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REPAIR AND SEISMIC STRENGTHENING OF RC STRUCTURAL ELEMENTS BY UHPFRC CONCRETES

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ABSTRACT

This research investigates the seismic behavior of RC (Reinforced Concrete) columns, that are representative of 1:6 scaled bridge piers, strongly damaged by an earthquake and then repaired and retrofitted. The damaged parts of the column are repaired using new shaped steel bars and a concrete jacket built by an Ultra-high-performance fiber reinforced concrete (UHPFRC) with steel fibers. This jacket replaces the damaged concrete parts at the base of the column in plastic hinge region without modifying the column dimension and assures the necessary seismic upgrading (ductility and shear strength capacity of the repaired column) by the steel fibers. This reduces much time and economic, social and environmental impacts of the repair strategy because new transversal steel reinforcement and/or external CFRP wrappings, that are usually used for the seismic upgrade of columns, are not necessary. The proposed techniques were tested both experimentally by cyclic tests on RC columns (1:6 scaled bridge piers) and numerically by fiber models built in OpenSees (a software framework for simulating the seismic response of structural and geotechnical systems). The first results about RC columns repaired by the proposed strategy, confirm that these techniques are: simple to realize in situ, able to restore the maximum strength, the stiffness and the energy dissipation capacity of the column and are effective to guarantee the necessary seismic performance. The numerical analysis permits to investigate in detail the experimental column behavior.

Keywords: structural seismic behavior, RC structure repair, longitudinal rebar substitution, RC structure retrofitting, UHPFRC jacket

1. INTRODUCTION

Repair strategies that aim to restore or improve the seismic capacity of existing reinforced concrete (RC) members seriously damaged after a strong earthquake **(Fiorentino et al. 2018)** can assure a significant reduction of economic, social and environmental impacts with respect to reconstruction **(Albanesi et al. 2009; Lavorato et al. 2015a; Ruano et al. 2014; Rabehi et al. 2014; He et al. 2013; Cheng et al 2003; Albanesi et al. 2008; Lavorato et al. 2010a, Lavorato et al. 2010b, Lavorato et al. 2011; Lavorato et al. 2015b; Zhou et al. 2015; Zhou et al. 2014; Lavorato et al. 2017; Su et al. 2017)** This paper presents a new repair strategy that consists in: (i) substitution of the damaged longitudinal rebar parts by new shaped rebar parts; (ii) construction of a concrete jacket by means of ultra-high-performance fiber reinforced concrete (UHPFRC) with steel fiber to restore the damaged concrete parts and assure the seismic retrofitting (column shear strength and ductility improvements). The main steps of the repair intervention are showed in **Figure 1**. This solution is applied on RC columns (1:6 scaled bridge piers) subjected to axial and cyclic bending actions with damage located at the column base in plastic hinge region.

