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Push-out tests on demountable shear connectors of steel-concrete composite structures

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ABSTRACT

Circular Economy refers to a move from linear business models, in which products are manufactured from raw materials, used and then discarded, to circular business models where products or parts are re-used, remanufactured or recycled. Structural steel is highly compatible with this concept; however, when steel-concrete composite structures are used, recycling becomes difficult and the potential for reuse is lost. In order to make steel-concrete composite structures reusable, bolted connections should replace the commonly used welded headed studs. Furthermore, the reusable parts should be designed to withstand repeated use.

This paper presents a desktop study and the corresponding laboratory experiments on demountable shear connectors that facilitate recyclability and even provide the potential for reusing complete structural elements. In the Laboratory of Steel and Composite Structures of the University of Luxembourg 15 push-out tests have been carried out using different bolted connection systems suitable for multiple use. The shear connectors have been evaluated based on their shear strength, stiffness, slip capacity, ductility and ability of demounting. The investigated systems included pre-stressed and epoxy resin injection bolts, solid slabs and solid slabs in combination with profiled steel sheeting. The results showed that the tested demountable shear connections could provide higher shear resistance than conventional shear connections. The critical failure mode was the shear failure of the bolts, which is a brittle failure. There was no visible damage observed on the connected members. The application of epoxy resin in the hole clearance resulted in lower slip capacity. The outcome provides an important basis for the justification of the forthcoming enhancement and validation of numerical models of the demountable shear connections. The failure behaviour, the observed damages and the resulting ability of the elements for re-use are discussed in detail.

1. Introduction

The circular economy is essential for a sustainable, resource-efficient and low-carbon future [1]. It refers to a move from linear business models, in which products are manufactured from raw materials, used and then discarded, to circular business models where products or parts are re-used, remanufactured or recycled. Efficient allocation of resources can minimise waste production and decrease carbon dioxide emissions. The RFCS Research Project “REDUCE” of the European Commission (Grant Agreement number: 710040) goes one step beyond the mere material recycling and investigates, how the philosophy of the circular economy can be used in the case of steel and steel-concrete composite structures. One target of the research was to develop structural solutions that allow for demounting and facilitate the future reuse of structures or structural elements.

With proper considerations at the design stage, whole buildings or parts of it can be deconstructed and re-erected elsewhere. If the structural steel elements are not worn, yielded or corroded, they are ideal candidates for re-use with no melting and new hot rolling process. However, the deconstruction of steel-concrete composite structures and the later separation of the materials require a large amount of cutting, which is a labour- and cost intensive work. This is because the composite action is usually provided by shear studs welded on the steel beam and embedded in the concrete deck. As a result, recycling is difficult and the potential for reusing entire elements is lost.

In order to make steel-concrete composite structures demountable, bolted connections should replace the commonly used welded headed studs. Furthermore, the reusable parts should be designed to withstand repeated use.

In the frame of the REDUCE project, 15 push-out tests have been

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conducted in order to determine the characteristics of the demountable shear connector systems. This paper presents the corresponding results and discusses the failure behaviour, the observed damages and the resulting ability of the elements for re-use.

2. Shear connectors for demountable composite beams

2.1. Reusability and demountability

The application of the principles of the circular economy can take place on different levels. Crowther [2] differentiates the following levels for the built environment: material recycling, component remanufacturing, component reuse and building relocation. This project aims to facilitate component reuse and building relocation for steel-concrete composite structures.

The ability to reuse building components depends on the ability to recover them. In current construction practice, after a building reaches the end of its life, usually demolition takes place, and the ability to reuse the components is lost. However, with proper design considerations, the demolition process can be replaced by a deconstruction process, which is essential for keeping the reuse potential of the components. Crowther [2] identifies several aspects of design for deconstruction. Among these aspects, the ones that are relevant for demountable composite flooring systems are the following: use dismountable mechanical connections and dry joints, use modular design and a standard structural grid, use prefabrication, provide access to all parts, provide tolerances for assembly and disassembly, use a minimum number of connectors and design to withstand repeated use.

Dallam [3] and Marshall [4] investigated high-strength bolted shear connectors already five decades ago. However, the research on demountable shear connections is still very limited when compared to welded studs. The questions of demountability and reusability are becoming increasingly important as more emphasis is placed on sustainability. Different types of demountable bolted shear connections have been developed in recent years. Kwon et al. [5] investigated the strengthening methods of existing non-composite bridge girders and the behaviour of post-installed shear connectors under static and fatigue loading [6]. Their investigation included demountable bolted shear connectors such as encased bolts (Fig. 1a), high-tension friction grip bolts (Fig. 1b) and different types of anchor bolts (Fig. 1c). Dai and Lam [7–9] developed a demountable shear connector that can be manufactured from standard studs (Fig. 1d). Embedded bolts with single or double nuts were investigated by Pavlovic [10,11], Moynihan and Allwood [12], Wang [13] and Lee [14]. Ban and Uy [15,16] investigated blind bolts in composite beams. A number of research studies were performed on high-strength friction grip bolts with precast concrete slabs by Lee and Bradford [14], Ataei, [17], Chen [18] and Liu [19]. Suwaed and Karavasilis [20] developed a demountable shear connection with through-bolts, where the bolt clearance is grouted after installation. More recently, Feidaki and Vasdravellis [21] conducted push-out tests on demountable shear connectors that use a steel-yielding mechanism provided by a steel section with slotted holes.

Although all of the presented solutions facilitate the demounting process, they provide different levels of reusability. For example, the embedded bolts (Fig. 1a) and studs (Fig. 1d) are not replaceable. When dismantled, they stand out of the concrete. This makes the threads vulnerable during transportation. If thread damage occurs, the reuse potential of the slab element is lost. In the case of through bolts (Fig. 1b), it is sufficient to replace the bolts while maintaining the reusability of the slab.

2.2. Demountable shear connectors

Taking into account the aspects listed in the previous section, two types of demountable shear connectors have been developed and tested at the University of Luxembourg. Each of them uses prefabricated composite slabs and high strength bolts. L-shaped steel profiles were cast into the concrete to provide edge protection during the repeated use of the elements. The secondary purpose of the L-profiles was to increase the friction resistance between the slab and the beam. Both shear connections were designed so that the damage would occur in the replaceable elements and not in the concrete nor in the steel beam.

