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Structural Behaviour of UHPC under Tensile Stress and Biaxial Loading

Summary

An extensive test programme on Ultra High Performance Concrete (UHPC) has been started at the Chair of Structural Concrete of Kassel University in summer 2004. The purpose of these investigations is to analyse the interaction of steel fibres and conventional bar reinforcement under tensile stress as well as the influence of cracking on the compressive strength of UHPC. The tests are conducted on panel-shaped structural members and on tensile members with different cross-sections. The objective of these tests is to develop constitutive models for cracked UHPC under tension and under compression. Based on this, calculation methods for ultimate limit state (ULS) as well as for serviceability limit state (SLS) shall be derived (calculation of crack width and deflections, ultimate limit state design).

Keywords: *tensile stress, biaxial loading, crack width, tension stiffening, concrete design*

1 Introduction

Ultra High Performance Concrete [1] is a very dense structured, fine- or coarse-aggregated concrete. A low water/cementitious binder ratio < 0.35 , high effective concrete admixtures and the use of cement and mineral admixtures which are selected in aggregate composition and aggregate form (low hollow space) lead to compressive strengths near to the yield strength of structural steel (approx. 150 to 230 N/mm²). Because of the compressive strength, this kind of concrete is named Ultra High Strength Concrete (UHSC). However, UHSC offers not only a high compressive strength but also a great spectrum of several other favourable characteristics.

As an example the high resistance against each form of physical or chemical attack, resulting from the dense structure, is to be mentioned. The whole of these favourable material properties is described by the term Ultra High Performance Concrete (UHPC).

From the structural point of view, the behaviour under tensile stress is of substantial interest as well. The tensile strength of UHPC attains a magnitude of 1/25 to 1/20 of the compressive strength. Compared with normal strength concrete, tensile strength increases less than propor-

tionally with increasing compressive strength. The failure at crack initiation is brittle. To increase the tensile strength and the flexural tensile strength respectively, and to attain a ductile behaviour in cracking and post-cracking, high-strength fibres, usually steel fibres, are added (UHPFRC). In this way the tensile strength of concrete is systematically usable for structural design calculations in ultimate limit state.

However, the load-carrying capacity of exclusively fibre-reinforced UHPFRC beams is limited by the (flexural) tensile strength. An utilisation of the compressive strength of UHPC can be achieved by adding bar reinforcement. As already examined at normal- and high-strength concrete, a combination of fibres and bar reinforcement affects the structural and deformation behaviour, both in the working load range and in ultimate limit state. By interaction of fibre reinforcement and bar reinforcement the crack width can be diminished, the crack propagation can be prevented or at least reduced and a good bond between the reinforcement and the surrounding concrete can be achieved [2, 3]. Moreover, a more uniformly distributed crack formation is possible [4]. In a test programme, the structural and deformation behaviour of UHPC and UHPFRC under tensile stress shall be investigated, using panel-shaped structural members and tensile members. In the following, the preparation of this series and the results of some preliminary tests will be presented.

Apart from the investigations under tension, the effects of cracking on the compressive strength of UHPC and UHPFRC are to be examined at panel-shaped test specimens. This biaxial loading, which occurs very frequently on panels or shells, leads to a decrease of the compressive strength in comparison with the uniaxial strength measured at cylinders. For more than three decades, experimental investigations were conducted on this topic at panel-shaped structural members of normal-strength concrete. However, the results differ a lot.

During last year, a test series on panels of normal strength concrete was executed at Kassel University. For the first time the effects of fibre addition on the biaxial strength were investigated [5]. On the basis of the results and the experience attained by these tests, a similar test program with test specimens of UHPC and UHPFRC is carried out at present. Chapter 3 deals with this topic.

2 Tension tests on reinforced UHPFRC

2.1 Determination of tensile strength of UHPFRC

The tensile strength of fibre reinforced concrete can be determined by a centric tensile test, e.g. at notched prisms. Due to the difficult load introduction, the determination of the tensile strength from a bending tensile test is preferred. However, the tensile strength obtained by this procedure depends on the height of the test specimen as well as on the placing direction and on the fibre orientation respectively. According to the DBV data sheet "steel-fibre concrete" [6], the flexural tensile strength of normal- and high-strength concrete is to be attained by a four-point-bending test (Fig. 1). Afterwards, a stress-strain-relationship is derived, using the stress-displacement-diagram of these tests (Fig. 2).

According to Teutsch [7], the test specimen, shown in Figure 1, can also be used for UHPFRC. Figure 3 shows the results of such kind of test.

