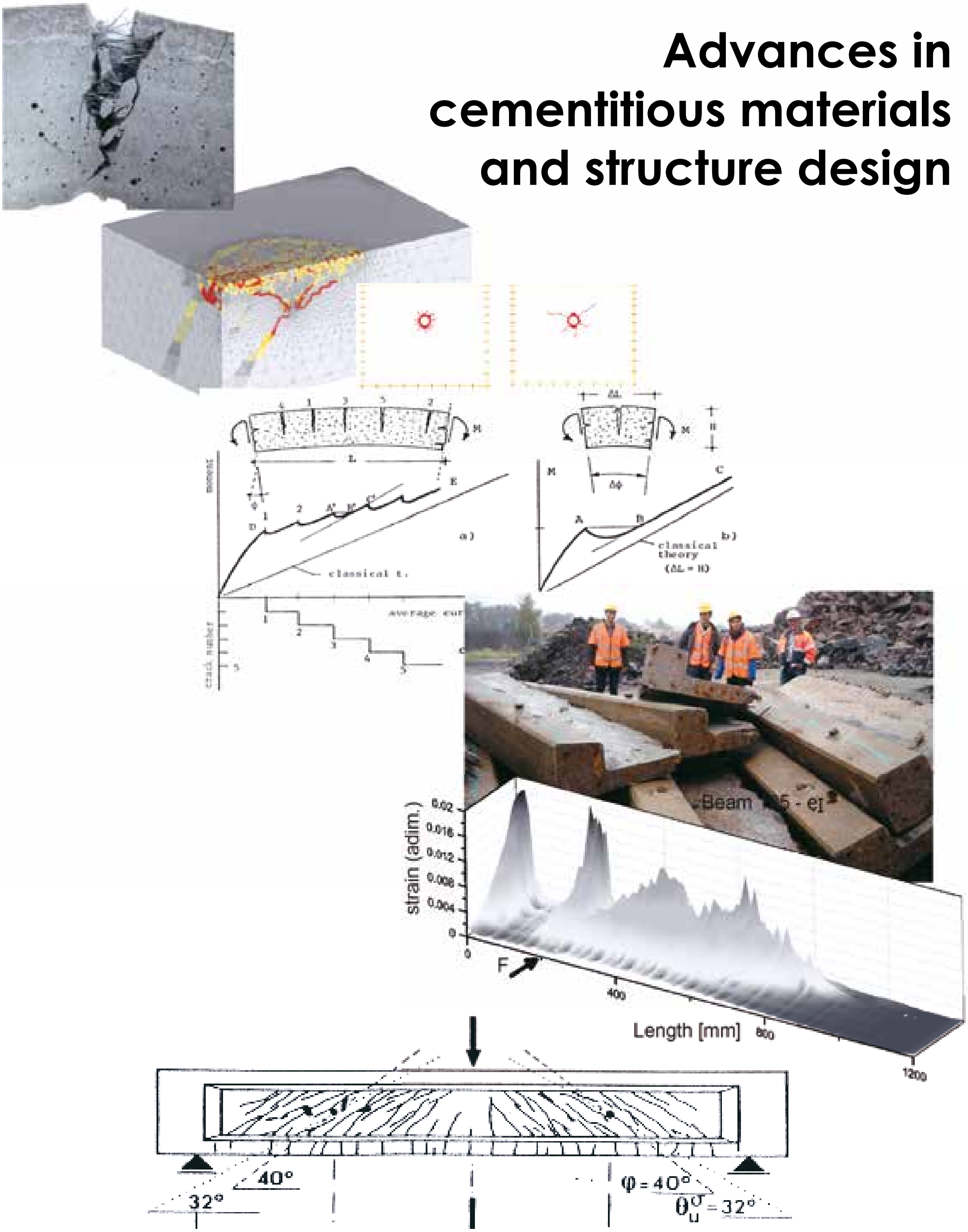
Advances in cementitious materials and structure design



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An application of high performance jacketing for the shear strengthening of RC beams

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ABSTRACT: The possibility of strengthening RC elements for increasing the bearing capacity under shear actions is an important issue in the retrofitting field. In RC existing structures, made in the '60s and '70s, the shear reinforcement is often not sufficient to satisfy the prescription of current codes. Hence, in the retrofitting of these structures it is often necessary to increase the shear carrying capacity. The possible use of thin high-performance jackets aimed at improving the shear capacity of existing R/C members is investigated in this paper. Full scale tests have been performed on 2.85 m beams, lacking stirrups. The jackets are made with a high performance fiber reinforced concrete (HPFRC), with or without an additional 2 mm diameter steel-wire mesh. A numerical analysis with a commercial program (DIANA) is also performed in order to better understand the behaviour of the strengthened beams.

