffness and strength but the blind bolt had a relatively brittle behaviour. Henderson *et al.* [12] discussed different types of shear connection under dynamic loading and reported that the removable shear connectors had very similar stiffness and strength as welded headed studs in composite beams. Hawkins [13] tested the anchor bolts without embedded nuts in a solid slab using different lengths and diameters of the bolts. It was found that the shear capacity of these anchor bolts was about 80% of the welded shear connectors. Ollgaard *et al.* [14] carried out extensive studies on welded shear connectors. Oehlers and Bradford [15] discussed different types of connectors used for composite beams including the bolted and demountable connectors. It was concluded that the bolts can be attached directly to the flange prior to the casting of concrete; or the concrete slab and the steel beam can be bolted together after casting by using bolts or friction grip bolts. Allwood and Moynihan [16] conducted three composite beam tests using M20 Gr 8.8 bolts; it was found that the Eurocode 4 prediction for the welded shear connector is conservative when compared to their experimental observation.

From the literature review, it is found that previous research on bolted connectors was carried out using high strength bolts with solid slabs. There is little research carried out using metal deck composite slabs. The only composite beam tests were carried out using the Gr 8.8 M20 bolts, but no push test was carried out using demountable shear connectors with metal deck composite slabs. In this research, beam test was carried out to investigate the slip behaviour and the ultimate moment capacity of composite beam with demountable shear connectors.

2 Shear Connector Capacity

2.1 Push Tests

To assess the shear capacity, stiffness and ductility of demountable shear connectors, a series of push tests, as shown in Figures 1 and 2 were carried out at the University of Bradford. In general, the push test arrangement is very similar to the one described in Eurocode 4. It consisted of two identical concrete slabs of size 610 × 510 × 150 mm connected through shear connectors with predrilled holes in a steel section (203 × 203 × 52 UB). The test specimens were divided into 6 groups as shown in Table 1. In each group, two replicate specimens were tested. These specimens covered different concrete strength, connector diameter and types.

The two composite slabs of each specimen were casted horizontally with the same concrete mixes and cured in an open-air environment. Ribbed steel bars with a diameter of 10mm were used for the reinforcement cage. The strength of the concrete slab was determined by taking the average strength of six 100mm cubes cured in the same condition as the test specimens and tested on the test day.

The clearance between the hole in flange and collar (shank) of the bolt was 1.0 mm and 1.0 mm clearance was also provided for the hole in the metal profiled decking. The nuts were tightened using tools that are commonly used by steel erectors in steelwork construction. The nominal height of the connectors in concrete for all the specimens was 120 mm, specimens S1, S2, D1, D2 and M1 to M6 have a shank diameter of 19 mm embedded in the concrete and a 17 mm diameter collar passing through the steel beam flange and a threaded portion of 16mm diameter (C1). The specimen M7 has a shank diameter of 22mm with a collar and a threaded portion of a diameter 20mm (C2). For the specimen M8, a pair of

M20Gr 8.8 bolts was used (C3).

