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TESTING OF REINFORCED HIGH PERFORMANCE FIBRE CONCRETE MEMBERS IN TENSION

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Abstract

Ultra-high performance fibre reinforced concrete (UHPFC) is a high performance construction material, coupling strength, ductility and compacity. Up to the present time, its mechanical properties have been widely studied, but design approaches and structural applications in which it can be efficiently applied are needed. To improve this knowledge, a study on the behaviour of real-scale UHPFC ties with additional ordinary steel bars constitues an important step. The results of a test series presented here indicate that ordinarily reinforced UHPFC members in tension provide an optimal behaviour at the serviceability limit state. At the ultimate limit state, even a large amount of additional ordinary reinforcement cannot avoid a strain localisation and a brittle failure.

Keywords

Fibres, reinforced concrete, tensile test, UHPFC.

1 Introduction

Ultra high performance fibre reinforced concretes (UHPFC) are a class of cementitious composites made of a strong and compact powder-based matrix and of steel or synthetic fibres as a distributed three-dimensional reinforcement. Amongst other properties, fibres provide ductility and, for a sufficiently large amount (more than 1.5% volume of steel fibres), they lead to a strain hardening behaviour in tension. An important research effort has been made in recent years to study UHPFC as a construction material. So far, the knowledge about its behaviour in real-scale structural elements remains fragmentary, and only few structures have been realized that really exploit UHPFC properties (Aïtcin et al. 1998, Blais and Couture 1999, Cheminet and Thibaux 1999, Simon et al. 2002, Behloul and Kicul 2003, Fehling et al. 2004).

The aims of the research project under way at the Ecole Polytechnique Fédérale de Lausanne (Switzerland) are to asses the effectiveness of UHPFC as a construction material and to identify best suited structural applications. By means of an experimental as well as theoretical approach, the present work focuses on the behaviour of reinforced UHPFC in tension. The study of reinforced members in tension is considered as the first task to improve the knowledge on UHPFC behaviour in structural applications. It provides a direct comparison with the well known case of ordinary reinforced concrete in tension. Fundamental information on the use of UHPFC ties as statically determinate structural

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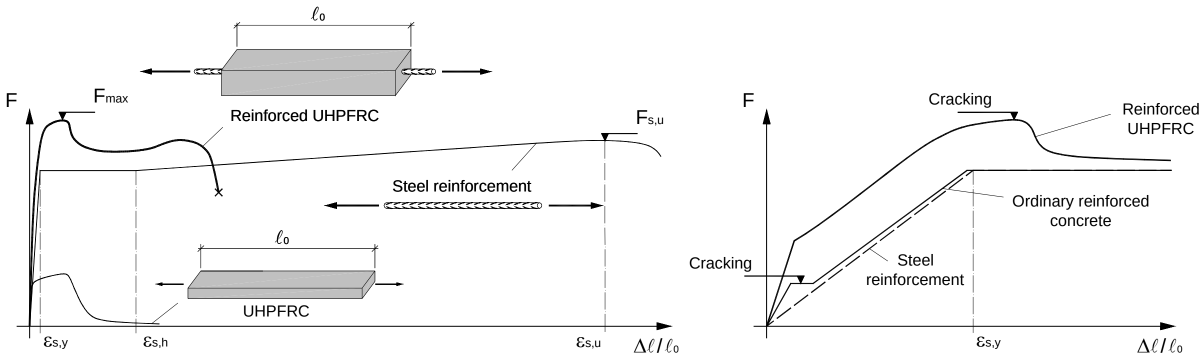
elements or as the tensile part of beams in bending and shear is also gained. A first test series has been carried out on reinforced UHPFC tensile elements to evaluate their effectiveness in structural applications and to observe the effect of using various steel types.

2 Reinforced UHPFC members in tension

2.1 State-of-the-art

So far, only a few experimental investigations have been performed on large scale reinforced UHPFC elements in tension (Fischer and Li 2002, Leutbecher and Fehling 2004). In the framework of a previous project carried out at the Ecole Polytechnique Fédérale de Lausanne, tests on UHPFC structural ties with hot-rolled steel reinforcement have been performed and a numerical model has been proposed (Jungwirth and Muttoni 2004, Jungwirth 2005).



a) b)

Figure 1 a) Force-elongation curve for UHPFC, hot-rolled steel and reinforced UHPFC in tension and b) comparison with ordinary reinforced concrete for small elongations

