Behaviour of UHPFRC-RC composite beams subjected to combined bending and shear

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Abstract
An emerging technique for strengthening of reinforced concrete (RC) slabs is to add a layer of Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) with steel rebars as an external tensile flexural reinforcement. This paper presents the results of an experimental campaign to study the structural response of composite UHPFRC-RC beams. The resistance of the UHPFRC layer subjected to combined tension and bending controls the structural response. The results show that the UHPFRC layer increases the member shear strength without impairing the deformation capacity. The Critical Shear Crack Theory is used to show the enhanced response of the composite beams.

1. Introduction

The addition of a thin overlay of Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) to reinforced concrete (RC) members is an emerging technique to protect and strengthen existing structures. UHPFRC belongs to a family of high-strength cementitious composites that have a quasi-impermeable matrix. Fine steel fibres distributed in the matrix increase the material's tensile resistance and provide its significant ductility. These properties make UHPFRC especially suitable as a top-layer external flexural reinforcement on RC slabs and bridge decks that are exposed to extreme mechanical and environmental actions.

A 40 to 60-mm-thick UHPFRC layer on an RC slab acts as an additional external bonded reinforcement, creating a composite member [1]. Flexural strengthening of RC members requires structural safety checks for shear resistance and deformation capacity. Unlike a pure flexural reinforcement, the UHPFRC layer resists both tangential and normal stresses; hence, it can contribute to the member flexural-shear resistance. There is currently no model for predicting the flexural-shear response, shear dimensioning and checking of structural safety of UHPFRC-RC composite (composite) members.

This paper presents an experimental investigation on the structural response of composite beams subjected to combined bending and shear. The aim is (1) to demonstrate the improved structural response of a composite member and (2) to quantify both the higher flexural-shear resistance and the deformation capacity provided by the layer of UHPFRC.

The existing models for estimating the shear resistance of RC members that are based on both equilibrium and compatibility conditions provide the means to estimate the contribution of the RC element to the flexural-shear resistance of a composite member. For example, the Critical Shear Crack Theory (CSCT) relates the shear resistance of members without shear reinforcement and with various ratios of tensile flexural reinforcement to an imposed rotation and the opening of a critical flexural-shear crack [2,3]. Tensile flexural reinforcement hinders the flexural-shear crack opening, thus reducing member deformation capacity while increasing the resistance. A purely flexural reinforcement (e.g., steel bars or FRP strips) does

not change the CSCT failure criterion [4]. Using this analogy, the CSCT is used to show the increase in the resistance and deformation capacity of a composite member provided by the UHPFRC layer with a shear resistance of its own.

2. Experimental investigation

2.1 Specimens and test setup

Figure 1 shows the typical geometry and reinforcement of the composite beam specimens and the static system of the cantilever-beam test setup. The specimens are from two different beam series (B and S series). The span length L_{span} and the lever-arm length a of all beams are 1600 mm and 800 mm, respectively. The UHPFRC layer on top acts as a negative flexural reinforcement. External vertical prestressing is added between the supports to avoid a failure outside the lever arm. The hydraulic jack applies monotonically increasing deflection to the end of the lever arm.

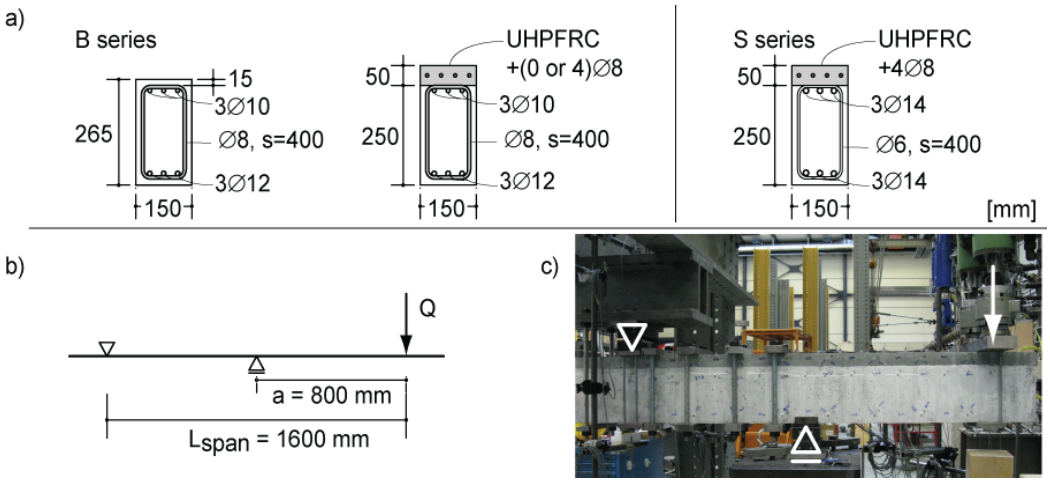

Figure 1: (a) Geometry and reinforcement details; (b) Static system; (c) Test setup