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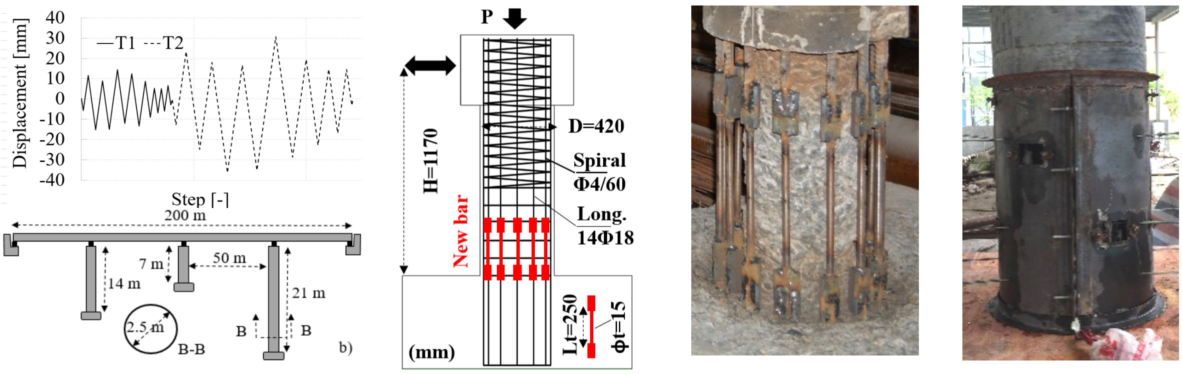
The anchorages of the original rebar in the damaged column are intact and so they can be used to anchor the new rebar (Figure 1). This is important because it is difficult to anchor new rebars in existing foundation that is usually congested with steel reinforcement. The longitudinal rebar substitution is performed by cutting the damaged rebar part only and connecting a new shaped rebar to the original anchorages and to the upper undamaged part of the rebar in the column. The connection between new and existing rebars is realized by a steel coupler and welding chords (Figure 1). The steel coupler is realized by means of two steel plates welded each other to realize a coupler with a V transversal section. This shape permits the correct location of the rebars before their connection by welding. The welding is realized between the rebars (butt welding) and between the rebar and the coupler by symmetric lateral welding. The new rebars are shaped to assure that the rebar plastic deformations are distributed along the new rebar only. This can be guaranteed simply if the new rebar has the maximum force smaller that the yield force of the existing rebar. In the present application, a new rebar that with yield, maximum stress and diameter similar to the ones of the original longitudinal rebar in the column is selected. A part of this rebar is turned to reduce the rebar diameter and so the maximum force in the new rebar is smaller than the yield force in the original rebar anchorages: the plastic deformations are distributed along the turned rebar part only. This is possible also thanks to the strong rebar connection that are designed in detail and tested in lab (these results are not showed here). A highway bridge (Figure 1) is selected as case of study to apply the repair strategy. The center pier of this bridge is the most stressed and damaged one after an earthquake because it is the shorter one (7 m) **(Lavorato et al. 2015a)Error! Reference source not found.**. The pier longitudinal steel reinforcement is designed by Chinese codes **(JTG D60-2004, JTG D62-2004, JTG/T B02-01-2008)** whereas the transversal reinforcement is insufficient to sustain the seismic shear and ductility demand. Some RC columns, that represent the shorter pier of the bridge by proper scale factors **(Lavorato et al. 2015a)**, were strongly damaged in plastic hinge region by cyclic tests and then repaired in **§2**. The repaired columns were tested again at the structural lab of Fuzhou University (China) in **§3**. The experimental results are presented and discussed in **§3** in term of maximum strength, stiffness, displacement capacity and shear strength of the repaired columns. The response of the repaired column is compared with the one of an undamaged column retrofitted by CFRP. The numerical fiber section analyses performed by means of OPENSEES (**McKenna et al. 2002**) in **§4** permit to investigate better the experimental results in term of maximum strength. Finally, the conclusions in **§5** show that the repair strategy results not only efficient to restore the column capacity but also to improve the seismic response of the column.