1 INTRODUCTION

In recent years the strengthening and the repair of existing RC structures have been among the most important challenges in civil engineering. The primary reasons for strengthening of structures include: upgrading of resistance to withstand underestimated loads; increasing the load-carrying capacity for higher permit loads; meet the requirements in seismic areas where older construction were not designed for earthquake actions; restoring lost load carrying capacity due to corrosion or other types of degradation caused by aging, etc. In this field, the possibility of increasing the bearing capacity of RC elements under shear actions is of great interest, due to a high probability of having structures lacking in shear reinforcement, typically when build in the '60 and '70.

Different technique have been developed to retrofit a variety of structural deficiencies. The techniques traditionally used, based on externally bonded steel plates or RC jacketing, have many limitation. In particular, the use of R/C jackets is possible by adding layers of concrete with thickness larger than 60–70 mm (Fib Bulletin 24, 2003) due to the presence of rebars requiring a minimum concrete cover, thus leading to an excessive increase of the section geometry. The main disadvantage of using steel plates is corrosion of steel which adversely affects the bond at the steel concrete interface. A solution that has recently gained favour concerns the use of externally bonded Fiber Reinforced Polymers (FRP). Unfortunately, this techniques may not satisfy minimum requirements for serviceability limit states and it could present problems for fire resistance.

Recently, FRC materials having a hardening behavior in tension, usually named High Performance Fiber Reinforced Concrete (HPFRC), are available for practical use 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. One of the most promising areas of application of this material is in the repair of concrete structures. The HPFRC materials offer the possibility of developing cement-based composite thin sheet for structural retrofit and are suitable for repairing concrete structures due to their compatible mechanical and physical properties.

The possibility of remarkably reducing the jacket thickness has been recently investigated by using high performance fiber reinforced concrete jackets (Martinola et al. 2010; Habel et al. 2007; Alaee and Karihaloo, 2003; Kuneida et al. 2010, Beschi et al, 2011a and b). The use of high performance fiber reinforced concrete jackets combined with high performance steel mesh has also been investigated (Marini and Meda, 2009).

In this research, the attention has been devoted to the effects of high performance jackets with reduced thickness on the increase of the carrying capacity under shear actions. The use of two kinds of high performance fiber reinforced concrete has been considered: a material having an almost self levelling rheology, able to cast very thin jackets, and a newly developed thixotropic material which can be easily used to create the jacket. The strengthening HPFRC jackets were cast after sandblasting of the existing concrete surface and, due to the high bond properties of the material, it was not necessary to apply any primer in order to ensure a perfect adhesion of the jacket to the original concrete.

Full-scale tests on 2.85 m long beams, lacking stirrups, have been performed under a four point bending configuration in order to verified the performances of the proposed solutions.

The experimental behaviour of the reinforced beams has been simulated by means of numerical FE analysis for deepening the knowledge of the main parameters and effects governing the behaviour of the HPFRC jacketed beams. The HPFRC constitutive law was determined by performing inverse analysis from uniaxial tensile and bending tests.

2 SPECIMEN PREPARATION

2.1 *Specimen geometry*

The effectiveness of shear reinforcement technique, based on HPFRC jacket application, was studied through five beam specimens having a length of 2.85 m and rectangular transverse section characterized by a with of 200 mm and a depth of 500 mm, as shown in Figure 1. The beams have been reinforced with longitudinal bottom rebars only, made with 4 steel bars having diameter equal to 20 mm, with a 30 mm net cover. The reinforcement ratio results equal to 1.50%. The beams were designed in order to have a shear failure under the chosen load configuration. Neither stirrups nor inclined reinforcement are present. At the ends of the beams, the longitudinal reinforcement were welded to two steel plates, in order to avoid bond slip and prevent sliding of the bars.

The beams were cast with a low strength concrete that is typical of structures built in the 60's and 70's: the average compressive strength, measured on 150 mm side cubes, is equal to 33 MPa. According to Eurocode 2 the concrete can be classified as a C20/25 material. Regarding the reinforcement, the steel used for the reinforcements was B500C and the rebars exhibited an average yield strength equal to 520 MPa and an average maximum strength equal to 616.21 MPa. One beam was used as reference specimen while the other four beams were strengthened by applying a high performance jacket as described in Table 1. For the beams SB, SC and SD, a wire mesh U-bent was placed within the thickness of the jacket, for the beam SA the wire mesh is not present because the small thickness of the jacket .

