Structural behavior of ultra-high performance (fiber-reinforced) concrete compression struts subjected to transverse tension and cracking

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Abstract
Struts and stress fields with transverse tension and cracks parallel to the direction of compression lead to a reduction of compressive strength in reinforced concrete structures. Biaxial tests on fine-grained ultra-high performance concrete (UHPC) and ultra-high performance fiber-reinforced concrete (UHPFRC) panels additionally reinforced with steel bars show that this reduction in compressive strength is already pronounced even with small transverse tensile strain and crack widths, but stabilizes for larger tensile strain. A reduction could also be observed with regard to compressive stiffness of the struts, which behave quasi-linear under compressive loading. Basically, the UHPFRC panels show less reduction in strength and stiffness than the UHPC panels. Compared with the results obtained from the tests on normal-strength reinforced concrete panels and fiber-reinforced concrete panels, the reduction of compressive strength is only little larger for UHP(FR)C. Based on the test results, a proposal for modeling the stress–strain behavior of UHP(FR)C in compression–tension stress state is provided.

KEYWORDS
compression, compressive strength, crack, model, stiffness, strut, tension, UHPC, UHPFRC, ultra-high performance concrete

1 | INTRODUCTION

Biaxial stress states with compressive stress and simultaneous transverse tension lead to a decrease in both compressive and tensile strength of concrete.\(^1,2\) This applies to concretes of all strength classes, but is more pronounced with increasing concrete compressive strength.\(^3,4\) For reinforced concrete, reaching the concrete tensile strength is of no significance, if reinforcement is provided which carries the tensile forces after crack formation. Then, the failure mode under compression–tension loading is always in concrete compression with or without previous yielding of the tension reinforcement perpendicular to compressive direction.

Numerous tests on the reduction of the compressive strength of cracked reinforced concrete have been carried...
out over more than 50 years,\textsuperscript{5–13} with very different results and conclusions. A survey and critical review of the relevant work can be found in Roos\textsuperscript{14} and Fehling et al.\textsuperscript{15}

Crack formation in biaxially loaded reinforced concrete structures basically causes separate compression struts whose dimensions are defined by the thickness of the structural member and the crack spacing. The crack spacing and the crack width, whereby the latter in turn is essentially dependent on the reinforcement configuration and the strain state, are given fundamental importance with regard to the reduction of compressive strength.\textsuperscript{7,11,12,15,16}

In ultra-high performance concrete (UHPC)\textsuperscript{17} under tension, cracks occur at very small distances due to the high bond stresses with the bonded reinforcement.\textsuperscript{18,19} With ultra-high performance fiber-reinforced concrete (UHPFRC), the crack spacings are further reduced and tension stiffening increases due to the contribution of the steel fibers.\textsuperscript{20–24} Thus, in case of simultaneous compressive loading perpendicular to tension, compressive stress meets very slender struts. The interaction between individual struts by aggregate interlocking is hardly possible in particular for fine-grained UHPC due to the small maximum grain size. Even with small crack widths, a significant reduction compared to uniaxial compressive strength may be expected. On the other hand, tests on normal-strength reinforced concrete panels showed that the reduction is significantly lower when adding steel fibers to the concrete mix.\textsuperscript{15}

Following the tests on normal-strength reinforced concrete panels mentioned before, Fehling, Leutbecher, Röder, and Stürwald\textsuperscript{25} carried out biaxial tests on UHP(FR)C panels at the University of Kassel. The aim of these tests was to investigate the influence of transverse tension and cracking as well as the significance of fiber addition on the reduction of compressive strength, which is reported in Fehling et al.\textsuperscript{26} These aspects have subsequently also been addressed by Lee et al.\textsuperscript{27}

However, establishing a constitutive model, such as a stress–strain relation (which is necessary for performing physically nonlinear finite element analysis) requires information on the influence of transverse tension and cracking on both strength and stiffness of UHP(FR)C compression struts. This was not examined before and is thus in the focus of the present study. Based on an accurate evaluation\textsuperscript{28} of the biaxial tests on UHP(FR)C panels performed at the University of Kassel, a proposal for modeling the compressive stress–strain behavior of UHP(FR)C structural members in compression–tension stress state is provided. At the beginning, an overview of the tests is given.

## Overview of the Experimental Investigations

### 2.1 Test program

The test program consisted in a total of 46 panel tests. Some of the panels were subjected exclusively to uniaxial tension or did not provide reliable results due to errors in test execution or data logging. Thirty-one panel tests were relevant with regard to strength and stiffness of UHP(FR)C compression struts. Only these tests will be reported below.