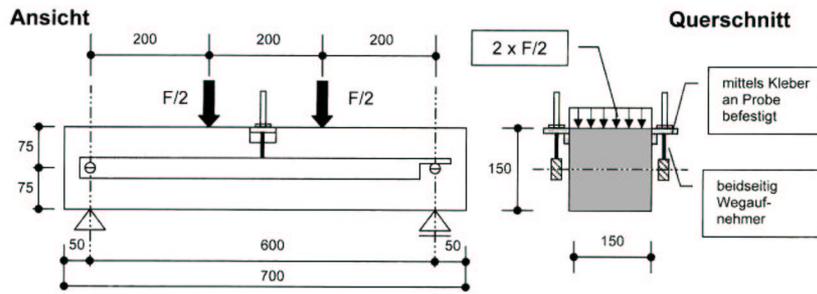


Figure 1: Four-point-bending test according to the DBV data sheet "steel fibre concrete" [6]

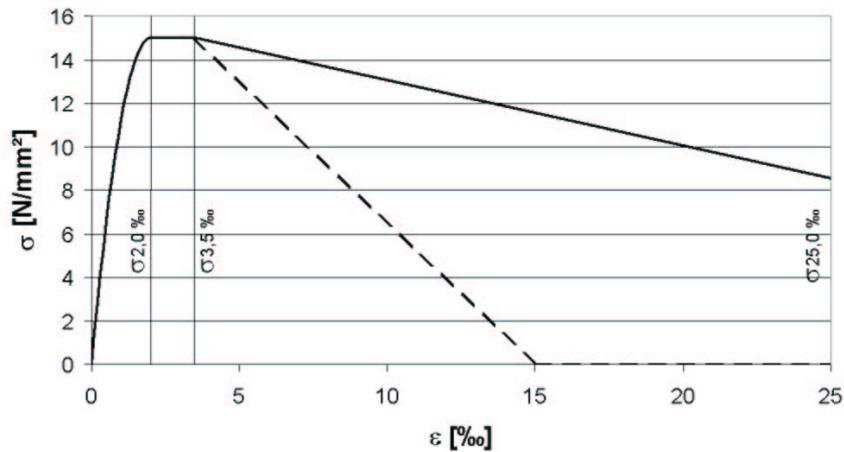


Figure 2: Stress-strain-relationship for steel fibre reinforced normal-/high-strength concrete and for UHPFRC [7]

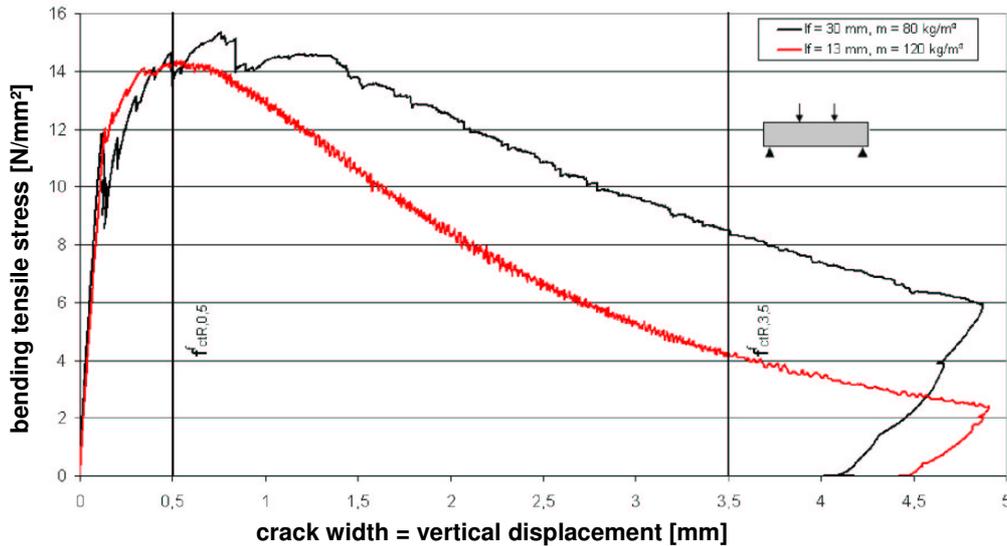


Figure 3: Results of bending tests on UHPFRC carried out by Teutsch [7]

According to Figure 3, with increasing crack width the sustainable load of short fibres decreases fast because of the pull out of the fibres. In contrast, long fibres are still able to carry high stresses even for large crack widths. Formulating a design model, this effect has to be consid-

ered, in order not to overestimate the effect of fibres. In the DBV data sheet [6] and the outline of the code for steel-fibre concrete [8] a limitation of strain is defined at $\varepsilon_u = 25 \%$. However, in the progress report on UHPC [7] of the German Committee for Reinforced Concrete (DAfStb) it is suggested to use an ultimate strain ε_u and an ultimate stress $\sigma_u = f_{3,5} \cdot \beta$ as a function of the ratio $f_{3,5}/f_{0,5}$ (Fig. 4). The coefficient β is determined by recalculating several test results.

