BEHAVIOUR OF HEADED SHEAR STUD CONNECTORS IN COMPOSITE BEAMS WITH UHPFRC CONNECTION

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Abstract

The outstanding properties of ultra-high performance fibre reinforced concrete (UHPFRC) promote innovative applications. The aim of the experimental program presented in this paper is to determine the performance in terms of capacity and ductility of the headed shear stud connectors in UHPFRC joints using a single side push-out test. A total of 5 specimens were tested with various parameters including, concrete type, haunches, stud length. The specimen performance related to the concrete type is considered. Test results showed that the failure mode of composite girder with UHPFRC connection is always by stud fracture through the weld collar. Compared with ordinary concrete, using shear connectors in UHPFRC leads to an increase in the load carrying capacity while presenting high ductility.

Résumé

Les performances exceptionnelles du béton fibré ultra-performant (BFUP) conduisent à l'innovation. Le but d'un programme de recherche expérimental présenté de cet article est de déterminer la performance en cisaillement de joints en BFUP in situ réalisés entre des dalles préfabriquées au-droit de poutres en acier. Des essais de type cisaillement simple ont été réalisés sur un total de 5 spécimens. Le type de béton (béton normal, béton fibré et BFUP) ainsi que la présence de goussets entre la semelle supérieure de la poutre et la dalle a été considérée. Les résultats expérimentaux ont illustré que la rupture des goujons gouvernait le comportement en présence d'un joint en BFUP. Les essais ont aussi démontré la supériorité des joints en BFUP en présence d'un gousset pour lequel une résistance moindre que celle prédite par les équations de conception a été obtenue en présence d'un béton normal. L'utilisation d'un joint en BFUP augmente la résistance de la connexion tout en présentant une ductilité élevée.

1. INTRODUCTION

The main strategic developments for precast composite bridge decks using UHPFRC field cast connections are to simplify interaction between reinforcing bars and shear connectors, accelerate construction on site while ensuring a higher performance and more durable decking system than traditional connections. The strength of composite slab does not only depend on the stud properties and stud configurations but also on the interaction with the surrounding concrete. It has been shown by different studies [1-3], that the compressive strength and stiffness of the concrete surrounding the studs influence the global behaviour. These properties mainly control the bending and tensile effects induced into the stud shank, hence the height over which the shear forces are transferred into the surrounding concrete, for normal and high strength concrete. With the use of UHPFRC, its ultra-high compressive strength and superior stiffness would have a direct impact regarding the load carrying behaviour of headed studs and ductility in a composite beam. However, because of its outstanding tensile properties, UHPFRC offers a higher tensile strength, shear strength, and consequently a better confinement around studs. Some studies considered the influence of UHPFRC in composite slabs [4-8]. Except Hegger et al. [5] who performed push-out tests on a specific set-up with a single stud embedded with UHPFRC cover, the other studies performed standard push-out tests on headed studs embedded in UHPFRC. In the case of composite bridge deck slabs, design requirements are more stringent than for the buildings, the fatigue-related aspects requiring, among other things, full composite action.

The general objective of this project is to analyse the behaviour of shear connectors embedded in UHPFRC with a test set-up that represents as closely as possible the load transfer between prefabricated bridge deck elements and steel girders through field cast UHPFRC composite connections with a composite action. This research will be used to develop guidelines and design rules for structures using this new construction technique.

2. EXPERIMENTAL INVESTIGATION

2.1 Single Side Push-Out Test

A Single Side Push-Out Test (SSPOT) has been developed to investigate the behavior of shear connectors in UHPFRC. The SSPOT consisted of a steel girder held in a vertical position connected on only one side with shear connectors to a reinforced concrete slab, as shown in figure 1. The concrete slab was connected to the beam flanges by shear connectors. Friction between concrete slab and bottom steel plate found in standard test is eliminated using a combination of a pin mounted on a bearing with rollers provided at both end of the test specimen. The steel girder is guided by a pinned-pinned strut at the upper level.

