UHPC JOINTS OF PRECAST ELEMENTS

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Abstract

Steel concrete composite bridges are very efficient structures. Their efficiency may be increased if the concrete slab is made of precast elements. Then the joints, where also the headed studs are anchored, may become weak points of the system. The experience from already erected structures shows that the joints can be cast with UHPC (ultra high performance concrete), which results in excellent structural performance and also in increased durability of the joints. The small size of the joint required testing of the bond between reinforcing steel and UHPC. The results of the bond tests and the experimental verification of the model of the joint will be presented in this paper.

Keywords: Precast concrete, ultra high performance concrete, bond, joints, anchorage length

1. Introduction

Composite steel concrete bridges are very usual structures. In order accelerate the construction the idea of the precast bridge deck is very interesting. However, there is a difficulty in joints. The reinforcement from individual parts of the precast slabs has to be connected in the area, where a number of stud connectors carry shear force between steel and concrete. The transversal bending moments are also extreme in this area. The application of the advanced high performance material may be a good alternative for this highly stressed detail. The dimensions of the joint could be significantly reduced and the high quality of the material can transfer the high stress in the area. Therefore the ultra high performance concrete (UHPC) was designed to fill the joint of the precast slabs. The experimental research which should verify this idea had two steps. 1. Verification of the anchoring length of the steel bars and 2. Verification of the joint on the model in the scale 1:1 on transversal bending moments.

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2. Bond of steel reinforcement and UHPC

There are various test methods how to determine bond between steel reinforcement and concrete. The RILEM RC6 recommendation assumes testing using a pull out test. The steel bar is embedded in the cube and pulled out (Fig. 1). The bond stress along the embedded length of the bar in concrete is not uniform. In order to simplify the evaluation of experiments the assumption of uniformly distributed stress – average stress – is accepted. The average shear stress is given by the ratio of tensile force in the reinforcement P_m and contact area (the product of the anchorage length of the steel bar a and the perimeter of the bar o) (Eq.1).

$$\tau_m = \frac{P_m}{a \cdot o} \qquad (1)$$

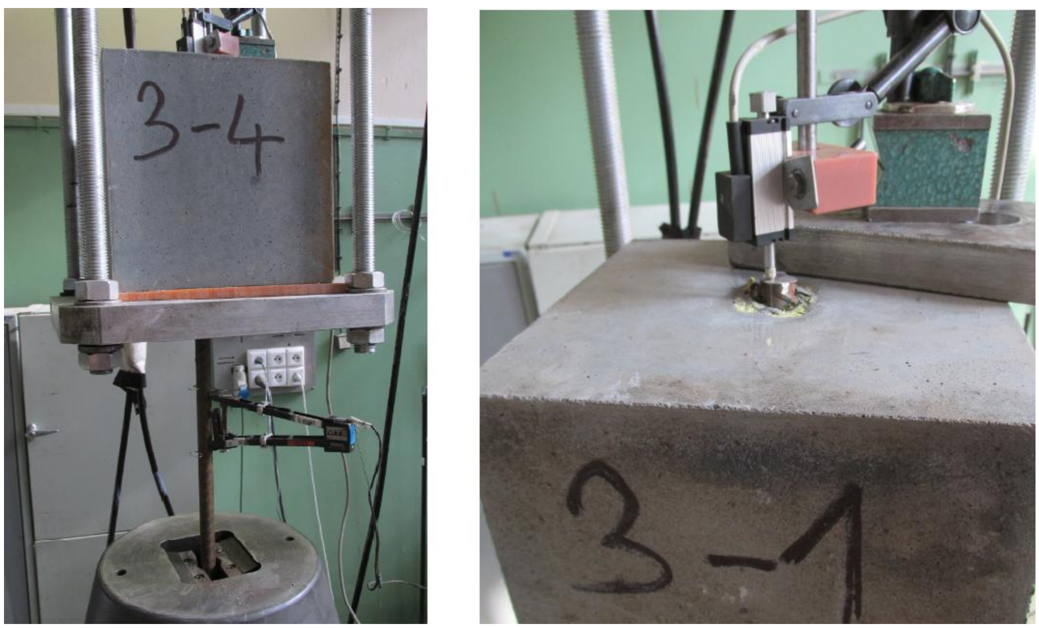

Fig. 1: Specimen with potentiometric sensor on the top of unloaded end of reinforcement

2.1 Different diameters of steel reinforcement

In the first part of the experiment the three diameters of bars – 12, 16 and 20 mm were tested. The steel bars were embedded into the concrete cube with dimensions 200 x 200 x 200 mm with the anchorage length of 5 x diameter of the steel bar (according to the RILEM recommendation). Then the bars were pulled out of concrete cube by tensile force induced by the testing machine (Fig. 1).