According to the experimental results, the following conclusions were drawn: as shown in figure 1, the contribution of the fibres starts to decrease (softening branch) for strain levels at which hot-rolled steel reinforcement is already in its yielding plateau. As a consequence, stress increases in steel bars cannot be activated to resist the stresses released by fibres, and the overall response of the reinforced element also softens. Depending on the element length, this can even lead to a snap-back behaviour. Furthermore, the combined strength of the fibres and the reinforcement across cracks leads to a peak tensile force (F_{max}) which is usually larger than the maximum strength of the steel reinforcement alone ($F_{s,u}$). This is very different from the behaviour of properly designed ordinary concrete elements, in which the yield strength of the reinforcement is larger than the cracking load (fig. 1b). For the UHPFC used in this research, $F_{s,u}$ can be higher than F_{max} only for very high amounts of ordinary steel reinforcement.

2.2 Present test series

The behaviour observed in the test described in (Jungwirth and Muttoni 2004) directly depends on the mechanical properties of the two materials (UHPFC and steel) and on their bond interaction. Relevant changes in the structural behaviour might result from using a different type of steel.

Within the framework of the current project, an additional test series was thus planned to validate the numerical model developed in (Jungwirth 2005) with additional data and to study the effect of a change in the mechanical properties and in the amount of steel. As in the previous test series, Ceracem© (Maeder et al. 2004) was used as UHPFC. Typical properties for this material are given in table 1. Two different types of reinforcing steel were selected, and a new test setup was developed.

Table 1

Mechanical properties of Ceracem©

Compressive strength	200	MPa
Tensile strength	12	MPa
Elasticity modulus	$50{\cdot}10^3$	MPa
Length of steel fibers	20	mm
Diameter of steel fibers	0.16	mm

The main task of this test series was to determine whether there is a beneficial effect associated with the use of cold-worked steel with a continuous strain hardening instead of hot-rolled steel. The absence of a yielding plateau was expected to provide a better crack control during the softening of the fibres, and to prevent the early crack localisation shown in figure 1.

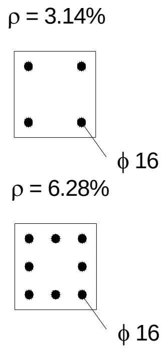
3 Test series

3.1 Layout and materials

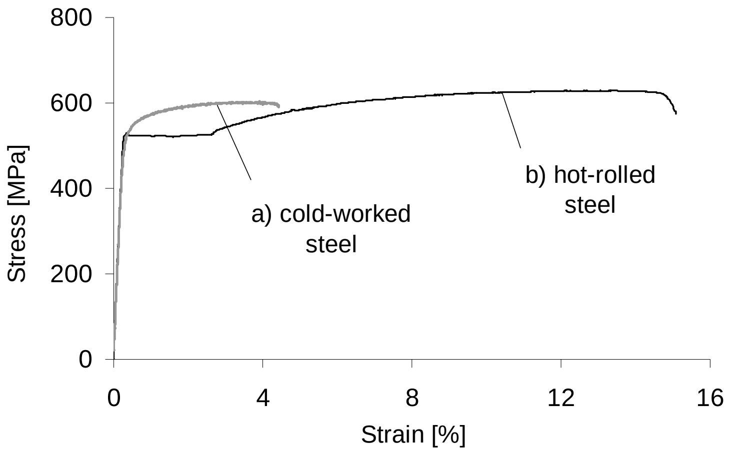
The current test series consists of six specimens with various steel grades and reinforcement ratios (tab. 2). The diameter of reinforcing bars was kept constant at 16 mm. Compression tests on 100 mm cubes were carried out at the same age as the tests on tensile members. The average compressive strengths for each specimen are given in table 2. Values higher than reference values (tab. 1) were obtained, probably due to aging.

Table 2 Test series description and average compressive strength

Specimen	*Steel type*	*Bars*	ρ [%]	Test age [days]	f_{cc} [MPa]
S-41	Hot-rolled	4 ϕ 16	3.14	68	229
S-42		4 ϕ 16	3.14	78	245
S-81	Cold-worked	8 ϕ 16	6.28	82	241
R-41		4 ϕ 16	3.14	63	244
R-42		4 ϕ 16	3.14	76	234
R-81		8 ϕ 16	6.28	82	235



Two grades of steel were used: cold-worked and hot-rolled ordinary steel available on the Swiss market. Tensile tests were carried out on 500 mm long bars. The measured stress-strain curves are given in figure 2.