The 31 test specimens are subdivided into four series. Three panels were made of plain UHPC without fibers (series C). Four panels were exclusively reinforced with steel fibers (series FRC), 12 panels were exclusively reinforced with steel bars (series RC), and 12 panels were reinforced with a combination of steel bars and steel fibers (series MRC). All panels of series C and FRC as well as three panels of series RC and four panels of series MRC were loaded exclusively in uniaxial compression. The remaining specimens were subjected to biaxial tension–compression loading. Table 1 gives an overview of the test program.

### 2.2 Test specimens

The panels were made of fine-grained UHPC with a maximum aggregate size of 0.5 mm. In series FRC and MRC, smooth straight steel fibers with a length $l_f = 17$ mm, a diameter $\phi_f = 0.15$ mm, and a tensile strength of approximately 2,500 MPa were added to the mixture. The fiber volume fraction was 1.0% in these series. Table 2 provides the mix design in detail.

The dimensions of the specimens were 500 mm in horizontal (tensile) direction, 350 mm in vertical (compressive) direction, and 70 mm in thickness (Figure 1).

In series RC and MRC ribbed high-strength steel with a diameter $\phi_s = 8$ mm was used as bar reinforcement. The steel grade was St 1420/1570 with a nominal yield stress $R_{p0.2} = 1,420$ MPa, a nominal tensile strength $R_{m} = 1,570$ MPa, and a modulus of elasticity

| TABLE 1 Test program |
|-----------------------|
| **Series** | **Uniaxial loading** | **Biaxial tension–compression loading (applied tensile strain)** |
| C  | 3  | – |
| FRC  | 4  | – |
| RC  | 3  | 9 (0.64–5.55‰) |
| MRC  | 4  | 8 (0.65–4.34‰) |
$E_s = 205,000 \text{ MPa}$. The specific rib area of the reinforcing bars was $i_R = 0.026$. Figure 2 shows the stress–strain curve of the steel. The reinforcing bars were arranged orthogonally in two layers. The bar spacing was 100 mm in tensile direction (represented by index $1$) and 200 mm in compressive direction (represented by index $2$). This resulted in reinforcement ratios of $\rho_1 = 1.64\%$ and $\rho_2 = 0.86\%$, respectively. The lateral distance between the bar axis and the specimen surface was 15 mm for the horizontal tension reinforcement and 23 mm for the vertical compression reinforcement. The horizontal reinforcing bars of the biaxially loaded panels were 2 m long and were provided with cold rolled threads at their ends for applying the tensile loading by the testing device (see Chapter 2.3.2). The vertical reinforcing bars were 350 mm long, corresponding to the height of the panel, and their ends were machined flat.

TABLE 2  Mix design

| Units                  |          |
|------------------------|----------|
| Cement CEM I 52.5 R HS-NA kg/m$^3$ | 744      |
| Silica fume Elkem microsilica® grade 940 uncompacted kg/m$^3$ | 234      |
| Quartz powder MILLISIL® W12 kg/m$^3$ | 186      |
| Quartz sand 0.125/0.5 mm H33 kg/m$^3$ | 1,023    |
| Superplasticizer FM 1254 kg/m$^3$ | 29.0–31.0|
| Water kg/m$^3$ | 175–193  |
| Steel fibers STRATEC Weidacon kg/m$^3$ | 78       |
| $l_f/\phi_f = 17 \text{ mm}/0.15 \text{ mm}$ (for series FRC and MRC only) |           |
| Fiber volume fraction $\rho_f$ (for series FRC and MRC only) % | 1.0      |
| Water–cement ratio $w/c$ | 0.26–0.28|

*Taking into account the water content of the superplasticizer.

FIGURE 1  Geometry of test specimens and arrangement of reinforcement (dimensions in mm)

The specimens were concreted upright. Compaction was performed for 120 s on a vibrating table at 50 Hz. After compaction, the upper side of the panels was flattened with a trowel and covered with a plastic foil. The formwork was stripped after 2 days. Immediately thereafter, the specimens were heat-treated for 48 hr at about 90°C. The specimens were then stored in room climate until the test was carried out.

For determining the compressive strength and the modulus of elasticity of the concrete, three to six cylinders with $h/d = 300 \text{ mm}/150 \text{ mm}$ were fabricated concomitantly to each panel. Before testing, the concrete cylinders were ground plane-parallel at the end faces. In order to ensure the flatness of the load introduction surfaces of the panels, an approximately 5-mm thick leveling layer of fine-grained UHPC was applied in the test rig.