2.2.1. Friction bolts with cast in cylinders (P3)

The effects of creep and shrinkage can cause loss of prestress in through bolts (Fig. 1b). In order to eliminate this effect, shear connector type P3 uses a cast-in steel cylinder welded to the L-profile, a top plate welded to the cylinder and a pre-tensioned M20 bolt with a grade of 8.8. The steel cylinder also protects the concrete from any damage that might occur due to bearing. The layout of this shear connector system is presented in Fig. 2.

This connection has pockets in the concrete in order to avoid any protruding part of the concrete surface and it provides access to the shear connectors from the top of the slab. This makes the assembly and the disassembly process safer as there is no need for workers being underneath the slab during construction and deconstruction. It also

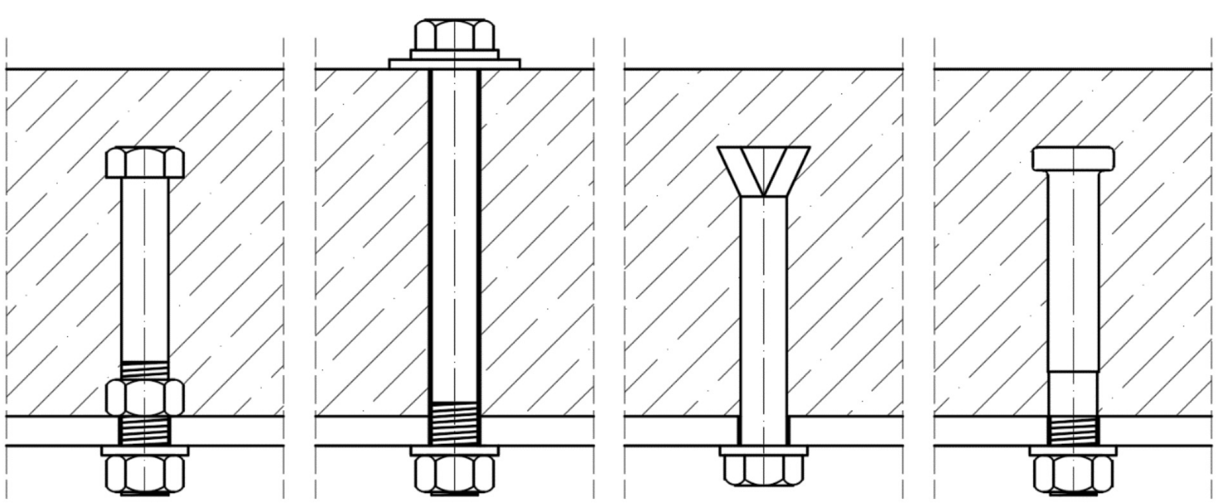

Fig. 1. Different types of demountable shear connectors.

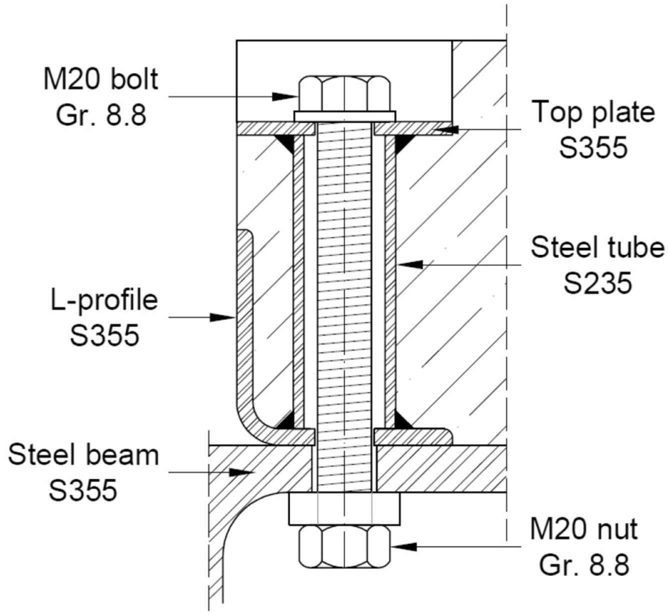

Fig. 2. The layout of the “cylinder system”, P3.

makes the alignment of the holes in the slab and the steel beam easier. In the forthcoming, this shear connection type is referred to as “cylinder system”.

2.2.2. Embedded mechanical coupler device (P15)

Shear connector type P15 uses an embedded mechanical coupler, an embedded bolt and a removable bolt placed from below. The coupler has a grade of 10.9, while the bolts are made of 8.8 material. The reason behind the higher material strength of the coupler is that this way it is possible to ensure that if thread damage takes place, then the damage will occur in the replaceable bolt, and not in the embedded coupler, which is not replaceable. Two variants of this connection type have been developed. The two variants are mostly identical, but P15.1 uses pre-tensioned bolts and P15.2 uses epoxy resin injected bolts i.e. the bolt hole is filled with resin around the bolt. This solution allows larger hole clearance in the steel beam as the resin prevents the slippage of the bolt. Fig. 3 shows the layout of the shear connector system. In the forthcoming, this shear connection type is referred to as “coupler system”.

Table 1
Test parameters.

Series	Shear connector	Slab type	Remark
P3.1	Cylinder system	Solid	
P3.2	Cylinder system	Solid + CF80	Galvanized elements
P3.3	Cylinder system	Solid + CF80	
P15.1	Coupler system	Solid	
P15.2	Coupler system	Solid	Injection bolts

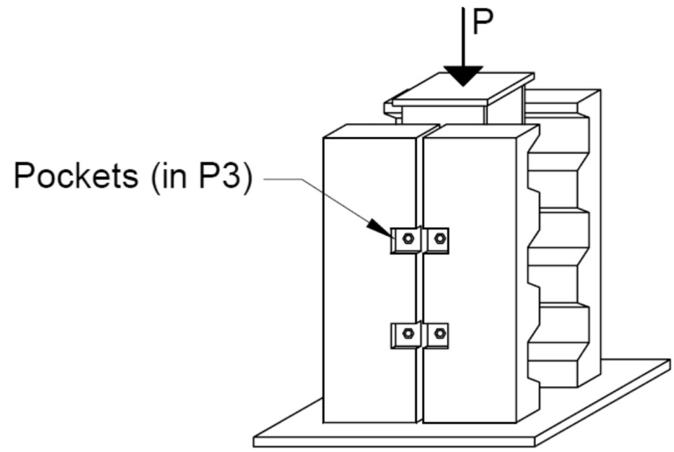

Fig. 4. The test specimen.

