UHPC Connection of Precast Bridge Deck

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Abstract: Ultra high performance concrete is very promising material for many construction applications. Its outstanding features are generally known [1][2]. UHPC should be used in complex structural details like joints of precast elements. Acceleration of bridge construction requires the development of new construction methods. In composite steel concrete bridges the cast in situ bridge deck may be replaced by a precast deck. For fast assembly the joints have to be solved adequately. UHPC joints represent an alternative which can satisfy the requirements on mechanical properties of joints, on construction and economy. The function of the joints and connections was experimentally verified using three different experiments. The performance of beams with precast slab with UHPC joint and cast in situ slab was compared.

Keywords: UHPC, connection, bond, precast deck, composite beams, testing

1. Introduction

One of the fields of application of ultra-high performance concrete (UHPC) may be found in composite steel concrete bridges with precast concrete slabs. The use of precast concrete slab can accelerate the construction and save some costs. The joints which might become weak points may be produced using an UHPC, so that the amount of the material is low, the costs are not influenced significantly, the hardening of UHPC is fast, the assembly may be fast and the stresses are transferred reliably since the quality of UHPC is high. The excellent bond between reinforcing steel and concrete allows for a significant reduction of the joints, where the reinforcement is connected. Additionally the joints are also located above the steel beams where the shear is transferred between the steel beam and a concrete slab. Stress concentrations can be also favorably transferred in the UHPC and a number of shear connectors may be reduced in comparison with their number embedded in ordinary concrete. In the paper, experimental verification of UHPC developed in the Czech Republic is described. First part deals with the experimental tests on bond of the steel in the UHPC, the second part is focused on the evaluation of the tests where the slab is subjected to bending and the last part describes the tests of the composite steel concrete beams.

2. Background – design of composite beams with UHPC joint

Excellent results observed worldwide in many real applications of UHPC leads to design experimental model of steel-concrete composite beam with UHPC joint. The tests should verify the performance of precast concrete slab with the joint made of UHPC over the steel beam and to compare it with performance of a traditional design, i.e. with the performance of a continuous cast in situ slab of the composite beam. In the joint over the steel beam, there is also a shear connection. Most often the headed studs are used as shear connectors. The headed studs are designed for application in ordinary concrete. They are design according to the codes to resist in ultimate and serviceability limit states. The studs may fail in two modes. a) Failure in concrete, i.e. the concrete surrounding the stud cracks of crushes. b) Failure in steel, i.e. concrete does not fail, but usually the welded joint between steel flange and the stud fails. The studs are designed so that the forces at failure of steel or concrete should be similar. Traditional studs designed for ordinary concrete are rather long and slender. Some of the innovative solutions of different types of shear connections were published at [5][6]. If the studs were embedded in the UHPC their shape would need to change. Their anchorage in concrete is very strong and then the load carrying capacity of concrete increases significantly. The studs for application in the UHPC should be shorter and their diameter should be larger. Such studs are not on the market. Therefore the perforated steel sheet was designed for the shear connection instead of the headed studs. The capacity of the perforated steel sheet which is welded along its length to the steel flange of the beam is determined by the area which resists to the shear force in the joint. The teeth on the top part of the perforated sheet increase this area, which is given by the height of the teeth and by the thickness of the perforated sheet. Similarly to the studs, the connection can fail in concrete or in steel. The dimensions of the perforated sheet should be designed so that the load carrying capacity of concrete and the load carrying capacity of the welding connection should be similar.

In the experiments the steel perforated sheet was designed initially for application in ordinary concrete. A continuous perforated sheet with the thickness of 6 mm with the teeth 30 mm deep was designed. For the transfer of shear between UHPC and steel beam only smaller elements were designed (not a continuous sheet) with the thickness of 10 mm. In simply supported beams the maximum bending moment is at the midspan and the maximum shear force is at the support. If a continuous beam is assumed, then large shear force and large bending moment in the longitudinal direction are located at the intermediate support area. Also the slab is subjected to the transversal bending induced by loading acting between the steel beams. In the support area the negative bending moments in longitudinal and transversal directions result in cracking of the slab, which may reduce the capacity of the shear connection. The experimental modelling of the support area became an objective of the research.

