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Retrofitting of R/C shear walls by means of high performance jackets

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ABSTRACT

A new technique for the strengthening of existing R/C shear walls based on the application of thin high performance jackets is presented in this paper. The strengthening jacket is made of high performance concrete, having a compression resistance higher than 150 MPa, and reinforced by means of an high strength steel mesh. The experimental study is carried out on a 1:3 scale R/C wall, proportioned to resist vertical loads only, and reinforced by means of a 15 mm thick high performance jacket. Cyclic loads of increasing magnitude are applied to the experimental shear wall up to collapse. The effectiveness of the technique is also verified numerically. The results show the efficiency of the proposed solution in significantly increasing the structure resistance, deformation capacity and ductility.

1. Introduction

In earthquake engineering, a growing interest is nowadays shown for the assessment and retrofit of existing R/C buildings. Different approaches can be followed in the seismic rehabilitation of R/C structures. Possible solutions include: the retrofit of existing R/C frames, the use of resisting shear walls (either by strengthening existing R/C walls or inserting new shear walls) or the adoption of dissipative devices.

In existing buildings R/C walls are often present, commonly located near the stair blocks and the elevator areas, or along the structure perimeter. These elements are often reinforced to resist the vertical loads only, without considering the seismic actions. In few cases, the elements may be reinforced against horizontal wind loads but may be ineffective against the design seismic actions recommended by modern standards.

The transformation of the existing R/C walls into anti-seismic shear walls is a solution that is often preferred [1]. It is worth noting that regardless of the strengthening strategy, the presence of the existing R/C walls should be considered anyway: as a matter of fact, even if new anti-seismic devices are adopted, the seismic action transferred to the existing R/C walls can be rather large as a result of their significant stiffness. This in turn could result in severe damage of the R/C walls, prior to the activation of the devised anti-seismic systems.

In order to transform the existing R/C walls into seismic resistant shear walls, different retrofit techniques are traditionally proposed. The retrofit can be obtained by applying either traditional reinforced concrete jackets, external post compression or externally bonded FRP [1–4]. However, these techniques are not always easy to apply to existing structures. For example, the strengthening of an existing R/C wall by means of traditional R/C jackets might require excessive thicknesses (80–100 mm), which could jeopardize the future usage of the structure; whereas the application of FRP may be problematic because of the difficult anchorage of the fibers.

A new technique based on the use of jackets made of high performance fiber reinforced concrete has been recently proposed for the strengthening of existing R/C beams [5,6]. The method resulted in a significant increase of the structure resistance; as a drawback, however, a limited ductility was observed.

The application proposed in this paper can be regarded as an enhancement of this technique, obtained by adding a high strength steel mesh in the jacket with the aim to increase the ductility. The solution is based on the use of a high performance jacket made of high strength steel mesh, having a tensile resistance higher than 1200 MPa, embedded in a thin layer of high performance concrete, having a compressive strength higher than 150 MPa. By using these high performance materials, the jacket thickness can be significantly reduced with respect to a traditional solution in ordinary reinforced concrete.

In this paper, the proposed technique is validated by means of an experimental test on a 1:3 scaled shear wall. The experimental specimen was designed by reference to an existing three storey R/C building, which was proportioned to resist the vertical loads only. As a proportioning criteria, the high performance jacket was

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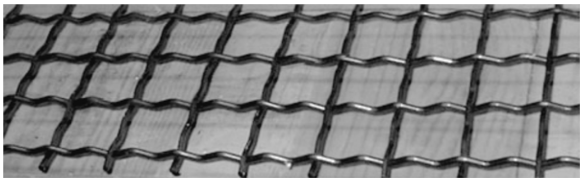

Fig. 1. High strength steel mesh made of bent wires. Single wires are weaved with each others to mechanically prevent their threading from the mesh.

