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ULTRA-HIGH-PERFORMANCE FIBRE-REINFORCED CONCRETE JACKET FOR THE REPAIR AND THE SEISMIC RETROFITTING OF ITALIAN AND CHINESE RC BRIDGES

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Abstract. *The seismic behavior of Chinese RC (Reinforced Concrete) bridge piers with insufficient seismic details, severely damaged after an earthquake and then repaired and retrofitted by rapid interventions is investigated. The proposed interventions are applied on the damaged steel rebar and concrete parts in plastic hinge zone to guarantee the proper plastic dissipation of the seismic energy and the necessary shear strength and ductility improvements. New longitudinal shaped rebars and concrete jackets are used to substitute the pier damaged parts. Two repair and retrofitting procedures are presented and applied on 1:6 scaled pier specimens designed on the base of Chinese codes [1]-[3]: the first one uses an self-compacting concrete to build the concrete jacket and carbon Fiber Reinforced Polymer (CFRP) wrapping to assure the seismic improvements; the second one uses an ultra-high performance fiber reinforced concrete (UHPFRC) with steel fiber to build the concrete jacket and to assure the seismic upgrading allowing time and cost saving. The results of the first experimental tests on 1:6 scaled pier specimens repaired and retrofitting by the two procedures, are compared and discussed.*

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1 INTRODUCTION

The repair and retrofitting interventions performed by means of well-tested materials and techniques can restore the functionality of RC (Reinforced Concrete) bridges in short time [4]-[25].

The present research introduces innovative interventions for the rapid repair and seismic upgrade of strongly damaged concrete and steel reinforcement parts due to cyclic loading at the base of cantilever RC piers due to the insufficient seismic details.

The irregular bridge shown in Table 1 is a critical structure in an infrastructures and structures network with seismic critical issues [26]-[29] and it should be repaired and retrofitted in very short time in case of damage. For that reason, this bridge represents a valid case of study to apply the proposed rapid interventions. The center pier of this bridge (7 m pier, Table 1) is the most stressed and damaged one after the seismic load application on the entire bridge [30].

The pier longitudinal steel reinforcement is designed by Chinese codes [1]-[3] whereas the transversal reinforcement is insufficient to sustain the shear and ductility demand [31], [32].

The 7 m pier behavior is studied experimentally by means of 1:6 scaled pier specimens subjected to axial load and cyclic displacement histories. Some pier specimens are retrofitted and strongly damaged in plastic hinge by cyclic tests to be repaired and retrofitted by the two different repair and retrofitting procedures (PR1 and PR2).

Each procedure repairs the damaged longitudinal rebar parts by partial substitution using new shaped rebar parts [22]-[25]. A strong connection system connects these new rebar parts to the undamaged original ones (anchorages and undamaged rebar part outside the plastic hinge). The shaped rebar is designed with a geometry that assures the correct distribution of the steel plastic deformations in plastic hinge where the seismic energy is dissipated.

The repair of the removed damage concrete parts and the seismic retrofitting of the piers are performed in two different ways: the first procedure (PR1) uses self-compacting concrete (SCC) to build a concrete jacket (CJ) and carbon Fiber Reinforced Polymer (CFRP) wrapping to assure the seismic improvements; the second procedure (PR2) uses ultra-high performance fiber reinforced concrete (UHPFRC) with steel fiber to build a CJ and to assure the seismic upgrading allowing time and cost saving. Experimental tests on UHPFRC material specimens with 2% of fiber content and a specific concrete mix design were carried out at Fuzhou lab to check the available shear strength of the UHPFRC CJ on the base of CNR-DT 204/2006 guideline [41] formulations.

Cyclic tests on two 1:6 pier specimens repaired and retrofitting by the two proposed procedures were carried out at the lab of the Sustainable and Innovative Bridges Engineering Research Center of Fujian Province (SIBERC) of the Fuzhou University (China) to evaluate the effectiveness of the proposed techniques.

The first experimental results are presented and discussed comparing the two proposed techniques (PR1 and PR2). Finally, numerical fiber section analyses were performed by means of OPENSEES [42] to understand better the experimental results.

2 PROPOSED REPAIR AND RETROFITTING SOLUTIONS

The repair and retrofitting solutions tested experimentally by pseudo-dynamic test (PSD) on Italian RC bridge piers in [4] and [5] can restore the original pier strength avoiding the original shear rupture but the connection system between undamaged original rebar parts (anchorages and rebar part out of the plastic hinge) and the new rebar parts used to substitute the damaged ones, has to be improved [5].

