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ARTICLE

Demountable composite beams for a circular economy: Large-scale beam tests

The circular economy approach has been introduced to the construction sector to design out waste, reduce carbon emissions and energy use in production of primary materials and achieve resource efficiency. 'Design for deconstruction' is increasingly important for a sustainable economy, especially for composite structure that is deemed as resource-efficient in the first cycle of use. With this background, research has been conducted to facilitate the use of demountable composite beams in steel-framed structures. This article presents the results of two 6 m full-scale composite beam tests with prefabricated composite slabs. Demountable shear connectors were used, which have been newly developed within the frame of an EU project. The nominal degree of shear connection of the composite beam specimens was 0.37, which was slightly below the required 0.4 specified in EN1994-1-1 for welded shear studs. Based on the experimental observations, it can be demonstrated that (1) the tested demountable composite beam produced higher resistance but lower stiffness values than a comparable reference composite beam with welded shear studs; (2) the beam specimens showed a good plastic behaviour with high ductility despite the non-ductile nature of the applied demountable shear connectors. Remarks on the demountability of the developed demountable composite beams are provided.

Keywords circular economy; composite structures; demountable composite beam; demountable shear connectors; experimental tests

1 Introduction and objectives

The concept of circular economy has recently been implemented in the construction sector in the EU and worldwide driven by the increasing pressures in sustainability and reduction of greenhouse gas emissions. One main principle of the circular model is to keep the products and materials within the use cycle. Drastic reduction in carbon emissions, energy use and waste can be made if building products are reclaimed from end-of-life structures and reused repeatedly in future structures. 'Design for deconstruction' is a key strategy to encourage and maximise the reuse when a building structure comes to demolition and is particularly important for composite structure that is deemed as resource-efficient in the first cycle of use. A lot of its attributes make the steel an ideal material for a circular economy. Thus, steel-based composite structures are of great interest and potential for deconstruction and reuse.

In steel-framed structures, current design for composite flooring systems is to connect the steel beam permanently with the floor slab through shear studs that are welded to the top flange of the beam. This design is not favourable from a demountability point of view. Bolted shear connectors for such flooring systems make the composite beams demountable and thus the steel beams and the floor slabs reclaimable in their entirety. Dallam [1] and Marshall [2] already investigated high-strength bolted shear connectors nearly five decades ago. However, the respective research on demountable shear connectors is still very limited compared to welded studs. Recently, a few types of demountable shear connectors have been proposed and investigated through standard push tests, beam tests and numerical simulations to assess the shear resistance, slip capacity of the connectors and composite action and behaviour of composite beams. Lam and Dai [2] developed a demountable shear connector that was machined from a traditional shear stud by cutting threads into the end. The performed push-out tests showed a high level of ductility of the shear connectors but a relatively low stiffness compared to welded studs. The stiffness of the demountable shear studs was later improved by introducing an additional nut above the top of the steel flange and embedded in concrete. Demountability of the flooring system using the developed studs and the reusability of the steel beam and floor slab were demonstrated by full-scale composite beam test by comparing first use with second use [4]. Further studies investigated high-strength bolted connections [5–10] and blind bolts [11–13]. Nijgh et al. [14] investigated resin-injected bolt-coupler shear connectors to reduce the effect of bolt clearance and to optimise the beneficial effect of composite action.

Among the aforementioned pieces of research work, the project REDUCE (funded by the Research Fund for Coal and Steel (RFCS) of the European Commission) [15] has provided a holistic approach and guidance on the design for deconstruction of steel-concrete composite structures in commercial and residential buildings. A respective design guidance on the demountable composite construction systems for UK practice [16] has been published based on the findings of REDUCE. One objective of the research work was to assess the composite action of the newly developed demountable shear connectors in push-out and full-scale beam tests, observe the modes of failure and obtain the respective structural behaviour at ultimate limit state. The demountable shear connection also allows for the replacement of bolts, should any bolt be damaged. Two experimental tests on full-scale demountable composite beams are presented. Remarks on the demountability of the system are also provided.

2 Overview of the experimental campaign

In the frame of the RFCS research project REDUCE [15], 15 push-out tests were conducted on different types of demountable shear connections at the University of Luxembourg. The push-out tests were then followed by two full-scale beam tests on demountable composite beams. Beam B7 had the dismountable shear connection type P3.3 and beam B8 had type P15.1 (details given in the following sections). The test setup of the demountable composite beams was chosen to be as close as possible to the test setup of a reference beam. The reference beam was the specimen 2-10 tested within the RFCS research project DISCCO [17]. It had a trapezoidal steel sheeting ComFlor® 80 [18] and a 150 mm thick concrete slab. The degree of shear connection was $\eta = 0.32$. The investigated shear connections are described in Section 3, and the conducted beam tests and the corresponding results are presented in Section 4.

