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Seismic Rehabilitation of RC Columns Under Biaxial Loading: An Experimental Characterization

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ABSTRACT

The main purpose of this paper is to present an experimental campaign of different strategies for seismic repair and retrofit of RC columns previously subjected to biaxial cyclic loading actions and evaluate the benefits concerning their structural behaviour when again subjected to biaxial cyclic loading. Column rehabilitation was performed in order to improve the ductility and/or strength characteristics and it was obtained through increasing the concrete ductility conditions, with efficient jacketing, or increasing the amount of longitudinal and transversal steel. The aim is, therefore, to contribute in developing and calibrating a procedure that enables the evaluation of the efficiency of different retrofit solutions, their possibilities and fields of application. The results are presented in terms of shear-drift, stiffness degradation, ductility and energy dissipation. The retrofitted results will be compared with the results of the original one, deducting about the structural efficiency by each type of retrofit technique adopted.

1. Introduction

The response of reinforced concrete (RC) elements under biaxial (2D) cyclic bending moment is recognized as a very important research topic for building structures in earthquake prone regions, but experimental research work on the inelastic response of RC members under compression axial force and 2D lateral cyclic bending loading conditions is currently very limited [1]. However some findings became commonly accepted and, besides the expected significant influence of axial loads on the hysteretic response of columns, the 2D transversal load cycles are found to be responsible for increased degradation of stiffness and strength, when compared to the 1D response. In addition, the failure mechanism of RC columns shows very dependent of the loading path and strongly affects both the ductility and energy dissipation capacity of the columns [2–4].

Concerning the behaviour of repaired, retrofitted or strengthened RC columns under 2D loading, the lack of results is even more evident, for which the present knowledge is still much behind that that one for 1D bending. Large number of RC columns were designed according to the practice that not taking into account the importance of plastic deformation and ductility capacity, is commonly deficient in flexural ductility, shear strength and flexural strength under strong seismic excitations. Lap slices in critical regions, premature interruption of longitudinal reinforcement and lack of lateral confinement are common practices causing such deficiencies [2].

The local modification of isolated components of structural and non-structural systems seeks for better deformation capacity of deficient components so that they will not reach their limit state as the building responds at the required level. Local and properly combined intervention techniques applied to a group of structurally deficient members may be used to obtain the desired behaviour for seismically designed structures [5–8].

Several experimental studies about the application of retrofit techniques to improve the column behaviour and repair of previously damaged columns have been developed to date. Some of them aim at improving concrete confinement, which stems from the well-known fact that confinement enhances the strength and, more importantly, the ductility of RC columns [9–15].

This work presents an experimental campaign composed of 6 RC columns that were tested under different loading histories, in order to evaluate the influence of the biaxial loading in the cyclic response of the columns. After that, four of the tested columns were repaired and submitted to different retrofit strategies in order to replace the original characteristics, and mainly to provide the columns a good ductility capacity to respond well under cyclic loads. The retrofit techniques adopted in the present work were: increasing the number of stirrups, steel plate jacketing and Carbon Fibre Reinforced Polymer (CFRP) sheets and plate jacketing. After this retrofit process all of the 4 RC

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columns were biaxial tested. The results are presented in terms of shear-drift, shear drift envelopes, ductility, energy dissipation and stiffness degradation and are compared with the results of the original one, deducting about the structural efficiency by each type of retrofit technique adopted.

2. Experimental procedure

2.1. Specimen description and experimental test setup

The specimens have a height of 1.70 m, and are cast in strong square concrete foundation blocks with dimensions of 1.30 × 1.30 m^2 in plan and 0.50 m high. The cross-section dimensions and the reinforcement detailing are presented in Fig. 1a. Four holes are drilled in the foundation block to fix the specimen to the laboratory strong floor. With the cantilever model it is assumed that the inflection point of a 3.0 m height column is located at its mid-height (1.5 m), representing the behaviour of a column at the base of a typical building when subjected to lateral demands induced by earthquakes. An extra 0.20 m height is added for attaching the actuator devices.

The test setup is illustrated in Fig. 1b. and it can be seen that the system includes two independent horizontal actuators to apply the lateral loads on the column specimen. A vertical actuator was used to apply the axial load. Since the axial load actuator remains in the same position during the test while the column specimen laterally deflects, a sliding device is used (placed between the top-column and the actuator), which was built to minimise spurious friction effects.

2.2. Loading condition

This experimental campaign, as already mentioned before, was composed of two different stages: i) six rectangular original and undamaged RC columns were constructed with the same characteristics and reinforcement detailing and were monotonic and cyclically tested for different loading histories. The first 3 original undamaged columns were submitted to monotonic uniaxial tests (PC01_N01, PC01_N02 and PC02_N03). The columns PC01_N01 and PC01_N02 were tested in the strong direction of the column (with different levels of axial load) and PC02_N03 in the weak direction of the column (see Table 1 and Fig. 2). Then the three remaining columns were submitted to cyclic biaxial lateral displacements with the diagonal load path of 30°, 45° and 60° (PC12_N04, PC12_N05 and PC12_N06 respectively) and were imposed at the top of the column with steadily increasing drift levels. These diagonal load paths were adopted in order to characterize the influence of the horizontal load path direction in the response of RC columns. ii) Four of the columns previously tested were repaired and retrofitted and were biaxial cyclically tested for the same loading history. The four repaired and retrofitted RC columns belonging to the second stage (PC12_N03R, PC12_N04R, PC12_N05R and PC12_N06R) were tested under the diagonal 45° biaxial loading path in order to evaluate the efficiency of each retrofit technique and also to compare these results with the PC12-N05 "as built" column that was tested to the same loading history.

In order to characterize the response of the column specimens, cyclic lateral displacements were imposed at the top of the column with increasing demand levels. Three cycles were applied for each of the following peak displacements: 3, 5, 10, 4, 12, 15, 7, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, and 80 mm. However, several times it was not possible to achieve the cycles of larger amplitude due the damage reached in the specimens.

3. Influence of the biaxial horizontal loading direction in the RC column behaviour — stage 1

The shear-drift diagrams (along the X and Y directions) of the stage 1 RC columns are presented in Figs. 3 and 4. Through the shear-drift results of the monotonic uniaxial loading displacement path in X (Fig. 3a), the difference between the two RC columns submitted to different axial loads values was observed. Both of the columns have similar initial stiffness but PC01-N02 shows more than 10% of maximum strength than the PC01-N02.

The shear-drift diagrams for the columns under biaxial horizontal displacement loading for each respective direction are presented in Fig. 4. It was also plotted for each direction the respective result of columns PC01-N01 (direction X) and PC02-N01 (direction Y) with the main purpose of evaluating the effect of the biaxial load in the behaviour of RC columns. The maximum strength of the biaxial tested columns is always lower than observed in the uniaxial tests, about 10–20% less in the strong direction (X) and mainly about 15–40% less in the weak direction (Y).

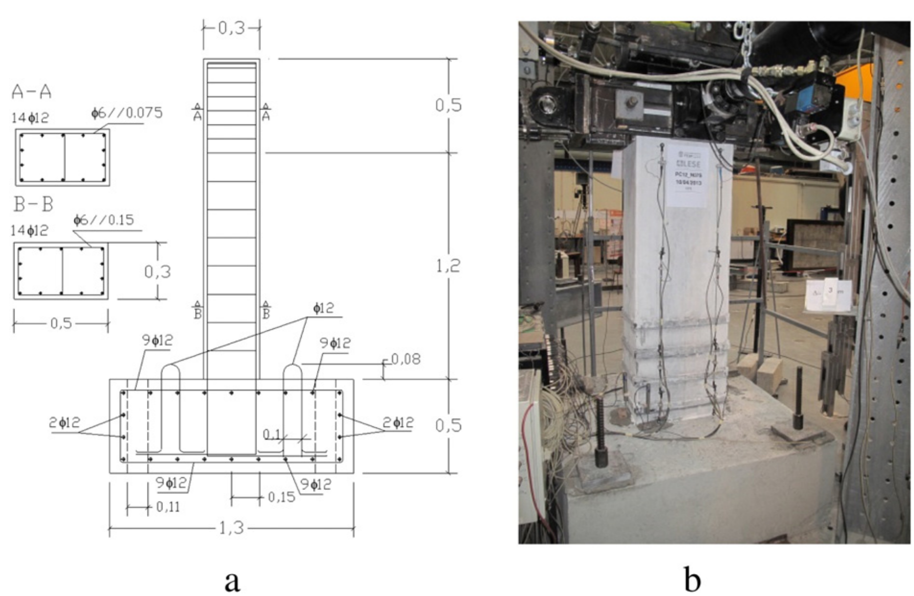


Fig. 1. a) RC column specimen dimensions and reinforcement detailing: b) General view of the test setup at LESE Laboratory.