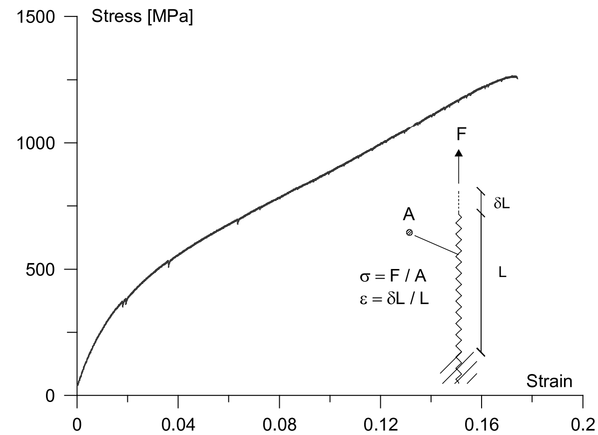

Fig. 2. Single mesh wire stress versus ideal strain relation-ship. Ideal strains are the result of both the wire straightening and deformation.

designed to entirely resist the seismic actions. Note that this assumption leads to a conservative proportioning of the strengthening solution. However, the assumption was made based on the fact that the resisting contribution of the existing wall is difficult to evaluate, due to the uncertainty in assessing the effectiveness of the transfer of shear forces at the base of the wall. The experimental results showed the efficiency of the proposed technique in increasing the structure bearing capacity and ductility.

A numerical study was also carried out to compare the performance of un-reinforced R/C and strengthened shear walls. The numerical model was validated against the experimental results and was used to analyze the performance of a full scale strengthened shear wall.

2. Materials

When performing a scaled test, a correct choice of the materials is necessary for the scaled model to be effectively representative of the full scale structure.

For the construction of the R/C wall, the concrete mix design was defined by reference to a scaled aggregate grading curve, by adopting a maximum aggregate size of 15 mm.

As for the reinforcement, hot rolled bars, having a diameter of 5 mm, were used. It is worth noting that, unlike hot rolled large diameter bars, smaller diameter bars are usually produced with cold formed steel. The different production process results in different deformability and ductility. Therefore, appositely hot rolled bars were produced and adopted for the construction of the scaled shear wall.

As for the jacketing, a high strength steel mesh was used. The mesh is made of 2 mm diameter bent wires, assembled with a spacing of 20 mm (Fig. 1). Since the high strength steel cannot be welded, the high strength steel mesh is made of bent weaved wires. The bend of the single wires mechanically prevents their unthreading from the mesh. The single wire geometry is illustrated in Fig. 1. The typical results of the tensile test performed on single wires are plotted in Fig. 2. The measured maximum strength was always higher than 1200 MPa.

For the reinforcing jacket, a high strength fiber concrete with a very compact matrix and with a maximum aggregate size of 2.7 mm was adopted. This material has also been used in other researches, in which the R/C strengthening jackets were made of concrete only, without any reinforcing steel meshes [5]. In those applications, fibers having a length of 12 mm were used. In this case, given the reduced scale of the model, 6 mm fibers having a diameter of 0.16 mm were selected, with a content equal to 2.5% by volume. The resulting high performance fiber concrete exhibits a hardening behaviour under tensile forces, and shows a compressive strength, measured on 100 mm cubes, higher than 150 MPa.

In order to assess the experimental behaviour of the reinforcing jacket, preliminary tests were performed on small specimens obtained with high strength steel mesh pieces embedded in a thin layer of high strength fiber concrete (Fig. 3). Different thicknesses of the high performance fiber concrete layer were considered (10 mm and 20 mm).

The tensile behaviour of the specimens was compared with the behaviour of the bare mesh. The experimental results are shown in Fig. 4. As expected, the presence of the concrete layer remarkably increases the stiffness of the bare mesh wires.

In the 10 mm thick specimen several smeared cracks developed throughout the test. Their location matched the position of the mesh transverse wires. Conversely, in the 20 mm thick specimen crack localization occurred, resulting in the specimen anticipated failure and in a small overall ductility.

It is worth noting that, in order to avoid crack localization and to guarantee larger structural ductility, mindful attention should be paid to the proportioning of the strengthening jacket. The coupling effect of the high strength steel mesh (having a small hardening behaviour) and the high performance concrete should be taken into account by controlling the ratio between the mesh diameter and the jacket thickness. To this purpose, further research is needed. At this stage, the aim can be pursued and the effective ductility of the jacket can be assessed, by performing preliminary tests on small specimens for varying ratios between mesh diameter and jacket thickness.

