Steel and Composite Structures, *Vol. 29, No. 1 (2018) 107-123*
DOI: https://doi.org/10.12989/scs.2018.29.1.107

Removable shear connector for steel-concrete composite bridges

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(*Received May 25, 2018, Revised July 3, 2018, Accepted July 31, 2018*)

Abstract. The conception and experimental assessment of a removable friction-based shear connector (FBSC) for precast steel-concrete composite bridges is presented. The FBSC uses pre-tensioned high-strength steel bolts that pass through countersunk holes drilled on the top flange of the steel beam. Pre-tensioning of the bolts provides the FBSC with significant frictional resistance that essentially prevents relative slip displacement of the concrete slab with respect to the steel beam under service loading. The countersunk holes are grouted to prevent sudden slip of the FBSC when friction resistance is exceeded. Moreover, the FBSC promotes accelerated bridge construction by fully exploiting prefabrication, does not raise issues relevant to precast construction tolerances, and allows rapid bridge disassembly to drastically reduce the time needed to replace any deteriorating structural component (e.g., the bridge deck). A series of 11 push-out tests highlight why the novel structural details of the FBSC result in superior shear load-slip displacement behavior compared to welded shear studs. The paper also quantifies the effects of bolt diameter and bolt preload and presents a design equation to predict the shear resistance of the FBSC.

Keywords: shear connectors; steel-concrete composite bridges; slip capacity; shear resistance

1. Introduction

Departments of transportation worldwide are in the need of more efficient methods for retrofit, widening and replacement of bridges so that they can reduce the socio-economic losses associated with long disruptions to traffic flow (e.g., delays in move of goods and people, business interruption, etc.). This need is of significant importance as many bridges in Europe and the USA show signs of significant deterioration (PANTURA 2011, ASCE 2014). Moreover, urbanization and climate change are expected to increase traffic flow, the allowable weight of vehicles, and the corrosiveness of the environment, and therefore, impose further demands on the bridge infrastructure. The concrete decks of a large number of bridges in America (i.e., close to 33% of the existing bridge stock) are typically replaced after 40 years of service life (ASCE 2014). Deterioration of the decks of bridges is also a concern in the UK (Long *et al*. 2008). Typically, replacement of the deck is preferable to repair as it guarantees a long extension of the bridge lifespan (Deng *et al*. 2016). However, replacement of the deck is not a straightforward process, especially in the case of steel-concrete composite bridges that use shear studs to achieve composite action. Shear studs are welded on the top flanges of the steel beams and are fully embedded within the deck, and therefore, replacement of the latter can only be achieved through a costly and time-consuming process involving crushing and drilling of concrete (Tadros and Baishya 1998).

A possible way to address the difficulties in replacement of the deck of composite bridges is to move towards a new design philosophy that will allow rapid bridge disassembly. Given the ability for disassembly, any structural component of a composite bridge (e.g,. bridge deck, shear connector, steel beam) that has experienced severe deterioration due to long-term stressors (e.g., corrosion, fatigue) would be rapidly replaced without long disruption to traffic flow. Apart from rapidly replacing deteriorating structural components, disassembly would also offer significant benefits for widening or any other changes in the size/geometry of the bridge so that new service requirements can be provided. To achieve disassembly in composite bridges, removable shear connectors shall be used instead of the non-removable welded shear studs. It is noted that composite bridges using dry joints (Hallmark 2012) between adjacent precast concrete slab panels would further enhance the potential for disassembly. In the latter case, separating one slab panel from the other would also be a straightforward process.

2. Background

When searching for an option alternative to welded shear studs to achieve disassembly, the obvious solution would be a bolted connection. Dallam (1968), Dallam and Harpster (1968), and then Marshall *et al*. (1971) experimentally investigated the use of high-strength friction-grip bolts as shear connectors. Dedic and Klaiber (1984) assessed the effectiveness of high-strength bolts to

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http://www.techno-press.org/?journal=scs&subpage=6
ISSN: 1229-9367 (Print), 1598-6233 (Online)

rehabilitate bridges and other structures. Kwon *et al*. (2010, and 2011) conducted a series of experiments on three configurations of post-installed shear connectors used to retrofit non-composite bridges. Their tests showed significant increases in strength and stiffness of the bridge as well as good fatigue performance. Mirza *et al*. (2010), Pathirana *et al*. (2013, 2015, 2016), Ban *et al*. (2015) and Henderson *et al*. (2015a, b) investigated the use of blind bolts as shear connectors with the goal of achieving demountable composite beams and found adequate structural performance. In parallel with the previous work in Australia, extensive research work has been conducted in the UK by Lam and Saveri (2012), Lam *et al*. (2013), Moynihan and Allwood (2014), Dai *et al*. (2015), and Rehman *et al*. (2016) on bolted shear connectors machined from standard welded studs. Pavlović (2013) and Pavlović *et al*. (2013) investigated the use of bolts as shear connectors and revealed low stiffness in the shear load - slip displacement behavior. The use of high-strength friction-grip bolts as shear connectors has been also assessed by Lee and Bradford (2013), Rowe and Bradford (2013), Bradford and Pi (2012), Ataei and Bradford (2014), Chen *et al*. (2014), Liu *et al*. (2014), and Ataei *et al*. (2016). Recently, Feidaki and Vasdravellis (2017) developed a highly ductile demountable steel hollow rectangular section as a shear connector for hollow-core precast slabs.

3. The problem

The results of previous works on the use of friction-grip bolts as shear connectors show a sudden slip of the bolts within their bolt holes when the shear load overcomes the friction resistance in the concrete slab - steel beam interface. This sudden slip is the main reason that the pre-standard of Eurocode 4 (BSI 1994) imposed major restrictions on the use of friction-grip bolts as shear connectors. In particular, that pre-standard allowed the designer to assume that the shear resistance is equal to the sum of the friction resistance in the concrete slab - steel beam interface and the shear resistance of the bolt itself only if this has been verified by testing. It should be noted that the final standard of Eurocode 4 (BSI 2004 and 2005a) does not cover friction-grip bolts or any other types of bolts as shear connector but provides design recommendations only for welded shear studs. An interesting discussion on the use of friction-grip bolts as shear connectors is provided by Johnson and Buckby (1986). One of their most important comments is that to exploit the full shear resistance of friction-grip bolts, grouting of the gaps among the bolts and the precast slabs shall take place after bolt tightening. This is to ensure that the bolt will bear onto the precast slab just after friction resistance in the concrete slab - steel beam interface is exceeded.

Full contact between the steel beam and a precast slab is not a realistic assumption due to their usual imperfections such as lack of straightness, lack of flatness, lack of fit and other minor eccentricities (BSI 2005b). Therefore, if bolts are used as shear connectors, adequate bolt fastening may not be possible, and cracks may be developed in the slab (Biswas 1986). It is also noted that bridge disassembly and replacement of the shear connectors in case of damage due to fatigue or corrosion cannot be achieved when bolts are embedded within the concrete.

The previous paragraphs reveal that previously proposed shear connectors using friction-grip bolts have certain disadvantages that may hinder their potential for practical application. To overcome such disadvantages and offer additional advantages, two demountable shear connectors were developed, namely the locking nut shear connector (LNSC) and the friction-based shear connector (FBSC). Although similar in overall geometry, the two shear connectors have different shear load transfer mechanisms. The development and experimental evaluation of the LNSC have been presented in detail in previous publications (Suwaed and Karavasilis 2017a and b). This paper presents the development and experimental evaluation of the FBSC using 11 pushout tests.