The cylinder compressive strength was tested according to EN 12390–3:2009\textsuperscript{29} and the modulus of elasticity was tested according to EN 12390–13:2013.\textsuperscript{30} The UHPFRC mix showed on average a cylinder compressive strength $f_{cm,cyl} = 169 \text{ MPa}$ and a modulus of elasticity $E_{cm,cyl} = 44.3 \times 10^3 \text{ MPa}$. The corresponding values for the UHPC mix (without fibers) were $f_{cm,cyl} = 130 \text{ MPa}$ and $E_{cm,cyl} = 40.6 \times 10^3 \text{ MPa}$. This remarkable difference in mechanical properties for similar mix compositions may be attributed to the fact that the fracture surfaces of the UHPC specimens showed agglomerations of silica fume and quartz powder, which indicates an insufficient mixing intensity of the compulsory mixer. Due to the higher energy application in case of fibers in the mix, these agglomerations were not observed with the UHPFRC specimens.
2.3 | Test setup and test execution

2.3.1 | Panels uniaxially loaded in compression

The panels, which were subjected exclusively to compression, were tested in a servohydraulic four-column testing machine with 6.3 MN maximum load (Figure 3). The machine was controlled via the piston stroke. The specimens were arranged concentrically in the testing machine and were continuously loaded at a constant rate of 1 μm/s until the compression failure occurred.

To measure the deformations, three linear variable displacement transducers (LVDTs) each in vertical and horizontal directions were attached to the front and back of the panel (Figure 4). The gauge length in vertical direction was 270 mm (measuring lengths $V_0 – V_2$) and in horizontal direction 420 mm (measuring lengths $H_0 – H_2$).

2.3.2 | Panels biaxially loaded in compression and tension

The biaxial tests were carried out in a load frame (Figure 5). The test setup essentially corresponded to the setup used in a previous series of tests on normal-strength reinforced concrete panels and steel fiber reinforced concrete panels.15

The horizontal tensile loading was applied by two coupled hydraulic jacks (No. 1 in Figure 6a), each with a nominal capacity of 400 kN. The tensile load was then transmitted via a crosshead (No. 2 in Figure 6a) and intermediate threaded rods and adapters (Figure 6b) to the reinforcement of the test specimen.

The vertical compressive loading was applied by two individually controllable hydraulic jacks (No. 3 in Figure 5), each with a nominal capacity of 2.5 MN. The compressive load was then transmitted via two roller bearings (No. 3 in Figure 6c), which enabled rotation in
the panel plane, to two load distributing steel panels (No. 4 in Figure 6c), which were positioned one above the other. The large distance between the line of action of the two hydraulic jacks for compressive loading and the short length of the test specimen causes significant bending deformation of the load distributing steel panels. In order to obtain a most uniform stress distribution at the top of the test specimen, a 40-cm long centering plate was arranged concentrically in a horizontal gap between the two steel panels (No. 5 in Figure 6c; the centering plate is partially hidden by a mounting tab). The vertical compressive loading was transmitted only through the centering plate. In addition, a self-constructed pot bearing (No. 6 in Figure 6c), which was filled with a layer of polytetrafluoroethylene (PTFE), was inserted between the lower steel plate and the top of the test specimen, in order to avoid increased edge pressure. A block of UHPC (No. 7 in Figure 5) with a flat steel plate on it served as support for the test specimen.

The test specimens were loaded displacement controlled using a separate measuring and control unit. The vertical compressive forces were measured by the load cells of the two 2.5 MN hydraulic jacks. The horizontal tensile forces were measured by the load cells of the two 400 kN hydraulic jacks and for control purposes on the opposite side (abutment side) by two further load cells. In addition, the tensile forces of the individual reinforcing bars were measured both on the side of load application and on the abutment side by means of load cells mounted between the reinforcing bars and the adjacent threaded rods (Figure 6b).

As in the uniaxial tests, six LVDTs were attached directly to the specimen’s surface each at the front and back side of the panel (arrangement see Figure 4). Four additional LVDTs were applied in order to control the tests. Two of them were arranged on the left and right side of the specimen for measuring the vertical displacements (No. 8 in Figure 6c). The other two LVDTs were mounted on reinforcing bars outside the specimen (No. 9 in Figure 6c) for measuring the horizontal displacements at the front and back side of the specimen.

The test specimens were arranged in the test rig in such a way that the compression load could be applied concentrically after the target tensile strain had been applied. In all biaxial tests, the tensile strain was applied first, followed by monotonous compressive loading. As in the majority of previous studies, sequential load application was preferred compared to proportional load application, because applying the tensile strain with simultaneous compression loading is handicapped by friction, which occurs between specimen and loading device. The sequence should, however, be of minor importance, since in either case the direction of principle strains is constant during the test and agrees with the direction of principle stresses.

The tensile load was applied at a rate of 2 μm/s until the initial crack and thereafter at a rate of 4–8 μm/s up to the target tensile strain. This value ranged between 0.64 and 5.55‰ and was thus always below the elastic limit of the high strength reinforcement. During the tensile loading, the visible cracks were marked.

After finishing the tensile loading process, the load distributing steel panels were placed on the pot bearing at the top of the test specimen. Initially, a compressive load of approximately 40 kN per hydraulic jack was applied by
manually moving the cylinder pistons. The compressive load was then increased continuously at a constant rate of 1 μm/s until compressive failure of the test specimen.