Table 1
Specimens' specifications, loading characteristics and retrofit technique.

Stage	Specimen	Geometry [cmxcm]	f_{cm} [MPa]	f_{ym} [MPa]	Axial load [kN]	Horizontal displacement path type	Retrofit technique
Stage 1	PC01-N01	30 × 50	23.3	478	300	Uniaxial — strong (monotonic)	Original "As built" columns
	PC01-N02				600	Uniaxial — strong (monotonic)	
	PC02-N03				300	Uniaxial — weak (monotonic)	
	PC12-N04					Diagonal — 30° (cyclic)	
	PC12-N05					Diagonal — 45° (cyclic)	
	PC12-N06					Diagonal — 60° (cyclic)	
Stage 2	PC12-N03R				300	Diagonal — 45°	Increase the transversal reinforcement
	PC12-N04R					(cyclic)	Increase the transversal reinforcement + CFRP jacketing
	PC12-N05R						Increase the transversal reinforcement + CFRP plate jackting
	PC12-N06R						Increase the transversal reinforcement + steel plate Jacketing

In order to evaluate the effect of the different biaxial horizontal displacement load directions in the response of the RC columns, the shear-drift envelopes are presented in Fig. 5. The ratio between the columns submitted to uniaxial loading (PC01-N01 in direction X and PC02_N03 in direction Y) and the columns submitted to biaxial horizontal loading (PC12-N04, PC12-N05 and PC12-N06) in terms of initial stiffness and maximum strength is illustrated in Fig. 6.

As expected, the maximum strength values in one specific column's direction were lower for all biaxial tests comparatively with those for the corresponding uniaxial tests (with the same axial load). The biaxial loading induces a reduction in the stronger direction, X, between 5% and 30%, increasing with the angle of diagonal loading direction. Regarding the weaker direction, Y, the reduction is between 25 and 65%. The strength reductions are more pronounced in the weak direction for a horizontal load path angle of 30° (PC12-N04) and in the strong direction for the 60° horizontal load path (PC12-N06) angle.

For the tested columns, the initial column stiffness in each direction was not significantly affected by the biaxial load path but otherwise the post-yielding plateau of the biaxial tests tends to be 10–15% more reduced, when compared with the uniaxial tests, and the softening was more pronounced, with more evidence decay of the column strength.

3.1. Columns' ultimate ductility

The procedure used in this work for the determination of the yield displacement is based on the method proposed by Park [16], but adapted in order to be used with the results of the complete cyclic test, while considering the possible hardening of the post-yield response. The procedure adopted is described as follows and illustrated in Fig. 7:

1) Evaluation of the maximum strength of the specimen in both test directions;
2) Identification of the cycle for which the strength is lower than three quarters of the previously evaluated maximum strength of the specimen;
3) Calculation of the secant stiffness (K_y) for the cycle identified in step 2;
4) Adjustment of the branch corresponding to the post-yield stiffness (K_{pl}).

Determination of the intersection points of K_y and K_{pl}, for each direction, leading to the yield displacement in each direction. The average of the yield displacements obtained in the two directions (d_y^+ and d_y^-) provides the reference yield displacement (Δ_y).

The method adopted to evaluate the yield displacement has a clear advantage in this analysis (in comparison to estimation by simplified formulae) because it is based on experimental test results.

The ductility was determined with the yield displacement determined as shown in Fig. 7, and the ultimate displacement was determined when the applied load drop until 80% of the maximum load registered during the test.

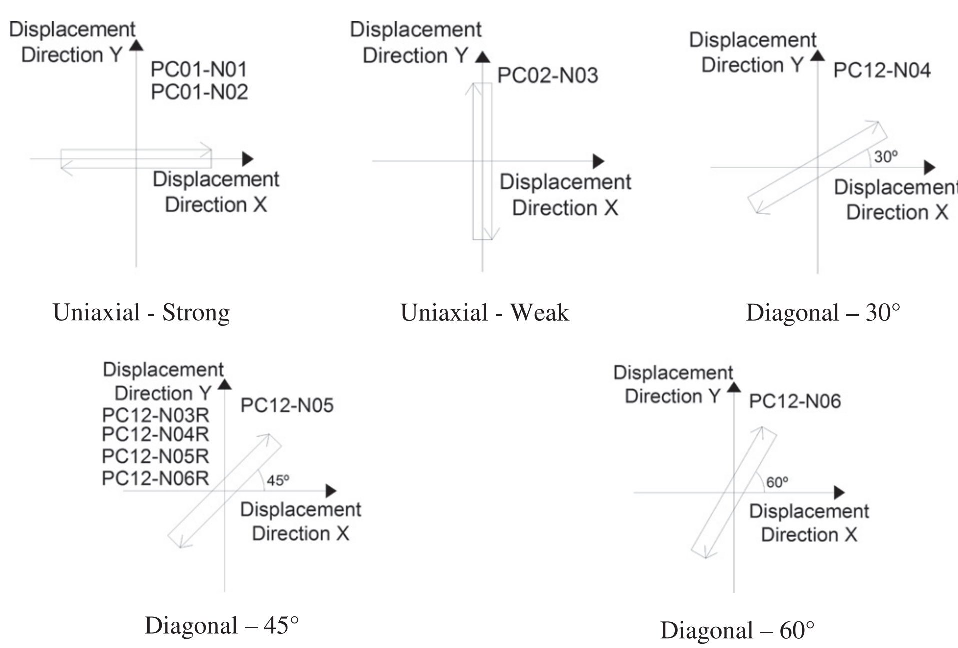


Fig. 2. Horizontal displacement path type.

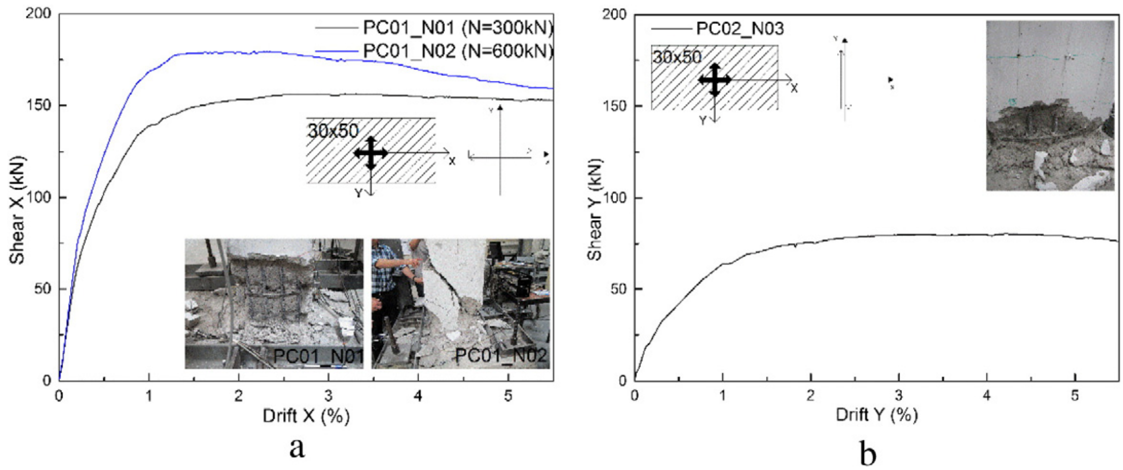


Fig. 3. Shear-drift diagrams for the columns a) PC01_N01 and PC01_N02 under uniaxial horizontal strong (with different axial load) b) PC02-N03 under uniaxial weak load path — Stage 1.

From the analysis of the maximum ductility values from the tests results performed in stage 1, presented in the shear-drift envelopes (Fig. 5) and in Table 2, the ratio between the ductility of the columns submitted to uniaxial loading and the columns submitted to biaxial horizontal loading illustrated in Fig. 8 was determined. It was observed that biaxial loading results in lower ultimate column ductility than does uniaxial loading (PC01_N01, PC01_N02 and PC02_N03). Amongst the imposed biaxial loading paths, the 30° diagonal path (PC12_N04) produces the lowest ultimate ductility. The loading in string direction yields ductility reductions of approximately 50–75% relative to those in the corresponding uniaxial case that are greater than in the weak direction (30–50%).