4. The FBSC

The FBSC is intended to be used in precast steel-concrete composite bridges, such as the one shown in Fig. 1, where prefabricated concrete panels are placed on the top of steel beams. As it shown in Fig. 1, FBSCs are placed within holes (also known as pockets) of the precast panels. The FBSC consists of several components, which are indicated in the 3D inside view of Fig. 2 as well as in the cross-section of Fig. 3. One of the main ideas that led to the development of the FBSC was to minimize the slip of the

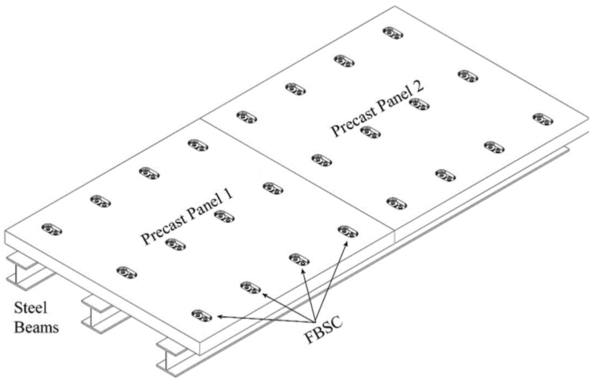

Fig. 1 Precast steel-concrete composite bridge using the FBSC

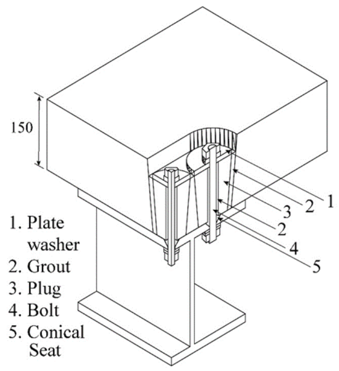

Fig. 2 3D inside view of the FBSC

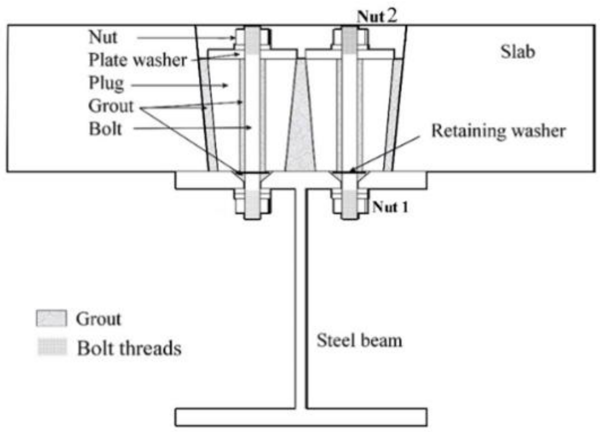

Fig. 3 Cross-section of a steel-concrete composite beam using the FBSC

slab with respect to the steel beam under service loading (i.e. < 50% of ultimate load). This was achieved by developing friction resistance in the interface between the concrete plug (see Fig. 3) and the upper face of the top flange of the steel beam. Obviously, the latter mechanism could be very beneficial in reducing fatigue effects that occur in other shear connectors, such as welded studs, under repeated traffic loading.

Fig. 3 shows that the FBSC uses a pair of high strength steel bolts (e.g., Grade 8.8 or higher), which have a smooth shank with 20 mm threaded ends. These bolts pass through chamfered countersunk seat holes drilled on the top flange of the beam. The bolts are placed in their final position with the aid of retaining washers designed per BS EN 3386 (BSI 2012). Fig. 4 shows a sample M16 retaining washer. The latter consists of a radial mounting shape having an external diameter equal to or less than the external diameter of the

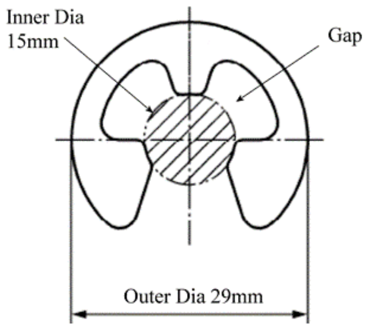

Fig. 4 Dimensions of retaining washer

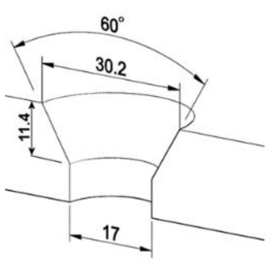

Fig. 5 Geometry of half countersunk hole

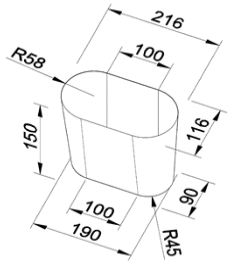

Fig. 6 Dimensions of slab pocket

chamfered countersunk seat hole; internal diameter equal to the bolt diameter minus 1mm; and several radial gaps. The main role of these washers is to hold the bolts in position prior to other important fabrication steps, which are described in the following paragraphs.

Fig. 5 displays the details of the chamfered countersunk seat which is similar to that of the LNSC (Suwaed and Karavasilis 2017a, b) with geometry that follows an angle of 60 degrees. It should be noted that bolt threads should be kept below the chamfered countersunk seat hole (Fig. 3). In this way, shear failure within the weak threaded length of the bolt (a failure seen in other types of bolt shear connectors like those recommended by Pavlović (2013), Kwon *et al*. (2011), and Dedic and Klaiber (1984)) is prevented. The reason for promoting such detailing was concluded from double shear tests on bolts that have shown a 30% increase in shear resistance when failure occurs in the shank instead of the threads of the bolt (Pavlović 2013, Chesson *et al*. 1965). A lower standard hexagonal nut (BSI 2005c) is used along with a hardened chamfered washer (BSI 2005d) as shown in Figs. 2 and 3. A 70-100% proof load is applied between the lower nut and the upper nut. The proof load represents 70% of the ultimate capacity of the bolt according to BSI (2009a).

The pocket of the slab has an inclination of 5 degrees based on recommendations available in the literature (Vayas and Iliopoulos 2014). The dimensions of the slab pocket of the test specimens described later on are provided in Fig. 6. Two inverted conical precast concrete plugs (see Figs. 2 and 3) with 5 degrees inclination angle (i.e., equal to that of the slab pocket) are inserted within the slab pocket. Fig. 7 illustrates the geometry of a plug with respect to the test specimens presented later on. To accommodate an M16 bolt with 10 mm clearance, the plugs have a circular hole with a 26 mm diameter. A well-known disadvantage of welded studs is that they impose highly concentrated loads into the concrete slab (Oehlers and Bradford 1995). Such loads will usually result in premature longitudinal shear failure and/or splitting of the concrete slab. Contrary to that, the relative increase in plug diameter as compared to a stud diameter ensures that the force exerted by the FBSC to the concrete slab is less concentrated. Furthermore, the plugs have diameter that is considerably smaller to that of the slab pocket. Therefore, the FBSC has fewer construction tolerance issues typically encountered during the construction of precast composite bridges (Hallmark 2012).

Grout is used to fill the gaps between the bolt and the plug as well as the gaps between the plugs and the slab pocket (see Figs. 2 and 3). In order to ensure grouting of such small annular space without developing voids, a flowable grout is recommended. This grout consists of, based on trial mixes, 1:1 Portland cement (Quickcem from Hanson): fine sand (internal plastering sand), and 0.5 w/c. Quickcem cement is basically an ordinary Portland cement but with fast setting and hardening characteristics. Its workability duration is 7 minutes, which makes it ideal to reduce the specimen construction time, otherwise ordinary Portland cement could be used. The 0.5 w/c ensures a flowable grout without bleeding. Fine sand (for internal plastering) is a vital requirement to avoid possible segregation of sand particles between the lower face of the plug and the upper face of the steel flange. The existence of radial gaps in the retaining washers (see Fig. 4) ensure that grout will flow into the chamfered countersunk seat holes and into the clearance gaps between the bolts and their holes in the steel beam. In this way the bolts are locked into the steel beam, and therefore, they do not experience sudden slip when friction resistance between the concrete plug and the steel beam is overcome.

Grout is used to ensure the development of dowel action, i.e., resistance of shear force through bending of the bolt before its fracture (Oehlers and Bradford 1995). In particular, the grouted gaps will allow the bolt to deflect in bending and shear. Moreover, the grout will work as a cushion that distributes the shear stresses of the bolt to the stronger concrete plug (80-100 MPa compressive strength). As was previously explained, the slip of the bolt within its hole is prevented by ensuring that the bolt is locked within a fully grouted countersunk seat. It has been shown by Oehlers (1980) that the strength of a stud depends on the concrete strength in the vicinity of its welded collar. This is actually the reason of using high strength concrete (preferably 80-100 MPa cubic compressive strength) for the plug. The height of the plug is 115 mm (i.e., less than the 150-mm height of the slab) to allow for additional cover or waterproof grout.

The hardened plate washers are shown in Fig. 3. The main role of these washers is to distribute the concentrated high preload of the bolt to the concrete plug without crushing it. These washers were manufactured from EN24T steel according to BS EN 10204 and have a 90 mm outer diameter, 18 mm inner diameter, and 10 mm thickness. The 10 mm thickness was necessary to increase the stiffness of the washer against bending under the proof load of the bolt. This is essential to minimize stress concentration on the upper face of the concrete plug. The 90 mm outer diameter was chosen to decrease the compressive stress in the concrete plug to less than 50% of its ultimate compressive strength when the bolt is preloaded to the proof load. The 18 mm inner diameter was used to accommodate M16 bolts with 2 mm clearance. The plate washers have a nominal tensile strength of 1000-1150 MPa and yield strength of 850 MPa. Tightening of Nut 2 (see Fig. 3) should be performed after grouting and before the grout hardens to ensure adequate bolting and avoid cracking in the slab due to imperfections in the concrete slab - steel beam interface (Badie and Tadros 2008).

Different configurations of the FBSC could be adopted. For example, one bolt in one plug within a single slab pocket would reduce the amount of grout. Alternatively, four bolts in a single plug within a single slab pocket would significantly increase the shear strength of the FBSC, and thus, reduce the number of the shear connectors needed for a particular precast composite bridge design.