3 TEST RESULTS AND EVALUATION

3.1 Overview

Table 3 presents the data of the 31 panel tests (see Data S1 for further information on each test). To designate the specimens the abbreviations introduced in Chapter 2.1 are used, followed by two digits separated by a hyphen. The first digit indicates the type of load (1 = uniaxial; 2 = biaxial). The second digit is a consecutive number counting the panels of the same type.

The strain $\varepsilon_{1,max}$ was calculated from the average of the horizontal displacements that were measured along the gauge lengths $H0$–$H2$ (see Figure 4) at the front and back of the panel at the end of the tensile loading process. The corresponding nominal tensile stress $\sigma_{1,max}$ results from the applied tensile load divided by the area of the gross concrete cross section in tension $A_{c1} = 350 \text{ mm} \cdot 70 \text{ mm}$.

The compressive stress $\sigma_{2,min}$ was determined by dividing the ultimate compressive load by the area of the gross concrete cross section in compression $A_{c2} = 500 \text{ mm}$

| Panel no. | $\varepsilon_{1,max}$ (‰) | $\sigma_{1,max}$ (MPa) | $\sigma_{2,min}$ (MPa) | $\varepsilon_{2,min}$ (‰) | $\sigma_{2,min}$ (MPa) | $\frac{f_{c, panel}}{f_{c, cyl}}$ (–) | $\sigma_{c, panel} \cdot 10^{3}$ (MPa) | $E_{cyl} \cdot 10^{3}$ (MPa) | $\frac{E_{c, panel}}{E_{c, cyl}}$ (–) |
|-----------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| C1-1      | –               | –               | –114.7          | –2.82           | –114.7          | 129.6           | 0.89            | 42.2            | 40.4            | 1.04            |
| C1-2      | –               | –               | –119.9          | –3.41           | –119.6          | 112.6           | 1.06            | 41.6            | 40.7            | 1.02            |
| C1-3      | –               | –               | –130.8          | –3.50           | –130.8          | 134.6           | 0.97            | 44.7            | 39.9            | 1.12            |
| FRC1-1    | –               | –               | –170.5          | –3.89           | –170.5          | 172.0           | 0.99            | 47.6            | 45.9            | 1.04            |
| FRC1-2    | –               | –               | –159.2          | –3.66           | –159.2          | 160.9           | 0.99            | 47.7            | 43.5            | 1.10            |
| FRC1-3    | –               | –               | –154.9          | –3.59           | –154.9          | 178.1           | 0.87            | 46.4            | 46.8            | 0.99            |
| FRC1-4    | –               | –               | –177.5          | –3.92           | –177.5          | 178.1           | 1.00            | 49.9            | 46.8            | 1.07            |
| RC1-1     | –               | –               | –131.1          | –3.04           | –125.7          | 135.8           | 0.93            | 43.2            | 42.2            | 1.02            |
| RC1-2     | –               | –               | –111.9          | –2.66           | –107.1          | 127.2           | 0.84            | 41.3            | 40.8            | 1.01            |
| RC1-3     | –               | –               | –133.1          | –2.99           | –127.8          | 151.4           | 0.84            | 44.7            | 41.2            | 1.08            |
| RC2-1     | 0.64            | 2.55            | –100.6          | –3.18           | –95.1           | 122.9           | 0.77            | 34.5            | 39.5            | 0.87            |
| RC2-2     | 1.56            | 4.60            | –102.8          | –3.24           | –97.0           | 122.9           | 0.79            | 31.6            | 39.5            | 0.80            |
| RC2-3     | 2.71            | 7.85            | –69.8           | –2.50           | –65.4           | 128.6           | 0.51            | 30.5            | 42.2            | 0.72            |
| RC2-4     | 3.68            | 11.20           | –72.8           | –2.89           | –68.0           | 128.6           | 0.53            | 29.3            | 42.2            | 0.69            |
| RC2-5     | 4.57            | 14.46           | –72.8           | –3.64           | –66.3           | 122.2           | 0.54            | 24.9            | 41.6            | 0.60            |
| RC2-6     | 5.55            | 17.15           | –62.5           | –2.70           | –57.7           | 122.2           | 0.47            | 24.0            | 41.6            | 0.58            |
| RC2-7     | 1.07            | 3.10            | –100.7          | –3.83           | –94.1           | 140.0           | 0.67            | 32.2            | 38.3            | 0.84            |
| RC2-8     | 1.91            | 5.27            | –86.7           | –3.32           | –80.8           | 140.0           | 0.58            | 30.5            | 38.3            | 0.80            |
| RC2-9     | 0.65            | 2.04            | –98.3           | –3.14           | –92.7           | 127.2           | 0.73            | 36.3            | 40.8            | 0.89            |
| MRC1-1    | –               | –               | –152.4          | –3.39           | –146.4          | 160.9           | 0.91            | 45.6            | 43.5            | 1.05            |
| MRC1-2    | –               | –               | –181.1          | –3.98           | –174.0          | 184.5           | 0.94            | 46.6            | 46.6            | 1.00            |
| MRC1-3    | –               | –               | –181.3          | –3.94           | –174.3          | 184.5           | 0.95            | 47.7            | 46.6            | 1.02            |
| MRC1-4    | –               | –               | –152.6          | –3.93           | –146.4          | 162.8           | 0.90            | 43.0            | 40.0            | 1.08            |
| MRC2-1    | 0.65            | 4.18            | –126.0          | –4.42           | –118.3          | 162.8           | 0.73            | 39.3            | 40.0            | 0.98            |
| MRC2-2    | 0.70            | 4.93            | –147.4          | –4.17           | –140.3          | 167.7           | 0.84            | 43.7            | 45.7            | 0.96            |
| MRC2-3    | 0.77            | 5.76            | –131.7          | –4.22           | –124.5          | 168.1           | 0.74            | 40.3            | 42.6            | 0.95            |
| MRC2-4    | 1.64            | 8.32            | –136.8          | –4.09           | –129.8          | 167.7           | 0.77            | 40.2            | 45.7            | 0.88            |
| MRC2-5    | 2.54            | 11.40           | –132.1          | –3.85           | –125.4          | 163.4           | 0.77            | 36.4            | 44.6            | 0.82            |
| MRC2-6    | 3.25            | 14.17           | –127.9          | –4.06           | –120.7          | 163.4           | 0.74            | 35.8            | 44.6            | 0.80            |
| MRC2-7    | 2.00            | 9.68            | –131.3          | –3.94           | –124.9          | 164.6           | 0.76            | 39.1            | 43.7            | 0.89            |
| MRC2-8    | 4.34            | 19.25           | –122.9          | –4.88           | –114.2          | 168.1           | 0.68            | 31.2            | 42.6            | 0.73            |
· 70 mm. For the panels of series C and FRC, $\sigma_{2,min}$ already represents the actual concrete compressive stress $\sigma_{c2,min}$. For the panels of series RC and MRC, however, $\sigma_{2,min}$ has to be corrected by subtracting the strain-dependent contribution of the reinforcement oriented in compressive direction. This was done according to Equation (1) assuming the strain in the bonded reinforcement is the same as that in the surrounding concrete.