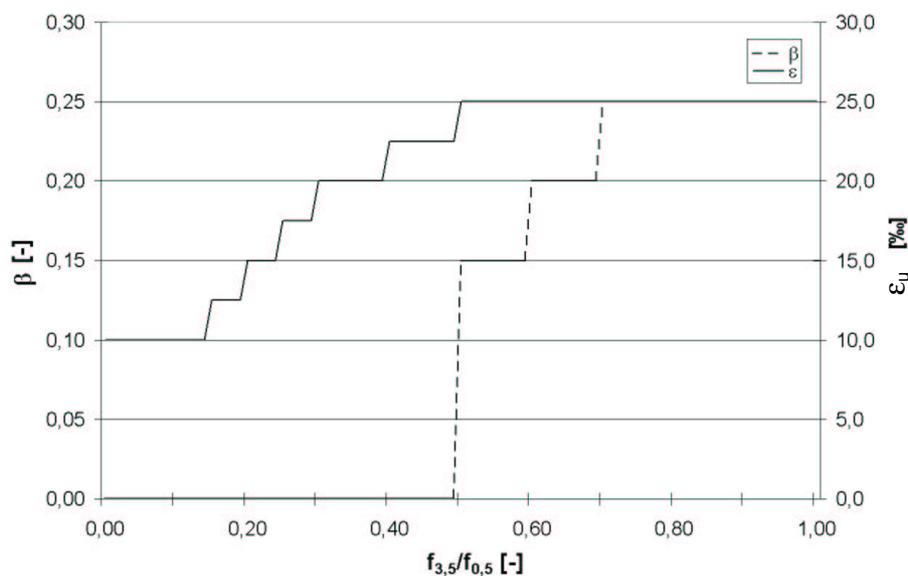


Figure 4: Ultimate strain ε_u and coefficient β to convert bending tensile strength in centric tensile strength according to [7]

2.2 Interaction of fibres and bar reinforcement

The systematic utilisation of the compressive strength of UHPC in structural members predominantly loaded in bending requires an additional reinforcement in the tension zone, using ordinary or prestressed reinforcement. However, for economic reasons, it is desirable to consider steel fibres (used for ductility reasons anyway) for the calculations in ultimate and serviceability limit state as well.

The consideration of the fibres for the ultimate limit state design requires knowledge concerning the fracture-mechanic relations. [6] and [8] contain models for the interaction of fibres and bar reinforcement for normal- and high-strength concrete.

On the one hand, for a systematic limitation of crack width under working load conditions, the consideration of discrete cracks is necessary. On the other hand design models based on smeared cracking should be developed to describe the stiffening influence of the fibres compared to non-fibred members, e. g. for a realistic calculation of deformations.

The intention of the testing programme on tensile members of UHPFRC, initiated by Kassel University, is to develop a constitutive model to describe the interaction of fibre reinforcement and bar reinforcement under tensile stress. Hence, models based on discrete cracking as well as on smeared cracks (calculation of crack width, ultimate limit state design) for reinforced UHPFRC are supposed to be developed.

2.3 Description of the tensile tests

Test specimens

The interaction of fibres and conventional bar reinforcement will be tested at approx. 30 panel-shaped structural members (Fig. 5), fabricated with and without fibres. Additionally, small tensile members with different cross sections are tested (Fig. 6).