The test parameters include the shear-reinforcement ratio in the RC element ($\rho_{sv}=A_{sv}/A_{cw}$, where A_{sv} is the area of stirrups in concrete and A_{cw} is the area of concrete in which the stirrups are effective), the ratio of total added reinforcement (i.e., a 50-mm UHPFRC layer and its rebars), as well as the type of reinforcing bars in UHPFRC. Smooth and ribbed rebars of various grades of steel are used to study the UHPFRC-rebar interaction at different states of stress and cracking [5].

The RC elements are designed with a C30/37 concrete with a maximum aggregate size of 16 mm. The average tested concrete cylinder compressive strength f_c is 42 MPa. The concrete elements are reinforced with B500B steel bars (with the nominal yield stress of 500 MPa).

The specimens are designed with the UHPFRC class HIFCOM13. Oesterlee [6] provides the mix properties and the tensile and flexural behaviour of UHPFRC plates. The elastic strength of the material in tension $f_{Ut,1}$ is 9 MPa (at a strain of 0.015%). The average tensile strength f_{Ut} is 11MPa (at a strain varying from 0.1 to 0.4%). The tensile strength corresponds to the end of the strain hardening of UHPFRC [6].

Table 1 lists the properties of the beams. Among other test parameters are the ratio of a to the effective static height d, ρ_{sv}, the stirrup spacing s, the characteristic of the steel rebars in UHPFRC (including the actual yield strength), as well as the mechanical ratio of the tensile reinforcement in concrete ω_s and the added reinforcement, i.e., the UHPFRC layer ω_U and its steel rebars ω_{sU}. For material i, this ratio is defined as $\omega_i = (A_i f_i)/(A_c f_c)$, where f_i is the elastic strength of i.

Table 1: Summary of specimen design

Series	Beam ID	a/d	s [mm]	ρ_{sv} [%]	steel reinforcing bars in UHPFRC				ω_s [%]	ω_U [%]	ω_{sU} [%]
					n.ϕ [mm]	Steel ID	Surface	f_{sU} [MPa]			
B	01BRC0	3.4	400	0.17	--				7.5	--	
	03BRCU0	3.2		0.17	--	--	--	--		4.3	--
	07B4x50R0	3.1		0.17	4ϕ8	B500 B	Ribbed	566		4.2	7.2
	10B4x50S0	3.1		0.17		11SMn30	Smooth	516		4.2	6.6
	14B4x70S0	3.1		0.17		ETG88	Smooth	702		4.2	9.0
S	18S4x70R0	3.2		0.09		UGIGRIP 1.4362	Ribbed	710	14.7	4.2	9.1

2.2 Results

Table 2 summarises the test results: the failure mode of each beam, the angle of the critical flexural crack in concrete causing member failure ($\theta_{failure}$), the applied vertical displacement of a beam at the jack with respect to the strong floor (Δ_{jack}), the calculated rotation at peak resistance (ψ_{peak}), the moment at peak (M_{peak}), and the residual moment (M_{res}) following the first drop in resistance due to the critical flexural-shear crack in the RC element.

Table 2: Summary of test results

Beam ID	Failure	$\theta_{failure}$ [deg]	Δ_{jack} [mm]	ψ_{peak} [rad]	M_{peak} [kNm]	M_{res} [kNm]	$\frac{M_{res}}{M_{peak}}$
01BRC0	Flexure-Shear	25	15.6	0.020	34.6	24	0.69
03BRCU0	Flexure	62	10.9	0.014	47.2	--	--
07B4x50R0	Flexure-Shear	30	14.5	0.018	72.6	54	0.74
10B4x50S0	Flexure	55	15.0	0.019	83.7	--	--
14B4x70S0	Flexure-Shear	32	15.0	0.019	79.7	45	0.56
18S4x70R0	Flexure-Shear	30	11.9	0.015	72.7	65.4	0.90