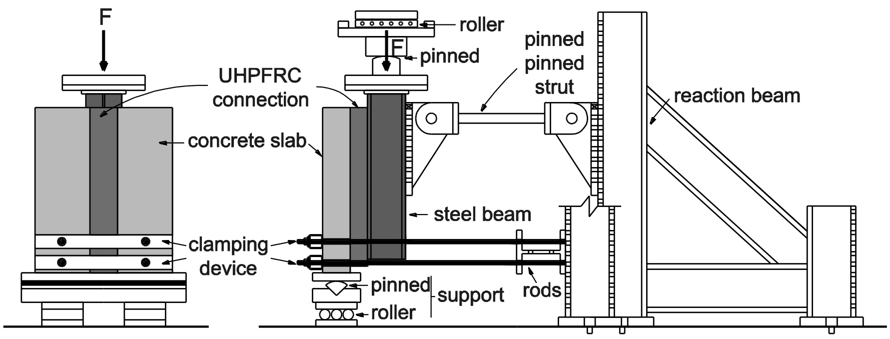


Figure 1: Single push-out test

2.2 Experimental program

The experimental project involved the testing of 5 specimens. Figure 2 shows the global dimensions, stud configurations and connection details. A summary of the geometrical properties, test parameters and the concrete properties are presented in Table 1.

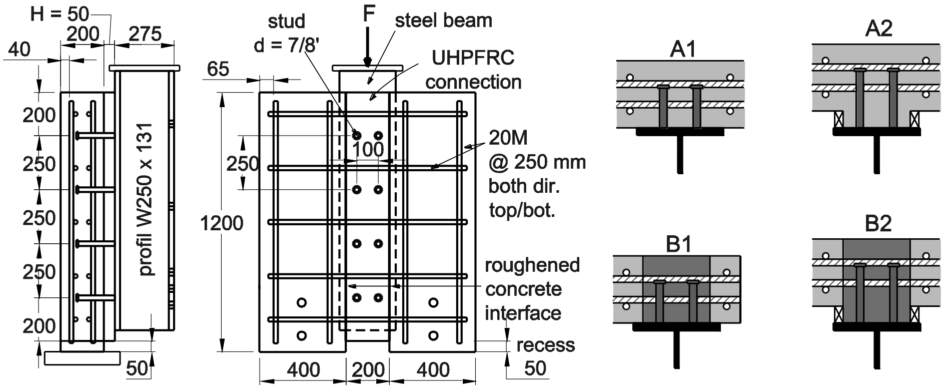


Figure 2: Specimen dimensions and stud configurations

The specimen dimensions were all 1200 mm long and 1000 mm wide. A recess of 50 mm at the bottom part of the slab was provided over a width of 200 mm equal to the width of the field cast UHPFRC connections. The reinforcement of the slab consisted of 20 mm diameter bars spaced at 250 mm in both directions, top and bottom. Headed stud of 22.23 mm (7/8') in diameter were employed. The studs were installed in two rows on the beam. A total of 8 studs was present for each specimen. The studs were spaced transversely and along loading direction center-to-center at 100 mm and 250 mm, respectively.

Table 1: Specimen Details

Specimen	Concrete		Haunch	Stud - d = 22 mm		Stud configuration	
	Slab	Connection	depth (mm)	Height (mm)	Aspect ratio (h/d)	Type	Lateral confinement (mm)
A1-N8-H0	NSC	-	-	132	5.9	A1	80
A2-N8-H50	NSC	-	50	182	8.2	A2	50
A2-F8-H50	HSFRC	-	50	182	8.2	A2	50
B1-U8-H0	HSFRC	UHPFRC	-	132	5.9	B1	80
B2-U8-H50	HSFRC	UHPFRC	50	182	8.2	B2	50

The parameters of the study were the type of concrete used for the slab, type of slab (solid slab or with haunch), and the presence of field-cast UHPFRC connection, as illustrated in figure 2.c and reported in Table 1. The experimental program can be separate in two series. In Series A, the slabs were all cast in place to serve as control specimens. In series B, specimens consisted of field-cast UHPFRC connections joining precast HSFRC slabs. The height of UHPFRC connection varied from 200 mm for solid slab specimen (B2-U8-H0) to 250 mm for specimen having a haunch. The *h*/*d* ratio of studs were all above 4, as per the code requirement [9] and varied according to the type of slab. For solid slabs, the *h*/*d* ratio was 5.9 and the distance between the axis of the stud shank and the end of the flange of the beam, corresponding to the lateral confinement provided by the slab to the stud, was 80 mm. For slab with haunch, an increase in stud height equal to haunch height was applied. It resulted in h/d ratio of 8.2 and a lateral confinement reduced to 50 mm (value still superior to 2 times the stud diameter).