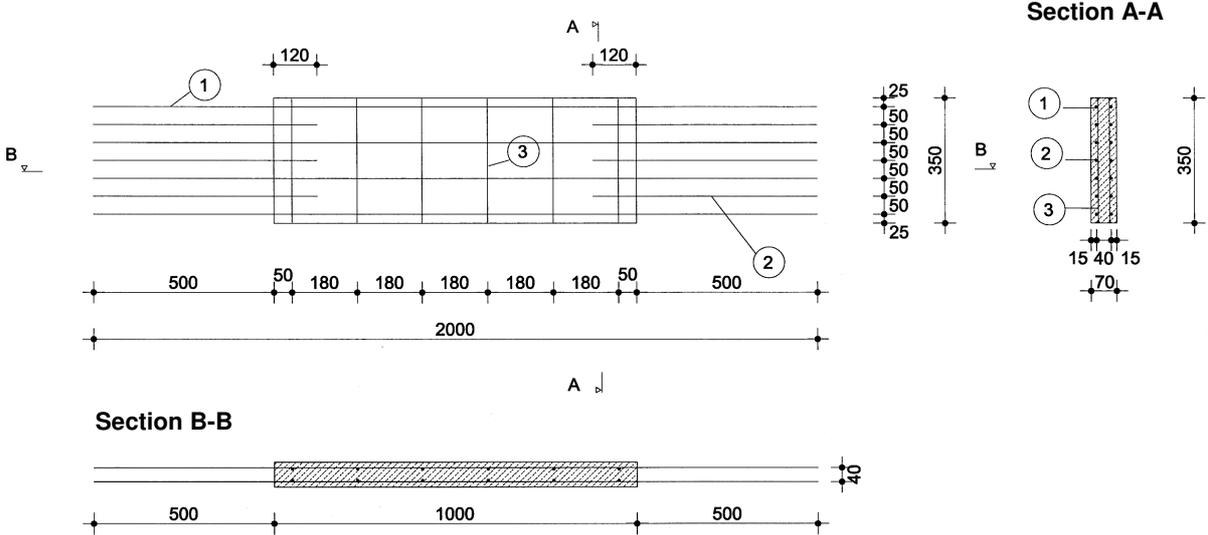


Figure 5: Panel-shaped structural members for the investigations on tensile stress (type L)

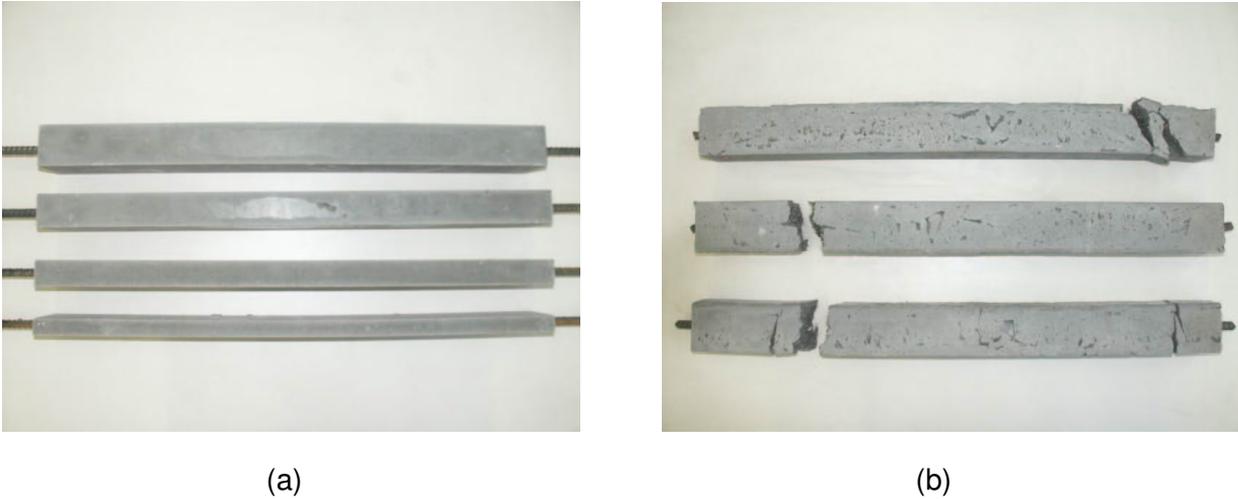


Figure 6: Tensile members with different cross sections for the investigations on tensile stress (a) and results of preliminary tests (b)

The panels are 1000 mm long (load direction), 350 mm high and 70 mm thick. They are reinforced in two directions with two layers of high-strength rebars in each direction, SIGMA prestressing steel St 1420/1570, diameter \varnothing 8 mm. In order to avoid an early failure at the junction points to the load device, reinforcing bars with rolled-up threads are used.

The tensile members are reinforced centrally with one SIGMA prestressing steel St 1420/1570. Both, the cross sections of the tensile member and the diameter of the reinforcement bar (\varnothing 8, 10 and 12 mm) will be varied.

For reinforcing steel BSt 500 commonly used today, the reinforcement ratio becomes unsatisfying high. Otherwise the steel stresses at crack initiation reach very high values near to the yield strength. For this reason high-strength reinforcement (prestressing steel type) was selected for the test series.

The panel-shaped test specimens are poured vertically in a rigid steel formwork. The top side of the panels is flattened with a smoothing trowel after concreting. The mix composition is selected in a way that allows a fabrication of the panels without or with just a small mechanical compaction (quasi-selfcompacting concrete). Through this, the danger of an unintentional alignment of the steel fibres, e. g. by using an internal vibrating machine, and a segregation of the fluid mix (settlement of the fibres) is avoided. After taking out of the formwork at an age of 2 days, the test specimens are heat treated at approx. 90°C for 48 hours. Afterwards they are stored under normal climate (20°C). In this way, the specimens reach their final strength already after 4 days.

