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Strengthening and repair of RC beams with fiber reinforced concrete

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ABSTRACT

The use of a jacket made of fiber reinforced concrete with tensile hardening behavior for strengthening RC beams is investigated by means of full-scale tests on 4.55 m long beams. A 40 mm jacket of this material was directly applied to the beam surface. Both the strengthening and the repair of RC beams were studied. In particular, in the latter case the beam was initially damaged and eventually repaired. A numerical analysis is also performed in order to better understand the reinforcement behavior. The experimental and numerical results show the effectiveness of the proposed technique both at ultimate and serviceability limit states.

1. Introduction

The interest for strengthening and repair of RC structures has increased in the last few years. Besides the well known problems of seismic retrofitting, the strengthening of structures can be also required by the degradation of structural materials (due to durability problems) or by an increase in design loads. Moreover, there are important infrastructures, such as bridges or tunnels, which have to be repaired to avoid the social costs related to the demolition and the reconstruction of new structures.

Designers refer usually to traditional strengthening techniques that were based on externally bonded steel plates or RC jacketing [1], while a new solution that has recently gained great favor concerns the use of externally bonded Fiber Reinforced Polymer (FRP) [2–4]. All these techniques can be successfully used but have some limits. In particular, the use of R/C jacket is possible by adding layers of concrete with thickness larger than 60–70 mm due to the presence of rebars that require a minimum concrete cover [5]. The use of externally glued steel plates as well as of FRPs may have problems for fire resistance. Furthermore, the use of these techniques may not satisfy minimum requirements for serviceability limit states.

During the last 10 years, the use of concrete reinforced with fibers has increased due to its enhanced properties after cracking of the cementitious matrix [6,7]. Fiber Reinforced Concrete (FRC) is nowadays extensively used in applications where fiber reinforcement is not essential for the structural safety (e.g. industrial pavements or shotcrete for early stage tunnel linings). Besides these applications, there are new ones where fiber reinforcement is used as total or partial substitution for conventional (rebars or welded mesh) reinforcement [8,9]. In particular, several studies demonstrated that fibers can be used for replacing part of shear reinforcement in beams [10] or transverse reinforcement in thin-web elements [11]. Because of the possibility of stress redistribution, in all these applications a FRC with a post-peak softening behavior in tension can be adopted.

Recently, FRC materials having a hardening behavior in tension, usually named High Performance Fiber Reinforced Concrete (HPFRC), are available for practical use [12–15] and allow newer applications. In fact, by using these materials, it is possible to design structures with new geometries and shapes that are no longer limited by the reinforcement detailing limitations. Both FRC and HPFRC will be included, and then considered as construction materials for new and existing structures, in the coming New *fib* Model Code [16].

A promising application of HPFRC concerns the retrofitting of RC structural elements. The possibility of developing cement-based composite thin sheet for structural retrofit, and its efficiency is analysed, in [17–19].

The use of HPFRC for strengthening or repair existing RC beams is proposed herein. In order to verify the effectiveness of this application, experimental tests on full-scale beams having a length of 4.55 m have been carried out. The proposed technique considers

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the use of a thin HPFRC jacket, having a thickness of 40 mm. Furthermore, numerical analyses have been performed for deepening the knowledge of the main parameters and effects governing the behavior of the HPFRC jacketed beams.

2. Strengthening of RC beams: experimental investigation

The application of the HPFRC jacketing technique was initially aimed at strengthening existing RC beams and was applied to undamaged elements that required a higher stiffness and load-carrying capacity. A typical example is represented by structures that now have to carry loads greater than the original design ones.

2.1. Beam geometry and material properties

Full-scale tests were performed on 4.55 m long beams with a depth of 500 mm and a width of 300 mm, as shown in Fig. 1a. One beam was cast without any reinforcement (neither rebars nor stirrups) while two additional beams were reinforced with two bottom longitudinal rebars (Ø = 16 mm), two top longitudinal rebars (Ø = 12 mm) and, at the beam ends, stirrups having a diameter of 8 mm and a spacing of 150 mm. The ends of the bottom longitudinal-rebars were welded to steel plates in order to guarantee a good anchorage and to avoid any slip during loading.

The beams were cast with a concrete having nominal cylindrical strength of 22 MPa. A low concrete resistance and a low reinforcement percentage (0.3%) were chosen in order to simulate the real case of a weak existing beam and to better highlight the strengthening effectiveness.

One of the reinforced beams was used as the reference specimen while a 40 mm thick jacket of HPFRC was applied to the other beams, with and without internal steel reinforcement, as shown in Fig. 1b. The un-reinforced beam (without rebars) strengthened with HPFRC was tested in order to verify the effect of the jacket only on the concrete beam, i.e. to check the behavior of the composite structure, in terms of stiffness and crack formation, without the influence of the steel reinforcement.

The strengthening material is a self-leveling mortar having a maximum aggregate size of 1.3 mm and water/binder (cement + microsilica) ratio equal to 0.17 by weight. The mortar is reinforced with 2.5% (by volume) of steel micro-fibers having a length of 12 mm and a diameter of 0.18 mm. The advantage of this material is that it can be adopted with a curing at ambient temperature and humidity. The compressive strength of the HPFRC, as measured on 100 mm side cubes, after 28 days of curing, was 177 MPa.