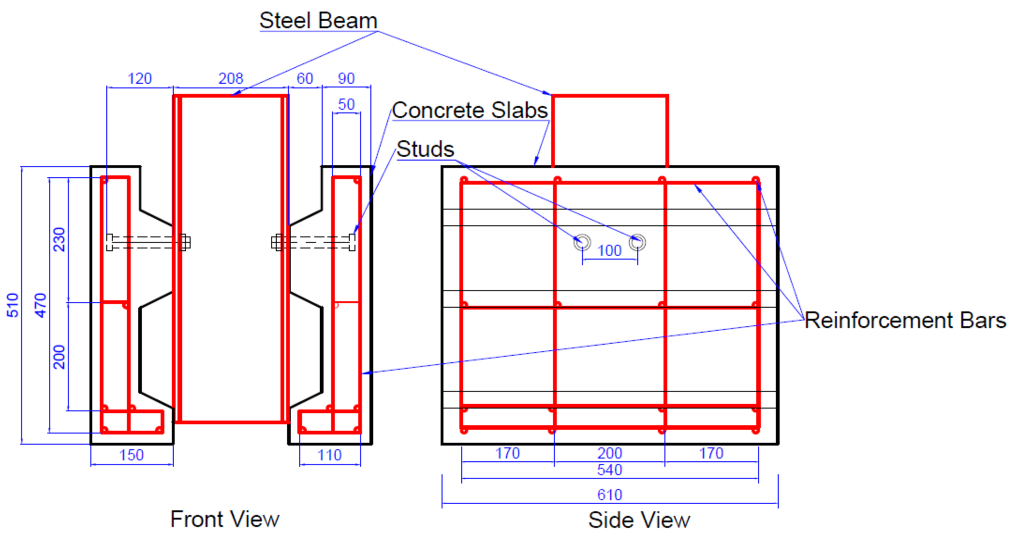

Fig. 1. Push-off test specimen with modified reinforcement cage

Table 1. Geometric configuration summary of tested specimens

	Test Specimen ID	Connector type	Concrete Strength (MPa)	Shear connectors per slab
Group 1	S1	C1	57.5	2
	S2	C1	54.5	2
Group 2	D1	C1	29.4	2
	D2	C1	28.5	2
Group 3	M1	C1	43.4	2
	M2	C1	40.9	2
Group 4	M3	C1	36.2	1
	M4	C1	30.5	1
Group 5	M5	C1	55.7	2
	M6	C1	58.1	2
Group 6	M7	C2	22.7	2
	M8	C3	19.2	2

Figure 2 shows the test setup of the push test. Eight linear variable displacement transducers (LVDTs) were installed at the top of the steel beam and concrete slabs, as shown in Figure 2 to measure the vertical displacements, which subsequently used to calculate the relative slip between the steel beam and the concrete slab.

During the test, the applied load was increased by 5kN interval; at each interval, a further 5 minutes between loading was allowed for the load to settle before the next load increment was applied. When the applied load reached 40% of the predicted failure load based on the Eurocode 4 equations, the load-control method was changed to the displacement-control method, in which a constant increment rate of 0.2mm/min was adopted until the failure of the specimens was observed, i.e. rapid reduction of the load capacity.

Fig. 2. Push test set up and test arrangement

2.2 Push Tests Results

Figures 3 and 4 show the two main failure modes observed in these tests. The first mode of failure is concrete cone failure with concrete crushing and cracks where no connector fracture was observed. In this type of failure, the concrete around the connector failed in compression before the connector is yielded. Crushing of concrete started from the vicinity of the connector head and cracks developed through the depth of the concrete slab forming a cone shape around the shear connectors. The second mode of failure is the shearing of the connectors where the connectors are usually sheared off at the base of the slab. Cracks on the outer surfaces of concrete slab were observed in tested specimens. These cracks were more evident in specimens with low strength concrete than those with higher concrete strength. The transverse concrete cracks first appeared on the outer surface of the slabs just near the middle of the slabs, but they did not propagate through the thickness of the slab due to the presence of the reinforcement cage.

These cracks are visible when the metal profiled decking was removed from the concrete slab after the test. The concrete damage patterns observed from different specimens are very similar irrespective to the shear connector arrangement and the concrete strength. However the concrete cone formed in a single shear connector per trough was less than that in specimens with pair shear connectors per trough.

Figure 4 shows the typical failure mode, which is characterised by the combination of connector fractured, concrete cone and the cracks around the root of the connector due to compressive forces. The average width of the cone was about 260 mm in the specimens with a pair of 19 mm diameter connectors, which is approximately double the bottom width of the slab trough (110 mm) and about 25% wider than the width of the steel beam flange. The average width of the cone was about 130mm with a single shear connector connection, which is half of that of specimens with pair connectors per trough. It was observed that the average cone width was up to 360mm for the specimen with pair connectors of a diameter of 22 mm, which is 100 mm larger than that of specimens with connector diameters of 19 mm. This is possibly due to the increases in the shear resistance and the higher compressive forces applied to the concrete around the demountable shear connectors.