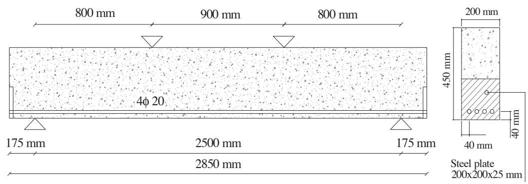


Figure 1. Beams for the tests

2.2 *Strengthening materials*

For strengthening the beams, two different types of concrete were used: a material having an almost self levelling rheology, that should be cast in moulds, and a recently developed thixotropic material that was spread directly on the beam surface.

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 results of the tests together with the specimen geometries are reported in Figure 2. The results of the tests show the strain hardening behaviour in tension of the two material.

The average characteristics of this two materials are reported in Table 2, it can be noticed that the self levelling material presents a higher strength value.

Both of this strengthening materials are reinforced with a straight steel fibers having a length of 15 mm and a diameter of 0.175 mm. The content by volume is 0.2% and the aspect ratio (l/Ø) is 85.7.

For the strengthened beam, a wire mesh was placed within the jacket thickness.

Table 1. Characteristics of the jacket

		Thickness	**Material**	**Bond properties**	**Type Mesh**
Beam SA	bottom surface	30 mm	self levelling	no primer	no mesh
	lateral surfaces	30 mm	self levelling	no primer	
Beam SB	bottom surface	50 mm	self levelling	no primer	U-shaped welded-wire mesh
	lateral surfaces	50 mm	self levelling	no primer	
Beam SC	bottom surface	50 mm	self levelling	no primer	U-shaped welded-wire mesh
	lateral surfaces	50 mm	thixotropic	Epoxy primer	
Beam SD	bottom surface	50 mm	self levelling	no primer	U-shaped welded-wire mesh extended for 20 cm above the bottom side
	lateral surfaces	30 mm	thixotropic	no primer	

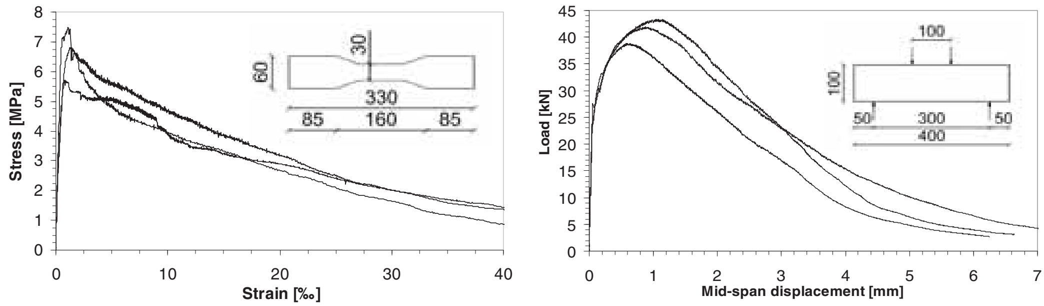


Figure 2. Material characterization: a) direct tensile test on dog-bone specimen (thickness=13mm); b) flexural test (100x100 mm cross-section).

This mesh is made of 2.05 mm diameter bent wires, assembled with a spacing of 25.4 mm. The results of the tensile tests performed on single wires show a maximum strength equal to 550 N/mm^2.

Table 2. Properties of the HPFRCs.

Material	f_{cc} [N/mm^2]	f_{ct} [N/mm^2]	E_c [N/mm^2]
self levelling	130	6	38000
thixotropic	90	5	33000

2.3 *Jacket application technology*

The beams were first sandblasted in order to obtain a roughness of about 1 mm, so as to ensure perfect bond between the existing concrete and the applied high performance concrete. This technique has been demonstrated effective in previous research (Martinola et al. 2010). Figure 3 shows the sandblasting of the beams and, in detail the difference of the surfaces before and after the treatment.

The HPFRC material was prepared in a vertical axis mixers, the self levelling concrete was cast in moulds without vibration, while the thixotropic material was applied directly on the side surfaces of the beams by spreading (Fig. 4). Since curing was carried out at ambient temperature and humidity, a plastic sheet was placed on the surfaces 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.

Figure 3. Sandblasting (left-hand); and surfaces before and after sandlasting (right-hand).

