New composite flooring system for the circular economy

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Abstract. Circular economy is an economic system aimed at minimizing wastes and making the most of the current resources. This regenerative approach contrasts with the traditional linear economy, which has been adopted by the construction industry. Developing new construction technologies for sustainable built environment is a top priority for the construction industry throughout the world. Much of the environmental impact from the construction industry is associated with the consumption of resources and generation of waste. The construction industry in Europe consumes over 70,000 million tonnes of materials each year and generates over 250 million tonnes of waste. Composite flooring formed by connecting the concrete slabs to the supporting steel beams has been widely used for many years and is well established as one of the most efficient floor systems in multi-storey steel frame building structures. However, shear connectors are welded through the steel decking to the steel beams and cast into the concrete; this made deconstruction and reuse of these components almost impossible. A new composite flooring system which allows for the reuse of the steel beams and composite floor slabs is developed and tested to assess its potential and suitability for reuse. This paper presents the results of a series of full-scale beam tests and demonstrates the reusability of this new form of composite flooring systems. Simplified hand calculations are also provided and compared against beam tests.

Keywords: Circular economy, design for deconstruction, design to Eurocodes, demountable shear connectors, composite beam tests, reusable composite floor system

1. Introduction

Steel manufacturing consumed a large amount of energy when comparing with the production of concrete. However, steel is 100% recyclable whilst concrete can only be down cycle or sent to landfill. Steel and concrete contribute to the most embedded energy in construction due to their huge usage. In building structures, flooring contributes to a large volume of concrete being used. If both steel and concrete components can be reused without remanufacturing at the end life of the structure, this could save significant resources and reduces carbon emissions, and move up the waste hierarchy from recycling to reuse. The global materials use is projected to be 167Gt in 2060 which is double the level in 2011 (OECD 2018), in the meantime, non-metallic minerals, such as sand, gravel and limestone, represent more than half of the total material use.

To achieve a circular economy, it is essential to create reusable components for a sustainable built environment. Allowing us to use structural components longer by repeated use. Creating high quality systems to avoid recycling or down cycling. Composite structure consists of steel beams connected to metal profiled deck concrete slabs is the most popular flooring system for steel framed building structures around the world. Composite action between the steel beams and the floor slabs led to increases in member stiffness and bending moment resistance, which in turn, enable the construction of long spans and shallow floors. However, the current composite flooring system normally achieved its composite action by use of through deck stud welding that has made the deconstruction of the composite floor system impossible.

Recently, researchers have been searching for innovative connection systems to overcome the weakness of the welded shear connectors to make the deconstruction of the composite system possible (Suwaed and Karavasilis 2017, 2018), bolts used as demountable shear connectors might be a solution (Liu et al. 2014, Pathirana et al. 2016, Uy et al. 2017,), however, so far bolts have not been extensively adopted in construction practice to fulfil the deconstruction aim. (Pavlovic et al. 2013) studied the M16 Gr8.8 bolted shear connector through push tests in solid slabs and compared the experimental results with welded headed shear studs in solid slabs. It was found that the Gr. 8.8 bolted shear connectors with a single embedded nut achieved about 95% of the shear resistance under static loads, but the stiffness was reduced by 50% compared to the welded headed stud. A full-scale composite beam test with profiled metal decking was reported by (Moynihan and Allwood 2014) using M20 Gr 8.8 bolts as shear connectors in a composite beam. Their research showed that these bolts may be used as demountable connectors and they behaved in a similar way to welded connectors and the slabs can be taken off easily from the steel

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beam. (Dai et al. 2015) investigated the load slip behaviour of modified demountable shear connectors through push tests and finite element modelling. (Rehman et al. 2016, 2017) and (Lam et al. 2017) studied the modified demountable shear connectors in composite slabs by push tests and full-scale composite beam systems. It was found that the demountable shear connectors completely fulfilled the aim of deconstruction of the composite system.

This paper presents recent research by the authors on the use of demountable bolted shear connectors using cast in-situ composite construction with profiled decking, the aim is trying to keep the first cycle of use as close as possible to the current construction practices. The behaviour and failure modes were analysed through a series of push tests and numerical simulations, which led to a better understanding to the behaviour of this form of shear connectors.

The three main characteristics for re-use are: it is not worn, yielded or corroded, it is not a superseded technology, and it can be interfaced with new components. The aims and objectives of the research is to develop an innovative composite floor system which is demountable and allow the reuse of both the steel beams and the concrete slabs. The composite system presented in this paper is closely mimicked the currently commonly used composite flooring system to encourage uptake by the construction industry. This paper investigates the reusability of the system by comparing its performance of first-use vs. re-use. The demountability and re-assemblability of the system is highlighted.

A sequence of experiments on a long-span asymmetric composite cellular beam with conventional welded shear connectors were previously performed at the Heavy Structures Laboratory, University of Bradford (Sheehan et al. 2016). The degree of shear connection used was less than half of the required number for such beams specified in EN 1994-1-1 (CEN 2004). The beam failed at an applied uniform load of 3.4 times of the design working load and shear resistance was 45% higher than predicted. Overall, the tests demonstrated the potential of unpropped composite beams with low degrees of shear connection.

To explore the potential of demountable shear connectors, the composite beams tested and presented in this paper, were cast unpropped and were designed for a 27% nominal degree of shear connection (actual 23.1%) which was much smaller than the minimum value of 40% required for welded studs specified in EN1994-1-1 (CEN 2004). Demountability of the structure and reusability of both the composite slabs and the steel beam was highlighted.

Notations	
A_a	Cross-sectional area of the steel beam section
A_{fc}	Cross-sectional area of the compression flange of the composite section
E_a	Modulus of elasticity of the steel beam
E_{cm}	Secant modulus of elasticity of concrete
$I_{y,a}$	Second moment of area of the steel beam
L	Span of the beam
M_a	Design plastic resistance moment of the steel beam
M_{Ed}	Design bending moment
$M_{pl,Rd}$	Design value of the plastic resistance moment of the composite section with full shear connection
N_a	Design value of the normal force in the steel beam
N_c	Design value of the compressive normal force in the concrete flange
$N_{c,f}$	Design value of the compressive normal force in the concrete flange with full shear connection
P_{Rd}	Design value of the shear resistance of a single stud
P_{Rk}	Characteristic value of the shear resistance of a single shear stud
f_{ck}	Characteristic value of the cylinder compressive strength of concrete
f_u	Ultimate tensile strength of the steel beam
f_y	Yield strength of the steel beam
k_{sc}	Stiffness of a shear stud
n_f	Number of stud connectors for full shear connection
t_w	Thickness of the web of the steel beam
t_f	Thickness of the flange of the steel beam
η	Degree of shear connection

