Experimental investigations on the shear bearing behavior of prestressed ultra-high performance fiber-reinforced concrete beams with compact cross-section

Kevin Metje | Torsten Leutbecher

Abstract

For ultra-high performance fiber-reinforced concrete (UHPFRC) beams subjected to shear the bearing mechanisms differ significantly depending on the type of cross-section. For I-shaped cross-section, existing models superimposing the contribution of concrete, prestressing, stirrups, and fibers were found to predict the shear bearing capacity appropriately. These models may, however, not be generally applicable. The aim of the present study is to investigate the shear bearing mechanisms for UHPFRC beams with compact cross-section and, in particular, the influences of fiber volume fraction and prestressing on the development of the critical shear crack and the shear bearing capacity. This paper presents the experimental results of 22 three-point test on 11 beams with varying fiber volume fraction and prestressing. It was found that increasing the fiber volume fraction increases the shear bearing capacity, but however, the benefit was less than proportional to the residual flexural tensile strength of UHPFRC. As expected, prestressing also affects the bearing capacity favorably.

KEYWORDS

beam, bearing capacity, compact cross-section, creep, digital image correlation, flexural test, prestressing, shear, shrinkage, UHPFRC

1 | INTRODUCTION

Ultra-high performance fiber-reinforced concrete (UHPFRC) has been subject to research for several decades. Extensive test series on the shear bearing behavior of UHPFRC beams have been carried out by many researchers, varying cross-sectional dimensions, fiber volume fraction, and prestressing. However, the majority of studies focused on beams with I-shaped cross-section. These beams typically fail in shear tension of the thin web. As the shear bearing mechanisms significantly differ for beams with compact cross-section, where a diagonal tension failure occurs, the findings on beams with I-shaped cross-section may, however, not be generally applicable. Studies focusing on UHPFRC beams with compact cross-section unfortunately provide only few tests on fiber-reinforced beams without conventional shear reinforcement and no tests on prestressed beams at all. This may be attributed to difficulties provoking shear...
failure in UHPFRC beams with wide web. Activating a large number of fibers increases the shear bearing capacity and prestressing prevents flexural and diagonal shear cracking.

Existing models for predicting the shear bearing capacity of UHPFRC beams superimpose the load bearing mechanisms of reinforced and prestressed concrete (aggregate interlock, dowel action, resistance of the compression zone), of conventional shear reinforcement, and of fibers. These bearing mechanisms may however have to be weighted differently depending on the type of cross-section. For example, for I-shaped cross-section, the massive flanges allow for a large compression zone and an uncracked tension cord, which contributes to the shear bearing capacity differently compared to compact cross-section with constant width and flexural cracks in the tensile zone. Shear stresses in the thin web of UHPFRC beams with I-shaped cross-section usually initiate multiple evenly distributed shear cracks which develop parallel and with virtually constant inclination over the total height of the web. These shear cracks widen uniformly before finally the crack opening concentrates into one critical shear crack leading to shear tension failure by fiber pull-out.\textsuperscript{4,7,9,10} In contrast, UHPFRC beams with compact cross-section fail in diagonal tension by formation of a single curved critical shear crack.\textsuperscript{13,15,17} Therein, the fiber contribution differs over the height of the cross-section depending on the crack width which increases toward the tension cord.

Due to lack of experimental data, the specific contribution of the different bearing mechanisms to the shear bearing capacity of (prestressed) UHPFRC beams with compact cross-section cannot be quantified yet. In the present study, a series of shear tests on UHPFRC beams with compact cross-section was performed and monitored by digital image correlation (DIC) in order to examine the influence of different fiber volume fraction and prestressing on the formation of the critical shear crack as well as on the shear bearing capacity. This paper gives a detailed presentation of the experimental campaign.

2 | EXPERIMENTAL INVESTIGATIONS

2.1 | Test program

The test program included 22 shear tests on 11 UHPFRC beams (hereinafter referred to as “shear specimens”) without conventional shear reinforcement (Table 1). Therein, the fiber volume fraction ($\rho_f = 0, 1, \text{or } 2\%$) and the total force applied to the strands ($P_0 = 0, 469, \text{or } 938 \text{ kN}$) were varied, resulting in nine different specimen configurations.

The nominal geometry of the shear specimen as well as the arrangement of the tendons, of the additional reinforcement, and of a system for measuring the longitudinal deformation due to prestress and time-dependent behavior of concrete (see section 3.1) are shown in Figure 1.

The total length of the shear specimen was 3 m to enable testing of two shear zones per specimen. The nominal effective depth with regard to the center of gravity of the prestressing strands was $d = 211 \text{ mm}$. After testing, the actual geometry and position of the strands was measured with 1 mm accuracy at multiple locations. Table 1 presents for each specimen the actual mean values of the width $b_w$, the height $h$, the effective depth $d$, and the cross-sectional area $A_c$.

Accompanying fabricated cubes with $d = 100 \text{ mm}$ as well as cylinders with $h/d = 200 \text{ mm}/100 \text{ mm}$ were tested according to EN 12390-3\textsuperscript{20} in order to determine the

| Specimen | $\rho_f$ (%) | $P_0$ (kN) | Batch | $b_w$ (mm) | $h$ (mm) | $d$ (mm) | $A_c$ (cm$^2$) |
|----------|--------------|------------|-------|------------|---------|---------|---------------|
| V101-F0-P0 | 0 | 0 | F0-1 | 112 | 267 | 209 | 350 |
| V111-F0-P0 | 0 | 0 | F0-3 | 114 | 266 | 207 | 352 |
| V102-F0-P50 | 0 | 469 | F0-2 | 113 | 269 | 212 | 354 |
| V103-F0-P100 | 0 | 938 | F0-2 | 113 | 269 | 212 | 353 |
| V104-F1-P0 | 1.0 | 0 | F1-1 | 112 | 267 | 210 | 350 |
| V114-F1-P0 | 1.0 | 0 | F1-3 | 114 | 267 | 209 | 354 |
| V105-F1-P50 | 1.0 | 469 | F1-2 | 113 | 270 | 212 | 355 |
| V106-F1-P100 | 1.0 | 938 | F1-2 | 112 | 265 | 209 | 348 |
| V107-F2-P0 | 2.0 | 0 | F2-1 | 112 | 272 | 215 | 357 |
| V108-F2-P50 | 2.0 | 469 | F2-2 | 113 | 268 | 211 | 353 |
| V109-F2-P100 | 2.0 | 938 | F2-2 | 114 | 267 | 210 | 355 |
concrete compressive strength after 28 days and at the day of the shear test. Cubes or cylinders with \( d = 150 \) mm were not applied due to limitation of the testing device.

