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SUSTAINABLE COMPOSITE BEAMS AND JOINTS WITH DECONSTRUCTABLE BOLTED SHEAR CONNECTORS

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ABSTRACT

Composite action between the concrete slab and steel beam in a building frame is typically achieved by using headed stud shear connectors that are welded to the top flange of the steel beam to produce robust shear connection, but their demolition necessarily is associated with much waste and uses considerable energy. Moreover, the slab is usually cast in situ onto profiled steel decking with conventional reinforcement placed on site, which is time consuming and labour intensive and which can increase the construction costs considerably. This paper describes the results of three push-out tests conducted to determine the load-slip behaviour of deconstructable bolted shear connectors for precast concrete slabs in a configuration which can be used in composite beams and joints. In this system, precast “green concrete” slabs having reduced emissions during their manufacture are attached compositely to the steel beam via post-installed pre-tensioned bolt shear connectors. Based on the experimental results, the structural behaviour of these new systems is assessed and compared with traditional stud shear connectors. The test results show that the behaviour of post-installed pre-tensioned bolted shear connectors is completely different to stud shear connectors, and these connectors provide reliable and adequate shear connection to composite beams and connections with precast concrete slabs.

KEYWORDS

Push-out test, shear connectors, bolted shear connectors, deconstructability, sustainability

INTRODUCTION

Traditional composite steel-concrete beams in building frames are produced by connecting the concrete slab to the steel beam using mechanical shear connectors (Oehlers and Bradford 1995). Headed stud shear connectors welded to the top flange of the steel beam are widely used because they are easily and rapidly installed with automated welding techniques. However, this connection leads to much waste at the end of the service life of the building when it is demolished and uses considerable energy. Moreover, the slab is usually cast in situ onto profiled steel decking with conventional reinforcement placed on site, which is time consuming and labour intensive and which can increase the construction costs considerably. Post-installed pre-tensioned bolted shear connectors can be an...
alternative solution which may solve many problems associated with traditional composite beam and beam-to-column composite joints.

The use of Ordinary Portland cement (OPC) in the concrete, one of the main components of composite members, results in the atmospheric emission of very large amounts of CO$_2$. Davidovits (1994) pointed out that the production of one tonne of OPC produces approximately one tonne of CO$_2$, contributing 65% of anthropogenic CO$_2$ emissions, and the demand for OPC has been increasing significantly over the years since this study. These anthropogenic emissions have become a major concern and several attempts have been conducted to reduce the use of OPC in concrete in many nations. Green concrete (GC), which is a much lower-carbon concrete with excellent performance, commonly utilises about 70% fly ash as a by-product of coal-burning power stations instead of OPC without impacting on structural performance. These concretes are also known to possess shrinkage and creep strains that 50% and 40% respectively of those in conventional concrete. Therefore, GC may be one of replacement for conventional OPC concrete in composite framed building structures.

Pre-tensioned high strength bolts installed through holes in precast slabs into pre-drilled holes in the steel beam flange produce a composite flooring system that can be deconstructed at the end of life of the structure (Bradford and Pi 2012a,b, 2013; Rowe and Bradford 2013; Ataei and Bradford 2013; Lee and Bradford 2013). Marshall et al. (1971) appear to be the first researchers to have reported the use of bolted shear connection, but the context of the usage is not clear. Twelve push tests using high strength bolts as shear connectors were carried out and reported by Dallam (1968). In these sets of tests, the bolts were embedded in the concrete slab and pre-tensioned by the turn-of-nut method after the concrete had aged 28 days. A series of tests was conducted on three types of 22-mm diameter post-installed shear connectors under static and fatigue loading by Kwon et al. (2010). Lee and Bradford (2013) conducted two series of push-out tests to obtain the behaviour of the post-installed pre-tensioned bolted shear connectors. The first and the second series of this experimental study included five and four push-out specimens respectively. Although some tests have been conducted on post-installed bolted shear connections, only a few of them focus on the deconstructability and sustainability of the composite beam and on beam-to-column composite joints.

This paper describes the results of three push-out tests conducted to determine the load-slip behaviour of deconstructable bolted shear connectors for precast concrete slabs in a configuration which can be used in composite beams and joints. In this system, precast GC slabs are attached compositely to the steel beam via post-installed pre-tensioned high-strength bolt shear connectors. Based on the experimental results, the structural behaviour of these new systems is assessed and compared with traditional stud shear connectors. The test results show that the behaviour of post-installed pre-tensioned bolted shear connectors is completely different to stud shear connectors, and these connectors provide reliable and adequate shear connection to composite beams and connections with precast concrete slabs.