3. Testing Methods

Experimental program was divided into three parts which will be described in the next chapters.

3.1. Bond of reinforcing steel in concrete

Extensive research program of bond between reinforcement and concrete and especially UHPC was carried out in Klokner Institute, CTU in Prague. UHPC is relatively new material a lot of experimental research and verification of material parameters need to be done [3]. For evaluating the average shear stress on the boundary of the steel bar and concrete the the pull-out tests according to RILEM RC6 recommendation and Czech Standard 73 1328 were used. In both documents anchorage length of 5x diameters of rebar (ribbed, yield strength 500MPa) embedded in cube 200x200x200 mm is used. In order to simplify the evaluation of experiments the assumption of uniformly distributed stress is accepted. The average shear stress is given by the ratio of tensile force in the reinforcement and contact area between steel bar and concrete.

In the first part of the experiment, the three diameters of steel bars embedded in UHPC were tested (12, 16 and 20 mm). For comparison the same tests were carried out using the cubes made of ordinary concrete of the class C30/37. The failure of steel bars was observed at all specimens made of UHPC. Compared to these results the failure of bond was observed at all specimens made of ordinary concrete. Therefore the average bond stress was significantly lower than that measured in the specimens made of UHPC. Tests of UHPC specimens proved that anchorage length according to RC6 recommendation (5x bar diameter) is more than sufficient.

The second series of tests was focused on the reduction of the anchoring length to 4 diameters, 3 diameters and 2 diameters of the steel bar. The balance between tensile strength of the bar and the bond capacity at the reduced anchoring length was searched. These tests showed that anchorage length of 4 diameters is still 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 highest average shear stress was reached by specimens with the anchorage length of 3 diameters. In this set of tests, some of specimens partly failed in bond and partly in steel. At this value of anchorage length, the largest average bond stress was observed. It can be concluded that using of UHPC significantly (about 2.5 times) increased the maximum average bond stress compared to that at ordinary concrete C30/37.

3.2. UHPC joint of precast deck

Because of the excellent bond results, the experimental joint of two precast elements was designed. The experiment 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 [4].

3.2.1. Design and fabrication of the UHPC joints of the precast deck

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. The second arrangement (type S) had the loops made of reinforcing bars which overlapped in the joint. The diameter of the steel was identical in both alternatives (14 mm).The precast 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 contained about 2% of short high strength fibres. 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, no cracks in the UHPC joint were observed resulting from shrinkage strains.

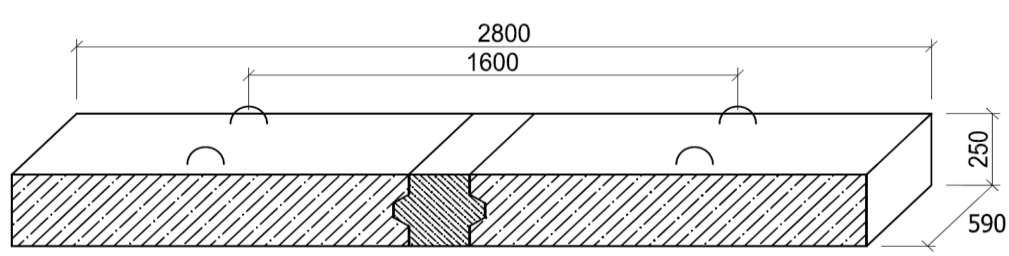

Figure 1. Scheme of the precast slab with UHPC joint

3.2.2. Test procedure

The testing procedure represents the transversal bending of the bridge deck above the flange of the steel beam. The slab with the joint in the middle was subjected to the three point bending. The experiment consisted of 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 for all specimens. The specimens were loaded in 5 cycles up to the level of the serviceability load (about 50% of estimated ultimate load) – 75kN. 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. The test was terminated after a significant decrease of load forces or if deflections grew at constant load. The research of the behavior of the slab with the joint was the main objective of the tests. The load displacement diagram was recorded and the crack pattern was observed during the complete test.

