Steel-Yielding Demountable Shear Connector for Composite Floors with Precast Hollow-Core Slab Units

ST7226

Steel-Yielding Demountable Shear Connector for Composite Floors with Precast Hollow-Core Slab Units

Eliza Feidaki[1], George Vasdravellis[2], Jun He[3], and Sihao Wang[4]

[1]PhD candidate, Institute for Infrastructure and Environment, Heriot-Watt University, U.K.
[2]Associate Professor, Institute for Infrastructure and Environment, Heriot-Watt University, U.K. (corresponding author). gv39@hw.ac.uk
[3]Associate Professor, School of Civil Engineering, Changsha University of Science and Technology, China
[4]PhD Candidate, Department of Bridge Engineering, Tongji University, China

ABSTRACT

A demountable shear connector for use in composite floors with precast hollow-core slab units is proposed. The proposed shear connector consists of a steel square hollow tube welded on a steel plate that is bolted on the steel section's top flange using four high strength bolts. Concrete is cast only around the connector allowing for an easy separation of the precast slab units from the steel sections during the deconstruction phase, with operations required only from the top of the composite floor. The shape of the hollow tube promotes a ductile elastic-plastic behavior under the longitudinal shear flow in a composite beam. Ten push tests using an horizontal testing arrangement were carried out to assess the structural performance of the novel shear connector. The experimental results show that the shear connector has stiffness comparable to that of a welded shear stud, strength that can be adjusted to achieve the desired degree of shear connection in a composite beam, and slip capacity that is much higher than the requirements of the current codes of practice. A finite element model was also calibrated against the tests and found capable of accurately reproducing the experimental behavior. The finite element model was then used to conduct a number of parametric studies to safely generalize the experimental results. Based on the experimental and numerical results, a design model is proposed to predict the strength of the

demountable shear connector based on either the yielding of the steel tube or the failure of the precast slab.

INTRODUCTION

The construction sector plays an important role in achieving future sustainability targets set by many countries, since the production of steel and cement, the two materials that are exclusively used in construction, accounts for fifteen percent of the total global man-made CO_2 emissions (IEA 2007). Moreover, it is foreseen that the material demand will be doubled globally by 2050, indicating that a more responsible consumption of natural resources should be adopted. In addition, the construction and demolition sectors are responsible for a large amount of waste sent to landfill (Burka 1993). Thus, high sustainability standards in construction can be achieved by developing structural systems that offer the advantage of deconstruction at the end of their service life, as opposed to demolition, and reuse of the structural components.

The 'design for deconstruction' principle, however, is difficult to be applied to steel-concrete composite floors, which are widely used in multi-storey buildings worldwide, because the headed shear studs are commonly welded on the steel sections and embedded in concrete. For this reason, recent research works recommend avoiding this type of construction when designing for reuse (Webster and Costello 2005; Densley Tingley and Davison 2011; Brambilla et al. 2019).

A number of recent research efforts have been devoted to develop novel methods that allow for the separation of the slab from the steel section in a composite floor and reuse of the components, using demountable shear connectors. Lam and Saveri (2012) and Dai et al. (2015) proposed the use of threaded Nelson shear studs that are bolted (instead of welded) on the steel

section's top flange. They investigated the performance of the bolted studs in pushout tests and performed a parametric study using the finite element method (FEM). They found that the bolted studs have comparable strength, higher slip capacity, and approximately half the initial stiffness of the welded studs due to the unavoidable clearance between the stud collar and the flange holes. Rehman et al. (2016) conducted twelve pushout tests using threaded shear studs in slabs with profiled metal decking and found a similar behavior as in solid slabs but with a slightly reduced ultimate strength. Pavlovic et al. (2013) carried out four pushout tests and numerical simulations to investigate the performance of high strength bolts (M16, grade 8.8) as shear connectors in precast slabs with cast-on-site openings. Compared to welded shear studs, the high strength bolts exhibited similar strength, half the initial stiffness, and their slip capacity was limited to 4-5 mm. Moynihan and Allwood (2014) conducted three composite beam tests using M20 bolts as shear connectors. They found that the flexural response of the specific beams with demountable connectors was comparable to that of previously tested beams with welded shear studs in terms of ultimate strength and ductility. Liu et al. (2015) proposed the use of high-strength friction-grip bolts as shear connectors in conjunction with precast geopolymer concrete slabs and evaluated them through pushout tests. The friction bolts exhibit a rigid initial behavior due to pretension, a sudden slip once the friction force is overcome, followed by a plastic response and subsequent bolt shear failure. Further tests on composite beams using high-strength friction-grip bolts and geopolymer concrete precast slabs (Ataei et al. 2016; Liu et al. 2017) showed good ductility and adequate ultimate strength. Pathirana et al. (2015) and Pathirana et al. (2016a) proposed the use of two blind bolt types to retrofit composite beams and compared their behavior with that of welded shear studs through pushout tests. The blind bolts in the pushout tests had a higher ultimate capacity than the welded studs, but smaller stiffness. They also tested the blind bolts in

full-scale composite beams, which showed comparable behavior to beams with welded studs in terms of strength, stiffness and ductility (Pathirana et al. 2016b). Blind bolts as an alternative to welded studs were successfully implemented in composite beams under sustained loads (Ban et al. 2015) and under dynamic loading (Henderson et al. 2015a; Henderson et al. 2015b; Henderson et al. 2017). Suwaed and Karavasilis (2017) proposed a 'locked-nut' shear connector for use in composite bridges with precast decks. They used a conical-shaped nut that 'locks' into counter-shaped corresponding holes on the steel section flange to eliminate the initial slip due to tolerances. The resulting force-slip response was characterized by high initial stiffness (comparable to that of a welded stud), and strength and slip capacities significantly higher than those of a welded stud. Note that the minimum slip capacity required by Eurocode 4 (BSI 2004) in order for a shear connector to be characterized as ductile is 6 mm.

A composite floor system using precast concrete slab units acting compositely with the steel sections offer the advantages of off-site quality controlled fabrication, flexibility in the geometric characteristics of the precast units, possibility for custom-made geometries, durability, and fewer operations on site (Couchman 2014). The use of precast hollow-core units (HCUs) offers the additional advantage of large spans without the need for secondary beams, thus enabling even greater economy in steel and fewer operations on site. Typically, HCUs are 1.2 m wide and alternate cores are left open to the top for a specific length (usually 0.5 m) to allow for the placement of rebars for the effective connection between adjacent units. Shear studs are welded on the steel section and *in situ* concrete is poured to fill the open cores and to connect the units with the studs. Fig. 1 shows a composite beam section using HCUs. Since the welded shear studs are fully embedded in concrete, the deconstruction of such composite floor systems is still problematic; however, demountable shear connectors for use with HCUs are not yet proposed in

the literature.