Direct tensile test on dog-bone specimens and four point bending tests on small beams were performed in order to characterize the material in tension. The experimental results, together with the specimen geometries, are reported in Fig. 2. As highlighted from the uniaxial tensile test, the material is characterized by a strain–hardening behavior in tension.

2.2. Jacket application technology

A preliminary investigation was carried out in order to define the procedure for the HPFRC strengthening layer application. Particular attention was devoted to the control of the adhesion between the base concrete and the new material. To this aim, a first series of tests was performed on 150 × 150 × 600 mm specimens, made with the same concrete used for the full-scale beams. After sandblasting of the surface, a layer of 40 mm of HPFRC material was cast. The adhesion between existing and strengthening concrete was verified by performing preliminary four point bending tests on the specimens (LVDTs were placed between the old and new concretes for measuring the relative displacements, see Fig. 3a): Experimental evidences show that no slip occurs between the two materials, up to the failure in bending [20].

After this initial phase, the technology of the HPFRC strengthening was defined. The surfaces of the full-scale beams were sandblasted in order to produce a 1–2 mm roughness, that was considered enough to avoid the use of bonding products (Fig. 3b). The HPFRC material was directly cast on the beam, without any vibration (Fig. 3b). Since curing was carried out at ambient temperature and humidity, a plastic sheet was placed on the surface in order to limit water evaporation. The HPFRC jacket was applied 3 months after casting the normal concrete beams while the tests were performed 28 days after the HPFRC jacketing.

2.3. Testing procedures

The full-scale beams were tested under flexure with a four point bending scheme. The beams were placed on a 4.35 m

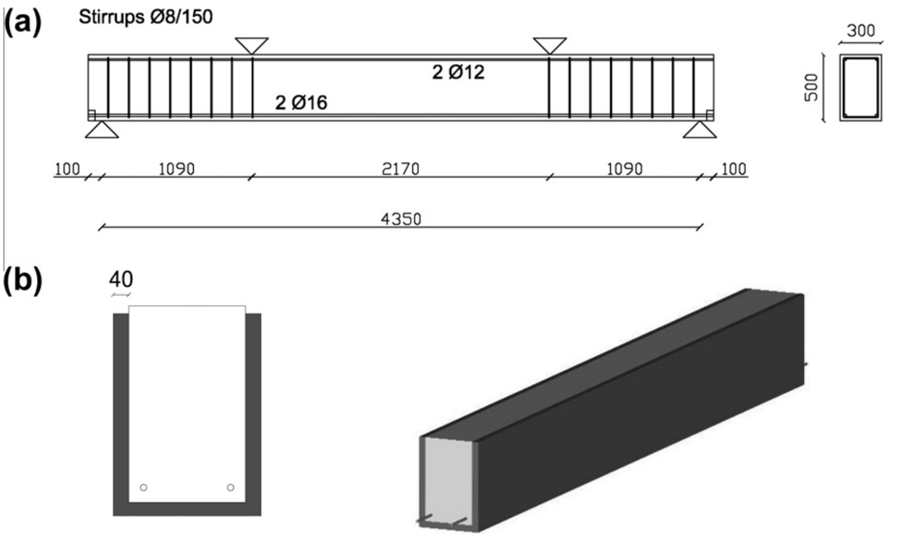

Fig. 1. Geometry of the specimens (a) and strengthening scheme (b).

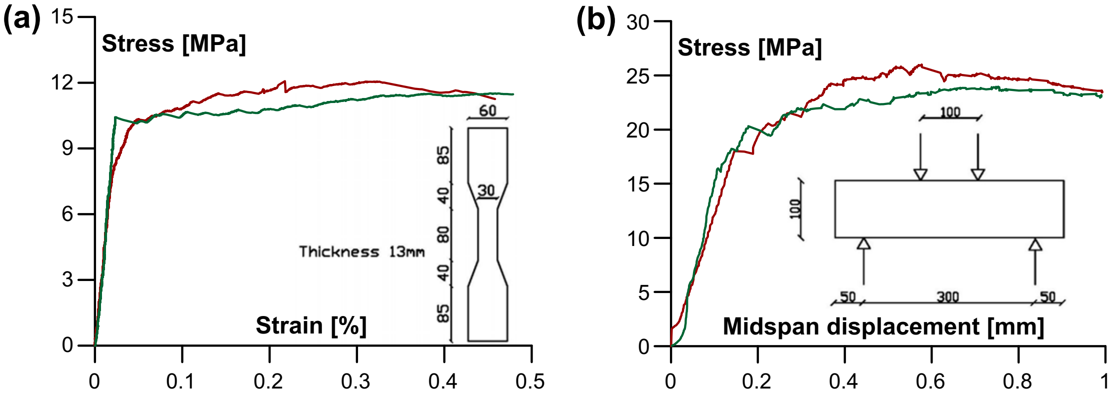

Fig. 2. HPFRC characterization: (a) direct tensile test on dog-bone specimen; (b) flexural test (100 × 100 mm cross-section).