2. APPLICATION OF THE REPAIR STRATEGY ON RC COLUMNS

Some RC columns were built using proper scale factors for concrete geometries and steel reinforcement **(Lavorato et al. 2015a)** to be representative of the 7 m pier of the bridge in Figure 1 (the most stressed and damaged one in case of a strong earthquake) before damage. These columns are labelled as ASB columns and their concrete geometries are: diameter equal to 420 mm and height equal to 1170 mm (pier dimensions in scale 1:6, Figure 1). The columns steel reinforcement consists in: 14 longitudinal rebar HRB335E with diameter equal to 18 mm, a spiral built by 4 mm HPB235 steel rod spaced 60 mm. The yield and maximum stress of the longitudinal rebars were obtained by monotonic tests on steel specimens at the lab of Fuzhou University (china) and resulted equal to 450 and 600 MPa respectively. The transversal reinforcement was insufficient to sustain the seismic shear resulting from the capacity design criteria and so seismic retrofitting was necessary. A CFRP wrappings is realized with discontinuous rings applied along the column height to improve ductility and shear strength of the columns by capacity design rules. The mechanical properties of the CFRP tissue used to build the rings were: thickness equal to 0.167 mm, elastic modulus equal to 242 GPa and maximum design deformation equal to 0.005. The column P16-2 is one of these ASB columns and the column label means that this column represents a 1:6 scaled pier that is retrofitted by a CFRP wrapping with rings built using 2 layers of CFRP tissue (strips). The retrofitted ASB columns were strongly damaged in plastic hinge region by applying a cyclic deformation history on the top of the column (Figure 1). Then the damaged columns were repaired by the repair strategy proposed in **§1**. The damaged ASB column P16-2 was repaired and labelled as the repaired column (RR) R16-0-250-15-UHS. The new label describes shortly the characteristics of the repair intervention: R16 means that the column is repaired, the first number 0 indicates that the CFRP wrapping is not applied, 250 and 15 indicate the length and diameter of the new turned part respectively and UHS indicate that UHPFRC with steel fiber is used to build the concrete jacket. The damaged concrete parts of the R16-0-250-15-UHS column were repaired by UHPFRC with 2% of steel fibers. The steel fibers were long 13 mm, had equivalent diameter equal to 0.20 mm, ultimate tensile strength of 2000 MPa and modulus of elasticity equal to 200 GPa. The concrete mix design includes: cement, silica fume, sand and water. The water/cement ratio was 0.26. The compressive strength (f_c) resulted by experimental tests on material specimens at the lab of Fuzhou University was equal to about 90 MPa. The concrete jacket did not have transversal steel reinforcement. This simplifies much the cast of the new concrete parts with steel fiber.



(a) (b) (c) d)

Figure 1 Proposed repair strategy for RC columns: a) displacement history applied on the RC column and geometries of the bridge selected as case of study; b) repaired column reinforcement and concrete geometries; c) longitudinal rebar substitution; d) concrete restoration by an UHPFRC concrete jacket with (2 %)

The shear strength of the repaired column was calculated as the sum of two contributions: the shear strength of the concrete jacket ($V_{Rd,\ UHFRC}$) and the one of the original column core ($V_{Rd,i}$). $V_{Rd,\ UHFRC}$ was calculated by CNR-204/2006 **equation 1** substituting the shear area A_{HS} to the product $b_w d$. A_{HS} value was calculated by the formulation in **Priestley et al 1994** for a crown thickness equal to 100 mm (the removed concrete thickness, P2 part in Table 1).

$$V_{Rd,UHFRC} = \left[\frac{0.18}{\gamma_c} \cdot k \cdot (100 \cdot \rho_1 \cdot (1 + 7.5 \cdot \frac{f_{Ftuk}}{f_{ctk}}) \cdot f_{ck})^{1/3} + 0.15\sigma_{cp} \right] \cdot b_w \cdot d \tag{1}$$

In **the equation 1**, γ_c is the partial safety factor for the concrete without fibers, ρ_l is the reinforcement ratio for longitudinal reinforcement, f_{ck} is the cylindrical characteristic compression strength of the concrete, k is a factor that considers the size effect, σ_p is the average stress acting on the concrete cross section, f_{ctk} is the characteristic tensile strength of the concrete matrix and f_{Ftuk} is the characteristic value of the ultimate residual tensile strength. The concrete properties measured by tests on the UHPFRC with 2% (volume content) of steel fibers performed at the Fuzhou university lab are f_{Ftu} 5.02 MPa and f_{Ftu} / f_{ctk}= 0.48. The shear strength contribution of the original specimen concrete core ($V_{Rd,i}$) can be evaluated by different formulation presented in literature (**Sezen et al. 2004**). The sum of the two contributions resulted sufficient to sustain the maximum base shear measured experimentally and it is also evident by damage survey because there is not shear rupture of the repaired column.
The pier geometries are not modified as the new jacket substitutes the removed concrete parts only. The repair is rapid as the selected UHPFRC concrete presents at fresh state a great pass-ability to simplify concrete cast and guarantees a rapid increase of the concrete compression strength in 2 days (up to 80 % of the maximum compressive strength). The intervention is durable because the steel fibers in UHPFRC reduce the concrete cracks opening and so the concrete cover thickness can be not

increased with reduction of intervention impacts.