4.1 Bolt preload loss

FBSC exploits the friction resistance between the lower face of the concrete plug and the upper face of the top flange of the steel beam. Friction resistance is achieved with bolt axial pretension. Thus, loss of bolt pretension with time, due to concrete creep, concrete shrinkage, and bolt steel relaxation, is a technical issue. Although this paper does not assess time-dependent effects on the behavior of the FBSC, the following recommendations were extracted from previously published research and development: (1) Tighten the bolts to a larger extent than that dictated by formal design calculations to account for loss in bolt tension with time (Nah *et al*. 2010). For example, design the FBSC for 60% of proof load pretension and tighten the bolts to 100% of proof load by assuming a 40% loss with time. (2) A large percent of loss in bolt pretension occurs during the first 24 hours after tightening (Heistermann 2011). Re-tightening after one day may reduce the loss in bolt pretension according to BS EN 1994-1-1 (BSI 1994). However, retightening bolts after one day might be, in some cases, time consuming and costly. (3) Use of high strength concrete with a high aggregate/paste ratio in plugs ensures less creep (Johnson 1967, Oehlers and Bradford 1995). (4) Precast concrete eliminates/reduces the effect of shrinkage in concrete. (5) High strength steel (Grade 8.8, 9.8, 10.9, or 12.9) for bolts helps to maintain bolt tension. (6) Use of special ‘spring’ washers that can restore the loss in bolt tension (e.g., the commercial Bellville washers). (7) Use of special ‘locking’ nuts or washers that prevent nut loosening over time (e.g., the commercial NordLock-washers). (8) Use Tension Control Bolts (TCB) with electrical wrenches to reduce nut self-loosening and shank torsional relaxation.

4.2 Accelerated bridge construction and deconstruction

The execution of a composite bridge using the FBSC involves prefabrication of all structural components in the shop (i.e., drilling of the chamfered holes, positioning of the bolts on the steel beams by fastening the washer-nut configuration, casting of precast concrete plugs, and casting of precast slabs) and rapid assembly on site. In particular, after placing the precast panels on the steel beams, flowable grout is poured into the slab pocket. Then the plugs are inserted into the slab pocket, and in that way, all gaps are filled with grout easily. Tightening of Nut 2 in Fig. 2 before the grout hardens completes the construction. It is noted that the FBSC reduces the amount of grout needed to fill the slab pockets in comparison to the case of using welded shear studs.

If there is a need to replace the precast panel (because of deterioration), removing the lower nuts (Nut 1 in Fig. 3) allows the precast panel (with its plugs and bolts) to be uplifted and replaced. The latter operation demands access underneath the deck. If this is not possible for any particular reason, removing the nuts at the top of the plugs (Nut 2 in Fig. 3) allows the precast panel (with its plugs) to be uplifted, while the bolts will be left in place. The latter operation is more feasible if the threaded part of the bolts is not in contact with the grout. The presence of retaining washers prevent the bolts from falling through the holes in the plug and girder flange.

If there is a need to remove and replace the FBSCs (because of fatigue or corrosion), this can be achieved in two ways. Provided that the bolts are slightly deflected, the plugs (with the bolts and grout) can be pulled out by first removing the lower nuts (Nut 1 in Fig. 3) and then applying uplift forces by using the slab as support, as shown in Fig. 8. The latter process becomes easier with the application of a durable release agent material (e.g., Pieri® Cire LM-33 from Grace Construction Products) on the surfaces of the slab pocket before grouting. If the bolts are not deflected, then they can be extracted and replaced using the previously described procedure; yet the plugs (with the grout) will remain in place. Then, the plugs can be removed using some mechanical procedures, e.g., by inserting a wedge expansion anchor bolt into the plug hole.

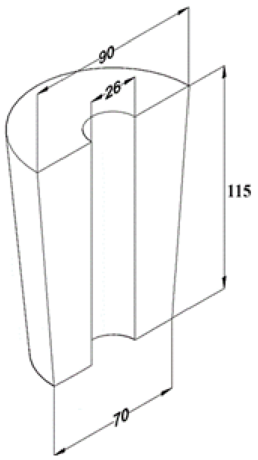

Fig. 7 Dimensions of half plug

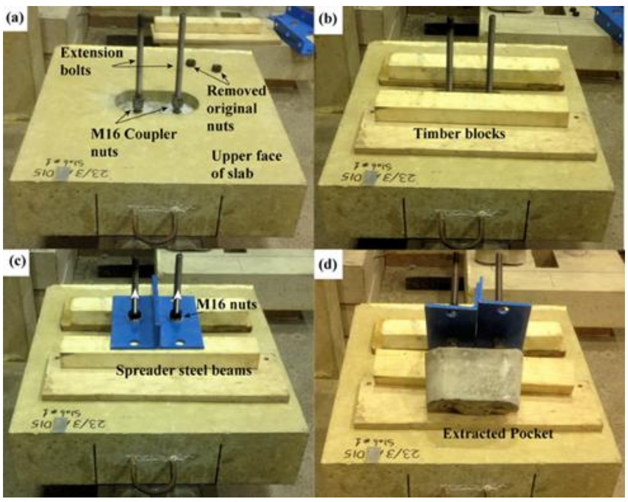

Fig. 8 Disassembly procedure

5. Experimental program

5.1 Overview

The experimental program consists of 11 pushout tests conducted on the FBSC. The program begins with five preliminary pushout tests, followed by pushout tests for evaluation of the characteristic shear resistance (i.e., 3 identical repeated tests) and investigation of the effect of different parameters (i.e., parametric tests). In the following subsections, the final test setup, instrumentation, specimens, and materials properties for the pushout tests are described in detail. In order to shorten the length of this paper, it should be emphasized that these subsections are exclusive for the final design, while the preliminary tests may have slightly different specifications (refer to Suwaed 2017 for full details).

Table 1 lists the specifications for the 11 FBSC push-out tests. The first five tests are preliminary, while the rest are final design tests. The preliminary tests represent five different designs with different structural details aiming to improve the behavior of the shear connector in terms of shear resistance, slip capacity, and potential for disassembly. In particular, the preliminary tests helped the authors to identify the final robust structural details of the FBSC described in the previous Sections.

Tests 6-11 in Table 1 represent two parametric studies based on the FBSC final design. In particular, Tests 6, 8, and 10 study the effect of bolt diameter (12-16 mm), while Tests 7, 9, and 10 study the effect of bolt pretension (55-77 kN). Test 11 was conducted so that we can have the required number of tests to evaluate the characteristic shear resistance of the FBSC. In all parametric tests, only one variable was changed while all other variables were kept as similar as possible.

5.2 Specimens

The specimen shown in Fig. 9 was used to conduct pushout tests on the FBSC. The specimen consists of two slabs connected to a steel beam with the aid of the FBSC. The UC254×254×89 beam has S355 steel grade, a length of 80 cm, and four holes (their geometry is shown in Fig. 5 for M16 bolts) drilled on its flanges. The FBSC specimen uses four bolts of Grade 8.8 per BS EN 14399-3 (BSI 2005c). Each bolt has smooth shank with 20 mm threaded ends. Four retaining washers per BS EN 3386 (BSI 2012) (their geometry for M16 bolts is shown in Fig. 4) are provided to hold the bolts in position before grouting of the slab pocket. The bolts with their retaining washers were inserted into the countersunk seat holes of the steel beam. Then, the lower nuts of grade 10.9 per BS EN 14399-3 (BSI 2005c) (Nut 1 in Fig. 3) were tightened by hand to temporarily hold the bolts in position. Fig. 10 reveals the inside view of the slab pocket where the bolts and their retaining washers can be seen locked within the countersunk seat holes of the flange of the steel beam.

The precast concrete slab has 650×600×150 mm dimensions along with a conical pocket with geometry shown in Fig. 6. Fig. 11 shows the reinforcement of the slab that was designed per Eurocode 4 (BSI 2004a).