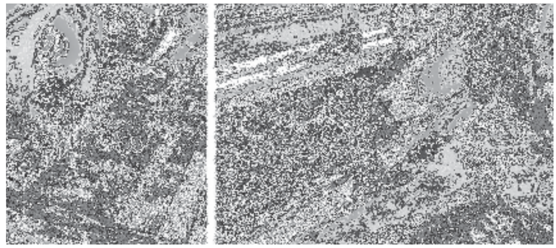


Figure 4. Casting of the jackets: self-leveling concrete (left - hand); and thixotropic concrete (right-hand).Copying old text onto new file

3 TEST SET UP

The beams were tested under a four point bending configuration, by adopting a steel reacting frame (Fig. 5). The load was applied by means of an electromechanical jack having a loading capacity of 1000 kN with a close loop control system. The tests were conducted by imposing the displacement of the actuator with a constant speed equal to 0.01 mm/sec. The beams were placed on roller steel supports with a span of 2.5 m. A steel beam was adopted in order to apply the load at two points having a distance of 0.9 m . The shear span ratio (i.e. ratio between the distance of the loading point respect to the support and the effective depth) was equal to 1.9. In order to measure the beam deformations, potentiometric and LVDT transducers were used for monitoring the vertical displacement at midspan (front and back side) and at loading points, the crack opening due to shear and bending and the supports displacement, are shown in Figure 6. The potentiometric transducers which were employed for measuring shear crack widths were placed with an inclination of about 45° to the horizontal; this was based on experimental observations from previous similar experiments, where it was found that shear cracks develop from the load point to the bottom chord of the specimens, at an approximate distance d from the external supports. For measuring vertical cracks, the potentiometric transducers were placed horizontally near the extrados of the beam. Before setting the instruments the beams were whitewashed and a 100

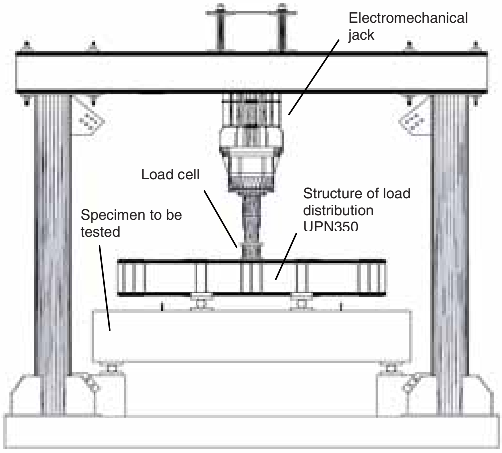


Figure 5. Loading frame.

mm spaced grid was drawn on the specimen surface in order to recorder the crack pattern by means of a high-resolution camera. Concerning the loading modalities, three preliminary elastic cycles were performed (up to two third of the estimated first crack load of each beam) to verify the suitability of the instrumentation.

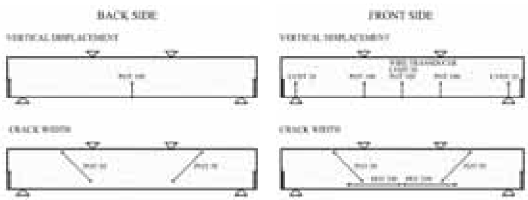


Figure 6. Instrumentation of the beams.

4 EXPERIMENTAL RESULTS

The curves of the load–versus–mid-span displacement of the five beams tested are plotted in Figure 7.

The mid-span displacement reported in Figure 7 is the average of the displacements measured at the front and back faces, cleansed of the average settlement measured at the supports.

4.1 *Reference beam*

In Figure 7, Beam SR exhibits a linear elastic behavior up to a load of 50 kN (onset of first vertical cracking in the zone between the two point loads). At the load value of 200 kN, shear cracks developed at an angle close to 30° to the axis, between the point loads and the supports (i.e., in the shear span). As soon as cracks (of either type) formed, a noticeable change occurred in the slope of the plots. At a load equal to 450 kN one of the shear cracks quickly widened and a sudden drop of the load occurred, followed by the closure of the other cracks. In the behavior of the beam, considerable reserve strength was apparently available owing to more efficient arch action. This failure result in concrete crushing near the point load, namely shear compression failure. The crack pattern at the end of test is reported in Figure 8.

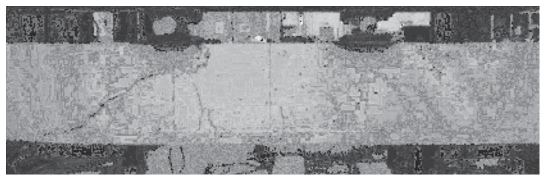
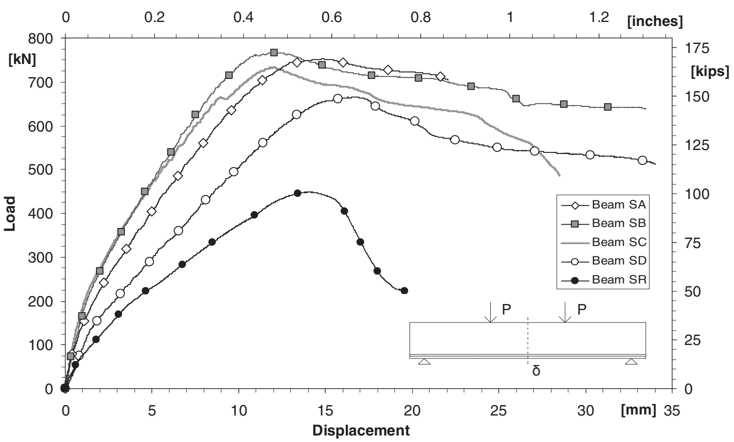


Figure 8. Reference Beam SR: crack pattern at collapse.