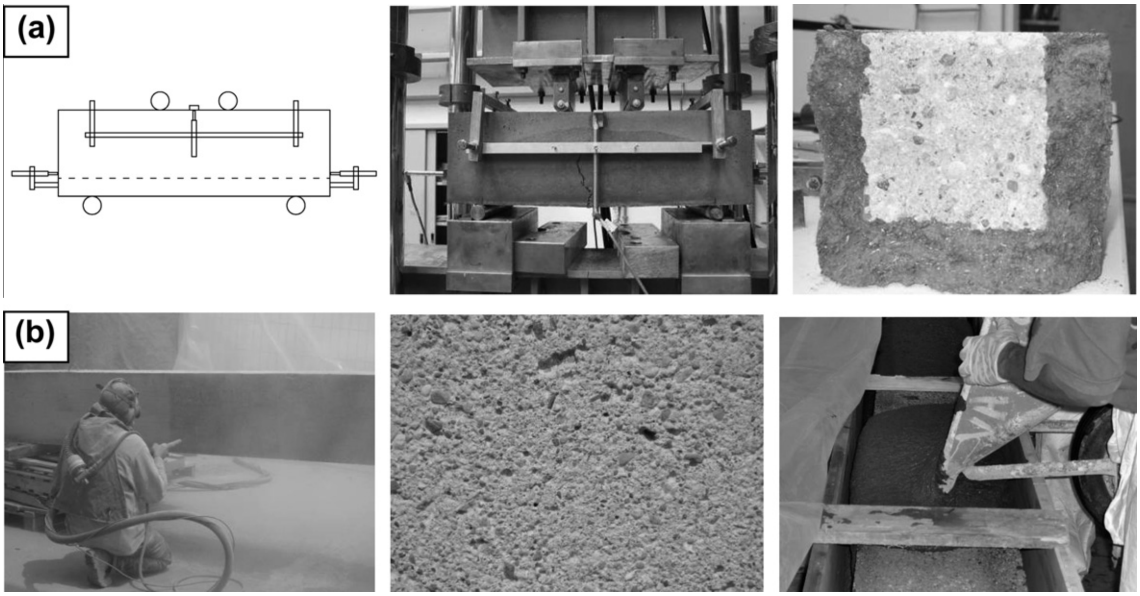

Fig. 3. Definition of the proposed technique: (a) preliminary tests for the evaluation of the adhesion: scheme of the LVDTs position, small composite beam under tests, beam after the test; (b) surface preparation of the full-scale beams, roughness of the surface and casting of the HPFRC jacket after sandblasting.

span and loaded at two points located at a distance of 1.09 m from the supports (shear span equal to 2.4), as shown in Figs. 1 and 4.

The tests were performed by imposing a constant displacement rate and adopting a 1000 kN electromechanical screw jack with a PID close loop control. Potentiometers and LVDTs were adopted for measuring the vertical displacement and the horizontal deformation (that includes crack opening), as described in Fig. 4.

2.4. Results

Firstly, the test on the reference beam (RC without the HPFRC strengthening jacket) was carried out. Fig. 5a shows the curve of the total applied load (including the self weight of the loading frame) versus the mid-span displacement.

The behavior of normal RC beams is well known, but some remarks have to be stressed for the following discussions. The main experimental observations are the following:

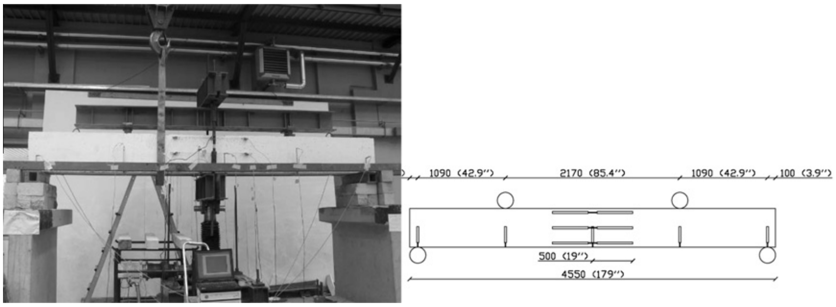

Fig. 4. Full-scale beam test set-up, and transducers positions.

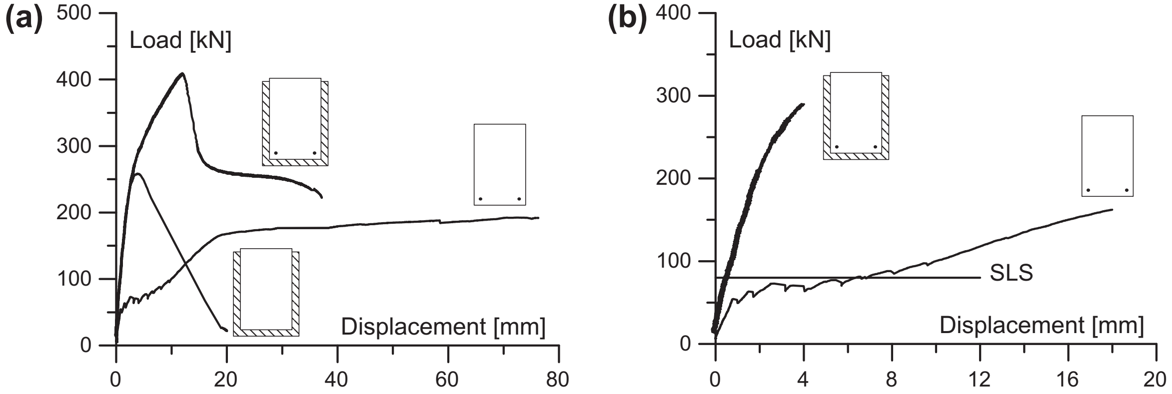

Fig. 5. Comparison of the load–displacement curves obtained from strengthened beams: (a) complete curves; (b) and initial part of the curves (for serviceability limit state).