Similar performance was observed at all tests. The first hair cracks appeared at the load level of about 40 kN. At the estimated serviceability load, the crack width was about 0.15 to 0.25 in average in dependence on the type of reinforcement of the joint (R or S). There was no failure of any specimen in bond of reinforcement in the UHPC joint, which was considered as an important conclusion. Such result would be completely acceptable for a characteristic load combination in SLS. No cracks were observed at the UHPC joint, they appeared in the interface between the two concretes or in the precast part of the model. The ultimate load varied in the range from 160 kN to 170 kN, in dependence on the type of reinforcement of the joint (R or S). The specimens with simple overlap of reinforcement exhibited slightly higher load carrying capacity which proved that reduced anchorage length is sufficient. The main failure crack was located either in the interface 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. 2.

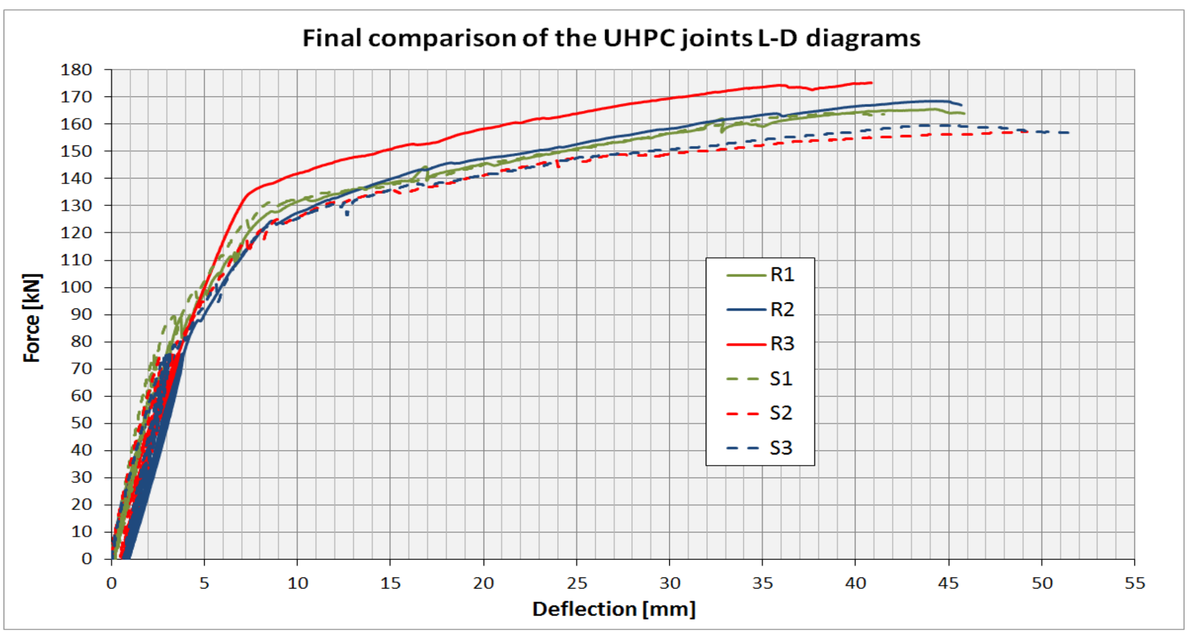

Figure 2. Scheme of the precast slab with UHPC joint

3.3. UHPC joint of precast deck

3.3.1. Design and production of experimental models

Taking the conditions mentioned above into account, simply supported beams with long cantilevers were designed. The load was applies on the cantilevers, and the short span represented partly fixed intermediate support of the beams. The loading was divided into two forces, which were active on the edges of the slab, so that the transversal bending moment was also developed.