	E *[GPa]*	f_y or $f_{0.2}$ *MPa]*	f_t *MPa]*
Hot-rolled	*200*	*525*	*632*
Cold-worked	*195*	*548*	*603*

Figure 2 Stress-strain curves for cold-worked steel and hot rolled steel

3.2 Test setup and specimen

The dimensions of the test specimen were chosen to be representative of actual structural elements and to provide a cross section large enough to place various types and amounts of reinforcement. A 160 x 160 mm cross-section on a 1.00 m measurement length was chosen. The load introduction system was designed to prevent the development of eccentricities or unexpected end rotations. Rigid end conditions were chosen as the best solution from a constructive point of view.

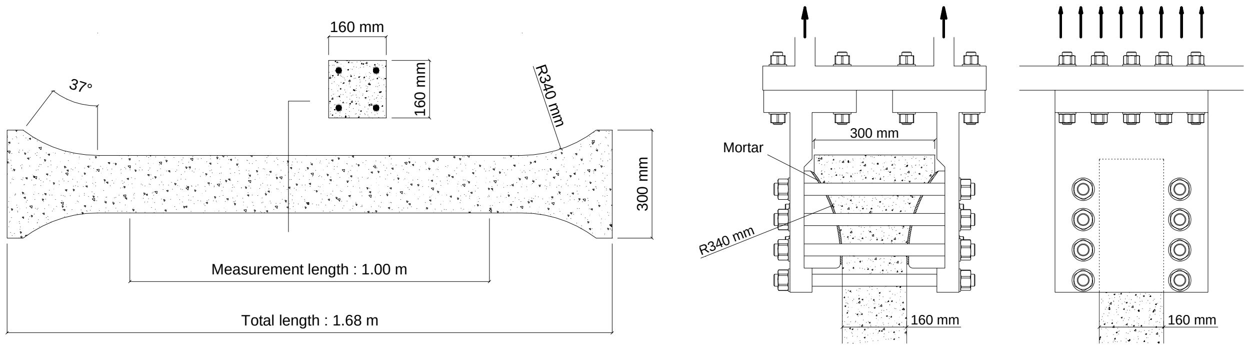


Figure 3 Dog bone shaped specimen and load introduction system

A curve dog-bone shaped specimen was chosen (fig. 3). The radius of curvature was optimized both to minimize transversal tensile stress concentrations in the specimen and to reduce the opening forces acting on the anchorage device. However, transversal forces up to 50% of the applied tension were estimated, for which a very stiff transverse system was designed (fig. 3). The final test set up is shown in figure 4.

Figure 4 Test set up and instrumentation

4 Test results

4.1 Load-displacement relationships

All tests were carried in a 10 MN universal testing machine under displacement controlled conditions. The imposed displacement speed was 0.5 mm/min. Test results for all the six specimens are presented in terms of load–displacement curves (fig. 5 and fig. 6). The load was measured by the testing machine sensor, while the displacement is the average value of the elongations measured on the sides of the specimen by four longitudinal inductive transducers on the 1.00 m measurement length.

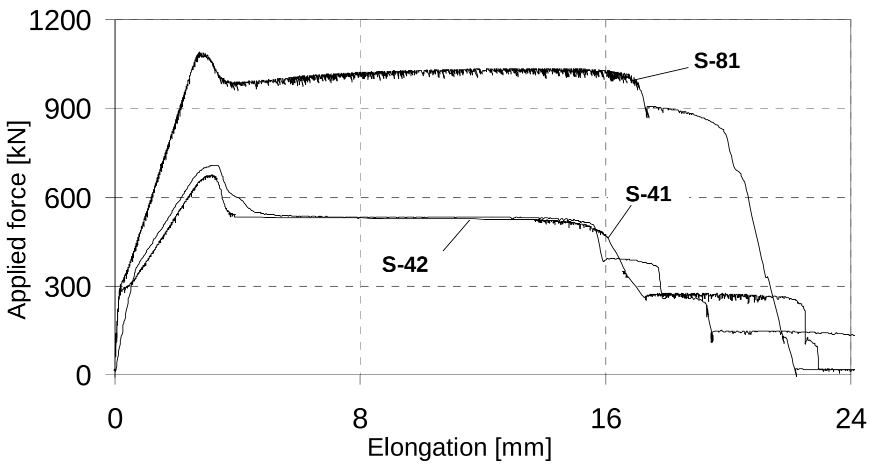


Figure 5 Load-displacement curves for hot-rolled steel (ℓ_0=1.00 m)