- at a load level of about 50 kN, flexural bending cracks developed in the zone between the two point loads, with a crack spacing ranging between 300 and 400 mm (the crack depth was around 430 mm with the thickness of the compressive chord of about 70 mm);
- afterwards, from every crack, at reinforcement level, a splitting crack developed and led to a loss of bond between longitudinal bars and concrete (Fig. 6), this is not usual effect and it is due to the low reinforcement ratio and the poor quality of the concrete;
- the load still increased with a lower stiffness till the reinforcement yielded (Fig. 5), with a load equal to 190 kN;
- due to the bond failure, the stiffness of the beam remarkably decreased and large deformations occurred;
- horizontal cracks developed from the upper compressive chord, defining an arch mechanism geometry (Fig. 6a).

The crack pattern at failure is shown in Fig. 6a. Other details can be found in Martinola et al. [20].

The second specimen tested was the beam without the longitudinal and transverse reinforcement, strengthened with the HPFRC jacket. The load versus mid-span displacement curve is shown in Fig. 5a. A brittle collapse occurred with a load equal to 258 kN and a single main crack developed close to the mid-span, as shown in Fig. 6b.

Finally, the RC beam strengthened with the HPFRC jacket was tested and the experimental results are shown in Fig. 5. Because of the presence of the HPFRC jacket, the beam exhibited the same behavior of the second beam (without rebars) up to a load level of 250 kN. Eventually, due to the presence of rebars, the load continued increasing up to 410 kN, with a slight stiffness reduction due to the development of cracks with a spacing of 500–600 mm.

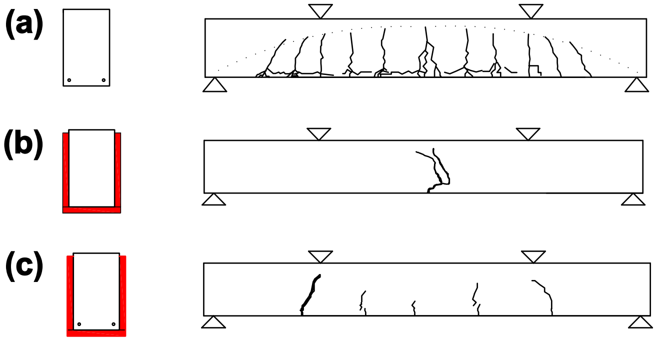

Fig. 6. Crack pattern at failure: (a) RC beam without HPFRC jacket; (b) beam without steel reinforcement with HPFRC jacket; (c) RC beam with HPFRC jacket.

After reaching the maximum value of 410 kN, the load decreased and stabilized at, approximately, 260 kN with a localization of a single crack under one of the point loads (Fig. 6c).

The experimental behavior evidences the three stages characterizing the structural response: the elastic stage, the second stage whose stiffness is governed by multiple cracking in the fiber reinforcement, the third softening branch, whose slope is due to the macro-crack localization and the residual strength.

The collapse occurred due to the tensile rupture of the longitudinal reinforcement, with the final crack pattern shown in Fig. 6c.

2.5. Discussion

The comparison of the results of the three tested beams is illustrated in Fig. 5. When focusing the attention on the two RC beams (with and without strengthening jacket), it can be noticed that the HPFRC jacket leads to an increase in the structural capacity of the beam (2.15 times), even if the post-peak behavior shows considerable softening. Due to the jacket, at the end of the softening branch, the load stabilized with a horizontal branch higher than that obtained in the RC beam without jacket. It should be underlined that the maximum load of the beam with the HPFRC jacket and without conventional reinforcement is higher than that exhibited by the RC beam without the HPFRC jacket. As far as the ductility is concerned, it is worth noting a decrease of the ultimate displacement in the strengthened beams, even if the apparent very large ductility of the unstrengthened element is due, at least in part, to a loss of bond. Anyway, if an increase of ductility is also required to the strengthened beam, a suitable steel reinforcement in meshes or small diameter rebars could be embedded in the jacket [21,22].

Another key-point of the proposed technique concerns service conditions. In fact, building codes require to verify the strengthened structure both at the ultimate limit state (i.e. increase of the bearing capacity) and at the serviceability limit state (i.e. deflection and crack opening control). From this point of view, the proposed technique produces a remarkable increase of the beam stiffness, with a behavior similar to the uncracked stage in the un-reinforced beam (Fig. 5b). Indeed, the HPFRC jacket limits the development of macro-cracks with evident advantages in terms of stiffness and durability, also considering the very low water/cement ratio of the high performance matrix. By assuming a service load of 80 kN, that corresponds to a maximum tensile stress in the rebars of about 250 MPa [23], the use of the HPFRC jacket leads to a decrease of the mid-span displacement from 6 mm to 0.5 mm (i.e. up to 12 times smaller; Fig. 5b), without visible cracking.