2. Demountable composite floor system

2.1 Hypothesis of research

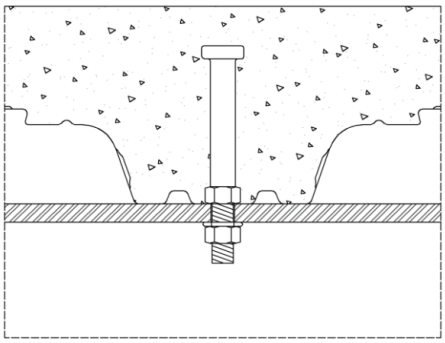
The hypothesis of the research is that for a composite flooring system, a cast in-situ composite floor can be cut into manageable smaller scale slab segments (in size according to lifting and transportation limits) and demounted from its supporting beam at the end of its life, so that the supporting beam and potentially the slab segments can be salvaged and reused in the subsequent life cycles instead of recycling for steel and downcycling or sent to landfill for concrete after their first use, to significantly reduce the associated energy use and embodied carbon emissions of this mostly adopted form of flooring system for multi-storey composite structures in the UK. The composite slab should be detachable from the steel beam, and the process of deconstruction, e.g. cutting the composite slab, and reconstruction should be minimal and practical. In the meantime, the reconstructed composite floor system should have a similar structural performance to the original system.

2.2 Demountable shear connection and its shear resistance and stiffness

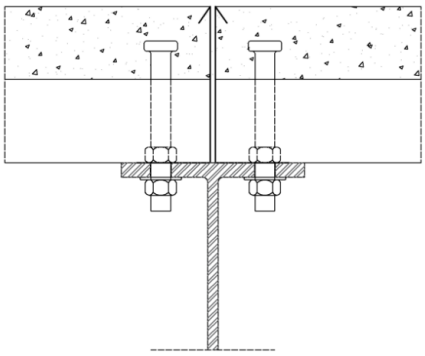
Details of the demountable shear connection system is shown in Fig. 1 (a) – (d). The commonly used headed shear studs is adopted as the demountable shear connectors with small modification to the diameter near the end, see Fig. 1(c). In this way, the same material properties of the connectors are retained and thus are easier to be adopted in practice.

Instead of welded onto the steel beam, the shear stud is threaded at the end in the workshop to allow it bolted to the top flange, thus can be disconnected from the steel beam and allow the composite slab detachable from the supporting beam. The stiffness of a demountable shear connector is usually smaller than that of a welded shear connector. To address this problem, and also ensure the stud hold in position during concreting, double nuts are considered with one nut above and one nut below the steel flange, see Fig. 1(a) and (b).

The nature of trapezoidal profiled steel sheeting can well accommodate transverse cutting to the span of the beam at the crest of the slab when the ribs run perpendicular to the beam. Longitudinal cutting is needed considering the typical 3m to 4.5m span of the slab (spacing between the beams). However, it is almost impossible to cut along the beam with no damage to the steel beam. To facilitate detaching without longitudinal cut along the beam span, full-depth double edge trims are cast in concrete to form the edge of the slabs at the centre of the steel beam in between the shear connectors, which makes the composite slab completely separable with no longitudinal cut along the centre of the steel beam, see Fig. 1(b) and (d).



(a)



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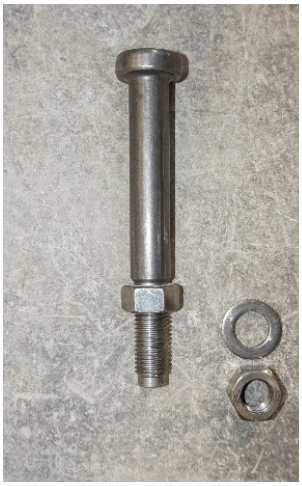


(c)

(d)

Fig. 1 Details of the demountable shear connection system, (a) cross-section view of bolted stud connectors in composite slab with profiled steel sheeting, (b) cross-section view of composite beam with separable composite slabs (with full-depth edge trims), (c) photo of a stud connector with nuts and washer, (d) photo of the system

The details and dimensions of the slab profile and demountable shear connection adopted in this research are described as follows:

- the overall depth of the composite slab is 150mm,
- the profiled steel sheeting used is SMD TR60, with 60mm depth and a 12mm re-entrant, 0.9mm in thickness, giving a slab topping of 90mm over the decking,
- the depth and thickness of the steel edge trims was 150mm (same to the slab depth) and 0.9mm, respectively,
- the stud connector has a shank diameter of 19mm with 16mm threaded end,
- the overall height of the stud (including head thickness) cast in concrete is 120mm (approximately 6.3 times of the shank diameter), with a projecting height of 60mm over the top of the decking profile and 30mm concrete cover

over the head of the stud,
- the longitudinal spacing of the studs/bolt holes in the decking and top flange is 333 based on the profile of the decking (stud placed centrally in the trough),
- the transverse spacing of the studs/bolt holes is 100mm in top flange,
- a torque of 120 N·m is used to tighten the demountable stud connectors, equivalent to the torque applied by the impact driver commonly used on site for steel frame erection,
- to prevent longitudinal spilling of the concrete slabs, standard U-bars (10mm in diameter) are used around the stud connectors placed on to the crest of the deck (approximately 60mm in height off the top flange of the beam).

The characteristic resistance (P_{Rk}) of demountable shear connectors in C30/37 concrete slab with TR60 sheeting was 50kN per stud which was determined from standard push tests and was taken as the minimum failure load reduced by 10%, according to Annex B in EN 1994-1-1 (CEN 2004). The stiffness of the stud connectors (k_{sc}) is determined as 20.5 kN/mm (Fig. 2) based on experimental result from standard push test by using the following expression, $k_{sc} = \frac{0.7P_{Rk}}{s}$ where s is the slip value at the load of $0.7P_{Rk}$.

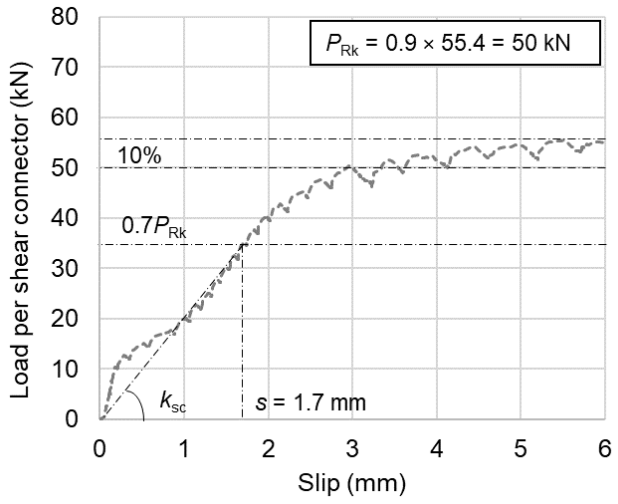


Fig. 2 Stiffness of shear connection, k_{sc}, determined from standard push test

3. Composite beam and materials

3.1 Composite beam specimens

The composite beam investigated in this study consist a rolled universal steel I-beam (UB 356 × 171 × 57 kg/m, 6.3m in length, 6.0m clear span, steel grade S355) with an effective width of 1.5m (Span/4), formed two pieces of separable composite slab (each piece 150mm depth × 750mm width × 6.1m length, concrete grade C30/37) that spans perpendicular to the beam. Two full-scale composite beam specimens, namely B1a and B1b, were constructed unpropped at the same time with the same concrete mixture, at the Heavy Structures Laboratory, University of Bradford. Setup of B1a and B1b before concreting and specimens after cast are shown in Fig. 3 and Fig. 4. Bended rebars were embedded in specimen B1a for lifting purpose, see Fig. 4.