3. Wall specimen and test set-up

In order to verify the effectiveness of the proposed strengthening technique an experimental test was carried out. Reference was made to a typical R/C wall of an existing three storey building. The R/C experimental wall was built in a reduced 1:3 scale.

The specimen, reproducing a typical stair block element of an existing building, was designed to resist the vertical loads only. The geometry of both the prototype R/C wall and the scaled specimen is shown in Fig. 5(a)–(c). The scaled wall specimen has a height equal to 3.2 m (reproducing a 9.6 m real R/C wall) and a 100 mm $\times$ 800 mm cross section. The reinforcement is made of 5 mm diameter longitudinal rebars, having a spacing of 70 mm, and 4 mm diameter stirrups having a spacing of 100 mm. The R/C wall foundation block is anchored to the testing bench.

Upon completion of the R/C wall casting and curing, the reinforcing jacket was applied. In order to ensure perfect bond to the high performance jacket, the R/C wall surface was previously sandblasted in order to obtain a surface roughness of approximately $1 \div 2$ mm was obtained. This roughness was demonstrated to prevent any slip of the jacket [5].

After the steel mesh was fixed to the wall lateral surface, the high performance self-leveling fiber high performance concrete mix was poured into the moulds. A 15 mm jacket thickness was selected in order to obtain both a homogeneous casting and a sufficient mesh cover. The selected thickness results in a 45 mm jacket in a full scale application (Fig. 5(c)).

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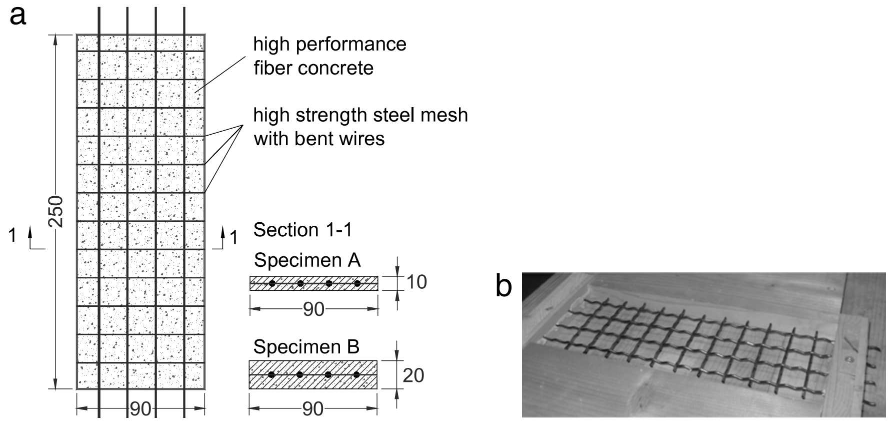

Fig. 3. (a) Geometry of the tensile test specimens [units in mm]. (b) Detail of the high strength steel mesh prior to the high performance fiber concrete cast.

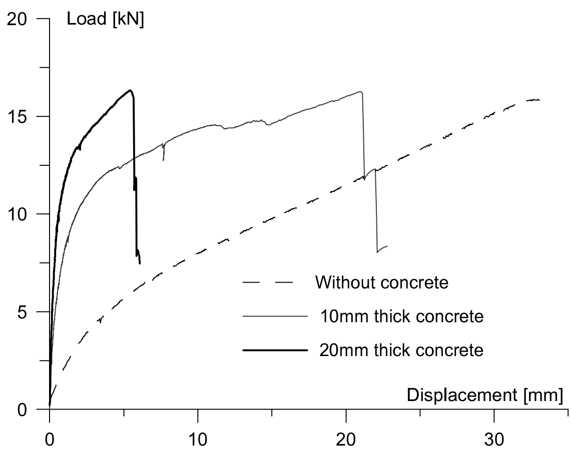

Fig. 4. Tensile tests performed on the bare high strength steel mesh and on specimens made of high strength steel mesh embedded in 10 and 20 mm layers of high performance concrete.