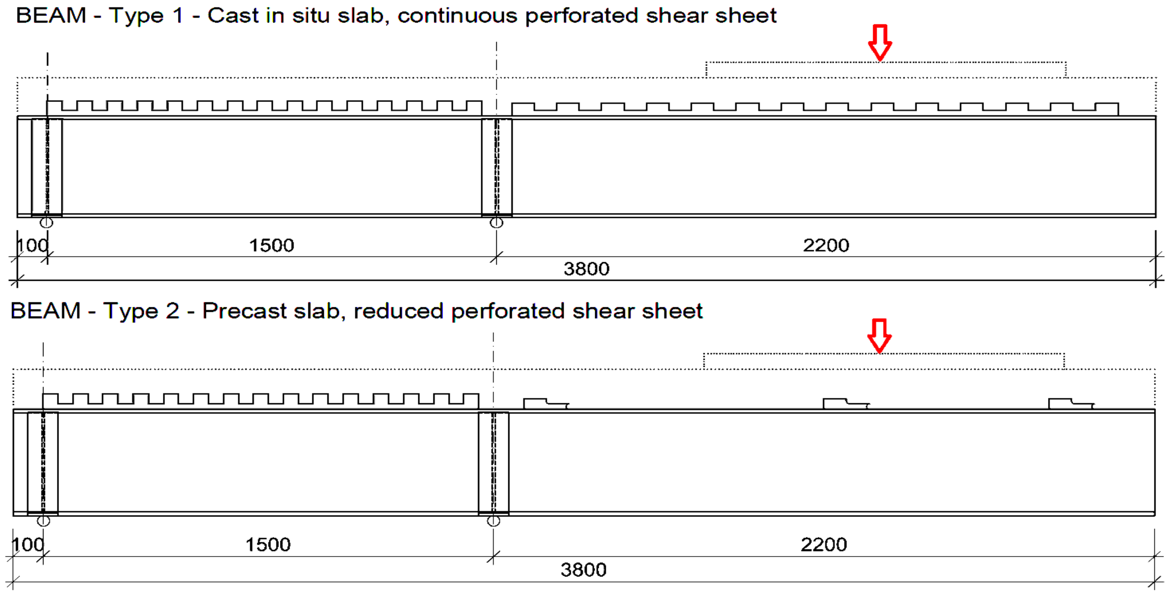

Figure 3. Longitudinal section of the tested beams

Two types of beams were tested. The beam of the type 1 was a classical composite beam with cast in situ slab without any joint. The beam of the type 2 was a model of a composite beam with a precast concrete slab. There was a joint over the top flange of the beam, which was cast later using UHPC. The longitudinal views on the individual beams are plotted in Fig. 3.The cross-section of the beams of the type 1 and 2 is plotted in Fig. 4. The reinforcement coming out from the precast edge parts of the precast slab overlaps in the joint. No other connection of the steel bars was made.