Two different repair and retrofitting procedures (PR1 and PR2) are presented in Figure 1. Firstly, the damaged concrete and steel rebar parts at the base of the pier are removed (Figure 1a).

Each procedure repairs the longitudinal damaged rebar parts by partial substitution using new shaped rebars (Figure 1b). The new rebar parts are connected to the original undamaged ones (anchorages and undamaged rebar part outside the plastic hinge) by means of steel coupler and welding joints. This connection system was studied extensively by numerical finite element models and tested experimentally with very good results but these results are not discussed here. The rebars connection results effective and the shaped rebar moves the plastic deformations along the new rebar part with a reduced section away from the rebar connection. The substitution of each rebar at the pier base assures the proper plastic deformation of each longitudinal rebar in plastic hinge only.

The repair of the damage concrete parts and the pier seismic retrofitting are performed in two different ways (Figure 1c, d): the first procedure (PR1) uses self-compacting concrete (SCC) to build a concrete jacket (CJ) and carbon Fiber Reinforced Polymer (CFRP) wrapping to assure the seismic improvements; the second procedure (PR2) uses ultra-high performance fiber reinforced concrete (UHPFRC) with steel fiber to build a CJ and to assure the seismic upgrading allowing time and cost saving. The pier geometries are not modified as the CJ substitutes the removed concrete parts only.

The repair is rapid as the selected SCC and UHPFRC mixes present at fresh state a great pass-ability to simplify concrete casting and guarantee a rapid increase of the concrete compression strength. The intervention results durable without modifying the cover thickness by using steel fiber in UHPFRC or specific components in SCC.

The retrofitting is included in the repair operation in case of procedure PR2 because the UHPFRC jacket improves the original insufficient shear strength of the pier by fibre contribution.

However, the retrofitting is rapid also in case of PR1 as the CFRP wrapping is simple to realize in situ.

(a) (b) (c) (d)

Figure 1 Repair and retrofitting solutions PR1 and PR2 for damaged RC piers: a) damaged concrete and rebar parts removal; b) longitudinal rebar substitution; c) concrete restoration by SCC (PR1) or UHPFRC concrete jacket (PR2); d) CFRP wrapping application (PR1 only).

3 DEFINITION OF THE CASE OF STUDY

This study focuses on the 7 m pier of the irregular RC bridge in Table 1 designed by means of Chinese codes [1]-[3]. The pier longitudinal steel reinforcements was calculated considering the design of the entire bridge, the seismic action with 0.2g PGA and the deck vertical loads of 200 KN/m as it is described in [4], [5], [17]-[25], [33].

The bridge materials are: the Chinese strength grade C30 (cylindrical characteristic compressive strength equal to 20.1 Mpa) for the concrete and the Chinese steel grade HRB335E (characteristics tensile strength 335Mpa) for the steel reinforcement. The transversal steel configuration was selected to reproduce the one of an existing bridge with seismic deficiencies.

4 REPAIRED AND/OR RETROFITTED BRIDGE PIER SPECIMENS

Some 1:6 scaled pier specimens representative of the 7 m pier (Table 1) were built using proper scale factors for concrete and steel reinforcement geometries [33], [34]. The section diameter and height of each specimen were equal to 420 mm and 1170 mm respectively (Table 1). The longitudinal steel reinforcement was composed by fourteen 18 mm HRB335E rebars whereas 4 mm HPB235 steel rods spaced 60 mm were used to build the transversal steel reinforcement (Table 1). The yielding and maximum stresses of the longitudinal rebars were obtained by monotonic tests 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.

Two pier specimens labelled as P16-1 and P16-2 were retrofitted by means of CFRP wrappings built by one or two layers respectively to increase the original insufficient shear strength and ductility (Table 1). The CFRP mechanical properties were: thickness of 0.167 mm, elastic modulus of 242 GPa and maximum design deformation equal to 0.005. These specimens were strongly damaged at plastic hinge zone by cyclic tests ([15]-[25]).