In this paper, a demountable steel-yielding shear connector is proposed for use in composite floors with HCUs. The proposed shear connector consists of a steel square hollow tube welded on a steel plate that is bolted on the steel section's top flange using four high strength bolts, and it has the following unique features: a) it is not fully embedded in concrete, thus operations only from the top of the floor are required to dismantle the composite floor system during deconstruction; b) it has a shape that promotes a ductile and predictable force-slip behavior under the longitudinal shear force in a composite beam; c) it has high initial stiffness, strength that can be adjusted to required levels, and slip capacity much higher than the minimum requirements of current codes of practice, a feature that enables the designer to use large spans with partial shear connection; and d) it eliminates the issue of tolerances in the reuse phase, as opposed to the case of bolted connectors that protrude from the slab and, therefore, they may not fit precisely in the pre-drilled holes of the steel sections. The proposed shear connector was experimentally evaluated through ten push tests using an horizontal testing arrangement. A detailed numerical model based on the finite element method (FEM) was then calibrated against the experimental results and found capable of accurately simulating the experimental behavior. The validated FEM model was used to carry out a number of parametric studies to safely generalize the experimental findings. Based on the experimental and numerical results, a design model to predict the strength of the proposed connector based on either the yielding of the steel element or the failure of the precast slab is proposed.

STEEL-CONCRETE COMPOSITE FLOOR WITH HOLLOW CORE SLABS AND A DEMOUNTABLE STEEL-YIELDING SHEAR CONNECTION MECHANISM

Description

The proposed composite floor system uses precast HCUs and a novel demountable shear connection mechanism consisting of a steel yielding device, denoted as the yielding pocket (YP). Fig. 2a shows a 3-d view of a beam segment with two HCUs, one YP installed and three open cores per HCU. One or two (depending on the degree of shear connection required, partial or full) cut-outs are made through the depth of the HCUs at the edges parallel to the beam axis, in order to accommodate the installation of the YPs. The YP consists of a steel square hollow section having a steel plate welded at the bottom and total length equal to the slab depth. Vertical elongated holes are cut on the sides of the YP that are parallel to the beam axis in order to form vertical steel strips. Aligned horizontally slotted holes are also opened on the same sides of the YP. A 3-d view of the YP is shown in Fig. 2a.

Four high strength bolts are used to clamp the YP on the top flange of the steel section, as shown in the longitudinal section of Fig. 2b. Rebars are placed in the open cores. A rebar is inserted through the horizontally slotted holes of the YP and placed in the middle open core, aiming to prevent uplift of the slab during the bending of the beam. A 50-mm gap is left between the YP and the HCUs, and *in situ* concrete is poured to fill the open cores and the gaps around the YP. Before pouring the *in situ* concrete, polythene foam is used as formwork around the YP to disconnect the bottom part of the concrete from the YP. In this way, the longitudinal shear force resultant is moved slightly above the base of the YP and resisted by bending of the vertical steel strips and the walls of the YP. The inset of Fig. 2b shows the expected deformed shape of the YP under the longitudinal shear force. Thus, a ductile failure of the YP is promoted as opposed to a brittle failure due to bolt shearing that would occur if the concrete was in full-depth contact with the YP.

To deconstruct the system, the concrete 'tooth' around the YP (see Fig. 2b) is removed using a standard concrete cutting tool; the rebar passing through the YP is cut; and the YPs are unbolted and removed, which is an operation performed from the top of the beam. The slab can then be removed and recycled using standard methods, while both the steel sections and the YPs can be reused directly in new building construction projects. Another possibility that this system offers is to reuse not only the steel sections and YPs but also the precast slabs. This would require removing the concrete strip above the steel section and cutting all the rebars in the middle. The HCUs will have the rebars protruding out of the concrete (since only the concrete strip above the steel section is removed during deconstruction as described above). To reuse them in a new building and avoid re-opening the cores, couplers can be used to connect the protruding rebars, as indicated in Fig. 2a.

Mechanics-based strength of the demountable shear connector

Figs. 3a and 3b show the geometric properties of a YP, which are the width of the vertical strips, w, the height of the vertical strips, h, and the thickness of the hollow section, t. The foam around the YP is placed in such a way that the bottom side of the concrete tooth is in line with the top end of the vertical strips. Under the longitudinal shear force in a composite beam, the YP deforms as shown in Fig. 3c. The resistance, therefore, is provided by the plastic bending of the vertical strips and the vertical walls. The section of the vertical walls is shown in Fig. 3b as hatched regions. Assuming fixed boundary conditions, the vertical strips are expected to develop two plastic hinges at their ends, as shown in Fig. 3c. The distance between the top and bottom plastic hinges is h_1, which is less than h. h_1 is taken as the distance between the sections of the strip with width w that are just before the semi-circular part of the elongated hole (see Fig. 3c). The plastic moment of resistance of a vertical strip section is given by:

$$M_{p,strip} = \frac{tw^2}{4} f_{ys} \qquad (1)$$

where f_{ys} is the yield strength of the vertical strip material. The force provided by all the vertical strips is given by:

$$F_{p,strips} = 2 \frac{M_{p,strip}}{h_1} n \qquad (2)$$

where n is the total number of strips.

Likewise, the vertical walls are expected to develop plastic hinges at their top and bottom ends. Using plastic sectional analysis, the plastic moment of resistance of the wall section (hatched region in Fig. 3b) is given by:

$$M_{p,wall} = Z_p f_{yw} \qquad (3)$$

where Z_p is the plastic section modulus of the wall section and f_{yw} is the yield strength of the material of the vertical walls. The force provided by the two vertical walls is:

$$F_{p,walls} = 4 \frac{M_{p,wall}}{h_1} \qquad (4)$$

Thus, the total strength of the YP is given by:

$$F_{YP} = F_{p,strips} + F_{p,walls} \qquad (5)$$

To allow for the above plastic shear-resisting mechanism to develop, concrete failure should be prevented before the force F_{YP} has been reached. Possible failure modes of the slab due to dowel bearing forces include (Oehlers and Bradford 1995): a) concrete shear failure with the formation of diagonal cracks; b) concrete splitting; and c) concrete ripping. From the above failure mechanisms, the concrete shear failure was found to be the most critical. The concrete shear strength was taken as the shear strength of a cracked plane, according to Eurocode 2 (BSI 2002a):

$$F_{conc} = 2(cf_{ct}A_c$$

$$+ \mu f_{yr}A_{tr}\kappa)$$ (6)

where f_{ct} is the lowest tensile strength between the *in situ* concrete and the HCU concrete; A_c is the area of the shear plane; f_{yr} and A_{tr} are the yield stress and area of transverse reinforcement, respectively; κ is a reduction factor to account for an insufficient anchorage between the reinforcement and the concrete, taken as 0.5 as recommended in FIB (2010); and c and μ are cohesion and friction coefficients equal to 0.5 and 0.9, respectively, as recommended in Eurocode 2 (BSI 2002a). Eq. 6 represents the contribution of adhesion, aggregate interlock and friction mechanism along a shear crack. The coefficient 2 accounts for the fact that there are two shear critical planes, one on each side of the YP.

Prototype steel-concrete composite floor system

Based on the above design procedure, the YP's strength can be adjusted by selecting the width and height of the elongated holes and the thickness of the hollow section. Fig. 4 shows a 3-d view of part of a composite floor system having 12 m-long main composite beams using the proposed structural details. The main beams are placed at 7.5 m distances, and 200 mm-deep HCUs are used to span this distance without the need for secondary beams. One cut-out is made in the mid length of each 1.2-m long HCU and half cut-out at its both ends. In this way, YPs are placed at 600 mm center-to-center distances on the steel section. Using a typical office building loading (BSI 2002b), and assuming simply-supported boundary conditions, the design of a main composite beam requires an IPE550 steel section and 24 YPs with strength equal to 280 kN each to achieve full shear connection. This prototype composite floor system served to select the specimen geometry for the experimental program.