Fig. 3. Concrete cone failure observed from test specimen M7

Fig. 4 Fracture of the shear connectors

In specimen M2, although the shear connector deformed significantly, it did not fail before the concrete failure occurred. The concrete cracks propagated longitudinally across the concrete slab and causing rib shear failure to the concrete slab as shown in Figure 5. The deformation of the hole in the profiled metal decking was also observed during the test. The tearing of the metal decking around the hole was more prominent in specimens with a single shear connector compared to the paired connector specimen as shown in Figure 6.

The second mode of failure is that the shear connector fractured at the collar. For this mode of failure, the connector fully yielded and fractured at the collar of the shear connector. The connector reached its maximum yield stress while the concrete slab did not reach its maximum stress. In the paired connector specimens M1, M5 and M6; the connectors sheared off as expected due to the high concrete strength. A similar connector failure was also observed in single shear connector specimens M3 and M4. Typically fracture occurred at the collar position close to the slab. The deformation of the shear connectors observed in specimens M5 and M6 with a high concrete strength slab was much smaller than that of specimen M2 with a low concrete strength slab.

Fig 5. Rib shear failure to the concrete slab

Fig. 6. Elongation of holes in the metal deck

Table 3 summarises the maximum shear capacity per connector, amount of slip at maximum load and at failure; and the mode of failure observed in the push tests. The load - slip curves of all push test specimens are presented in Figure 7. These load-slip curves have a clear elastic and plastic portion. In the elastic region, the load-slip curves show an almost linear relationship for all specimens.

Table 3. Summary of maximum shear resistance and failure mode

Test Specimen ID	Concrete cube strength (MPa)	Max Shear capacity (kN/stud)	Slip at Maximum load (mm)	Slip value at failure (mm)	Mode of Failure
S1	57.5	60.0	5.5	10.0	Premature failure
S2	54.5	44.5	6.4	9.2	Premature failure
D1	29.4	61.5	5.2	7.8	Cone failure
D2	28.5	42.2	4.3	6.5	Cone failure
M1	43.4	69.9	9.2	21.2	Stud fracture
M2	40.9	68.2	7.2	18.7	Cone failure
M3	36.2	80.0	16	17.9	Stud fracture
M4	30.5	79.6	26	28.2	Stud fracture
M5	55.7	76.1	6.8	10.6	Stud fracture
M6	58.1	82.6	7.0	9.5	Stud fracture
M7	22.7	66.3	4.1	6.0	Cone failure
M8	19.2	63.5	5.8	6.6	Cone failure