A total of thirty-eight shear studs was used per beam. There were fourteen stud connectors from the support to the first load point (2.25m). The holes in the steel beam and the profiled steel sheeting are pre-drilled, with hole diameter of 18mm and 17mm, respectively.

B1a B1b

Fig. 3 Setup of B1a & B1b before concreting

B1a B1b

Fig. 4 Specimens after the concrete was cast

Mesh reinforcement (A193, mesh size 200m × 200mm, 7mm wires) was placed on to the crest of the steel decking with rebar spacers. The top concrete cover to the mesh reinforcement was approximately 30mm. One ϕ10 U-bar was put around per two shear studs and tied to the mesh reinforcement with metal wires and concrete spacers. A close view of the reinforcement is given in Fig. 5.

Temporary props were attached to the beams (Fig. 6) to support the decking during concreting to simulate unpropped construction and were removed after concrete was hardened. The use of unpropped construction is not explicitly covered in EN1994-1-1 (CEN 2004). One of the advantages of unpropped construction of composite beam is that there is zero stress to the shear connectors caused by dead loads after

the concrete has hardened, resulting in less slip requirement compared to propped construction.

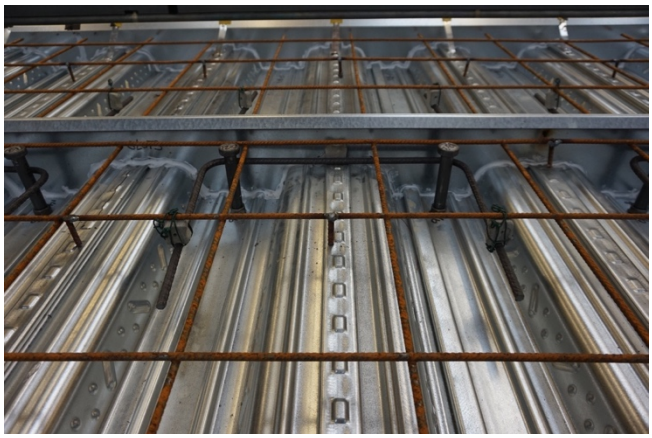


Fig. 5 Close view of U-bar and mesh reinforcement

Fig. 6 Temporary supports attached to the steel beam to facilitate un-propped construction

3.2 Material properties

The concrete used was in grade C30/37, with an average 14-day cube strength of 45.7 N/mm^2, 28-day strength of 51.2 N/mm^2 and test-date strength of 54.7 N/mm^2. The average tested compressive strength of grout was 19.4 N/mm^2 after 24hrs, 38.8 N/mm^2 after 48hrs and 53.5 N/mm^2 on test day of the composite beams.

Tensile tests for the steel beam was performed. Coupons were extracted from both the flange and the web. The average tested yield strength (f_y) of the flange and web was 387.7 N/mm^2 and 442.4 N/mm^2, respectively. The average tested tensile strength (f_u) of the flange and web was 491.4 N/mm^2 and 517.1 N/mm^2, respectively. The average modulus of elasticity of the steel beam (E_a) was 210GPa.

4. Beam tests and results

4.1 The philosophy of testing

The composite beams were designed to investigate the structural behaviour and composite action of the beams at two scenarios / two life cycles of the components:

- First use composite beam with cast in-situ slabs, and
- Reused steel beam and slab segments with transverse grout joints, see Fig. 7.

Overall, three beam tests were planned and conducted as follows:

1) TEST-1, failure test on specimen B1b,
2) TEST-2, loading the specimen B1a up to serviceability limit,
3) TEST-3, failure test on specimen B1a with reused steel beam and slab segments.

The behaviour of specimen B1a from TEST-2 (first use) and TEST-3 (reuse) will be compared against that of specimen B1b from TEST-1 (first use). Then, the reusability of the steel beam and reclaimed slab segments will be highlighted.

Fig. 7 Reassembled/reused slab segments with grout joints (specimen B1a REUSE)

4.2 Test setup and instrumentation

A 250-tonne actuator was employed to exert the compressive load at the mid-span of the beam. Loading setup and instrumentation are illustrated in Fig. 8. Rollers were placed underneath the composite beam ends and the loading beam ends. LVDTs and/or dial gauges were used to measure the deflections of the beam at mid-span and loading points, and slips between the beam and the slabs. Strain gauges were used to capture the longitudinal strains at beam sections (Fig. 8) and slab (at mid-span).

4.3 Loading protocol

The beams were subjected to 2-point loads so that a defined zone of the beam was subjected to constant shear (2.25 m). The specimens were subjected a sequence of loading cycles, excluding the self-weight of the specimens (composite slab 2.67 kN/m^2, steel beam 0.19 kN/m^2) and loading system (3.1kN, 0.17 kN/m^2). Specimen B1b was tested to failure after (TEST-1) a few cycle loadings up to the design working load (5 kN/m^2), 40% of estimated failure load and serviceability state limit (span/300). Specimen B1a experienced cycle loadings up to 40% of tested failure load (determined from B1b) and serviceability limit before unloading (TEST-2). Afterwards, the continuous slab (B1a) was cut into segments, detached and reconnected to the beam for a second use /test (TEST-3). The tests on specimen B1b and reused specimen B1a were terminated after the maximum load was reached and the steel beam fully yielded.