On the contrary, at this load stage, the crack pattern in the RC beam is completely developed, as described above.

The main experimental results are summarized in Table 1.

Table 1
Experimental results.

Beam	Maximum load (kN)	Residual load (kN)	Peak displacement (mm)	Residual displacement (mm)	SLS displacement (mm)
R/C	190	–	80	–	6
Strengthened without steel rebars	258	–	4	–	0.5
R/C strengthened with jacket	410	260	12	38	0.5
R/C repaired with jacket	365	270	12	43	0.8

3. Strengthening of RC beams: numerical investigation

The experimental behavior of the reinforced beams has been simulated with nonlinear FE analyses with the aim of a better understanding of the mechanisms governing the flexural response of the analysed reinforced beams. The main parameters affecting the local and global behavior are highlighted in order to optimize the proposed strengthening technique.

The numerical analyses have been performed with the FEM program Diana 9 [22]. Preliminary studies on the RC beam without HPFRC jacket were carried out in order to test the program and to characterize the concrete and steel properties. The concrete in compression has been simulated by means of the Thorenfeldt curve [24,25], with a cylindrical stress of 20 MPa, a Young's modulus of 28 GPa and a tensile peak stress of 2.36 MPa. The tensile behavior of concrete is simulated with a "multi-linear total strain rotating crack model" that defines the pre-peak behavior as a linear stress–strain relationship, and the post-peak response in terms of linear stress-crack opening law, up to a maximum crack opening of 0.05 mm. The stress–strain relationship is evaluated along the principal strain directions and rotates with them.

The steel behavior is characterized by an elastic–hardening relationship, whose parameters, (yielding stress ≈ 560 MPa, tensile strength ≈ 680 MPa, ultimate strain ≈ 10%) have been experimentally determined from the steel rebars used in the tests.

A 3D model with iso-parametric 20-nodes brick-elements was adopted. Both the longitudinal and transverse reinforcement were included in the model, by assuming a perfect bond between steel and concrete; this assumption can be accepted for the tested beam, since the mechanical response was mainly governed by the flexural behavior while the bond loss, occurred for high values of the applied load, leads to a slight decrease of post-yielding stiffness. As a consequence, in the numerical model, smaller values of the displacement, with respect to the experimental ones, are expected due to the bond loss. Furthermore, it is worth remarking that, for the sake of equilibrium, the flexural ultimate load is almost equal to the ultimate force related to the formation of an arch mechanism (that occurred in the tested beam).

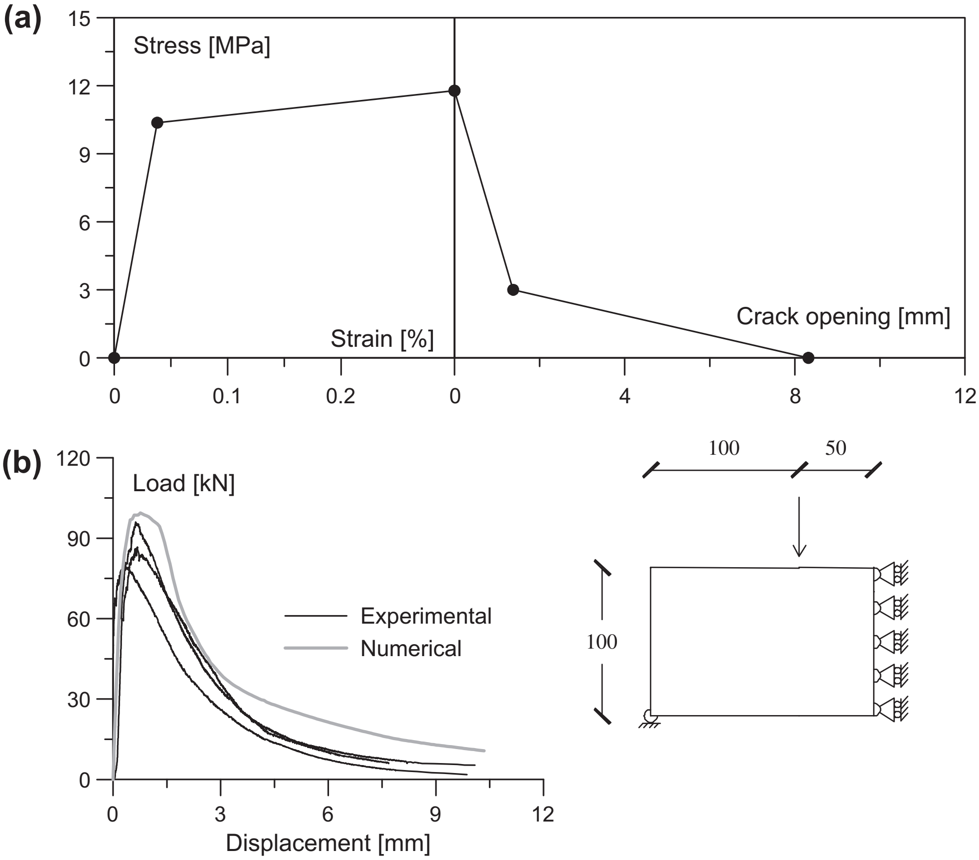

Fig. 7. Numerical definition of the tensile behavior: (a) modeling law of the HPFRC material; (b) numerical versus experimental results for the four point bending beam (100 × 100 × 400 mm).

