

ANALYTICAL MODELLING OF R-UHPFRC – RC COMPOSITE MEMBERS SUBJECTED TO COMBINED BENDING AND SHEAR

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Abstract

The addition of a thin overlay of reinforced UHPFRC (R-UHPFRC) to a reinforced concrete (RC) element creates a composite member which can be used for the strengthening of an existing structure and in the design of new structures. With its strain hardening and softening behavior and high resistance in tension, the UHPFRC layer serves as an external tensile reinforcement contributing to both flexural and shear resistance of the RC element. The main failure modes of a composite section were identified during previous experimental campaigns. In this paper, experimental results on a composite slab strip are presented to demonstrate how the layer contributes to the shear resistance. Using this example, an analytical model to predict the behavior and calculate the resistance of a composite member is presented and applied.

Résumé

L'ajout d'une couche de BFUP armé sur un élément de béton armé permet de créer une section composée. Cette technique peut être utilisée pour le renforcement de structures existantes ou dans la conception de nouveaux ouvrages. Avec son comportement écrouissant et adoucissant ainsi que sa grande résistance en traction, la couche de BFUP sert de renforcement externe en traction et contribue à augmenter la résistance à la flexion et à l'effort tranchant de l'élément de béton armé. Les principaux modes de rupture des éléments composés ont été identifiés lors de précédentes campagnes expérimentales. Dans le présent article, les résultats d'un essai subséquent sont utilisés afin d'illustrer comment la couche de BFUP contribue à la résistance à l'effort tranchant d'un élément composée. En utilisant cet exemple, un model analytique permettant de prédire le comportement et de calculer la résistance d'une section composée est présenté et mis en application.

1. INTRODUCTION

Many structural elements are submitted to high shear forces combined with bending moments, such as structural systems with hogging moments and the examples shown in

figure 1. To increase the ultimate resistance of these concrete elements without increasing significantly the dead load of the structure, it is possible to add a layer of 30 to 60 mm of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) with or without small diameter steel reinforcement bars, thus creating a composite section [1, 2] (figure 2). The layer of UHPFRC serves as a tensile reinforcement for the reinforced concrete (RC) section.

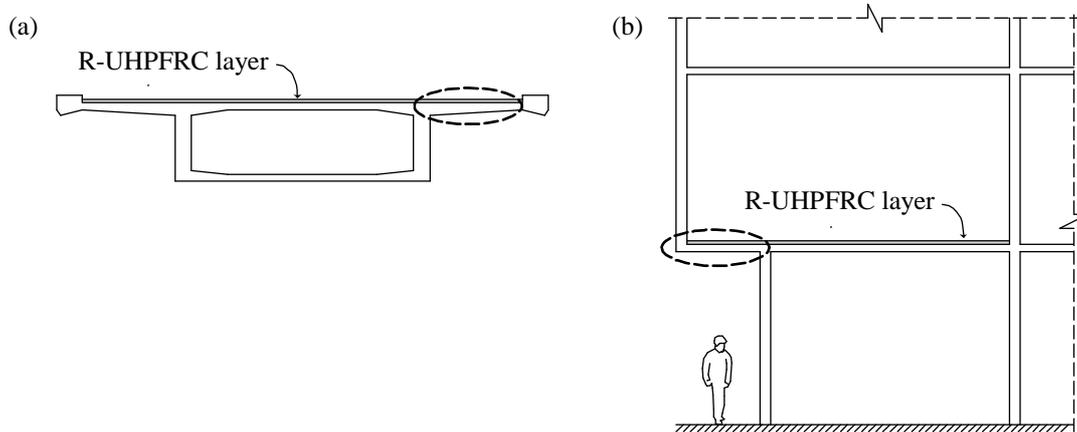


Figure 1: Example of structures submitted to combined bending and shear:

(a) cantilevers on a box girder bridge; (b) cantilever floor in a building.