4. Repair and retrofitting of previously tested RC columns

4.1. Repair process of damaged columns

After the cyclic test of the as built specimens took place up to failure, four specimens were repaired and retrofitted (PC02_N03, PC12_N04, PC12_N05 and PC12_N06 which will be designed with the letter “R” at the end in this stage 2). The repair and retrofitting processes consisted of internal reconstruction of the damaged zone (as described next), with increase of the transversal reinforcement by shortening the spacing to 0.075 m in all of these 4 columns. Externally, these specimens were retrofitted with three different techniques namely: CFRP sheet jacket; CFRP plate's jacket; and steel plate's jacket (see Table 3).

The repair process of the tested columns has involved a series of procedures, which were based on previous research works that have been performed in the Laboratory of Earthquake and Engineering (LESE) [18–20], and followed the next 6 steps:

1) Delimitation of the repairing area (the critical section at the plastic hinge region), typically from the footing up to 50 cm along the column height);
2) Removal and cleaning of the damaged concrete (Fig. 9a);
3) Alignment and replacement of the longitudinal reinforcement bars, which required cutting short pieces (few cm) of the buckled and failed rebars in order to ensure their alignment. Additional bars were welded to the existing ones to ensure lap splicing as described in more detail below (Fig. 9b and c);
4) Replacement of the transversal reinforcement with the half of the initial space of 0.075 m (Fig. 9d);
5) Application of formwork and new concrete (a pre-mixed micro-concrete, modified with special additives to reduce shrinkage in the plastic and hydraulic phase; not shown) (Fig. 9d to f);
6) Application of the retrofitting technique.

4.2. Retrofit of repaired RC columns and design criteria

In poorly confined columns, which are expected to be modified so as to withstand large rotations on the plastic hinges, the priority should be increasing their ductility capacity. Inelastic deformation capacity of flexural plastic hinge regions can be increased by confining the RC column concrete with a composite fibre or steel jacketing system, which is quantifiable by the volumetric confinement ratio referred below. RC column jacketing can be designed according to the several criteria found in the literature. In this work, following the experience of previous studies [21–23], the proposed criteria by Priestley et al. [12,24] and the Monti et al. proposal of [25] were adopted.

Both proposals can be used for jacketing with CFRP sheets and plates and steel plates. For the jacketing with steel plates, the design can be done in the same way, by introducing the necessary adaptation to specific details, namely a proper distribution of the calculated amount of material for the plates that materialize the reinforcement. In Sections 4.2.1, 4.2.2 and 4.2.3 it will be presented all of the design criteria adopted in the present study, and several observations and comments in order to help understand all of the retrofit processes.

4.2.1. CFRP sheet retrofit

The Priestley et al. approach [12,24] was used to calculate the jacket thickness of the jacket for rectangular columns. This proposal recommends the procedure which relates the volumetric confinement ratio with the required plastic rotation. The jacketing thickness t_j, for the case of composite materials, can be obtained from:

$$t_j = \frac{0.4(\varepsilon_{cu} - 0.004)f'_{cc}}{f_{uj}\varepsilon_{uj}}\left[\frac{bh}{b+h}\right] \tag{1}$$

where ε_{cu} is the ultimate compressive strain, ε_{uj} and f_{uj} are the composite strain and stress of the composite reinforcing material, f_{cc}' is the confined concrete strength, and b and h are the columns' cross section width and depth respectively.

For steel jacketing solution, there's a slight difference between these expressions in particular the jacketing thickness can be obtained by Eq. (1) replacing the value 0.4 by 0.357 and the parameters f_{uj} and ε_{uj}, respectively, by f_{yh} and ε_{su}.

For the present study, the CFRP sheet jacketing was designed taking into account the following properties, namely: the CFRP ultimate strain ε_{uj} = 0.0155 and strength f_{uj} = 3800 MPa, the layer thickness t_{jl} =

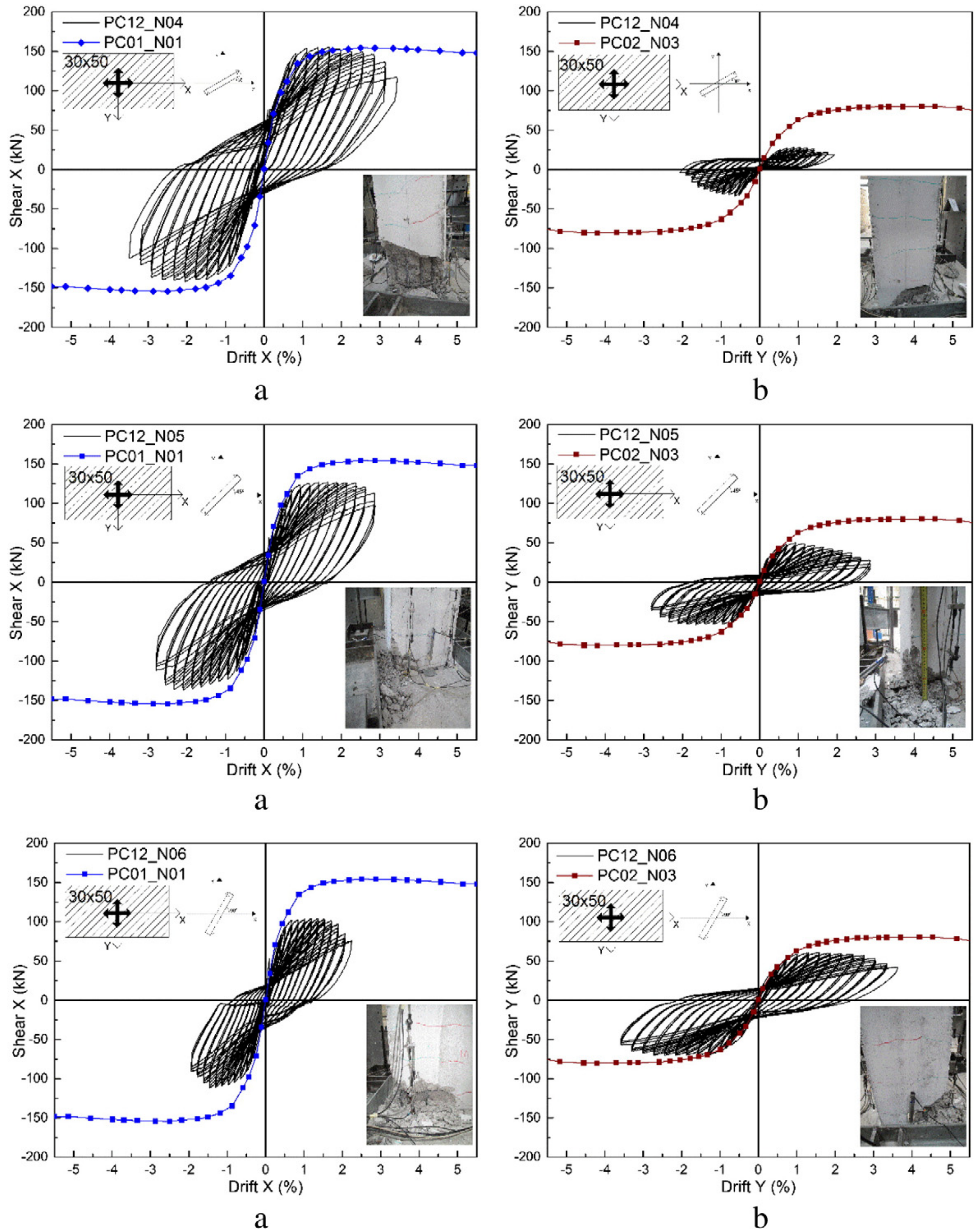


Fig. 4. Shear-drift diagrams for the columns under 30°, 45° and 60° diagonal biaxial horizontal load paths (PC12-N04 to PC12-N06) and with the shear-drift results for monotonic load path (PC01-N01 and PC02-N03). a) Strong direction b) weak direction — Stage 1.

0.176 mm, as well as the cross-section geometry and reinforcement of the column specimen PC12_N04 previously tested and repaired using micro-concrete with 50.74 MPa compressive strength.