Mix compositions

Table 1 shows the mix compositions of the UHPC and the UHPFRC. The percentage of steel fibres of the UHPFRC is 0.95 vol.-%. The high-strength steel fibres used are sized 17 mm long with a diameter of 0,15 mm. The consistency of the concrete mixtures is capable of flowing.

Table 1: Mix composition

| Mix Composition | | UHPC | UHPFRC |
|------------------------------|-------------------|-------------|---------------|
| cement CEM I 52,5R HS-NA | kg/m ³ | 733 | 733 |
| quartz sand 0,125/0,5 | kg/m ³ | 1008 | 1008 |
| silica fume | kg/m ³ | 230 | 230 |
| steel fibres (17 mm/0,15 mm) | kg/m ³ | - | 74,6 |
| quartz powder | kg/m ³ | 183 | 183 |
| superplasticiser | kg/m ³ | 28,6 | 28,6 |
| water | kg/m ³ | 161 | 161 |

Preliminary tests

In some preliminary tests, tensile members of UHPFRC, reinforced centrally with one reinforcement bar $\varnothing = 10$ mm, BSt 500, were loaded until failure of the steel occurred. The tests were carried out path controlled. Figure 7 shows the force-displacement-relationship of two tensile members with different cross section and the characteristic curve of the plain steel.

Because of the larger (concrete) cross section, the influence of the steel fibres is greater for smaller reinforcement ratio. Starting with the first crack, the contribution of concrete (tension stiffening and effects of steel fibres) is nearly constant until reaching the yield strength of steel. Figure 7 shows as well, that the ultimate load capacity of the tensile members is approximately

attained at the yield point of the bar reinforcement. This can be explained by the fact, that the tensile stress transferred by the fibres is still remaining on a high level for small and middle crack widths (see Fig. 3). The sustainable load in a crack results from the sum of reinforcement and steel fibre loads.

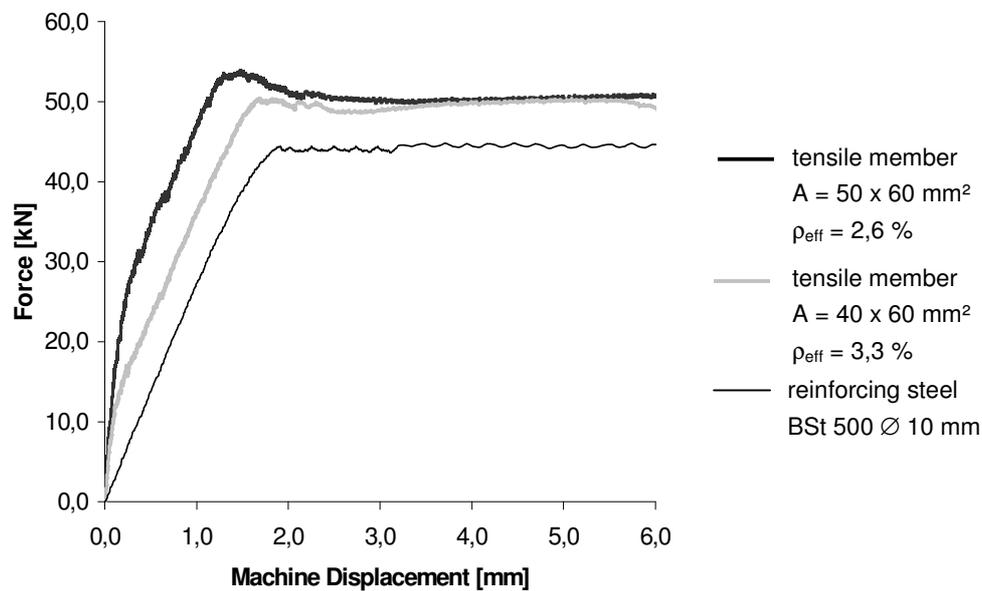


Figure 7: Results of two preliminary tests on tensile members of UHPFRC

Coincidentally or affected by concreting/compacting the distribution and the alignment of the fibres vary over the length of the tensile member. Therefore the load distribution between fibres and bar reinforcement is different in each crack. The bar reinforcement in the crack with the most unfavourable fibre distribution/alignment reaches the yield strength first. A further load increase in this cross section would have to be sustained exclusively by the fibres. However, with increasing crack width, the destabilisation of the fibre-reinforced concrete (fibre pull-out and/or yielding of the steel fibres) is starting. Hence, the load-carrying capacity of the tensile member decreases gradually to the pure load-carrying capacity of the hardening reinforcing steel. By the hardening of the steel the load-carrying capacity of one specimen was increasing a little bit. Thus it was possible to generate steel yielding in a second crack (Fig. 6).