3 Push-out tests and the behaviour of the applied shear connections

3.1 Shear connection tested in the RFCS-DISCCO project

Within the RFCS research project DISCCO [17], the behaviour of shear stud connections in composite beams with deep decking was investigated. Nellinger [19] conducted several push-out tests and two full-scale beam tests. The push-out tests that represent the shear connections applied in beam 2-10 were denoted as 3-01-3. This test had eight shear studs with a diameter of 19 mm and a height of 125 mm (Köco® SD 19×125 [20]) placed in 160 mm deep concrete decks with ComFlor® 80 [18] profiled steel sheeting. The studs were welded through the profiled sheeting. Fig. 1 shows the corresponding load-slip curve. The tensile strength of the stud material was $f_u = 550$ MPa. The measured shear connection resistance was 53 kN/shear connector. The measured concrete cylinder strength was $f_{cm,e} = 40.4$ MPa.

3.2 Demountable shear connectors tested in the RFCS-REDUCE project

A number of push-out tests were carried out on demountable shear connectors within the frame of the REDUCE project [15], and the experimental results were presented by Kozma et al. [21]. Based on the experimental observations of these tests, two shear connection variants have been selected and implemented in the beam specimens B7 and B8 with prefabricated deck elements [22]. As explained in Section 2, specimen B7 used shear connection type P3.3 (Fig. 2a). This connection provides accessibility from the top of the slab through pockets in the concrete. Specimen B8 used shear connection type P15.1 (Fig. 2b), which uses an embedded coupler device, an embedded

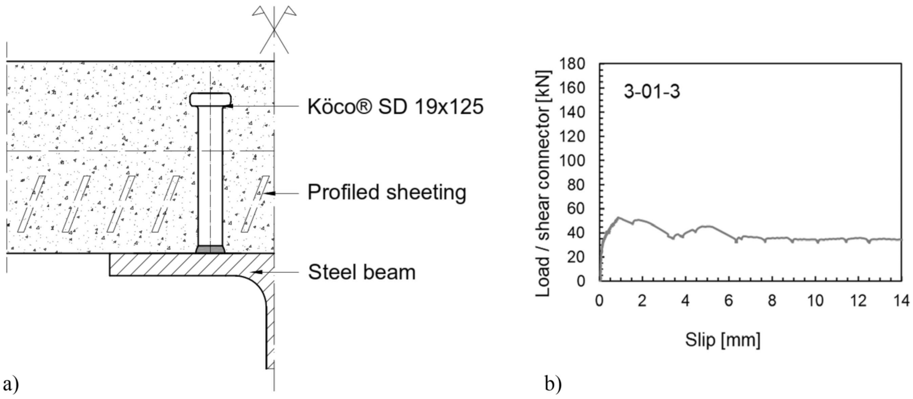


Fig. 1 a) Shear connection applied in beam 2-10 [19]; b) the corresponding load-slip curve

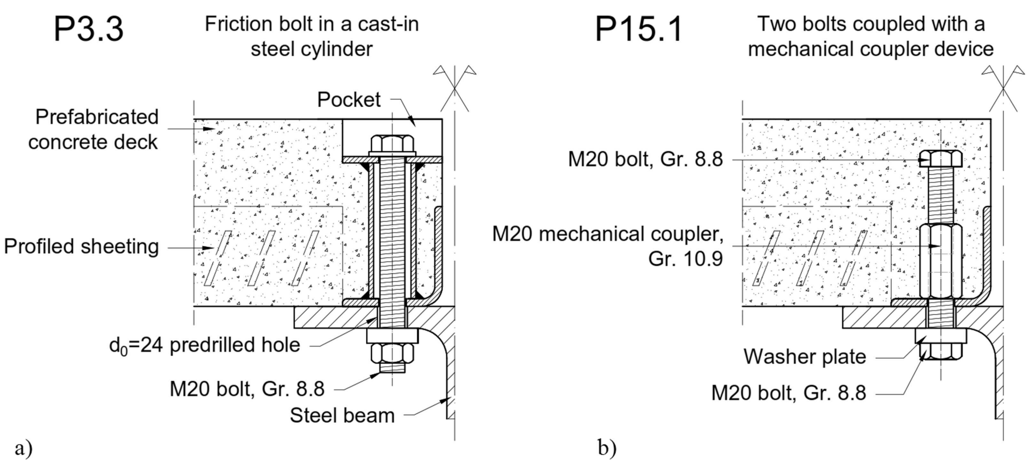


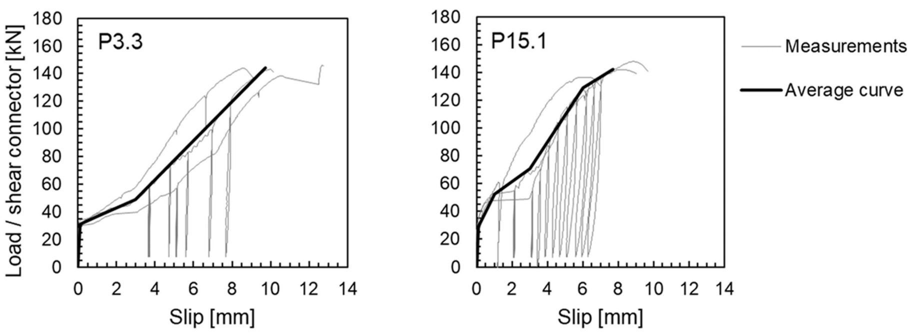
Fig. 2 Tested shear connectors: a) system P3; b) system P15

bolt and a bolt placed from underneath the top steel flange. The coupler is of grade 10.9 and the bolts are of grade 8.8 steel. Both shear connection system utilise friction in the early load stages and bolt bearing afterwards. The corresponding load-slip curves are presented in Fig. 3.