4.2 *Strengthened beams*

The four jacketed beams exhibited a bending-type failure. Their load-displacement curves are plotted in Figure 7. All beams behaved according to the same failure pattern. Under increasing loads, a first vertical crack appeared in the central portion of each specimen, accompanied by a slope change in the load-displacement curve.



BEAM SA	Bottom surface	30 mm	SL
	Lateral surface	30 mm	SL
BEAM SB	Bottom surface	50 mm	SL
	Lateral surface	50 mm	SL
BEAM SC	Bottom surface	50 mm	SL
	Lateral surface	50 mm	T
BEAM SD	Bottom surface	50 mm	SL
	Lateral surface	30 mm	T

SL: self leveling T: tihxotropic

Figure 7. Curve of the load versus mid-span displacement for shear beams.

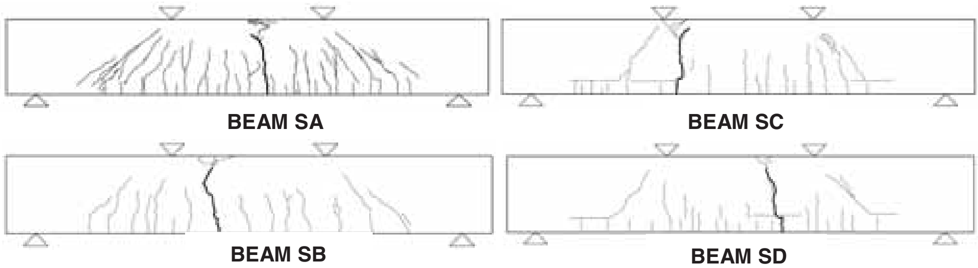


Figure 9. Jacketed beams: crack patterns at collapse.

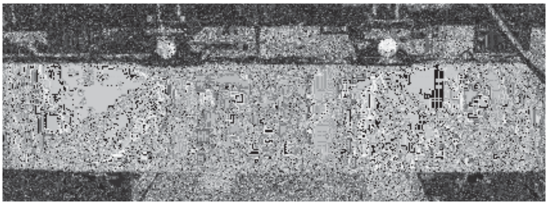


Figure 10. Crack distribution at the end of test for beam SA

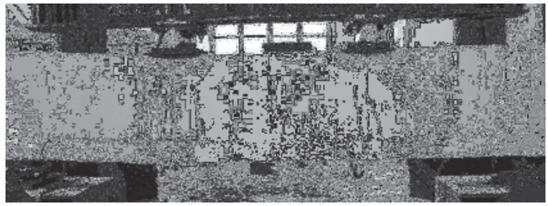


Figure 11. Crack distribution at the end of test for beam SC

Later, a few small vertical cracks developed between the two point loads and, for larger loads, beyond the two point loads. Subsequently, shear cracks appeared in both shear spans. The collapse was triggered by the development of a macro vertical crack, located between the two point loads. Later, however, the load decreased and stabilized at a level equal to about 80% of the maximum load. The tests were interrupted after a sort of rather long plateau, followed by the crushing of the concrete. The crack patterns at collapse are shown in Figure 9. Furthermore, in Beams SC and SD cracking developed along the joint between the self-levelling and the thixotropic concrete. Such a cracking accompanied the formation of the shear cracks and was located mostly in the shear spans. Figure 10 shows the crack pattern at the end of the test on Beam SA (jacket made of self-levelling concrete). The cracks are regularly spaced by 100 mm, and propagate into the zones between the point loads; their inclination in the shear spans is close to 45°. Also the crack pattern of beam SB (jacket made of self-levelling concrete of thickness equal to 50 mm) is very similar to the one of beam SA, albeit a larger crack spacing.

The crack patterns exhibited by Beam SC (side faces of the jacket made of thixotropic concrete) is reported in Figure 11. The cracks, however, are less regularly distributed than in Beam SA and are concentrated in the zones under the point loads. Shear cracks occur outside the point loads (inclination close to 30°) and are extended along the interface between the self-levelling and the thixotropic concrete, towards the end sections of the beam. In the shear spans there is a loss of bond between the HPFRC jacket and the original concrete. This loss occurred at the end of the test and did not compromise the results. The crack pattern at collapse of beam SD is similar than the one of beam SC.