Table 1 Specifications of push-out tests

Test No	Bolt Dia. (mm)	Bolt preload (kN)	Slabs (mean)		Plugs (mean)		Grout (mean)
		Nuts 1–2*	Comp. strength (MPa)	Tensile strength (MPa)	Comp. strength (MPa)	Tensile strength (MPa)	Comp. strength (MPa)
1	16	88-106	31	2.5	65	4.2	83
2	16	88-106	31	2.5	65	4.2	83
3	16	88-106	31	2.5	65	4.2	38
4	16	88-106	37	3.7	82	4.1	37
5	16	88-106	37	3.7	74	3.7	48
6	16	63**	50	4.0	90	4.8	41
7	14	68-81	40	3.7	72	4.0	40
8	12	47-56	40	3.7	80	4.3	51
9	14	77**	39	3.7	82	4.9	45
10	14	55**	40	3.7	85	4.7	40
11	16	59**	50	4.0	100.1	5.0	42

* See Fig. 3 for locations of Nuts 1 and 2
** Washer load cell actual readings; otherwise, DTI washer predictions

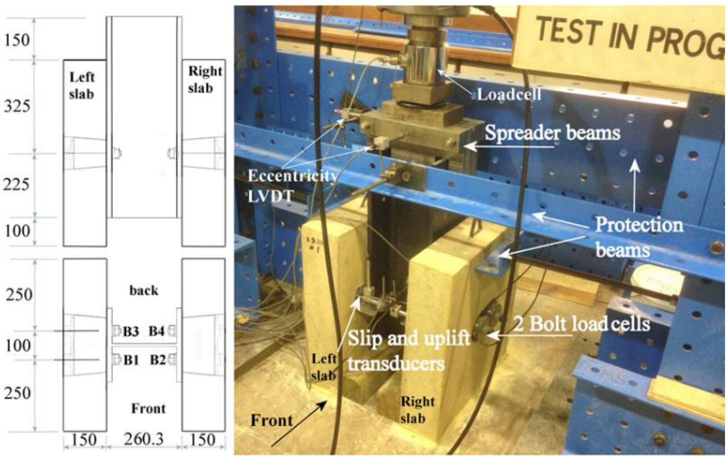

Fig. 9 Typical setup for push-out tests and instrumentations

Fig. 10 Inside view of a slab pocket showing bolts and retaining washers

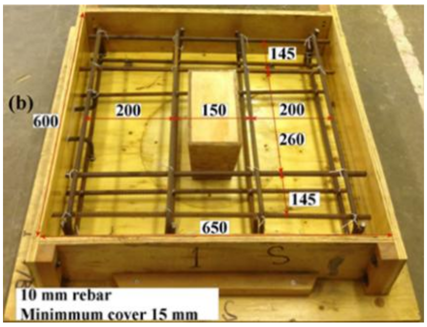

Fig. 11 Slab reinforcement details

Two layers of a release agent (Pieri® Cire LM-33 from Grace Construction Products) were applied to the surface of the slab pocket, and after that, grout was poured into the slab pockets. Then, a precast plug with geometry shown in Fig. 7 was gradually inserted into the slab pocket around each bolt, and the upper nut was tightened. Such process ensured that grout passed through all gaps without leaving

Table 2 Typical mix proportions for slabs, plugs, and grout

Material	Slabs (kg/m3)	Plugs (kg/m^3)	Grout (kg/m^3)
Cement	313	500	910
Cement type	CEM II A-L 32.5 R	CEM I 52.5N	Hanson Quickcem
Water	189	182	455
Sand	825	713	910 'fine sand'
Gravel	1093 (size 10 mm)	1011 (size 10 mm)	-
Superplasticizer	0.8% of cement weight	1.2% of cement weight	-

Table 3 Sieve analysis of fine sand used in grouts

Sieve size (mm)	Cumulative (% by weight)	Passing (% by weight)	BSI (1976), Table 1, Type B, Passing (% by weight)
0.6	0	100	55 - 100
0.3	34	66	5 - 75
0.15	58	8	0 - 20
0.063	8	0	< 5

any voids. Table 2 lists the mix proportions used for the grout, the plugs, and the concrete slabs for the final pushout tests (but not for the preliminary tests discussed in the next paragraph). It should be emphasized that if there are gaps (> 1.0 mm) between the concrete slab and the steel beam due to imperfection, then they should be sealed before grouting the pockets. This should be done to avoid leaking of the grout.

5.3 Materials properties

The mean concrete compressive and tensile strengths were obtained on the day of each push-out test according to BS EN 12390-3 (BSI 2009c) and BS EN 12390-6 (BSI 2009d), respectively. The compressive strengths of the slabs and plugs were evaluated by using six standard cubes of 100 mm sides; the compressive strength of the grout by using six 75 mm cubes; and the tensile strengths of the slabs and plugs by using three standard cylinders of 100 mm diameter and 200 mm height.

In each test, all bolts had approximately the same preload to ensure a symmetrical behavior on loading. The bolt preloads were measured either by washer load cells or Direct Tension Indicator (DTI) washers per BS EN 14399-9 (BSI 2009a). The maximum size of the gravel was 10 mm. Table 3 provides the sieve analysis (BSI 1976) for the 'fine' sand used in designing a flowable grout. This is essential to avoid segregation of sand particles between the top flange of the beam and the plug.

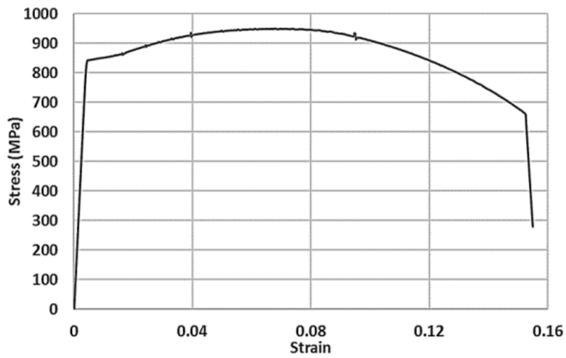

Fig. 12 Typical stress-strain behavior of bolts from tensile coupon tests

Tensile tests per BS EN ISO 6892-1 (BSI 2009b) were conducted on nine steel coupons machined from bolts. A typical stress-strain relationship from one coupon test is shown in Fig. 12, while average values of the properties of the steel bolts are listed in Table 4.

5.4 Test setup and instrumentation

The experimental setup shown in Fig. 9 was used to conduct pushout tests on the FBSC. The test setup as well as the geometry and details of the specimen are according to

Table 4 Mechanical properties of Grade 8.8 bolts

Test	Modulus of elasticity (GPa)	Yield stress (MPa)	Tensile strength (MPa)	Maximum elongation (%)	Bolt tensile resistance (kN, calculated)
Avg. of 9 specimens	209	787	889	8	-
Min.	201	719	832	5	-
Max.	215	847	950	15	-
Standard deviation	5	50	41	5	-
D12 mm	-	-	-	-	100.5
D14 mm	-	-	-	-	136.9
D16 mm	-	-	-	-	178.7

Annex B of Eurocode 4 (BSI 2004a) with the exception that no grease was applied in the steel-concrete interface to ensure the required frictional resistance.

The slip between the steel beam and the concrete slabs was measured with four linear variable displacement transducers (LVDTs) placed close to the position of the four bolts (B1 to B4 in Fig. 9). The separation (i.e., uplift displacements) of the concrete slabs from the steel beam was measured with four additional LVDTs placed close to the positions of the four bolts. An LVDT was used to monitor the jack displacement as shown in Fig. 13. The hydraulic jack used to apply vertical loading has a capacity of 100 tons, while a load cell (see Figs. 9 and 13) with the same capacity measured the force just under the jack.

Dental paste was used to bed the base of slabs into the floor to prevent horizontal sliding, similarly to Oehlers (1980) and Yuan (1996). As mentioned by Oehlers and Bradford (1995), the configurations of a standard pushout test setup results in the development of a horizontal force in the concrete slab-steel beam interface. For that reasons, supporter steel beams (Fig. 9) were added just 5 mm away from each concrete slab to hold them after failure. Additional safety beams were used to hold the steel beam in case of possible tilting.

It is very essential to have symmetry in the test setup, i.e., the load of the hydraulic jack to be symmetrically resisted by four reactions (shear resistances of the four bolts). In this case, the pushout test failure is more likely to be by simultaneous fracture of the four bolts, which means that the failure load is the mean of the failure loads of the four bolts. On the other hand, if eccentric loading takes place, this will result in failure of one bolt only (the more heavily loaded one) and the corresponding failure load will represent the value of the lowest value. For this reason, Eurocode 4 (BSI 2004a) recommends using eight shear connectors, arranged into two levels, i.e., four connectors in each level. By doing this, rotational stiffness is provided that prevents any tilting of the steel beam. However, using 8 bolts would require designing and fabricating a 200 t capacity rig, which was beyond the limitation of the Structures Lab at the University of Warwick. Nevertheless,