The required plastic rotation θ_p of the plastic hinge was obtained by dividing the target displacement of $\Delta = 50$ mm by the column height. Then, the plastic curvature ϕ_p was found assuming the gap distance $g = 0$ and the longitudinal reinforcement data for the plastic hinge length L_p estimation. The maximum required curvature ϕ_m was obtained by Eq. (2), where the equivalent bilinear yield curvature (ϕ_y) was estimated from moment–curvature analysis which also provided an estimate for the neutral-axis depth c to allow computing the maximum required compression strain.

$$\phi_m = \phi_y + \phi_p \tag{2}$$

Through the transverse reinforcement data, the lateral confining pressure f_l was determined (assuming $K_e = 0.75$), as well as the confined concrete strength f_{cc}', where $f_{co}' = 50.74$ MPa. Finally, assuming $\varepsilon_{cu} = \varepsilon_{cm}$, the required total jacketing thickness $t_j = 0.389$ mm was

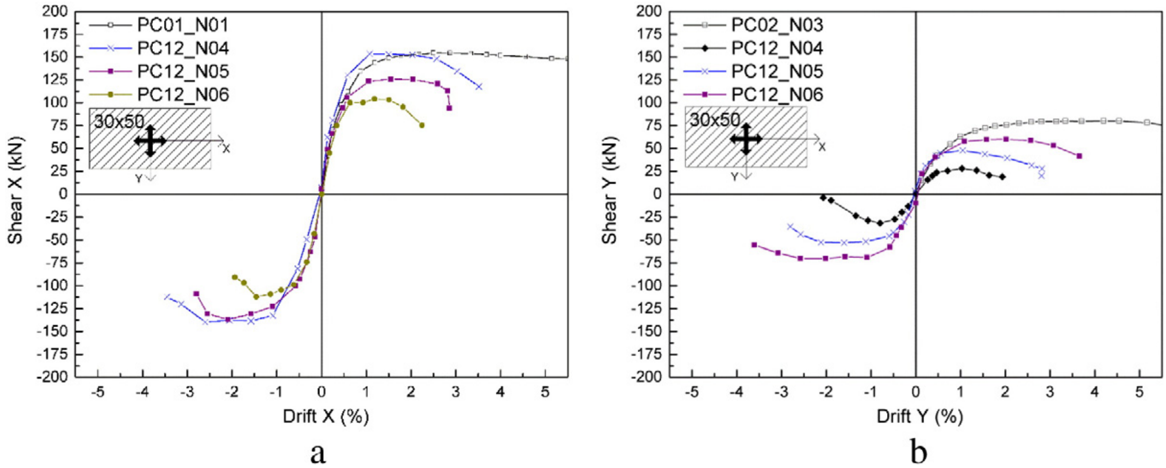


Fig. 5. Shear-drift envelopes for different horizontal load paths a) strong direction b) weak direction — Stage 1.

obtained, which was materialized by adopting 4 layers, 0.176 mm each one, of CFRP sheets.

The retrofitted region with the CFRP confinement extends over the distance equal to the larger value between the cross section depth *h* on the horizontal loading direction and the product of 0.25 × L, where L is the column length. In this case, a retrofitted height $L_j = 0.50$ m was adopted, after having checked that all the conditions for applying jacketing with composite materials have been met. Fig. 10 shows the general view of the CFRP sheet jacketing scheme and the implementation in the specimen ready for the PC12-N04R column testing.

4.2.2. CFRP plate retrofit

In order to achieve the same target displacement mentioned at the first technique (CFRP sheet jacketing), the column specimen PC12-N05R was retrofitted with CFRP plates after a previous repair procedure. The CFRP plates were designed following the Monti approach [25], and after calculating the CFRP jacket thickness, it was multiplied by the jacket length obtaining the total area per face. To reduce this jacket area into plates, a fixed width was chosen leading to a new jacket thickness. The following parameters, presented in Table 4 are required for the Monti et al. [25] approach:

According to the Monti et al. approach, the section upgrading can be expressed by the index $I_{\text{sec}} = \delta_\chi^{tar}/\delta_\chi^{ava}$, which for the present case is taken equal to 1, once the main objective of the columns' repair was to reestablish the columns performance when subjected to biaxial horizontal loadings.

Considering the previously described CFRP properties and the corresponding characteristic value of the elastic modulus $E_{fk} = 240{,}000$ MPa, the design procedure can be summarized in the following steps:

- Calculation of the transverse reinforcement ratio: $\rho_{st} = \frac{4A_{st}}{s\,d}$
- Definition of parameters for the selected material (with $\gamma_f = 1.5$):

design elastic modulus: $E_j = 0.9\,E_{fk}$

ultimate design strain calculated by Eq. (3):

$$\varepsilon_{ju} = \min\left(0.9\frac{f_{fk}}{\gamma_f E_f}, 0.9\frac{\varepsilon_{fu}}{\gamma_f}\right) \tag{3}$$

- Calculation of confinement pressure required from CFRP jacketing:

Confinement pressure provided by the stirrups (Eq. (4))

$$f_l = \frac{1}{2}K_e\ \rho_{st}\ f_{yh} \tag{4}$$

assuming $K_e = 0.80$

Concrete confinement stress due to the stirrups (Eq. (5))

$$\frac{f'_{cc,st}}{f'_{co}} = 2.254\sqrt{1+7.94\frac{f_l}{f'_{co}}} - 2\frac{f_l}{f'_{co}} - 1.254 \tag{5}$$

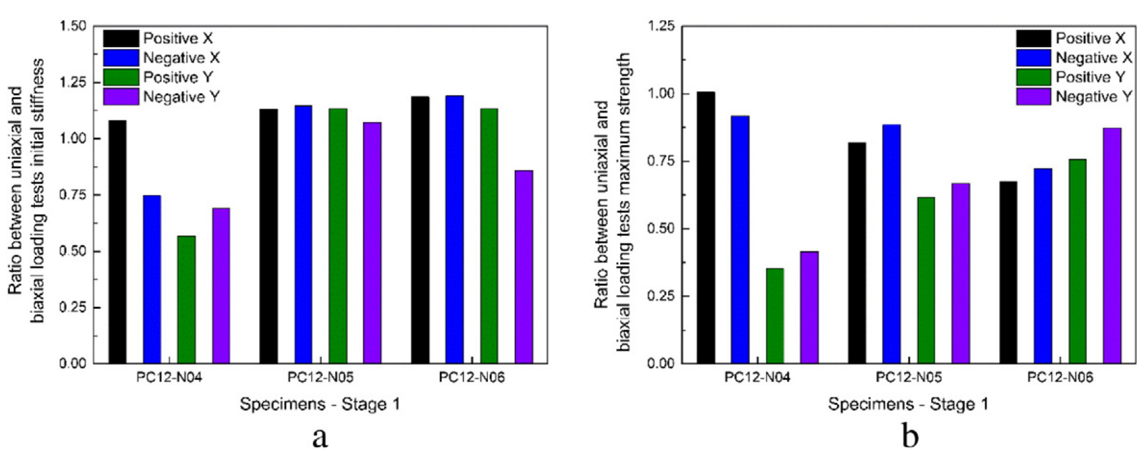


Fig. 6. Ratio between uniaxial and biaxial loading tests a) initial stiffness b) maximum strength — Stage 1.

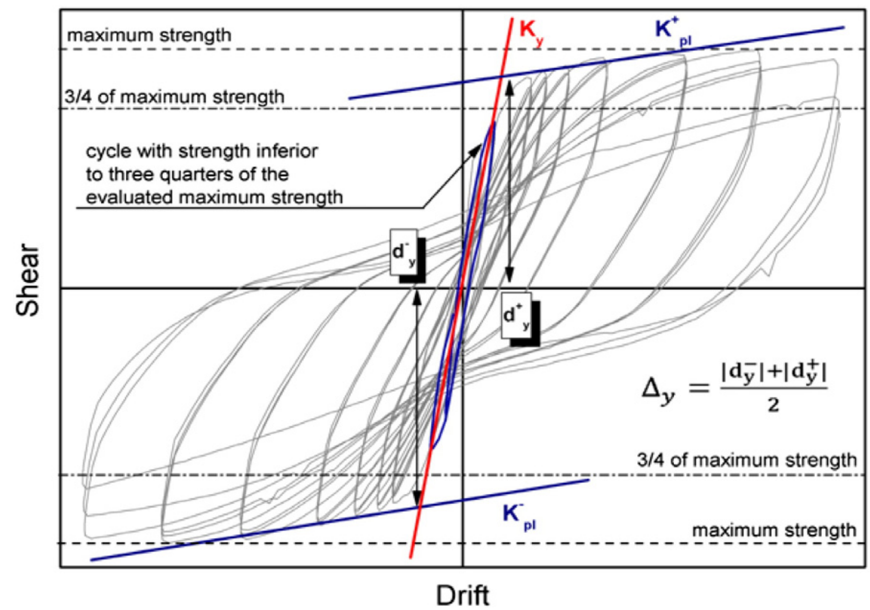


Fig. 7. Proposed method for the definition of yield displacement [17].