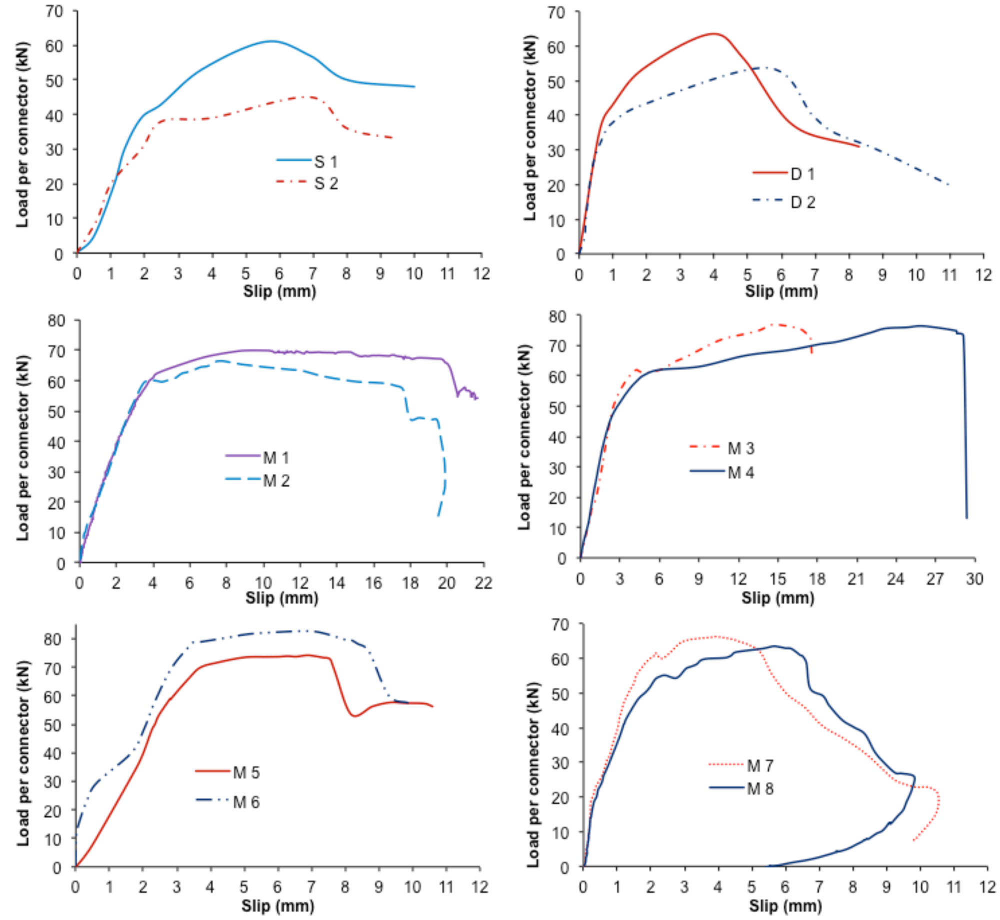

Fig. 7. Load-slip curves of push-off test specimens

2.3 Proposed Design Rules for the Demountable Shear Connector

Currently there is no specific assessment method available for demountable shear connectors. The methods available for headed shear connectors in Eurocode 4 and bolts in the Eurocode 3 [17] are summarised in Table 4 and are used to predict the shear capacity of demountable shear connections. The use of these equations is illustrated in Figure 8 according to different failure modes as shown in Figures 3 to 6.

Table 4: Proposed equations for demountable shear connector

Eurocodes	Expression	
EN1994-1--1	$P_{Rd,C} = \dfrac{0.29\alpha d^2\sqrt{f_{ck}E_{cm}}}{\gamma_v}$	(1)
	$P_{Rd,S} = \dfrac{0.8 f_u \dfrac{\pi d^2}{4}}{\gamma_v}$	(2)
	$k_t = \dfrac{0.7}{\sqrt{n_r}}\dfrac{b_0}{h_p}\left(\dfrac{h_{sc}}{h_p}-1\right) \leq 0.85$ for n_r =1 and 0.7 for n_r =2	(3)
EN1993-1-8	$F_{V,Rd} = \dfrac{\alpha_v f_{ub} A}{\gamma_{M2}}$	(4)