All tests of bars embedded in UHPC cube finished by failure of the steel bar in tension, while no damage in concrete was observed (Fig. 2). It means that the anchorage length of 5 diameters is long enough that no failure of concrete was observed and the tensile force is reliably transferred from steel bar to concrete [1].

Fig. 2: Types of steel failure (specimen made from UHPC)

For comparison the same tests were carried out using cubes made of ordinary concrete of the class C30/37. The failure of bond was observed at all tested specimens. The average bond stress was significantly lower than that measured in the specimens made of UHPC (Fig. 3).

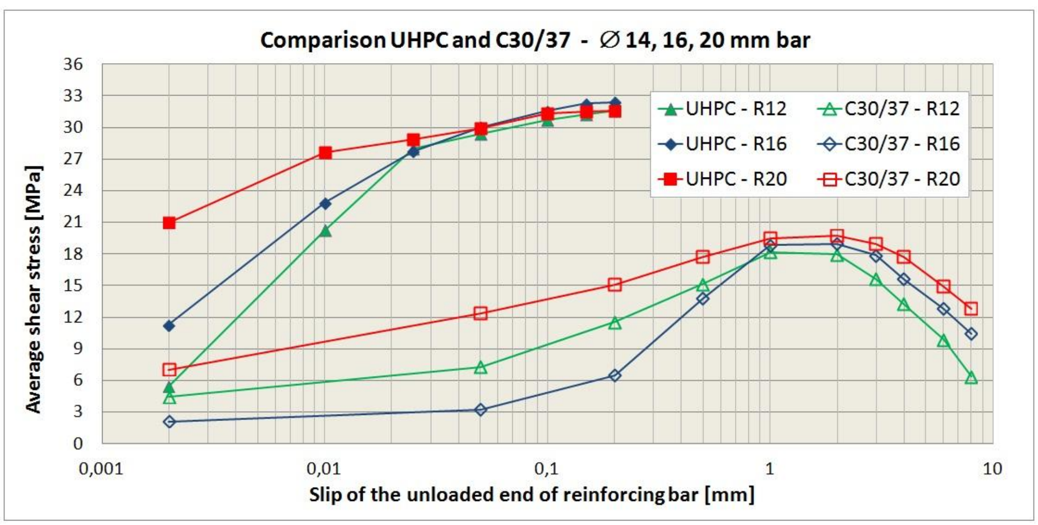

Fig. 3: Comparison of the bond behaviour of UHPC and C30/37 with different bar diameters

2.2 Reduction of the anchorage length

The second series of tests was focused on the reduction of the anchoring length to 2 diameters, 3 diameters and 4 diameters. The tests were executed using the reinforcing bars 16 mm in diameter. The balance between tensile strength of the bar and the bond capacity at reduced anchoring length was searched. The same test procedure as in the first part of experiment was used.

These tests showed that anchorage length of 4 diameters is sufficient (the failure of steel bar was similar to those with the anchorage length of 5 diameters). The failure in bond appeared when the anchorage length was reduced to 2 diameters of steel bar. The tests with the anchorage length of 3 diameters resulted partially in failure of steel and partially in failure of bond. The maximum average shear stress between the steel bar and UHPC was 55 MPa while at the specimens made from C30/37, the maximum average shear stress was only about 18 MPa (Fig. 4.) [1].