A similar deformation behaviour could be observed at specimens, tested in the context of the biaxial compression-tension tests on panels of normal strength concrete. Over the whole length of the panel (1000 mm), the bar reinforcement was reaching the yield strength only in one crack, even at high strain. The other cracks retained their crack widths (Fig. 8). With the opening of the crack the load-carrying capacity decreased.

Recapitulating it is to be stated that the maximum load-carrying capacity of the tensile members was obtained for a relative small strain still within the elastic range of the reinforcing steel. The crack widths were still at a level of serviceability at this time ($w < 0,2$ mm). After unloading these cracks closed almost completely.

For the ultimate limit state design, the influence of the fibres has to be considered not only to achieve an adequate safety but also to attain a ductile behaviour with an announcement of failure by excessive cracking.

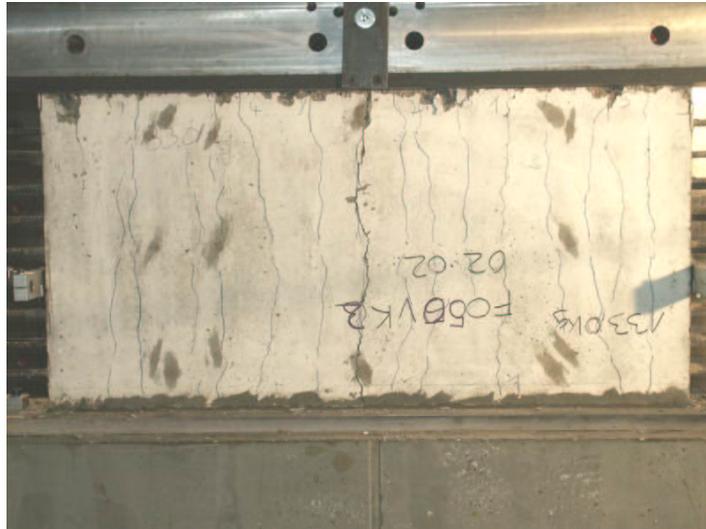


Figure 8: Opening of only one crack in the middle of the panel at a strain of 5.5 %.

3 Biaxial compression-tension tests on UHPC-panels

3.1 Influencing variables on the compressive strength of cracked concrete

In the literature, the crack width and the crack spacing (Schlaich/Schäfer [9]), the tensile stress (Kollegger/Mehlhorn [10]), the main tensile strain and the load path (Belarbi/Hsu [11]), the main tensile strain (Vecchio/Collins [12]) or the tensile stress and the crack initiation (Tanabe/Wu [13]) are mentioned as major influences on the decrease of the uniaxial compressive strength due to biaxial loading.

Due to the cracking of the concrete member, a partitioning into several compression struts occurs. The dimensions of the struts are defined by crack spacing and thickness of the panel. Affected by the crack initiation the compression struts are shaped irregularly.

There are many parameters influencing the crack growth and final crack spacings. A special dependence exists regarding the direction and the stress of the tensile reinforcement. The direction of the reinforcement affects considerably the crack direction. Cracks, which do not run parallel to the main compression stress direction, represent a disturbance of the load path and reduce the load-carrying capacity of the compression struts. For the reinforcement, reaching the yield strength means a crucial border. Above this border, the tensile strain can serve as a suitable size to describe the cracking.

Affected by the crack opening under very large tensile strain, the damage of the panel is increasing strongly. Interaction between the compression struts by aggregate interlock at inclined cracks (Fig. 9a) is not possible anymore. Also the dowel effect of the reinforcement (Fig. 9b), which causes a mutual support of the slim compression struts, is almost lost beyond the yield point. Besides this, very high bond stresses near to the cracks appear, which contribute to a

further destruction of the grain structure. For this reason the stresses have to be redistributed from the edges to the centre of the compression struts.

Due to small crack spacings, for small compression struts the risk exists, that they are already destroyed by tensile stress or split up along the reinforcement from bar to bar by compression stress. If the panels are reinforced in two directions, as common for concrete structures, the reinforcing bars can buckle in compression direction, effected by an insufficient concrete cover and missing support respectively. This may cause a premature failure of the whole structure. A detailed theoretical investigation on the topic of biaxial loading is given by Roos [14].