These damaged specimens were repaired and retrofitted by the solution PR1 or PR2 presented in §2 to be tested again: the specimen P16-1 was repaired and retrofitted by the procedure PR1 and labelled as R16-SCC-SR-1 whereas the specimen P16-2 was repaired and retrofitted by the procedure PR2 and labelled as R16-HSF-SR-0. The new labels describe shortly the characteristics of the repair and retrofitting interventions for each specimen: R16 means that the 1:6 scaled specimen is repaired, SCC and HSF indicate the SCC or UHPFRC material used to build the concrete jacket, SR indicates the use of shaped rebar to substitute the damaged rebar and the last number of the label is the number of the layers of the CFRP wrapping.

The shaped rebar (SR) of each repaired specimen had a part with a reduced diameter of 15 mm and length of 250 mm (Table 1). The damaged concrete of the specimen R16-SCC-SR-1 was repaired by a SCC jacket with transversal stirrups Φ4/60 mm. The SCC cylindrical compressive strength was evaluated by compressive tests on material specimens and was equal to 25 MPa. The retrofitting of this specimen was performed applying the same CFRP wrapping of the specimen P-1 (Table 1).

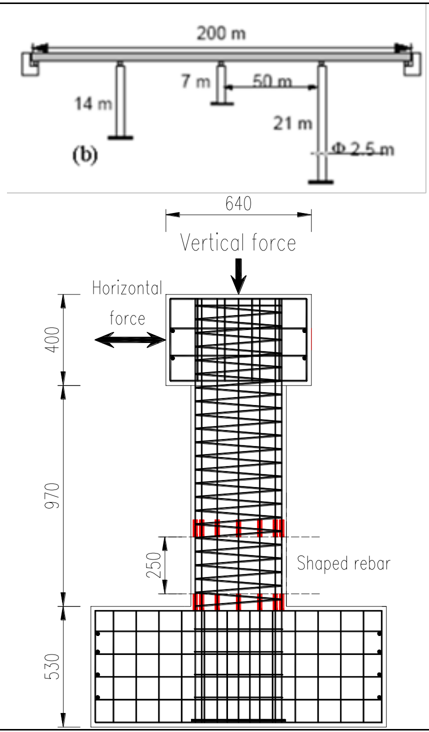
The damaged concrete parts of the R16-HSF-SR-0 specimen were repaired by UHPFRC with 2% of steel fibers (HSF). This fiber content was selected to build a HSF jacket with a shear strength sufficient to retrofit the damaged pier specimen [22], [23].

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 was composed by cement, silica fume, sand and the water/cement ratio was 0.26. The compressive strength resulted by experimental test on material was equal to about 90 MPa.

The HSF jacket did not have transversal steel reinforcement but the HSF jacket contribution calculated by CNR-DT 204/2006 guideline [41] considering also the original pier specimen core contribution resulted sufficient to sustain the maximum base shear measured experimentally on the retrofitted specimens P16-2 (Figure 2).

Table 1 – Geometries and reinforcement configuration for the retrofitted pier specimens (P16-1, P16-2) and the repaired and retrofitted pier specimens (R16-SCC-SR-1, R16-HSF-SR-0): concrete jacket (CJ) material, length (L_s) and diameter (Φ_{SR}) of the longitudinal shaped rebar part (Shaped Reb.) in the repaired zone, transversal rebar (Tr. Reb.) of CJ and the number (n_e) of the CFRP wrapping layers [mm]; "-" indicates absence of the intervention; b) irregular bridge geometries

Pier	CJ material	Shaped Reb.		CJ Tr. Reb.	CFRP
		L_s	Φ_{SR}		n_e
P16-1	-	-	-	Φ4/60	1
P16-2					2
R16-SCC-SR-1	SCC	250	15		1
R16-HSF-SR-0	UHPFRC			-	-



5 CYCLIC TEST ON PIER SPECIMENS

Cyclic tests were carried out on the retrofitted (P16-1, P16-2) and on the repaired and retrofitted pier specimens (R16-SCC-SR-1, R16-HSF-SR-0) at Fuzhou University Lab applying a constant axial vertical load of 266 kN (deck weight) and the same horizontal displacements history on the top of the specimen. The displacement history was representative of the one recorded on the central bridge pier when Tolmezzo (PD1) and Tolmezzo scaled to double (PD2) accelerograms are applied on the entire bridge [5]. The experimental responses of the retrofitted specimens, which were designed to guarantee the necessary seismic performances, represented the target responses for the repaired and retrofitted specimens. These tests permitted to evaluate the effectives of repair and retrofitting operations performed by means of the two procedures PR1 and PR2.