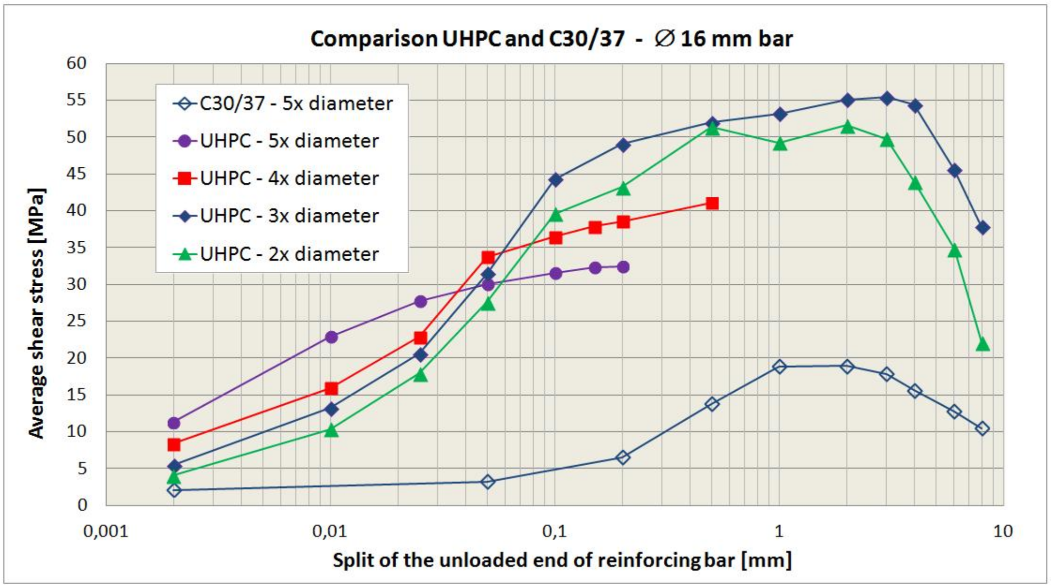

Fig. 4: Comparison of the bond behaviour of UHPC and C30/37 with different anchorage length

3. UHPC joints of precast elements

The research is focused on the performance of the precast concrete deck of a steel concrete composite bridge. The precast elements of the slab will have the longitudinal joints above the steel beams and transversal joints which will be perpendicular to the steel beams. The longitudinal joints are subjected to a large bending moment in transversal direction. Additional to that, the stud connectors welded to the top flange of the steel beam will be embedded in the joint, which results in additional stress in the joint. The application of UHPC as a filling of the joint has significant advantages. 1. The high strength of UHPC allows for reduction of the width of the joint, because the anchoring length of steel bars can be reduced (as it was already shown). 2. The smaller dimensions of the joint result in reduction of the costs, since the UHPC is expensive. 3. The high strength of UHPC will contribute to the load carrying capacity of the shear stud connectors. 4. The joint can be easily located only above the top flange of the steel beam, i.e. no additional formwork is necessary. The precast slabs can be supported on the edges of the steel flange and still there is enough space for the joint. In the first step only the bending behaviour of the joint was experimentally verified.

3.1 Design and fabrication of the UHPC joints of the precast elements

The thickness of the bridge deck slab of 250 mm was assumed, since it corresponds to the realistic bridge design. The model was designed 0.6 m wide and 2.8 m long. The width of the joint was only 200 mm, which allows for a comfortable placement of the reinforcement and also the volume of the joint is not large (Fig. 5). The two arrangements of the reinforcement of the joint were tested. The first arrangement (type R) had only straight bars coming out from the precast slabs (Fig. 6 left). The second arrangement (type S) had the loops made of reinforcing bars which overlapped in the joint. The profile of the steel was identical in individual alternatives (14 mm).

The precast elements – model of deck panels were made from ordinary concrete C40/50. After hardening of the panels the joint was cast using UHPC. The UHPC has a cylinder concrete strength over 150 MPa, the flexural strength about 18 MPa and it contains about 2% of short high strength fibres.