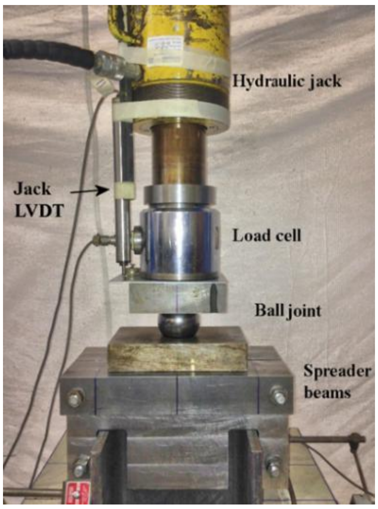

Fig. 13 Load transfer through ball joint in pushout tests

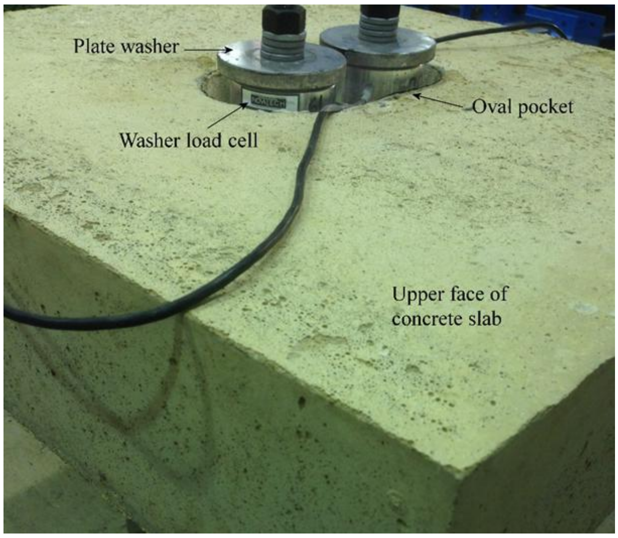

Fig. 14 Nut and washer load cell on the top of the concrete plugs

because only four shear connectors were used in the pushout tests presented herein, two eccentricity LVDTs were positioned at the upper tip of the steel beam to detect possible horizontal movements of the specimen, as shown in Fig. 9.

To ensure that the load is applied directly to the centroid of the steel section without any eccentricity, the test setup includes a ball joint along with two spreader beams (see Fig. 13). The latter ensured that the point load from the ball joint is evenly distributed to the two flanges of the steel beam. Fig. 14 shows that washer load cells (F313CFR0K0 from NOVATECH) of 200 kN capacity were used to measure the force inside the bolts of the FBSC. Each load cell is placed within two plate washers and then secured with a nut on the top of each concrete plug. The push-out tests were carried out under load control of 40-60 kN/min during the initial linear shear load-slip displacement behavior phase, and then, under displacement control of 0.1-0.2 mm/min during the nonlinear shear load-slip displacement behavior phase. Following the recommendation of Eurocode 4 (BSI 2004a), a continuous constant loading rate was applied to ensure that failure does not occur in less than 15 minutes.

5.5 Experimental results

5.5.1 Load – slip behavior and failure mode

Fig. 15 plots the load-slip behavior of the FBSC from three ‘identical’ tests, namely: Tests 5, 6, and 11. Table 5 specifies the shear resistance and slip capacity for these tests. The load-slip displacement curve consists of three main phases. The first phase, which starts from 0.0 mm slip displacement to a range of 0.1 to 0.3 mm slip displacement, corresponds to loads from 55 to 70 kN (i.e., 25% of the shear resistance). This phase is characterized by linear elastic deformations in the concrete slab and the steel beam. The shear force is transferred from the steel beam to the concrete slabs through friction resistance in their interface. The initial stiffness of the M16 FBSC during the 2-10 kN

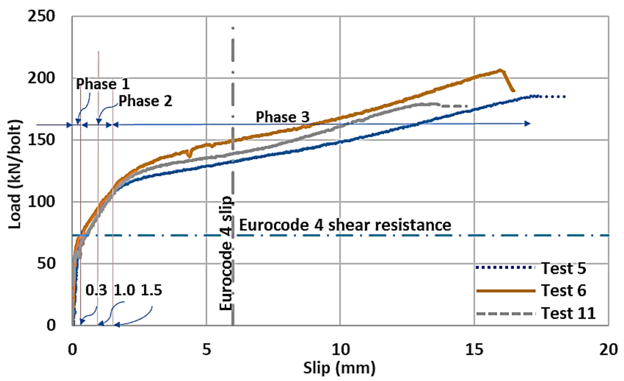

Fig. 15 Load-slip behavior from three identical tests

loading period is extremely high, i.e., 230-900 kN/mm. The static friction coefficient can be calculated by dividing the load of phase one (e.g., 55 kN/bolt from Test 11 in Fig. 15) by the preload of the bolt (e.g., 59 kN from Test 11 in Table 1). Thus, for Test 11, the friction coefficient is 0.9, which is larger than the typical one for steel-concrete interfaces (i.e., ≈ 0.5-0.6). Based on this, we conclude that apart from friction, other factors contribute to the shear resistance during phase one. These factors are the chemical bond (adhesion) between the grout and the steel flange and interlocking between the grout and the steel flange due to roughness of the flange surface (see Fig. 10). No noticeable slip occurs until the resistance due to the chemical bond and interlock connection is exceeded. As the applied force increases, the frictional resistance of the FBSC is overcome, and the bolt resists the shear force through bearing. A slight sudden slip displacement (e.g., Test 6 in Fig. 15) may occur if there are gaps between the bolts and their surrounding grout and concrete. These gaps are the results of preload adjustment that took place before Test 6. The preloads in the four bolts were intentionally adjusted to ensure equal initial tensile forces. Otherwise, the elastic linear performance would continue until a slip displacement of 0.3 mm, which indicates the beginning of phase two. Eurocode 4 recommends applying the load in pushout tests first in cycles between 5% and 40% of the maximum load. It should be noted that the main reason of this is to ensure that the tested shear connector is not susceptible to progressive slip (Johnson 2012). Suwaed (2017) showed that theses cycles did not result in noticeable slips in the FBSC. In particular, the accumulated slip due to 25 cycles was equal

Table 5 Results of Tests 5, 6, and 11

Test No.	Shear resistance (kN/bolt)	Slip capacity (mm)
5	185	17
6	206	16
11	179	14
Average	190	15.7
Standard deviation	11.5	1.2
Coeff. of variation CV %	6	7.6

to 0.05 mm. Thus, the first 25 cycles can be safely ignored in the FBSC push-out tests due to the frictional resistance of the latter that essentially minimizes slip at low shear loads.

Phase two represents a linear load-slip displacement behavior and covers slip displacements from 0.3 to 1.5 mm, where the shear load reaches values up to 110 kN, i.e., approximately equal to 58% of the shear resistance. The slip displacement is due to bolt bending and bearing against the surrounding grout and concrete. The previously mentioned resistance due to interlocking and chemical bond in phase one is exceeded, and therefore, the stiffness of phase two is less than the stiffness of phase one. It should be noted that Eurocode 4 assumes the stiffness of a shear connector equal to the secant stiffness at 50% of the shear resistance. The 50% of the average shear resistance (from Table 5) is 95 kN, and by referring to Fig. 15, the corresponding slip displacement is equal to 0.91 mm. Thus, the stiffness of the M16 FBSC is equal to 95/0.91 = 104 kN/mm. It is noted that the stiffness that can be offered by a 19 mm diameter welded stud is 100 kN/mm according to Eurocode 4 (BSI 2004a), which reveals the superior stiffness of the FBSC.

Phase three covers slip displacements from 1.5 mm to 14-17 mm, where the load reaches the shear resistance. The behavior in this phase is nonlinear for a short interval, linear for most of its part, and ends with a nonlinear descending part. The slip displacement capacity of the FBSC is about 11 times the slip displacement at which plastic deformation develops (i.e., phase three in Fig. 15). The same ratio for welded shear studs is equal to three (Oehlers and Bradford 1999, Oehlers and Sved 1995).

The typical failure modes are illustrated in Figs. 16 and 17 for Test 6 which similar to Tests 5 and 11. Fig. 16 shows that the deflected shape of the bolts includes two plastic hinges that are 20-40 mm apart. Fig. 16 also shows that shear and tensile deformations are concentrated within a length of 5-6 mm close to the bolt base; following a deflection angle (β) of about 45°. Fig. 17 shows that the grout in the countersunk seat forms a cushion that allows the bolt to deflect. It should be also mentioned that the recorded slab separations (uplift displacements) in all pushout tests are very small (< 0.5 mm), and similarly to the LNSC (Suwaed and Karavasilis 2017a, b), have a negligible effect on the behavior of the FBSC.

5.5.2 Preliminary tests

Each concrete plug of the specimens of Tests 1 and 2

Fig. 16 Deflected shapes of bolts from Test 6

Fig. 17 Concrete crushing in Test 6

has two longitudinal holes to accommodate two M16 Grade 8.8 bolts threaded along their whole length. These tests showed modest shear resistance and large slip capacity. The specimen of Test 3 uses two bolts per plug but with a countersunk seat hole of 120° at the lower part of each bolt hole to reduce sudden slip of the bolt.