3. Push-out tests

3.1. Tests specimens

A total of 15 push-out test specimens were fabricated with a geometrical layout similar to the one recommended by Eurocode 4 [22]. In order to represent prefabricated construction accurately, the slabs were non-continuous above the steel beam. Five different test configurations were designed, and for each configuration, three identical specimens were fabricated. Three series used solid slabs, and two used solid slabs

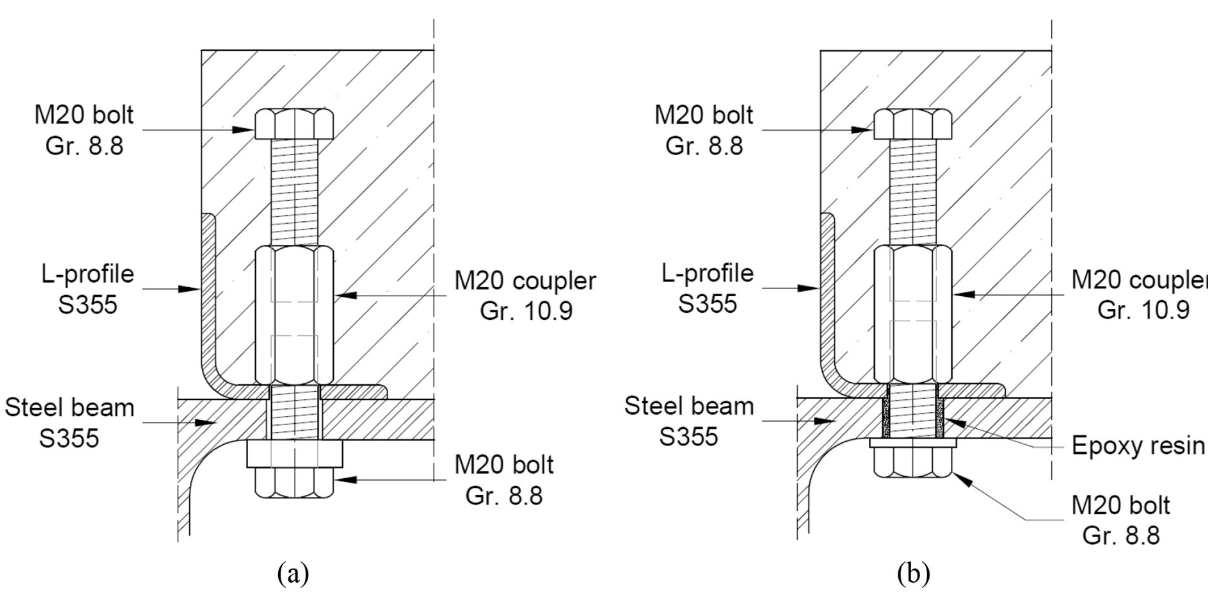

Fig. 3. The layout of the “coupler system” (a) P15.1 with pre-tensioned bolts and (b) P15.2 with injection bolts.

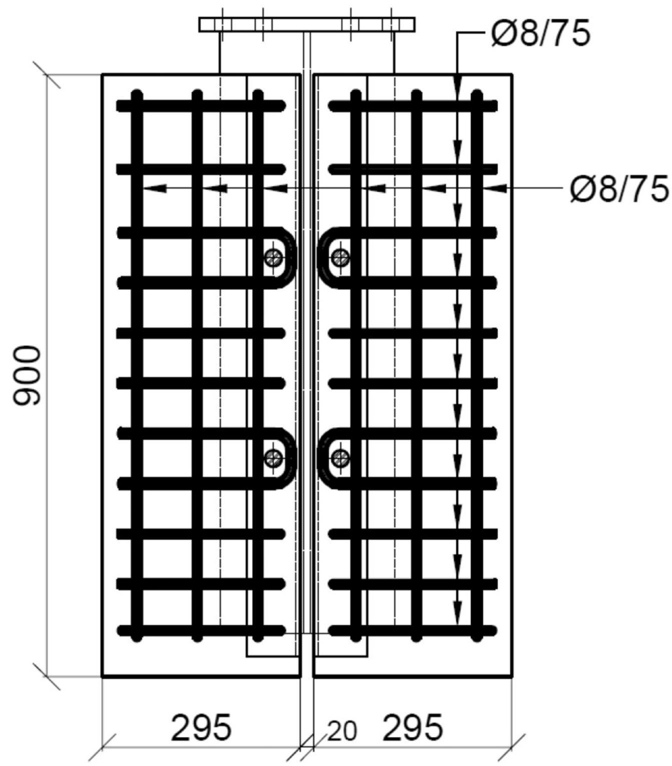

Fig. 5. Reinforcement layout.

Table 2
Concrete strength and reinforcement.