In order to properly anchor the steel mesh in the critical section at the wall footing, a 100 mm deep pocket was made in the foundation block.

The efficiency of the technique was verified by analyzing the structure performance in the critical zone, extending over the first inter-storey height, in which all the structure ductility is attained. For the sake of simplicity, the seismic load distribution that might be expected in the real structure, where the lateral forces from the earthquake are exerted at each storey, was replaced by a point load applied to the top edge of the experimental wall (F in Fig. 6(a)). In this way, the ratio of the shear force with respect to the bending moment at the base of the experimental wall is smaller (i.e., for a given base shear force, the bending moment is overestimated) than in the real structure. This simplification of the load boundary conditions does not significantly affect the results of the test, as it was designed mainly to assess strength and ductility under bending actions.

The point load was applied by means of an electromechanical jack. Cyclic tests were carried out in displacement control. The reaction frame adopted for the experimental test is schematized in Fig. 6(a). The wall was fixed to the frame with eight pre-tensioned bars (P) in order to avoid significant displacements and rotations of the foundation block. A constant vertical load ($N = 60$ kN) was applied by means of two hydraulic jacks, in order to simulate the actual vertical load of the prototype.

The applied force was measured by means of a load cell placed between the electromechanical jack and the wall top edge. The top displacement was monitored by two different measurement system. By reference to Fig. 6(b), two linear potentiometers (one for each wall side top edge) were used to monitor the small displacements (Pot. 7–8), whereas a potenziometric wire transducer

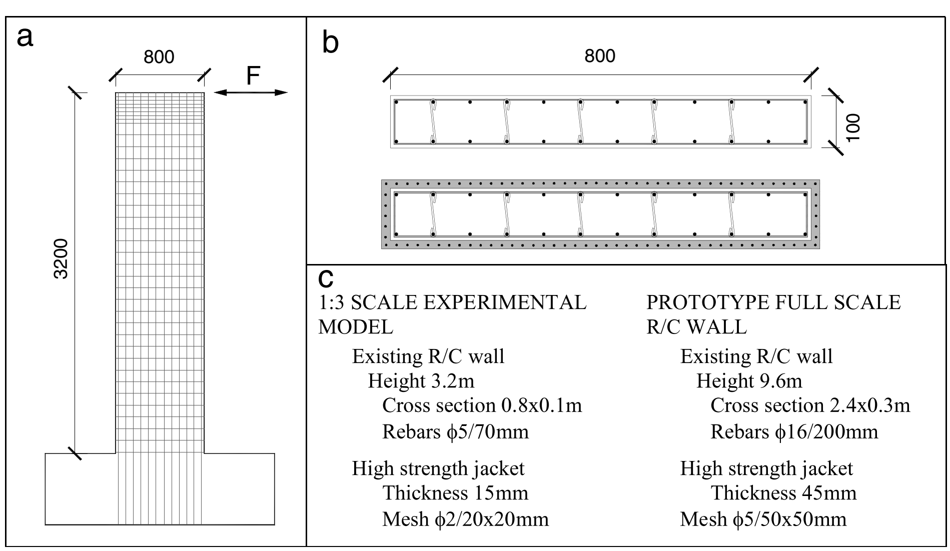

Fig. 5. (a) Front view and (b) cross section of the experimental model; (c) geometric properties of the 1:3 scaled model and the full scale prototype R/C wall (Units are in mm).