The residual flexural tensile strength of UHPFRC was determined in three-point tests according to EN 14651\textsuperscript{21} on accompanying fabricated flexural beams with \( b/h/l = 150 \) mm/150 mm/550 mm.

With regard to the action of prestress, the creep behavior of UHPFRC was examined in a separate creep test which was performed on an accompanying fabricated cube with \( d = 100 \) mm.

### 2.2 Fabrication of test specimens and material characterization

The specimens were fabricated from a total of eight batches with a volume of between 0.170 and 0.320 m\(^3\) each. The batch numbers are presented in Table 1. A single batch included one or two shear specimens as well as all related cubes, cylinders and flexural beams.

The specimens V101 and V104, cast from batches F0-1 and F1-1, did not obtain the targeted 28-day compressive strength due to a mix error. Thus, these specimen configurations were fabricated a second time (from batches F0-3 and F1-3) and designated V111 and V114.

All specimens were reinforced by seven-wire strands of grade St 1660/1860 steel with a nominal elastic modulus of 193 GPa.\textsuperscript{22} The strands showed a nominal diameter of 12.5 mm (0.5\textdegree) and a cross-sectional area of 0.93 cm\(^2\) each. In tensile tests, the 0.1\% proof-stress, the 0.2\% proof-stress, and the tensile strength of the steel were determined to 1601, 1667, and 1879 MPa, respectively.

The specimens V103, V106, and V109, a force of approximately 134 kN was applied to each strand resulting in a total force of \( P_0 = 938 \) kN (hereinafter referred to as “P100” and “100\% prestressing”). In case of specimens V102, V105, and V108, a force of approximately 67 kN was applied to each strand resulting in a total force of \( P_0 = 469 \) kN (hereinafter referred to as “P50” and “50\% prestressing”). In order to ensure proper anchorage of the strands even for the specimens without fibers, the anchorage zones at the ends of the specimens were rectangular-shaped and reinforced by stirrups. The cubes, cylinders and flexural beams were cast next to the shear specimens.

The mixture design and the most important material characteristics are presented in Table 2. The maximum aggregate size was 8 mm. The smooth and straight steel fibers had a length-to-diameter ratio of 20 mm/0.40 mm, a nominal tensile strength of 1250 MPa, and a nominal elastic modulus of 200 GPa. A compulsory mixer was used for mixing. The overall mixing process lasted 11 min (F0 batches) or 14 min (F1 and F2 batches).

The molds of the shear specimens and of the flexural beams were filled in two layers by means of a concrete bucket with screw conveyor which passed over the formwork in its longitudinal direction twice. Thereafter, the molds of the cubes and cylinders were filled by hand. All specimens were compacted by vibrating the stressing bed for 30 s. After casting and compacting, the molds were covered with foil and polystyrene insulation.

All specimens were demolded 3 days after casting. Then, the prestressing was applied to the shear specimens by cutting the wires of each strand separately. The shear specimens and four accompanying cubes were stored outdoors under tarpaulin until testing. One cube from batch F0-2 was used for determining the creep coefficient of UHPFRC by means of a creep test (see Section 3.1). The remaining cubes, cylinders and flexural beams were cured in water according to EN 12390-2\textsuperscript{23} until testing. Before compressive strength test, the load application faces of both cylinders and cubes were surface ground.

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**FIGURE 1** Nominal geometry of the shear specimens and arrangement of reinforcement, prestressing strands, and system for measuring longitudinal deformation

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[Diagram of shear specimens with reinforcement and prestressing strands]
The mean 28-day compressive strength of the different batches is between 115.9 MPa/126.2 MPa (cylinder/cube, batch F0-1) and 161.3 MPa/169.1 MPa (cylinder/cube, batch F1-3). When comparing cylinder and cube strength of the different batches it can be stated, that the difference between cylinder and cube strength decreases with increasing concrete compressive strength and increases with increasing fiber volume fraction. This is in line with the findings of former studies.25,26 The coefficients of variation (CV) range from 0.7 to 2.7%.

### 2.3 Setup and procedure of the shear tests

The shear specimens were tested in three-point tests. For each specimen two shear zones were tested. The tests were conducted in a servo-hydraulic testing machine with a maximum load of 400 kN. The loading device consisted of one loading roller and two supporting rollers. Figure 2 exemplary shows the setup for shear test 1 of specimen V111. For load distribution, sets of steel plates were arranged at the top of the supporting rollers and one steel plate below the loading roller. On the side opposing the examined shear zone, a sheet of greased polytetrafluoroethylene was placed between the steel plates to enable a low-friction horizontal displacement, as the supporting rollers were only capable of rotating freely around their axis but restrained to move horizontally. Since the specimens were filled from the top, a thin leveling layer of rapid hardening polyurethane resin was poured before placing the steel plate on top.

The shear span-to-depth ratio \( a/d \) was varied in the range of 3.5–5.0 for the individual tests, in order to eliminate effects of a potential direct load transfer between loading point and supports (strut-and-tie mechanism) as well as to enable flexural cracking for the prestressed specimens. To disable substantial shear cracking outside the examined shear zone, the opposite shear span was strengthened by external bracings consisting of rectangular steel hollow sections and threaded rods, which were tightened using a wrench.
The load was applied displacement-controlled with a constant rate of 0.5 mm/min. In order to examine the crack formation and propagation, the displacement of the shear zone was measured by means of three-dimensional DIC at one side of the specimen. In addition, the vertical displacement was measured by linear variable displacement transducers (LVDT) at the centers of support and in the axis of load application. The DIC measurement was executed with variable frequency (0.2 – 10 Hz) depending on the load level. The actual failure process was recorded in 100 ms increments.