3. EXPERIMENTAL VALIDATION OF THE REPAIR STRATEGY

The retrofitted ASB column (P16-2) and the repaired column (R16-0-250-15-UHS) were tested at Fuzhou University Lab applying the same horizontal displacement history on the top of the column and a constant axial load P = 266 kN (deck seismic load). This displacement history is the one recorded on a repaired column during the tests in **Lavorato et al. 2015a** and so it is representative of the response of a pier in a bridge when the bridge is subjected to a strong earthquake. The number of the cycles applied are sufficient to evaluate the low cycle fatigue response of the column. The yield displacement of the column is 5 mm by preliminary tests, that are not presented here, and so the displacement ductility demand is investigated up to 7. The labels T1 and T2 indicate the displacement response of the column in **Lavorato et al. 2015a** when the bridge is subjected to Tolmezzo EW component earthquake (strong demand) or to the Tolmezzo earthquake scaled to double respectively (very strong demand). This earthquake was selected in **Lavorato et al. 2015a** because the response of the bridge in term of acceleration spectra has a peak in correspondence of the bridge natural period.

The experimental hysteretic (base shear - top column displacement) curves measured for the retrofitted ASB column P16-2 (black line) and for the repaired column R16-0-250-15-UHS (dashed black line) during the T1 and T2 displacement history are compared in Figure 2. The retrofitted ASB column (P16-2) presents wide cycles thanks to the CFRP wrapping confinement. The ultimate displacement is assumed in correspondence of the drop of the maximum base shear equal to 15%. The displacement ductility evaluated considering the ratio between the yield displacement equal to 5 mm and the ultimate displacement equal to 25-30 mm is at least 5. There are not abrupt loss of the load carrying capacity during the cycles and so rebars remain intact. A flexural failure mode is observed without pinching of cycles due to shear rupture.

The repaired column (R16-0-250-15-UHS) shows strength, stiffness and ductility capacity very similar to the ones measured for the column P16-2. The stiffness recovery is important because it is obtained without performing resin injection in the original column cracked core. The injections are difficult in situ and so the use of the UHPFRC is a very good solution for this problem. The hysteretic cycles are wide and the energy dissipation capacity is very similar to the one of the retrofitted undamaged column P16-2. The steel fibre contribution permits to obtain a seismic improvement by the new concrete jacket that is comparable to the one provided by CFRP wrapping applied on the undamaged columns. The flexural failure mode of the column is guaranteed by the new jacket only: no pinching of the hysteretic cycles or shear cracks were observed during the tests. The new shaped rebar and the connections are intact at the end of the test after the cover removal; it is also evident by the absence of abrupt loss of the load carrying capacity during the cycles. A reduction of the maximum base shear was expected for R16-0-250-15-UHS column because the new shaped rebars have a reduced diameter but this is not observed. The high compressive strength of the UHPFRC can explain this experimental result. It is investigated numerically by fibre section analysis in **§4**