EXPERIMENTAL PROGRAM

Test setup

To study the structural behavior of the proposed demountable shear connector, ten push tests were conducted using the horizontal testing arrangement shown in Fig. 5. The reason for using an horizontal test setup instead of the standard vertical setup prescribed in Eurocode 4 (BSI2004) is practical, i.e. the size of the HCUs would make the fabrication and handling of the specimens problematic in the Lab environment. Other reasons for conducting push tests in an horizontal arrangement when using precast HCUs are given in Lam (2004).

An hydraulic actuator of 1000 kN force capacity and 500 mm displacement capacity with attached load cells was used to push the HCUs against the steel section through a strong spreader beam, indicated in Fig. 5. The actuator was attached on a strong reaction frame bolted on the strong floor through eight M28 high strength bolts. To fix the steel section of each specimen on the strong floor, four 30 mm-thick steel plates were welded on the bottom flange and perpendicular to the section, and then bolted on the strong floor using M28 steel rods, as shown in Fig. 5. The bolts and the rods were pre-tensioned to their proof load to create a strong friction connection and, thus, eliminate relative slip. The slab was protruding from the steel section by 100 mm to allow the movement of the spreader beam against the steel section. The centroid of the actuator was in line with the mid-depth of the slab. The spreader beam was supported by a steel base. The top surface of the base was greased to minimize the friction force during the sliding of the actuator on it. Two configurations were employed to transfer the load to the specimens, shown in Fig. 5b. The first configuration used wooden parts of a rectangular section between the HCUs and the flange of the spreader beam. The second configuration used a mortar tooth cast before testing between the flange of the spreader beam and the HCUs. All the specimens were tested using the first configuration

apart from specimens SP1A and SP6, for which the second configuration was used.

Specimen description and design parameters

A plan view and a longitudinal section of a typical push specimen are shown in Figs. 6a and b, respectively. The specimens' dimensions were selected based on the prototype composite floor design. Each specimen consisted of two HCUs connected to a UB533x210x92 (equivalent to an IPE550 or W530x101) steel section using a single YP. The HCUs used for all specimens had depth, nominal width and length equal to 200x1200x800 mm respectively, nine cores, and provided by *Bison Precast Limited.* They had a compressive cube strength equal to 55 N/mm^2. Five cores were left open to the top for the placement of transverse reinforcement, i.e. the cores 1, 3, 5, 7 and 9 shown in the inset of Fig. 6b. The edge cut-out in each HCU was 300 mm long (in the direction parallel to the steel section) and 70 mm wide. Polythene foams were placed around the YP to form the concrete tooth before pouring the *in situ* concrete. The five open cores were filled with *in situ* concrete after the 12 mm-diameter rebars were placed inside the cores.

Based on Eq. 6 and one trial test, it was concluded that the core located just before the YP should be filled with concrete as well, as otherwise the slab would fail prematurely due to the large stress field concentrated in that region before an effective shear transfer takes place. This result was confirmed using the FEM model, as will be described later. For this reason, the core designated as '4s' in Fig. 6 was left open to the top and filled with *in situ* concrete in all specimens.

Each YP was manufactured from a square SHS180x180x8 or SHS180x180x10 hollow tube of S355 steel grade. The steel plate welded at the bottom of a YP had dimensions 200x200 mm, was 20 mm thick, and made of S275 steel grade for all specimens. The depth of the horizontally slotted hole of the YP was designed to allow the placement of a 12 mm-diameter rebar with minimum tolerance.

The parameters varied in the tests were the YP geometry and the *in situ* concrete strength. A summary of the test matrix is provided in Table 1. The specimens are designated by the prefix 'SP' followed by a number and a letter. The number indicates a specific YP geometry group and the letters indicate different tests with identical YP geometry. When there is no letter, only one test was conducted with the specific YP geometry. Note that the *in situ* concrete cube strength was different in all tests.

The geometric parameters of the YP were changed to produce a different F_{YP} according to Eq. 5. The specimens in groups SP1 to SP4 were constructed using a relatively high strength *in situ* concrete mix, with cube strength ranging from 38 to 68 N/mm^2, in combination with a relatively low YP yield strength. The YP strength was gradually increasing from SP1 to SP4 group. F_{YP} according to Eq. 5 had values ranging from 150 kN in SP1 to 263 kN in SP4 group. Specimens SP5 and SP6 were constructed using a normal strength *in situ* concrete mix, with cube strengths equal to 30 and 35 N/mm^2 in SP5 and SP6 respectively, and stronger YPs. F_{YP} was 345 kN in SP5 and 310 kN in SP6 according to Eq. 5. The ratio F_{YP}/F_{conc} is given in Table 1. This ratio was low in specimens SP1 to SP4, assuming values in the range 0.34-0.73, and high in specimens SP5 and SP6, with values 1.17 and 1.11, respectively. Thus, failure in specimens SP1 to SP4 was expected to occur due to YP yielding, whereas in specimens SP5 and SP6 the concrete slab was expected to be involved in the failure modes, considering also the hardening behavior of the YP that would actually produce ultimate strengths higher than F_{YP}.

Material property tests

Both concrete and steel material tests were performed prior to tests. Concrete tests were conducted on the *in situ* concrete mix of each specimen. They consisted of standard cube compressive tests and splitting tensile tests (BSI 2009). The latter aimed to determine the tensile

strength of the concrete. Three cube compressive tests were carried out on the same day as the corresponding push test. The concrete cubes were casted on the same day as the corresponding specimen using the same concrete mix. The cubes were 100x100x100 mm, while the splitting tensile tests were performed on 100x200 mm standard cylinders as per BSI (2009). Table 2 summarizes the average compressive concrete strength of three cubes and the average splitting tensile strength of two cylinders on the test day. The maximum aggregate size used for the concrete mix was 10 mm. A medium to high workability concrete was produced using a polycarboxylate high range water reducing admixture.

The material properties of the YP were obtained from standard tensile coupon tests (ASTM 2011) taken from the flat and corner regions of the SHS180x180x8 and SHS180x180x10 tubes used to manufacture the YPs. The properties obtained are the modulus of elasticity, the 0.2% proof stress, the ultimate stress and the tensile strain corresponding to the ultimate stress. The results are given in Table 3 as average values of three coupon tests.