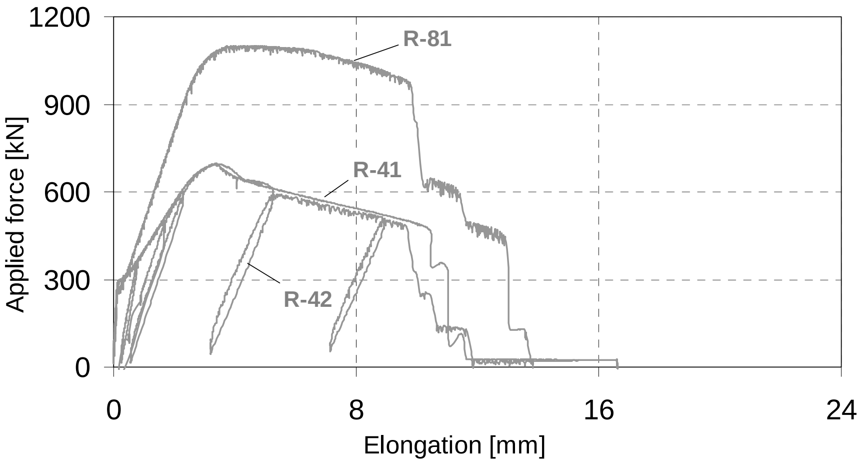


Figure 6 Load-displacement curves for cold-worked steel (ℓ_0=1.00 m)

From figures 5 and 6, the different performance of hot-rolled ("S" series) and cold-worked ("R" series) steel clearly appears. The first series provides a much larger elongation at ultimate. In the second series, cold-worked steel better counters the localisation of cracks immediately after the onset of softening.

Regardless of the type and the amount of steel, neither early strain localisation nor insufficient ultimate steel strength can be avoided. For cold-worked steel (fig. 6), the absence of a yielding plateau provides a much better crack control around the peak value, which is clearly visible for the higher reinforcement ratio. However, this effect in insufficient to delay strain localisation up to a higher deformation level. Hot-rolled steel specimens (fig. 5) exhibit a strong post peak localization, which is practically not influenced by doubling the reinforcement ratio, because of the yielding plateau. In the post peak branch, hot-rolled steel with 6.28% reinforcement ratio allows a significant stress increase to take place at high strain levels. Once again, this stress increase is insufficient to reach again the peak stress value.

4.2 Cracking

For all the specimens, various cracking phases were observed (fig. 7). The uncracked linear elastic phase ended into a micro-cracking phase, characterized by very thin cracks spaced some 10 mm apart (fig. 7a). This crack pattern was already observed in former tests (Jungwirth 2005), and is due to the bridging effect of the fibres across the crack. For loads of about 70% of the peak load, several visible cracks start to develop, spaced from 20 to 100 mm (fig. 7b). Like for ordinary reinforced concrete, the crack pattern is governed by the transfer length between bars and concrete, which depends on bond. For UHPFC, however, this interaction is influenced by the presence of fibres across the cracks, which leads to an irregular crack pattern and causes cracks to be more closely spaced and thinner than in ordinary concrete. After the peak load, softening of the fibres starts at one of the cracks, and leads to strain localization. For increasing imposed displacement, the applied load decreases. The localised crack opens widely (fig. 7c) up to nearly complete pull-out of fibres. Ultimately, the steel bars break up one after another (fig. 7d).

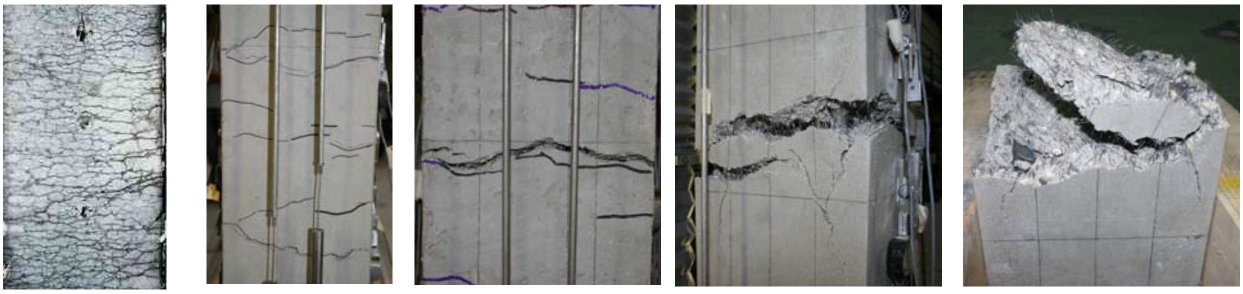


Figure 7 Crack pattern at different stages (from left to right): a) micro cracks formation (Jungwirth 2005), b) visible cracks, c) strain localisation, d) crack opening at the failure of the steel bars, e) final crack surface

At large strains, concrete tended to spall off in the vicinity of the main crack, with longitudinal cracks spreading along the location of the bars. However, no brittle splitting took place thanks to the transverse retaining effect of the fibres (fig. 7d). The final crack pattern is very irregular (fig. 7e) due to the local orientation of fibres in the crack zone.