$$\sigma_{c2,min} = \sigma_{2,min} - \rho_2 E_s \varepsilon_{2,min}. \quad (1)$$

In Equation (1) $\sigma_{2,min}$ represents the stress related to the area of the gross concrete cross section $A_{c2} = 500$ mm · 70 mm determined from the sum of forces of the two vertical hydraulic jacks at ultimate compression load, $\rho_2$ is the reinforcement ratio in compressive direction, $E_s$ is the modulus of elasticity of the reinforcing steel, and $\varepsilon_{2,min}$ is the compressive strain of the panel at ultimate compressive load.

In order to protect the LVDTs directly attached to the specimen’s surface (Figure 4) from damage, these LVDTs were removed before the expected failure load was reached. Therefore, the compressive strain $\varepsilon_{2,min}$ which is necessary for evaluating Equation (1) was derived analytically from the piston stroke of the four-column testing machine (uniaxial tests) or from the data of the two vertical displacement transducers used to control the biaxial tests. This was done—following the principle of a series connection of two springs—by subtracting the deformation of the four-column testing machine or that of the load frame from the total displacement. The stiffness of the “spring” representing the four-column testing machine or the load frame was calibrated at small and medium load level by comparing the total displacement and the displacement of the test specimen which was obtained from gauge lengths V0–V2. This stiffness was assumed to be representative on a higher load level as well.

The modulus of elasticity $E_{c,panel}$ corresponds to the maximum increase of the stress ($\sigma_{c2}$)-strain ($\varepsilon_2$) curve of the panels in the lower to medium stress range, where $\sigma_{c2}$ was determined by subtracting the strain-dependent contribution of the reinforcement, following Equation (1).

The ratios $|\sigma_{c2,min}|/f_{cm,cyl}$ and $E_{c,panel}/E_{cm,cyl}$ refer to the mean values of the mechanical properties of the cylinders which were fabricated concomitantly to each panel.

### 3.2 Uniaxial tests

The panels of series C showed a brittle, sometimes an explosive type of failure. Only in case of Panel C1-2 (Figure 7a) the failure pattern gave an idea of the original geometry of the specimen.

The failure of the panels of series RC was also brittle. The part of the concrete cross section, which was not covered by the two reinforcement layers, was completely spalled (Figure 7b), combined with buckling of the vertical compression reinforcement. Due to the high damage of the structure, the core cross section covered by the reinforcement layers did not show any residual load-bearing capacity.