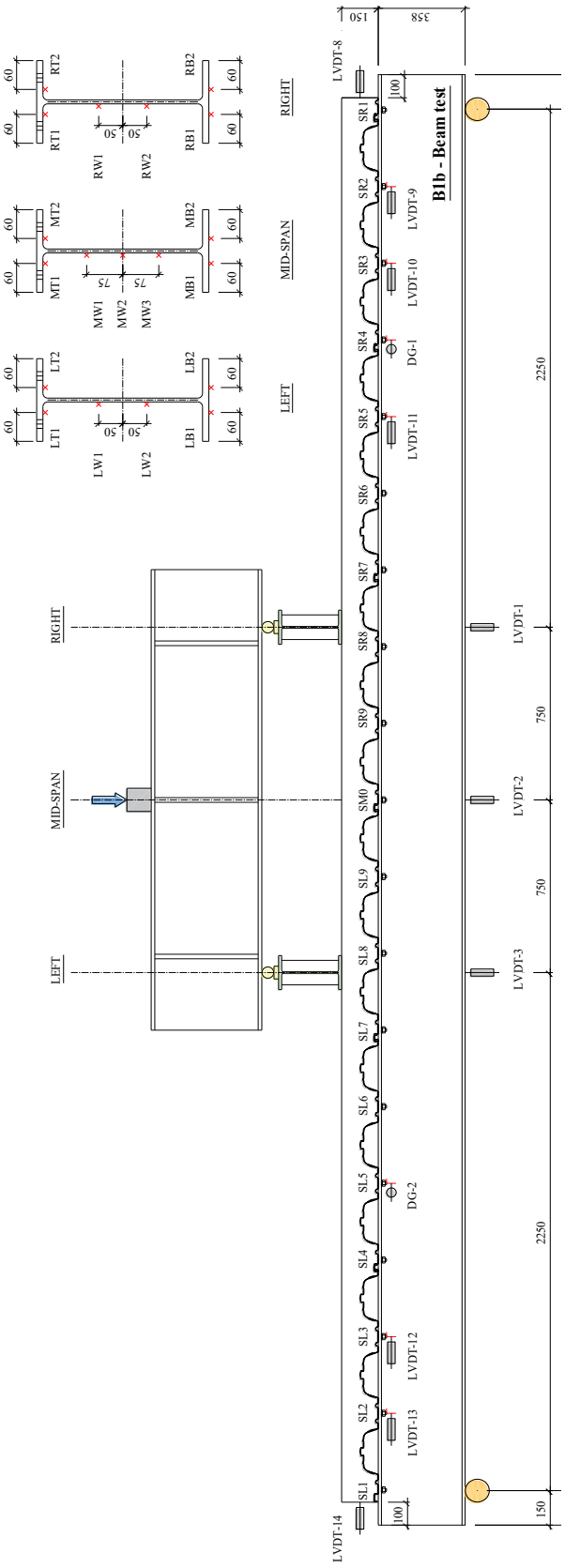


Fig. 8 Loading setup and instrumentation

Loading protocols for B1a and B1b is given as follows:

Specimen B1b (TEST-1):

- loading to 90 kN (design working load of 5 kN/m^2), 5 cycles between 25 kN (5% of estimated failure load) to 90 kN
- loading to 200 kN (40% of estimated failure load / 2.2 times design working load), 5 cycles between 25 kN to 200 kN
- loading to 20 mm of mid-span deflection (serviceability state limit, span/300), (287 kN – 280 kN, approximately 3.1 times design working load), 2 cycles between 25 kN to 20 mm of mid-span deflection
- loading to failure
- unloading

Specimen B1a before dismantling (TEST-2):

- loading to 200 kN (40% of failure load), 5 cycles between 25 kN to 200 kN
- loading to 275 kN (20 mm of mid-span deflection, serviceability limit), 10 cycles between 25 kN to 275 kN, wait for 5 mins each time at 275 kN
- unloading

Specimen B1a after reassembling with grouting (TEST-3):

- loading to 275 kN (20 mm of mid-span deflection, serviceability limit), 1 cycle between 25 kN to 275 kN, wait for 5 mins at 275 kN
- loading to failure
- unloading

4.4 Experimental results

4.4.1 Load vs. mid-span deflection

Load vs. mid-span deflection curves of the beams are compared in Fig. 9. Total load was excluded of self-weight of beam and slab. Both composite beam experience large mid-span deflection which demonstrated superior ductility. Summarized failure load, failure moment and maximum mid-span deflections from the failure tests of specimens B1a reuse and B1b were listed in Table 1. The difference in resistance from the two tests (TEST-1 and TEST-3) was approximately 2% which demonstrated good reusability of the steel beam and composite slabs and a similar composite action compared to the original composite beam with cast in-situ concrete. Measured maximum deflections and residual deflections at the end of each load cycle is given in Table 2. The residual deflections at the end of each load cycle grew slightly with the increasing of the load, but was relatively small (up to 7.49 mm at 25 kN, span/800). The mid-span deflections of both specimens reached the serviceability limit of span/300 at a load level of approximated 3.1 times of the design working load. The residual deflection of specimen B1a after the first test was 3.33 mm and was decreased to be negligible after the composite slab was cut into segments and detached from the steel beam.

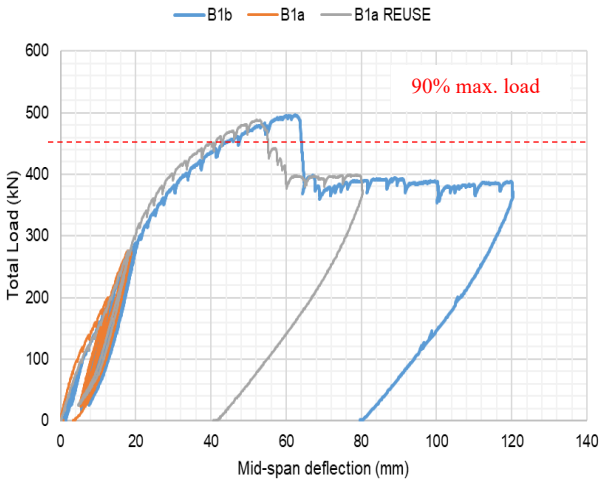


Fig. 9 Total load vs. mid-span deflection

Table 1 Failure load, moment and corresponding deflection of the beams (mm)

Specimen	Failure load kN	Failure moment kN·m	Max. mid-span deflection mm
B1a REUSE	487.9	548.9	52.2 (span/115)
B1b	496.0	558.0	62.4 (span/96)
Difference in resistance = 1.7%			

Table 2 Measured mid-span deflections at the end of each loading cycle excluding self-weight (mm)

Specimen	5 kN/m^2		2.2 × 5 kN/m^2		3.1 × 5 kN/m^2		
	At max.	At 25 kN	At max.	At 25 kN	At max.	At 25 kN	At 0 kN
B1a	/	/	13.20	5.41	18.77	6.45	3.33
B1a REUSE	/	/	/	/	17.95	4.76	/
B1b	5.46	2.85	13.90	6	19.95	7.49	/

Note: deflections listed included initial displacement caused by loading beams – 3.1 kN

4.4.2 End slips

The mean slip capacity measured from end slips of the beam at 90% resistance of the beam was 6.17 mm, as shown in Fig. 10 and Table 3, which suggested a ductile manner of the demountable shear connectors and that the shear connectors have sufficient deformation capacity for the beam to develop composite action and achieve a composite resistance.