Series	Concrete cube strength $f_{c,28}$ [N/mm^2]	Reinforcement	Applied pretension [kN]
P3.1	59.4	2 layers ϕ8/75	100
P3.2	59.4	1 layer ϕ8/75	120
P3.3	59.4	1 layer ϕ8/75	120
P15.1	44.3	2 layers ϕ8/75	176
P15.2	44.3	2 layers ϕ8/75	0

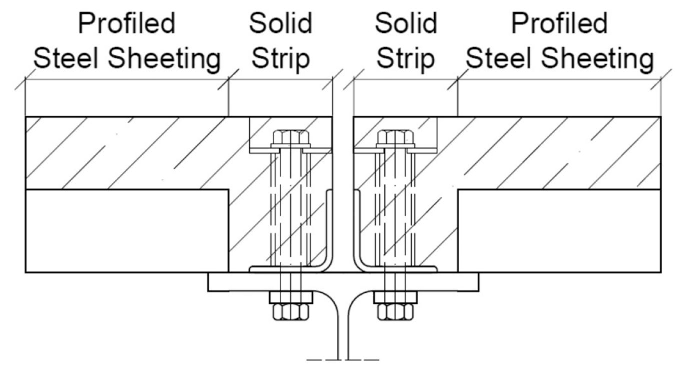

Fig. 7. Solid strip in combination with profiled steel sheeting.

in combination with ComFlor® 80 [23] profiled steel sheeting. Series P3.2 was identical to P3.3, but the beam and the bolts were galvanized in order to see the effect of galvanization on the friction resistance. The overview of the test parameters is presented in Table 1.

Fig. 4 presents the schematic view of a test specimen. Each specimen consisted of an HE 260B steel beam and four pre-fabricated slab elements connected to the beam with the demountable shear connectors.

Fig. 5 shows the general reinforcement layout. In all cases, ϕ8/75 reinforcement was applied with a material grade of B500 B. The solid slabs had two layers of reinforcement in both directions, while the slabs where profiled steel sheeting was also applied had one layer. U-bars were placed around the shear connectors as defined in Eurocode 4 [22] for shear connectors that are placed near the edge of the concrete slab. The concrete strength was measured at the age of 28 days on standard cube specimens. The measured concrete strength values and the applied reinforcement for each specimen are presented in Table 2. The reason behind the relatively high concrete strength is to represent reusable elements, that have an extended lifespan and they need to withstand repeated use. They have a high demand for robustness, and it is necessary that they have high resistance against mechanical impacts. The configuration of the push-out test specimens is shown in Fig. 6.

As shown in Fig. 7, the profiled steel sheeting was shorter than the concrete part, enabling the slab to have full depth like a solid slab without profiled sheeting in the vicinity of the shear connectors. As a result, it was possible to reduce the weight of the slab without compromising the shear connection behaviour.

The bolt holes in the flange of the steel beam have been oversized

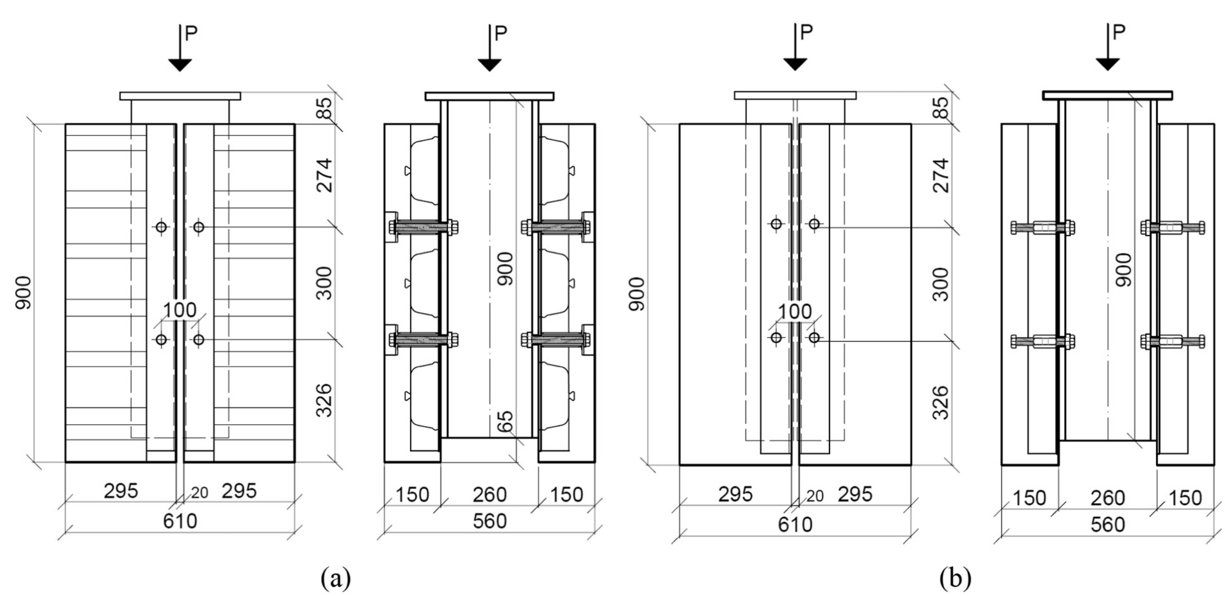

Fig. 6. Configuration of the specimens (a) P3.3 and (b) P15.1.

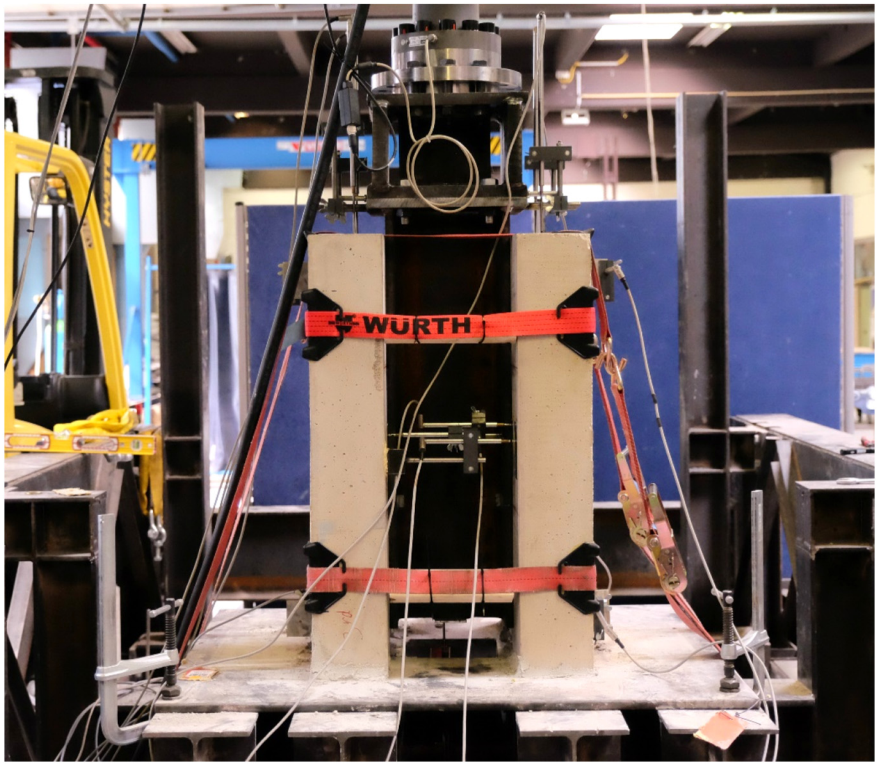

Fig. 8. Test setup.