**EXPERIMENTAL PROGRAM**

All tests were designed and conducted in accordance with Eurocode 4 (2004), and details of the push-out test specimens are summarised in Table 1. The geometric and design details are also presented in Figure 1. A push-out specimen consists of a 460 UB 82.1 steel beam and two precast concrete slabs, which were bolted to the steel beam. The dimensions of the precast slabs were $615 \times 600 \times 120$ mm. Two sizes of reinforcing bars, N10 in the vertical direction and N16 in the horizontal direction, were used to provide two-layer reinforcement meshes placed near the top and bottom layers of the concrete slabs. N16 bars were used to prevent splitting of the precast concrete slabs that may result from tension induced by concentrated forces at the bolt locations. A roller support below one concrete slab and a fixed support below the other were used in each push out test. The roller support eliminates any frictional horizontal resistance being imposed by the precast concrete slabs. Two different high-strength structural bolts were used, viz. M16 8.8 and M20 8.8, pre-tensioned to their specified pretension loads of 95 kN and 145 kN respectively, as specified by AS/NZS 1252 (1996). To confirm the minimum required pretension force in the bolts, an electric control torque with Squirter Direct
Tension Indicating (SDTI) washers were used. The clearance between the pre-tensioned bolts and the holes in the precast concrete slabs and steel beams was 2 mm and 1 mm respectively. A photograph of the experimental setup for the push-out tests shown in Figure 2(a) while Figure 2(b) shows the push-out specimens ready to be tested.

Figure 1. Details of push out test specimens

![Section A-A](image1)

![Section B-B](image2)

![Section C-C](image3)

Figure 2. (a) Experimental setup photograph for push-out test specimens; (b) Push-out test specimens ready to be tested

Table 1. The details of push-out test specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam</th>
<th>Connector</th>
<th>$H_{cs}$ (mm)</th>
<th>$H_{sb}$ (mm)</th>
<th>$C_1$ (mm)</th>
<th>$C_2$ (mm)</th>
<th>$P_b$ (kN)</th>
<th>$f_c$ (MPa)</th>
<th>$N_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT1</td>
<td>460 UB 82.1</td>
<td>M20</td>
<td>24</td>
<td>22</td>
<td>2</td>
<td>1</td>
<td>145</td>
<td>59.5</td>
<td>4</td>
</tr>
<tr>
<td>PT2</td>
<td>460 UB 82.1</td>
<td>M16</td>
<td>20</td>
<td>18</td>
<td>2</td>
<td>1</td>
<td>195</td>
<td>59.5</td>
<td>4</td>
</tr>
<tr>
<td>PT3</td>
<td>460 UB 82.1</td>
<td>M20</td>
<td>24</td>
<td>22</td>
<td>2</td>
<td>1</td>
<td>145</td>
<td>59.5</td>
<td>4</td>
</tr>
</tbody>
</table>

*Notes:* $H_{cs}$ = hole diameter in slab; $H_{sb}$ = hole diameter in steel; $C_1$ = clearance between bolt and hole in slab; $C_2$ = clearance between bolt and hole in beam; $P_b$ = applied bolt pretension; $f_c$ = average compressive cylinder strength of concrete slab; $N_c$ = number of connectors.

**LOADING PROCEDURE**

Loading was conducted in accordance with Eurocode 4 (2004) under displacement control by means of a hydraulic jack with a maximum load capacity of 5000 kN. In order to check the test set-up, a small load of about 10% of the predicted ultimate capacity of the specimens was applied before the main testing. In accordance with Eurocode 4 (2004), each push-out specimen was initially loaded to 40% of its expected ultimate load and then cycled 25 times between 5% and 40% of this expected ultimate load. Finally, the specimen was loaded monotonically until failure.
TEST RESULTS

At the early stages of loading, the magnitude of the slip between the precast concrete slab and steel beam was almost zero and this indicated very stiff and near to full shear interaction due to the friction generated by the pre-tensioned bolted shear connectors. Following this stage, the magnitude of the slip increased with a different trend until eventual fracture of the specimens. As a larger vertical load was applied, there was a separation at the interface of the precast concrete slab and the steel beam at the lower half of the specimens at the roller support side. Fracture of the four bolts was the failure mode of these three push-out test specimens.

As can be seen in Figure 3, the maximum load capacities were 193.6 kN, 127.2 kN and 207.6 kN per bolted shear connector with average interface slips of 14.93 mm, 12.80 mm and 18.23 mm being recorded for PT1, PT2 and PT3 respectively. Figure 3 shows that the bolted shear connectors have three distinctive stages that involve a region of full shear interaction, a region of almost zero shear interaction and a region of partial shear interaction. In these specimens at the first and third stages of shear interaction, composite action is produced by interface friction due to the pretension load and by bearing of the concrete and bolts, respectively. The first slip was observed to occur at loads of around 40 kN, 36 kN and 56 kN for PT1, PT2 and PT3 respectively, being followed by the onset of the second stage. In this second stage, the slip increased significantly due to the oversized holes and the clearance between the bolts and precast concrete slab and between the bolts and holes in steel beam. The value of slip for the second stage, which is around 3 mm, is almost same as the summation of the clearance between the bolts and holes in the precast slab, which is 2 mm, and between the bolts and steel beam, which is 1 mm. Once the slip exceeded this clearance, the third stage commenced and the bolts started to bear on the precast concrete and steel beam. In this stage, the shear interaction is very stiffer than the second stage and the applied load on the specimens increased significantly to around 172 kN, 105 kN and 173 kN for PT1, PT2 and PT3 respectively, followed by decreasing of the stiffness.

