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Longitudinal reinforcement of RC columns repaired with welded steel bars solutions: Experimental evaluation

H. Rodrigues [(1)], A. Furtado [(2)], P. Teixeira [(3)], A. Arêde [(4)], N. Vila-Pouca [(5)], H. Varum [(6)]

[(1)] *Senior Lecturer, RISCO, School of Technology and Management, Polytechnic Institute of Leiria, Portugal, hugo.f.rodrigues@ipleiria.pt*
[(2)] *Researcher, CONSTRUCT-LESE – Faculty of Engineering (FEUP), University of Porto, Portugal, afurtado@fe.up.pt*
[(3)] *Civil Engineer, CONSTRUCT-LESE – Faculty of Engineering (FEUP), University of Porto, Portugal, ec02898@fe.up.pt*
[(4)] *Associated Professor, CONSTRUCT-LESE – Faculty of Engineering (FEUP), University of Porto, Portugal, aarede@fe.up.pt*
[(5)] *Assistant Professor, CONSTRUCT-LESE – Faculty of Engineering (FEUP), University of Porto, Portugal, nelsonvp@fe.up.pt*
[(6)] *Full Professor, CONSTRUCT-LESE – Faculty of Engineering (FEUP), University of Porto, Portugal, hvarum@fe.up.pt*

Abstract

The study of the reinforced concrete columns' response to horizontal cyclic loads is of full importance to understand how earthquakes affect the integrity of structures, essentially those already built and especially vulnerable to this type of action, as is the case of many existing buildings on significant seismic activity zones which are not adequately prepared for that eventuality. This importance is mainly justified by that the actual behaviour of RC building columns during an earthquake is strongly related to the random characteristics of the seismic event, the building's three-dimensional response characteristics and its structural irregularities; and finally that the biaxial bending-moment cyclic demands applied to an RC column tend to reduce its capacity and cause stiffness and strength deterioration during successive load reversals. Large number of RC columns were designed according to the practice that not taking into account the importance of plastic deformation and ductility capacity, are 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.

However, experimental research on the inelastic response of RC elements under compressive axial force and biaxial lateral cyclic bending load conditions is currently very limited. The limited research may be due to the uncertainties concerning the relation between the two orthogonal horizontal loading paths and the complexity of the experimental setup required to perform appropriate tests. Consequently, the current knowledge regarding the inelastic response of RC columns under biaxial cyclic moments still lags behind the understanding of one-dimensional (1D) cyclic bending behaviour under a compressive axial load. Recent earthquakes should motivate further studies with the objective of develop also repair and/or retrofit strategies that could be restore the original capacity of the columns and improve their behaviour when subjected to multi-axial loadings.

With this aim an experimental work was conducted at the Laboratory of Earthquake and Structural Engineering – LESE to study the cyclic behaviour of eight RC columns, two of them "as built" and the six remaining damaged that were submitted to repair procedures through welding process of the longitudinal reinforcement and were subjected to uniaxial and biaxial bending combined with constant and variable axial load. The results will be presented in terms of damage evolution along the tests, shear-drift hysteretic curves and total energy dissipation and each repaired column result will be compared with the results of the original ones, deducting about the structural efficiency repair procedures adopted. From the experimental results it was observed similar results between the repaired and the original columns results in terms of initial stiffness, maximum strength and energy dissipation. Globally the repair procedures were very efficient to restore the capacity of the columns

Keywords: RC columns, biaxial loading, welded reinforcement, repair procedures, energy dissipation

1. Introduction

The study of the reinforced concrete (RC) columns' response to horizontal cyclic loads is essential to understand how earthquakes affect the integrity of structures, essentially those already built and especially vulnerable to this type of action, as is the case with many existing buildings on significant seismic activity zones - among which is Portugal - which are not adequately prepared for that eventuality. 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 [1-4]. 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.

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 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, are commonly deficient in flexural ductility, shear strength and flexural strength under strong seismic excitations [5]. Some retrofit techniques for RC columns can be found in the literature, 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 [6-11].