and had a diameter of 24 mm in order to allow for fabrication and construction tolerances. The application of resin in the boltholes allowed larger tolerances, thus 6 mm clearance was applied in the series P15.2.

The pre-tensioning was applied using the combined method defined by EN 1090 [24]. Different levels of pre-tension were analysed. The applied force was assessed by complementary tests defined by EN 1090 [24] Annex H. Table 2 summarises the applied pre-tension forces in the different test series.

3.2. Tests setup, measurements and procedure

The specimens were placed into a mortar bedding one day before the tests. The tests were conducted using a hydraulic jack with a load capacity of 1000 kN. During the test, belts were put around the specimens to prevent the parts from falling apart once the continuity was lost (see Fig. 8). The force in the hydraulic jack and the displacements were continuously monitored during test conduction.

For each specimen 15 displacement transducers (LVDTs) were employed to measure the (i) relative vertical displacement between the steel beam and the slab elements, (ii) the vertical displacement of the beam measured to the ground floor, (iii) the transversal separation between the steel beam and the slab elements, (iv) the relative horizontal displacement between the adjacent slab elements and (v) the relative horizontal displacement between the slabs on the different flanges of the beam. Fig. 9 shows the layout of the applied displacement transducers.

The loading procedure for standard push-out tests includes 25 load cycles between 5% and 40% of the expected failure load. The applied loading regime is illustrated in Fig. 10. In order to determine the failure load, the first test (LR1) of each series was conducted without cycles. In the second (LR2) and third tests of each configuration, the cycles were performed between 5% and 40% of the previously measured failure load. During the third tests (LR3), in addition to the 25 cycles, several unloading – reloading cycles were performed after 0.5 mm–1 mm increments in the relative slip in order to determine the actual stiffness at larger displacements. The specimens were loaded in force-controlled mode with 20 kN/min load rate until the first slip had occurred. Afterwards, the loading procedure continued in displacement-controlled mode with a speed of 0.5 mm/min.

The applied bolts had in all cases at least 20% overstrength compared to their characteristic strength value. As a result, the 1000 kN capacity of the hydraulic jack was not sufficient to cause failure when 8 bolts were applied in test P15.2–1 (Fig. 18). Therefore, in the subsequent test series, the tests started with 8 bolts until the total load level of 500 kN in load regime 1 (LR1). Then the specimens were unloaded, and the 4 bolts in the upper row were removed. Afterwards, the specimens were tested with only the four lower bolts. In LR2 and LR3, the four upper bolts were removed before testing. Because the rigidity and the bearing capacity of the steel beam and the concrete elements are very high compared to the shear stiffness of the connection, the relative slip and the load on the upper and lower bolt row was assumed to be equal. This assumption was supported by the experimental measurements as no difference could be observed between the load-slip curves of the specimens with 8 bolts and with 4 bolts. The use of only one row of bolts along the loading direction could lead to tilting and separation of the slabs relative to the beam. In order to justify that the applied test setup is suitable for the push-out tests, the following experimental results were used: LVDTs (iv) (see Fig. 9) measured the tilting of the slab elements, which was between 0.16 and 0.74 degrees in all cases. The maximum transverse separation at failure measured by LVDTs (iii) was 0.3 mm. The maximum relative horizontal displacement measured by LVDTs (v) was 1.1 mm. Based on these measurements it had been concluded, that the setup with four bolts was suitable for push-out tests.

3.3. Cylinder system (P3)

3.3.1. First assembly

The load-displacement curves of the cylinder system with solid slabs (P3.1) are shown in Fig. 11, where the relative slip corresponds to the