4.3 *Discussion of the results*

The four tests demonstrate the effect that the HPFRCC jackets have on the collapse mode, as well as on the post-cracking behaviour, and on crack formation and evolution. Contrary to the reference beam, all jacketed beams fail in bending, with limited shear effects. Therefore the jacket plays a role that is close to that of the shear reinforcement and may be considered as an alternative to shear reinforcement.

It is worth noting that Beams SB and SC (50mm thick jacket, same reinforcement) exhibited a very similar load-displacement curve, at least until the peak load was reached.

HPFRCC layers allow to increase the capacity of the beam. For Beams SB and SC (50 mm jacket), as well as for Beam SA (30 mm jacket, self-levelling concrete), the capacity increases 1.7 times, while in Beam SD (30 mm thixotropic jacket along the lateral sides) the increment of the capacity is smaller (1.5 times).

The proposed technique allows also to markedly increase the structural stiffness, as shown by the mid-span displacements, which are greatly reduced for the same load level, before the peak load. Such a behaviour is more evident in Beams SA, SB and SC, and less pronounced in Beam SD, probably because of some imperfections in the bonding between the original beam concrete and the HPFRCC jacket. On the other hand, the response of the beam after cracking testifies that the contribution of the HPFRC jacket anyway leads to a considerable increase of

Table 3. Main results for the tests.

Specimen	Failure mode	P_u	δ	M_u	$M_{u,fl}$	$M_u/M_{u,fl}$
		[kN]	[mm]	[kNm]	[kNm]	-
Beam SR	Shear	450	13.2	180	227	0.79
Beam SA	Bending	751	14.9	300	280	1.07
Beam SB	Bending	773	11.9	309	312	0.99
Beam SC	Bending	741	12.1	296	299	0.99
Beam SD	Bending	670	16.9	268	279	0.96

both stiffness and strength of the reinforced beam with respect to the unreinforced beam.

Finally, note that, while the reference beam SR has a brittle failure (shear-type failure), the four jacketed beams SA, SB, SC and SD have a very ductile failure, as demonstrated by their post-peak behaviour, that is characterized by a rather stable softening.

In Table 3, for each specimen the failure mode, the maximum load (= peak load), the displacement at the peak load and the ratio between the actual ultimate moment and the calculated bending capacity are reported.

5 NUMERICAL INVESTIGATION

The experimental behaviour of the beams has been simulated with nonlinear FE analyses with the aim of better understanding the mechanisms governing shear response. The numerical analyses have been performed with the FEM program Diana 9.3.

5.1 *Geometry and materials*

By taking advantage of the symmetry in cross-section of the beams and loading, only one quarter of the full beam is modeling. This approach reduced computational time and computer disk space requirements significantly.

The beams were simulated with a 3D model using a twenty-node isoparametric solid hexahedral elements (element type CHX60) based on quadratic interpolation and Gauss integration on a 3x3x3 integration scheme. The mesh density adopted was approximately equal to 25x25x25 mm. To avoid problems related to local concentration of strains, also the steel plates used for supports and points load were modeled.

The reinforcement was embedded in structural elements, the so-called mother elements, and reinforcement strain were computed from the displacement field of the mother elements. A perfect bond between steel and concrete was assumed. This assumption can be accepted for the tested strengthening beam being the response governed by the flexural behaviour. As a consequence, small values of displacements were expected, whit respect to the experimental results. Based on the experimental results obtained, perfect bond was assumed at the interfaces between concrete substrate and HPFRC. To avoid that the loading plate would act in compression together with concrete, an interface element was placed between the loading plate and the concrete. This interface element is constructed between the load plate and the concrete and describes a relationship between the tractions and the relative displacement across the interface.

The concrete (concrete substrate and HPFRC) was modelled with a *Total Strain Rotating Crack Model* in which the stress-strain relationships are evaluated in the principal direction of the strain vector. Concerning the concrete, the compression behaviour was modelled by means of Thorenfeldt constitutive law (Figure 12A) and the tensile behaviour was modelled using the Hordijk constitutive law (Figure 12B). The steel behaviour was characterized by an elastic-hardening relationship and identical in tension and compression. Table 4 reports the main values used for the analysis for concrete and steel materials.

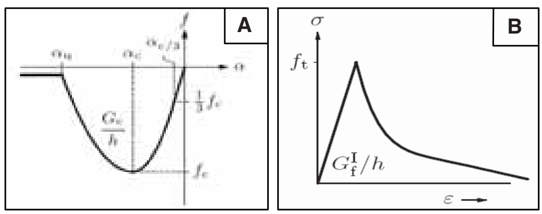


Figure 12. Thorenfeldt (A) and Hordijk law (B) for concrete.