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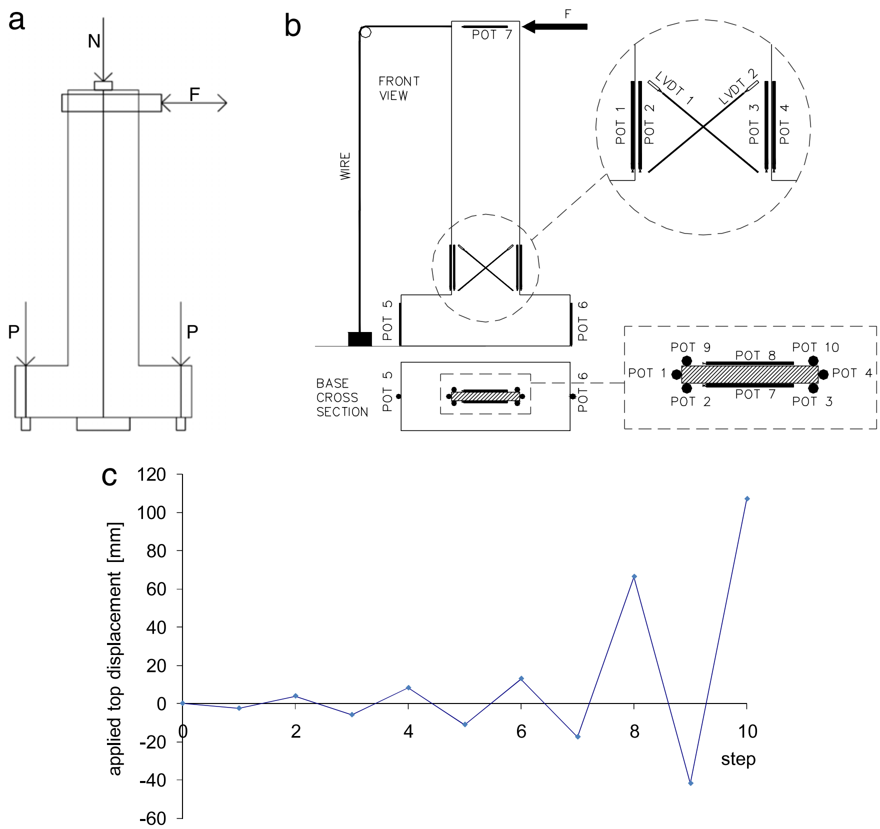

Fig. 6. (a) Scheme of the loading modalities; (b) instrument set up; (c) history of the top displacement applied to the structure.

was adopted for the larger displacements. The rotation at the wall footing was measured by means of a series of linear potentiometers (Pot. 1–4, 9,10). The wall base shear deformation was surveyed by two LVDTs (LVDT 1 and 2). The rotation of the foundation block was measured by means of two linear potentiometers (Pot. 5–6).

4. Experimantal results

The experimental test was carried out by applying cyclic loads of increasing amplitude up to the structure collapse (Fig. 6(c)). Fig. 7(a) shows the horizontal load versus top displacement curve. The behaviour is almost linear up to 50 kN with a limited dissipated energy. By increasing the cycle amplitude the dissipated energy increases and the behaviour becomes remarkably non linear. The structure yielding is recorded at a top displacement equal to 12 mm (δ_y); whereas the collapse is reached at a top displacement of 107 mm (δ_u) and a maximum load equal to 81 kN. The structure collapse is induced by crushing of the strengthening jacket at the wall base (Fig. 8(c)), with the internal concrete being already cracked. The structural ductility (δ_u/δ_y) was estimated as being equal to 9.

Fig. 8(a) shows the pronounced deformation of the shear wall at collapse, as well as the details of both the crack pattern in the critical zone and the crushing of the jacket at the base. The cracks are almost equally spaced and have a limited opening (Fig. 8(b)). The cracks are mainly horizontal proving that the structure behavior is governed by the bending moment up to collapse, with no appreciable influence of the shear effects. At collapse, i.e., at the ultimate applied top displacement (δ_u), the maximum shear distortion (γ_u) is approximately equal to 0.002 rad. This shear distortion is responsible of about 6% of the total top displacement.

The depth of the cracked area was almost 1.2 m (approximately equal to 1.5 times the shear wall cross section height). Neither crack localization, nor high strength mesh anchorage problems in the critical zone were observed. This way, plasticization could develop along a large portion of the shear wall throughout the test, resulting in remarkable structure deformability capacity. Unlike traditionally reinforced shear walls, a critical zone rather than a critical section was identified. Fig. 9 shows the bending moment versus curvature at the base section. The experimental curvature was determined by referring to the displacement measured by the potentiometric transducers placed at the wall base edges (Pot. 1, 4, Fig. 3(b)).