4 Beam tests

4.1 Reference beam test 2-10 of the RFCS-DISCCO project

Nellinger [19] explored the behaviour of composite beams with low degree of shear connection within the frame of the RFCS project DISCCO [17]. Test specimen 2-10 was selected as a reference case for the composite beams with demountable shear connectors presented in this article as beams B7 and B8. It had an IPE 360 steel beam, a 150 mm thick and a 1500 mm wide composite slab with ComFlor® 80 metal sheet decking and 19 mm diameter headed studs as shear connection with a nominal height of 125 mm (Köco® SD 19×125 [20]) welded, as usually in the building industry, through the metal decking. It contained 10 pairs of shear connectors placed on the half-length of which 8 pairs were placed within the shear length, defined between the support and the closest load application point in the two-point loading test. The longitudinal and the transversal spacing of the connectors were 300 and 100 mm, respectively. The shear resistance of the welded shear studs was 53 kN per stud obtained by push-out tests. For the beam specimen, the measured concrete cylinder strength was $f_{cm,e} = 49.0$ MPa, and the cube strength was $f_{cu,m,e} = 57.6$ MPa. The resulting degree of shear connection on the basis of EN 1994-1-1 [23] for the tested composite beam was 0.32, which was below the required minimum degree of shear connection of 0.4 for welded shear connectors. Fig. 4 presents the schematic view of the test setup and Fig. 5 shows pictures of the test specimen.



a) b)

Fig. 3 Load-slip curves of the tested shear connectors: a) system P3.3; b) system P15.1

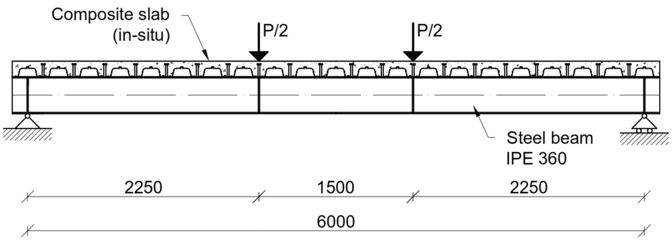
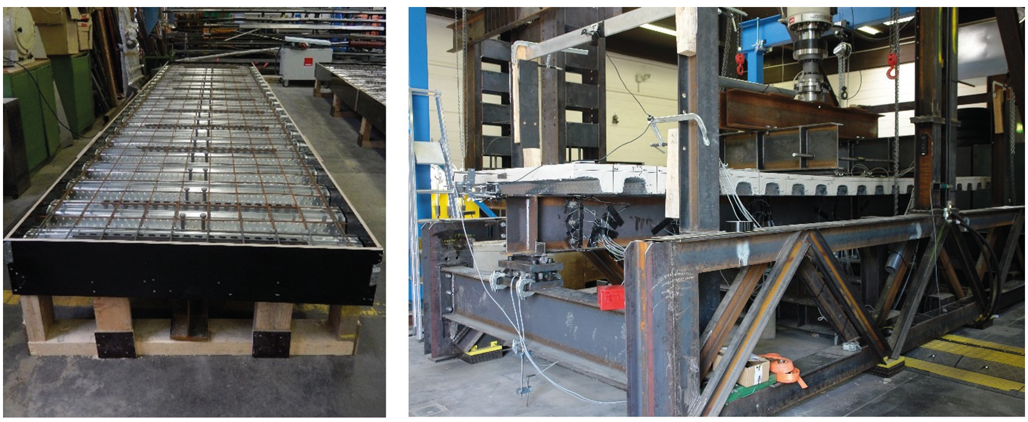


Fig. 4 Schematic view of beam 2-10 [19]

4.2 Demountable beam tests B7 and B8 of the RFCS-REDUCE project

4.2.1 Geometry and material properties

Compared to the beam tested in the RFCS project 'DISCCO', each specimen in this project had a 6.3 m long IPE



a) b)

Fig. 5 Beam 2-10: a) specimen before concreting; b) specimen in the testing frame

360 steel beam with a grade of S355 and two pre-fabricated composite slab elements with a depth of 150 mm and a width of 790 mm using ComFlor® 80 [18] metal sheet decking. The clear span of the beams was 6 m. The total width of the specimens was 1600 mm (see Fig. 6). This width corresponds to the effective width defined by Eurocode 4 [23]. Five pairs of shear connectors were placed within the half-length, among which four pairs were located within the shear length. The transversal spacing (b_0) of the connectors was 100 mm and a longitudinal spacing was 600 mm. Note that the shear connectors were placed in solid strips where there was no metal decking above the top flange of the steel beam (see Fig. 6).