Instrumentation and loading procedure

A number of linear variable differential transformers (LVDTs) and strain gauges were used to monitor the experimental behavior of the push specimens. The relative slip between the steel section and the HCUs was measured using two LVDTs horizontally placed near the YP, on each side of the steel section. The base of each of these LVDTs was attached to the bottom side of the HCU and the tip on the top flange of the steel section. The vertical separation between the slab and the steel section, or uplift, was measured during the tests of specimens SP1A and SP6 using two LVDTs vertically placed near the YP on each side of the steel section. Each of these LVDTs was connecting a point of the bottom steel section flange to a point of the bottom side of the HCU. Two more LVDTs were used in SP1A and SP6 to measure the differential horizontal displacement

between the top and bottom flange of the steel section, and, thus, to check if significant second-order bending was introduced in the beam due to the eccentric application of the loading with respect to the beam's centroid in the specific test setup. The horizontal load was measured by the built-in load cell of the hydraulic actuator. Strain gauges were installed on the steel strips and vertical walls of the YP and on the concrete slab around the YP to monitor the development of strains. Four strain gauges were also attached to the transverse rebars of specimen SP6. Fig. 7 shows the instrumentation used for specimens SP1A and SP6. In the rest of the specimens, only the relative horizontal slip in the HCU-steel section interface, the strains developed at the five points on the concrete slab shown in Fig. 7, and the strains developed in the YP were measured.

The load was initially applied under force control up to 40% of the expected yield load and then cycled 25 times between 5% and 40% of the expected yield load. The rate of loading during the cyclic loading was around 1 kN per second. After the cyclic loading, the load was applied under displacement control at a rate of 0.2 mm per minute up to failure.

EXPERIMENTAL RESULTS

Failure modes and observations

Table 4 summarizes the failure modes of the specimens. Specimens in groups SP1 to SP4 (low YP strength - high strength *in situ* concrete mix) failed in a ductile mode with plastic deformations concentrated in the YPs, specimen SP5 failed due to concrete slab cracking without significant deformations in the YP, and specimen SP6 failed in a mixed mode that included both the YP plastic bending and the failure of the concrete slab at later stages of the loading.

Specimen SP1A was loaded until all possible failure modes were met. The response was ductile due to plastic bending of the YP up to a slip of 47 mm corresponding to a force of 357

kN. The plastic deformations were concentrated in the YP's vertical strips and walls and no signs of cracks in the slab were observed up to that point. Fig. 8a shows the deformed YP at the end of the test; due to the very large inelastic deformations, cracks formed at the ends of the vertical strips at the positions of plastic hinges, as indicated on the same figure with an arrow. At a slip of 47 mm, the deformed transverse wall of the YP came in contact with the row of bolts close to the loaded side of the specimen, as shown in Fig. 8b. As the specimen was further loaded, the slab units started moving upwards and sliding over the YP, due to the large inclination of the YP's walls. This new deformed position created secondary hogging moment which finally led to the flexural failure of the slab by the sudden formation of a large crack through the width of the slab units at a position just before the YP. This failure happened at a slip of 58 mm and is shown in Fig. 8c. Although the slab has ultimately failed in a brittle manner due to the sudden transverse cracking, the response of the specimen SP1A is characterized as ductile because the main source of plasticity up to an excessive slip of 47 mm was the plastic bending of the YP. Note that slip demands of more than 20-22 mm can be met in composite beams used in buildings, e.g. in design cases where long spans and partial shear connection are used (Zona and Ranzi 2014). The proposed connector with the large ductile slip capacity can be safely used in such cases where force redistribution is needed.

Specimen SP5, which had a combination of high strength YP and normal strength *in situ* concrete, failed due to concrete cracking in the pattern shown in Fig. 9a. A transverse crack started to form on the surface of the slab at a load of 130 kN and corresponding slip of 0.9 mm. The crack was formed in the interface between the fourth core of the HCU and the *in situ* concrete on one side of the YP. A second transverse crack was formed on the surface of the specimen between the fifth core of the HCU and the *in situ* concrete at a load of 200 kN and corresponding slip

of 2.2 mm. Shear diagonal cracks started forming at the corners of the YP at a load of 314 kN and 7.2 mm of slip, propagating diagonally towards the spreader beam at an angle of approximately 45 degrees to the beam axis. Further increase of the load caused more shear cracks to form and the specimen ultimately failed at 321 kN and corresponding slip 8.3 mm, followed by gradual drop in the load to 286 kN, at which point the test was stopped. After dismantling the specimen, very little deformation was observed in the YP. Fig. 9b shows the YP at the end of the test. Note that, although the shear failure in this specimen was predicted by Eq. 6 (since $F_{YP}/F_{conc}=$ 1.17 in Table 1), the transverse (ripping) cracks were unexpected and they are attributed to eccentricities that were introduced in the application of the loading due to the imperfections on the surface of the specific HCUs used in this test and, in turn, resulted in secondary moments and torsional effects, which were extremely difficult to be quantified. The diagonal cracks were not symmetric with respect to the beam's centerline, as can be seen from Fig. 9a, due to torsional eccentricity during the test. The torsional eccentricity was recorded by the LVDTs that were measuring the horizontal slip and monitored higher slip values on the side where cracks were more prominent.

Specimen SP6 failed in a mixed mode. Up to 19 mm of slip, the response was ductile due to YP plastic bending, while there were no signs of cracks on the slab. At a force equal to 427 kN and corresponding slip of 19.6 mm, diagonal shear cracks started to form. The cracks were symmetric with respect to the beam's centerline. Further loading of the specimen caused the force to drop gradually to 380 kN and the test was stopped. Fig. 10a shows the failure mode of the slab at the end of the tests and Fig. 10b shows the deformed YP after the test. The YP has plastically deformed but much less than in test SP1A (Fig. 8a).

In specimen SP1C, an inadequate amount of *in situ* concrete was placed adjacent to the

sides of the YP that were parallel to the beam axis. This resulted in the out of plane bending of the device and the inability to develop the full plastic strength according to Eq. 5. The deformed shape of the YP in this test is shown in Fig. 11.

A common secondary failure mode observed in all tests is the local crushing of the bottom part of the concrete tooth due to the concentration of compressive stresses at that region as the YP deforms. This local failure is shown in Fig. 12.

Experimental versus theoretical strength

Table 4 summarizes the experimental yield ($F_{y,exp}$) and maximum ($F_{max,exp}$) strengths achieved in each specimen; the interface slip corresponding to maximum force; the ratio of the experimental to theoretical yield strength of the YP, $F_{y,exp}/F_{YP}$; and the ratio of the maximum experimental force to the experimental yield strength, $F_{max,exp}/F_{y,exp}$. In specimen SP1A, the maximum force and corresponding slip are taken as those recorded just before the bearing of the YP walls on the bolts, i.e. 357 kN and 47 mm, respectively. The yield force of specimen SP5 is not defined because there was no significant deformation of the YP in that test.

The ratio $F_{y,exp}/F_{YP}$ has a mean value of 1.06 with standard deviation 0.05, which indicates that Eq. 5 based on simple mechanics predicted the yield force of the YP in the specimens with no early failure of the concrete slab with good accuracy. Based on the failure modes of the specimens, it is concluded that the ratio F_{YP}/F_{conc}, reported in Table 1, which was used for the initial design of the specimens, predicted accurately the slab failure in specimens SP5 and SP6. These results will be generalized using the FEM model described later in the paper.

The maximum experimental strength is higher than the yield strength due to the hardening behavior of the YP. The ratio $F_{max,exp}/F_{y,exp}$ is approximately 2.0 in the SP1 group, 1.8 in the SP2 group, 1.6 in specimen SP3, 1.4 in the SP4 group and 1.33 in the SP6 specimen,

indicating the considerable hardening behavior of the YP. This hardening behavior of the proposed connector is different than the typical force-slip curve of a welded stud, which has a well-defined plateau before the ultimate failure.