Test 3 showed increased shear resistance and reduced slip capacity compared to Tests 1 and 2. The specimen of Test 4 has a rectangular conical pocket in each slab, two concrete plugs similar to those shown in Fig. 7, and a countersunk seat hole at the upper part of each bolt hole. The shear resistance and slip capacity were 165 kN/bolt and 14.1 mm, respectively. These values represent further improvement in shear resistance and slip capacity compared to those of Tests 1 to 3. Test 5 is similar to Test 4 apart from the upper part of the bolt hole, which was chamfered to create a countersunk seat with an inclination angle of 60° instead of 120°. Test 5 showed shear resistance of 185 kN/bolt and slip capacity of 17 mm. Finally, Test 6 was conducted on a specimen representing the actual robust structural details of the FBSC. The results show that the shear resistance and slip capacity were 206 kN/bolt and 16 mm, respectively. A simple comparison between Tests 1 to 6 highlights that the novel structural details of the FBSC result in superior structural performance. More details on the preliminary tests and their results are provided by Suwaed (2017) and are not repeated herein to reduce the length of the paper.

5.6 Characteristic shear resistance

Limit states design is based on characteristic values of the resistance of structural members divided by partial factors that consider different sources of uncertainty (BSI 2010). Typically, three pushout tests are required on nominally identical specimens to determine the characteristic shear resistance P_{Rk} of a shear connector. If the results of the three tests are within 10% of the mean value, then from Johnson (2012) and Eurocode 4 (BSI 2004a), $P_{\mathrm{Rk}} = 0.9\,P_{\mathrm{min}}$, where P_{min} is the lowest of the three measured shear resistances. For this reason, Tests 5 and 6 were followed by one additional test (i.e., Test 11 in Table 1) with similar material specifications. Table 5 lists the shear resistances and slip capacities from Tests 5, 6 and 11 as well as their very low coefficients of variation.

The deviation of the shear resistance of any individual test from the average shear resistance is about 6%, which is significantly lower than the previously mentioned 10% limit of Eurocode 4. Therefore, the characteristic shear resistance of the FBSC is calculated with confidence as $P_{\mathrm{Rk}} = 0.9 \times 179 \approx 161$ kN. It is noted that typical deviations from tests on welded shear tests can be up to +/- 30% (Oehlers 1980). It has been reported (Xue *et al*. 2008) that the results of the pushout tests based on welded studs are widely scattered. The main reason is the lack of uniform distribution of bearing stresses in the area around the collar of the studs due to the existence of voids and/or the variation in local arrangement of the aggregate particles (Johnson 2004). Furthermore, as diameter of stud decreases, the scatter increases because of the corresponding collar size in relation to that of aggregate particles (Oehlers 1980). It should be mentioned that the slip displacements listed in Table 5 are those at maximum loads and not the characteristic slips. Eurocode 4 defined slip as the one measured when the load drops from maximum at least 20% and the characteristic slip as the minimum of three identical tests times 0.9. This is beyond the capacity of the testing rig. Further discussion on this aspect is provided by Suwaed (2017).

5.7 Comparison with welded studs

According to Eurocode 4 (BSI 2004a), the shear resistance of welded shear studs is calculated as the minimum of (without considering partial factors)

$$P_{\mathrm{R}} = 0.8\,f_{\mathrm{u}}\,\pi\frac{d^2}{4} \tag{1}$$

and

$$P_{\mathrm{R}} = 0.29\,d^2\sqrt{f_{\mathrm{ck}}E_{\mathrm{cm}}} \tag{2}$$

where d is the shank diameter of the welded stud, f_{u} is the ultimate tensile strength of the steel material of the stud, f_{ck} is the characteristic compressive cylinder strength of the concrete slab, and E_{cm} is the elastic modulus of the concrete. f_{ck} and E_{cm} can be calculated using the procedure presented in Dai *et al*. (2015). By substituting in Eqs. (1) and (2) the concrete slab strength (not the plug strength), stud diameter, and tensile strength of the FBSC from Test 6, the shear resistance of the corresponding welded shear stud is calculated equal to 73 kN from Eq. (2), i.e., slab concrete failure controls the shear resistance. This result is supported by similar values obtained from tests conducted by Xue *et al*. (2008). Thus, the shear resistance of the FBSC is significantly higher (i.e., approximately 2.5 times higher) than that of welded studs. Furthermore, the slip capacity of the FBSC from pushout Test 6 is 16.0 mm. The slip capacity of welded studs with similar specifications was found equal to 7.7 mm by Xue *et al*. (2008). Thus, the FBSC slip capacity from Test 6 is 2.1 times the slip capacity of welded studs.

In practice, it is often advantageous to use fewer connectors (i.e., partial shear design) than the number required for full shear design, following the recommendations of Clause 6.6.1 of Eurocode 4.1. Shear connectors in partial shear design are more widely spaced, while the amount of the transverse slab reinforcement is also reduced. It is important here to note that partial shear design is more economical than full shear design for composite beams (Johnson and May 1975). Partial shear design for composite bridges with large spans cannot be achieved with welded shear studs due to their modest 6.0 mm slip capacity (Johnson 1981). This reveals further benefits that could be gained when using a shear connector with large slip capacity such as the FBSC in large spans bridges.

The reasons behind the superior mechanical characteristics of the FBSC are: (1) the use of high strength steel (e.g., Grade 8.8 and above); (2) the use of high strength concrete (80-100 MPa) for the plugs; (3) failure is due to fracture of the bolts and not due to concrete splitting; (4) the use of smooth flowable grout around the bolts without variation in voids or aggregates sizes; (5) the exploitation of friction resistance between the concrete plug and the steel flange; (6) shearing-off the bolts through their smooth shanks; and (7) shear failure through an elliptical (not circular) cross-section of the bolt. It should be mentioned that although studs with Grade 8.8 were assumed in the previous comparison, welding of high grade steel, such as 8.8, to a normal grade mild steel beam is not allowed (BSI 2016).

5.8 Experimental parametric studies

The aim of the parametric tests is to explore the effect of the bolt diameter and preload on the behavior of the FBSC. The effect of the concrete strength of the plugs was not investigated on the basis of the results from LNSC, which has similar configuration to the FBSC. The results showed negligible effects on shear resistance and slip capacity (Suwaed and Karavasilis 2017a).

5.8.1 Effect of bolt diameter

Three different bolt diameters, i.e., 12, 14, and 16 mm, were used in the pushout Tests 8, 10, and 6, respectively. Material properties of these tests are listed in Table 1, while their results are listed in Table 6 and plotted in Fig. 18. Table 6 shows that for all tests, the ratio of the shear resistance to the bolt tensile resistance is almost the same; having an average ratio of 1.12. Eq. (3) gives the typical ratio of the shear resistance to tensile resistance for Grade 8.8 bolts, which is 0.6. These ratios indicate the large contribution of friction to the shear resistance of the FBSC. Fig. 18 shows the shear load-slip displacement behavior of the FBSC with three different diameters. The figure proves that the FBSC, in contrary to welded studs, has slip displacement capacity larger than 6 mm even when the diameter is less than 16 mm.

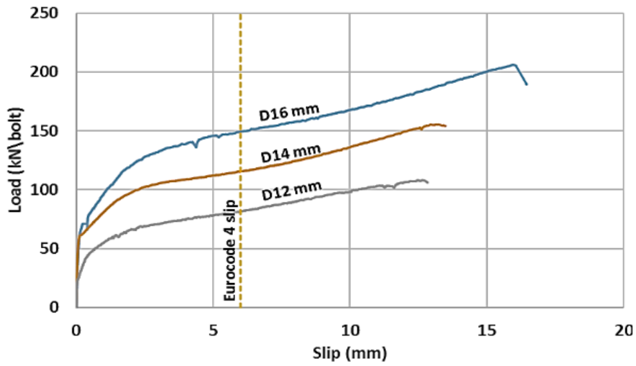

Fig. 18 Effect of bolt diameter on load-slip behavior

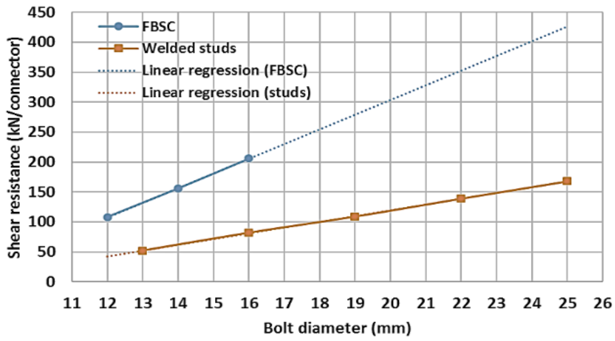

Fig. 19 Effect of bolt diameter on shear resistance of FBSC and studs

Fig. 19 shows the effect of changing the diameter of the bolt and stud on the shear resistance of the FBSC and welded shear studs, respectively. Fig. 19 is based on results listed in Table 6 for the FBSC, while the results of welded studs were obtained from similar experimental data in BS 5400-5 (BSI 2005f). Fig. 19 shows that for every 1 mm increase in diameter, the shear resistance of the FBSC increases by 24.5 kN, while the shear resistance of welded studs by 9.6 kN. The experimental results of welded studs from BS 5400-5 (BSI 2005f) in Fig. 19 show a linear relation between shear resistance and stud diameter when the stud diameter is within the range of 13 to 25 mm. Because of similarities in overall behavior, test setup, and