The slab elements were stabilised with diagonal struts so that no tension force arose in the shear connectors from the self-weight of the composite slabs. The diagonal struts were not connected to the slabs by any mechanical connectors. They only provided a vertical support of the deck elements; therefore, they did not influence the longitudinal flexural behaviour. A cross-sectional view of the composite beam is illustrated in Fig. 6. The beam configurations and material properties are summarised Tab. 1.

The beams were designed for a degree of shear connection of 0.37. However, it is important to note that during the design process the actual material parameters were unknown, so the degree of shear connection was determined using the 'expected' values of the materials: the yield strength of the structural steel $f_{y,m} = 394$ MPa was assumed on the basis of the JCSS Probabilistic Model Code [24], and the concrete strength $f_{cm} = 43$ MPa was determined using Eurocode 2 [25]. The calculated $\eta = 0.37$ value corresponds to a uniform spacing of 600 mm in pairs. In the DISCCO project [17] beam 2-10, the degree of shear connection was 0.32.

4.2.2 Fabrication and assembly

The formworks for the deck elements were fabricated from plywood for assembly, placed on standard European pallets. The solid strips were created by placing profiled foam fillers between the base plate of the formwork and the holes under the crests of the profiled steel sheeting. L-profiles with pre-drilled holes were placed between one end of the profiled sheeting and the side wall of the formwork.

In specimen B7, expanded polystyrene (EPS) foam blocks were glued to the top plate of the shear connectors

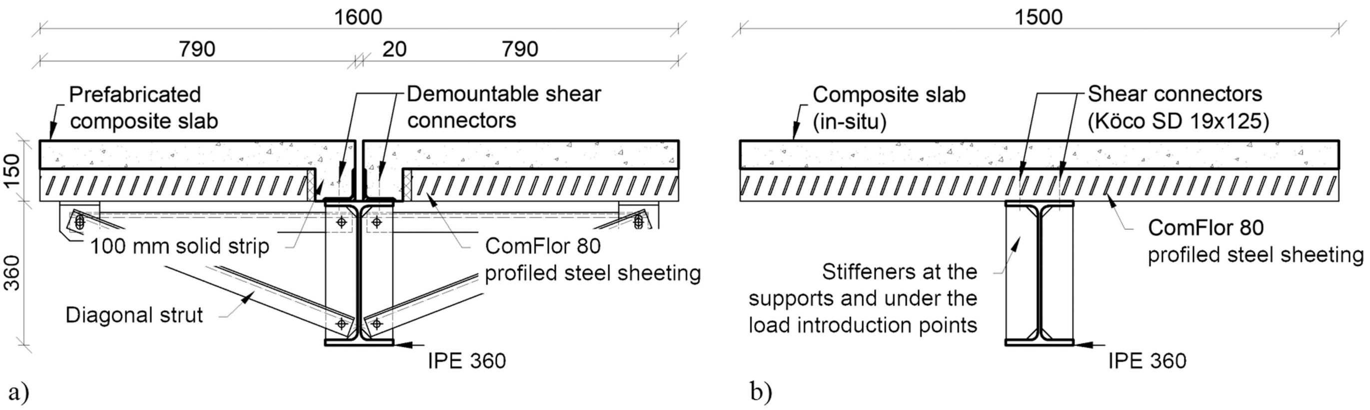


Fig. 6 Cross sections of the tested beams: a) REDUCE (B7 and B8); b) DISCCO (2-10)

Tab. 1 Tested beam configurations and materials properties

Test no.	Clear span, *L*	Section	Slab	Shear connection type	Spacing of connectors
2-10	6 m	IPE 360 S355 ($f_{y,e} = 382$ MPa)	Cast-in-situ, CF80, $d = 150$ mm C35/45 ($f_{cu,m,e*} = 49.0$ MPa) a)	Köco SD 19×150 ($f_u = 550$ MPa)	2 Ø 19 mm 300 mm
B7	6 m	IPE 360 S355 ($f_{y,e} = 382$ MPa)	Precast, CF80, $d = 150$ mm C35/45 ($f_{cu,m,e} = 64$ MPa)	P3.3 Through bolts Gr. 8.8 ($f_{u,e} = 949$ MPa)	2 Ø M20 bolt 600 mm
B8	6 m	IPE 360 S355 ($f_{y,e} = 382$ MPa)	Precast, CF80, $d = 150$ mm C35/45 ($f_{cu,m,e} = 64$ MPa)	P15.1 Coupled bolts Gr. 8.8 ($f_{u,e} = 969$ MPa)	2 Ø M20 bolt 600 mm

a) Calculated from the measured cylinder strength ($f_{cu,m,e*} = 0.85\, f_{cm,e}$)

in order to create pockets in the concrete (see Fig. 7a). The EPS blocks were removed after the concrete had hardened.