To study the behavior and identify the main failure modes of composite elements when subjected to combined bending and shear, a vast experimental investigation was undertaken [3, 4, 5]. Two main failure modes were then observed: a flexural failure and a flexure-shear failure. Following this experimental campaign, collapse mechanisms of composite beams were identified and an analytical model to predict the response and resistance of the beams was proposed [6, 7].

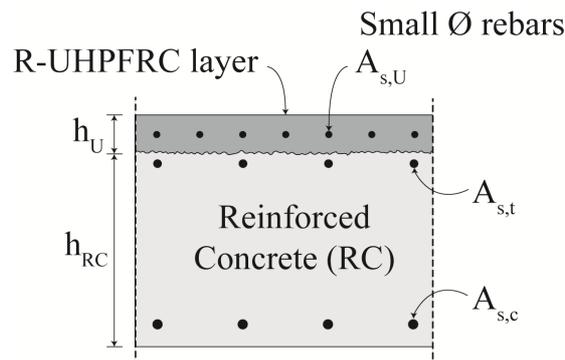


Figure 2: Typical composite section [2]

This paper presents the results of an experiment carried out later on a wider specimen representing a slab strip. This test will be used to illustrate how the layer of UHPFRC contributes to the shear resistance of the concrete element. The analytical model developed in [6] will then be presented and applied to this example. To conclude a field application on a bridge in Switzerland is briefly described.

2. EXPERIMENTAL RESULTS

2.1 Previous results

The main experimental campaign on composite beams was carried out using specimens similar to slab ribs: they had a width of 150 mm and a total height of 300 mm including the 50 mm layer of UHPFRC. Two types of setup were used: a four-point bending setup and a cantilever setup.

The flexural tests showed that the layer of R-UHPFRC, when used as a tensile reinforcement, increases the resistance up to 165% compared to a reference RC beam [8]. A flexural failure happens along a vertical crack where the highest moment is applied. Close to ultimate resistance, the vertical critical crack widens in the RC section while, over it, in the UHPFRC layer, distributed microcracks develop; no debonding is observed at the interface between the UHPFRC layer and the concrete [9]. In the post-peak domain, fracture of R-UHPFRC occurs with the localization of a macrocrack followed by the formation of a crack in the concrete parallel to the interface as illustrated by figure 3. This horizontal cracking appears near the tensile reinforcement in the concrete and its progression is limited. It is thus supposed that the behavior of composite beams is monolithic when submitted to pure flexural moments.

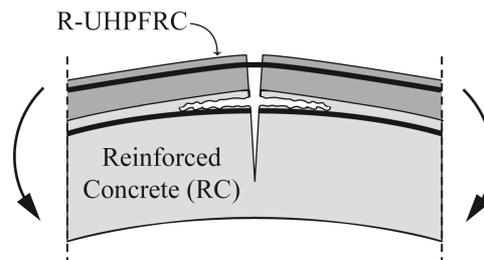


Figure 3: Failure of composite section in bending [9]

Subsequently to these flexural tests, composite beams placed in a cantilever test setup were submitted to combined bending and shear. These tests demonstrated that the layer of UHPFRC increases the resistance of the RC sections without modifying the rotation capacity when failing in shear [3]. In the case of a RC section with insufficient shear capacity, an additional layer of UHPFRC can modify the failure mode from flexure-shear failure with little deformation to a ductile flexural failure.

This test series allowed studying the flexure-shear failure mode of a composite beam and understanding how the UHPFRC layer contributes to the shear resistance of the member. A flexure-shear failure occurs when a flexural vertical crack in the RC section rotates towards the roller support and develops diagonally. The widening of this inclined crack creates a prying action on the UHPFRC layer which induces a softening of the concrete below, as illustrated in figure 4. This softening occurs between the mouth of the inclined crack and the point where the force acts and is known as the Intermediate-Crack induced Debonding (ICD) [3]. With this debonding, the section can no longer be considered as monolithic and the member stiffness decreases. Over the ICD zone, the R-UHPFRC layer resists to the prying action and is submitted to double curvature.