The measured maximum end slips and residual slips at the end of each load cycle was up to 2 mm at the maximum load of the cycle (serviceability limit), 1.22 mm at 25 kN and 0.79 mm after unloading, respectively. This implies that the end slips and residual slips of the composite beam in design for first life cycle of use should not beyond these values in order to get a similar structural behaviour for the second use.

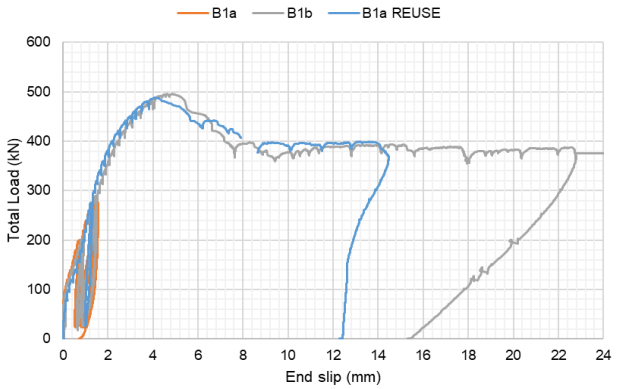


Fig. 10 Total load vs. end slip

Table 3 Measured end slips after the loading cycles and after tests (mm)

Specimen	5 kN/m^2		2.2 × 5 kN/m^2		3.1 × 5 kN/m^2			End slip at 90% max. load
	At max.	At 25 kN	At max.	At 25 kN	At max.	At 25 kN	At 0 kN	
B1a	/	/	1.50	1.06	2.04	1.22	0.79	/
B1a REUSE	/	/	/	/	1.25	0.99	/	5.90
B1b	0.14	0.12	0.92	0.68	1.46	0.86	/	6.43
Average								6.17

4.4.3 Strains

Strains measured at the bottom beam flange and concrete surface for specimen B1a during the first use/ test (Figs. 11 and 12) provided evidence that the beam section and composite slab did not yield during the first use before the reuse test. Negative and positive values indicate compressive strain and tensile strain, respectively.

Total load vs. strains measured at the mid-span of the beam bottom flange for specimens B1a reuse and B1b are plotted in Fig. 13, which shows that the beam yielded before the fracture of the demountable shear connectors. This demonstrates that the demountable shear connectors used have sufficient slip capacity and that their shear resistance can be achieved in the composite beam.

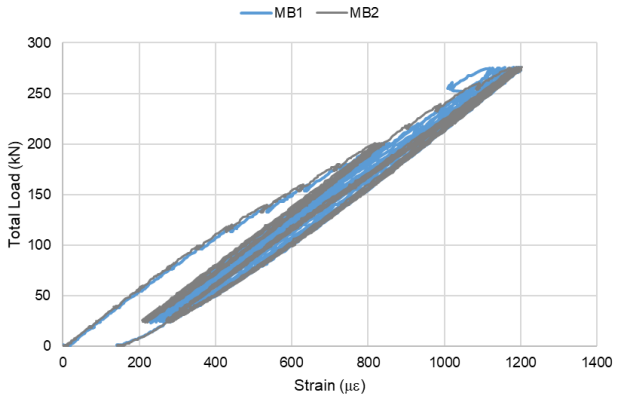


Fig. 11 Total load vs. strain at the bottom beam flange (Mid-span; B1a)

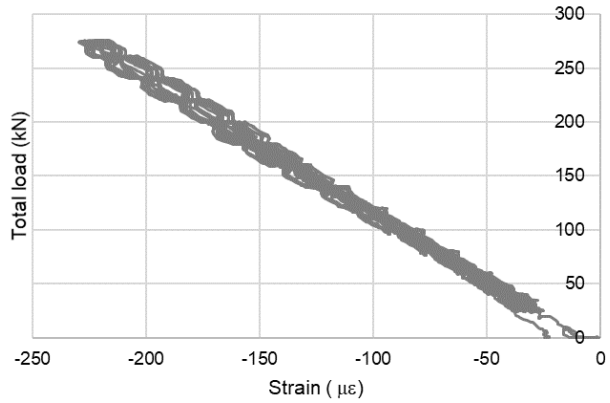


Fig. 12 Total load vs. strain at concrete (B1a at mid-span)

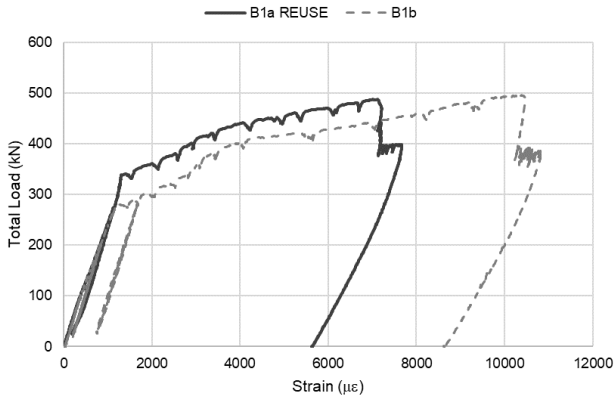


Fig. 13 Total load vs. strain at the bottom beam flange (mid-span; B1a-REUSE and B1b

4.4.4 Observed failures

The final failure of the composite beams was fracture of the shear connectors from beam end up to the first loading point at one shear span, as shown in Fig. 14. No concrete crushing was observed. For the first test of specimen B1a, cracks in concrete slab at loading points were observed at high load e.g. 275 kN; (Fig. 15) nearly half way of the slab depth with measured crack width of 0.01mm. After the 10th cycle between 25 and 275 kN; the specimen was unloading to 0 load. Cracks were propagated to full depth. This could the other reason that caused 2% lower resistance of specimen B1a at the 2nd test (reuse) compared to specimen B1b.

Deflection of the composite beam is presented in Fig. 16 in which plastic deformation of the beam at loading points can also be observed. Although the composite beam experienced large deflection, e.g. up to 120 mm for specimen B1b, the composite slab can still be easily detached after cutting, which can be seen from Fig. 17. This figure also shows intact shear connectors after the test at the one shear span, in contrast to the stud fracture occurred at the other shear span.