The tests were aborted either after shear failure or, in some tests, before reaching the expected flexural bearing capacity in order to avoid flexural failure. In one case, no shear cracking was observed. Thus, the shear zone was tested a second time with modified shear span-to-depth ratio.

3 | TEST RESULTS

3.1 | Time-dependent deformation and loss of prestress

In order to derive the loss of prestress due to time-dependent deformation, a measuring system was installed at six selected shear specimens before placing the concrete. This system consisted of two 21 mm by 21 mm steel prisms with 2380 mm length which were arranged on the side faces of the shear specimen approximately at the height of the center of gravity (Figure 1). In each steel prism two holes were drilled in a distance of 2200 mm (= measuring length). There, polyvinyl chloride rings were pressed in with very tight fit (hole: +10 to −0 μm and shaft: +6 to −0 μm), in which measuring pins were installed with tight fit (hole: +10 to −0 μm and shaft: +0 to −25 μm) for exact placement and easy installation.

The prisms were mounted at the bottom side of wooden formwork panels which were installed inside the molds in the stressing bed in order to adjust the width of the web $b_w$. After casting, the bottom half of the measuring pins was embedded in the concrete. Formwork and prisms were removed at the beginning of the demolding process and before applying the prestress to the concrete, while the measuring pins remained in the specimens. The specimens' longitudinal deformation $\Delta l$ was measured using a long steel gauge and feeler gauges. The steel gauge was equipped with one circular hole (width with +10 μm fit) and one slotted hole (width with −0 to +10 μm fit) in order to place it over both measuring pins simultaneously. The longitudinal deformation $\Delta l$ was measured at the slotted hole in 50 μm = 0.05 mm increments using the feeler gauges. The measurement was repeated multiple times (Figure 3). Within the accuracy of the measuring system, no significant change of longitudinal deformation could be observed after 28 days.

The final longitudinal strain was found to be approximately −0.6 mm/m for specimen V107 ($\rho_t = 2\%$, without prestressing, see Figure 3). This value may be assumed to represent approximately the total shrinkage strain $\varepsilon_{cs}$ which agrees quite well with data from literature for coarse-grain UHPFRC.27,28 In comparison, shortening of specimen V104 ($\rho_t = 1\%$) and specimen V101 ($\rho_t = 0\%$) was found to be smaller (approximately −0.4 to −0.5 mm/m, see Figure 3). This can be attributed to macroscopic shrinkage cracks which formed within these specimens due to internal restrain induced by the
strands. Cracking results in an elongation of the member compared with the uncracked one. Consequently, the total strain measured with the cracked specimens V104 and V101 is smaller than the actual shrinkage strain of concrete. In contrast, specimen V107 with ρ_f = 2% stayed visually uncracked until testing.

For the prestressed specimens of series P50 and P100 the final longitudinal strain was found to be approximately −1.0 mm/m and −1.5 mm/m, respectively. Besides shrinkage strain, both values include elastic and creep deformation due to prestressing. In order to judge the effect of concrete creep, a creep test was performed before applying the prestress to the concrete (t_0 = 3 days) while the second half is assumed to develop afterward affine to creep deformation. Finally, the creep deformation of the shear specimens may expected to be about 15% smaller than that of the tested cube due to different storing conditions (cf. material law proposed by Anders and Müller). Based on these assumptions, the actual creep coefficient results in approximately

\[
\varphi(t, t_0) = \frac{\varepsilon_{c,tot} - \varepsilon_{c,el}}{\varepsilon_{c,el}} = \frac{-2.44 \text{ mm/m} + 0.81 \text{ mm/m}}{-0.81 \text{ mm/m}} = 2.0
\]

(1)

The loss of prestress can be calculated according to Equation (3) (EN 1992-1-1, 2004), which is based on the approximation by Trost.

In order to check the validity of the assumptions mentioned before, the time-dependent development of concrete strain is determined computationally (details see Supporting Information Part B) and compared with the measured data (Figure 3). Considering the tolerances of the measurement system, the calculation fits quite well with the actual behavior of the shear specimens.

Applying this value would, however, overestimate the actual creep deformation since the measured \(\varepsilon_{c,tot}\) also includes some portion of shrinkage strain which developed after starting the creep test. In addition, the shear specimens were stored outdoors at RH ≈ 80% until testing, while the creep test was performed at RH ≈ 50%.

Thus, the following assumptions and corrections are made for evaluating the time-dependent deformation process and determining the loss of prestress computationally. The total shrinkage strain \(\varepsilon_{cs}\) is assumed to be −0.6 mm/m (see above). Since autogenous shrinkage is dominant with UHPFRC a significant portion of shrinkage strain develops in the first days after casting. Thus, one half of total shrinkage strain is assumed to develop before applying the prestress to the concrete (\(t_0 = 3\) days) while the second half is assumed to develop afterward affine to creep deformation. Finally, the creep deformation of the shear specimens may expected to be about 15% smaller than that of the tested cube due to different storing conditions (cf. material law proposed by Anders and Müller). Based on these assumptions, the actual creep coefficient results in approximately

\[
\varphi(t, t_0) = \frac{\varepsilon_{c,tot} - \varepsilon_{c,el} - 0.5 \cdot \varepsilon_{c,el} \cdot 0.85}{\varepsilon_{c,el}} = \frac{-2.44 \text{ mm/m} + 0.81 \text{ mm/m} + 0.5 \cdot 0.6 \text{ mm/m}}{-0.81 \text{ mm/m}} \cdot 0.85 = 1.4
\]

(2)

In order to check the validity of the assumptions mentioned before, the time-dependent development of concrete strain is determined computationally (details see Supporting Information Part B) and compared with the measured data (Figure 3). Considering the tolerances of the measurement system, the calculation fits quite well with the actual behavior of the shear specimens.