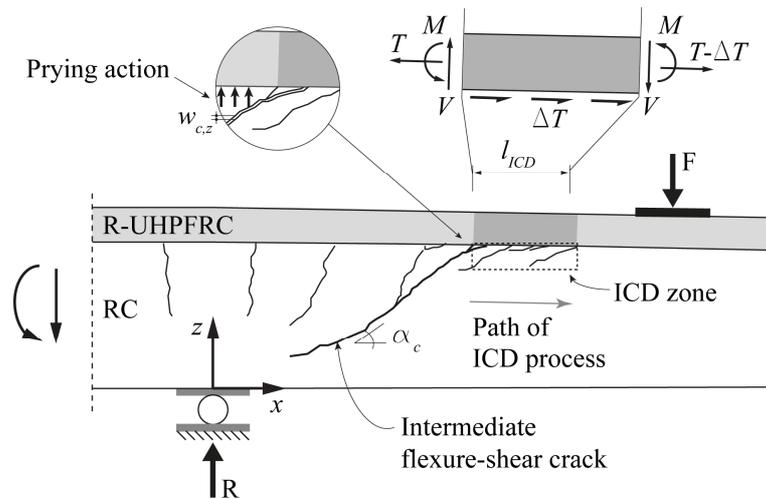


Figure 4: Flexure-shear failure mode of composite beams [3]

2.2 Specimen and test setup

To illustrate this flexure-shear failure mode, a test conducted on a slab-like beam is presented hereafter. The composite beam tested had a length of 1900 mm and resembled a slab strip with a width of 400 mm and a total height of 220 mm (figure 5). The concrete section had a tensile reinforcement ratio of 1.2% and no shear reinforcement except for three stirrups located at the middle point and at both end of the beam for construction reasons. The concrete used to cast the RC section had a compressive strength of 64.5 MPa with a maximum aggregate size of 16 mm.

The layer of R-UHPFRC was 50 mm thick with four 10 mm rebars. The mix used was developed in-house and is called HIFCOM 13. It has a 3.0 % (vol.) content of 13 mm straight steel fibers with a diameter 0.16 mm. HIFCOM 13 has a hardening and a softening behavior in tension. Its average elastic limit and ultimate tensile resistances are of 10 and 12 MPa. All steel rebars used for this specimen are of B500B with a nominal yield limit of 500 MPa.

The concrete section was first cast upside down. The bottom of the formwork was painted with retarder which prevented the hydration of the cement. After de-molding, this surface was washed off in order to obtain a rough surface with exposed aggregates. It is on this surface that the UHPFRC layer was cast, over 28 days after. This insured a good connection between the UHPFRC layer and the concrete section. For a rehabilitation project, this surface may be obtained by high pressure water jet (hydro-demolition).

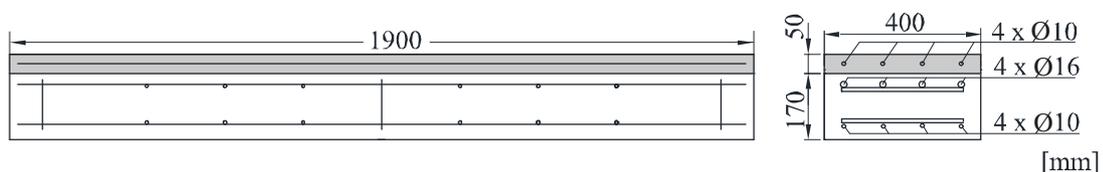


Figure 5: Dimensions and reinforcement layout of composite slab-strip

This slab-like beam was tested in a cantilever test setup illustrated on figure 6. All plates used below the applied force and at the supports were 200 mm wide and distributed the force and the reactions on all the width of the specimen. The end of the beam is considered to be

fully fixed. This was verified using a finite element model with a plane stress elements and elastic materials that allowed verifying the internal efforts in the beam. The obtained moment and shear diagrams are also shown in figure 6. The highest moment is over the roller support and the shear force in the shorter span of 700 mm is 2.14 times the force F . It was thus expected that the failure would occur in flexure over the support or in shear in the short span.