Fig. 14 Stud fracture at one shear span (B1b)

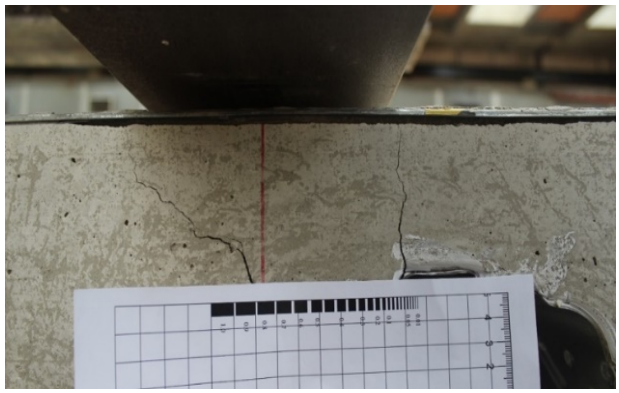


Fig. 15 Concrete cracking at and between loading points (B1b)

Fig. 16 Deflection of beam and slab and yielding of beam section (B1b)

Fig. 17 Studs at the other shear span (no fracture during testing and dismantling; B1b failure test)

5. Remarks on demountability and reusability

After the first round of tests, the composite beams were easily dismantled with the slabs cut into segments. The composite slab cross-section and reclaimed slab segments after cutting is shown in Fig. 18. For specimen B1a, the beam and composite slab segments were reassembled after the first test/use. Segment sizes were planned based on the following concerns: 1) minimum cuts, 2) manageable weight, 3) storage space, and 4) width for future transportation. A total of 4 × 2 cuts was made (10 slab segments) for each specimen. The segment size was approximately 1000 mm × 750 mm for the two mid-span segments, 1210 mm × 750 mm for the four end segments and 1340 mm × 750 mm for the other four segments. The blade of the diamond saw used for cutting was 5 mm in thickness. The gap between adjacent slab segments after cutting was between 5 mm to 7 mm, which was enough for flowable grouting when reconstruct the specimen.

Beam specimen B1a was loaded to serviceability limit and unloaded after 10 cycles between the limit to 5% of failure load. The mid-span deflection of the beam was measured before concreting, and after the slab was cut into segments, and detached from the beam. It was found from the measurements that the beam had no residual deformation, which was align with the stain readings that showed the beam did not yield. The difference in resistance of specimen B1b and reused specimen B1a was approximately 2%, demonstrated a similar composite action between first use and reuse. From the test observations, the grout could transfer in-plane forces and there was no problem to easy separation of the floor units.

For the developed demountable composite floor system, it is possible to re-use both the steel section and the composite slabs, i.e. two cycles of use can be achieved from the experimental evidence. Further cycles of use in composite form or non-composite form may be possible, provided that the shear connection, the composite slab and the steel beam are in good condition. The beams could also be re-used individually. For re-use of salvaged composite floor slabs, the slab segments may be cut to the required length and re-used as precast floor elements with similar composite action (2nd use) or with no (3rd use) composite action. Grout will be used to fill the gap between the slab segments.

Fig. 18 Composite slab section after cutting and slab segments after test

6. Design to Eurocode 4

6.1 Plastic resistance with partial shear connection

For a composite beam with partial shear connection, direction calculation method for the resistance moment of the critical cross-section of the beam, M_{Rd}, has been given in the Chapter 4.2 in SCI publication P428 (Girão Coelho et al. 2020) for bolted shear connectors. In this Section, the simplified method specified in EN 1994-1-1 (CEN 2004) for the determination of M_{Rd} is described and given as:

$$M_{Rd} = M_{pl,a,Rd} + \eta\left(M_{pl,Rd} - M_{pl,a,Rd}\right) \quad (1)$$

where,

$M_{pl,a,Rd}$ is the plastic resistance moment of the steel beam,

$M_{pl,Rd}$ is the plastic resistance moment of the composite section with full shear connection,

η is the degree of shear connection, and is given as $\eta = N_c/N_{c,f}$,

N_c is the compressive force in the concrete flange, and is dependent of the number of shear connectors (n_{sc}) provided between the points of zero and maximum moment and the shear resistance of a single shear connector (P_{Rd}), and is obtained from $N_c = n_{sc}P_{Rd}$,

$N_{c,f}$ is the compressive force in the concrete flange with full shear connection, given by the smaller of the compression resistance of the concrete slab or tensile resistance of the steel beam.

For a composite beam with full shear connection and with profiled composite slab, the compression resistance of the concrete slab ignoring the contribution from the profiled steel sheeting:

$$N'_{c,f} = \frac{0.85 f_{ck} b_{eff} h_c}{\gamma_c} \quad (2)$$

where,

b_{eff} is the effective breadth/width of concrete, taken as the sum of the transverse spacing of the pair shear connectors and beam span over 8 on each side of the web, but not greater than the actual width of the slab,

h_c is the effective area of the concrete flange with a profiled steel decking in compression, and is calculated by $h_c = h_s - h_p$, where h_s is the overall depth of the slab and h_p is the height of the decking profile,

f_{ck} is the characteristic value of the cylinder compressive strength of concrete,

γ_c is the partial factor for concrete.

The design value of the tensile resistance in the steel beam, N_a, can be calculated from the below equation:

$$N_a = \frac{A_a f_{yd}}{\gamma_{M0}} = N_{a,w} + 2N_{a,f} \quad (3)$$

where,

$N_{a,w}$ is the tensile resistance of the web, and is given by $N_{a,w} = N_a - 2N_{a,f}$,

$N_{a,f}$ is the tensile resistance of the flange, and is given by $N_{a,f} = b_f t_f f_{yd}$,

b_f is the width of the flange,

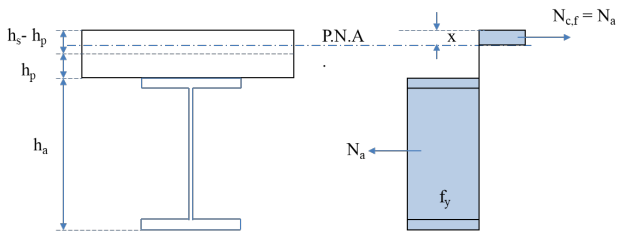
t_f is the thickness of the flange,

f_{yd} is the design strength of the steel beam,

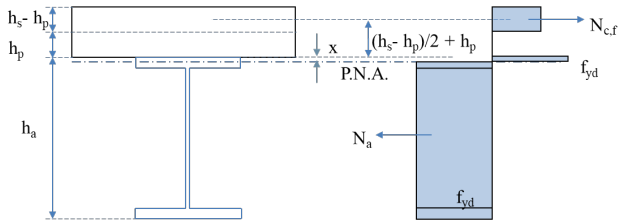
A_a is the cross-section area of the steel beam,

γ_{M0} is the partial factor.