The numerical model for the analysis of the reinforced beam requires particular care for the definition of the constitutive parameters characterizing the tensile response of the HPFRC.

The behavior in compression of HPFRC is again simulated with the Thorenfeld curve [24,25], with a maximum stress equal to 177 MPa and a Young's modulus of 44 GPa.

The tensile properties (cracking stress and strain, peak stress and strain, and maximum crack opening) of HPFRC (characterized by hardening behavior) were experimentally determined from uniaxial tensile tests on dog-bone specimens. A stress–strain relationship before cracking and a stress-crack opening law after cracking (Fig. 7a) was adopted; this constitutive law was validated by simulating with FE analyses the HPFRC beams (100 × 100 × 400 mm) tested in flexure (Fig. 2b). The good agreement between the experimental and numerical outcomes, obtained by adopting the same relationship for thin (dog-bone) and thicker (beam) specimens, highlights the negligible effect of the fiber orientation [26].

Once the HPFRC constitutive relationships were defined, the full-scale beam was numerically simulated. Based on the preliminary experimental results obtained from small beams, a perfect bond was assumed at the interfaces between concrete and HPFRC.

Due to the symmetry of the scheme, only half of the beam is considered (Fig. 7b). Second order iso-parametric brick elements with 20 nodes and a size of about 250 × 200 × 200 mm are adopted. The numerical results are depicted in Fig. 7b and, when compared with the experimental ones, show a very satisfactory agreement.

Fig. 8 shows the mesh size and the numerical results obtained with the FE analyses; it can be noted that the elastic stiffness of the reinforced beam without jacketing is perfectly captured by the model. The small differences in the second stage are probably related to the presence of splitting cracks (along the rebars), present in the experiment but not observed in the numerical analyses because of the assumed hypotheses of perfect bond between concrete and the steel rebar. The analysis of the crack pattern at collapse further confirms the effectiveness of the numerical model; as in the experimental results, the cracks appear close to the tensile steel rebars, with multiple cracks in the shear span and a wide inclined crack close to the point load.

As far as the beam with the HPFRC jacket is concerned (Fig. 8b), the numerical load–displacement curve compared with the experimental one exhibits a significant difference in the second and third stages. This phenomenon could be related to a kind of "notch-effect" due to the development of macro-cracks in the ordinary concrete that can induce a premature strain localization in the HPFRC material [27–29]. As a matter of fact, even if the fiber reinforcement delays the macro-cracks formation and propagation, the contact with a more brittle material where the strains tend to concentrate, anticipates the development of macro-cracks. On the basis of this assumption, the behavior of the fiber reinforcement in tension has been modified [18], shifting the softening branch in order to reduce the hardening phase, as shown in Fig. 9. The adoption of the new constitutive relationship provides the global response of the beam depicted in Fig. 10a, which appears in good agreement with the experimental results. In particular, the model catches well the main phases characterizing the structural response and the numerical final crack pattern is similar to the experimental one (Fig. 10b).

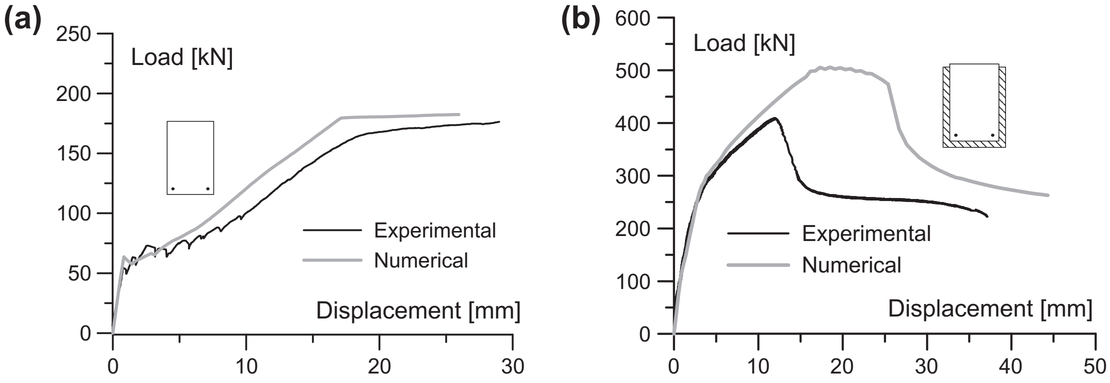

Fig. 8. Numerical result of the beam without jacketing (a) and with the HPFRC jacket (b).