Table 6 Results of Tests 6, 8, and 10

Test No.	Bolt dia. (mm)	Shear resistance (kN/bolt)	Slip capacity (mm)	Deflection angle β (degrees)	Shear resistance/ tensile resistance*	Bolt internal load/ tensile resistance*
6	16	206	16	45	1.15	0.59
8	12	108	12.6	33	1.08	-
10	14	156	13.2	39	1.14	0.51
Average	-	-	-	-	1.12	0.55
Standard deviation	-	-	-	-	0.031	0.04
CV %	-	-	-	-	3	7

* Bolt tensile resistance is provided in Table 4

specimen geometry, a similar linear relation for the FBSC diameters above 16 mm can be assumed. Based on this assumption, the linear regression extension line for FBSC when d > 16 mm was constructed. For FBSC diameters between 12 to 16 mm, no noticeable differences can be seen between the dotted line (linear regression) and the continuous line (experimental results). This indicates the precise testing setup and well controlled material variations. For FBSC diameters > 16, the linear regression line predicts that the FBSC shear resistance can reach 430 kN/bolt when the diameter is 25 mm. More pushout tests are required to confirm this approximate estimation and to validate Eq. (4) experimentally for predicting the shear resistance of the FBSC for bolts of larger diameter (> 16 mm).

5.8.2 Effect of bolt pretension

The bolt pretension was measured either by washer load cells (Fig. 14) or DTI washers. In case of DTI washers, the procedure outlined in Suwaed (2017) was followed. Please note that some of the preload is lost with time due to plug concrete creep and bolt steel relaxation. For example, the bolts of Test 5 were preloaded to 88-106 kN (DTI washer prediction), while at the test date the preload was only 66.7 kN (Suwaed 2017). In order to clarify this point, the preloads of the bolts in Test 10 were monitored for 7 days after tightening and before execution of the actual pushout test. The bolts were initially preloaded with a 60 kN tensile force (about 44% of tensile resistance). Fig. 20 shows the preload loss during the first 7 hours after tightening, which is about 3.4% to 11.6% of the initial preload. These results are also reported in Table 7, which also lists the preload loss corresponding to 2 and 7 days after tightening. The positions of Bolts 1 to 4 are shown in Fig. 9.

Table 7 shows that bolt preload decreases by 11% at 7 days after tightening. Moreover, Fig. 20 shows that the preload rapidly decreases following initial tightening, then decreases at a slower rate following a logarithmic curve. The preload-time curve for Bolt 4 after tightening, for example, can be represented through the following equation

$$T = -1.02\ln(t) + 54 \tag{3}$$

where T is the residual preload of Bolt 4 of Test 10 in kN and t is the time after tightening in hours. Using Eq. (5), a rough estimation of the preload of Bolt 4 of Test 10 after 100 years is 49.3 kN (i.e., 18% loss).

The curves plotted in Fig. 20 show that the preload losses of the four bolts are not the same due to several factors, i.e., (1) shrinkage of grout and plug; (2) creep of grout, plug, nuts, washers (i.e., hardened, DTI, and plate washers), and flange of steel beam; (3) relaxation of bolt material (Johnson and Buckby 1986); (4) self-loosening of nuts combined with torsional relaxation of bolts after tightening; (5) friction between the nuts and bolts threads; (6) friction between the nuts and their underneath washers; and (7) accuracy of fit of parts together as related to tolerance variations (Bickford 1995). Similar loss rates were found in the literature, for example, a loss of 2-11% of preload immediately after tightening, followed by additional loss of 3.6% in the next 21 days, were recorded for similar high strength bolts by Bickford (1995).

The effect of the preload on the shear resistance can be assessed using Tests 7, 9, and 10. These tests use three different preloads (i.e., 62, 77, and 55 kN, respectively), while all other parameters are similar. The results of the tests are listed in Table 8, while Table 9 and Fig. 21 illustrate the effect of preload variations on the load-slip behavior. As preload increases, the load at first slip increases due to the higher frictional resistance. This is highlighted in Fig. 22, which shows the experimental curves

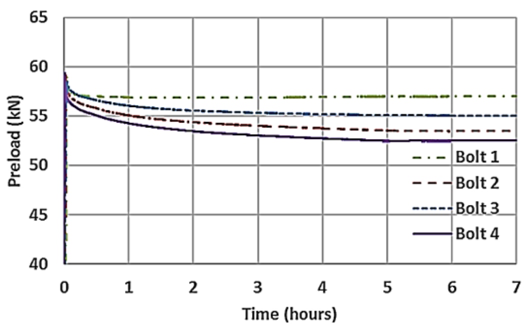

Fig. 20 Preload loss of four bolts in Test 10

Table 7 Preload Loss in Test 10

Bolt No.	Preload loss (%)		
	7 Hours	2 Days	7 Days
1	3.41	3.55	7.16
2	9.91	10.02	11.56
3	6.46	6.58	9.37
4	11.55	12.52	15.36
Avg.	7.67	8.17	10.86

Table 8 Results of Tests 7, 9, and 10

Test No.	Bolt dia. (mm)	Shear resistance (kN/bolt)	Slip capacity (mm)	Deflection angle β (degrees)	Preload/ tensile resistance*
7	14	134	14.4	21	0.45
9	14	141	12.4	19	0.56
10	14	156	13.2	39	0.41

* Bolt tensile resistance is provided in Table 4

Table 9 Effect of preload on shear resistance

Test No.	Preload (kN)	Load at 1st slip $\boldsymbol{P_{1st}}$ (kN/bolt)	Shear resistance $\boldsymbol{P}$ (kN/bolt)	Deviation of $\boldsymbol{P}$ from average (%)
9	77.2	79.3	140.5	2
7	62.0	65.9	134.1	7
10	55.5	61.3	156.0	8
			Avg. of $\boldsymbol{P}$	143.5

* Bolt tensile resistance is provided in Table 4

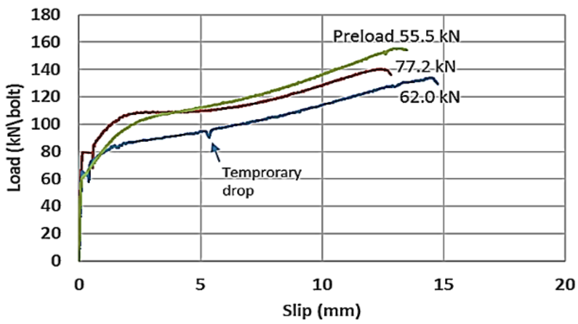

Fig. 21 Effect of preload on load-slip behavior from three tests

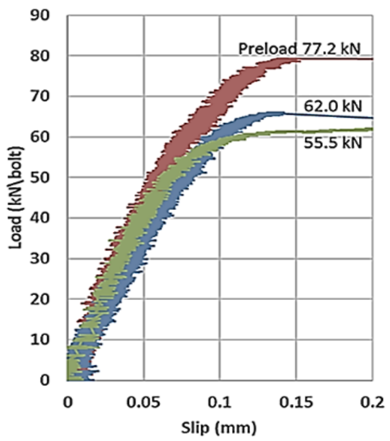

Fig. 22 Effect of preload in the first phase of load-slip behavior

of the three tests for displacements up to 0.2 mm. The same figure shows also that an increase of 40% in preload results in an increase of frictional resistance by 29%. Fig. 22 also shows that changing the preload does not affect the initial stiffness. The deviation of the shear resistance of any of the three tests from the average is less than 8%. As was previously explained, Eurocode 4 considers three tests to be identical if the shear resistance of any individual test deviates less than 10% from the average value (BSI 2004a). Therefore, Tests 7, 9, and 10 could be considered as identical as their different bolt preloads do not significantly affect their shear resistance. Further tests are though recommended to cover a wider variation in bolt preloads.

5.9 Design equation

Under service conditions, the bolt of the FBSC experiences torsion (due to tightening), tension, shear, and bending stresses. The behavior of the FBSC is affected by friction in the steel beam-concrete plug interface as well as by the stiffness of the concrete (i.e., grout and plug) surrounding the bolt. Thus, the derivation of an analytical design equation for computing the shear resistance of the FBSC seems a rather complicated task. Most design standards use empirical equations (Oehlers and Bradford 1995), which have been empirically derived from pushout and beam tests (e.g., Oehlers and Johnson 1987).