A Linear Variable Differential Transducer (LVDT) was used to measure the deflection of the specimen at the point where force F acts. A load cell was placed between the specimen and the jack to record force F . The test was controlled by actuator displacement.

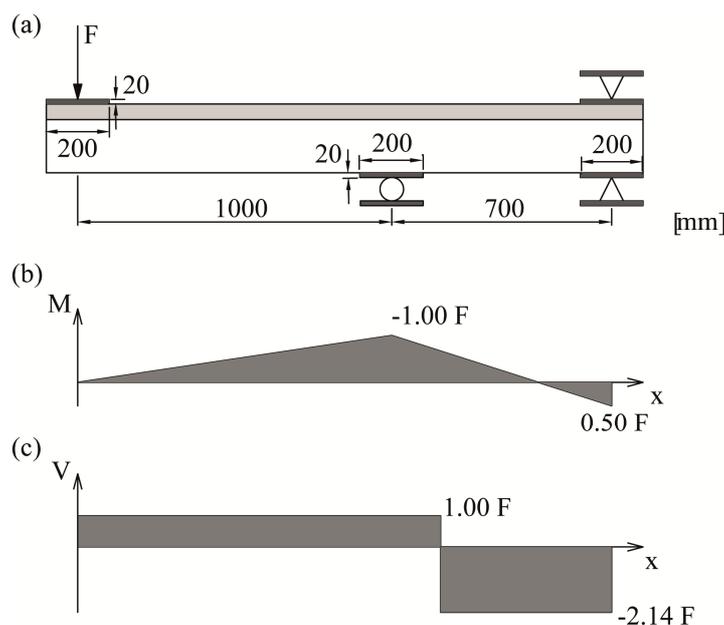


Figure 6: Test setup and static system: (a) position of supports; (b) moment diagram; (c) shear diagram

2.3 Test results

The recorded force-displacement curve is shown in Figure 7. Until maximum force, i.e. ultimate resistance, the specimen behaved monolithically. Over the central roller support, where moment is the highest, vertical flexural cracks were observed in the concrete mainly, spaced at 100 mm. Figure 8 shows the specimen when the force was 80 kN. At 90 kN, which is the maximum recorded force, a diagonal crack appeared in the shorter 700 mm span, where the shear force is highest. It rapidly widened causing the drop in the force-displacement curve and debonding of the UHPFRC layer. Figure 9 shows the fully developed crack pattern at the end of the test which is useful to identify the failure mechanism. Circled are the two plastic hinges created in the UHPFRC layer to resist to the development of the ICD zone. The hinge closer to the central support is a crack that opened from the top of the layer while the second hinge opened from the bottom. This shows that the layer is submitted to double curvature over the ICD zone.