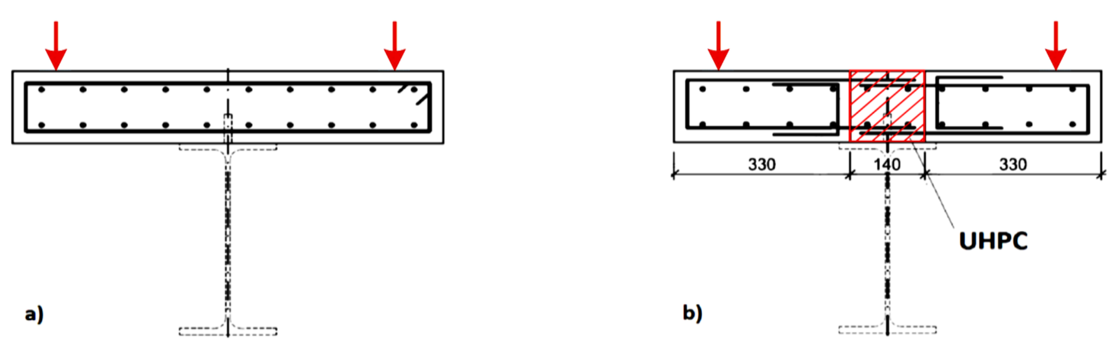

Figure 4. Cross-section of the beams. a) Type 1, b) Type 2

3.2.2. Test procedure

The load acting as two forces on the cantilever at the distance 1.3 m from the intermediate support was distributed along the length of about 1 m using steel elements located under the hydraulic jacks. During the loading, the deflections were measured at the end of the cantilever, under the point loads and also the displacements of the supports were monitored. The potential slip between the concrete slab and the top flange of the steel beam was also measured at the three locations, at the end of the cantilever, in the middle of the cantilever and close to the intermediate support. The crack development was recorded during the loading process. The transversal cracks appeared over the intermediate support due to the longitudinal bending and longitudinal cracks were recorded preferably on the top surface of the cantilever between the point loads. First the beams with cast in situ slab were tested. They were considered as reference beams. Then the beams with joint were tested. The test setup is illustrated in Figs. 5 and 6. During the testing, it was found that the stability of the bottom flange under the intermediate support is the weakest point of the system and the reason for finishing the test, without failure of the shear connection, or top slab due to cracking. The bottom flange was at some specimens stiffened, which led to higher load carrying capacity and stiffness. Then larger cracking of the top slab could be observed.

Figure 5. Beam Type 1 (Cast in situ slab)

Figure 6. Beam Type 2 (Precast slab)

4. Results

Comparison of the structural response of the beams with cast in situ and with precast slab was the main result of the experiments. The slab was subjected to the longitudinal and transversal bending and to shear forces. It is a very unfavourable loading situation which was experimentally investigated. The perforated steel shear sheet worked well at all experiments. A very little slip was measured during the tests, with exception of the section close to the intermediate support at high load levels (close the failure load). The slip of the order of 0.6 mm was recorded, which may be a consequence of the opening of large cracks in the support area. The slip in the middle of the cantilever was smaller and the slip at the end was almost negligible. The slip at the beams with the UHPC joint was a little bit larger than that at the cast in situ beams. It may be explained by elastic deformation at the connectors, where large stress concentrations appear. It is not the case of the cast in situ slabs, where the shear is transferred continuously. At the serviceability load level the slip would be about 0.1 mm at all beams, which is a negligible value. It may be concluded that the perforated steel shear sheet is a good alternative of studs in studied beams. Transversal cracks were observed at rather small loads. It is completely all right, since the slab is in tensile zone of the beam. At the cast in situ slabs, the cracks were continuous across the complete width of the slab. At the precast slabs with the joint, the cracks developed more in the precast part (ordinary concrete) and only some of them went through the UHPC joint. In the joint the number of cracks was lower. During the loading process they opened more, finally the crack width in the UHPC joint was larger than the crack width in ordinary concrete. At the serviceability limits state the crack width was small at all beams (0.2 - 0.3 mm). Longitudinal cracks developed in the area between the loads on the top surface of the cantilever. At the beams with cast in situ slab, the first longitudinal cracks were observed in the middle of the concrete slab, above the shear connectors. Then further parallel cracks appeared in the slab. At the beams with the joint, no cracks were located in the joint. The longitudinal cracks developed in ordinary concrete or in the interface between the UHPC and precast slab. It means no crack was in the position of the shear connectors.