Table 4. Values of concrete and steel.

CONCRETE			
E_{cm} [MPa]	f_{ctm} [N/mm^2]	G^I_f [Nmm/mm^2]	G_c [Nmm/mm^2]
29660	2.21	0.06	20
STEEL			
E_s [MPa]		f_y [N/mm^2]	
210000		517.94	

The numerical model required particular care for the definition of the constitutive parameters characterizing the response of the two HPFRC materials. The behaviour in compression was again simulated with the Thorenfeldt curve: the cylindrical strength for self-levelling HPFRC was assumed as equal to 107.9 N/mm^2 while for the thixotropic HPFRC was taken as equal to 74.7 N/mm^2.

The tensile properties (cracking stress and strain, peak stress and strain and maximum crack opening) of the HPFRC material, which are characterized by a hardening behaviour, were experimentally determined from uniaxial tensile test on dog-bone specimens. The stress-strain relationship for these two concrete was represented by a multi-linear model consisting of two parts: the first representing the behaviour before cracking, characterized by a hardening behaviour, the second concerning the response after cracking, when a softening behaviour occurs. The stress-strain curve adopted for the HPFRC materials in tension is shown in Figure 13.

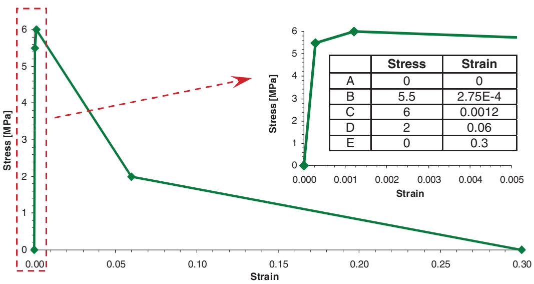


Figure 13. Stress - strain relationship for self-levelling HPFRC material

5.2 *Results of the analysis*

The analysis was conducted in displacement control with an increment of 0.1 mm for each step. Figure 14 shows the experimental and numerical load-deflection plots for the four strengthened beams. The results of the numerical analysis for beams SB and SC are in good agreement with the experimental data, whereas, using the rotating crack model with the same parameters of the other two beams, the structural response for beam SA and SD is in poor agreement with the experimental results, due to the inability of capturing the correct collapse mechanism.

To improve the results, a number of measures were taken. An incremental-iterative process was performed using a smaller increment size equal to 0.05 mm and the rotating crack model has been replaced with the fixed crack model with a shear retention factor β=0.2. Using these measures, the structural response is in reasonably good agreement with the experimental data also for beams SA and SD , and the collapse is governed by the beams reaching their moment capacity.

For all of the beams, the finite element model shows a stiffer response than the one experimentally observed both in the linear range and after cracking.