The loss of prestress can be calculated according to Equation (3) (EN 1992-1-1, 2004), which is based on the approximation by Trost.

FIGURE 3 Longitudinal deformation and strain of specimens V101, V104, V107, V106, and V109 as function of time. Black and gray curves from measurement, red curves determined computationally, F0, F1, and F2: fiber volume fraction 0, 1, and 2%, P0, P50, and P100: level of prestressing 0, 50, and 100%
\[
\Delta \sigma_{p,c,s+\varepsilon} = \frac{E_c \cdot \sigma_p + 0.8 \cdot \Delta \sigma_{pr} + \frac{E_p}{E_{cm}(t_0)} \cdot \sigma_{c,QP} \cdot \phi(t, t_0)}{1 + \frac{E_p}{E_{cm}(t_0)} \cdot \frac{\Delta \epsilon}{\Delta \epsilon_{c,E}} \left( 1 + \frac{A}{A_n} \cdot z_{n,p}^s \right) \cdot [1 + 0.8 \cdot \phi(t, t_0)]}
\]

(3)

With the data mentioned before, the total loss of prestress results in approximately 26.0% for series P50 and approximately 20.5% for series P100. Resulting, the prestress force at the time of experiment (after all immediate and time-dependent losses) is \( P_t \approx 317 \text{ kN} \) for series P50 and \( P_t \approx 681 \text{ kN} \) for series P100 (details see Supporting Information Part B).

### 3.2 Residual flexural tensile strength of UHPFRC

In order to determine the residual flexural tensile strength of UHPFRC, three-point tests were conducted for batches F1-1, F1-3, and F2-1 on six notched beams each. The beams with \( l/b/h = 550 \text{ mm}/100 \text{ mm}/100 \text{ mm} \) were prepared and tested according to EN 14651. The midspan deflection was measured by two LVDTs, one on each side of the beam.

Figure 5 shows the load–deflection curves of the tests. The residual flexural tensile strength \( f_{Rm,j} \) at midspan deflection \( \delta_j \) is calculated by Equation (4) according to EN 14651 for midspan deflections \( \delta_1 = 0.47 \text{ mm}, \delta_2 = 1.32 \text{ mm}, \delta_3 = 2.17 \text{ mm}, \) and \( \delta_4 = 3.02 \text{ mm} \). The strength values and CV are presented in Figure 5.

\[
f_{Rm,j} = \frac{1}{6} \sum_{i=1}^{6} 3 \cdot F_{j,i} \cdot l_{i} \quad \frac{E_{cm}^{t_{0}}}{l_{b,i} \cdot h_{sp,i}^{2}}
\]

(4)

In Equation (4), the index \( i \) is the sequential number of beam \( (i = 1–6) \).
In addition to code related evaluation, the maximum residual flexural tensile strength \( f_{Rm,max} \) is calculated from the postcracking peak load \( F_{max} \) and is ranging from 9.3 MPa for batch F1-1 to 17.6 MPa for batch F2-1. The CV of \( f_{Rm,max} \) is ranging from 8.4 to 13.8%, which is within the typical scatter of three-point test on notched beams.30

### 3.3 | Results of shear tests

#### 3.3.1 | Overview

Table 3 provides information about the shear tests, including specimen age, mean cube compressive strength \( f_{cm,cube100,test} \) at the day of the shear test, shear span-to-depth ratio \( a/d \), span \( l \), shear cracking load \( V_{cr} \), ultimate shear load \( V_{u} \), and failure type. A detailed documentation of the shear tests is provided in Supporting Information Part A.

The large period of time between fabrication and testing allowed for further hardening of the concrete, resulting in an increase of compressive strength \( f_{cm,cube100,test} \) compared to the 28-day strength \( f_{cm,cube100,28d} \) depicted in Table 2.

\( V_{cr} \) was determined at the moment when the inclined shear crack developed toward the loading point and toward the tension cord, which in most tests was accompanied by a small drop of load or immediate shear failure.

| Specimen | Test | Age (days) | \( f_{cm,cube100,test} \) (MPa) | \( a/d \) (–) | \( l \) (m) | \( V_{cr} \) (kN) | \( V_{u} \) (kN) | Failure type | Notes |
|----------|------|------------|-------------------------------|-------------|----------|----------------|----------------|-------------|-------|
| V101-F0-P0 | 1    | 430        | 144.1                         | 3.5         | 1.90     | 57.2           | 57.2           | S           | -     |
| V101-F0-P0 | 2    | 434        | 144.1                         | 3.5         | 1.90     | 54.9           | 54.9           | S           | -     |
| V111-F0-P0 | 1    | 477        | 169.2                         | 3.5         | 1.90     | 62.5           | 62.5           | S           | -     |
| V111-F0-P0 | 2    | 492        | 169.2                         | 4.0         | 1.80     | 63.7           | 63.7           | S           | -     |
| V102-F0-P50 | 1    | 836        | 168.1                         | 4.0         | 1.80     | 118.4          | (159.1)**     | -           | DLT   |
| V102-F0-P50 | 2    | 840        | 168.1                         | 5.0         | 2.20     | 96.6           | (113.7)**     | -           | DLT   |
| V103-F0-P100 | 1    | 846        | 168.1                         | 5.0         | 2.20     | 146.2          | (151.5)**     | -           | DLT   |
| V103-F0-P100 | 2    | 847        | 168.1                         | 5.0         | 1.90     | 148.9          | (148.9)**     | -           | DLT   |
| V104-F1-P0 | 1    | 455        | 163.4                         | 3.5         | 1.90     | 115.3          | 115.3          | S           | -     |
| V104-F1-P0 | 2    | 456        | 163.4                         | 3.5         | 1.90     | 115.0          | 146.3b        | S + FC       | DLT   |
| V114-F1-P0 | 1    | 478        | 178.1                         | 3.5         | 1.90     | 103.8          | 103.8          | S           | -     |
| V114-F1-P0 | 2    | 490        | 178.1                         | 4.0         | 1.80     | 107.6          | 109.6          | S           | DLT   |
| V105-F1-P50 | 1    | 893        | 175.1                         | 5.0         | 2.00     | 145.1          | 145.1          | S           | -     |
| V105-F1-P50 | 2    | 894        | 175.1                         | 5.0         | 2.00     | 142.8          | 146.3b        | S + FC       | DLT   |
| V106-F1-P100 | 1a   | 895        | 175.1                         | 5.0         | 2.20     | 107.5          | (156.8)**     | -           | No shear crack |
| V106-F1-P100 | 1b   | 895        | 175.1                         | 5.0         | 2.20     | (197.6)**      | (197.6)** FC  | DLT         |
| V106-F1-P100 | 2    | 909        | 175.1                         | 4.0         | 1.90     | 204.3          | 217.6** S + FC | DLT         |
| V107-F2-P0 | 1    | 820        | 192.0                         | 3.5         | 1.90     | 128.9          | 183.1b        | S + FC       | DLT   |
| V107-F2-P0 | 2    | 825        | 192.0                         | 4.0         | 1.80     | 123.8          | 123.8          | S           | -     |
| V108-F2-P50 | 1    | 896        | 184.3                         | 4.5         | 2.00     | 157.1          | 159.6b        | S + FC       | DLT   |
| V108-F2-P50 | 2    | 899        | 184.3                         | 4.5         | 2.00     | 150.3          | 157.6b        | S + FC       | DLT   |
| V109-F2-P100 | 1    | 902        | 184.3                         | 4.0         | 1.90     | 204.3          | (209.2)** FC  | DLT         |
| V109-F2-P100 | 2    | 907        | 184.3                         | 4.0         | 1.90     | (204.2)** FC   | (204.2)** FC  | DLT         |