Ultimate strain of the concrete confined by stirrups (Eq. (6))

$$\varepsilon_{cu,st} = 0.004 + \frac{1.4\rho_{st}\ f_{yh}\ \varepsilon_{su}}{f'_{cc,st}} \tag{6}$$

Required confinement pressure due to CFRP jacketing (Eq. (7))

$$f_l = 0.4I^2_{\text{sec}}\frac{f'_{cc,st}\ \varepsilon^2_{cu,st}}{\varepsilon^{1.5}_{ju}}. \tag{7}$$

– Finally, for rectangular section columns the required jacket thickness can be solved from Eq. (8), where d_j is the largest cross section dimension:

$$t_j = \frac{f'_l}{2}\frac{d_j}{E_j\varepsilon_{ju}}. \tag{8}$$

Considering the same properties adopted for the specimen PC12_N04R design, namely the concrete compressive strength in the repaired region ($f_{co}' = 50.74$ MPa) and the mentioned elastic modulus ($E_{fk} = 240{,}000$ MPa), the total thickness of CFRP jacketing $t_j = 0.342$ mm was obtained for the total repaired zone height (500 mm).

Table 2
Summary of test results for columns PC01_N01 to PC12_N06 — Stage 1.

Specimen	Loading direction		F_{max} [kN]	Δ_y [mm]	F_y [kN]	$\mu = \Delta_u/\Delta_y$
PC01_N01	X	+	154.6	11.7	121.3	7.69
PC01_N02	X	+	179.4	11.3	145.6	7.96
PC02_N03	Y	+	80.3	16.3	71.8	5.18
PC12_N04	X	+	155.5	7.1	120.3	2.81
		−	141.8	7.8	111.5	2.32
	Y	+	28.3	10.3	20.4	4.68
		−	33.4	12	26.6	3.875
PC12_N05	X	+	126.6	10	98.7	4.38
		−	136.8	10	110.4	4.25
	Y	+	49.5	5.3	34.9	6.04
		−	53.7	7.3	35.4	5.41
PC12_N06	X	+	104.2	7.3	78.2	4.25
		−	111.9	8.0	79.6	3.75
	Y	+	60.7	13.3	48.5	3.58
		−	70.2	7.3	52.1	6.45

The solution composed by 6 CFRP plates with 80 mm width, spaced at 70 mm, each one with 6 layers of CFRP sheet thickness of 0.176 mm, having a 40 mm gap between the footing and the first plate was adopted. Fig. 11 shows a general schematic view of the CFRP plates jacketing for specimen PC12_N05R.

4.2.3. Steel plate retrofit

In order to achieve the same target displacement above mentioned (for the cases of CFRP sheets retrofit), the specimen PC12_N06R was retrofitted with steel plates after a previous cyclic test. The steel plates were designed following the Priestley approach for steel jacket [12, 24]; each steel plate thickness and width was then calculated as for the CFRP plate jacketing.

The design procedure can be summarized in the following steps:

– Using the Priestley [12,24] approach as described in Section 4.2.1, the thickness of the steel jacket is obtained by Eq. (1);
– For easier comparison, the steel jacket was assumed with the same height as the CFRP jacket, i.e., 500 mm.
– A total area of the steel jacket per face was obtained; this area was divided by the number of steel strip plates (four plates were adopted

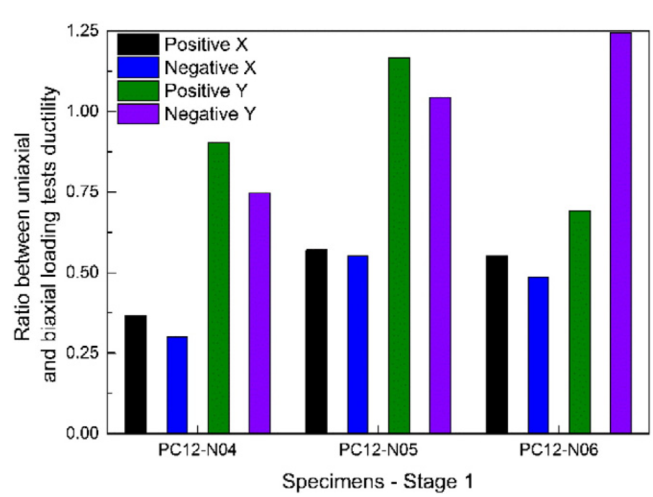


Fig. 8. Ratio between uniaxial and biaxial loading tests ductility — Stage 1.

Table 3
Specimen's retrofit techniques.

Specimen	Retrofit technique
PCN03R	Increase the transversal reinforcement
PCN04R	Increase the transversal reinforcement + CFRP sheet jacketing
PCN05R	Increase the transversal reinforcement + CFRP plate jacketing
PCN06R	Increase the transversal reinforcement + steel plate jacketing

for retrofitting the column PC12_N06 and some of the original/strengthened; however tests have shown that the upper plate was useless and therefore it was not included in the last cases);

– Finally, fixing the plate width (30 mm in the present case), a new thickness of the steel plates was obtained and fixed in 5 mm.

Considering the same material properties for concrete in the repair zone as for the previous cases, the yielding steel strength of 275 MPa and the ultimate strain of 0.015, four steel plates with $30 \times 5mm^2$ dimensions were adopted.

The steel plates were L-shape folded, bonded to the column with epoxy resin and welded in-situ in two corners to complete the collar. The plates were placed in four previously defined levels at increasing distances from the footing. After welding, the voids between the plates and the concrete were filled with injection of two component epoxy resin in order to ensure full contact and early efficiency of the

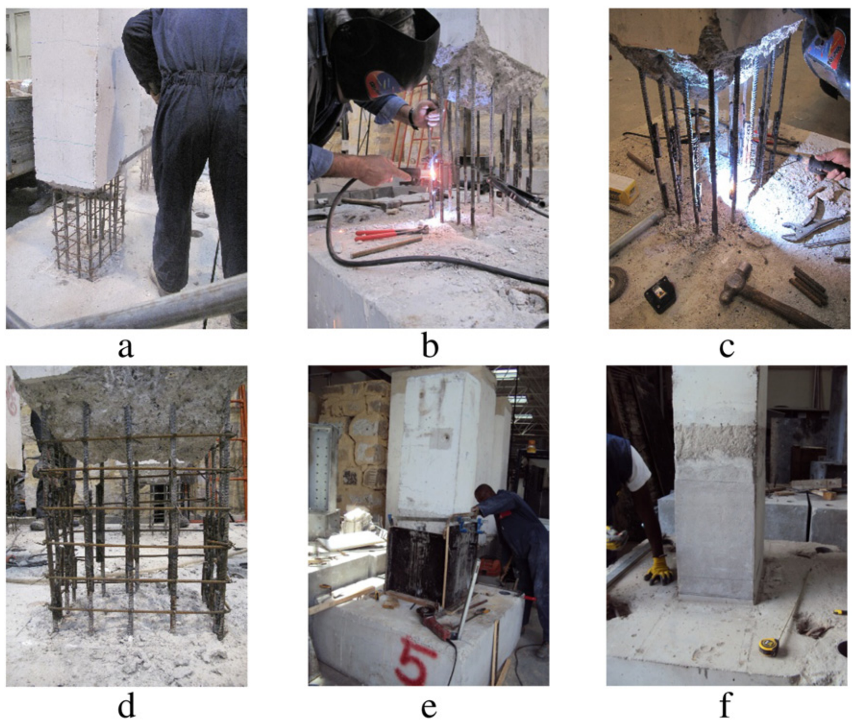


Fig. 9. Repair and retrofit processes of damaged columns: a) Removal and cleaning of the damaged area b) repair of the longitudinal reinforcement c) replacement of the transversal bars d) CFRP column jacketing e) CFRP column plate jacketing f) steel column plate jacketing.