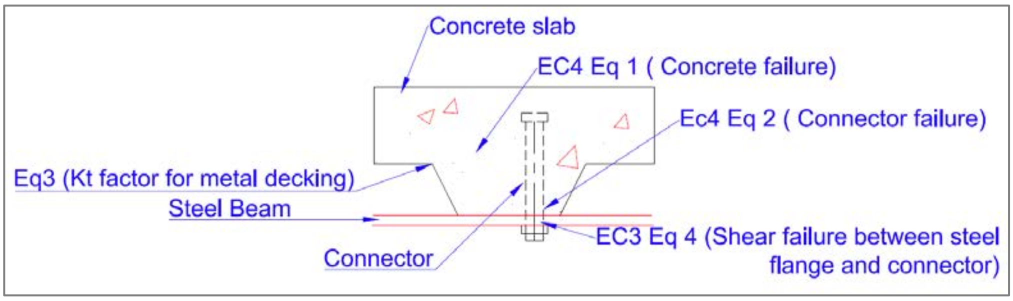

Fig. 8. Application of EC4 and EC3 on different failure modes

Table 5 summaries the comparison of the experimental results and predictions using the method provided in Eurocodes 4 and 3. It can be seen that Eurocodes slightly underestimated the shear resistance of demountable shear connectors regardless to the modes of failure in specimens. Currently, with no code providing any design guidance for this form of demountable shear connectors, a combination of Eurocode 3 and 4 equations recommended by the authors provide a reasonable accuracy in predicting the shear resistance of this form of demountable shear connectors.

Table 5. Comparison of test results with different code's predictions

	Experiment P_{Test} (kN/Stud)		Combined EC4 and EC3		P_{Rd} (kN/Stud)		
	Max. Load	Failure	$P_{rd,C}$	$P_{rd,S}$	$F_{v,Rd}$	P_{Rd}	P_{Rd}/P_{Test}
S1	60	Slab	94.9	90.8	68.1	68.1	1.13
S2	44.5	Slab	91.8	90.8	68.1	68.1	1.50
D1	61.5	Concrete	62.6	90.8	68.1	62.6	1.01
D2	42.2	Concrete	54.6	90.8	68.1	54.6	1.29
M1	69.9	Stud	79.6	90.8	68.1	68.1	0.97
M2	68.2	Stud	76.8	90.8	68.1	68.1	0.99
M3	80	Stud	77.8	90.8	68.1	68.1	0.85
M4	79.6	Stud	86.4	90.8	68.1	68.1	0.85
M5	76.1	Stud	93.0	90.8	68.1	68.1	0.89
M6	82.6	Stud	95.6	90.8	68.1	68.1	0.82
M7	66.3	Concrete	59.3	150.8	113.1	59.3	0.89
M8	63.5	Concrete	53.6	209.5	117.6	53.6	0.85
						Ave.	0.97
						COV	0.19

3 Beam Test

3.1 Test Set Up

A full-scale composite beam with demountable shear connectors along with an identical composite beam with welded shear connectors were constructed and tested. IPE-300 beam and Arcelor Cofraplus60 metal profiled deck were used for the test specimens. For the demountable composite beam, holes of 18mm diameter were predrilled at the metal profiled decking and at the top flange of the steel beam. The details of the test specimens are given in Table 6 below. The beam was cast un-propped with profiled decking laid perpendicular to the longitudinal axis of the steel beam. The demountable shear connectors were fastened using traditional construction tools used by steel erectors on site. After the shear connectors were fixed, torque wrench was used to measure the torque achieved by these tools. A torque of 60 Nm was recorded.

A single shear connector per trough was used for the composite beam as it is designed for partial shear connection. 19mm diameter × 100mm height demountable shear connectors with 17mm collar diameter were used for the test. The connectors were placed stagger along the steel beam as shown in Figure 9. All test specimens were cast un-propped, after reaching the required concrete strength, a four point loading system was used to apply loads on the simply supported composite beam as shown in Figures 10 and 11. The vertical downward load was applied using the 1000kN hydraulic jack and distributed equally using a spreader beam. Eleven linear variable displacement transducers (LVDTs) were used to measure end slip (Figure 12), vertical deflection and relative slippage between steel beam and the concrete slab at various connector positions (Figure 13).