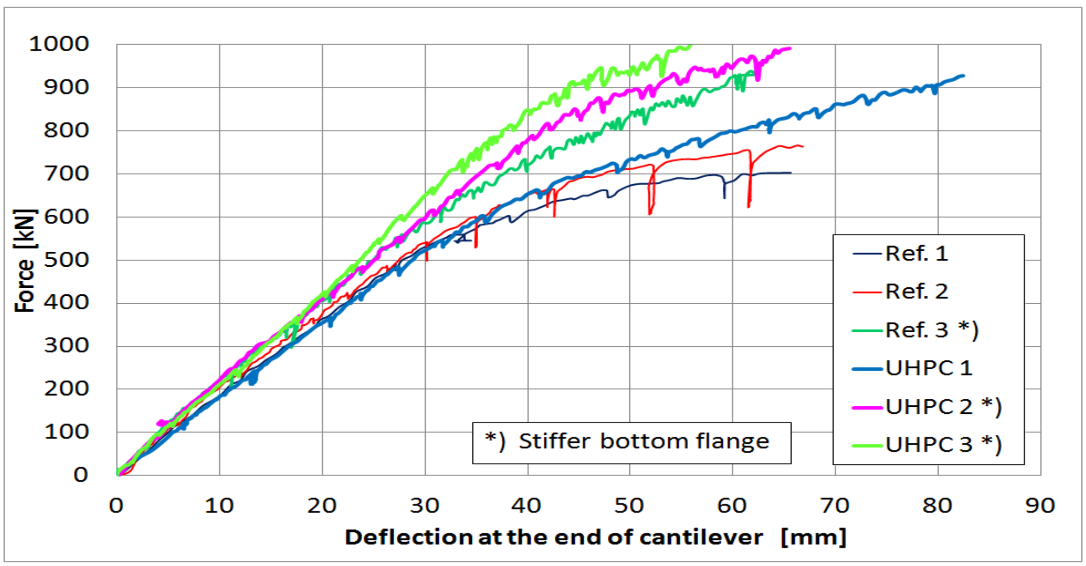

Figure 7. Cross-section of the beams. a) Type 1, b) Type 2

The beams with cast in situ slab are designated as reference beams. The two kinds of curves may be observed. Lines Ref 1, 2 and UHPC 1 had a weaker bottom flange of the steel beam. The stiffness of all three beams is lower than that of the others, but very uniform until the load of about 600 kN, which is well above the serviceability limit state. At larger load levels the beam with the UHPC joint is stiffer and load carrying capacity is higher. The second group of beams with stiffened bottom flange of the steel beam exhibits a similar behaviour. The stiffness is very similar again up to the level of 600 kN. Then the beams with the UHPC joint are slightly stiffer, the ultimate load is identical. The ultimate load was not achieved by direct failure of some part of the beam. The slow loss of stability of the bottom flange of the beams and crack opening of the cracks over the support were the reasons for finishing the tests at the max. load level of 1000 kN. This value is well above the designed load carrying capacity.

5. Conclusions

The UHPC developed by TBG Metrostav, Ltd, was tested in several tests. First the bond of reinforcing steel and UHPC was investigated. It was found that the anchorage length of the rebars may be significantly reduced. Such reduction may be effectively used in joints of the concrete elements. Their dimensions may be reduced because of the short overlapping length. Next step was focused on the joint of the slabs subjected to bending. Very good load carrying capacity was achieved using a simple arrangement of the reinforcement. Cracking never appeared in the joint, but in the ordinary concrete or in the interface between the UHPC and ordinary concrete. In the last step the composite beams with the precast slabs connected with the UHPC joint were experimentally investigated. Their response was compared with the response of the reference beams with cast in situ slabs without any joints. The behavior of the precast slabs with the UHPC joints was similar or even slightly better than that of cast in situ slabs. On the basis of the tests which were evaluated, it can be concluded that the precast slabs with UHPC joints represent an equivalent design to the cast in situ slab. However, at the UHPC joint there is an advantage of the simpler arrangement of the shear connectors, which results savings in the steel beam and makes the design of locally grouped shear connectors possible. It will also simplify the design of precast slabs arrangement at composite steel concrete bridges.

6. References

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7. Acknowledgements

This contribution is the result of the research supported partly by the grant projects of the Czech Science Foundation - project No. P105/12/G059 and partly by the project of the Technological Agency of the Czech Republic (Research centre CESTI, project No. TE 01020168).