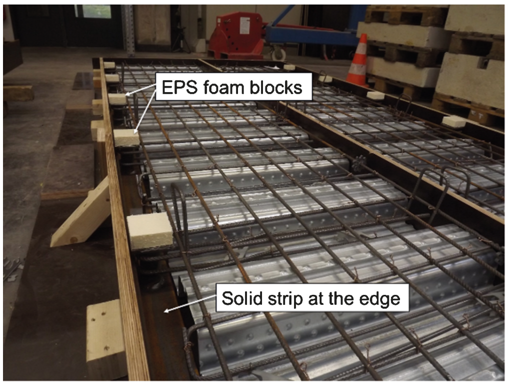
In specimen B8, holes were drilled into the base plate of the formwork through the pre-drilled holes of the L-profiles. Then, the mechanical coupler device with the embedded bolt was placed above the holes and dummy bolts were placed from below the base plate of the formwork into the coupler device to keep them in place during concreting (Fig. 7b).

One layer of Ø 8/95/135, B500 B reinforcement mesh was placed on the top of the profiled steel sheeting (see Fig. 8a). Additionally, two layers of Ø 10 U-bars were placed around the shear connectors (see Fig. 7b). The composite deck elements were fabricated in 6 m long pieces. This means that no transversal joints were applied. However, in practice it is possible that the application of transversal joints is unavoidable. Lam et al. [4] showed that grouted joints can be applied for the transversal connection of the deck elements without compromising the flexural behaviour. Both beams were cast on the same day from the same concrete mixture. After the concrete hardened, the slabs were lifted and placed on the top of the steel beam. For each deck element, it was necessary to align the holes of the concrete deck and the steel beam. Due to the L-profiles at the edge with pre-drilled holes (see Fig. 8b), low tolerances could be achieved and the hole alignment was successful.

The composite slab elements were fixed to the beam with M20 high-strength bolts through 24 mm diameter pre-drilled holes in the top flange of the beam. The beams were continuously supported along the total length during the assembly, which corresponds to a propped construction method. In both cases, the bolts were tightened by rotating the bolt head using a pneumatic impact wrench. Direct tension indicator (DTI) washers (see Fig. 9) were applied in accordance with the requirements of EN 1090-2 [26] in order to control the pre-tension force.

4.2.3 Loading protocol and instrumentation

The tests were conducted using a 1000 kN capacity hydraulic jack. Two-point loading was applied to the beam (Fig. 10) using a spreader beam, and two additional beams with steel rods welded on their bottom flange. Steel plates and neoprene layers were placed under the load application rods in order to counteract the uneven and rough surface of the concrete and equilibrate the force distribution on the surface to prevent local failure of the concrete due to the high concentrated forces. Both the steel and the neoprene layers had a thickness of 10 mm and a width of 100 mm.

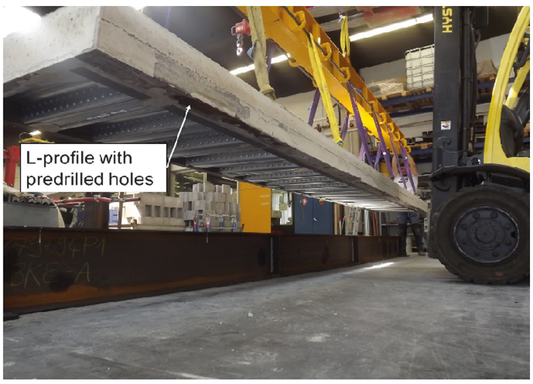


a)

b)

Fig. 7 Photographs of the test specimens: a) beam B7 with EPS foam blocks; b) beam B8 with the coupler device, which is kept in place by a dummy bolt from below the formwork

a)



b)

Fig. 8 Photographs of the test specimens: a) beam B7 before concreting; b) beam B8 during assembly