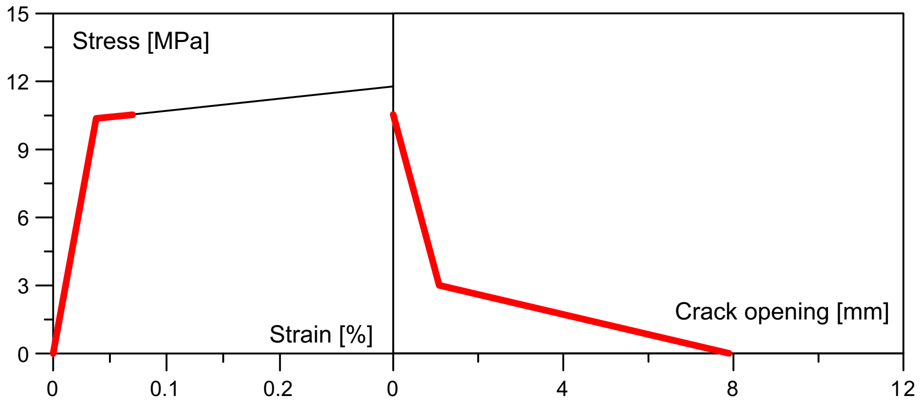

Fig. 9. Modeling low with the reduced hardening branch.

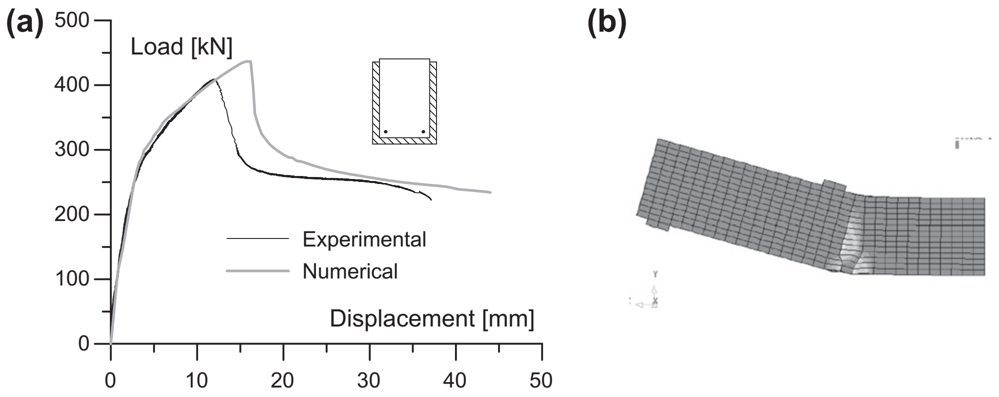

Fig. 10. Numerical study with the reduced hardening branch: comparison between numerical and experimental results (a) and FE mesh (b).

4. Repair of existing RC beams

The HPFRC jacket technique was also tested for the repair of damaged RC beams.

A reinforced concrete beam totally similar to the beam shown in Fig. 1, cast with the same concrete and reinforced with the same rebars, was initially damaged up to the yielding of the longitudinal reinforcement, as it may happen in old structures due to an increase of the service load. Eventually, the beam was repaired with a HPFRC jacket.

Fig. 11a shows the load versus displacement curve in the damaging phase: it can be noticed that the first cracking load (70 kN) and the yielding load (195 kN) are similar to the previously tested beams. When the yielding in the reinforcement was reached, the beam was unloaded and the residual mid-span displacement was equal to 7 mm. The cracks spacing, similarly to the case of RC beam without HPFRC jacket, was about 300–400 mm while the maximum crack opening was about 0.4 mm.

The damaged beam was repaired by directly pouring the HPFRC; it should be noticed that the beam surface was sandblasted but the existing cracks were not repaired, as it usually occurs in practice.

In Fig. 11a the test results from the repaired beam are also reported, in terms of load versus mid-span displacement curve. The behavior is similar to the response of the strengthened beam: it is characterized by an initial branch up to a load of 250 kN, followed by a branch with a decreasing stiffness until the maximum load is reached (365 kN). The initial stiffness of the repaired beam is slightly lower than the strengthened one (Fig. 12), while no significant differences are observed for the post-yielding stiffness, up to the peak stress (see Fig. 12, in which the two curves are almost parallel in this phase).

It can be noticed (Fig. 12) that the use of HPFRC for the repair of a damaged RC beam allows an increase of its bearing capacity of 1.90 times. This performance is slightly lower with respect to the strengthened beam due to the presence of larger cracks (about 0.1 mm after unloading) in the damaged beam that were not repaired (as already said the jacket was applied directly on the damaged beam). This choice was due to the interest in evaluating the jacket effect in a limit case; probably, a preparation on the damaged beam by chiseling to open cracks can help with the penetration of the materials between cracked surfaces.

The post-peak behavior and the collapse mechanism (Fig. 12b) are similar to those observed in the strengthened beam (Figs. 5 and 12), such as the crack pattern at failure (Fig. 11b), characterized by a crack spacing of about 400–500 mm and a final strain localization with a subsequent higher opening of one crack close to the point load.

As far as serviceability limit state (SLS) is concerned, the proposed technique allows for a remarkable increase in the beam stiffness (Fig. 12) also for the repair of structural members. As a matter of fact, for a service load of about 80 kN, evaluated as described in a previous paragraph, the use of the HPFRC jacket on a damaged beam leads to a mid-span displacement up to 12 times smaller with respect to the RC beam (Fig. 12c), without any visible cracking on the jacket.