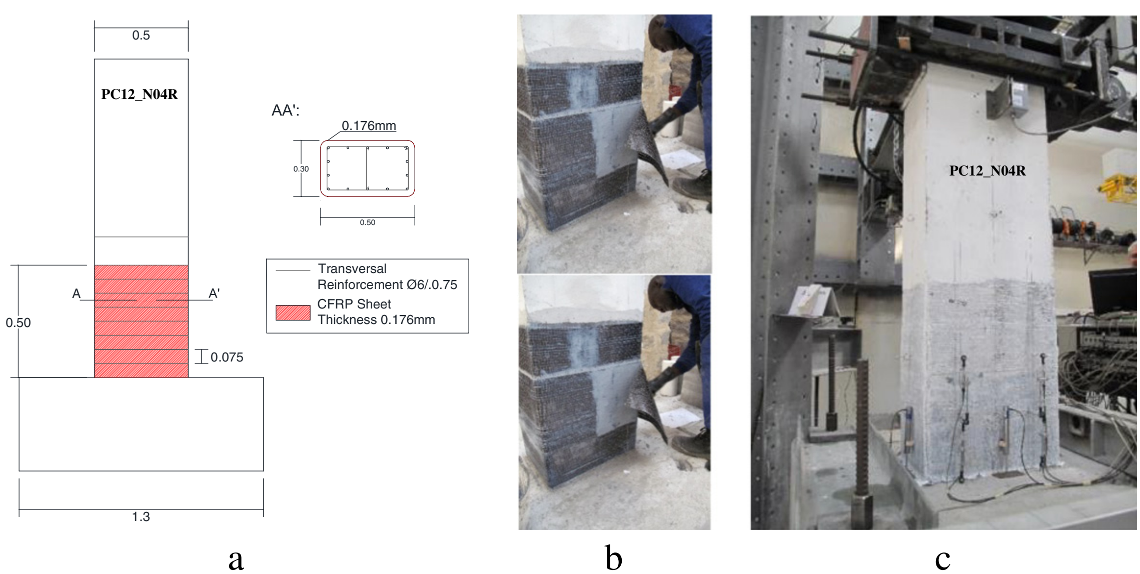


Fig. 10. Column specimen retrofitted with CFRP sheet jacketing PC12_N04R a) schematic layout of the retrofitted column design b) retrofit process of the RC column) actual column PC12-N04R ready for testing.

Table 4
Specimen's retrofit techniques.

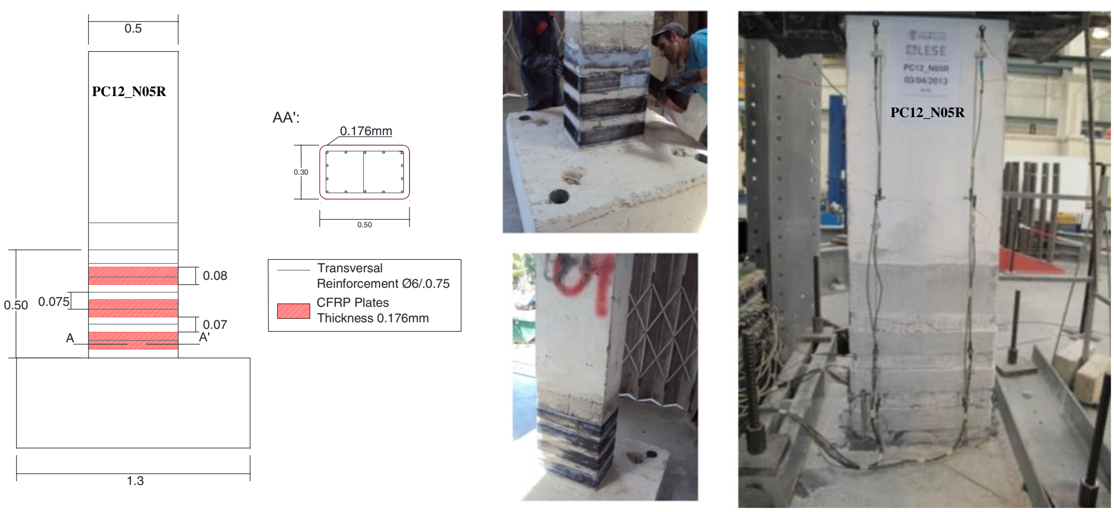
Concrete and steel	CFRP
f_{cc}' — peak compressive strength of unconfined concrete f_{sy} — tensile strength of longitudinal reinforcement f_{yh} — tensile strength of transverse reinforcement (stirrups) A_{st} — transverse reinforcement area -s — stirrups spacing For the column concrete and steel: d_s — diameter of confined concrete core	f_{fk} — characteristic value of tensile strength E_{fk} — characteristic value of elastic modulus ε_{fu} — ultimate design strain γ_f — partial safety factor $I_{sec} = \frac{\delta_\chi^{tar}}{\delta_\chi^{data}}$ - index for section upgrade

external strengthening. Fig. 12 shows the general schematic view of the specimen retrofitted with steel plates jacketing.

5. Cyclic test of the retrofitted specimens — stage 2

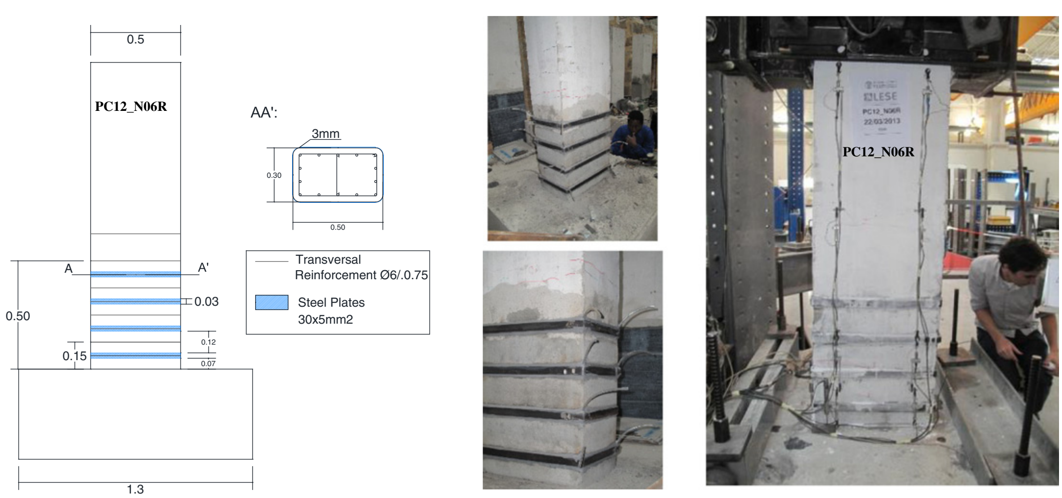
In order to evaluate the retrofitting technique efficiency in the behaviour of RC columns, the measured displacement and shear force paths (along the X and Y directions) are presented in Fig. 13. For a better comparison, the original columns tested under the same biaxial load path (PC12_N05) are also plotted. The envelope of all retrofit columns and the original tested column is illustrated in Fig. 14 in order to simplify the comparison, and the results can be summarized as follows:

- From the observation of the shear-drift curves of the retrofitted columns, the three main branches can be identified in their envelopes. However, in the retrofitted tests the plateau, after yielding, tends to be larger and the softening starts for higher drift deformation (see Fig. 13);
- In both directions, the initial stiffness, in the retrofitted columns, is typically equal or lower when compared with the original column



a b c

Fig. 11. Column specimen retrofitted with CFRP plates jacketing PC12_N05R a) schematic layout of the retrofitted column design b) retrofit process of the RC column) actual column PC12-N05R ready for testing.



a b c

Fig. 12. Column specimen retrofitted with steel plates jacketing PC12_N06R a) schematic layout of the retrofitted column design b) retrofit process of the RC column) RC column PC12-N06R ready for testing.