2.3 Specimen preparations

The specimens were manufactured at BPDL precast plant, to replicated actual industrial conditions. Each slab and field-cast connection were cast horizontally. Casting was carried out from the middle of the slab formwork. No preferential orientation was therefore imposed for HSFRC slab. UHPFRC connections were cast after 28 days of slab curing. UHPFRC was poured from only one location at the middle of the connection without any vibration. Just after casting, the forms were sealed with a plastic film. After demoulding at day one, six days of moist curing was applied. Specimens were then stored under laboratory conditions until the testing day.

2.4 Material properties

The three concrete mixes used in this study are reported in Table 2. HSFRC were fabricated using 1% fibre volume fraction with hooked-end fibres (L_f = 30 mm, d_f = 0.55). The UHPFRC used were prepared from ultra-high performance premix, Ductal® (Lafarge) with straight fibres (L_f = 12.7 mm, d_f = 0.2) and a fibre volume fraction of 2%.

Table 2: Concrete mixes (Unit: kg/m³)

Concrete	Cement	Silica Fume	Sand		Ground Quartz	Coarse Aggregate	Superplast-icizer	Water	Steel Fibres
			Coarse	Fine					
NSC-35	370	-	804		-	1000	28	137	80
HSFRC-70	650	-	163	653	-	602	28	172	80
UHPFRC	712	231	-	1020	211	-	31	109	160

Table 3 summarizes the mean value of material properties of concrete tested after 50 days at the specimen testing date. Concrete cylinders were tested for compressive strength and modulus of elasticity. For HSFRC and UHPFRC direct tensile strength was obtained from 3 dog bone specimens with a constant cross section of 100 × 50 mm over a length of 300 mm. At maximum tensile strength, the UHPFRC exhibited a plastic plateau up to strain of 0.002, followed by a softening behaviour in the post cracking stage. Moreover, three and four points bending tests were performed. A detail of characterisation test results can be found in Gascon [10].

Table 3: Mechanical properties

Concrete	f'_c (MPa)	E_c (MPa)	v	f_t (MPa)
NSC	40	31000	0.26	3.42[(a)]
HSFRC	83	36000	0.23	4.5[(b)]
UHPFRC	170	58000	0.18	9.6[(b)]

[(a)] obtained from 3 splitting tests
[(b)] obtained from 3 dog bone specimens

Material tests on headed studs were not conducted. The material properties were obtained from the certified material test report. The yield strength and the ultimate tensile strength of studs reported were 398 MPa and 500 MPa, respectively.

The reinforcing bars were made of Grade 400. The measured yield strength and the ultimate tensile strength of reinforcing bars was 452 MPa and 586 MPa, respectively.

2.5 Loading protocol and instrumentation

Figure 3 presents the experimental set-up with the specimen and instrumentation installed. The specimens were tested in a servo-hydraulic testing. Each specimen was first cycled 25 times between 5 and 40% of the expected failure, as suggested by Eurocode 4 [11]. After cyclic loading, the specimens were tested under monotonic loading with a displacement control. The tests were stopped when the load had dropped to 20% below the maximum load.

Linear variable displacement transducers (LVDT) were arranged to evaluate the longitudinal relative slip between the steel beam and the concrete slab, as well as the separation of the concrete slabs from the steel beam, using triangulation principles. The relative slips between the steel section and the slab was taken as the average of LVDTs installed at the top and bottom of the specimen.

3. EXPERIMENTAL RESULTS

3.1 Load slip curves

The load slip curves are presented in Figure 3. The experimental results are compared to the design equation in CSA-S6-14, which specifies the design strength of headed stud as:

$$P_{Rd} = 0.5\phi_{sc} A_{sc} \sqrt{E_c f_c^{'}} \leq \phi_{sc} A_{sc} f_u \qquad (1)$$

where ϕ_{sc} = resistance factor for shear connectors; A_{sc} = area of shear connector; E_c = modulus of elasticity of concrete; f'_c = compressive strength of concrete cylinders and f_u = specified minimum tensile strength equal to 450 MPa for commonly available studs. Typically, for a concrete strength greater than 30 MPa, the design strength is governed by the ultimate tensile strength of the studs.