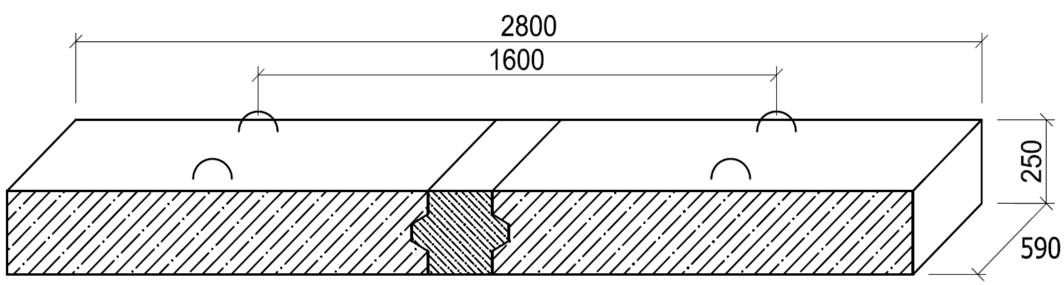

Fig. 5: Scheme of the specimen with dimensions

Fig. 6: Straight bars in UHPC joint (type R) and bars in the loop (type S)

The self-compacting UHPC was used; therefore no additional compacting was necessary. This is also important for future practical applications, since the compacting would not be easy due to the relatively dense reinforcement of the joint. The surface of UHPC was left without any additional smoothing and it was carefully treated with water and covered with PE foil to prevent evaporation. After hardening, there were not found any cracks in the UHPC joint resulting from shrinkage strains.

3.2 Test procedure

The testing procedure represents the transversal bending of the bridge deck above the flange of the steel beam. The test setup is arranged in the opposite position, i.e. the reaction of the flange is represented by a force ***F*** acting in the middle of the model (Fig. 7). The top flange of the beam is simulated by the steel plate under the loading force.

The experiment involved testing of 6 specimens. 3 specimens had the reinforcement of the joint of type R and the other 3 specimens were reinforced by the type S. The other parameters were identical.

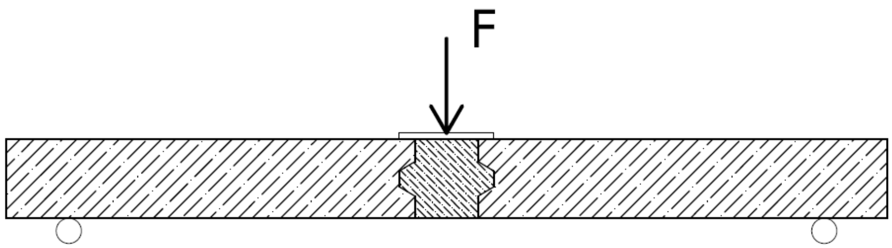

Fig. 7: Scheme of the test

The deflection was measured at the midspan at two points and also the displacements at the supports were monitored. The potentiometric displacement sensors were used. The specimens were loaded in 5 cycles up to the level of the serviceability load (about 50% of

Fig. 8: Test specimen during test

estimated ultimate load). Then the load was increased until failure. The loading process was controlled by force when loading up to the serviceability level and by deflection growth, when loading until failure. Loading rate was set at 0,2 kN/s at the repeated loading and then the speed of the deflection growth was set to 0,05 mm/s for the failure loading. The test was terminated after a significant decrease of load forces or if deflections grew at constant load. The objective of the tests was twofold: 1. Load – deflection diagram and 2. Observation of cracks in relation to the load level and their position in relation to the joint.