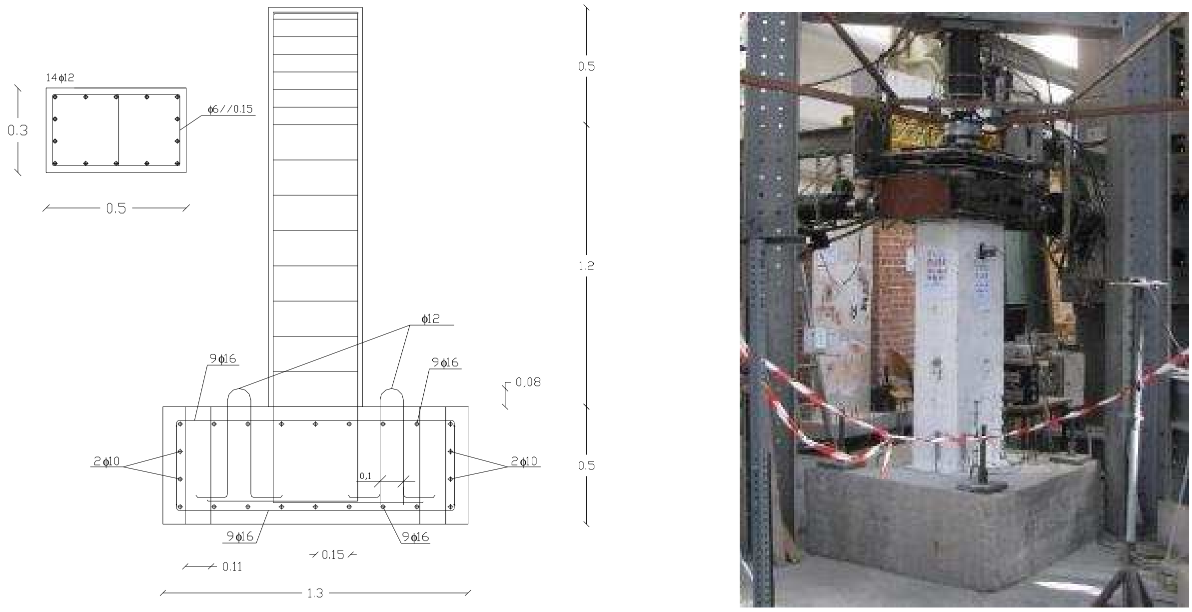
This work presents an experimental campaign composed by eight RC columns, two of them "as built" and the remaining six previously damaged that were submitted to two different repair procedures through welding of the longitudinal reinforcement according to BS EN ISO 17660-1:2006 [12]. The columns were tested under two different horizontal displacement paths and different axial load condition (constant or variable). The results will be presented in terms of evolution of the observed damage, shear-drift hysteretic curves and total energy dissipation and each repaired/retrofitted column will be compared with the results of the original ones, deducting about the structural efficiency repair procedures adopted.

2. General overview of the testing campaign

2.1 Specimen description and test setup

The specimens were constructed at full scale, assuming that the inflection point of a 3.0m height column deflected shape is located at the column mid-height, representing the behaviour of a column at the base of a typical building when subjected to lateral demands induced by earthquakes. All the columns are 1.70m high (with an extra 0.20m height that is added for attaching the actuator devices), and were cast on a strong square RC foundation blocks with dimensions 1.30x1.30m2 in plan and 0.50m high. The cross-section dimensions are 30x50cm^2 and the longitudinal reinforcement is composed by 14ø12 with the transversal reinforcement of ø6//.15m as presented in Fig. 1a. Four holes are drilled at the foundation block to fix the specimen to the laboratory strong floor with prestressed steel rods. This process avoids sliding and overturning of the column footing during the test.

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 (one with a capacity of 500 kN and a +/-150mm stroke and the other with a capacity of 200 kN and a +/-100mm stroke). A vertical actuator with the capacity of 700kN was used to apply the axial load. Two steel reaction frames and a concrete reaction wall form the reaction system for the three actuators. Because the axial load actuator remains in the same position during the test while the column specimen laterally deflects, a sliding device is placed between the top of the column and the actuator. The sliding device was built so that spurious friction effects were minimized.