Table 6: Details of the test specimen

Span of steel beam	5600 mm
Steel section (S355)	IPE 300
Steel strength	420 N/mm^2
Concrete compressive strength	37 MPa
Length of composite concrete floor	5400 mm
Width of composite concrete floor	1340 mm
Thickness of composite concrete floor	130 mm
Height of metal decking	58 mm
Position of reinforcement mesh	Top of steel decking
Number of connectors	26
Connector spacing	227 mm
Reinforcement	A142 mesh

Fig. 9. Set up before casting of the composite beam

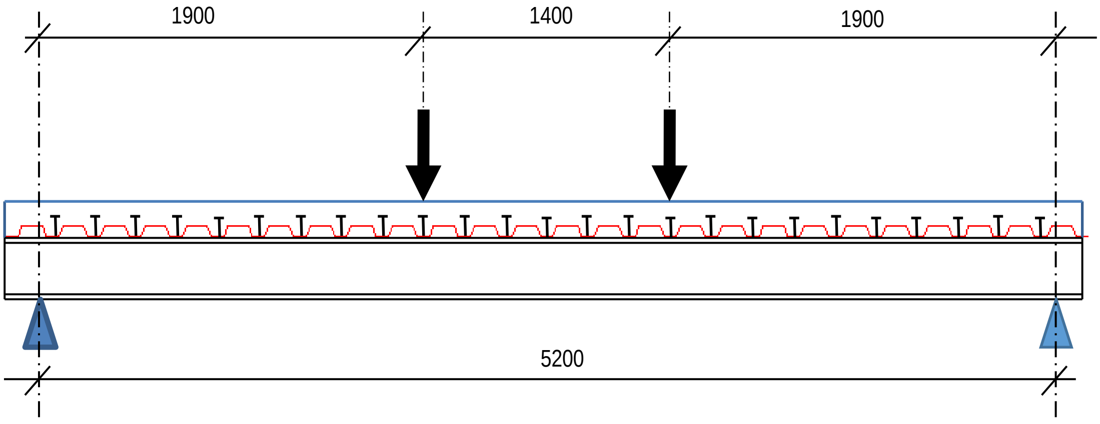

Fig. 10. Loading arrangement

Fig. 11. Test set up

Fig. 12. LVDT for the end slip

Fig. 13. LVDTs at the 2nd and 4th shear connector's position

Strain gauges were used to measure the strain in the steel beam and the strain on the top surface of the concrete slab at the mid-span of the beam. The strain measurements are used to determine the position of the neutral axis of composite beam. The specimen was subjected to a sequence of loading cycles, starting with an applied load of 5 kN/m^2 (excluding the weight of the specimen and spreader beams) and subsequent cycles of 7.5 kN/m^2, 15 kN/m^2 and finally loaded to failure.

3.2 Beam Test Results

Table 7 presents the test results of composite beam with demountable shear connectors (CB-DSC) and composite beam with the traditional welded shear connectors (CB-WSC). Figure 14 shows that load deflection curves of composite beam with demountable shear connectors and the equivalent composite beam with the traditional through deck welded shear connectors respectively. Initially, the beam was loaded up to 5 kN/m^2 (working load) and this was cycled for three times to observe any initial slip and deflection. Then the load was increased to 7.0 kN/m^2 and 15 kN/m^2 respectively. Then, the load was increased by an increment of 1kN/m^2 until 70% of the predicted failure load was reached. After that, the load was increased by displacement until failure was reached.

From Figure 14, it can be seen that at ultimate load, the mid-span deflection for the composite beam with demountable shear connectors reached 80 mm (more than span / 70), this showed that the beam exhibited excellent ductility. The test was terminated at this point as steel was fully yielded while the load deflection behaviour remains stable. There were some concrete cracks on the top surface in the longitudinal direction along the steel beam. No major cracking was observed on the side of the concrete slab. No deck separation, shear connector failure or concrete pull out were observed during the test. In addition to this, there was little evidence that uplift of composite slab occurred throughout the test.

Figure 14 shows the comparison of welded and demountable specimen. The demountable composite floor system showed almost similar capacity but slightly lower initial stiffness. However, the overall stiffness was higher than the beam with welded connectors. The composite beam with demountable shear connectors has demonstrated significantly higher ductility. There was an initial deflection due to the clearance holes in demountable specimen, which was not observed in the welded specimen.