Force-slip responses

Figs. 13 to 15 show the force versus slip responses of the specimens. The slip history plotted in these graphs is the average value recorded by the two LVDTs on the right- and left-hand side of the YP. The force corresponds to the feedback output given by the load cell of the actuator. Fig. 13 shows the response of specimens in the SP1 group. The sudden increase of the force in SP1A that occurs at 47 mm corresponds to the bearing force that was developed after the YP came in contact with the bolts (as described in the previous section). As can be seen from the graph, the force increases suddenly from 357 kN to 520 kN. The maximum force corresponds to a slip of 52 mm. The response after this point starts to soften due to the gradual failure of the slab and at a slip of 56 mm the curve drops suddenly due to the complete failure of the slab (Fig. 8c).

Following the test of SP1A that was brought to complete failure, the rest of the specimens that were designed to fail in a ductile mode, i.e. the specimens in groups SP1 to SP4, were tested up to approximately 30 mm of slip. At this slip value the tests were stopped for the following reasons: a) in all of the above tests there were no signs of cracks in the slab while the YP was plastically deforming, and, therefore, it was assumed that the failure mode would be similar to the one observed in SP1A if the slab was further pushed against the steel section; b) to avoid damaging the steel sections and slab units so that they can be reused in subsequent tests and assess the deconstructability of the system; and c) because 30 mm of slip is already a much higher value of slip than that expected in typical composite beams.

As can be seen from the force-slip graphs in Fig. 13, specimen SP1B follows exactly the

same force-slip path as the SP1A up to 34 mm of slip. Specimen SP1C has a significantly reduced initial stiffness and force compared to the other two tests of the series due to the inadequate amount of *in situ* concrete placed around the YP that resulted in its out of plane bending, as described in the previous section. The force is reduced by 40 kN, and the initial stiffness is reduced by 20 percent. This behavior is both undesirable and unpredictable, and, therefore, the YP should be fully restrained by *in situ* concrete.

Specimen SP2A was deconstructed after the test by cutting and removing the *in situ* concrete surrounding the YP, and both the steel sections and the slab units were reused for the test on SP2B with new *in situ* concrete cast around the YP. Fig. 14 shows that the original and reused specimens have equal stiffness and a slight difference in strength, i.e. SP2B is about 10 kN weaker than SP2A. The test of specimen SP3 was a standalone one, while Fig. 14 shows that specimens SP4A and SP4B had identical responses.

Fig. 15 shows the force-slip responses of specimens SP5 and 6. It can be seen that, despite the higher YP strength of specimen SP5 (345 kN), the specimen SP6 with F_{YP}= 310 kN achieved a higher stiffness and strength. The reason for this is the early concrete failure in SP5 for the reasons explained in the previous section. The slip at max force of SP5 is 8.3 mm, which is the smallest of all tests, but still higher than the 6 mm of min slip requirement in Eurocode 4. In SP6, the response starts plateauing at a slip of 19.6 mm, where the first visible diagonal crack on the slab was formed.

In Fig. 14, a comparison is made between the force-slip behavior of the proposed connector and that of a welded shear stud in HCUs tested in Lam et al. (1998). Although the comparison is not fully consistent, since the welded stud was tested in a 150-mm thick HCU and the *in situ* concrete had different properties in the study of Lam et al. (1998), it shows in a

qualitative way the significantly higher strength and slip capacity that is achieved using the YP. In

Fig. 16, the initial stiffness of specimen SP3, including the initial 25 cycles between 5 and 40

percent of the expected yield strength, is compared with the same welded shear stud in HCUs

(Lam et al. 1998), and it demonstrates that the initial stiffness of the YP is comparable to that of a

welded stud.

The LVDTs measuring the uplift showed that the vertical separation between the slab and

the steel section in specimens SP1A and SP6 was at any given time much less than half of the

horizontal slip, and thus it is considered acceptable as per Eurocode 4 (BSI 2004).

Strain recordings

The microstrain ($\mu\varepsilon$) values recorded by the strain gauge attached at the mid-width of the

concrete tooth, i.e. gauge 3 in Fig. 7, indicate almost zero strain. This indicates that the

compressive stress on the face of the YP is not significant and can be explained considering the

fact that the YP deforms in such a way that the top side of the YP and the top fibres of the

concrete tooth disconnect from each other. The strain gauges placed closer to the corners of the YP

recorded small compressive strains, i.e. gauges 2 and 4. This indicates a compressive stress field

developed from the introduction of the load from the spreader beam to the YP. The recordings of

the strain gauges installed closer to the sides of the YP, i.e. gauges 1 and 5 indicate larger tensile

strains up to 3000 $\mu\varepsilon$. This justifies the shear crack pattern observed in specimen SP6.

Fig. 17a shows the force versus microstrain graphs as recorded by the strain gauges

attached on the steel rebars of specimen SP6. The results show that the strain of the rebars started

increasing after the load reached 350 kN approximately and corresponding relative slip of 6.5

mm. The recorded strains increased rapidly from below 70 με to 400 $\mu\varepsilon$ as the load approached

the failure load of the slab due to shear failure at 400 kN applied load. The strains developed in

the rebars correspond to a stress of approximately 100 N/mm^2, while the yield stress of a rebar is 500 N/mm^2. This result indicates that the transverse rebars contributed to the shear transfer mechanism, although they did not yield.

The results from the strain gauges attached to the YP's steel strips and walls are shown in Fig. 17b. The yield strain of the YP material is approximately 1950 $\mu\varepsilon$, based on the material test results of Table 3 for the flat regions. Large strains up to 6000 $\mu\varepsilon$ developed in the top of the YP's steel strips and walls for the specimen SP1A. The YP of the specimen SP5 appeared to have limited deformation at the end of the test. The recordings of the strain gauges installed on this YP indicated minor yielding of the steel strips and onset of yielding at the transverse wall. However, these results are sensitive to the precise position of the strain gauges along the width of the steel strip and wall.

FINITE ELEMENT MODEL

Model description

The finite element analysis program Abaqus (Dassault Systémes 2014) was used to perform numerical simulations of the push tests. Due to symmetric geometry, only half of a push test specimen was built and is shown in Fig. 18. The concrete slab, YP, bolts, and steel section were modelled using the 8-node reduced integration brick finite element C3D8R. The transverse rebars were modelled using the 2-node linear truss element T3D2. The mesh of the YP and the concrete slab around the YP was refined to give accurate predictions of the failure modes and force-slip responses. Fully fixed boundary conditions were assigned to the bottom of the steel beam.

Displacement was imposed on the 'Loading' surface shown in Fig. 18. The displacement

was applied slowly by a smooth amplitude function, which allows arbitrary time variations of displacement to be given throughout the analysis. The solution was obtained using the dynamic explicit method since it is very efficient in modelling complex nonlinear problems. Hard contact and penalty friction formulations were used to describe the normal and the tangential behavior of concrete interfaces in contact with the YP and of the bolts in contact with the welded plate and the flange of the steel section. The friction coefficient was chosen as 0.4 in the contact boundary between steel and concrete and 0.25 between steel components. Tie constraints were imposed between the HCUs and the *in situ* concrete filled cores. The transverse rebars were embedded in the *in situ* concrete cores of the HCUs, and therefore perfect bond between the rebars and the surrounding concrete was assumed in the simulations.