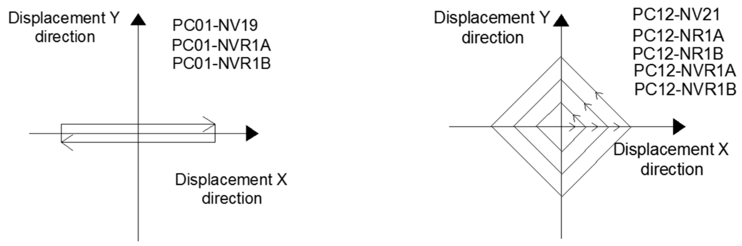


a) b)

Fig. 1 – Experimental campaign: a) RC column specimen dimensions and reinforcement detailing; and b) General overview of the test setup.

2.2 Loading condition

In order to characterise the response of the column specimens, cyclic lateral displacements were imposed at the top of the column with steadily increasing demand levels. Three cycles were repeated for each lateral deformation demand level. This procedure allows for the understanding of the column's behaviour, a comparison between different tests and provides information for the development and calibration of numerical models. The adopted load paths are summarised in Fig. 2, and the following nominal peak displacement levels (in mm) were considered: 3, 5, 10, 4, 12, 15, 7, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80.



Uniaxial – Strong direction (Type 1) Diamond (Type 2)

Fig. 2 – Horizontal displacement path type: a) Strong direction (type 1); and b) Diamond (Type 2).

Constant or variable axial load was applied for the each specimens. Prior to the tests with varying axial loading, the peak capacities in terms of displacements and the strengths corresponding to the first yield were numerically evaluated based on numerical simulations of the uniaxial tests. With this information the columns axial load was considered variable and proportional to the imposed lateral drift applied until the yielding drift. In the biaxial tests the axial load variation is relative to the displacement observed in the strong direction. Beyond the yielding point the axial load was kept constant. The initial axial load was set on 300kN and variations of ±150kN were considered as can be observed in Fig. 3. The different specimens' characteristics and horizontal displacement path type and axial load are presented in Table 1.

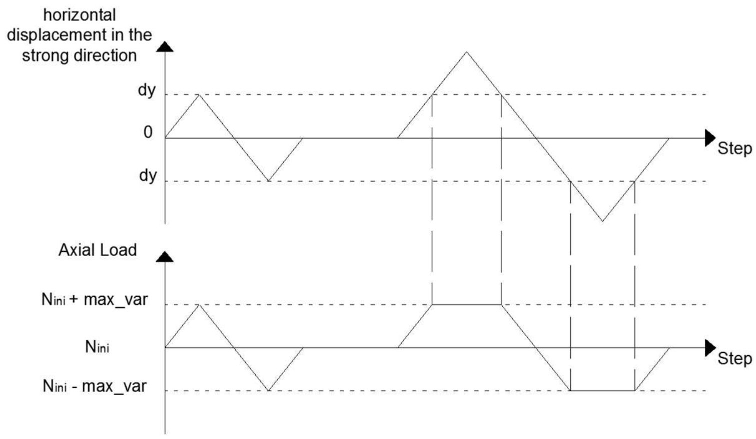


Fig. 3 – Axial loading condition adopted for the tests under varying axial load.

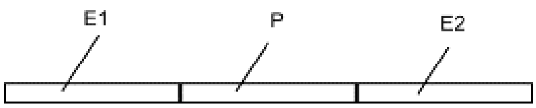
Table 1 – Specimen's specification, repair solution, material properties and loading condition.

Group	Specimen	Repair solution	f_{cm} (MPa)	f_{ym} (MPa)	Loading condition	
					Axial load	Horizontal Displacement path type
Original	PC01NV19	N/A	27.92	575.6	300 (±150kN)	Type 1
	PC12NV21	N/A				Type 2
Repaired	PC12N1B	ST1B			300	Type 2
	PC12N1A	ST1A			300	Type 2
	PC01NV1B	ST1B			300 (±150kN)	Type 1
	PC01NV1A	ST1A				Type 1
	PC12NV1B	ST1B				Type 2
	PC12NV1A	ST1A				Type 2

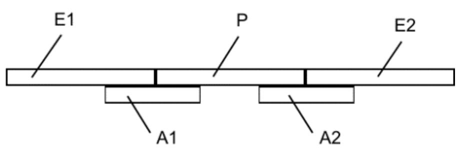
2.3 Repair procedures of previously damaged RC columns