Figure 2a shows the force-displacement (Q-D) response of the beams, that is the plot of the measured resistance of the beam Q versus the applied displacement Δ_{jack}. In these plots, a sudden drop in resistance is shown with a dotted line. The tests were stopped either after yielding of the tensile flexural reinforcement in concrete or within the post-peak regime after the full development of the critical flexural-shear crack in the RC element (flexural-shear failure). In Figure 2a, the test results are compared to the calculated CSCT failure criterion for a reinforced concrete beam without shear reinforcement that has an average d of 250 mm [2]. Figure 2b shows the crack pattern of the beams either after their flexural-shear failure or flexural failure and yielding of the tensile reinforcement in the RC element. Note that only the macro cracks in the UHPFRC layer are traced.

3. Discussion

The shear resistance of the RC element of each specimen is primarily provided by the friction and aggregate interlock along the inclined flexural-shear cracks [3,7]. As illustrated in Figure 2a, Beam 01BRC0 (the reference beam) has a flexural-shear failure (the drop in resistance caused by the full development of the critical flexural-shear crack) just after yielding of the tensile steel reinforcement. Following this failure, the dowel action and membrane action of the compressive and tensile reinforcement, respectively, continue to provide between 45 and 70% of the peak resistance [8].

The UHPFRC layer in Beam 03BRCU0 prevents the flexural-shear failure of the RC element. In the cases of Beams 07B4x50R0, 14B4x70S0 and 18S4x70R0, however, the over reinforcement of the RC elements provokes the flexural-shear failure. Regardless of their failure mode, the specimens have a higher deformation capacity than the CSCT prediction.

Note that the deformation capacity of Beams 07B4x50R0 and 14B4x70S0 is close to that of Beam 01BRC0.

Beam 07B4x50R0 has the least gain in ductility and resistance prior to its flexural shear failure. At Δ_{jack} of 14.4 mm (a/Δ_{jack} = 55) corresponding to the peak resistance of 90.3 kN, the deformation capacity and the resistance are respectively 1.2 and 1.6 times the values estimated by the CSCT for the beam with an equivalent ratio of tensile flexural reinforcement.