Material modelling

The material model for steel was defined using an elastic-plastic constitutive law. The stress-strain relationship of the YP's steel material followed the uniaxial tensile coupon tests, which can be simplified as bi-linear curves. A bilinear elastic-perfectly plastic stress-strain relationship was used for the rebars, bolts and base plate of YP using nominal material values.

The uniaxial compressive stress - strain behavior of concrete was defined according to the relationship provided in FIB (2010), including the descending (softening) branch after the peak compressive strength, f_c' , has been reached. For un-cracked concrete subjected to tension, a linear stress-strain relationship was adopted when the stress is less than the tensile stress, $f_t = 1.4(f_c'/10)^{2/3}$. The Young's modulus in tension was assumed to be the same as that in compression. The post-failure behavior for direct straining across cracks was modelled using the tension stiffening option and determining a linear reduction until stress is zero at a strain value of 10 times the strain at f_t. The damaged plasticity model was used to define the plastic behavior of

concrete (Dassault Systémes 2014). The parameters of the damaged plasticity model were set to their recommended values, i.e., dilation angle ψ= 30°; flow potential eccentricity ϵ= 0.1, viscosity parameter μ= 0; biaxial to uniaxial compressive strength ratio σ_{b0}/σ_{c0}= 1.16; and ratio of second stress invariant on the tensile meridian to that on the compressive meridian K= 2/3.

FEM model validation

The force versus slip curves resulting from the numerical analyses of the push specimens are plotted together with the experimental curves in Fig. 19. Comparison of the results shows that the experimentally obtained strength and stiffness of the shear connection can be predicted by the numerical solutions with good accuracy. Note that the analysis of specimen SP1A was stopped at a slip of 47 mm, i.e. when the transverse wall of the YP came in contact with the row of bolts close to the loaded side of the specimen.

Table 5 summarizes the yield strength, maximum strength, and slip at max force, obtained from the tests and numerical analyses. The numerical to experimental yield and maximum strength ratios have mean values 1.0 and 1.01 and standard deviations 0.05 and 0.05, respectively; while the numerical to experimental relative slip at max force ratio has mean value 1.04 with standard deviation 0.05. The maximum difference between the experimental and numerical maximum strength is 9% for specimen SP5, and it is attributed to eccentricities that were introduced in the application of the loading due to the imperfections on the surface of the specific HCUs (as described in previous section) and in turn resulted in secondary moments and torsional effects, which were extremely difficult to be simulated in the FEM analyses. Thus, the proposed finite element model can reasonably predict the yield strength, ultimate strength and relative slip of the YPs.

Fig. 20 shows contour plots of the equivalent plastic strain (PEEQ) on the deformed shape

of the YPs at the end of the analyses of specimens SP1A, SP5 and SP6. Plastic deformation concentration occurs at the ends of the steel strips and at the ends of the YP's walls for SP1A; the base plate also developed some plastic deformation. Minimum plastic deformation was observed in the YP of specimen SP5, and plastic deformation has occurred for specimen SP6 but mush less than in test SP1A. These observations are consistent with the experimental evidence and the strain recordings of the gauges placed on the YPs.

Fig. 21 shows the principal tensile strain distribution in the slabs of specimens SP1A, SP5 and SP6. In specimen SP1A, no plastic strains were observed in the concrete slab until the slip of 47 mm, indicating that the concrete slab did not fail up to that point, as observed in the corresponding push test. The relative magnitude and orientation of the plastic strains of SP5 and SP6 indicate that the slab failure zone in the model is consistent with the concrete failure modes observed during the tests, i.e. in the corners of the YPs towards the spreader beam.

From the above comparisons between numerical and experimental results, it is concluded that the FEM model is reliable and can be used for further parametric studies.

PARAMETRIC STUDY

Effect of *in situ* concrete in the core before the YP

To investigate the effect of *in situ* concrete in the core just before the YP, i.e. the core 4s in Fig. 6, on the strength and failure modes of the specimens, the *in situ* concrete was removed from core 4s and the analyses rerun, keeping the other parameters identical.

Fig. 22 shows a comparison of force-slip curves for all test specimens with or without *in situ* concrete in core 4s, denoted as W-4s and W/O-4s, respectively. The *in situ* concrete in core 4s does not affect the stiffness of the shear connection in the elastic stage; however, it improves the

strength and ductility of the shear connection. The ratios of ultimate strengths of specimens SP1 to SP6 with *in situ* concrete in core 4s to those without are 1.10, 1.21, 1.29, 1.04, 1.33, and 1.12 respectively. From the graphs of Fig. 22a (SP1 to SP4), it can be seen that there is significant improvement in the post-elastic response of specimens, and particularly in specimens SP3 (with *in situ* concrete strength 38 N/mm^2) and SP2 (with *in situ* concrete strength 47 N/mm^2). Notably, specimen SP3 without *in situ* concrete in core 4s failed immediately after the end of the elastic stage, at a slip of around 6 mm, showing no ductile behavior. The effect is also significant in specimens SP5 and SP6, as shown in Fig. 22b, with SP5 showing a much weaker and less ductile response when there is no *in situ* concrete in core 4s.

The placement of *in situ* concrete in the core before the YP affected the failure modes of the specimens. Fig. 23 shows the failure modes of specimens SP4 and SP5 with or without *in situ* concrete in core 4s, in terms of maximum principal plastic strain distribution. In specimen SP4, the failure mode changes from concrete shear failure to coupled shear and ripping failure when *in situ* concrete is removed from the core 4s. In specimen SP5, the failure mode changes from shear failure of concrete slab to compressive crushing at the local area of the "concrete tooth" between the YP and core 4s when *in situ* concrete was removed from the core 4s.

Effect of *in situ* concrete strength

To investigate the effect of *in situ* concrete strength on the response of the proposed demountable shear connector, the compressive strength of *in situ* concrete in the cores was varied from 25 to 60 N/mm^2, i.e. f_c= 25, 30, 40, 50, and 60 N/mm^2 for all the specimens (SP1 to SP6). Except for the *in situ* concrete strength, all other parameters were the same as the tested specimens.

Fig. 24 shows the force versus slip responses of all specimens with different *in situ* concrete strength. The *in situ* concrete strength does not affect the initial stiffness of the shear

connection; however, it slightly affects the ultimate shear strength and significantly affects the ductility of the specimen. The curves in Fig. 24a indicate that specimen SP1 always fails due to YP yielding, i.e. without any type of concrete failure involved, regardless of the concrete strength used. In specimens SP2, SP3 and SP4 (Figs. 24b, c, d), concrete failure occurs earlier when f_c= 25 or 30 N/mm^2, while for the rest of the *in situ* concrete grades the responses are almost identical to the experimental response.

There is a much more significant effect of the *in situ* concrete strength on specimens SP5 and SP6. As can be seen from Figs. 24e and f, increasing the *in situ* concrete strength delays the failure of concrete. For example, if *in situ* concrete with f_c= 60 N/mm^2 were used, then both SP5 and SP6 would be able to reach more than 30 mm of slip and more than 400 kN of ultimate strength.