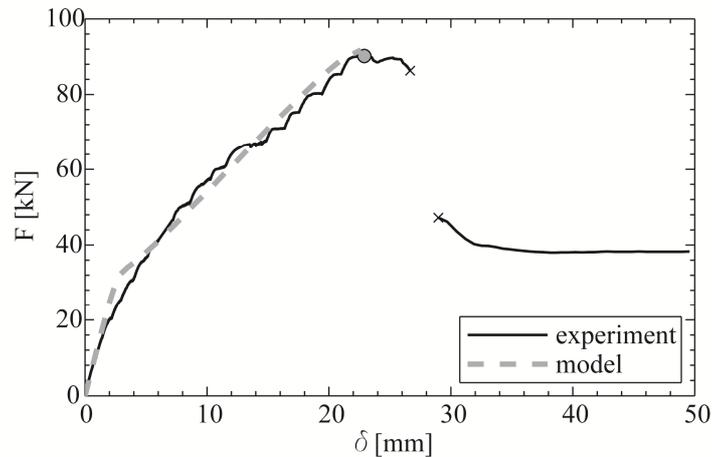


Figure 7: Force-displacement diagram

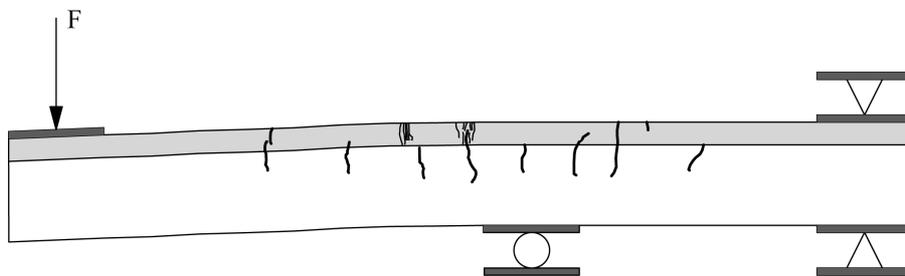


Figure 8: Crack pattern at 80 kN

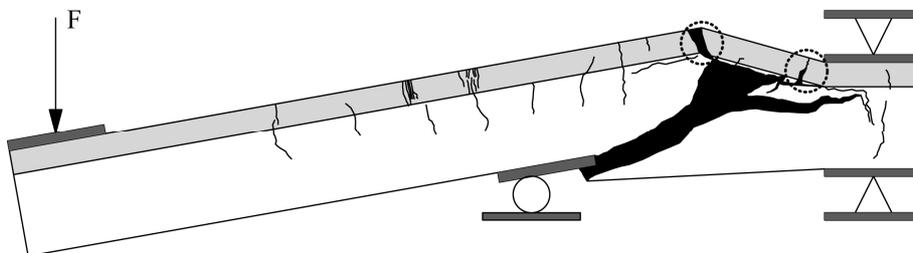


Figure 9: Fully developed crack pattern

3. EXPERIMENTAL RESULTS

3.1 Bending behavior

As shown by the test results, up to peak, the beam mainly presented a bending behavior with the formation of vertical cracks only. No debonding between the UHPFRC and concrete was observed. The composite beams thus behaved monolithically and the hypothesis of Bernoulli - plane sections remain plane - is valid. To calculate the moment-curvature response up to peak, it is proposed to use a plane section analysis as proposed in [2] and illustrated in figure 10. This model can also be used to calculate the ultimate bending resistance of a composite section.

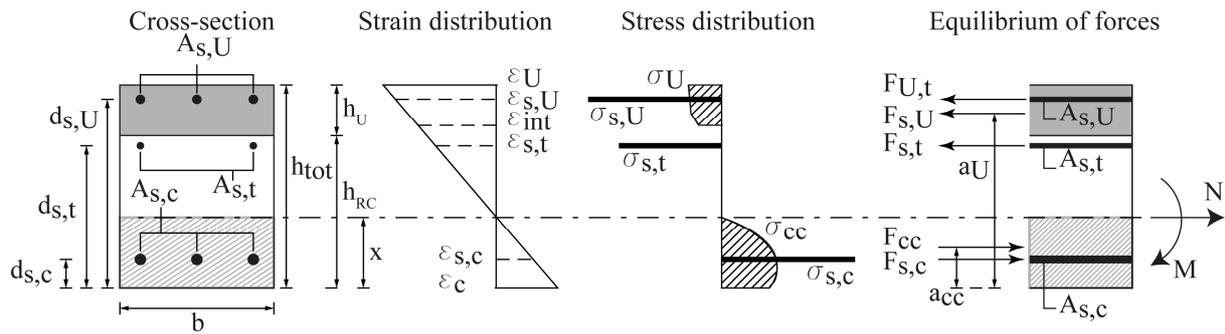


Figure 10: Analytical model for bending [2]

For the specimen presented here, the force-deflection response up to the peak was calculated according to this model. The result of this calculation is presented on the graph of figure 7 with the dashed line in grey. The model is in good agreement with the experimental result up to maximum resistance which in this case happened in flexure shear at 90 kN. According to the model, this beam would have failed in flexion at a force of 106 kN.