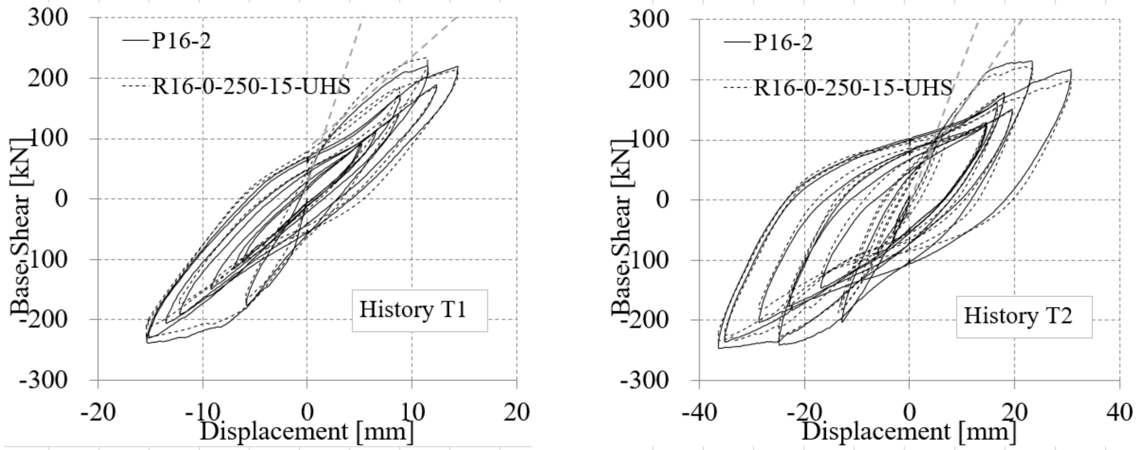


Figure 2 Experimental cyclic tests on the repaired column R16-0-250-15-UHS (dashed black lines) and the retrofitted ASB column P16-2 (continuous black lines) during T1 and T2 displacement hystory.

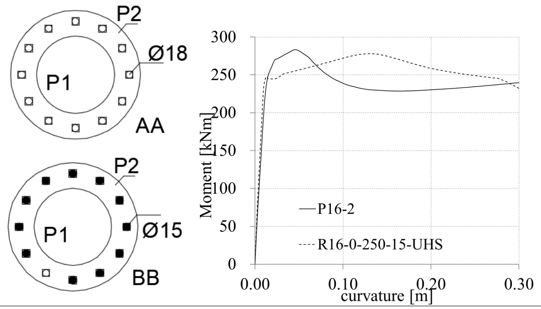
4. NUMERICAL SIMULATION OF THE REPAIRED COLUMN SECTION CAPACITY

Numerical analyses performed by means of some detailed fiber models of the column sections built in OpenSees (McKenna et al. 2002), permit to investigate the maximum experimental base shear of the columns tested in **§3.** The model base geometry is the same for each column: two concrete patches, a concrete ring (P2) and a concrete circular core (P1) divided in concrete fibers, and 14 steel fibers to represent the longitudinal rebar in the section (**Table 1**). The concrete ring P2 was used to model the new UHPFRC jacket of the column R16-0-250-15-UHS with a high concrete strength. The longitudinal rebars diameter is 18 mm or 15 mm for the section model of the column P16-2 or the column R16-0-250-15-UHS respectively. In fact, the weaker section of the repaired column is the one with the turned rebar (BB, **Table 1**) and so the maximum base shear is related with the maximum moment of this section. The section of the P16-2 column is the same along the entire column (AA, **Table 1**). The nonlinear behavior of the section is included in the model by the nonlinear stress-strain behavior of the fibers. The behavior of the concrete fibers of the column P16-2 is simulated by the model proposed by **Hosotani et al. 1998** for concrete confined by CFRP and stirrups. The difference between concrete confined by CFRP only or CFRP and stirrups is modest in this case and so one only material model is calibrated for P1 and P2 part in the section.

The behavior of the concrete fibers of the section part repaired by UHPFRC in the column R16-0-250-15-UHS is described considering the experimental curves given in **Su et al. 2017** for a very similar material. The results of the compressive tests given in **§2** for the UHPFRC used in this application permit to select the proper curve. The concrete core of the repaired column is confined by the new concrete jacket with steel fibers. The behavior of this concrete was assumed very similar to the one of a concrete confined by a CFRP wrapping built by one layer of CFRP tissue. The steel fiber behavior was described by a multilinear model with initial elastic modulus of 200 GPa and yield and maximum stress given **§2** by experimental tests. The stress-strain curve of each fiber is approximated in OPENSEES by the uniaxial Hysteretic material model (a multilinear model). The numerical analyses were performed applying on the section the experimental axial load (266 kN) and an increasing monotonic curvatures history. The results of these analyses are showed in **Table 1** in term of maximum moment on the section (M_{max_num}), maximum base shear (F_{max_num}) and height of the section compression zone (z_{c_num}) for each column. The numerical analyses reproduce well the maximum base shear measured on the column during the experimental tests (F_{max_exp}). The approximations about the stress-strain behaviors of steel and concretes result correct for the aims of these comparisons. The damaged core of the repaired column does not collaborate much in the section and so the model of the core concrete of the repaired column is not much important.