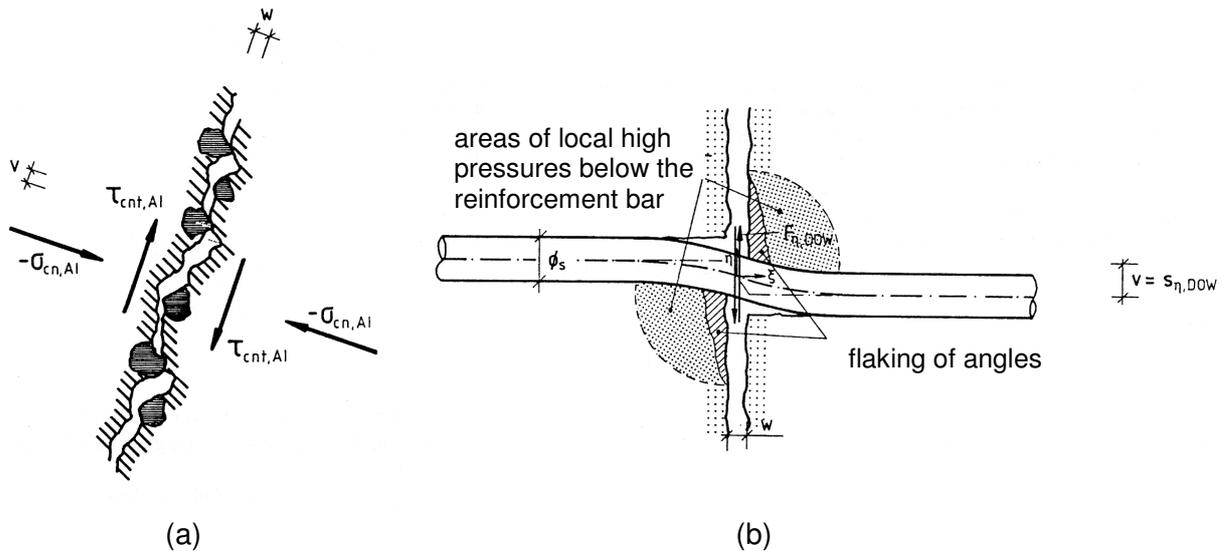


Figure 9: Aggregate interlock (a) and dowel effect of the reinforcement (b) [14]

3.2 Description of the biaxial compression-tension tests

Test programme

In the context of the test series concerning the structural behaviour of reinforced UHPFRC panels under tensile stress also the biaxial compressive strength is to be investigated. The panels of UHPC and UHPFRC are tested in the Laboratory of Civil Engineering at Kassel University (Fig. 10). Thereby the influences of tensile strain, crack orientation and steel fibres on the compressive strength of UHPC are analysed.

Test specimen

The dimensions and the reinforcement of the tested panels (type K) are represented in Figure 11. Due to the limitation of the vertical force of the load device on 5.5 MN, compared to Figure 5 (type L), the dimensions of the panel had to be reduced. The panels used are 500 mm long (tension), 350 mm high (compression) and 70 mm thick. In this way compressive stresses up to 157 N/mm² can be obtained. In case of a distinct decrease of the compressive strength of UHPC due to the applied transverse strain, the longer type of panels (type L) may also be used within this test series.

The test specimens are fabricated as described in chapter 2.3. For the determination of the modulus of elasticity, the tensile splitting strength and the compressive strength several cylin-

ders of the same concrete mix are fabricated. The test can be scheduled very flexibly, because the final strength is reached after only 48 hours of thermal treatment with approx. 90° C. To ensure an approximately uniformly distributed load over the length of the panel, on the top side and on the bottom side of the panel a very thin levelling layer of UHPC is applied one day before the test.



Figure 10: Test setup for the biaxial compression-tension tests on concrete-panels