Note: \( V_{cr} \) in brackets occurred simultaneously with flexural failure due to crushing of the concrete compression zone. \( V_{u} \) in brackets is maximum acquired shear load without shear failure. S: shear failure (diagonal tension) shortly after initiation of the critical shear crack. FC: flexural failure due to crushing of the concrete compression zone. DLT: crack pattern favorable for direct load transfer between support and loading point (strut-and-tie mechanism).

*Abort at given shear load and unloading of the specimen. Shear crack was initiated, without failure.

*Load increase after initiation of the critical shear crack until combined shear and flexural failure due to crushing of the concrete compression zone occurred. The ultimate shear load is given.

*Abort at given shear load and unloading of the specimen. Only flexural cracks were initiated.

*Shear crack was initiated simultaneously with beginning flexural failure due to crushing of the concrete compression zone. The maximum acquired shear load is given.
Out of 22 tests presented in Table 3, the four tests on specimens V102 and V103 without fibers were aborted after initiation of the critical shear crack, but before predicted flexural failure occurred in order to prevent damage to the testing equipment in case of brittle failure of the concrete compression cord. The remaining 18 tests were conducted until failure. The failure types can be divided into two groups: flexural failure and shear failure. In three tests, a flexural failure occurred by crushing of the concrete compression zone (type FC). The 15 tests failing in shear (diagonal tension failure) can be further divided into two types: shear failure immediately after initiation of the critical shear crack (type S) and combined shear failure with flexural failure of the concrete compression zone (type S + FC, also known as shear compression failure). The different types of shear failure are discussed in the following.

Figure 6 shows the crack and failure patterns of the specimens without prestressing. The crack patterns, which were recorded by DIC, show different curvature and position along the longitudinal axis of the diagonal shear crack. For type S, the flexural crack, which develops into the critical shear crack, is farther from the loading point and the shear crack develops with variable inclination intersecting the direct connection between loading point and support. Tests showing this crack pattern (Figure 6(a–e,g,i)) failed shortly after formation of the critical shear crack by penetration of the compression zone. In three tests (Figure 6(f,h,i)) the shear crack developed closer to the loading point, showing a crack pattern favorable for direct load transfer between loading point and support (strut-and-tie mechanism). This enabled an increase of loading even after formation of the critical shear crack until the concrete compression zone finally failed under combined action of shear and flexure (type S + FC). However, one of these tests (Figure 6(h)) showed no crushing but just penetration of the compression zone by the critical shear crack (type S).

Crack and failure patterns of the five specimens with prestressing showing a shear failure are depicted in Figure 7. One test (Figure 7(a)) shows type S crack pattern and failure. The other four tests (Figure 7(b–e)) show type S + FC failure. For these four specimens the given ultimate shear loads exceed the theoretical flexural bearing capacity.

Figure 8 depicts the load–deflection curves of the 22 shear tests. The specimens without prestressing are presented in the top diagram and the specimens with prestressing are presented in the bottom diagram. The ordinate shows the shear load and the abscissa the deflection below the loading point measured by DIC. The triangles mark the shear cracking load $V_{cr}$. The type of failure is given next to each curve.

3.3.2 Results of the shear specimens without prestressing

The five specimens without prestressing were tested twice, one time for each shear zone (tests 1 and 2). All specimens failed in shear (type S or S + FC, see Figure 6). In order to investigate the influence of shear span-to-depth ratio $a/d$, seven out of ten tests were conducted with $a/d$ of 3.5 and three tests with $a/d$ of 4.0.

The specimens without fibers (V101 and V111) reached average ultimate shear loads $V_u$ between 56 and 63 kN and show type S failure. In test 2, V111 failed brittle after initiation of the critical shear crack, while the other three tests showed a less brittle softening behavior after reaching $V_u$.

Increasing the fiber volume fraction to 1% (V104 and V114) resulted in $V_u = 115$ kN for V104 (test 1 with type S failure) and $V_u = 107$ kN on average for V114. In test 2 of V104, a type S + FC failure occurred resulting in $V_u = 146$ kN. However, DIC evaluation showed that the critical shear crack already formed at about 115 kN. As depicted in Figure 8, test 1 of V104 failed brittle after shear cracking (marked with triangle), while test 2 shows a monotonous load increase until failure of the concrete compression zone. Both tests on V114 showed a less brittle softening behavior after reaching $V_u$.