Because the FBSC does not have a collar as compared to welded studs, Eq. (1) of Eurocode 4 cannot be used. Instead, Eq. (4), which represents the shear resistance of high strength bolts, can be adopted, after some modifications. According to Clause 3.6.1(1) of BS EN 1993-1-8 (BSI 2005e), the shear resistance P_s of a Grade 8.8 bolt is given by (after omitting partial safety factors)

$$P_s = 0.6\, f_u\, A_s \tag{4}$$

where f_u is the tensile strength of the bolt, and A_s is the tensile stress area of the bolt where the shear plane passes through the threaded portion. In case that the shear plane passes through the unthreaded portion of the bolt, A_s is the gross cross-sectional area.

Moreover, a design equation for the FBSC shall account for the friction resistance in the steel beam–concrete interface; the effect of the inclination of the deflected shape of the bolt; and the effect of shear failure through an elliptical cross-section of the bolt. On the basis of the derivations provided in Suwaed and Karavasilis (2017a), the shear resistance of the FBSC can be calculated by

$$P = \frac{\pi d^2 f_u}{4}\left(\frac{0.6}{\cos\beta} + \frac{T}{F_u}(\sin\beta + \mu\ \cos\beta)\right) \tag{5}$$

where T is the preload of the bolt, F_u is the tensile resistance of the bolt, while the rest of the variables were previously defined. The tensile resistance is based on the nominal cross-sectional area and not on tensile stress area as the fracture of bolts always occurs through the unthreaded portion of the bolt. It should be noted that Eq. (5) does not contain safety factors or strength reduction factors.

The validity of Eq. (5) is confirmed with five tests. First, it is evaluated by using the results from Test 6. This test used a preload T of 60 kN with β = 45° (from Fig. 16); therefore, the shear resistance is equal to P = 214 kN, i.e., only 4% different than the test result (206 kN) in Table 5. In similar way, validity of Eq. (5) was checked using information from Tests 8 and 10 that use M12 and M14 bolts, respectively. Tests 8 and 10 have preloads of 18.8 kN and 55.5 kN, respectively. Fig. 23 shows the deflected shapes of the fractured bolts from Test 8, which have an average deflection angle β of 33°. Fig. 24 shows the fractured bolts from Test 10, which have an average deflection angle β of 39°. By substituting appropriate

Fig. 23 Deflected shapes of bolts (Test 8)

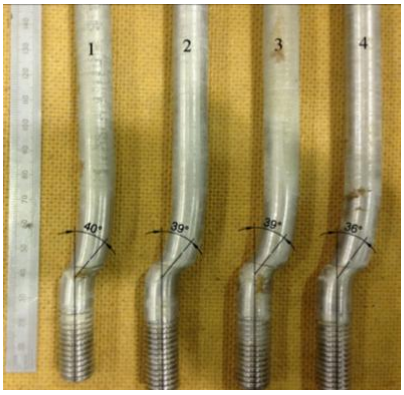

Fig. 24 Deflected shapes of bolts (Test 10)

values into Eq. (5), the shear resistance is estimated equal to 107 kN and 163 kN for the Tests 8 and 10, respectively. These values are only 1% and 5% different than their experimental counterparts. Lastly, validity of Eq. (5) was verified using the results from Tests 7 and 9 listed in Table 8. Eq. (5) estimates shear resistances of 139 kN and 148 kN for Tests 7 and 9, which are 4% and 5% different than the experimental ones.

In summary, the effectiveness of Eq. (5) to predict the shear resistance of the FBSC was assessed using five pushout tests. The results are listed in Table 10 and are shown in Fig. 25, which indicates a maximum difference of 5% without using safety factors. It appears that Eq. (4) can be used, after applying a suitable safety factor, to predict the shear resistance of the FBSC for cases with: plug concrete cube strengths between 65-100 MPa; bolts with steel strength of 889 MPa; diameters from 12 to 16 mm; grout

Table 10 Comparison among the predictions of Eq. (5) and push-out tests results

Test No.	Shear resistance (kN/bolt)	Eq. (5) (kN/bolt)	Difference %
6	206	214	4
7	134	139	4
8	108	107	1
9	141	148	5
10	156	163	5

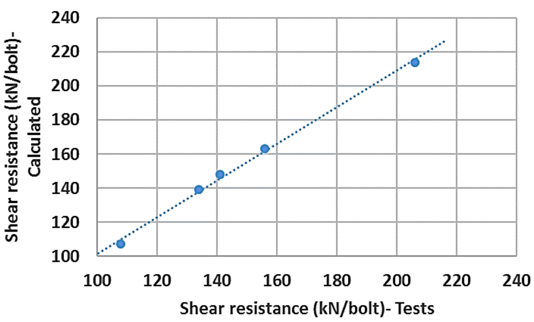

Fig. 25 Comparison of the shear resistance predictions from Eq. (5) and the push-out test results

compressive strengths from 35 to 50 MPa; and an initial bolt preload in the range of 40% to 70% of the tensile resistance.

6. Conclusions

A removable friction-based shear connector (FBSC) for precast steel-concrete composite bridges has been presented. The FBSC uses high-strength steel bolts, which are fastened to the top flange of the steel beam using a grouted countersunk hole that prevents sudden slip of bolts inside their holes. Pre-tensioning of the bolts provides the FBSC with significant frictional resistance that prevents slip displacement under service loading. The bolts are surrounded by conical precast high-strength concrete plugs, which easily fit within the precast slab pockets. Grout is used to fill all the gaps between the bolts, the precast plugs, and the precast slab pockets, while tightening of a nut at the top of the FBSC secures the plugs in place before grout hardening. 11 push-out tests were conducted to fully illustrate why the novel structural details of the FBSC result in superior shear load-slip displacement behavior compared to welded shear studs. The tests also serve to assess the characteristic shear resistance of the FBSC and to quantify the effects of the bolt diameter and bolt pretension. A simple design equation to predict the shear resistance of the connector is proposed. Based on the results presented in this paper and within the specific boundaries of the experimental work and parameters studied, the following conclusions are drawn:

- The FBSC allows bridge disassembly and time-efficient replacement of deteriorating structural components such as the precast deck panels, the shear connectors, and the steel beams.
- The FBSC promotes accelerated bridge construction by taking full advantage of pre-fabrication. Fabrication of all structural components is carried out in the shop, while the bridge assembly on site. Moreover, the FBSC reduces the amount of grout needed to fill the slab pockets in comparison to the case of using welded shear studs.
- The FBSC has very high shear resistance and stiffness in comparison to welded shear studs, and therefore, results in reduction of the required number of shear connectors and slab pockets. The characteristic shear resistance and stiffness of the FBSC for an M16 bolt were found equal to 161 kN and 104 kN/mm, respectively.
- The FBSC has large slip capacity, i.e., about 16 mm, and therefore, could allow partial shear design of steel-concrete composite bridges with large spans with the goal of further reducing construction cost. Partial shear design cannot be safely exploited with welded shear studs due to their modest slip displacement capacity, i.e., about 6 mm.
- The FBSC minimizes the slip displacement of the slab with respect to the steel beam under service loading due to its appreciable frictional resistance in the steel beam - concrete plugs interface. The latter

characteristic could be very beneficial in reducing fatigue effects that occur in other shear connectors, such as welded studs, under repeated traffic loading.
- The shear load-slip displacement behavior of the FBSC shows repeatability and negligible scatter. Among three identical push-out tests (according to Eurocode 4 recommendations), the maximum deviations in shear resistance of any individual test from the average was less than 8%. Such deviation in the case of welded shear studs can reach values up to +/- 30%.
- Increasing the bolt preload in the FBSC by 40% was found to increase the frictional resistance by 29%.
- The proposed design equation (Eq. (5)) was evaluated against test results from five specimens with different bolt diameters and preloads and was found to predict the shear resistance with maximum absolute deviation less than 5% without using safety factors.
- The shear resistance of the FBSC could be approximately estimated equal to 1.1 times the bolt tensile resistance for preliminary design purposes.

Further push-out tests including also fatigue ones are needed to expand our knowledge on the structural performance of the FBSC. Furthermore, full-scale precast steel-concrete composite beam tests are needed to evaluate the behavior of the FBSC within more realistic boundary conditions.

Acknowledgments

This work was financially supported by the Iraqi Ministry of Higher Education and Scientific Research (3 years PhD scholarship to the 1st author) and by the Strategic EPSRC Impact Fund of the University of Warwick (awarded to the 2nd author). Hanson Cement & Packed Products Ltd and Grace Construction Products Ltd donated raw materials for the fabrication of the test specimens. Emeritus Professor Roger P. Johnson of the University of Warwick kindly reviewed interim technical reports. Professor J. Toby Mottram of the University of Warwick provided a thorough review on the PhD thesis of the 1st author. The authors acknowledge with thanks the aforementioned support. Any opinions, findings, and conclusions expressed in this paper are those of the authors and do not necessarily reflect the views of the aforementioned sponsors and supporters.

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