Table 7. Test results of composite beams

Test Specimen ID	Concrete Strength (MPa)	Max. load at failure (kN)	Max. moment capacity (kNm)	Deflection at max. load (mm)
CB-DSC	37	340.6	323.6	79.9
CB-WSC	36	333.1	316.5	59.1

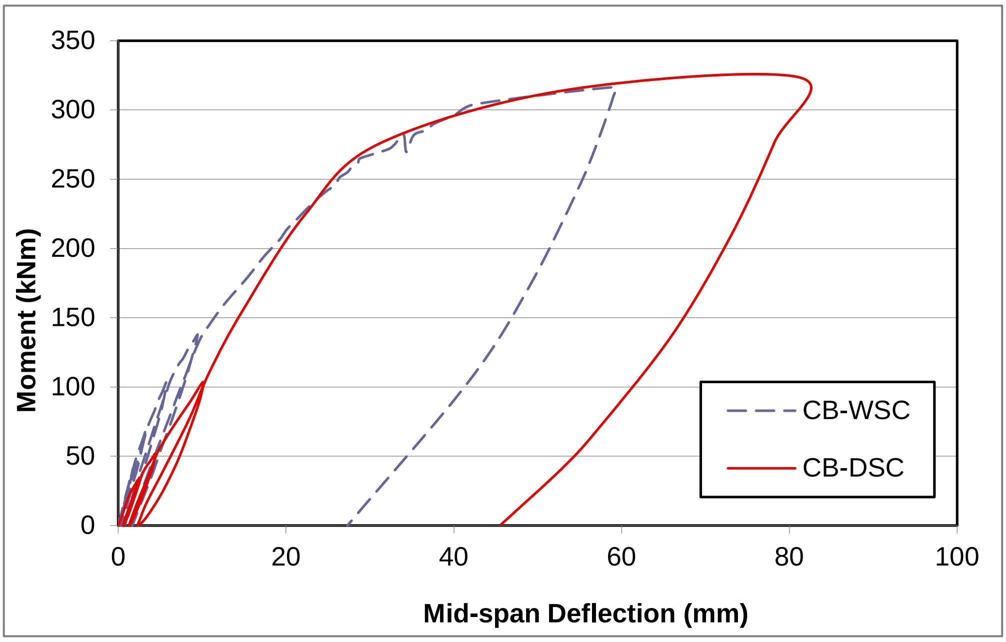

Fig. 14. Moment vs. mid-span deflection

The maximum mid-span deflection and residual defection against applied floor load obtained from the test is presented in the Table 8. The residual deflection of 0.6mm was observed at the first cycle of loading. This was due to the hole clearance in steel flange. But no further residual deflection was seen as this loading cycle was repeated for three more time. The moment vs. deflection curve (Figure 14) is linear up to the 70% of ultimate loading. This shows a stable linear behaviour in the elastic region.

The load vs. end slips is presented in Figure 15. The end slips at both ends show that there is a modest asymmetry in the behaviour of shear connectors, more than 6 mm of slip were recorded at both ends. This fulfils the EC4 requirement for ductility. At the first loading, an initial average slip of 0.09mm was observed due the clearance between the connector and predrilled holes in the steel flange. But this initial slip did not increase after repeating the same loading cycle for three times.

Table 8. Deflection at mid span

Load cycles	1.0 x working load 5.0 kN/m^2	1.5 x working load 7.5 kN/m^2	3.0 x working load 15.0 kN/m^2	Maximum load at failure 47.0 kN/m^2
Deflection at each loading cycle (mm)	2.8	1.9	4.1	71.1
Residual deflection (mm)	0.6	0.7	1.0	43.3
Cumulative residual deflection (mm)	0	0.6	1.3	2.3
Cumulative deflection (mm)	2.8	5.3	10.1	82.2