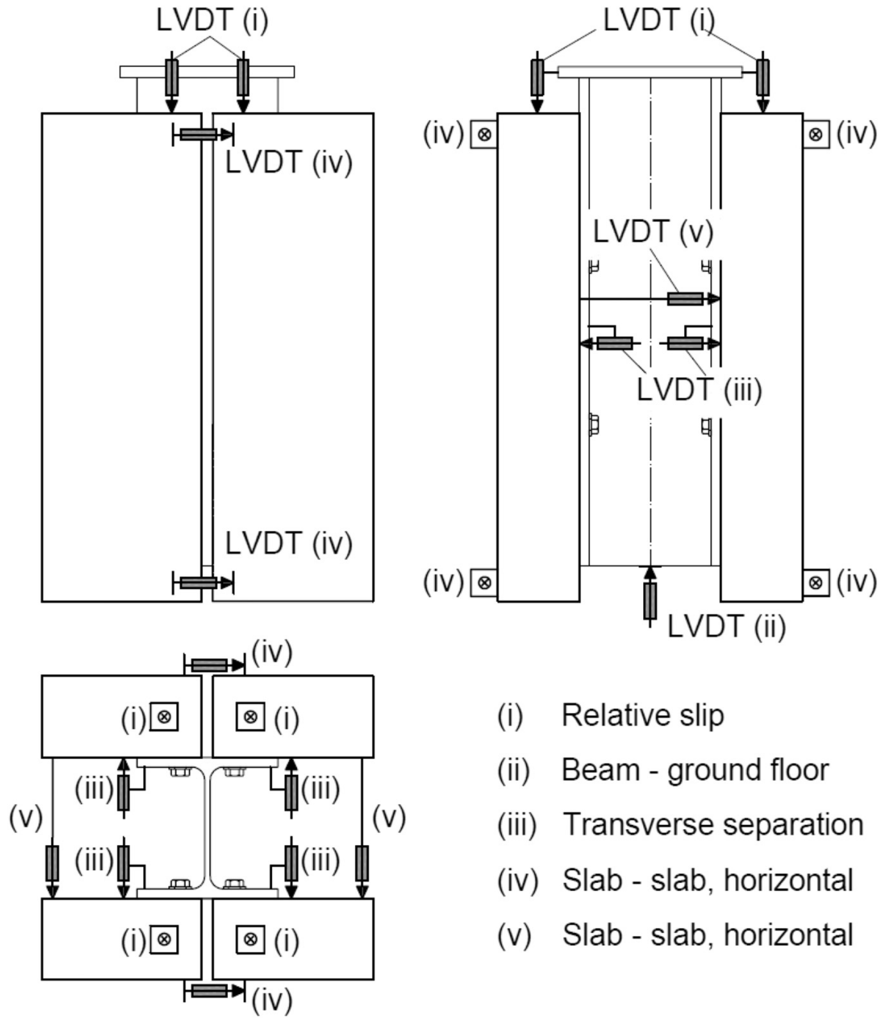

Fig. 9. The layout of the LVDTs.

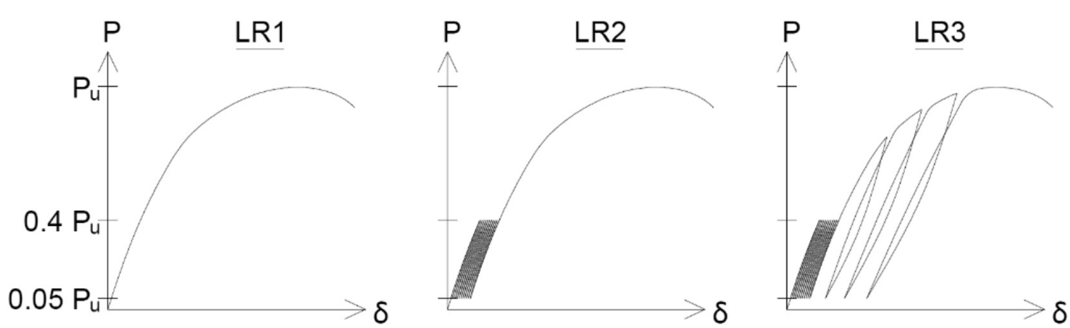

Fig. 10. Loading regime.

average measured slip between the steel beam and the four slab elements. Due to the pre-tensioning, the initial stiffness of the system was high (250 kN/mm) in the early load stages until the friction resistance was overcome at a load level of 26 kN/shear connector. Afterwards, the stiffness decreased significantly to 15 kN/mm. There was only minor nonlinear behaviour observed. In all cases, shear failure of the bolts occurred at an average load level of 141 kN/shear connector.

The failure happened in a very brittle way with no or minor descending branch between 7 and 10 mm relative slip. The load-displacement curves of specimens P3.1-1 and P3.1-3 are in a good agreement, while P3.1-2 shows larger slip capacity. This is due to the fact that the hole clearance was not deducted from the presented slip values, and the bolts have been positioned randomly inside the holes. In the case of P3.1-1, the sudden jump in the curve at 7 mm slip is caused by the failure of one bolt, which failed earlier than the others.

As shown in Fig. 12, minor damages were observed on the elements of the specimens: bearing deformation of the L-profiles, and thread penetration on the bearing surface of the holes in the steel beam.

The load-displacement curves of the specimens where solid slabs were applied in combination with profiled steel sheeting are illustrated in Figs. 13 and 14.

In the case of series P3.2 and P3.3, the initial stiffness was 500 kN/mm and 300 kN/mm respectively. In the case of the galvanized specimens (P3.2), the first slip occurred at a load level of 57 kN/shear connector, while in the case of specimens with no surface finish (P3.3), the friction resistance was 31 kN/shear connector. The stiffness

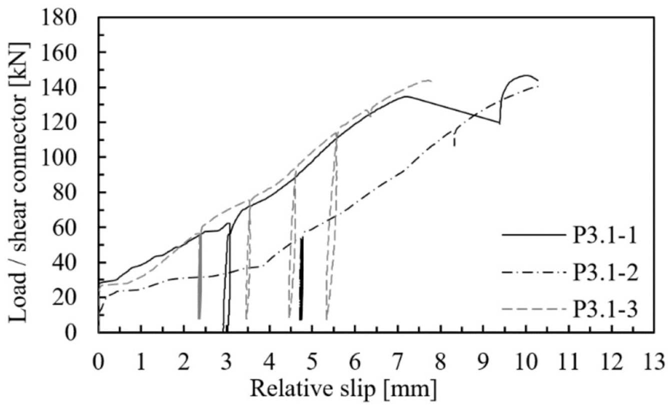

Fig. 11. Cylinder system with solid slabs.

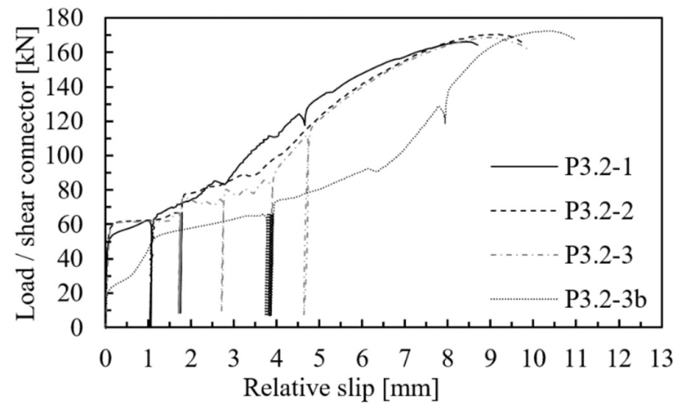

Fig. 13. Cylinder system with solid slabs in combination with profiled sheeting and galvanized elements.

decreased to 15 kN/mm after the first slip had occurred. Fig. 15 shows the failure surface of failed galvanized bolts.