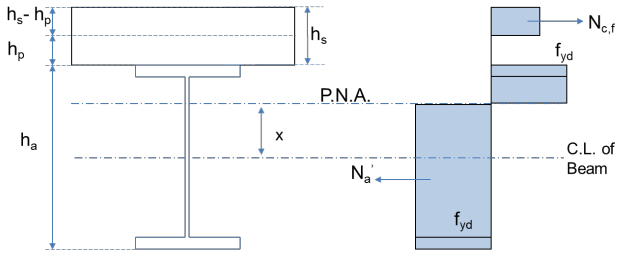
For a composite beam with full shear connection and with a symmetric steel beam, the plastic resistance moment of the composite section, $M_{pl,Rd}$, is classified into three cases according to the positions of the plastic neutral axis (p.n.a) of the composite section. The plastic stress distributions under sagging moment for the different cases are shown in Fig.19



(a) Case 1 ($N'_{c,f} \geq N_a$, p.n.a in the concrete flange)



(b) Case 2 ($N_{a,w} < N'_{c,f} < N_a$, p.n.a in the steel flange)



(c) Case 3 ($N'_{c,f} < N_{a,w}$, p.n.a in the steel web)

Fig. 19 Plastic stress distributions under sagging moment

The distance of x illustrated in Fig. 19 and the plastic resistance moment $M_{pl,Rd}$ are given below:

Case 1 ($N'_{c,f} \geq N_a$, p.n.a in the concrete flange),

$$x = \frac{N_a}{N'_{c,f}}(h_s - h_p) \quad (4)$$

$$M_{pl,Rd} = N_a\left(h_s + \frac{h_a}{2} - \frac{N_a}{N'_{c,f}}\frac{h_s - h_p}{2}\right) \quad (5)$$

Case 2 ($N_{a,w} < N'_{c,f} < N_a$, p.n.a in the steel flange),

$$x = \frac{N_a - N'_{c,f}}{2b_f f_{yd}} \quad (6)$$

$$M_{pl,Rd} = N_a\frac{h_a}{2} + N'_{c,f}\left(h_p + \frac{h_s - h_p}{2}\right) - \frac{\left(N_a - N'_{c,f}\right)^2}{4b_f f_{yd}} \quad (7)$$

Case 3 ($N'_{c,f} < N_{a,w}$, p.n.a in the steel web),

$$x = \frac{N'_{c,f}}{2t_w f_{yd}} \quad (8)$$

$$M_{pl,Rd} = M_{pl,a,Rd} + N'_{c,f}\left[\frac{h_a}{2} + \left(\frac{h_a + h_s + h_p}{2}\right)\right] - \frac{{N'_{c,f}}^2}{4t_w f_{yd}} \quad (9)$$

where h_a is the depth of the steel beam.

6.2 Load-deflections

The imposed load deflection depends on the properties of the composite beam, e.g. the second moment of area of the composite section. The second moment of area of the uncracked section for composite beams with flexible shear connection may be determined as (Lawson et al. 2017, Girão Coelho et al. 2020):

$$I_{y,eff} = I_{y,a} + \frac{I_{y,c}}{n} + \frac{A_c/n}{1 + \frac{A_c}{n\,A_a} + \left(\frac{\pi}{L}\right)^2\left(\frac{E_a}{k_{sc}/s_{sc,eq}}\right)\left(\frac{A_c}{n}\right)}a^2$$

where,

a the centroidal distance between the concrete ab and the steel beam, and is given as $a = \frac{1}{2}h_c + h_p + \frac{1}{2}h_a$,

A_c the cross-sectional area of the concrete slab,

$I_{y,a}$ is the second moment of area of the structural steel section,

$I_{y,c}$ is the second moment of area of the un-cracked concrete section,

L is the span of the composite beam,

n is the modular ratio,

$s_{sc,eq}$ is the longitudinal spacing of the shear connectors.

For a simply supported beam with two equal symmetrical point loads, the deflection under either load is determined from:

$$\delta = \frac{PL_1^2}{6E_a I_{y,eff}}(3L - 4L_1)$$

where,

P is the point load,

L_1 is the distance from the point of zero moment (support) to the nearer point load.

6.3 Comparison with test on short span beam

Dimensions and properties of the composite beam are summarized below:

Span of beam	L	=	6000 mm
Effective slab with	b_{eff}	=	1500 mm
Slab depth	h_s	=	150 mm
Decking depth	h_p	=	60 mm
Beam depth	h_a	=	358 mm
Cross-sectional area	A_a	=	7300 mm^2
Second moment of area	$I_{y,a}$	=	161×10^6 mm^4
Plastic section modulus	$W_{pl,y}$	=	1018×10^3 mm^3
Yield strength of beam	f_y	=	(387.7+442.4)/2= 415 N/mm^2
Modulus of elasticity of steel	E_a	=	205000 N/mm^2
Concrete cube strength	f_{cu}	=	54.7 N/mm^2
Secant modulus of elasticity of concrete	E_{cm}	=	35500 N/mm^2
Resistance of shear stud	P_{Rk}	=	50 kN
Longitudinal spacing of shear studs	s_{sc}	=	333 mm
Stiffness of shear stud	k_{sc}	=	20.5 kN/mm

Calculations for failure load from beam tests ignoring the partial factor are given as follows. From Eqs. (2)-(3), the resistances of the steel section and concrete slab are:

$$N_a = 7300 \times 415 \times 10^{-3} = 3030 \text{ kN} \quad (10)$$

$$N'_{c,f} = 0.85 \times 0.8 \times 54.7 \times 10^{-3} \times 1500 \times (150 - 60) = 5021 \text{ kN}$$

Then Case 1 ($N'_{c,f} \geq N_a$, p.n.a in the concrete flange),

therefore, Eq. (5)

$$M_{pl,Rd} = 3030 \times \left(150 + \frac{358}{2} - \frac{3030}{5021} \times \frac{150-60}{2}\right) \times 10^{-3} = 915 \text{ kN·m}$$

Number of shear stud in shear span $= 2 \times 7 = 14$
Degree of shear connection:
$\eta = N_c/N_{c,f} = 14 \times 50/3030 \times 100\% = 23.1\%$

$$M_{pl,a,Rd} = 415 \times 10^{-3} \times 1018 \times 10^3 \times 10^{-3} = 423\ kNm$$

Using linear interpolation, Eq. (1),

$$M_{Rd} = 423 + 0.231 \times (915 - 423) = 536.7 \text{ kN·m}$$

Then the total failure load $= 536.7/2.25 \times 2 = 477$ kN,

compared against the failure load, 496 kN, for specimen B1b, the error is 4%.

Calculations for load deflections are provided as follows. Other data: equivalent spacing of single shear connectors, $s_{sc,eq}$ = 161 mm (shear span 2.25 m with 14 shear studs).