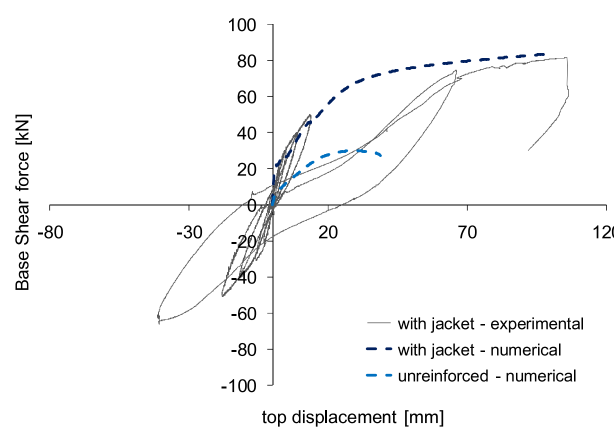

Fig. 7. Applied horizontal load versus top displacement curve.

A remarkable pinching effect can be observed in the last cycles of the experimental structure response. This is the result of the crack closing in the case of load reversal.

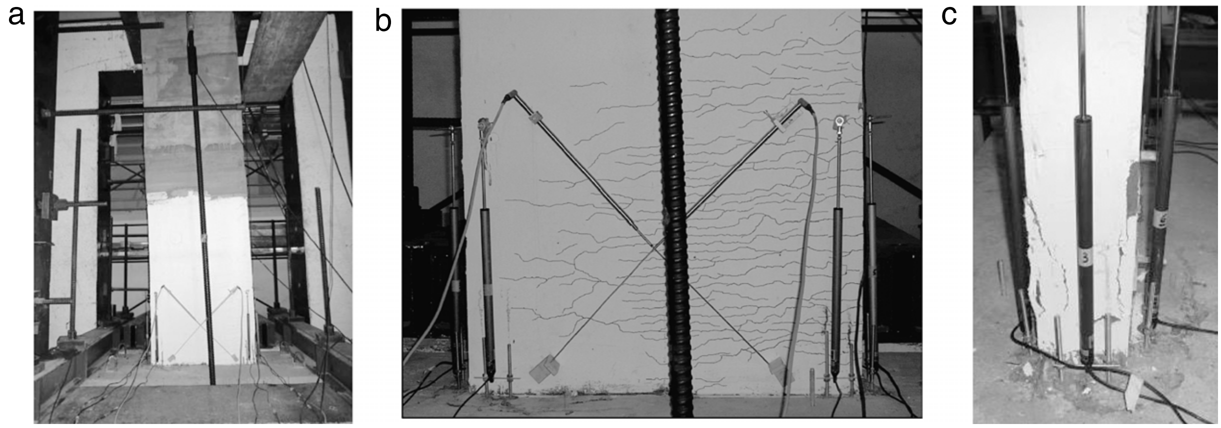

Fig. 8. (a) Side view of the deformed shear wall at collapse; (b) detail of the crack pattern in the critical zone; (c) collapse induced by the crushing of the jacket.

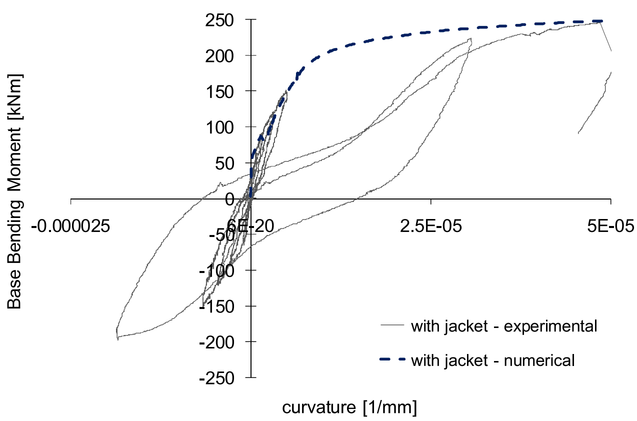

Fig. 9. Bending moment versus curvature experimental curve for the strengthened shear wall and numerical response for the unreinforced R/C wall.