The base shear - top pier displacement curves for the retrofitted specimens P16-1 and P16-2 (black line, Figure 2) and the ones for the repaired and retrofitted specimens R16-SCC-SR-1 and R16-HSF-SR-0 (dashed black line, Figure 2) are compared during the cyclic tests in Figure 2. The cyclic behaviours of the repaired and retrofitted specimens were very good: the cycles were wide and stable and so the shaped rebars assure an effective seismic energy dissipation. The connections among new shaped rebar and existing undamaged rebar parts were not damaged during the test and so this connection solution results strong.

Each specimen did not show premature shear ruptures. The repair of the specimen R16-SCC-SR-1 is effective and the same retrofitting solution already tested on the specimen P16-1 is valid. This experimental result was much more interesting in case of the specimen R16-HSF-SR-0: the HSF jacket resulted effective to perform not only the concrete parts repair but also the seismic upgrade of the specimen. This specimen shown a seismic behaviour very similar to the one of the retrofitted specimen P16-2 with an external wrapping built by means of two CFRP layers (Figure 2).

The specimen R16-SCC-SR-1 shown a maximum base shear smaller than the one of the P16-1 specimen (Figure 2). It is expected as the shaped rebar part in R16-SCC-SR-1 had a reduced diameter respect to the one of the original rebar in P16-1 (Table 1) and so the maximum resisting moment of the R16-SCC-SR-1 pier section with shaped rebar was smaller.

The same base shear reduction should be expected for R16-HSF-SR-0 specimens that has the same shaped rebar of R16-SCC-SR-1 (**Table 1**) but this specimen showed the maximum base shear value similar to the one of P16-2 without shaped rebar (Figure 2). This may be due to the high compressive strength of the UHPFRC that reduces the concrete compression zone of the section and increases the section inner lever arm. Furthermore, the steel fibre contribution in longitudinal direction can also increase the section resisting moment. Further investigations are necessary to understand better this behaviour but it is partially studied by simple numerical analyses in §6.

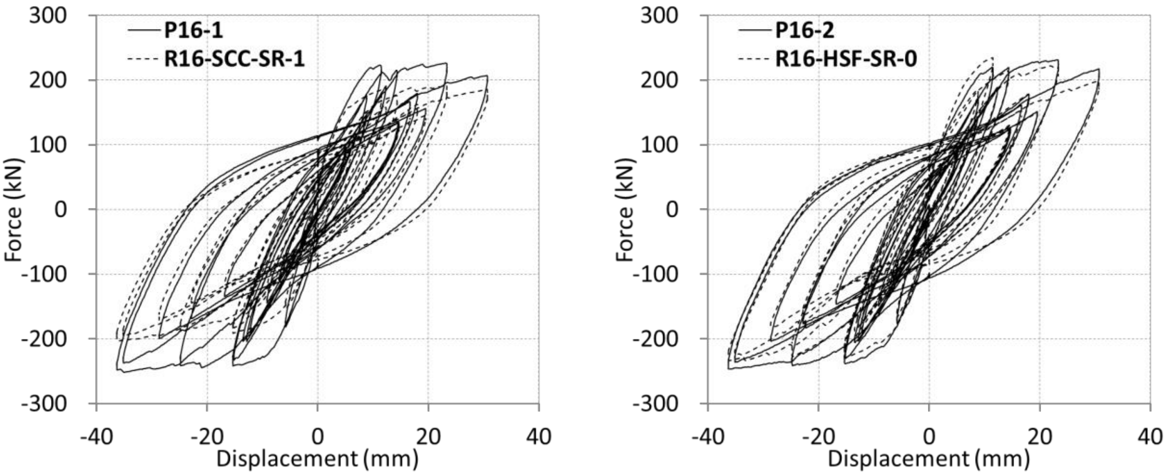


Figure 2 Experimental cyclic tests on 1:6 pier specimens: base shear (force) vs top pier displacement curves for repaired and retrofitted specimen R16-SCC-SR-1 and R16-HSF-SR-0 (dash black lines) and for the undamaged retrofitted specimens P16-1 and P16-2 (continuous black lines).

6 NUMERICAL FIBER SECTION ANALYSES

Numerical analyses were performed to understand better the experimental results previously discussed in §5 about the cyclic tests on the 1:6 pier specimens in term of maximum base shear. The numerical base shear of the specimen is obtained by the ratio of the numerical maximum moment at the base of the specimen and the pier specimen height.