The failure of the UHPFRC panels was less brittle. The failure modes of the panels of series FRC (Figure 7c) and MRC (Figure 7d) hardly differed. In most cases a shear failure combined with local concrete spalling was observed. However, even with these two types of panels, the load suddenly dropped to zero when reaching the ultimate load, despite the fact that the tests were performed displacement controlled.

The stress–strain behavior of the uniaxially loaded panels was almost linear until reaching the ultimate load, irrespective of the reinforcement configuration. The curves in the lower stress range (thick line) in Figure 8 are determined from the mean value of the displacements measured directly at the specimen’s surface (gauge lengths V0–V2 in Figure 4). The curves in the upper stress range (thin line) are—except for Panel C1-1—calculated from the piston stroke of the four-column testing machine, as described in Chapter 3.1. The displacements of Panel C1-1 were measured by the LVDTs directly attached to the panel until compression failure. During compression loading, transverse strains were measured, which relate to Poisson’s effect.

In Figures 9 and 10, the columns represent the mean ratios of the concrete compressive strength $|\sigma_{c2,min}|/f_{cm,cyl}$ and the modulus of elasticity $E_{c,panel}/E_{cm,cyl}$ of the four types of panel. The error indicator (black line) marks the
scatter range of the individual test results (minimum and maximum value).

The concrete compressive stress $\sigma_{c2,min}$ and the modulus of elasticity $E_{c,\text{panel}}$ of series C and FRC could be determined directly from the measured loads and displacements. For the panels with bar reinforcement, the ratios were calculated, as described in Chapter 3.1, by subtracting the strain-dependent contribution of the reinforcement from the measured load, so that $E_{c,\text{panel}}$ virtually represents the deformation behavior of the net UHP(FR)C cross section for series RC and MRC. The ratios $E_{c,\text{panel}}/E_{c,m,cyl}$ obtained in this way vary between 1.04 and 1.06 and thus are of similar magnitude for all four types of panel.

Since the gross density of the concrete of the panels tended to be slightly below the gross density of the accompanying cylinders, the systematically higher modulus of elasticity of the panels is most likely not attributed to a higher concrete quality, but to the method of determining $E_{c,\text{panel}}$ described in Chapter 3.1, which deviates from the method according to EN 12390–13:2013.30

In contrast to the ratios $E_{c,\text{panel}}/E_{c,m,cyl}$, the ratios $|\sigma_{c2,min}|/f_{c,m,cyl}$ differ for the four types of panel. While the failure of panels in series C and FRC occurred at a stress level approximately equal to the cylinder compressive strength, the panels of series RC failed on a stress level significantly below the corresponding cylinder compressive strength. The panels of series MRC are classified between the series mentioned before.

Due to their sensitivity to tolerances (flatness of the load introduction surface), the panels of series C show the largest scattering of the individual test results. Due to the larger cross-sectional area, it is much more difficult to apply a uniform load to the panels than to the cylinders. This and the greater slenderness of the panels ($h/d = 5$) compared to the cylinders ($h/d = 2$) could be the reason for ratios slightly below 1.0 for the panels of series C (0.97) and FRC (0.96). In general, the influence of the
specimen’s slenderness on the compressive strength is, however, only small with UHPC.31,32

The significantly lower ratio in series RC ($\sigma_{c,\text{min}}/f_{\text{cm,cyl}} = 0.87$) indicates that the load-bearing capacity of these panels cannot be determined by superposing the virtual load-bearing capacities of concrete and reinforcement. In particular, the reinforcing bars perpendicular to compressive direction cause local stress concentrations and splitting tensile stresses, which provoke early failure of the concrete cross section. In addition, the reinforcing bars hinder the free shrinkage of UHP(FR)C, which causes restraint with tension in the concrete even in the unloaded state. Thus, extraordinary transverse tension, resulting in compression–tension stress state even under uniaxial loading, might also have affected the load-bearing capacity of the panels of series RC. By adding fibers, the impact of these effects is partially eliminated for the panels of series MRC ($\sigma_{c,\text{min}}/f_{\text{cm,cyl}} = 0.92$). This may be attributed to the crack-bridging effect of the fibers and the increased deformation capacity of UHPFRC.

The phenomena described before regarding the four different types of panel could be observed in a similar manner and magnitude in a previous study on normal-strength reinforced concrete panels and steel fiber reinforced concrete panels.15

3.3 | Biaxial tests

The biaxial tests were carried out on panels of series RC and MRC. Depending on the applied tensile strain, the panels showed a different number of cracks, where the cracks sometimes propagated with significant inclination over the panel height (Figure 11). As expected, in the state of stabilized cracking (Figure 11b,d) the crack spacings were smaller and the number of cracks was larger with the panels of series MRC than with the panels of series RC. Especially at small tensile strains, the cracks of the UHPFRC panels were hardly visible due to their small width.