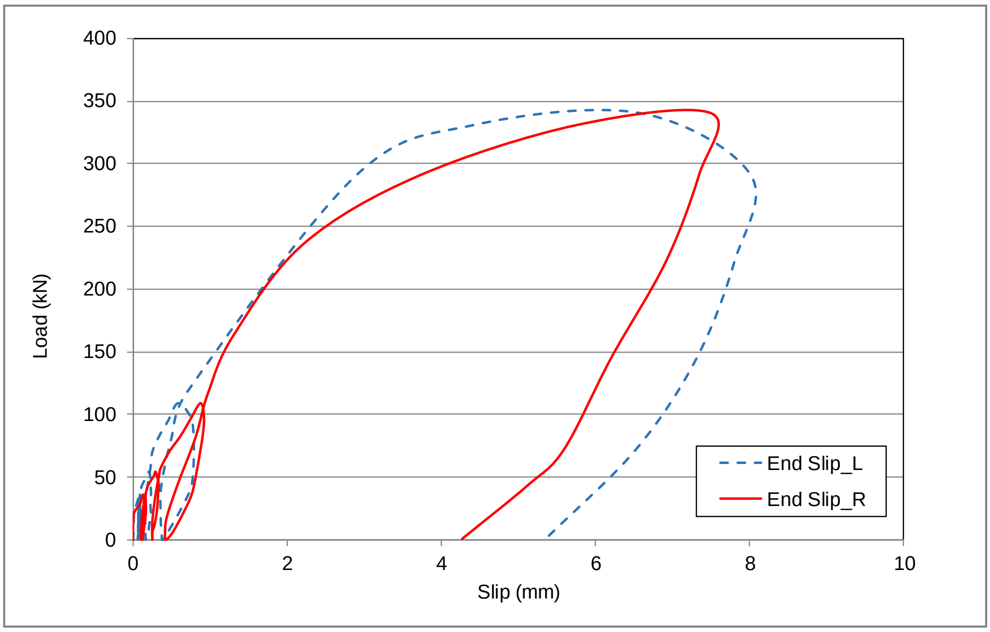

Fig. 15. Load-slip curves of the composite beam with demountable shear connectors

After the test, the demountable composite floor system was evaluated for the purpose of demountability and to determine whether the structural parts of the demountable composite floor system can be reused at the end of the design life. All the demountable shear connectors were undone after the test was terminated, even those some permanent deformation was observed in the beam, the composite concrete slab could still be easily lifted off from the steel beam without any problem at the end of the test. This will allow the steel beam to be reused after the structural design life without going into recycling process. Figure 16 shows the demountability of this form of composite beam.

Fig. 16. Demountability of the composite beam

4 Conclusions

A demountable shear connector in the form of headed stud shear connectors is developed. A series of the push off tests of demountable stud with profiled decking have been carried out to assess its shear capacity. Test results shown that these shear connectors can be easily demounted after test and have a similar shear capacity when comparing with the welded headed studs. Current research shows that Eurocode 4 and 3 equations could be safely used to evaluate this form of demountable shear connectors. Beam test with demountable shear connectors was carried out and compared with the beam test with welded shear connectors, results showed that the beam with demountable shear connectors has similar stiffness and superior ductility. After the test, the shear connectors can be unbolted and the beam and slab can be removed for reuse. A demountable system has two additional advantages over traditional connectors: no welding and increased flexibility. Welding studs alters their material properties, whereas bolts' material properties are unaffected by installation. In addition, welding is susceptible to fatigue under cyclic or seismic loading, so bolts may be preferred in these circumstances. In addition, site welding involves extra health & safety risks that bolting could also be avoided.

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Authors

Prof Dennis Lam
School of Engineering
University of Bradford
Bradford, UK. BD71DP
d.lam1@bradford.ac.uk

Dr Xianghe Dai
School of Engineering
University of Bradford
Bradford, UK. BD71DP
x.dai@bradford.ac.uk

Prof Ashraf Ashour
School of Engineering
University of Bradford
Bradford, UK. BD71DP
a.f.ashour@bradford.ac.uk

Mr Naveed Rehman
School of Engineering
University of Bradford
Bradford, UK. BD71DP
n.rehman9@student.bradford.ac.uk