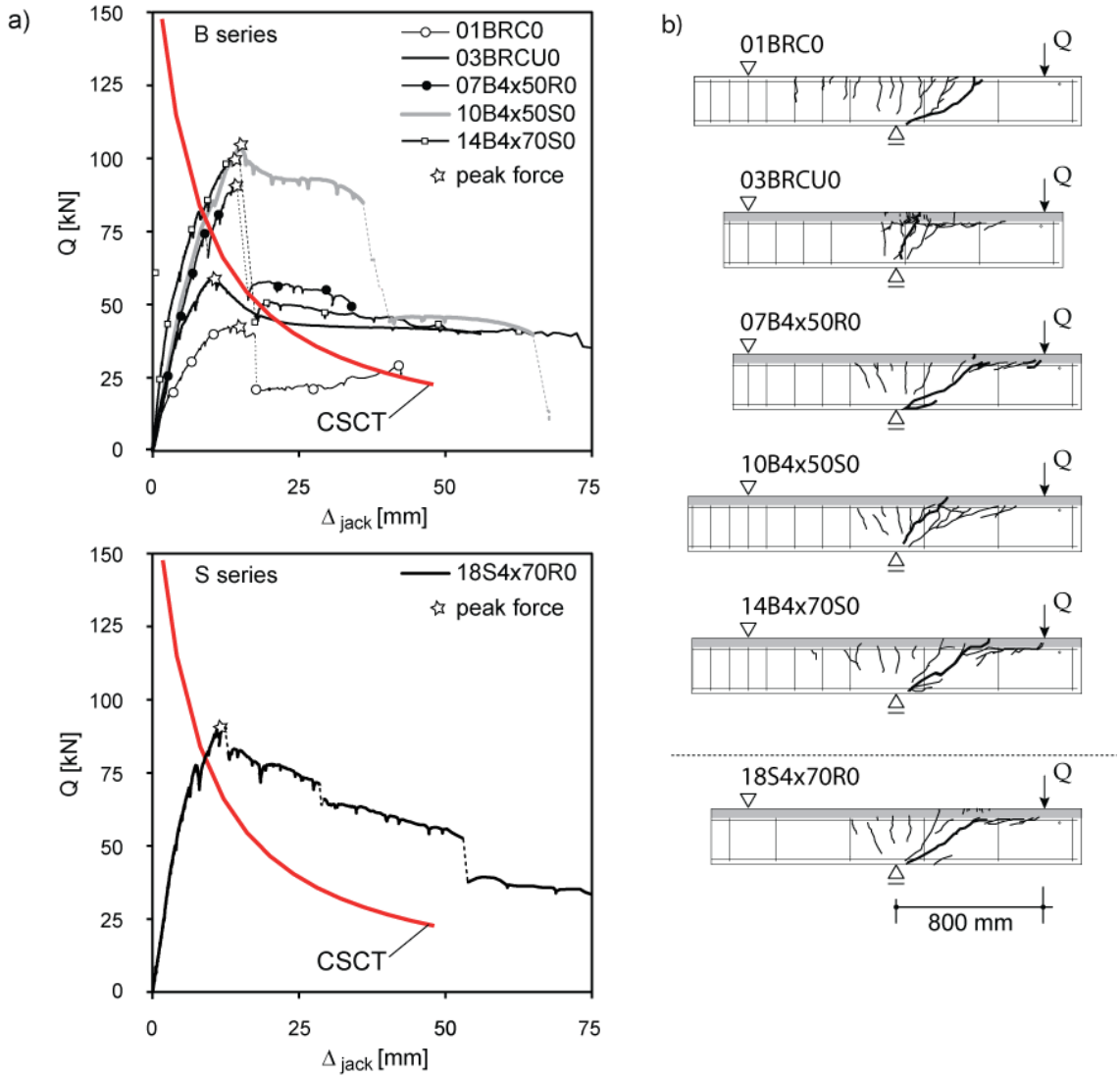

Figure 2: (a) Structural response and estimated CSCT failure criterion;
(b) Crack pattern after peak

The surface characteristic and bond condition of the rebars in UHPFRC influence the failure mode of the composite beams. Smooth bars enable an arching stress field that transfers a portion of the stresses from the jack to the support. The resistance of the arch mechanism depends on the magnitude of the stresses carried by the arch and the resistance of the concrete compression zone at the support. Take Beams 07B4x50R0, 10B4x50S0, and 14B4x70S0: Beam 10B4x50S0 (with smooth rebars) has the same reinforcement ratio as Beam 07B4x50R0 (with ribbed rebars), yet the first one fails in flexure. The larger magnitude of the arching stresses in Beam 14B4x70S0 (with a higher ω_{sU} than Beam 10B4x50S0) leads to the formation of horizontal cracks in the compression zone, thus the strength reduction of the RC element and the flexural-shear failure of the beam.

The deformation capacity, load bearing mechanisms, and mode of failure of the composite beams strongly depend on the cracking behaviour and the strength of the compression zone in the RC element. Figure 3 shows the crack pattern of Beam 18S4x70R0 before and after its peak resistance and flexural-shear failure. The cracking of concrete along the UHPFRC-RC interfacial zone changes the bond condition between the composite elements and the resistance mechanisms of the member. This figure also shows the assumed evolution of the distribution of the shear stresses (τ_{int}) transferred between the elements as a function of the crack opening.

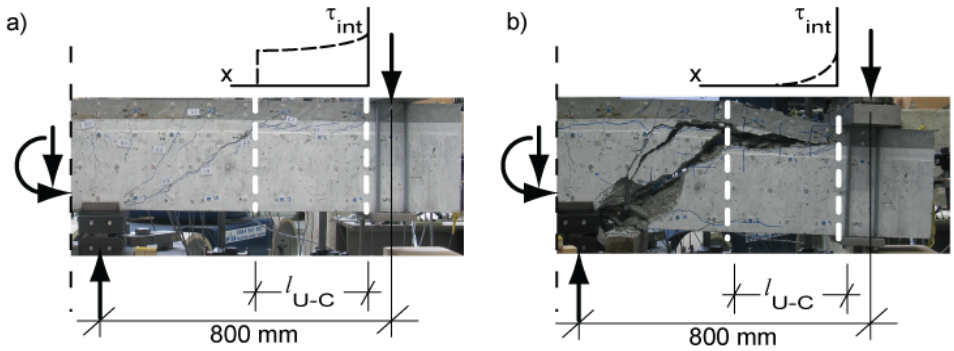

Figure 3: Crack pattern and assumed shear transfer between the elements of Beam 18S4x70R0 (a) at peak force and (b) in the post-peak regime