A fiber model of the base section of the 1:6 scaled specimens was built in OPENSEES [42] to evaluate the numerical maximum moment. This model (Table 2) consisted of an external concrete ring and a concrete circular core which were divided in fiber elements, and single fiber for each longitudinal rebar (Table 2). The concrete ring was used to model the section part with confined concrete by CFRP only (rebar cover thickness) for the specimen P16-1, P16-2 and R16-SCC-SR-1 or the section part built by UHPFRC (jacket thickness) in case of the specimen R16-HSF-SR-0.

The behaviors of the cover and core concretes of the specimens P16-1, P16-2 and R16-SCC-SR-1 were described by means of the analytical stress-strain model proposed by Hosotani et al. [43] for concrete confined by CFRP and/or steel stirrups.

The confinement effect of the stirrups (ϕ4/60 mm) on the core concrete was modest but it was included in the analyses. The concrete maximum deformation was evaluated by Hosotani et al. [43] equation using the maximum value of the experimental CFRP deformation measured during the cyclic tests (about 0.005).

The SCC used to build the CJ of the repaired and retrofitted specimen R16-SCC-SR-1 had the same mix design of the SCC concrete used to build the specimens P16-1 and P16-2: the compressive strength of the unconfined SCC, that was measured experimentally in §4, was assumed equal for these three specimens. For that reason, the analytical stress-strain curves for the core and cover concretes of the specimens P16-1 and R16-SCC-SR-1 which had the same stirrups and were retrofitted by the same CFRP wrapping were assumed equal (Table 1). The cover and core concretes of P16-2 had stress-strain analytical curves defined considering the effect of the confinement due to two layers of CFRP and stirrups in Table 1.

The confinement effect of the UHPFRC CJ on the SCC core of the specimen R16-HSF-SR-0 was not investigated in detail here: it was assumed a little smaller than the one due to one layer of CFRP. The unconfined UHPFRC stress-strain curve was obtained by scaling the experimental curve presented in literature [47] using the maximum tensile and compressive stresses measured during the experimental tests described in §4.

The steel fiber stress-strain behavior was described by a multilinear model with initial elastic modulus of 200 GPa, yielding stress and maximum stress measured by tensile tests at Fuzhou lab and given in §4.

The stress-strain curve for the cover and core concrete fibers and for the steel fibers were modelled in OPENSEES by the uniaxial Hysteretic material model. This material model is a multilinear curve that can be used to simplify the concrete stress-strain curve described by the Hosotani et al. [43]. This simplification is valid for the aims of these first analyses which intend to reproduce the maximum base shear of the specimen only.

The base section placed at pier anchorages, where the moment is greater and the longitudinal rebar had diameter equal to 18 mm, was analyzed in case of the retrofitted specimens P16-1 and P16-2.

The section just above the rebar connection (Figure 1) was considered in case of repaired and retrofitted specimen R16-SCC-SR-1 and R16-HSF-SR-0 as it was the one that must plasticize for the first according the design (each longitudinal rebar of this section had reduced diameter equal to15 mm).

The numerical axial load on each section was the experimental one that was equal to 266 kN whereas the section curvatures were increased monotonically during the numerical analyses.

The results of these analyses are shown in Table 2 in term of maximum moment on the section (M_{max_num}), maximum base shear (F_{max_num}) of the specimen and height of the section compression zone (z_{c_num}).

The numerical analyses can reproduce well the maximum base shear of the specimens measured 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 numerical analyses about P16-1 and R16-SCC-SR-1 specimens confirm that the experimental reduction of the maximum base shear of R16-SCC-SR-1 respect to the one measured on P16-1 (Figure 2) is essentially due to the shaped rebar diameter reduction as the only difference between the P16-1 and R16-SCC-SR-1 fiber section models was the diameter of the longitudinal rebars.

The numerical results are much interesting about the specimen R16-HSF-SR-0 that had shaped rebars but showed experimental maximum base shear very similar to the one of the retrofitted specimen P16-2 without shaped rebar. The numerical fiber section model of the specimen R16-HSF-SR-0 differed from the one of the specimen P16-2 not only for the reduced diameter of each longitudinal rebar but also for the material used to build the concrete jacket: the UHPFRC was much more resistant than the SCC.

The high value of the compressive strength of the UHPFRC reduces the height z_{c_num} of the section compression zone and increases the inner lever arm of the section: it is evident in Table 2 comparing the values of the z_{c_num} calculated for R16-HSF-SR-0 with the one of P16-2.