The behaviour of the tensile elements is in good accordance with the predictions of the numerical model (Jungwirth 2005). The test series has allowed observing cracking phases that were not clearly detected during the previous test series:

- the formation and stable propagation of several visible cracks before peak (fig. 7b);
- the progressive crack opening in the post-peak, up to the failure of the reinforcing bars (fig. 7d)

5 Reinforced UHPFC for structural applications

5.1 Serviceability and ultimate limit states

In terms of stiffness, crack control and tension stiffening effect, reinforced UHPFC outperforms ordinary reinforced concrete. At the ultimate limit state, on the contrary, the joint action of the fibres and of the reinforcing bars across the crack leads to a shortcoming: strain localization takes place when steel reinforcement is yielding, and the stress released by the fibres cannot be picked up by the reinforcement. Changing the kind and the amount of steel cannot fully avoid this effect.

5.2 Minimal reinforcement for UHPFC

In ordinary concrete, minimal reinforcement to prevent brittle failure at cracking is defined as:

$$\rho_{\min} = \frac{f_{c,t}}{f_{s,y}} \tag{1}$$

For UHPFC, an approach similar to that of ordinary concrete can be followed, taking into account its higher tensile strength and the fact that crack localisation takes place when the reinforcing steel is already in its plastic range:

$$\rho_{\min} = \frac{f_{c,t}}{\left(f_{s,u} - f_{s,loc}\right)} \qquad (2)$$

$f_{s,u}$ is the ultimate strength of steel and $f_{s,loc}$ is the stress in the steel, at crack localisation. As shown by the experimental results, $f_{s,loc}$ corresponds closely to the yielding stress of steel. Thus:

$$\rho_{\min} = \frac{f_{c,t}}{\left(f_{s,u} - f_{s,y}\right)} \qquad (3)$$

For the ordinary steels used in our tests, very high reinforcement ratios are obtained (20% for hot-rolled steel and 11% for cold-worked steel).

5.3 Alternative solutions

Ordinary reinforced concrete in tension needs special care to assure a satisfactory serviceability behaviour. UHPFC greatly improves crack control at the serviceability limit state, but it can lead to an unsuitable behaviour at the ultimate limit state. For the UHPFC used in this experimental research, it has been shown that ordinary steel can improve the behaviour at ultimate only if very high reinforcement ratios are used. From a practical point of view, alternative solutions need to be found. Three approaches are proposed.

- Use special steel products, with a more pronounced continuous strain hardening, and much higher ratios between ultimate and yielding stresses.
- Adapt the UHPFC composition to obtain different values of its mechanical properties: cracking stress, tensile strength and strain at peak stress. This may be the most viable solution to avoid strain localisation at yielding of steel while guaranteeing at the same time a good crack control in service.
- Use steel bars with different bond properties to modify the relationship between the opening of the critical crack and the corresponding local deformation of steel. This might prevent the strain softening of the fibres and the yielding of steel bars to develop at the same time.

6 Conclusions and future work

A test series has been carried out on UHPFC reinforced with two types of ordinary steel reinforcement.

- At the serviceability limit state, the behaviour of tensile members is very positively affected by the interaction between UHPFC and steel. Up to significant load levels, cracks are very thin and closely spaced. Moreover, tension stiffening is much more effective than in ordinary concrete, leading to a substantial increase in stiffness.
- UHPFC tensile members cannot be made fully ductile by adding ordinary reinforcement. This results from the inherent mechanical properties of the materials. Alternative solutions need to be found, and three approaches have been proposed for future work.

- An alternative definition of the amount of minimum reinforcement to avoid brittle failure in UHPFC tensile members has been proposed.

A further test series will be carried on UHPFC tensile members to study the effects of using high performance steel reinforcement and different types of fibre reinforced concretes. A theoretical study will be carried out to model the behaviour of UHPFC tensile members in statically indeterminate systems. Work will then focus on the behaviour of UHPFC in compression.

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