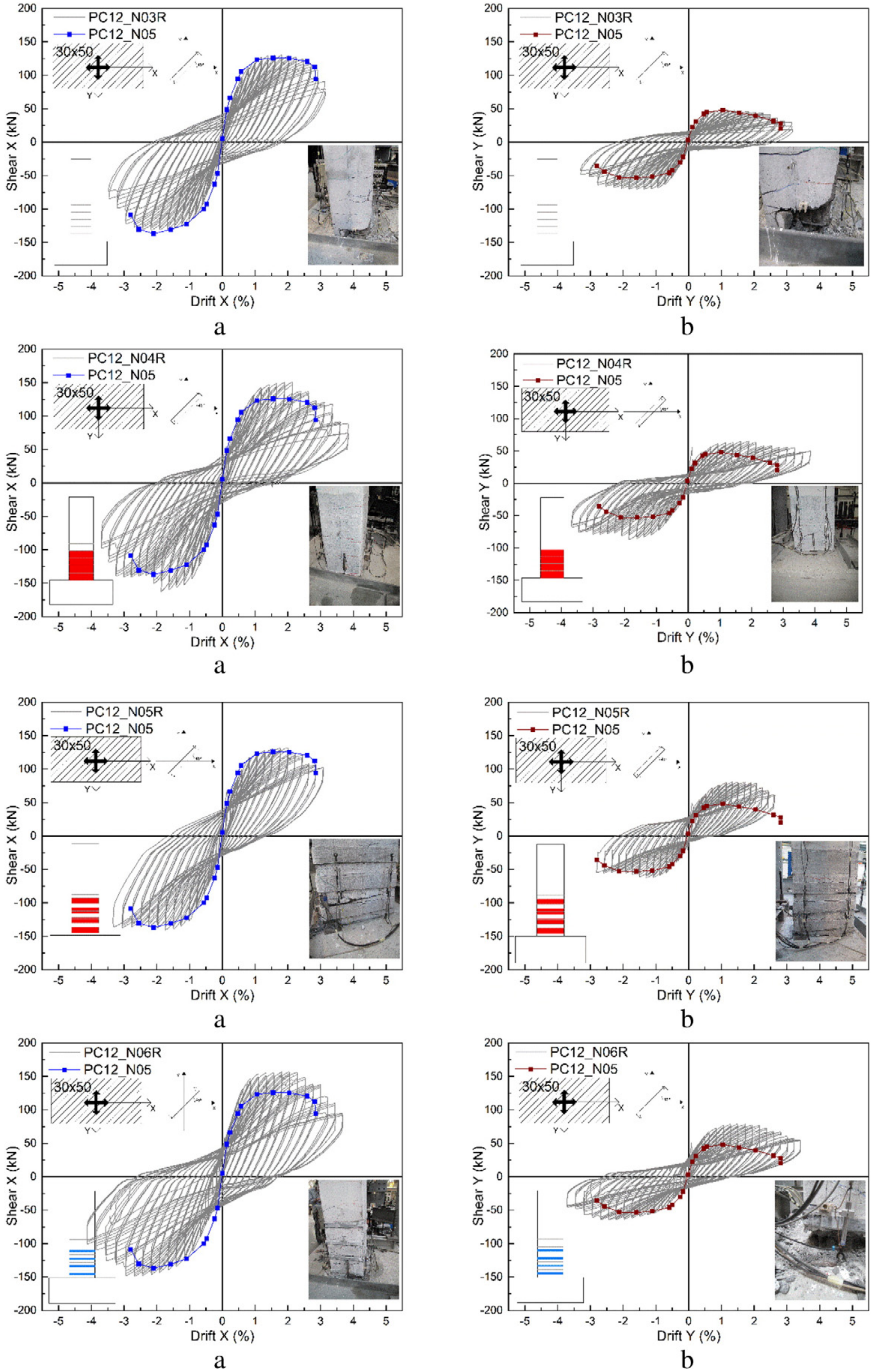


Fig. 13. Shear-drift diagrams of the retrofitted columns under biaxial horizontal load path (PC12-N03R to PC12-N06R) with the "As built" RC column (PC12-N05) a) strong direction b) weak direction.

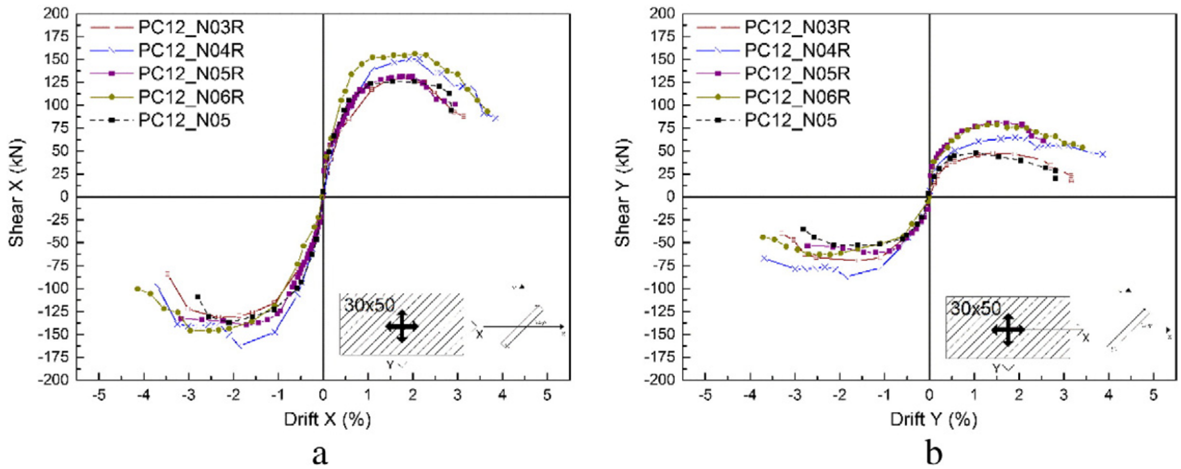


Fig. 14. Shear-drift envelopes of the all retrofitted RC columns (PC12_N03R to PC12_N06R) and the original column (PC12_N05) a) strong direction b) weak direction.

(in majority of the cases in weak direction), as can be observed in Fig. 13, which means that yielding occurs for larger drift demands;

- As for the maximum strength, the three different repair/retrofitting solutions lead to distinct results: i) considering that only the increase of transversal reinforcement (PC12_N03R) yields practically the same maximum strength values; ii) the use of CFRP sheet and plate jacketing (PC12_N04R and PC12_N05R) always leads to larger strength in weak direction than that in the original column (around 20%); for the strong direction just CFRP sheet jacketing increases significantly the maximum strength value iii) the use of steel plate jacketing (PC12_N06R) also leads to increased strength around 10% (see Fig. 14).

5.1. Columns' ultimate ductility

One of the main objectives of the present study was to verify the efficiency of the retrofitting techniques in restoring or if possible improving the columns' ultimate ductility. The yielding displacement and maximum ductility observed in each test are summarized in Table 5.

The obtained results show difficulty in re-establishing the maximum ductility observed in the original tests. A reduction of 20 to 50% can be observed dependent on the retrofitting technique. Column PC12-N03R, was the only that presents lower ductility reductions and PC12_N05R and PC12_N06R show higher reductions.

However, this fact should not be analysed without looking at the ultimate displacement and yielding displacement. Regarding the yielding displacement all the retrofitted columns exhibit larger values: when using CFRP's or steel plates this increase is between 70% and 120%. Concerning the ultimate displacement, the increase is also evident, though not so pronounced as for yielding. When only the column repair is considered, with the increased transversal reinforcement, the ultimate displacement is practically the same, when externally retrofitting is added this increase is around 20 to 50%. Therefore, since ultimate displacement increases less than the yielding one, the obtained ductility becomes lower than that of original columns. This is not the expected outcome, but it does not mean that a bad result was obtained because both the strength and the ultimate deformation capacity were restored and even slightly increased. Thus, the adopted repair and retrofitting were found efficient.

Table 5
Summary of test results of the all retrofitted RC columns (PC12_N03R to PC12_N06R) and the original column (PC12_N05).

Retrofit technique	Specimen	Loading direction		F_{max} [kN]	Δ_y [mm]	F_y [kN]	$\mu = \Delta_u/\Delta_y$
Original "As built"	PC12_N05	X	+	126.6	10	98.7	4.38
			−	136.8	10	110.4	4.25
		Y	+	49.5	5.3	34.9	6.04
			−	53.7	7.3	35.4	5.41
Increase of stirrups	PC12_N03R	X	+	130.1	11.4	100.9	3.87
			−	131.6	12.8	101.2	3.40
		Y	+	47.5	5.3	34.3	7.13
			−	70.4	9.4	52.3	4.47
Increase of stirrups + CFRP sheets	PC12_N04R	X	+	150.8	13	120.3	3.84
			−	162.6	12.4	131.6	3.81
		Y	+	65	9.1	49.8	5.55
			−	87	14.7	63.4	3.56
Increase of stirrups + CFRP plates	PC12_N05R	X	+	141.3	11.5	109.4	3.69
			−	142.8	14.1	122.7	2.99
		Y	+	75.9	9.4	46.8	4.26
			−	65.5	12.1	49.1	2.70
Increase of stirrups + Steel plates	PC12_N06R	X	+	152.4	10.5	122.1	4.49
			−	145.8	16.7	113.4	2.93
		Y	+	55.3	10.0	42.3	4.00
			−	59.2	16.0	47.3	2.975

5.2. Damaged evolution in the tested columns

It is generally accepted that when RC elements are subjected to biaxial horizontal loading the stiffness and strength deterioration is intensified, and consequently the damage evolution of the element is increased or occurs for earlier deformation levels [2,26,27] when compared with the columns subjected to uniaxial horizontal loading.