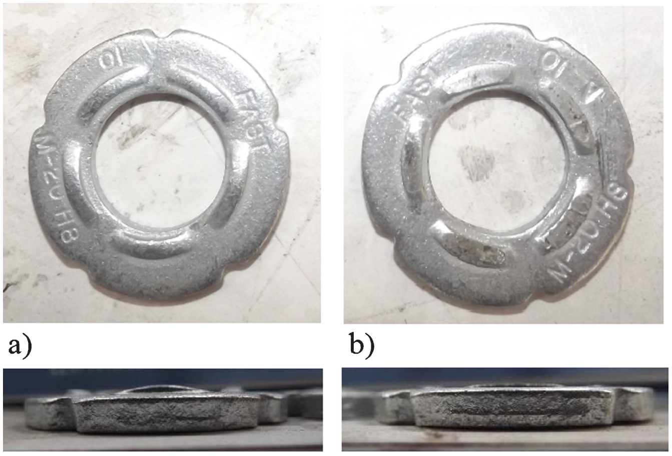


Fig. 9 DTI washers: a, c) before usage; b, d) after usage

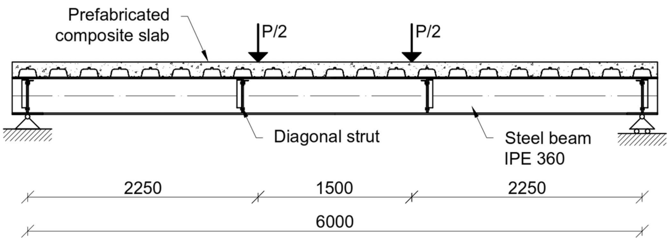


Fig. 10 Schematic view of the beam tests

The loading protocol consisted of two parts. Firstly, 25 cycles were conducted between 5 % and 40 % of the expected failure load P_{exp}. Secondly, an incremental cyclic loading was performed: five cycles were performed after every 10 % load increment followed by a monotonic loading until failure. The loading was held on the upper level for 5 minutes after each five cycles for short-term relaxation. The expected failure load was pre-determined using the non-linear finite element software Abaqus® [27]. The cycles were performed using the force-controlled mode, while the monotonic loading was applied using displacement-controlled mode.

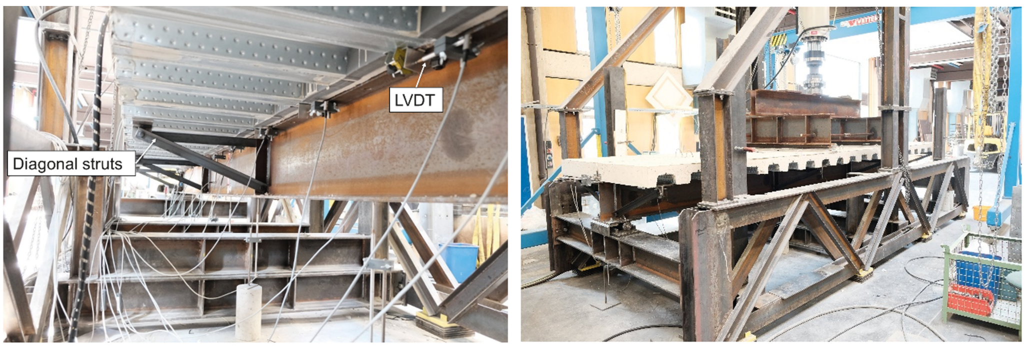
For each specimen, 28 displacement sensors (LVDTs) were applied to measure the end slip of each composite slab, the slip values at the shear connectors, the transverse separation of the slabs relative to the beam and the deflection values of the beam in different positions (see Fig. 11).

Additionally, one inclinometer was applied on the beam and 10 linear strain gauges: 4 on the web of the beam and 6 on longitudinal reinforcement bars inside the slabs. Furthermore, the travel and the force values of the hydraulic jack were continuously monitored during the tests. Additionally, compression tests on concrete cube specimens and uniaxial tensile tests on steel coupon specimens were carried out to obtain information about the properties of the applied materials. According to these tests, the mean values of the concrete cube strength and the steel yield strength were $f_{cu,e,m} = 64$ MPa and $f_{y,e,m} = 382$ MPa, respectively.

4.2.4 Experimental results

The measurements started after the assembly of the test setup. Therefore, the self-weight of the composite beam is not included in the measured data. The calculated self-weight deflection is 0.77 mm. The load-deflection curves of specimen B7 and B8 are presented in Fig. 12 and 13. Beam test B7 failed at a total load level of 600 kN by concrete crushing. This load level corresponds to a bending moment of 675 kN·m. The mid-span deflection at failure was 198 mm ($L/30$). Cracks appeared above the crests of the profiled sheeting due to tensile stresses arising from bending. The shear connectors did not fail during the tests; however, shear deformation of the bolts and thread penetration in the steel beam were observed. During dismantling, all bolts could be removed from the specimen.

The failure of specimen B8 occurred by shear connector failure at a load level of 569 kN. The corresponding bending moment capacity is 640 kN·m. At this load level, the deflection of the beam was 133 mm ($L/45$). After the first shear connector had failed, the deflection was further increased and the failure of the subsequent shear connectors occurred. This mechanism is represented by the stepwise dropping of load on the load-deflection curve. Finally, concrete crushing took place. Thread penetration could be observed in the holes of the steel profile, similar to the case of test B7. The concrete slab cracked around the outermost shear connectors, above the crests of the sheeting and longitudinally, parallel to the inner edge. Fig. 14 shows the tested beams under failure conditions



a) b)