The stress–strain behavior in compression of the biaxially loaded panels was almost linear over a wide range (Figure 12), but on the other hand showed some characteristics which differ from the uniaxially loaded panels.

The nonlinearity of the curve, which could often be observed in the lower stress range, combined with a gradual increase in stiffness, is due to the fact that—depending on the degree of damage caused by transverse tension—not all struts of a panel that were separated by cracks could be activated to the same extent at the beginning of compressive loading. This start-up effect is particularly pronounced for Panel MRC2-8 (Figure 12d).

In addition, the stress–strain curves of some panels show a certain plastic deformation capacity at or immediately before reaching the ultimate load (particularly pronounced for Panel RC2-3, Figure 12b). This behavior can be explained by the fact that the cracked panels initially failed only partially, for example, when reaching the bearing capacity of individual struts. Obviously, this failure could be compensated by transferring the load to previously less intensively loaded struts. In some cases the load could even slightly be increased. In other cases, the panels failed brittle when reaching the ultimate load without noticeable plastic deformation (e.g., Panel RC2-1, Figure 12a).

Unlike the uniaxially loaded panels, the biaxially loaded panels showed no Poisson’s effect under compression loading. The applied transverse tensile strains stayed virtually constant until failure of the panel, which can be attributed to the existing tensile cracks.

Since the stress–strain curves are affected by the start-up effect and the somehow arbitrary behavior at ultimate load level, the compressive strains $\varepsilon_{2,\text{min}}$ given in Table 3 leave some scope for interpretation.

The failure patterns and failure modes of the biaxially loaded panels did not differ from those of the uniaxially loaded panels reinforced in the same manner.

Figure 13 shows the ratios of the concrete compressive strength $|\sigma_{c,\text{min}}|/f_{\text{cm,cyl}}$ obtained in the biaxial tests as function of the applied tensile strain $\varepsilon_{1,\text{max}}$. The ratios of the panels of series RC are between 0.79 and 0.47. Starting from $|\sigma_{c,\text{min}}|/f_{\text{cm,cyl}} = 0.87$ with the uniaxially loaded panels of the same type and excluding Panel RC2-2, a nearly linear decrease of relative concrete compressive strength can be observed for tensile strains $\varepsilon_{1,\text{max}}$ less than approximately 2.0–2.5‰.
For larger tensile strains $\varepsilon_{1,\text{max}}$, the ratio $|\sigma_{c2,\text{min}}|/f_{cm,\text{cyl}}$ stabilizes at a level of approximately 0.5. This observation may be attributed to the gradual decrease of aggregate interlock mechanism, which is virtually no longer active at larger tensile strain (crack opening). Then, further crack opening has only minor effect.

The ratios $|\sigma_{c2,\text{min}}|/f_{cm,\text{cyl}}$ of the panels of series MRC are between 0.84 and 0.68. For small tensile strains, there is a similar decrease of the relative concrete compressive strength as for the panels of series RC. In contrast to these, the results of the panels of series MRC do, however, not show a significant further decrease for larger tensile strains $\varepsilon_{1,\text{max}}$. In fact, the ratio $|\sigma_{c2,\text{min}}|/f_{cm,\text{cyl}}$ stabilizes at a level of about 0.7.

Consequently, the panels of series MRC show the more favorable behavior in comparison to the panels of series RC, despite the larger number of cracks and smaller crack spacings. Obviously, the fibers bridge the cracks and thus stabilize the compression struts, which are considerably more slender than those of the exclusively bar reinforced panels of series RC. In addition, the smaller crack widths of the UHPFRC panels facilitate the
interaction of neighboring struts by shear friction even at inclined cracks.\textsuperscript{15}

Figure 14 shows the influence of tensile strain and cracking on the stiffness of the panel under compressive loading, represented by the ratio $E_{c,\text{panel}}/E_{cm,\text{cyl}}$.

Due to the start-up effect mentioned above, $E_{c,\text{panel}}$ was determined from the stress–strain curve at medium load level. After evaluating the $\sigma_2$–$\varepsilon_2$ diagram, the compressive stress for which the stress–strain curve turns into a linear slope (no further increase in stiffness) was determined specifically for each panel. Thus, $E_{c,\text{panel}}$ corresponds to a range of load level where the cross-sectional area of the panel struts is activated to the most extent.

The ratio $E_{c,\text{panel}}/E_{cm,\text{cyl}}$ of the panels of series RC is between 0.89 and 0.57. The relation between the relative modulus of elasticity and the applied tensile strain is virtually linear. If the trend is extrapolated linearly, the point of intersection with the ordinate is, however, significantly below 1.04, which is the ratio obtained for the uniaxially tested panels of series RC.