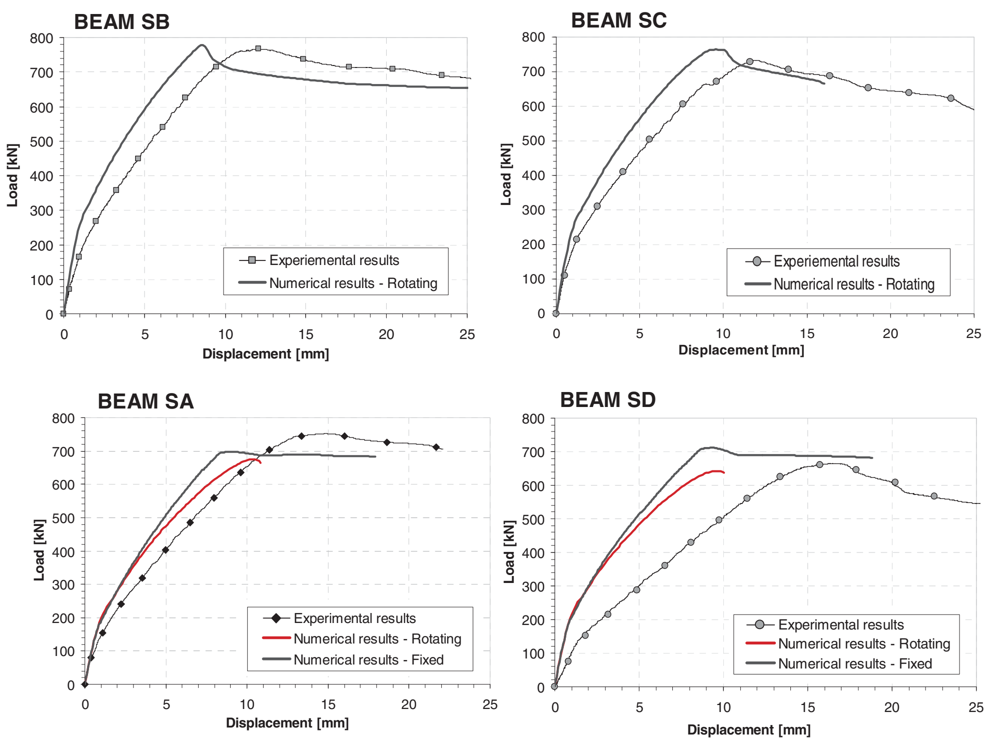


Figure 14. Numerical vs. experimental results for the jacketed beams

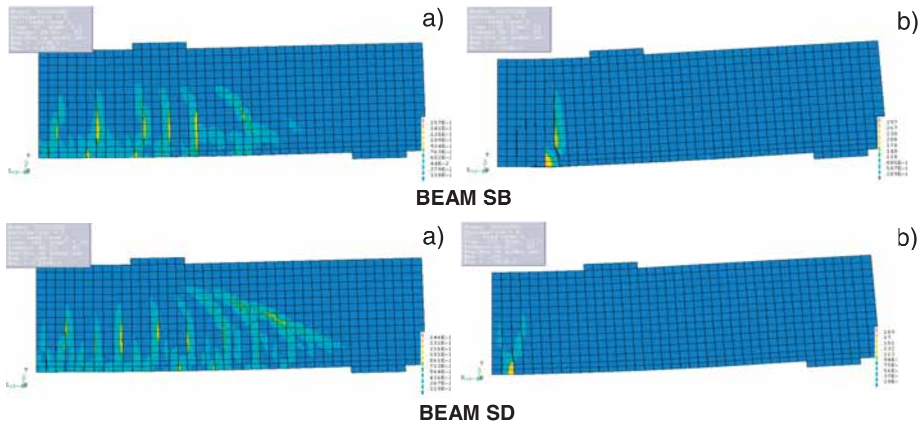


Figure 15. Strain in the fiber reinforced jacket obtained by the analysis for beams SB and SD – a) Mid-span displacement 6 mm, b) Mid-span displacement 15 mm

There are several effects that may cause the higher stiffness in the finite element models.
First, microcracks are already present in the concrete for the experimental beams, and could be produced by drying shrinkage in the concrete and/or handling of the beams. The microcracks reduce the stiffness of the experimental beams and are not included in the numerical model. Next, perfect bond between the concrete and steel reinforcing is assumed in the finite element analyses, but the assumption would not be true for the experimental beams. Thus, the overall stiffness of the experimental beams is expected to be lower than for the finite element models.

In order to investigate failure mechanisms in RC beams, strain and crack patterns in different load steps can help explaining the behavior of the members and fracture process adequately. Strain in the fiber reinforced jacket obtained by the analysis for the beam SB and SD are shown in Figure 15, where load steps labelled (a) are just before the peak load, and those labelled (b) are after the peak load (because of rescaling, minor cracks are not evident in these latter figures). As can be seen in these figures, crack formation and extension is well in agreement with the experimental evidence. As in the experimental test, after the peak load, the strains in the jacket tend to localize in a single macro crack.

Table 5 compares the first cracking and maximum loads for the experimental and numerical results.

The maximum load obtained from the numerical solution is higher than experimental load for three beams, but is slightly smaller for beam A.

6 CONCLUSION

The possible use of HPFRCs for strengthening R/C beams has been investigated by means of full-scale tests on simply-supported R/C beams, that were partly encased in HPFRC jackets. The results on specimens subjected primarily to shear confirm the effectiveness of this technique for flexural and shear strengthening.

On the basis of the experimental results the following conclusions can be drawn:

- the application of HPFRCC jackets brings in a sizable increase in both the bearing capacity and the stiffness in R/C beams;
- in the shear tests, using HPFRCC jackets shifts the failure mode from shear to bending, with very limited shear effects, thus showing the effectiveness of the technique for strengthening shear-deficient beams;

Table 5. Main results for the tests

	First cracking load [kN]			Maximum load [kN]		
	$P_{lf,ex}$	$P_{lf,FEM}$	% Difference	$P_{max,ex}$	$P_{max,FEM}$	% Difference
Beam SB	210	225	+7	767	777	+1
Beam SC	150	210	+40	741	764	+3
Beam SA	150	160	+7	752	698	-8
Beam SD	135	180	+33	670	712	+6

- coupling thixotropic and self-levelling concretes - or even using thixotropic concretes – seems to be a cost-effective means to cast jackets for a variety of structural application
- 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;

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