Using Eq. (10),

$$I_{y,eff} = 161 \times 10^6 + \frac{1500 \times 90^3}{12 \times 6.0} + \frac{(135/6.0) \times 10^3}{1 + \frac{135}{6.0 \times 7.3} + \left(\frac{\pi}{6000}\right)^2 \left(\frac{205 \times 161}{20.5}\right)(135/6.0) \times 10^3} \left(\frac{90}{2} + 60 + \frac{358}{2}\right)^2 = 306 \times 10^6 \text{ mm}^4$$

From Eq. (11), the deflection

$$\delta = \frac{496/2 \times 10^3 \times 2250^2}{6 \times 205000 \times 306 \times 10^6}(3 \times 6000 - 4 \times 2250) = 30 \text{ mm}.$$

Plot total load of 496kN versus deflection of 30mm, against the beam test result of specimen B1b, see Fig. 20. It can be seen that the calculations on load deflections have reasonably accuracy up to the serviceability limit of 20mm (span/300).

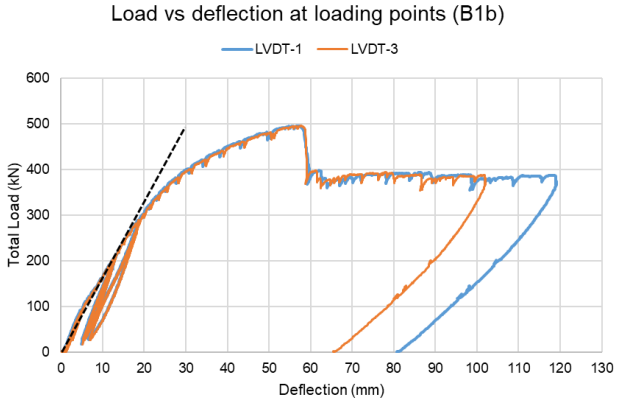


Fig. 20 Total load vs. deflection at loading points

7. Conclusions

A series of beam tests were carried out to investigate the behaviour of composite beam with demountable shear connectors. Comparable results were obtained from the first-use composite beam and the composite beam with reused steel beam and composite slab segments. The less than 5% differences in load resistance could because of cast in-situ vs. precast, further analysis is currently on-going. Testing of the demountability and re-assemblability showed that the flooring system can be easily reused without additional erection tolerances. The degree of shear connection of the tested beams with demountable shear connectors was 23.1 %, much smaller than the lower bound of 40% specified in EN 1994-1-1 (CEN2004) for welded shear studs, the slip capacity at 90% of beam resistance was 6 mm in average and fulfilled the ductility requirement. This finding demonstrates the benefits of unpropped composite construction, in terms of reduced slip requirement and lower degree of shear connection. Comparisons were made between hand calculations and the beam test; reasonably accuracy was obtained.

Acknowledgments

This research is part of a research project jointly investigated by the University of Bradford and the University of Manchester. The project is funded by the Engineering Physical Sciences Research Council (EPSRC) and Structural Metal Deck Ltd, their financial supports are greatly acknowledged.

References

Dai, X.H., Lam, D. and Saveri, E. (2015), "Effect of concrete strength and stud collar size to shear capacity of demountable shear connectors", Journal of Structural Engineering, American Society of Civil Engineers, 141(11), 04015025. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001267

EN1994-1-1: Eurocode 4 – Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings. Brussels, Belgium: European Committee for Standardization (CEN); 2004.

Girão Coelho, A. M., Lawson, M., Lam, D., Yang, J. (2020), Guidance on demountable composite construction systems for UK practice (P428), The Steel Construction Institute, Ascot, UK. ISBN 13: 978-1-85942-245-8.

Lam, D., Dai, X., Ashour, A. et al. (2017), "Recent research on composite beams with demountable shear connectors", Steel Construction: Design and Research, 10(2), 125 –134. https://doi.org/10.1002/stco.201710016

Lawson, R. M., Lam, D., Aggelopoulos, E. S., Nellinger, S. (2017), ''Serviceability performance of steel–concrete composite beams'', Proceedings of the Institution of Civil Engineers - Structures and Buildings,170:2, 98-114. https://doi.org/10.1680/jstbu.16.00048

Liu, X., Bradford, M.A. and Lee, M.S.S. (2014), "Behavior of High-Strength Friction-Grip Bolted Shear Connectors in Sustainable Composite Beams", Journal of Structural Engineering, 141(6):04014149 https://doi.org/10.1061/(ASCE)ST.1943-541X.0001090

Moynihan M.C. and Allwood J. (2014), Viability and performance of demountable composite connectors. Journal of Constructional Steel Research 2014; 99, 47 –56. https://doi.org/10.1016/j.jcsr.2014.03.008

OECD (2018), Global Material Resources Outlook to 2060: Economic Drivers and Environmental Consequences, OECD Publishing, Paris. https://doi.org/10.1787/9789264307452-en

Pavlović M., Marković Z., Veljković M. and BuĎevac D. (2013), "Bolted shear connectors vs. headed studs behaviour in push-out tests", Journal of Constructional Steel Research; 88, 134 –149. https://doi.org/10.1016/j.jcsr.2013.05.003

Pathirana, S.W., Uy, B., Mirza, O. and Zhu, X. (2016), "Flexural behaviour of composite steel–concrete beams utilising blind bolt shear connectors", Engineering Structures, 114, 181-194. https://doi.org/10.1016/j.engstruct.2016.01.057

Rehman N, Lam D, Dai X and Ashour A (2016), "Experimental study on demountable shear connectors in composite slabs with profiled decking", Journal of Constructional Steel Research, 122, 178–189. https://doi.org/10.1016/j.jcsr.2016.03.021

Rehman N, Lam D, Dai X and Ashour A (2017), "Testing of composite beam with demountable shear connectors", Proceedings of the Institution of Civil Engineers - Structures and Buildings, 171(1), 3 –16. https://doi.org/10.1680/jstbu.16.00172

Sheehan, T., Dai, X. and Lam, D. (2016), "Experimental study on long spanning composite cellular beam under flexure and shear." Journal of Constructional Steel Research, 116, 40 –54. https://doi.org/10.1016/j.jcsr.2015.08.047

Suwaed, A.S.H. and Karavasilis, T.L. (2017), "Novel Demountable Shear Connector for Accelerated Disassembly, Repair, or Replacement of Precast Steel-Concrete Composite Bridges", Journal of Bridge Engineering, American Society of Civil Engineers, 22(9):04017052. https://doi.org/10.1061/(ASCE)BE.1943-5592.0001080

Suwaed, A.S.H. and Karavasilis, T.L. (2018) "Removable shear connector for steel-concrete composite bridges", Steel and Composite Structures, 29(1):107-123. https://doi.org/10.12989/scs.2018.29.1.107

Uy, B., Patel, V., Li, D. and Aslani, F. (2017), "Behaviour and Design of Connections for Demountable Steel and Composite Structures", Structures, 9, 1-12. https://doi.org/10.1016/j.istruc.2016.06.005