3.2 Flexure shear failure

The failure mechanism in flexure shear was presented in figure 4 and illustrated by the fully developed crack pattern of the tested specimen. This mechanism is used in [6] to propose a simplified formulation to evaluate the flexure-shear strength of a composite beam, V_{RU-RC} . This model is illustrated in figure 11.

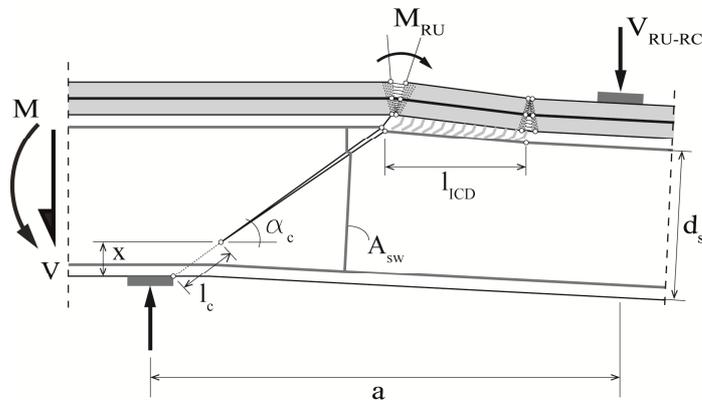


Figure 11: Analytical model for shear [6].

The total shear resistance of the composite member is calculated with equation 1 as the sum of the contribution of concrete (V_c), stirrups in steel (V_s) and UHPFRC layer (V_{RU}).

$$V_{RU-RC} = V_c + V_s + V_{RU} \quad (1)$$

The shear resistance of concrete V_c along the diagonal crack is obtained with equation 2 proposed in [10]. In this equation, f_{ce} is the effective strength of concrete and can be taken as $0,8f_c$. The height of the compression zone, x , is obtained when the sectional analysis is done for the composite beam (see figures 10 and 11). The angle of the diagonal crack with respect

to the longitudinal axis angle is α_c (see figures 4 and 11). The value of α_c should be between 20° and 60° .

$$V_c = \frac{f_{ce} b}{2} \left[\frac{x}{\sin \alpha_c} (1 - \cos \alpha_c) \right] \quad (2)$$

The shear resistance of steel is simply the area of shear reinforcement times the yield limit of the steel, as given by equation 3.

$$V_s = A_{sw} f_{sw} \quad (3)$$

Finally, the shear resistance of the UHPFRC layer submitted to bending in double curvature is a function of its flexural resistance $M_{RU,max}$ and the maximum possible length of the ICD zone $l_{ICD,max}$, as stated by equation 4. The bending resistance of the layer can be calculated with a plane section analysis while the ICD length is obtained geometrically using the angle of the diagonal crack α_c and the span length. The maximum ICD zone develops below the interface between the mouth of the diagonal crack and the top support or force point. It is given by equation 5 where d_s is the static height of the RC section as shown in figure 11.

$$V_{RU} = \frac{2M_{RU,max}}{l_{ICD,max}} \quad (4)$$

$$l_{ICD,max} = a - \frac{d_s}{\tan \alpha_c} \quad (5)$$

3.3 Application to test results

These equations were used to calculate the shear resistance of the specimen presented in this paper. The results of these calculations with the important parameters are given in table 1. Since there are no stirrups in this slab-like specimen, the contribution of steel is 0. In a design case, the angle of the diagonal crack α_c is chosen in order to obtain the smallest value for V_{RU-RC} . When α_c increases, the contribution of concrete, V_c , increases as well, but the length of the ICD zone is also increased which means a lower contribution of the UHPFRC layer to the resistance. For this experimental result, the angle of the critical crack was measured at 25° on the tested specimen.

The calculated value for V_{RU-RC} must be divided by 2.14 as given by the shear diagram in figure 6, in order to compare it with the peak force measured during the test. Thus, according to the model, the force that can be applied at the end of the 1000 mm cantilever to obtain a flexure-shear failure in the shorter 700 mm span is 96 kN. The peak force measured during the test is 90 kN. The simplified calculation presented here was thus able to predict the strength of the beam with an error of 7%.