All the six damaged RC columns, except the "as built" ones (PC01NV19 and PC12NV21), were subjected to the same repair procedure that can be summarized in the following five steps:

1) Delimitation of the repairing area, illustrated in Fig. 4a (the critical section at the plastic hinge region), typically from the footing up to 50 cm along the column height) – Phase 1;
2) Removal and cleaning of the damaged concrete (Fig.4b) – Phase 2;
3) In the context of the present study, it was intended to apply de recommendations given by BS EN ISO 17660-1:2006 [11] and four different configurations were considered to repair the longitudinal reinforcement of the columns and are illustrated in Fig. 4, namely: a) butt joint welding b) unilateral lap joint welding (type 1) c) unilateral lap joint welding (type 2) and d) Bilateral strap joint welding.



a)



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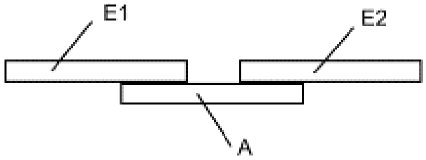
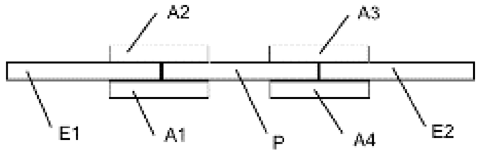


c)



d)

Fig. 4 – Welding longitudinal reinforcement steel re-bars configurations according to BS EN ISO 17660-1:2006 [12]: Butt joint welding b) Unilateral lap joint welding (type 1) c) Unilateral lap joint welding (type 2) and d) Bilateral strap joint welding. Where E represents the existing steel rods from the column and the foundations that are connected to the emended P (but joint) and to the unilateral and bilateral steel rods welded

According to Riva et al [13] the cyclic behaviour of butt joint (Fig. 4a) welding solution is not satisfactory, so this solution can be applicable only in simultaneous with unilateral lap joint welding (Fig. 4b) or strap bilateral welding (Fig. 4d) for seismic repair of columns. The strap bilateral welding solution (Fig. 4d) was also discarded since it corresponds to an excessive increase of the reinforcement area, when compared with the initial situation, being this type of solution is more properly consider for strengthening purposes and not for only repair procedures.

In the LESE laboratory it was conducted 50 tensile strength tests (monotonic and cyclic) of original steel re-bars and specimens of the solutions Type 1 and Type 2. From this tensile tests the following conclusions were drawn:

- The use of an adequate electrode and a suitable cord length well designed and executed for the pretended application guarantee the integrity of the welding re-bars, being the rupture conditioned by the resistance of the steel re-bars as observed in the experimental campaign of tensile strength of different welding solutions.
- The monotic tests showed better response in terms of ultimate strength, however it was verified the same not occurred for other parameters such as elasticity modulus, yielding extensions and maximum strength. The results regarding to the samples cyclically tested it was verified slight increase of the ultimate strength, however the other parameters were similar to the ones observed in the simple re-bars.

Thus the welding solution Type 1 was considered the most appropriated and recommended to adopt for the repair proceeding of the longitudinal reinforcement. It was considered two variations for the unilateral lap joint welding configuration where the E represents the existing steel rods from the column and the foundations that are connected to the emended P (but joint) and to the unilateral and bilateral steel rods welded. For the first variation it was considered for the centre span length equal to 5 diameters and is designed as repair solution ST1A. For the second variation it was considered a centre span length equal to 10 diameters, designed ST1B. In Table 1 is described the repair solution adopted for each specimen.

Finished this stage, the column was lined up according to the welded steel rebar's, followed by the final welding process between them and the columns steel rebar's (Fig. 4c and d): starting from the lateral weld splicing followed by the opening of the chamfer for posterior realization of the butt welding. It should be noted that the sequence of alignments to welding most follow a symmetric logic from the previous alignment, also taking into account some time waiting so as to enable the cooling and reestablishment of the initial length of the steel rebar's, relieving the increase of stress caused by the dilation (temperature effects). At the end of this step it was observed that no significate curvature/buckling was observed and thus can be concluded that this is the best option for similar works to be performed in the future – Phase 3.