Effect of an additional transverse rebar in the core before the YP

The effect of placing an additional 12-mm-diameter transverse rebar in core 4s was investigated. For this purpose, the specimens that experienced shear failure of the concrete slab, i.e. specimens SP2 to SP4 with *in situ* concrete strength 25 N/mm^2, SP5 and SP6, were re-analysed with the additional rebar installed.

Fig. 25 plots the force versus slip responses of specimens with or without additional rebar in core 4s. The additional rebar contributes slightly to the ultimate strength of the specimens, and it increases the slip capacity (and hence the ductility) of the shear connection by delaying the concrete failure. For example, the slip at ultimate force for SP6 increased from 21 mm to 26.2 mm (about 25% increase) when adding an additional 12-mm diameter transverse rebar in core 4s).

The FEM model showed that the additional rebar redistributes the load to the cores. For

example, the normal stresses in rebars in cores 1s and 3s (see Fig. 6) for specimen SP6 are 251 N/mm^2 and 135 N/mm^2, respectively, but reduce to 227 N/mm^2 and 110 N/mm^2 after adding the rebar in core 4s, while the normal stress in the additional rebar is 210 N/mm^2.

EVALUATION OF DESIGN EQUATIONS

The push tests and the FEM parametric analyses showed that two main failure modes control the ultimate strength of the proposed demountable shear connector: the ductile failure of the YP and the failure of the concrete slab. The correlation of F_{YP} from Eq. 5 with the experimental and numerical yield strength ($F_{Y,exp}$ or $F_{Y,FEM}$) is shown in Fig. 26a. The mean value of $F_{YP}/(F_{YP,FEM}$ or $F_{YP,exp})$ is 0.95 with standard deviation 0.06. The correlation of F_{conc} from Eq. 6 with the experimental and numerical strength of specimens that failed due to shear cracking in the slab ($F_{conc,exp}$ or $F_{conc,FEM}$) is shown in Fig. 26b. The mean value of $F_{conc}/(F_{conc,exp}$ or $F_{conc,FEM})$ is 0.91 with standard deviation 0.12. The above experimental and numerical agreement with the design equations show that Eq. 5 can be used to predict the yield strength of the YP with good accuracy, while Eq. 6 can be conservatively used as an upper bound for concrete shear failure, since it slightly underestimates the concrete failure. In addition, the condition $F_{YP}/F_{conc}<1$ should be used to ensure a ductile failure mode and at least 6 mm of slip of the proposed demountable shear connector, provided that the cores before and after the YP are filled with *in situ* concrete.

CONCLUSIONS

A steel-yielding demountable shear connector, denoted as yielding pocket or YP, was proposed for use in composite floors with precast hollow core slab units (HCUs). The proposed connector was experimentally validated through ten full-scale push tests that were conducted in a horizontal

testing arrangement and using sections extracted from a prototype composite floor. A numerical model was calibrated to the experimental results and used to carry out a number of parametric studies. Based on the findings of the experimental and numerical study, the following conclusions can be drawn:

- The YP can provide an adequate shear connection between the steel section and the HCUs if properly designed. The experimental results show that a YP can be designed to have high strength, high initial stiffness, and very high slip capacity.

- The YPs provided longitudinal shear resistance in the range of 170-320 kN in the tests reported herein. Specimens designed to fail in a ductile mode had at least 30 mm slip capacity. Specimens that failed due to concrete shear cracking still achieved a slip capacity much higher than that required by Eurocode 4, i.e. higher than 6 mm.

- Two main failure modes control the strength and ductility of the proposed shear connection, i.e. the yielding of the YP walls and vertical strips, and the shear failure of the slab units.

- The cores of the HCUs before and after the YP should be filled with *in situ* concrete in order to avoid premature concrete failure and to ensure that the longitudinal shear force can be transferred from the YP to the concrete component.

- The strength of the *in situ* concrete does not affect the stiffness and strength of the shear connection, but it significantly affects its ductility.

- An additional transverse rebar in the core before the YP has small effect on the strength and slip capacity of the shear connection.

- The YP should be adequately restrained by the *in situ* concrete on all sides, as inadequate restraint will cause out of plane bending that results in lower strength and stiffness.

- The deconstruction was assessed by reusing the steel section and the HCUs of specimen SP2A

to specimen SP2B. The dismantling of the specimen was implemented in the Lab environment, and the force-slip responses of the two specimens were within ten percent difference.

- The strength of the YP can be accurately predicted using a plastic analysis of the device, and the strength of the concrete slab can be conservatively predicted using an equation based on the shear strength of a cracked plane, adopted from current codes of practice. In addition, the condition that the strength of the YP should be less than that of the shear strength of the concrete slab is a reliable predictor of ductile force-slip response of the shear connector.

REFERENCES

ASTM (2011). *E8/E8M – 11, Standard Test Methods for Tension Testing of Metallic Materials*. ASTM Standards.

Ataei, A., Bradford, M. A., and Liu, X. (2016). "Experimental study of composite beams having a precast geopolymer concrete slab and deconstructable bolted shear connectors." *Engineering Structures*, 114, 1–13.

Ban, H., Uy, B., Pathirana, S. W., Henderson, I., Mirza, O., and Zhu, X. (2015). "Time-dependent behaviour of composite beams with blind bolts under sustained loads." *Journal of Constructional Steel Research*, 112, 196–207.

Brambilla, G., Lavagna, M., Vasdravellis, G., Castiglioni, C.A. (2019). "Environmental benefits arising from demountable steel-concrete composite floor systems in buildings". *Resources, Conservation & Recycling*, 141, 133-142.

BSI (2002a). *EN1992-1-1. Eurocode 2. Design of concrete structures. Part 1-1. General rules and rules for buildings*. British Standards Institution, European Committee for Standardization (CEN), Brussels.

BSI (2002b). *Eurocode 1: Actions on structures. Part 1-1: General actions - Densities, self-weight, imposed loads for buildings*. British Standards Institution, European Committee for Standardization (CEN), Brussels.

BSI (2004). *EN1994-1-1. Eurocode 4. Design of composite steel and concrete structures. Part 1-1. General rules and rules for buildings*. British Standards Institution, European Committee for Standardization (CEN), Brussels.

BSI (2009). *BS EN 12390-6:2009. Testing hardened concrete, Part 3: Compressive strength of test specimens*. British Standards Institution, London.

Burka, L. P. (1993). "A hypertext history of multi-user dimensions." *MUD history*, <http://www.ccs.neu.edu> (Dec. 5, 1994).

Couchman, G. (2014). "Design of composite beams using precast concrete slabs in accordance with eurocode 4." *Steel Construction Institute Publication P401*.

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*, 141(11), 04015025.

Dassault Systémes (2014). *ABAQUS documentation*. Dassault Systémes Simulia, Providence, RI, 6.14 edition.

Densley Tingley, D. and Davison, B. (2011). "Design for deconstruction and material reuse." *ICE Energy*, 164(11), 195–204.