It can be noticed that for the bolts of series P3.2 the resistance is higher (168 kN) when compared to the results of P3.3 (143 kN). This is in agreement with the results of the uniaxial tensile tests that had been conducted on four bolts of each type resulting in mean values of the ultimate strengths of 1045.6 MPa and 948.7 MPa respectively.

All bolts failed in shear and similar minor damages were observed as in series P3.1.

3.3.2. Re-assembly after failure

In order to assess the effect of these minor damages on the reusability, the most heavily loaded specimens (P3.2-3 and P3.3-3) were reassembled with new bolts and the tests were repeated. The results of these tests are indicated as P3.2-3b and P3.3-3b in Figs. 13 and 14 respectively. In the second tests, the failure mode was again bolt shear leading to similar resistance values as in the case of the original tests. However, lower friction resistance and larger relative slip were observed in the case of the galvanized specimen (P3.2-3b).

3.4. Coupler system (P15)

The load-slip curves of the tests of the coupler system with pretensioned bolts are presented in Fig. 16. The initial stiffness was 70 kN/mm. The first slip occurred at a load level of 50 kN/shear connector. Then, after a slip of 2 mm, the stiffness reduced to 30 kN/mm. In all cases, brittle shear failure of the bolts occurred at an average load level of 142 kN/shear connector.

As shown in Fig. 17a, no bearing deformation was observed in the L-

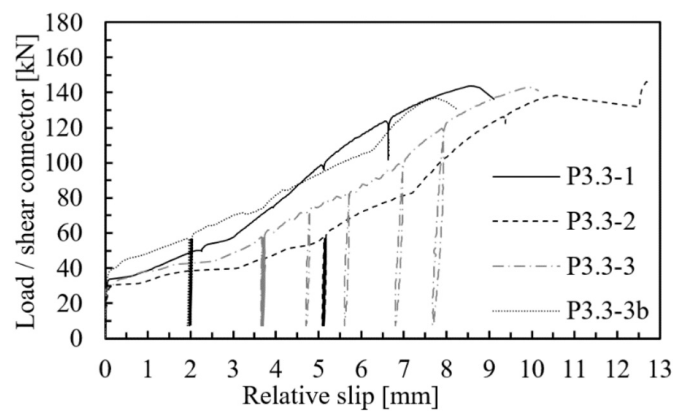

Fig. 14. Cylinder system with solid slabs in combination with profiled sheeting.

profile. However, thread penetration occurred in the bearing surface of the holes in the steel beam (Fig. 17b).

Fig. 18 presents the results of the tests with resin injection bolts. The load-slip curves have three parts: an initial part with a stiffness of 100 kN/mm until the load level of 50 kN, the second part with a stiffness of 30 kN/mm until the load level of 110 kN and the final part with a stiffness of 5 kN/mm until failure. The shear failure of the bolts occurred at an average load level of 131 kN. As the resin in the bolt holes prevented the slippage of the bolts, the curves are in good accordance despite the varying loading regime. Bolt shear was the only observed damage on the specimens. Fig. 19 shows a bolt after failure. The resin

(a) (b)

Fig. 12. Observed damages (P3.1-1) (a) bearing deformation and (b) thread penetration in the steel beam.

Fig. 15. Failure surface of bolts (P3.2-1).

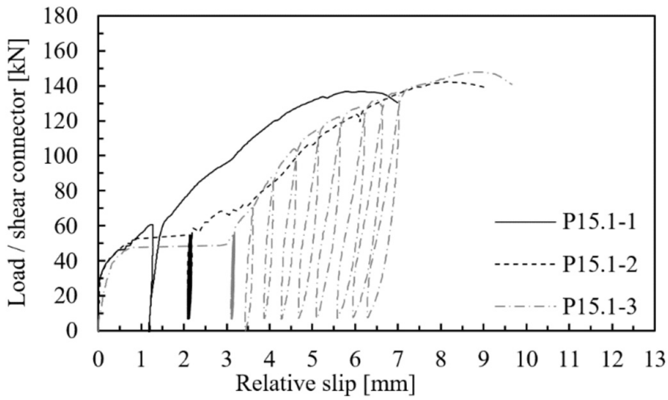

Fig. 16. Coupler system with pre-tensioned bolts.

remained intact during the test. Because of the threaded shape of the resin, the use of a wrench was necessary for the removal of the bolt head from the steel beam.

4. Discussion

4.1. Resistance, stiffness and ductility

The load-slip curves can be divided into three parts. First, due to the pre-tensioning, the initial stiffness of the specimens is high (250–500

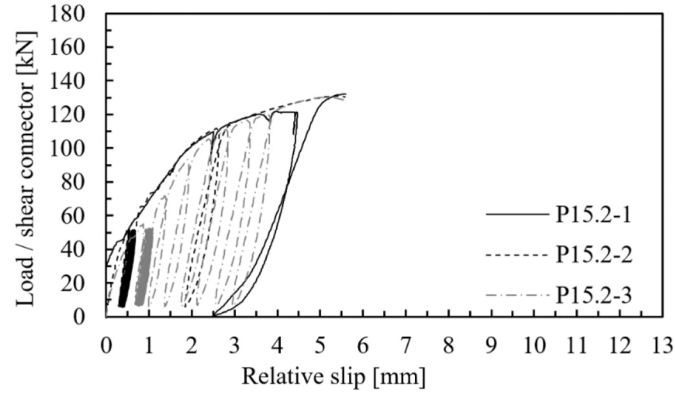

Fig. 18. Coupler system with resin injected bolts.