Table 1. Numerical fiber section analyses by OPENSEES: height of the section compression zone (z_{c_num}), numerical maximum moment (M_{max_num}), numerical (F_{max_num}) and experimental (F_{max_exp}) maximum base shear for the ASB (P16-2) and the repaired (R16-0-250-15-UHS) column; section fiber model for the ASB (AA) and the reapired column (BB)

	P16-2	**R16-0-250-15-UHS**
z_{c_num} [mm]	127.0	75.5
M_{max_num} [kNm]	283.3	278.1
F_{max_num} [kN]	242.8	238.4
F_{max_exp} [kN]	231.1	234.5



The force equilibrium in the section subjected to axial and bending actions is between the compression force in the compressed concrete part and rebars and the tension force in the rebar in tension. The compression zone of the section includes practically the new concrete parts only that are built with a concrete with high strength. The high value of the compressive strength of the UHPFRC reduces the height z_{c_num} of the compression zone in the section. A modest compressed zone of new concrete part is sufficient to guarantee the equilibrium of forces in the section and so the section inner lever arm increases. The increasing of the lever arm balances the decreasing of the maximum tensile force due to the rebar diameter reduction. This is a numerical verification of the experimental evidence in **§3**.

5. CONCLUSIONS

A rapid repair strategy is presented and applied on strongly damaged RC bridge piers or columns. This strategy consists in: the use of shaped rebar parts to substitute the damaged parts of the longitudinal rebars; a UHPFRC jacket with 2% of steel fibers to restore the concrete part and to improve the seismic performance of the column. Cyclic tests were carried out on RC columns to test the repair strategy effectiveness. The tested columns are: an undamaged column retrofitted by two layers of CFRP wrapping; a column repaired by the proposed techniques.

The comparison between the experimental results obtained for the two columns show:

- The repair strategy can be applied in short time (3-4 days) in situ in a simple way on seriously damaged columns and bridge piers
- The new shaped rebar sustains low cycles fatigue action and remains intact at the end of the test also if a very strong earthquake is applied on the column.
- Similarly, to the new rebar, the connection between new and existing rebars by a simple steel coupler and welding joints is efficient because the connections are intact at the end of the tests
- The repair strategy restores strength, stiffness and ductility capacity of the retrofitted undamaged column. The stiffness recovery is obtained without performing resin injection in the cracked concrete core.
- The repair strategy assures that the seismic capacity of the repaired column is very similar to the one of the retrofitted undamaged column. The UHPFRC jacket with steel fibers guarantees the necessary seismic performance that usually is obtained by CFRP wrapping. The advantage in the use of this jacket is the great time and cost saving because new steel stirrups and the CFRP wrapping are not applied.
- The high compressive strength of the UHPFRC reduces the height of the section concrete compression zone. This is good because the original cracked core contribution is modest and so new concrete parts only can sustain the seismic demand. The section inner lever arm increases and balances the reduction of the maximum steel tensile force due to the reduction by turning of the diameter of the new rebars. The strength of the column is restored. This is verified also numerically by OpenSees section analyses.
- The repaired column shear failure is avoided and the correct flexural failure is guaranteed. This is checked by damage survey and by the shape of the hysteretic cycles that did not show pinching.

The repair strategy will be tested also in case of the column response when the bridge is subjected to non-synchronous seismic action (**Lavorato et al. 2017**), which can be significant in case of long structures as bridges and can produce effects not studied in case of synchronous actions.

6. ACKNOWLEDGMENTS

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