The deformability reduction and the following delay in the cracks formation and opening contribute to a durability improvement of the structural element. The main experimental results are compared with the previous ones, related to normal RC and strengthened beams, in Table 1.

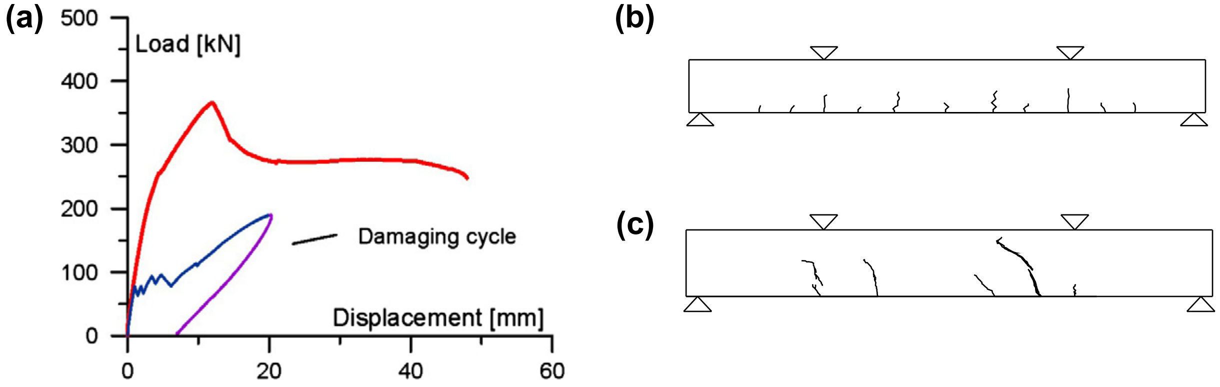

Fig. 11. Repaired beam: (a) damaging cycle of the RC beam, and load–displacement curve of the beam repaired with the HPFRC jacket; (b) crack pattern at failure of the repaired beam.

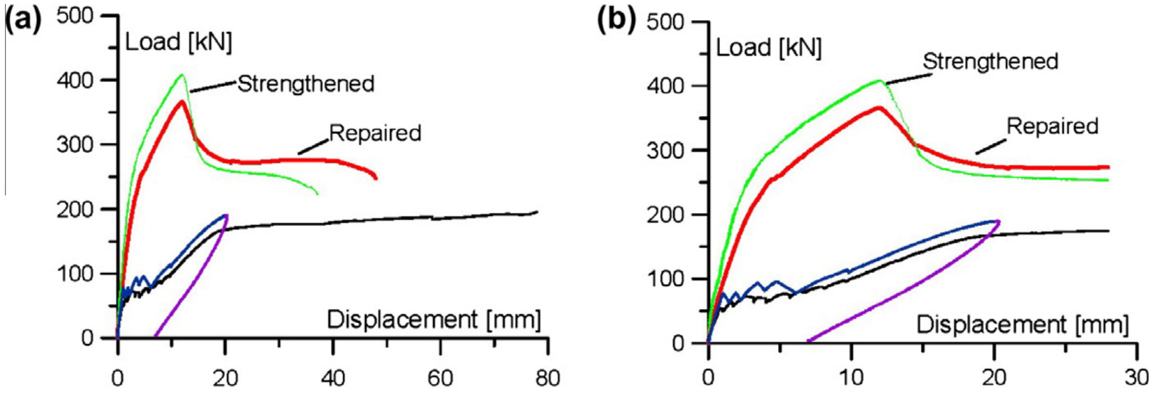

Fig. 12. Comparison of the load–displacement results: (a) complete curves; (b) initial part of the curves (for serviceability limit state).

5. Concluding remarks

This paper provides information on structural behavior of strengthened and repaired beams by using a thin layer of High Performance Fiber Reinforced Concrete (HPFRC).

Experimental and numerical results demonstrated that this appears a promising technique.

On the base of the results discussed in the paper, the following remarks can be drawn:

- the application of a 40 mm thick HPFRC jacket on a RC beam provides an increase of the ultimate load of 2.15 times; referring to a pre-damaged beam, this increase is about 1.90 times. The experimental and numerical results point out the effectiveness of the proposed technique in improving the bearing capacity, in both the cases of strengthening and repair;
- the proposed technique provides a significant structural enhancement at the serviceability limit state; due to the remarkably increase of the beam stiffness under service load, the mid-span displacement can be remarkably reduced. As a matter of fact, the jacked acts like a sort of external prestressing, by keeping the initial uncracked stiffness of the element.
- The proposed numerical models, validated by means of the comparison with the experimental results, provide a valuable tool for designing the strengthening or the repair of existing RC beams, but particular care has to be devoted to the simulation of the localization of strain and/or notch effects that can arise when two concrete materials (the base and the HPFRC external jacket) with different stiffness values, work together.
- The technology of the HPFRC application is relatively simple: a curing at ambient temperature and humidity is sufficient to allow the development of the strength characteristics of the HPFRC; due to the self-leveling property, the material can be cast in a thin layer, and a normal sandblasting of the beam surface ensures a good adhesion of the jacket without using any primer.

Finally, it is worth mentioning the possibility of increasing the durability of the structure by applying the HPFRC jacket, due to the reduced crack openings and to the compactness of the HPFRC matrix.

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