Figure 4 shows the observed crack pattern in the RC element and the formation of a pair of hinges in the UHPFRC layer of a composite beam at its flexural-shear collapse:

- Crack 1 is the critical flexural-shear crack that reduces the RC element contribution to the member shear resistance with increasing crack opening.
- Crack 2 appears in concrete below the interface at the level of the tensile reinforcement in the RC element. This crack is induced by Crack 1. By reducing the shear stress transfer between the RC and UHPFRC elements, Crack 2 increases the pre-peak deformation capacity of the member.
- At the locations 3 and 4, two plastic hinges develop in the UHPFRC layer which is subjected to combined tension and bending (in double curvature). These hinges form as the UHPFRC strain-hardening begins; a pair of cracks becomes visible with increasing deformation when the UHPFRC is in its strain-softening phase. Test results show that the plastic hinges provide a significant amount of shear resistance to the member resistance, which makes up for the loss of the RC element contribution.
- The flexural-shear collapse mechanism is attributed to the full development of Crack 1, bridged by the steel rebars and the UHPFRC layer, which continue to resist stresses in the post-peak regime.

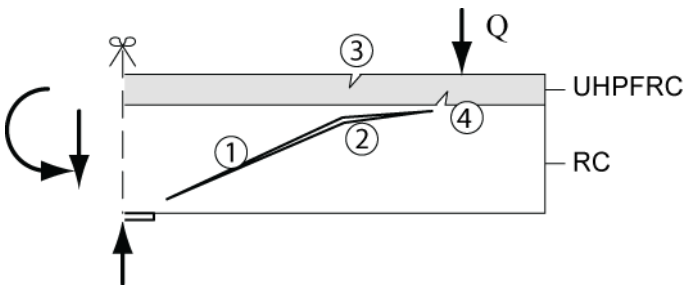

Figure 4: Flexural-shear collapse mechanism: typical crack pattern and formation of hinges

The post-peak response of a composite beam is mainly characterised by the response of the plastic hinges in the UHPFRC layer described by a plastic moment – rotation relation for the interaction of bending moment and tension force in the layer.

4. Conclusions

This paper shows that the UHPFRC layer in a composite UHPFRC-RC member subjected to combined bending and shear not only increases member shear resistance but also can significantly increase member deformation capacity.

Unlike a purely tensile reinforcement, the UHPFRC layer provides an alternative load bearing mechanism by carrying a portion of the stresses in bending. Member resistance is controlled by the cracking behaviour and the degree of composite action between the elements.

The resistance and ductility of a composite member depend on: (1) the critical flexural-shear crack in concrete reducing the contribution of the RC element, (2) the concrete crack below the interface reducing the composite action between the elements, and (3) a pair of plastic hinges in the UHPFRC layer subjected to combined tension and bending. The material properties and layer thickness as well as the steel rebars in UHPFRC control the structural response.

Future work includes (1) development of a model as an explicit function of crack opening and (2) validation of the model based on test results.

References

[1] K. Habel, *Structural Behaviour of Elements Combining UHPFRC and Concrete*, PhD Thesis no. 3036, EPFL, Switzerland, 2004.
[2] A. Muttoni "Punching shear strength of reinforced concrete slabs without transverse reinforcement", *ACI Structural Journal*, vol. 105, no. 4, pp. 440-450, 2008.
[3] A. Muttoni and J. Schwartz. "Behavior of Beams and Punching in Slabs without Shear Reinforcement". *IABSE Colloquium*, vol. 62, pp. 703-708, 1991.
[4] A. Muttoni, M. Fernández Ruiz, "Exemples de renforcement contre le poinçonnement". *Sécurité structurale des parkings couverts*, SIA, D 0226, pp. 57-74, Zürich, 2008.
[5] C. Oesterlee et al., "Tragverhalten von Verbundbauteilen aus bewehrtem UHFB und Stahlbeton ", *Beton- und Stahlbetonbau*, vol. 104, no. 8, pp. 462-470, 2009.
[6] C. Oesterlee et al., "Strength and deformability distribution in UHPFRC panels", CONMAT 09, Nagoya, Japan, 24-26 August 2009. pp. 390-397, 2009.
[7] M.P. Collins and D. Mitchell, *PC Structures*, Response Publications, Toronto, 1997.
[8] Y. Mirzaei, *Post-punching behavior of RC slabs*, PhD Thesis no. 4613, EPFL, 2010.