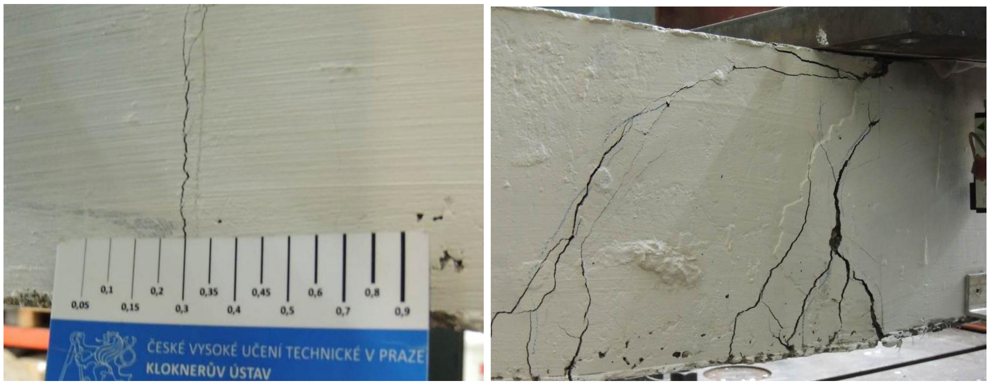

Fig. 9: Measurement of crack width, expansion of main cracks

3.3 Results

Similar pattern was observed at all tests. The first hair cracks appeared at the load level about 40 kN. At the estimated serviceability load, the crack width was about 0.15 to 0.25 in average in dependence on the reinforcement (Tab.1). Such result would be completely acceptable for a characteristic load combination in SLS. No cracks were observed at the UHPC joint, they were on the boundary between the two concretes or at the precast part of the model, i.e. in ordinary concrete (Fig. 9).

Tab.1: Test results

Number of panel	Loading force - first crack [kN]	Maximum crack width under serviceability load 75 kN [mm]	Ultimate loading force [kN]
R1	40,0	0,2	167,5
R2	35,0	0,25	168,5
R3	38,0	0,3	177,2
Average of R	37,7	0,25	171,1
S1	45,0	0,1	164,1
S2	36,0	0,15	157,4
S3	40,0	0,15	159,6
Average of S	40,3	0,13	160,4
Average of all specimens	39,0	0,19	165,7

Collapse of the model was achieved at the load level slightly higher than expected (in the range 160 kN to 170 kN, in dependence on the reinforcement type). The main failure crack was located either on the boundary between UHPC and ordinary concrete (at the loop reinforcement – type S) or in ordinary concrete (at the majority of specimens with straight reinforcement – type R). The load displacement diagrams of all specimens are plotted in Fig. 10.

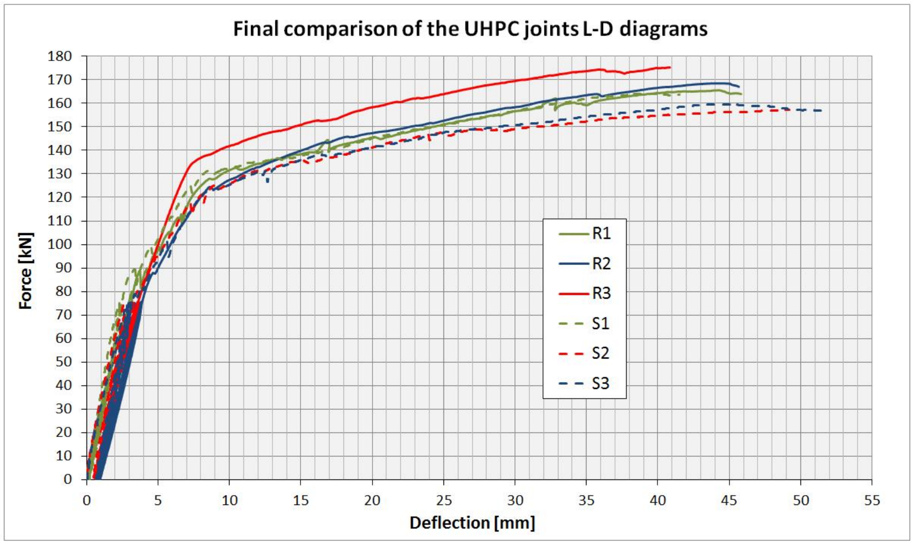

Fig. 10 Load – deflection diagrams of all specimens

4. Conclusions

The bond tests showed that the anchoring length of the reinforcing bars in the UHPC can be significantly reduced in comparison with that used in ordinary concrete. The anchoring length of only 4 diameters appeared as completely sufficient, the bars failed in tension, while the bond was not subjected to any damage. Such result was optimistic for design of the reinforcement of the UHPC joint of precast elements of the bridge deck of the steel concrete composite bridge. The 6 tests of the joint were carried out. Under the serviceability load the cracks appeared only in ordinary concrete or at the border between ordinary concrete. Their width was in majority of specimens about 0.15 – 0.25 mm, which complies with requirements of codes. The ultimate load was slightly higher than expected, and again no cracks appeared in the UHPC. The existing cracks originated at lower load levels extended until complete failure of models. The experiments showed that the concept of the UHPC joints of precast elements works well and can be used in structures.

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