Fig. 11 Beam B8 in the testing frame; a) view on the struts, b) view of setup

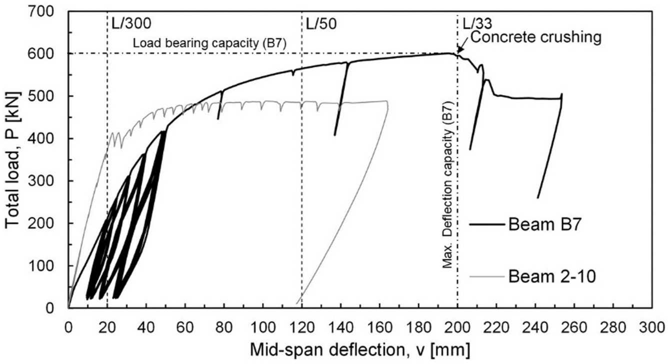


Fig. 12 Experimental load-deflection curves of tests B7 (P3.3, dismountable) and 2-10 (Köco SD 19×125 welded connection)

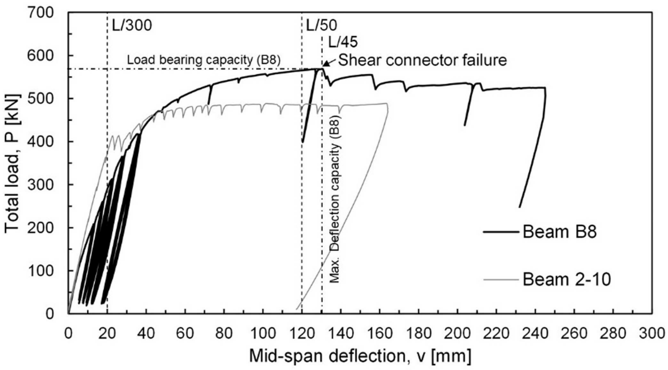


Fig. 13 Experimental load-deflection curves of tests B8 (P15.1, dismountable) and 2-10 (Köco SD 19×125 welded connection)

and Fig. 15 presents photographs of the observed damages.

Both tested beams produced higher resistance than the similar beam tested in the frame of the DISCCO project [17] (beam 2-10). Both beams were loaded up to failure and the deflection was further increased until ~220 mm (L/27). After the tests, the specimens were disassembled. The disassembly process required less time and effort than in the case of standard composite beams, where welded studs provide the shear connection resulting in the separation of the materials in standard composite beams requiring a large amount of cutting. The concrete slab elements could be separated from the steel beam using a normal wrench. Hence, the demountability of the specimens was proven.

4.3 Discussion of the results and observations

4.3.1 Degree of shear connection

The stress block method of Eurocode 4 [23] enables to calculate the bending capacity of the composite section with partial shear connection based on plastic theory. The method assumes that the developing compression force in the concrete is ηN_{cf}, where η is the degree of shear connection. In the case of the test, the bending capacity can be calculated from the equilibrium with Eq. (1). Using the aforementioned method, the degree of shear connection was calculated backwards from (1) the measured bending moment capacity (M_{ult}), (2) the calculated bending capacity of the steel beam alone ($M_{pl.a}$) and (3) the calculated bending capacity of the composite section in the case of full shear connection ($M_{pl.f}$).

Tab. 2 summarises the determined experimental degree of shear connection for both of the tested specimens and the beam tested within the frame of the DISCCO project [17]. In this table, P_{ult} and M_{ult} also include the self-weight of the beams, which means an additional 37 kN force and 28 kN·m bending moment to the measured values.

$$M_{\mathrm{ult}} = \frac{P_{\mathrm{ult}}}{2} \cdot 2.25\,\mathrm{m} \tag{1}$$

4.3.2 Stiffness and degree of interaction

The degree of interaction ψ is governed by the stiffness of the shear connection. If there is no shear connection the degree of interaction is zero, while in case of a perfectly rigid shear connection its value is one. The load level at which the beam would reach this deflection ($P_{L/300}$) was calculated using elastic theory for both extreme cases ($\psi = 0$ and $\psi = 1$). The actual degree of interaction was determined using linear interpolation:

$$\psi = \frac{P_{\mathrm{L/300}} - P_{\mathrm{L/300},\psi=0}}{P_{\mathrm{L/300},\psi=1} - P_{\mathrm{L/300},\psi=0}} \tag{2}$$

Tab. 3 summarises the total load at the deflection of L/300 ($P_{L/300}$) and the corresponding stiffness values ($S_{L/300}$). As shown, both tested demountable composite beams

a) b)