FIB (2010). *First complete draft. Vol. 1. Comite Euro-International du Beton*. Model Code 2010, Secretariat permanent, Case Postale 88, CH-1015 Lausanne, Switzerland.

Henderson, I., Zhu, X., Uy, B., and Mirza, O. (2015a). "Dynamic behaviour of steel-concrete composite beams with different types of shear connectors. part i: Experimental study." *Engineering Structures*, 103(Supplement C), 298 – 307.

Henderson, I., Zhu, X., Uy, B., and Mirza, O. (2015b). "Dynamic behaviour of steel-concrete composite beams with different types of shear connectors. part ii: Modelling and comparison." *Engineering Structures*, 103(Supplement C), 308 – 317.

Henderson, I., Zhu, X., Uy, B., and Mirza, O. (2017). "Dynamic behaviour of steel-concrete composite beams retrofitted with various bolted shear connectors." *Engineering Structures*, 131(Supplement C), 115 – 135.

IEA (2007). "Tracking industrial energy efficiency and CO2 emissions." International Energy Association.

Lam, D. (2004). "Capacities of headed stud shear connectors in composite steel beams with precast hollow core slabs." *Journal of Constructional Steel Research*, 63(9), 1160–74.

Lam, D., Elliot, K., and Nethercot, D. (1998). "Push-off tests on shear studs with hollow-cored floor slabs." *The Structural Engineer*, 76(9).

Lam, D. and Saveri, E. (2012). "Shear Capacity of Demountable Shear Connectors." *Proceedings of the 10th International Conference on Advances in Steel Concrete Composite and Hybrid Structures*, 767–774.

Liu, X., Bradford, M. A., and Ataei, A. (2017). "Flexural performance of innovative sustainable

composite steel-concrete beams." *Engineering Structures*, 130(Supplement C), 282 – 296.

Liu, X., Bradford, M. A., and Lee, M. S. S. (2015). "Behavior of high-strength friction-grip bolted shear connectors in sustainable composite beams." *Journal of Structural Engineering*, 141(6), 04014149.

Moynihan, M. C. and Allwood, J. M. (2014). "Viability and performance of demountable composite connectors." *Journal of Constructional Steel Research*, 99(Supplement C), 47 – 56.

Oehlers, D. J. and Bradford, M. A. (1995). *Composite steel and concrete structural members. Fundaental Behviour.* Pergamon, Elsevier Science Ltd., The Boulevard, Langford Lane, Kidlington, Oxford, U.K.

Pathirana, S. W., Uy, B., Mirza, O., and Zhu, X. (2015). "Strengthening of existing composite steel-concrete beams utilising bolted shear connectors and welded studs." *Journal of Constructional Steel Research*, 114, 417–430.

Pathirana, S. W., Uy, B., Mirza, O., and Zhu, X. (2016a). "Bolted and welded connectors for the rehabilitation of composite beams." *Journal of Constructional Steel Research*, 125(Supplement C), 61 – 73.

Pathirana, S. W., Uy, B., Mirza, O., and Zhu, X. (2016b). "Flexural behaviour of composite steel-concrete beams utilising blind bolt shear connectors." *Engineering Structures*,114(Supplement C), 181 – 194.

Pavlovic, M., Markovic, Z., Veljkovic, M., and Bucevac, D. (2013). "Bolted shear connectors vs. headed studs behaviour in push-out tests." *Journal of Constructional Steel Research*, 88(Supplement C), 134 – 149.

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(Supplement C), 178 – 189.

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*, 22(9), 04017052.

Webster, M. and Costello, D. (2005). "Designing structural systems for deconstruction: how to extend a new building's useful life and prevent it from going to waste when the end finally comes." *Proc., Green Building Conference*, Atlanta, USA.

Zona, A. and Ranzi, G. (2014). "Shear connection slip demand in composite steel–concrete beams with solid slabs." *Journal of Constructional Steel Research*, 102, 266 – 281.

Table 1. The test matrix

Specimen	YP geometry (mm)			Concrete cube strength (N/mm^2)	F_{YP} (kN, Eq. 5)	F_{conc} (kN, Eq. 6)	F_{YP}/F_{conc}
	t	w	h				
SP1A	8	17	75	61	150	440	0.34
SP1B	8	17	75	52	150	388	0.39
SP1C	8	17	75	50	150	370	0.41
SP2A	8	13	55	47	186	370	0.50
SP2B	8	13	55	56	186	434	0.43
SP3	8	17	55	38	215	333	0.65
SP4A	10	14	75	68	263	440	0.6
SP4B	10	14	75	47	263	360	0.73
SP5	10	18	65	35	345	295	1.17
SP6	10	14	65	30	310	280	1.11

Table 2. *In situ* concrete material properties

Specimen	Age at testing (days)	Compressive strength (N/mm^2)	Splitting tensile strength (N/mm^2)
SP1A	27	61	4.3
SP1B	17	52	3.1
SP1C	11	50	2.9
SP2A	7	47	2.9
SP2B	17	56	3.6
SP3	12	38	2.5
SP4A	11	68	4.5
SP4B	35	47	2.80
SP5	10	35	2.1
SP6	12	30	1.9

Table 3. YP's steel material properties (average values of three coupon tests)

Coupon region	Modulus of elasticity (N/mm^2)	0.2% proof stress (N/mm^2)	Tensile stress (N/mm^2)	Strain at ultimate stress
Flat region SHS180x8	193600	390	460	0.168
Corner region SHS180x8	221000	390	425	0.02
Flat region SHS180x10	202850	500	565	0.087
Corner region SHS180x10	215500	525	590	0.028

Table 4. Experimental strength and failure modes

Specimen	$F_{y,exp}$ (kN)	$F_{max,exp}$ (kN)	Slip at max force (mm)	$F_{y,exp}/F_{YP}$	$F_{max,exp}/F_{y,exp}$	Failure mode
SP1A	170	357	47	1.13	2.10	Ductile
SP1B	170	325	34	1.13	1.91	Ductile
SP1C	160	250	30	1.07	2.23	Ductile
SP2A	200	356	35	1.08	1.78	Ductile
SP2B	200	339	35	1.08	1.79	Ductile
SP3	230	359	36	1.07	1.56	Ductile
SP4A	260	378	33	0.99	1.37	Ductile
SP4B	260	375	33	0.99	1.44	Ductile
SP5	N.A.	321	8.3	N.A.	N.A.	Slab shear failure
SP6	320	427	19.6	1.03	1.33	Mixed

Table 5. Comparison of experimental and numerical results

Specimen	Yield strength (kN)			Ultimate strength (kN)			Slip at max force (mm)		
	Test	FEM	Test/FEM	Test	FEM	Test/FEM	Test	FEM	Test/FEM
SP1A	170	185	0.92	357	350	0.98	47	45	0.96
SP2A	200	190	1.05	356	341	0.96	35	36	1.03
SP3	230	225	1.02	359	357	0.99	36	37	1.03
SP4A	260	267	0.97	378	385	1.02	33	35	1.06
SP5	N.A.	N.A.	N.A.	321	350	1.09	8.3	9	1.08
SP6	320	300	1.07	427	417	0.98	19.6	21	1.07
Mean:			1.0			1.01			1.04
St. dev.:			0.05			0.05			0.05

List of Figures