The increasing of the inner lever arms may contrast the decreasing of the steel tensile resulting force due to the shaped rebar diameter reduction. This numerical evaluation will be studied in future investigations by a more detailed definition of the stress-strain curves of the UHPFRC used to build the CJ and of the original SCC core concrete confined by the UHPFRC CJ.

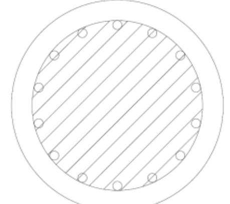
	P16-1	P16-2	R16-SCC-SR-1	R16-HSF-SR-0	
z_{c_num} [mm]	130.8	127.0	108.3	75.5	
M_{max_num} [kNm]	276.7	283.3	222.1	278.1	
F_{max_num} [kN]	237.2	242.8	190.4	238.4	
F_{max_exp} [kN]	230.3	231.1	183.8	234.5	

Table 2 Numerical fiber section analyses: height of the section compression zone (z_{c_num}), numerical maximum moment (M_{max_num}), numerical maximum base shear (F_{max_num}) and experimental maximum base shear (F_{max_exp}) for the retrofitted (P16-1 and P16-2) and the repaired and retrofitted (R16-SCC-SR-1 and R16-HSF-SR-0) pier specimens; section scheme for fiber model in OPENSEES

7 CONCLUSIONS

Two rapid repair and seismic retrofitting procedures (PR1, PR2) for strongly damaged RC bridge piers are presented and tested by cyclic tests on 1:6 pier specimens. Each procedure uses shaped rebar parts to substitute the damaged parts of the longitudinal rebars and to assure the seismic energy dissipation by steel plastic deformations.

The procedure PR1 restores the damaged concrete parts of the pier by a SCC jacket and the pier retrofit is assured by a CFRP wrapping.

The procedure PR2 restores the damaged concrete parts of the pier and assures the necessary seismic improvement by a UHPFRC jacket with 2% of steel fibers.

The same vertical load and displacements history were applied on two pier specimens to perform cyclic tests on undamaged and retrofitted specimens (P16-1, P16-2). After these tests the damaged pier specimens (R16-SCC-SR-1, R16-HSF-SR-0) were repaired and retrofitted by PR1 or PR2 procedures and tested again applying the same load and displacement history used for P16-1 and P16-2. The first experimental results shown:

- Each procedure can be performed in short time on the pier specimen
- The shaped rebar connection system is not only simple to realize in situ but it is also efficient as no connection ruptures were observed at the end of the tests.
- The repaired specimens R16-SCC-SR-1 and R16-HSF-SR-0 did not show shear rupture and so the two different retrofitting techniques are efficient. However, the procedure PR2 assures the seismic improvement of the pier by means of the UHPFRC jacket only and so it permits time and cost saving as the stirrups and the CFRP wrapping are not applied in the repaired zone.

- The maximum base shear of the pier specimen R16-SCC-SR-1 repaired by PR1 is smaller than the one of the undamaged pier P16-1 retrofitted with the same CFRP wrapping. This is due to the new shaped rebar parts with a reduced diameter used for the specimen R16-SCC-SR-1. It is evident also by OPENSEES fibre section numerical analyses. However, the deformations of the shaped rebars and the confinement of the concrete due to the CFRP wrapping increase the ductility of the pier and so the pier energy dissipation capacity: the design force may be reduced.
- The maximum base shear of the pier specimen R16-HSF-SR-0 repaired by PR2 without CFRP wrapping is similar to the one of the undamaged P16-2 retrofitted by a wrapping built with two layers of CFRP. The high compressive strength of the UHPFRC reduces the height of the section compression zone and so the section inner lever arm, the maximum moment and the corresponding maximum base shear result greater. It is also showed numerically by OPENSEES fibre section analyses.

The repair and retrofitting interventions will be tested and upgraded to consider also the effects of the nonsynchronous seismic action [35]-[40], which can be significant in case of long structures as bridges. This issue is much important as existing bridges are usually designed without considering the effects of the nonsynchronous action.

The stainless steel that is much more resistant to corrosion than ordinary carbon or alloy steels and has high values of ductility ([15]-[20], [44] and [45], [46]) can be a valid material to perform the rebar substitution ([4], [5]) and so it will be considered for future improvement of the repair operations.

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