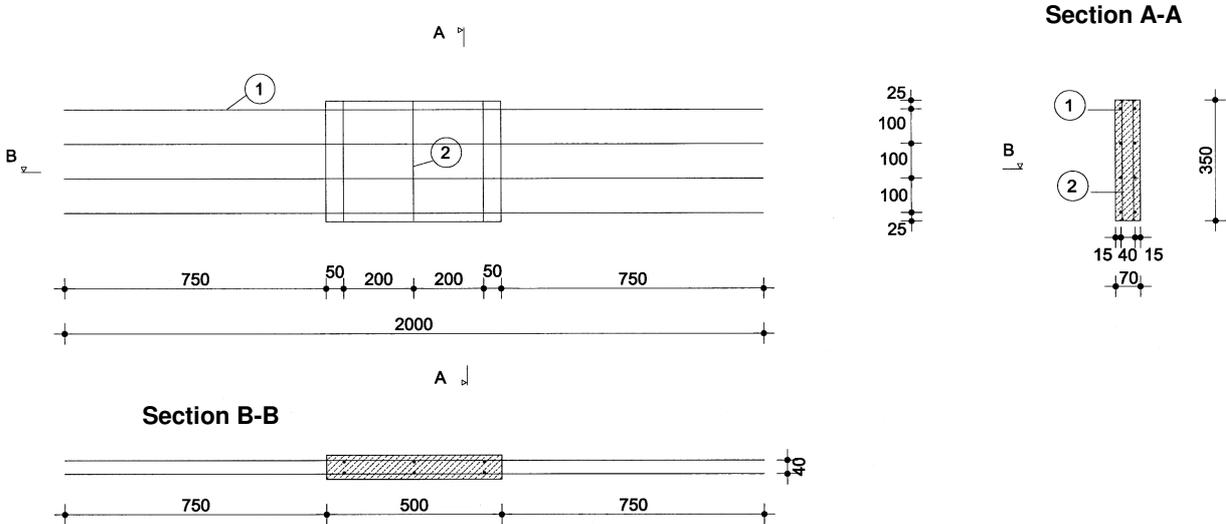


Figure 11: Panels for the investigations on biaxial compression-tension loading (type K)

Mix composition

For the investigations on the biaxial compression-tension strength of UHPC, the compressive strength of the test specimens has to be adapted to the maximum force of the load device. For this purpose, the compressive strengths of the UHPC mixtures, designed at Kassel University reach too high values. In order to reduce the compressive strength systematically without affect-

ing the grain structure substantially, parts of the quartz sand 0.125/0.5 contained in the mixture according to table 1 are replaced by adding glass foam adapted in grain size and volume. This leads to an increased air content, which reduces the compressive strength to an objective of 150 N/mm².

Performance of tests

First the appropriate strain in horizontal direction is applied. Subsequently, the test specimens are loaded in vertical direction at failure (sequential load applying).

In the context of the investigations on panels of normal-strength concrete a preliminary test on a panel of UHPC reinforced exclusively with bar reinforcement was carried out, too. The characteristic cylinder compressive strength of the mixture used was approx. 160 N/mm². The panel was extended in horizontal direction at a strain of 25 ‰ before applying the compressive load. In contrast to the fibre reinforced panels, as described in chapter 2.3 (Fig. 8), the yield strength of the reinforcement was reached in almost every crack (Fig. 12). Even after reaching the yield point successive crack propagation was recognized. Finally, the crack spacing was in the range of 30 to 50 mm.

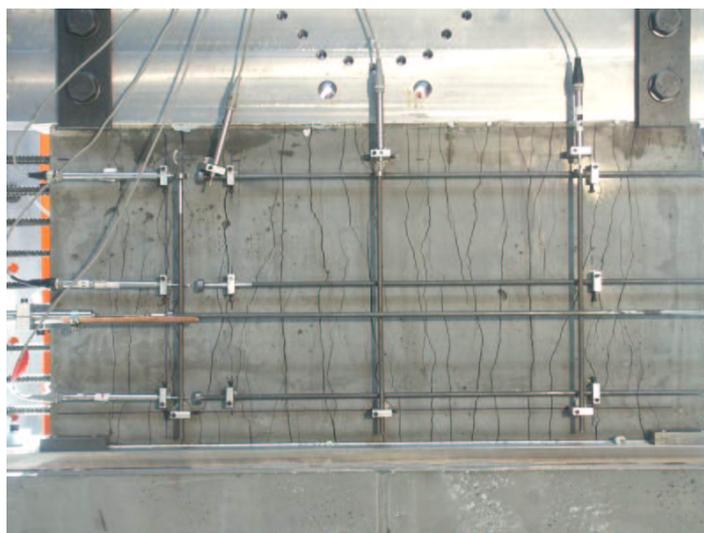


Figure 12: Test specimen of UHPC (without fibres) after applying the horizontal strain (25 ‰)

As expected, the very narrow and irregularly shaped concrete compression struts had only a minor remaining load-carrying capacity. The reduction of the uniaxial compressive strength was significantly more pronounced than the maximum reduction attained in the tests on normal-strength reinforced concrete panels (without fibres). The average crack spacing measured at specimens of normal-strength concrete was approximately 100 mm. This result points out the great influence of the crack pattern on the reduction of the compressive strength. For the fibre-reinforced panels of UHPFRC the reduction is expected to be less due to the different crack development (cp. chapter 2.3).

4 Conclusion

In the context of this paper the investigations on the structural behaviour of UHPC under tensile stress und under biaxial compression-tension loading, that have been initiated at the Chair of structural Concrete of Kassel University, were presented. The first preliminary tests show the necessity of these investigations in the sense of an economic and safe design of structural UHPC- and UHPFRC-members. The results refer to both, common features and differences existing in these topics between normal- or high-strength concrete on the one hand and Ultra High Performance Concrete on the other hand.

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