For specimen V107 with $\rho_t = 2\%$, $V_u$ was 183 kN in test 1 with type S + FC failure and 124 kN in test 2 with type S failure. In test 1, the critical shear crack formed at about $V_{cr} = 129$ kN. The load–deflection curve of this test shows two small hystereses (marked with “a”) in Figure 8) which result from a temporary stop of the testing machine which became necessary for processing, saving and restarting the DIC recording. Apart from that, test 1 shows a monotonous load increase until brittle failure of the compression zone occurred. Failure of test 2 was slightly less brittle.

3.3.3 Results of the shear specimens with prestressing

The six specimens with prestressing were tested at least one time for each shear zone. Individual tests differ by shear span-to-depth ratio, span, and loading history in order to enable flexural and shear cracking after decompression of the tension cord.

The specimens V102 and V103 without fibers were tested with $a/d = 4.0$ and 5.0, respectively. In test 2, V103 underwent one load cycle at about $V = 90$ kN in order to promote initiation of cracking. However, in all four tests no flexural crack near the support could be provoked due to high prestressing. Instead, a flexural crack near the
FIGURE 6  Shear tests on specimens without prestressing. Crack patterns observed via digital image correlation (DIC) at ultimate shear load $V_u$ (left), and after shear failure (right), F0, F1, and F2: fiber volume fraction 0, 1, and 2%, $a/d$: Shear span-to-depth ratio
loading point developed into a shear crack, enabling direct load transfer to the support. $V_{cr}$ was about 118 kN (test 1) and 97 kN (test 2) with V102 and about 146 kN (test 1) and 149 kN (test 2) with V103. Test 2 of V103 underwent a second load cycle at the level of $V_{cr}$. Finally, the tests were aborted prior to predicted brittle flexural failure. Thus, no failure pattern is available for the specimens without fibers.

Specimen V105 with $\rho_f = 1\%$ was tested twice with $a/d = 5.0$. In test 1, two parallel shear cracks were initiated until shortly thereafter the shear crack farther from the loading point penetrated the compression zone leading to a brittle type S failure at $V_u = 145$ kN. Similar to test 1, two parallel shear cracks were initiated in test 2 at $V_{cr} = 143$ kN, but the shear crack closer to the loading point developed into the compression zone. After small load increase the compression zone failed (type S + FC) at $V_u = 146$ kN. Specimen V106 was tested three times with different shear spans in order to provoke flexural and shear cracking near the support. However, in test 1a with $a/d = 5.0$, no shear crack could be initiated and the test was aborted before reaching the predicted flexural bearing capacity. For test 1b, the supports were rearranged shortening the shear span ($a/d = 4.0$) in order to lower the moment-to-shear ratio. In this test, a small shear crack occurred simultaneously with beginning flexural failure of the concrete compression zone (FC). The test was aborted at about $V = 198$ kN before reaching $V_u$ in order to avoid crushing of the concrete compression zone. Thus, no failure pattern is available for this test. In test 2 of V106, which was executed until failure of the specimen, the shear crack formed at $V_{cr} = 204$ kN with a crack pattern favorable for strut-and-tie mechanism. After load increase, the concrete compression zone ruptured explosively (type S + FC failure) at $V_u = 218$ kN.

The specimens with $\rho_f = 2\%$ were tested twice with $a/d = 4.5$ (V108) and 4.0 (V109), respectively. V108 showed multiple parallel shear cracks which localized into a single critical shear crack near the loading point. This enabled strut-and-tie mechanism and resulted in type S

![Figure 7](image-url) Shear tests on specimens with prestressing, showing S or S + FC failure. Crack patterns observed via digital image correlation (DIC) at ultimate shear load $V_u$ (left), and after shear failure (right), F1 and F2: fiber volume fraction 1 and 2%, P50 and P100: level of prestressing 50 and 100%, $a/d$: shear span-to-depth ratio

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FIGURE 8  Load–deflection curves of shear tests. Top: specimens without prestressing, bottom: Specimens with prestressing. F0, F1, and F2: fiber volume fraction 0% (blue/violet curves), 1% (orange/red curves), and 2% (green/brown curves), P0, P50, and P100: level of prestressing 0, 50, and 100%, a/d: Shear span-to-depth ratio, $V_{cr}$ (triangles): shear cracking load, empty triangles: shear cracking initiated simultaneously with beginning flexural failure due to crushing of the concrete compression zone, S: shear failure, S + FC: combined shear failure and flexural failure due to crushing of the concrete compression zone, FC: flexural failure due to crushing of the concrete compression zone, (a) hysteresis caused by a temporary stop of the testing machine and (b) test was aborted before failure for safety reasons.
+ FC failure in both tests. $V_{cr}$ was about 157 kN and 150 kN, while $V_u$ was 160 kN and 158 kN for test 1 and 2, respectively. For the highly fiber-reinforced and prestressed specimen V109 it was, as expected, hardly possible to provoke shear failure prior to flexural failure. In both tests, a shear crack close to the loading point was initiated simultaneously with beginning flexural failure due to crushing of the concrete compression zone. Since no shear failure occurred, the failure patterns of the two tests on V109 are not depicted in Figure 7.

4 | EVALUATION OF THE SHEAR TESTS

4.1 | Development and inclination of the critical shear crack

As expected, the specimens without fibers show fewer flexural cracks with larger crack width, while the fiber-reinforced specimens show a more distributed crack pattern (Figures 6 and 7). This also applies to the shear cracks, which developed from the flexural cracks. With the fiber-reinforced specimens, parallel shear cracks occurred before failure localized in a single critical shear crack.

The position of flexural cracks and the position of the critical shear crack significantly depend on the level of prestressing. In eight out of ten tests on specimens without prestressing, the shear crack developed from a flexural crack near the support. In four out of five tests on prestressed specimens showing type S or type S + FC failure, no flexural crack could be initiated near the support due to the high compressive stress in the bottom cord. Thus, the shear crack developed from a flexural crack near the loading point which is favorable for enabling strut-and-tie mechanism. With the prestressed specimens and some of the fiber-reinforced specimens without prestressing the critical shear crack developed almost linear between tension cord and loading point (Figures 6(h,i) and 7), while with most of the specimens without prestressing the critical shear crack developed with variable inclination toward compression zone (Figure 6(a–g,j)).