To observe and record in detail the damage evolution during the cyclic loading, each test was stopped at the end of the last cycle of each displacement level in order to highlight and register new cracks in the last cycle and/or the evolution of existing ones.

Fig. 15 illustrates the damage evolution in the original column PC12-N05 and in the retrofitted columns for the same drift demands. The visual observation of the damage evolution during the tests yielded the following information:

- For all tests, a horizontal crack was observed at the base of the column. This crack is reported in several studies of columns under horizontal loading and it is associated with the yield penetration of the vertical reinforcing steel bars [16,28]. In all the original tests horizontal cracks distributed along the column length, associated with the flexural dominant response of the columns, were observed (see example of PC12-N05 in Fig. 15).
- For each drift demand level, retrofitted columns presents a lower level of damage in the column base when compared with the original one.
- In the retrofitted columns tests, the damage tends to be concentrated in the column base horizontal crack; this is more evident in the column PC12_N06R, where this damage concentration leads to the

Fig. 15. Damage evolution in the columns of the all retrofitted RC columns (PC12_N03R to PC12_N06R) and the original column (PC12_N05).

premature failure and rupture of the longitudinal rebars, while for columns with CFRP and steel plates the damage, although more concentrated than in the original columns, still show spread a bit more above the base.

5.3. Stiffness degradation

The evolution of stiffness degradation was evaluated by comparing the peak-to-peak secant stiffness of the first cycle of each imposed peak displacement. Fig. 16 shows the lateral peak-to-peak stiffness degradation, for each column and in each direction. From previous studies it was clear that the load path history has an irrelevant influence in the stiffness degradation evolution [3,29].

In the present study, for the two independent directions the stiffness degradation evolution of the repaired columns is much different when compared with the original columns tested. The comparisons based on experimental results, allowed concluding that all retrofit techniques had more pronounced strength degradation than the original column, besides column PC12_N03R. Lower stiffness degradation at the same drift demand is observed, comparatively to the original solution, as more evidenced in column PC12_N05R. These results, although indicating better performance of the increase of the transversal reinforcement, do not allow inferring a simple rule for stiffness degradation of retrofitted columns to be considered in the definition of simplified numerical models.

5.4. Energy dissipation

For the present study the results in terms of evolution of cumulative dissipated energy are presented in Fig. 17 for the columns with different

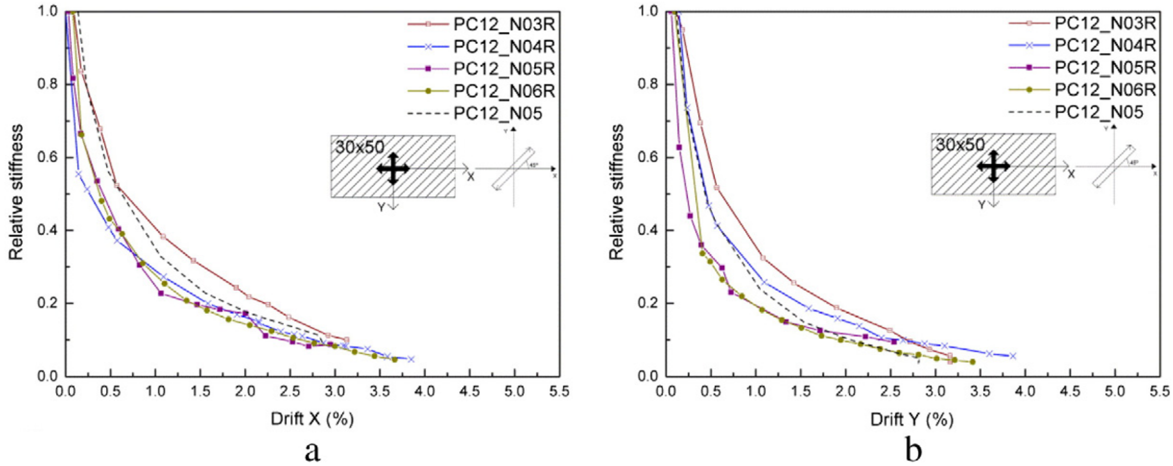


Fig. 16. Stiffness degradation for of the all retrofitted RC columns (PC12_N03R to PC12_N06R) and the original column (PC12_N05) a) strong direction b) weak direction.

repairing solutions. For each displacement amplitude level, the plotted value of dissipated energy corresponds to the end of the third cycle. The results' analysis shows that the repair technique of the damage zoned with increase of transversal reinforcement leads to very similar dissipated energy as the original column. The retrofitting with CFRP's plates also exhibits similar energy dissipation evolution than the CFRP sheet jacketing, slightly less than the original column. By contrast, steel plate jacketing leads to slightly more increase of energy dissipation, for the same drift demand, when compared with the original solution. This increase is clearly evident, when for example, for 2% drift the retrofitted columns have 50% more cumulative dissipated energy than the original column, which is related with the fact that damage is a bit more spread above the base section than for the other solutions.

6. Summary and conclusions

Three monotonic uniaxial tests were carried and three columns were cyclic tested with biaxial lateral displacements, after retrofit interventions according to different techniques were implemented in four of the tested columns. The experimental results on the original columns show that the biaxial loading patterns have significant effects on the non-linear behaviour and capacity of the column.

Four of the previously tested RC columns were repaired and before being submitted again to cyclic biaxial load, all the columns have been subjected to different retrofit strategies such: increase of the transversal reinforcement, CFRP sheet and plate jacketing and steel plate jacketing.

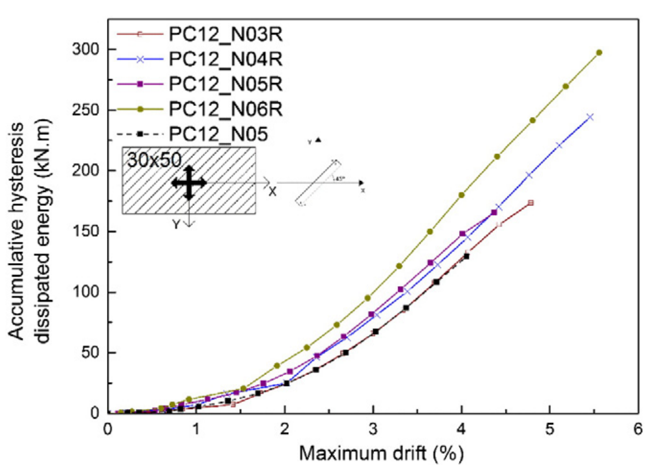


Fig. 17. Comparison of cumulative dissipated energy for retrofitted columns.

All of the design procedures and criteria were presented in order to help the designers during the retrofit process of existing RC columns.

The experimental results on the column retrofitting show that the initial stiffness is typically lower and softening starts for higher drift demands. Also retrofitted columns tend to have an increase of the maximum strength around 20% maximum.

Generally an increase of the ultimate displacement in the retrofitted columns was observed, however ultimate ductility is lower than the original columns. This fact is related with the lower initial stiffness of the column that leads to a higher yielding displacement.

It was observed that damage of the columns is more pronounced in the original column, when compared with the retrofitted, for the same drift demand, however the retrofitted columns tend to concentrate the damage in the base of the column. It was observed that the stiffness degradation is significantly affected by retrofitting solutions.

It was also observed, that submitting the column to and repair and retrofit process can restore or improve the energy dissipation capacity of the columns.

Finally, it is worth emphasising that many questions are still open in the field of the repair and retrofitting of columns under biaxial axial load. The lack of experimental results in this fields, especially those associated with the response dependency of the load paths, presents many questions to the designers. Therefore, additional experimental research should be developed in this domain, particularly considering other displacement panthers. Nevertheless, this research work is expected to contribute to a better understanding of the retrofitting solutions the effect and its interaction with the biaxial response of RC columns and for the calibration of suitable numerical models for the biaxial lateral response of reinforced concrete columns under cyclic loading reversals.

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