The shear resistance of the RC section alone was also calculated using equation 2 and the results are given in Table 1. In this case, the chosen angle for the diagonal crack is the smallest angle possible to connect the bottom roller support to the top support at the end of the short span. This should be the preferred path for the compression strut in the RC section with no added external tensile reinforcement. The concrete shear resistance alone is 52 kN or 24 kN if measured at the point load, at the end of the cantilever (figure 6). It is 4 times lower than the resistance of the composite section.

Table 1: Analytical calculation of flexure-shear resistance

	RU-RC	RC
x	65 mm	29 mm
α_c	25°	20°
$M_{RU,max}$	7 kN.m	N/A*
$l_{ICD,max}$	235 mm	N/A
V_c	145 kN	52 kN
V_s	0 kN	0 kN
V_{RU}	60 kN	N/A
V_{RU-RC}	206 kN	N/A

*N/A: Not Applicable

This also illustrates that the UHPFRC layer contributes in two ways. First, by adding tensile reinforcement, the compression zone becomes higher which increases the resistance of the concrete section to shear. During the test, the diagonal crack forms and starts to open at a higher load than for the same member without a UHPFRC layer. Second, the layer of UHPFRC controls the widening of the diagonal crack by resisting to the development of the ICD zone. It then contributes to the shear resistance by its bending capacity.

4. APPLICATION

The 50 year-old drivable reinforced concrete floor of a fire brigade building had insufficient load carrying capacity in view of heavier future fire engines. The concept to increase the structural capacity of the existing slab of 720 m² area was to pour a 40mm thick UHPFRC layer on top of the existing RC slab in replacement of the existing cementitious overlay (figure 12). The UHPFRC layer leads to a thicker load carrying slab providing better distribution of local wheel loads, increase in static height and high strength material to resist both compression or tension stresses.

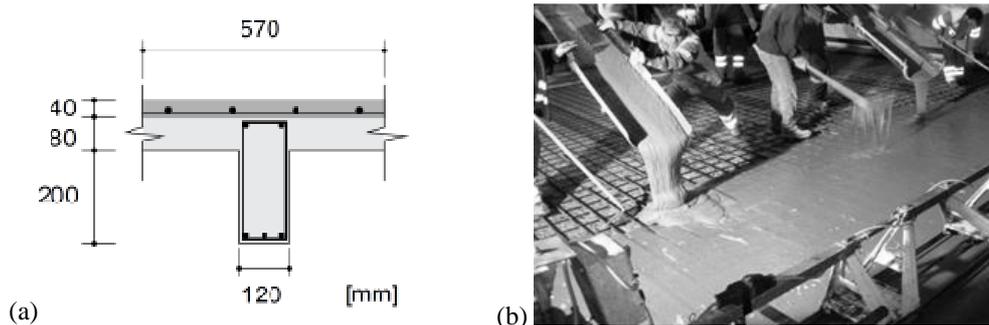


Figure 12: Application case: (a) cross section with R-UHPFRC layer (in dark grey);
 (b) view of UHPFRC casting.

The use of the UHPFRC solution turned out to be very economic (compared to the conventional solution of slab demolition and reconstruction). Additionally, the utilization of the fire workers building was only slightly restricted during the intervention and thus user costs could be kept minimal [1].

5. CONCLUSIONS

A test on a slab-like specimen illustrated the flexure-shear failure mode and how a UHPFRC layer contributes to the shear resistance by its bending capacity. Two main failure modes were identified for composite structures: a flexural mode and a flexure-shear mode.

The analytical model to calculate the shear resistance was applied to an experimental example which proved it to be reliable and simple to use.

It was also shown that a layer of UHPFRC can improve the ultimate resistance of RC members both in bending and in shear.

This is an efficient technique to improve the design of new structures or to strengthen existing structures.

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