4.2 | Shear bearing capacity

4.2.1 | Shear cracking load

Generally, shear failure may occur immediately after appearance of the critical shear crack. However, as presented in Table 3 and discussed in previous sections, some shear tests showed a (significant) load increase after appearance of the critical shear crack until the specimen finally failed at $V_u$. This observation can be attributed to (arbitrary) crack patterns favorable for strut-and-tie mechanism. Thus, for better comparison of the individual test results, the following evaluation focuses on the shear cracking load $V_{cr}$.

Figure 9 depicts $V_{cr}$ with respect to the fiber volume fraction. The symbols represent the different levels of prestressing, where specimens of series P0, P50, and P100 are indicated by dots, triangles, and diamonds, respectively. The numbers represent the mean values of $V_{cr}$ for each configuration. Tests with shear cracking initiated simultaneously with beginning flexural failure, which are marked with $V_{cr}$ in brackets in Table 3, are not included.

As expected, $V_{cr}$ increases with increasing either the level of prestressing or the fiber volume fraction.

4.2.2 | Influence of prestress

Increasing the level of prestressing for specimens without fibers (series F0) from 0% (V101 and V111) to 50% (V102) or 100% (V103) results in an increase of $V_{cr}$ from an average of 60 kN to an average of 108 kN (+80%) and 148 kN (+147%), respectively. Increasing the level of prestressing for specimens with $\rho_f = 1\%$ (series F1) from 0% (V104 and V114) to 50% (V105) or 100% (V106) results in an increase of $V_{cr}$ from an average of 110 kN to an average of 144 kN (+31%) and 204 kN (+85%), respectively. When increasing the level of prestressing for specimens
with $\rho_f = 2\%$ (series F2) from 0\% (V107) to 50\% (V108), $V_{cr}$ increases from an average of 126 kN to an average of 154 kN (+22\%). Comparing the specimens with different levels of prestressing, $V_{cr}$ increases by an average of 37 kN from series P0 to series P50 and by an average of 50 kN from series P50 to series P100. The effect of prestressing on $V_{cr}$ seems to be basically independent of the fiber volume fraction.

### 4.2.3 Influence of fiber volume fraction

Increasing the fiber volume fraction for specimens without prestressing (series P0) from $\rho_f = 0\%$ (V101 and V111) to $\rho_f = 1\%$ (V104 and V114) or to $\rho_f = 2\%$ (V107) results in an increase of $V_{cr}$ from an average of 110 kN (+83\%) and 126 kN (+110\%), respectively. Increasing the fiber volume fraction for specimens with a level of prestressing of 50\% (series P50) from $\rho_f = 0\%$ (V102) to $\rho_f = 1\%$ (V105) or $\rho_f = 2\%$ (V108) results in an increase of $V_{cr}$ from an average of 108 kN to an average of 144 kN (+33\%) and 154 kN (+43\%), respectively. When increasing the fiber volume fraction for specimens with a level of prestressing of 100\% (series P100) from $\rho_f = 0\%$ (V103) to $\rho_f = 1\%$ (V106), $V_{cr}$ increases from an average of 148 kN to 204 kN (+38\%). Comparing the specimens with different fiber volume fraction, the shear cracking load increases by an average of 47 kN from series F0 to series F1 and by an average of only 13 kN from series F1 to series F2. Thus, the effect of increasing the fiber volume fraction from $\rho_f = 1\%$ to $\rho_f = 2\%$ is 72\% smaller than with the increase from $\rho_f = 0\%$ to $\rho_f = 1\%$, which can also be recognized by the change of inclination of the dotted lines in Figure 9. In contrast, the residual flexural tensile strength $f_{Rm,\text{max}}$ obtained in the three-point test on notched beams (see Section 3.2) is on average 10.2 MPa for $\rho_f = 1\%$ and 17.6 MPa for $\rho_f = 2\%$. With regard to $f_{Rm,\text{max}}$, the effect of increasing the fiber volume fraction from $\rho_f = 1\%$ to $\rho_f = 2\%$ is only 27\% smaller than the effect of $\rho_f = 1\%$. The different efficiency of fibers may be attributed to a deviating distribution and orientation of the fibers in the shear specimens and flexural beams due to different geometry and presence of reinforcement. For this reason, distribution and orientation of fibers will be further investigated by optoanalytical method.31

### 4.2.4 Influence of shear span-to-depth ratio

Three out of four tests on specimens of series P0 without fibers (V101 and V111) were performed with $a/d = 3.5$ and one test with $a/d = 4.0$. All tests showed a shear failure (type S) immediately after initiation of the critical shear crack. The shear bearing capacity was obviously not influenced by the small variation of shear span-to-depth ratio in tests 1 and 2 of V111. The same applies to $V_{cr}$ of the fiber-reinforced specimens V114 and V107. However, two tests on fiber-reinforced specimens of series P0 (V104 and V107) showed a significant load increase after appearance of the critical shear crack and before reaching $V_u$ which can be attributed to a crack pattern favorable for strut-and-tie mechanism. Both tests were performed with $a/d = 3.5$, while the specimens tested with $a/d = 4.0$ failed immediately after initiation of the critical shear crack. With regard to the prestressed specimens (series P50 and P100), the high compressive stress in the bottom cord prevented formation of flexural cracks near the support even in tests performed with $a/d > 4.0$. Thus, only one out of twelve tests on specimens with prestressing showed type S failure immediately after initiation of the critical shear crack.