The ratio $E_{c,\text{panel}}/E_{cm,\text{cyl}}$ of the panels reinforced with a combination of steel bars and steel fibers is between 0.98 and 0.73. For this type of panel also an approximately linear decrease of the relative modulus of elasticity can be observed with increasing applied tensile strain. The course is almost parallel to that of the panels exclusively reinforced with steel bars. The loss of stiffness due to transverse tensile stress and cracking is therefore lower with the addition of fibers than for a comparably stressed specimen exclusively reinforced with steel bars.

Irrespective of the type of panel, the ratio $E_{c,\text{panel}}/E_{cm,\text{cyl}}$ is greater than the ratio $|\sigma_{c,\text{min}}|/f_{cm,\text{cyl}}$ in all biaxial tests. However, it should be kept in mind that for the uniaxially tested panels the ratio $E_{c,\text{panel}}/E_{cm,\text{cyl}} = 1.04$ was already significantly higher than the corresponding ratio of concrete compressive strength which was affected by local stress concentrations due to transverse reinforcement.

\textbf{4 \ | \ SUMMARY AND CONCLUSIONS}

The findings on the effects of transverse tension and cracking on the stress–strain behavior of UHP(FR)C compression struts may be summarized as follows:

1. As with normal-strength and high-strength concrete, the compressive strength of reinforced UHP(FR)C is reduced by transverse tension and cracking. The reduction is significantly pronounced already for small transverse strains. In the tests the decrease in strength was up to 53\% for fine-grained UHPC and up to 32\% for fine-grained UHPFRC, each in comparison to the (uniaxial) cylinder compressive strength. The reductions in compressive strength are only insignificantly larger than the values determined in Ref. 15 on normal-strength reinforced concrete panels and steel fiber reinforced concrete panels (there: up to 47\% without fibers; up to 28\% with fibers).

2. Besides the reduction in compressive strength, transverse tension and cracking also lead to a significant
reduction in compressive stiffness. This can be attributed to a reduced effective cross-sectional area of the struts, which are shaped irregularly due to unsteady and inclined path of crack.

3 Item 2 suggests the conclusion that also the observed reduction in compressive strength is essentially caused by the geometry of the struts, but that local weak points have more impact in this case than with the reduction in compressive stiffness. Hence, the reduction in compressive strength observed in the tests is essentially influenced by the arbitrariness of the crack pattern and the length of the struts. In addition, the compressive strength is reduced also by stress concentrations due to transverse reinforcement, so that the reduction in compressive strength is somewhat larger than the reduction in compressive stiffness.

4 By adding fibers, the crack spacings become smaller and thus the compression struts more slender with the same tensile strain. Nevertheless, the addition of fibers has a favorable effect on both compressive strength and compressive stiffness since fibers reduce the effects of local stress concentrations and facilitate the interaction of neighboring struts.

5 For modeling the compressive stress–strain behavior of biaxially loaded UHP(FR)C structural members, a linear course until reaching the peak stress, as commonly used in uniaxial loading case, seems to be appropriate. A plastic branch should not be considered since it was not found to be reliable. Indicative values for the strain-dependent reduction of compressive strength and compressive stiffness are provided by the dotted lines in Figures 13 and 14, respectively. Even though the decrease in stiffness varies to some extent from the decrease in strength, the inclination of the stress–strain curve may—as a simplification—be reduced in the same manner as the compressive strength. Then, the compressive strain at peak stress stays unchanged compared to uniaxial stress–strain curve.

6 The presence of reinforcement perpendicular to compressive direction causes local stress concentrations and splitting tensile stresses which reduce the compressive strength by about 10–15% even under uniaxial compression loading. This effect should carefully be considered for design of UHP(FR)C structural members.

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NOTATIONS

| Symbol | Description |
|--------|-------------|
| C      | plain concrete |
| FRC    | fiber-reinforced concrete |
| MRC    | mixed reinforced concrete |
| PTFE   | polytetrafluoroethylene |
| RC     | reinforced concrete |
| d      | thickness or diameter of the specimen |
| $f_{cm,cyl}$ | mean cylinder compressive strength of UHP(FR)C |
| h      | height of the specimen |
| $i_R$  | specific rib area of the reinforcing bars |
| $l_f$  | fiber length |
| w/c    | water–cement ratio |
| $A_{c1}$ | area of the panel’s gross concrete cross section in tensile direction |
| $A_{c2}$ | area of the panel’s gross concrete cross section in compressive direction |
| $E_{c,panel}$ | modulus of elasticity of UHP(FR)C panel |
| $E_{cm,cyl}$ | mean modulus of elasticity of UHP(FR)C cylinder |
| LVDT   | linear variable displacement transducer |
| $R_m$  | nominal tensile strength |
| $R_{p0.2}$ | nominal yield strength |
| UHPC   | ultra-high performance concrete |
| UHPFRC | ultra-high performance fiber-reinforced concrete |

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