Thus, considering the nominal ultimate tensile strength, f_u, of the stud equal to 450 MPa and a shank diameter of the stud equal to 22.23 mm, the design strength, P_{Rd}, is expected to be at least 1397 kN for 8 studs, with ϕ_{sc} = 1. The ultimate strength of studs configuration used in this study, $P_{R,u}$, with the real stud tensile strength reported equal to 500 MPa is 1552 kN.

The global behaviour, ultimate load and ductility of the tests vary according to the type of material (NSC, HSFRC, UHPFRC). Specimen A1-N8-H50 with NSC slab and a 50 mm haunch reached a maximum capacity less than the design strength predicted by equation (1). Specimen A2-N8-H0 and A2-N8-H50 reached maximum load of the same order of magnitude as the ultimate strength calculated using the real stud tensile strength. Both specimen with UHPFRC connections (B1-U-N8-H0 and B2-U-N8-H50) exhibited very similar behaviour and the influence of a 50 mm haunch does not seem to affect the global behaviour of the composite beam. The maximum load was clearly higher than the reference specimens in NSC and the specimen with HSFRC slab, with less but still largely sufficient ductility. Figure 3 shows that UHPFRC properties influences the stiffness and deformation capacity of the shear connection.

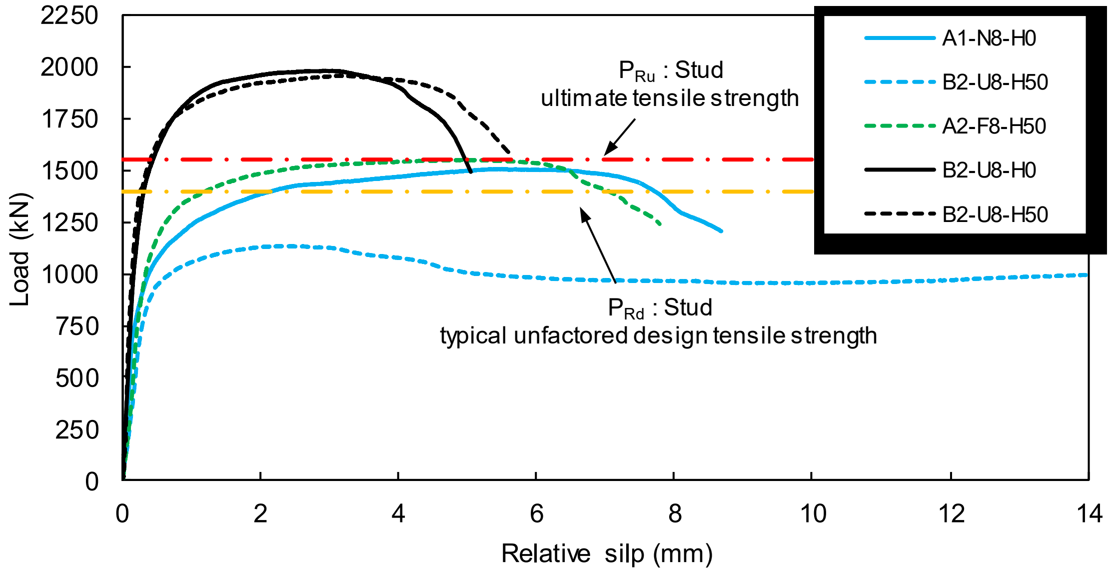


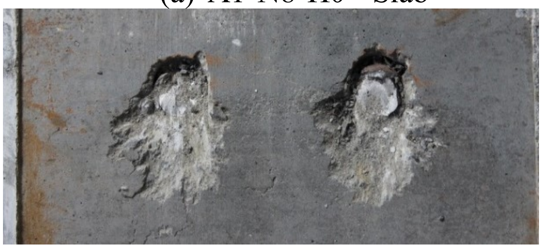
Figure 3: Load-slip curves with height headed studs

3.2 Failure modes

For NSC specimens, the failure mechanism with haunch are quite different to the one experienced in solid slab. Figure 4 illustrates the haunch effect on the failure mode. Specimen A1-N8-H0 failed by stud fracture (Figure 4.a) with local crushing failure of the concrete at the bottom of the studs (Figure 4.c).