5. Numerical analyses

Besides the experimental investigation, the efficiency of the proposed technique was verified numerically by comparing the performance of unreinforced and strengthened shear walls. The numerical model was initially validated through comparison with the experimental results on the 1:3 scale R/C wall, previously illustrated.

The shear walls, either unreinforced or jacketed, were modeled by means of force-based fibre elements, proposed in [7], implemented within the Finite Element Code FEAP [8]. The element formulation is general and yields an exact solution within the Timoshenko beam theory [7]. A simple, nonlinear, shear force–shear deformation law is used at the section level, together with a classical fiber section for the axial and bending effects [9]. Shear deformations are thus uncoupled from axial and bending effects in the section stiffness, but shear and bending forces become coupled at the element level because equilibrium is enforced along the beam element.

Force-based elements are computationally more demanding than displacement-based elements, but they offer the main advantage of being "exact" within the beam theory framework used for the formulation [8]. This leads to the use of one element per structural member (beam or column) in a frame analysis, thus requiring a lower number of nodal degrees of freedom. This results in the faster assembly of the numerical mesh, which might in turn be a significant advantage for practitioner engineers. Therefore, for the numerical analysis, the shear wall was modeled by means of a single element, having 5 control sections.

As for the materials, ordinary and high strength concrete and steel were described by Kent-Park and Menegotto-Pinto constitutive laws, respectively. The mechanical properties were obtained experimentally, by testing small specimens collected during the shear wall construction. The material parameters are summarized in Table 1.

In order to validate the model the experimental test was simulated. Accordingly, preliminary nonlinear pushover analyses were carried out by applying horizontal displacements of increasing amplitude to the 1:3 scale wall top edge. The horizontal load versus top displacement curve of the strengthened shear wall is shown in Fig. 7. The numerical response closely approximates the envelope of the experimental curve, both in terms of initial stiffness and strength (Fig. 7).

The response curve of a 1:3 scale unreinforced R/C wall is also illustrated in Fig. 7. The remarkable stiffening and strengthening effect of the reinforcing jacket can be observed. The maximum base shear force is more than twice as large as the maximum resistance of the traditional R/C wall. More importantly, the ductility is significantly increased. The same comparison is made in terms of bending moment versus curvature at the base section in Fig. 9.

Upon completion of the preliminary tests, the unreinforced and jacketed full scale shear walls were modeled to provide an estimate of the bearing capacity and ductility in an actual repaired R/C wall. The geometry is briefly described in Fig. 5(c). The material properties are those of Table 1. The vertical load was amplified, to reproduce the cross section stress level of the 1:3 scaled model ($N = 540$ kN).

The load versus top displacement curves of the full scale unreinforced R/C wall and the strengthened shear wall are shown in Fig. 10. The response of the unreinforced R/C wall initially shows a piecewise linear behavior up to the yielding of the steel reinforcement (point 2′ in Fig. 10). A significant reduction of the structure stiffness is observed upon overcoming the concrete tensile strength of the inner core (point 1′). For increasing applied displacement the curve is nonlinear as a result of the progressive yielding of the reinforcement occurring along the element cross section. The structural capacity is reached when the rebars yield in compression (point 3′), whereas the structural failure occurs when concrete crushes in compression (point 4′).

The response of the shear wall strengthened by means of high strength 45 mm jacket is shown in Fig. 10. The tensile strength of concrete is first overcome in the original wall (point 1) and later in the reinforcing jacket (point 2). The stiffness reduction is observed when the reinforcement of the inner core yields in tension (point 3). The high strength steel mesh starts yielding when the load is almost twice as large as the shear capacity of the unreinforced R/C wall (point 4). The stiffness reduces as a result of

Table 1
Material properties.