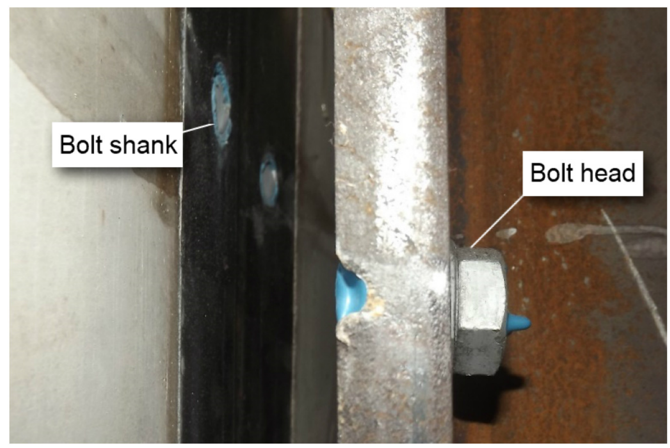

Fig. 19. Sheared bolt after failure (P15.2-1).

kN/mm). In the second part, after the friction resistance is overcome, slip occurs and the stiffness is reduced to a very small value. The third part represents the bearing and shear deformation with around 15 kN/mm–20 kN/mm stiffness, which is very low compared to traditional welded stud shear connector. This behaviour is in accordance with the observations of Lee and Bradford [14].

The highest resistance (168 kN) was measured in tests series P3.2 which consisted of the cast-in cylinders and galvanized elements. However, this is the result of the higher material strength of the bolts. In all cases, the shear failure of the bolts was the governing failure mode. The cylinder system (P3) produced higher resistances, 150 kN on average, than the coupler system (P15) which had 137 kN on average.

(a) (b)

Fig. 17. Observed damages (P15.1-1) (a) sheared bolt and (b) thread penetration.

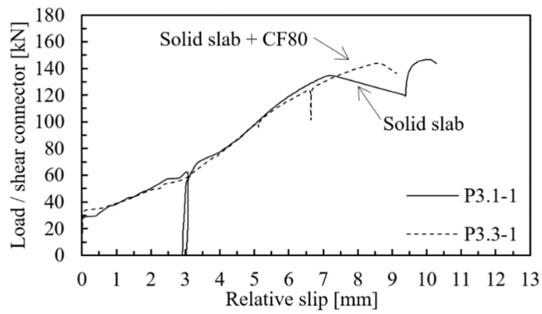

Fig. 20. Comparison of solid slabs with solid slabs in combination with profiled sheeting.

Table 3
Preload and friction resistance.

Series	Preload [kN]	Friction resistance [kN]	Friction coefficient [−]
P3.1	100	26	0.26
P3.2	120	57	0.48
P3.2b	120	27	0.23
P3.3	120	31	0.26
P3.3b	120	33	0.27
P15.1	176	46	0.26

The cylinder system (P3) had in all cases larger slip capacity than the coupler system (P15). In almost all cases, the load-slip curves showed, that the slip capacity has a high sensitivity on the position of the bolts inside the holes. The only exception was series P15.2 where the epoxy resin was injected into the bolt hole.

As shown in Fig. 20, the specimen where the solid slab was used in combination with profiled steel sheeting behaved similarly to the specimen with fully solid slabs. The solid strip in the shear connection region was sufficient to prevent the profiled sheeting from compromising the shear connection behaviour.

Eurocode 4 [22] considers a connector as ductile if the characteristic slip capacity is at least 6 mm. It defines the characteristic slip as the maximum slip measured at the characteristic load level, which is 90% of the failure load after failure. Most of the tested configurations had larger deformation capacity than 6 mm, except series P15.2. However, after the maximal load was reached, the specimen failed earlier than the 10% load drop could have happened. Furthermore, contrary to traditional shear connectors where the maximal load is usually reached after 1–2 mm of relative slip, the tested demountable shear connection systems showed monotonously increasing load-slip behaviour and reached their maximal load at slip between 6 mm and 10 mm. Only minor descending branch was observed after failure. Therefore, if the ultimate load is taken to determine the design resistance on the basis of EN 1994-1-1, Annex B [22], the load-slip behaviour cannot be considered as ductile.

4.2. Friction resistance

The tests showed that the specimens with no surface finish had a friction coefficient of 0.26, while the galvanized specimens had 0.48. However, after re-assembly of the failed specimens, the galvanized specimens' friction coefficient was reduced to 0.23, while there was no change in the case of the specimens with no surface finish (see Table 3). This reduction of friction coefficient could be explained by the flattening of the surface asperities during the first test.

4.3. Demountability and reusability

In load regime 1 (LR1), the specimens were loaded until 500 kN (except P15.2-1, which was loaded until 1000 kN), then unloaded, and the four upper bolts were removed. The successful removal of these bolts proved the demountability of the tested systems. Specimens P3.2-3 and P3.3-3 were re-assembled after failure with new bolts. Afterwards, they were loaded until failure. Their second test showed similar behaviour to their original tests in the means of resistance, stiffness and slip capacity. Because the failure occurs in the bolts and not in the connected members, the developed composite flooring systems are robust and therefore adequate for reuse.

4.4. Challenges in design and application

Some factors can make the design and application of demountable shear connections challenging. First, the fabrication and installation process requires special care for tolerances. If the tolerances are too small, the construction process can become difficult or even impossible. However, too large tolerances lead to an increase in slip and a reduction in stiffness. Second, the tested shear connections showed lower stiffness than standard welded studs, which can lead to increased deflections when applied in a beam. Third, the observed load-slip behaviour is different from an ideally plastic curve that can represent the traditional shear studs. This will have an effect on the design of composite beams or on the definition of the ultimate load-bearing capacity of the shear connection.

5. Conclusion

The results of 15 push-out tests on demountable shear connectors were presented. Based on these results, the structural behaviour of bolted connections was assessed and the following conclusions were made:

1. The developed solutions are robust and adequate for reusable composite flooring systems.
2. The tested pre-tensioned demountable shear connectors behave rigidly before the friction resistance is overcome. Afterwards, bolt slip occurs followed by bearing and shear deformation.
3. The bolt position inside the hole has an influence on the load-slip curves.
4. It is sufficient to provide a strip of solid slab above the flange in order to prevent the profiled steel sheeting from compromising the shear connection behaviour.
5. With the application of epoxy resin, larger tolerances can be allowed without compromising the load-bearing capacity where the execution of large deck components makes it necessary.
6. In order to be able to use the rules of Eurocode 4 for beams with partial interaction, the definition of the ultimate load-bearing capacity of the shear connection has to be reassessed.

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