(a) A1-N8-H0 - Slab

(b) A2-N8-H50 - Slab



(c) A1-N8-H0 – Local concrete failure

(d) A2-N8-H50 - Local concrete failure

Figure 4: Final cracking pattern after testing for NSC specimens

In presence of a 50 mm haunch (Figure 4.b), lateral confinement provided by concrete cover was reduced and the stud flexibility was increased by 40%. Thus, the bearing pressure exerted by the studs on the surrounding concrete led to spalling of concrete cover due to a lack of lateral confinement, which resulted to a smaller resistance for specimen A1-N8-H50 than predicted (Figure 4.b). The concrete near the studs was subjected to greater displacements with a more pronounced bending effect at the base of the shear connectors (Figure 4.d) which explains the major difference in the damage observed between the two reference specimens. The concrete in the haunch area governed the failure mode before shear connectors can developed their full capacity. For both specimens, all stud fractures appeared above the welded collar.

Haunches are commonly used in bridge construction due to deck drainage slope. From the design equation (1), the maximum resistance of studs in concrete haunches is taken as that for solid slab configuration. This assumption appears to be unconservative, according to the present experimental program, which reflects a typical haunch details used in practice. It can be stated that the design strength calculated from this equation should be applied only for solid slab.

For the HSFRC specimen with haunch (A2-F8-H50), the bearing pressure and bending effect exerted by the studs in the haunch did not generate premature concrete failure. The improved tensile properties of the HSFRC, with a softening stress-crack opening behavior led stud fracture related to the ultimate tensile strength of stud, with only local crushing failure of the concrete at the bottom of the studs (Figure 5.a).

(a) A2-F8-H50 - Slab

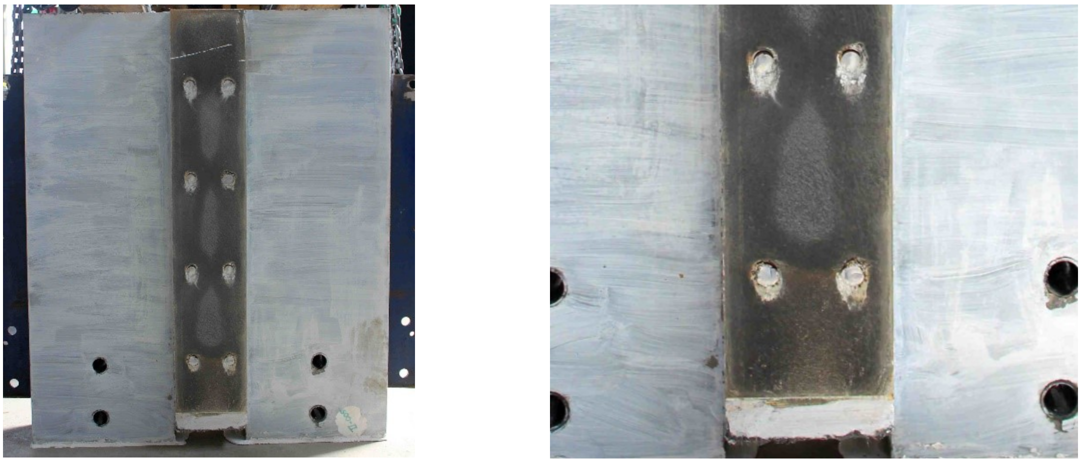
(b) A2-F8-H50 - Local concrete failure

Figure 5: Final cracking pattern after testing for NSC specimens

Figure 5.b shows the local concrete crushing of the concrete at the bottom of the stud, and some tensile cracks formed for which the fibre bridging effect controlled efficiently the crack opening and propagation in the concrete cover, after stud facture located above the welded collar. The relative slip obtained with this specimen with higher h/d ratio was comparable to specimen in NSC with solid slab. This result, however applies to a 50 mm thick haunch. Further tests would be required for thicker haunches.