Comparisons of the ultimate load capacity, the load for the first major slip, the slip at first bearing, the average ultimate slip and the friction coefficient at the interface are summarised in Table 2. It can be concluded that the average slip at first bearing for all the specimens are similar as would be expected, and that the friction coefficient between the precast concrete slab and the steel beam varies between 0.275 and 0.38. In addition, the first major slip for PT1 and PT3 was observed to occur at different loads; about 40 kN and 56 kN for PT1 and PT3 respectively. These would not be expected since the same sized bolted shear connectors were used in both specimens. The reason for these results may be due to the different pre-tension load in the bolts and different locations of the shear plane as can be seen in Figure 4. This also caused an increase, of about 22%, of the average ultimate slip for PT 3.

COMPARISON OF STUD AND PRE-TENSIONED BOLTED SHEAR CONNECTORS

The test results were compared with those obtained from push-out tests of Loh et al. (2006) on stud shear connectors and the design value provided by AS 2327.1 (1992). As shown in Figure 3, for stud shear connectors there are two distinctive stages that involve a region of full interaction and then a region of almost no interaction. The composite action between the concrete slab and steel beam is produced by the bearing of the concrete and studs. However, for bolted shear connectors as illustrated in Figure 3, there are three distinctive regions as has been discussed.

Figure 3 also shows that the load and deflection capacities of the bolted shear connectors are much higher than those for stud connectors. The slip for the stud shear connectors is less than 1 mm until the load reaches around 100 kN, whereas the first slip for the bolted shear connector depends on the amount of pretension load applied in the connectors and usually commences at a load between 30 kN to 65 kN, which means that at the first stage of loading the studs resist larger loads than bolted shear connectors due to the oversized holes in bolted shear connection. However, after bearing of the bolts in the concrete slabs, the shear strength of the bolts increases significantly compared with the studs. In addition, unlike stud shear connectors, most of the shear strength of bolted shear connectors is in their
third stage, when the slip of the bolts exceeds the first major slip, which equal to clearance between the holes and bolts.

![Graph](image)

Figure 3. Load-average slip response for PT 1, 2 and 3.

![Images](image)

Figure 4. Detachment of the precast concrete slab from steel beam and fracture of bolts for (a) PT1; (b) PT2; (c) PT3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_f$ (kN)</th>
<th>$P_u$ (kN)</th>
<th>$S_a$ (mm)</th>
<th>$S_{au}$ (mm)</th>
<th>Shear plane</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT1</td>
<td>40</td>
<td>193.6</td>
<td>3</td>
<td>14.93</td>
<td>Threads</td>
<td>0.275</td>
</tr>
<tr>
<td>PT2</td>
<td>36</td>
<td>127.2</td>
<td>3</td>
<td>12.8</td>
<td>Shank</td>
<td>0.379</td>
</tr>
<tr>
<td>PT3</td>
<td>56</td>
<td>207.6</td>
<td>3</td>
<td>18.23</td>
<td>Shank</td>
<td>0.38</td>
</tr>
</tbody>
</table>

*Notes: $P_f$ = load per bolt for first slip; $P_u$ = ultimate load per bolt; $S_a$ = average slip at first bearing; $S_{au}$ = ultimate average slip; $\mu$ = friction coefficient.*

**CONCLUSIONS**

The results of three push-out tests with post-installed pre-tensioned bolted shear connectors have been reported. Based on these, the structural behaviour of bolted connections were assessed and discussed. It can be concluded from these push-out tests that:

- The behaviour of post-installed pre-tensioned bolted shear connectors is completely different to that of stud shear connectors. Stud connectors have two distinct stages that involve a region of full interaction and then a region of almost no interaction, whilst bolted shear connectors have three distinctive stages that involve a region of full shear interaction, a region of zero shear interaction and a region of partial shear interaction.
- Post-installed pre-tensioned bolted shear connectors provide robust and adequate shear connection for a precast concrete slab and steel beam system.
• The load and deflection capacities of the bolted shear connectors are much higher than for stud shear connectors.
• During the early stage of the testing, the friction between the concrete slab and the steel beam created by pre-tensioned bolted shear connectors carry the initial interface shear forces and the bolts experience little shear.
• The placement method of the bolted shear connectors and the position of the thread may have effect on the shear capacity and the ultimate slip of the connection.

ACKNOWLEDGMENTS

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