Fig. 14 The tested beams under failure conditions: a) beam B7; b) beam B8

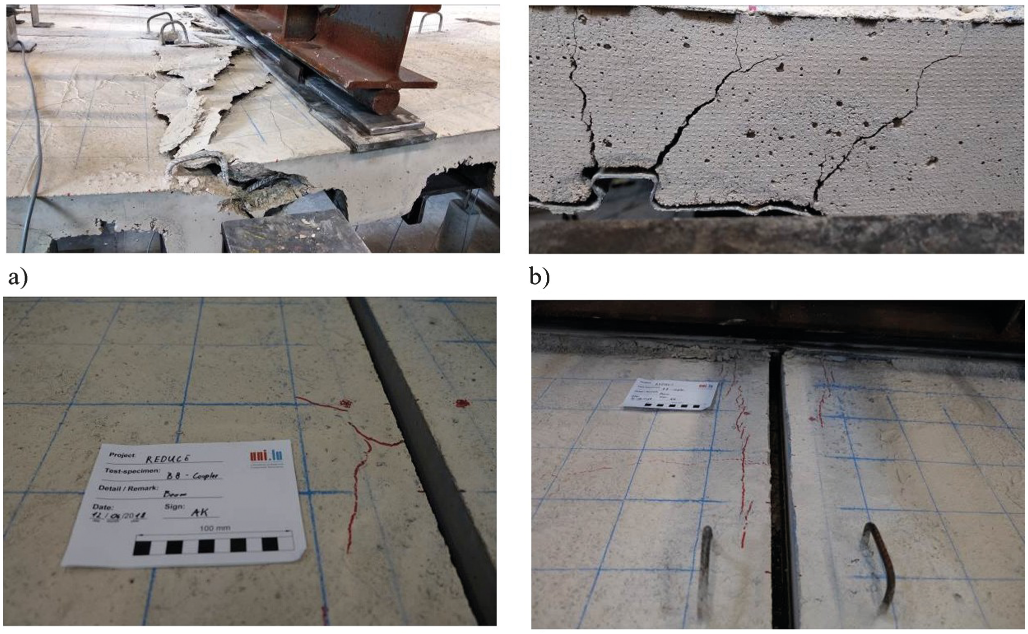


Fig. 15 Observed damages: a) concrete crushing in beam B7; b) bending cracks in beam B7; c) cracks around the shear connectors of beam B8; d) longitudinal cracks in beam B8

Tab. 2 Load-bearing capacity and degree of shear connection of the tested beams and comparison of capacity

Test no.	Shear connection	P_{ult} [kN]	M_{ult} [kNm]	$M_{pl.a}$ [kNm]	$M_{pl.f}$ [kNm]	η [a] [−]
2-10	Welded stud	494	542	372	825	0.32
B7	P3.3	637	703	382	861	0.50
B8	P15.1	606	668	382	861	0.41

Test no.	Shear connection	Capacity at L/50 [kN]	2/3 Capacity at L/50 [kNm]	Deflection at 2/3 capacity at L/50 [mm]	Comparison to 2-10 [%]
2-10	Welded stud	488	325	16	100
B7	P3.3	603	414	40	250
B8	P15.1	602	413	29	181

[a] η analysed based on the beam test results

Tab. 3 Stiffness of the tested beam configurations

Test no.	Shear connection	$P_{L/300}$ [kN]	P_{El} [kN]	$S_{L/300}$ [kN/mm]	$P_{L/300,\psi=0}$ [kN]	$P_{L/300,\psi=1}$ [kN]	ψ [−]	η [−]
2-10	Welded stud	319	409	16.0	153	584	0.34	0.32
B7	P3.3	210	389	10.5	173	596	0.10	0.50
B8	P15.1	289	405	14.5	173	596	0.27	0.41

had lower stiffness than the beam with welded studs tested in the DISCCO project [17]. In the case of welded studs, the degree of interaction is similar to the degree of shear connection ($\psi = 0.34$ vs. $\eta = 0.33$). This is not the case for the demountable beams (B7 and B8), where the degree of interaction is relatively low despite the higher values of η. This means that in the design of demountable composite beams, special attention is required for the occurring deflections.

4.3.3 Demountability

The demountability and reusability of the tested systems had already been proven during the push-out tests [21]. Therefore, the specimens were not disassembled during the beam tests. After the beam tests – by when the steel beam, the concrete slab and the shear connectors had all been highly deformed (see Fig. 16) – the specimens were disassembled with standard hand tools. Hence, the demountability was proven again. No signs of cracks, damages or plastic deformations were observed before reach-

Fig. 16 Specimen B7 (right) and B8 (left) after removing them from the testing frame

ing $L/300$ deflection, which corresponds to the serviceability limit. This indicates that beams that do not observe loads beyond the serviceability limit state remain reusable.

5 Conclusions

In the frame of the presented research, two demountable composite beams were investigated using different types of demountable shear connections. Based on the experimental investigations, the following conclusions could be drawn:

1. Demountable composite beams using high-strength bolted shear connections behave similar to composite beams with welded stud shear connectors. However, the tested demountable composite beams produced higher resistance but lower initial stiffness values than the comparable composite beam with welded studs tested in a preceding research project called DISCCO [17].
2. The ultimate load-bearing capacity of the tested demountable composite beams was reached at relatively large deflection levels (around $L/50$). The test specimens showed a plastic behaviour and a high ductility despite the non-ductile nature of the applied shear connectors.
3. According to the lower initial stiffness, the deflection of the demountable composite beam at about 2/3 P(at $L/50$) was larger than when compared to the composite beam with welded headed shear connection.

Acknowledgements

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