Ordinary concrete	High strength concrete	Ordinary steel	High strength steel
Kent & Park	Kent & Park + Tens	Menegotto & Pinto	Menegotto & Pinto
$f_c = 17$ MPa $\varepsilon_{fc} = 0.002$ $\varepsilon_{20\%fc} = 0.007$	$f_c = 140$ MPa $\varepsilon_{fc} = 0.002$ $\varepsilon_{20\%fc} = 0.014$ $f_{ct} = 6$ MPa $E_{ts} = 20\,000$ MPa	$f_y = 480$ MPa $E_s = 200\,000$ MPa $b = 0.005$ (hardening ratio) $r_0 = 20.0$ (exponential transition elastic-plastic) $a_1 = 18.5$ (coeff1 variation r_0) $a_2 = 0.15$ (coeff2 variation r_0)	$f_y = 1000$ MPa $E_s = 200\,000$ MPa $b = 0.03$ (hardening ratio) $r_0 = 20.0$ (exponential transition elastic-plastic) $a_1 = 18.5$ (coeff1 variation r_0) $a_2 = 0.15$ (coeff2 variation r_0)

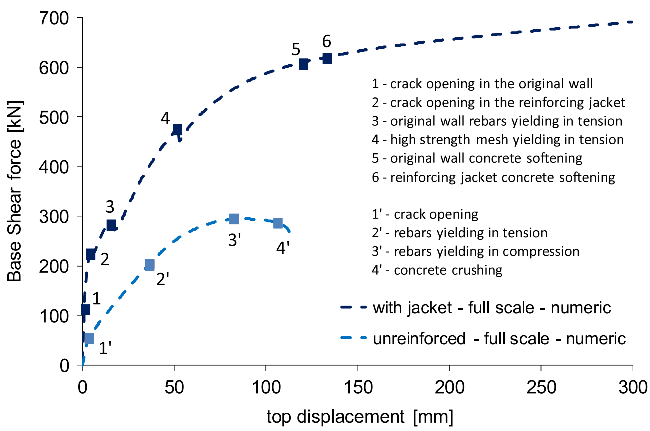

Fig. 10. Full scale model: applied horizontal load versus top displacement curve.

the softening of the concrete, which occurs upon overcoming the compressive resistance of the inner core and of the fiber reinforced concrete (points 5 and 6, respectively). The further increment of the structure resistance is governed by the hardening behavior of the high strength steel mesh. Unlike the experimental wall, no high strength concrete crushing was observed in the numerical model at this level of deformation. Concrete crushing was observed for a much larger drift of 5%.

6. Conclusions

The strengthening of existing R/C walls by means of a new technique based on the use of thin high performance jackets, made of high performance fiber concrete reinforced with high strength steel meshes, has been investigated. As a result of the strengthening, the RC walls of existing buildings, which are usually designed to resist the vertical loads only, are transformed into shear walls capable of adequately resisting the seismic actions.

The results of the experimental test, also confirmed by the numerical analyses, show that the use of a very thin high performance jacket allows to increase the structure ultimate resistance (more than double in the analyzed specimen). Furthermore, and perhaps more importantly, the proposed technique allows us to largely increase the structure deformation capacity and ductility, which are the main goals of every seismic strengthening design.

Referring to the performed test, the main results are that the structure failure is characterized by the wall base concrete crushing and by a crack pattern uniformly extending over a critical zone. The depth of the critical zone is approximately equal to 1.5 times the shear wall base. Unlike traditionally reinforced shear walls, showing a large crack opening at the wall base at collapse and thus having a limited ductility [10], no damage localization in a single critical section is observed. Furthermore, the structure behavior is governed by flexure up to collapse, with no appreciable influence of the shear effects.

The proposed technique can be easily used in structural applications, provided that its construction requires neither special works or special man labor. The high strength steel meshes can be folded by the producer prior to its transfer to the construction site and the high performance concrete can be easily pumped.

As a drawback, however, the strengthening of the existing RC walls results in a significant increase of the horizontal shear forces to be transferred to the foundations, which in turn may need structural strengthening works.

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