### 5 CONCLUSIONS AND OUTLOOK

Based on the current evaluation of 22 shear tests on UHPFRC beams, the following conclusions can be drawn:

- The time-dependent deformation of the shear specimens due to creep and shrinkage of UHPFRC results in a significant loss of prestress. The experimentally determined data could be reproduced and validated computationally.
- When increasing either the fiber volume fraction or the prestressing, the shear bearing capacity significantly increases. Furthermore, the test results suggest that the favorable effects of fibers and prestressing superimpose each other.
- The effect of prestressing on the shear cracking load seems to be quite proportional to the level of prestressing and basically independent of the fiber volume fraction.
- When increasing the fiber volume fraction, the increase of shear cracking load is significantly less than proportional to the increase of residual flexural tensile strength of UHPFRC obtained from three-point test on notched beams.
- The fibers favor multiple parallel shear cracks. In case of smaller shear span-to-depth ratio, this may lead to the formation of a critical shear crack near the loading point, allowing for strut-and-tie mechanism. However, the shear cracking load is virtually unaffected by the shear span-to-depth ratio in case of specimens without prestressing.
Due to high compressive stress in the bottom cord in case of prestressing, the shear crack develops from a flexural crack near the loading point enabling strut-and-tie mechanism even with $a/d > 4.0$. Further experiments should be performed, investigating a larger variety of shear span-to-depth ratio.

Further research and evaluation will focus on the formation and development of the critical shear crack, the fiber distribution and orientation, as well as comparison of test results with code regulations and other experiments from literature.

ACKNOWLEDGMENT
Both authors would like to thank the company Fertigbau Lindenberg OTTO QUAST GmbH & Co. KG for fabricating the test specimens.

DATA AVAILABILITY STATEMENT
The data that support the findings of this study are available in the supplementary material of this article.

ORCID
Kevin Metje https://orcid.org/0000-0002-6225-0457
Torsten Leutbecher https://orcid.org/0000-0003-2836-1900

NOTATIONS

- $a$: shear span
- $A_c$: gross cross-sectional area of concrete
- $A_n$: net cross-sectional area of concrete (without cross-sectional area of prestressing tendons)
- $A_p$: cross-sectional area of prestressing tendons
- $b$: width of the cross-section
- $b_w$: width of the web of cross-section
- $d$: effective depth of cross-section/side length of cube/diameter of cylinder
- $E_{cm}$: elastic modulus of concrete
- $E_p$: elastic modulus of prestressing steel
- $F$: load
- $F_j$: load at midspan deflection $\delta_j$
- $F_{max}$: postcracking peak load
- $f_{cm}$: mean concrete compressive strength at 28 days (cube $d = 100$ mm)
- $f_{cm}$, cube100,28d: mean concrete compressive strength at 28 days (cylinder $d/h = 100/200$ mm)
- $f_{cm}$, cyl100,28d: mean concrete compressive strength at time of the shear test (cube $d = 100$ mm)
- $f_{Rm,j}$: mean residual flexural tensile strength according to EN 14651 at midspan deflection $\delta_j$ ($j = 1–4$)
- $f_{Rm,max}$: mean residual flexural tensile strength of three-point test at postcracking peak load
- $h$: height of the cross-section
- $h_{np}$: distance between the tip of the notch and the top of the cross-section
- $I_n$: second moment of area of the net concrete section
- $l$: span of the beam/total length
- $P_0$: tensile force applied to the tendons (pretensioning)
- $P_t$: prestress force at time of the shear test
- $t$: time at the moment considered
- $t_0$: time at the transfer of prestress to concrete (releasing the pretensioned tendons from the anchorages)
- $V$: shear load
- $V_{cr}$: shear cracking load
- $V_u$: ultimate shear load
- $w$: deflection at loading point
- $z_{np}$: distance between the center of gravity of the net concrete section and the center of gravity of the cross-sectional area of prestressing tendons
- $CV$: coefficient of variation
- DIC: three-dimensional digital image correlation
- $F_0$, $F_1$, $F_2$: series with fiber volume fraction (0, 1, or 2%)
- LVDT: linear variable displacement transducer
- $P_0$, $P_{50}$, $P_{100}$: series with total force applied to the strands $P_0 = 0$, 469, or 938 kN (level of prestressing 0, 50, or 100%)
- PTFE: polytetrafluoroethylene
- PU: polyurethane
- PVC: polyvinyl chloride
- RH: relative humidity
- UHPFRC: ultra-high performance fiber-reinforced concrete
- $\delta_j$: midspan deflection according to EN 14651 ($j = 1–4$): $\delta_1 = 0.47$ mm, $\delta_2 = 1.32$ mm, $\delta_3 = 2.17$ mm, $\delta_4 = 3.02$ mm
- $\Delta l$: longitudinal deformation of shear specimen
- $\Delta \sigma_{p,c+s+r}$: absolute value of the variation of stress in the tendons at time $t$ due to creep and shrinkage of the concrete and relaxation of the prestressing steel
- $\Delta \sigma_{pr}$: absolute value of the variation of stress in the tendons at time $t$ due to relaxation of the prestressing steel
- $\varepsilon_{c,el}$: instant elastic compressive strain at time of loading $t_0$
- $\varepsilon_{c,tot}$: total strain at the end of creep test
- $\varepsilon_{cs}$: total shrinkage strain
- $\phi(t,t_0)$: creep coefficient at time $t$ due to a constant compressive stress applied at time $t_0$
- $\rho_f$: fiber volume fraction
stress in the concrete adjacent to the tendons due to self-weight, initial prestress, and other quasi-permanent actions causing creep

ORCID
Kevin Metje ©https://orcid.org/0000-0002-6225-0457
Torsten Leutbecher ©https://orcid.org/0000-0003-2836-1900

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AUTHOR BIOGRAPHIES

Kevin Metje
Chair of Structural Concrete, Department of Civil Engineering
University of Siegen
Siegen, Germany
metje@bau.uni-siegen.de

Torsten Leutbecher
Professor and Head of Section, Chair of Structural Concrete, Department of Civil Engineering
University of Siegen
Siegen, Germany
leutbecher@bau.uni-siegen.de;
https://www.bau.uni-siegen.de/subdomains/massivbau/

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How to cite this article: Metje K, Leutbecher T. Experimental investigations on the shear bearing behavior of prestressed ultra-high performance fiber-reinforced concrete beams with compact cross-section. Structural Concrete. 2021;22:3746–62. https://doi.org/10.1002/suco.202100337