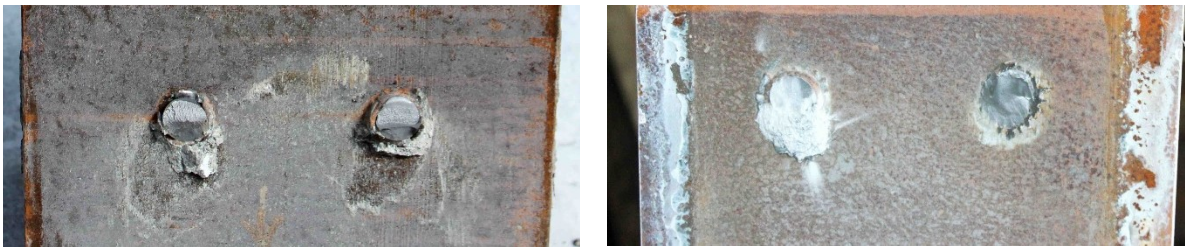
For specimens with UHPFRC connection, the failure mode was identical. Both specimens experienced fracture of the studs. Figure 6.a shows the final cracking pattern for specimen with 50 mm thick haunch (B2-U8-H50). Compared to specimens with NSC and HSFRC, the local crushing failure in front of the stud were markedly smaller, and the fractured studs remained nearly upright, with a smaller cavity formed behind each stud (Figure 6.b). No visible tensile cracks were noted.

The upper surface of the beam flange is shown in Figure 7, for solid slab specimen with NSC and UHPFRC connection. Two fracture positions were observed. For NSC, the facture surface was in the stud shank above the welded collar, with stud clearly tilted at this location (Figure 7.a). For studs embedded in UHPFRC, a clear sheared off at the bottom of the welded collar appeared (Figure 8.b).



(a) B2-U8-H50 - Slab (b) B2-U8-H50 - Local concrete failure

Figure 6: Final cracking pattern after testing for NSC specimens



(a) B2-N8-H0 (b) B2-U8-H0

Figure 7: Detail of failure surface location from the upper surface of the beam flange

The strength of studs embedded in UHPFRC were beyond the upper bound adopted in the design equation considering the real ultimate tensile strength (500 MPa), with resistance value higher than 400 kN. This significant over-strength could be attributed to a more active friction at the interface between the UHPFRC connections and the steel beam interface in front of studs. Indeed, when tensile stresses developed into the shank during the load transfer, a compressive strut develop in the concrete under the head of the stud to counterbalance this tensile force. This compressive forces activate additional friction locally at the steel concrete interface. Considering a tensile stress of 500 MPa in the studs, the compression force exerted at the interface is 1552 kN, which implies that the total friction contribution could reaches 500 kN, which corresponds to an effective friction coefficient of around 0.3. This phenomenon is possible with studs embedded in UHPFRC because its higher strength and stiffness reduce considerably the deformation capacity of stud, which could ensure that a static mode of friction is maintained until the stud fracture. Another probable contribution could be supplied by the welded collar portion, because the majority of shear forces were transferred through it, as found by Luo [8].

4. CONCLUSIONS

The objective of this paper was to experimentally investigate the behaviour of shear connectors in UHPFRC, with a configuration that represents as closely as possible the load transfer between prefabricated bridge deck elements and steel girder through field cast UHPFRC composite connections. The following conclusions can be drawn.

1) The use of a 50 mm haunch with a NSC slab led to premature failure and a lower strength than predicted by the CSA-S6 code. This failure was due to a very small concrete confinement on either side of the haunch. The design strength equation should in future take into account the stud location with respect to the lateral confinement provided by the surrounding concrete.

2) With a HSFRC slab containing 80 kg/m³ of fibres, the full capacity related to the ultimate tensile strength of studs was achieved, even in the presence of a 50 mm haunch. This behaviour is due to the brittleness elimination of the concrete supplied by the HSFRC.

3) The strength of studs embedded in UHPFRC are significantly higher than those of studs embedded in NSC and HSFRC, due to its high strength and stiffness which make possible to develop a greater friction force, but this over-strength should not be included in design equation for the moment.

4) Connection with UHPFRC showed a highly ductile behaviour but less than those made with NSC and HSFRC slab.

This experimental program demonstrated that the design strength of this type of connection can be calculated using the current design equation in the code CSA-S6 for HSFRC ($V_f = 1$ %) and UHPFRC ($V_f = 2$%) within the geometric parameters of this study. Additional tests on full scale composite beams would be needed to verify that the shear connection strength and failure modes experienced in the proposed single side push test are similar.

5. ACKNOWLEDGEMENTS

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