5) Replacement of the transversal reinforcement with the half of the initial space of 0.15m. Note that it was placed at the middle of the central span of the welding solution one transversal rebar – Phase 4;

6) Application of formwork and new micro-concrete to restore the previously sanitized area of the column, that consists in a moisture of a structural repair mortar – Phase 5;

a) b) c) d)

Fig. 5 – Repair procedures adopted for previously damaged RC columns a) Phase 1; b) Phase 2; c) and d) Phase 3.

3. Experimental results and discussion

This section presents the results of the repaired RC columns testing campaign based on: i) damage evolution of the columns through the visual observation along the tests; ii) shear-drift curves where parameters such as initial stiffness, maximum strength, strength degradation and ductility will be discussed; total energy dissipation. For a better comparison the results of the "as built" specimens will be compared with the results of the repaired ones, deducting about the efficiency of the repair procedures.

3.1 Damage evolution

In all the tests (for uniaxial or biaxial loading), horizontal cracks distributed along the column length (associated with the flexural dominant columns' response) were observed for early demand stages, illustrated in Fig. 6 and 7. For each drift demand level, the biaxial loading induced a higher level of damage in the column base than the uniaxial loading. In the biaxial tests, after the formation of horizontal cracks form, larger lateral demands initiate concrete spalling in the column corners. For uniaxial tests, when concrete spalling is observed in the column corners, it promptly expands along the whole section width of the column. For biaxial tests, a bar located at the corner of the column base is always the first to break. In all cases where the plastic hinge region comprised a stirrup, its failure was observed.

As observed in previous studies [5, 14], it was observed higher level of damage in column base for the biaxial tests when compared with the uniaxial tests with constant axial loading. In the case of the RC columns subjected to biaxial loading combined with variable axial load, it is observed a non-uniform damage in each faces of column, which was most evident in the north and south faces, since the maximum axial load coincides with the maximum horizontal displacement according to the strong direction of the column. As expected, the buckling of the repaired re-bars was restricted to the section composed by a simple re-bar between the splicing's, since the overlap of the re-bars in the splicing provided high stiffness to this sections. Consequently, the rupture of the re-bars occurred in this zones, and it is important to note that as observed in the tensile strength testing campaign, no re-bar fractured in the welding sections.

Drift=0.33% *Drift*=0.67% *Drift*=1.00% *Drift*=1.33% *Drift*=2.00%

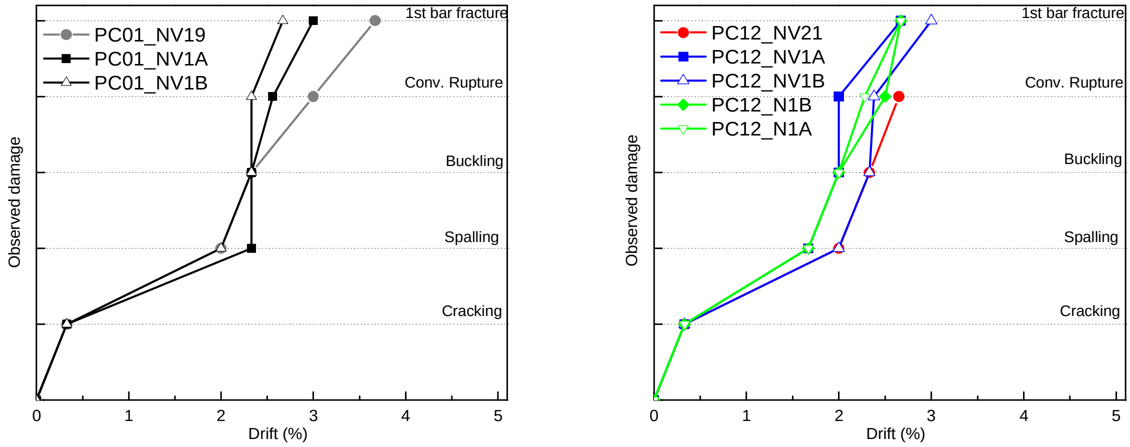
Fig. 6 – Damage evolution: PC12NV1A.

Drift=0.33% *Drift*=0.67% *Drift*=1.00% *Drift*=1.33% *Drift*=2.00%

Fig. 7 – Damage evolution: PC12NV1B.

From the observation of all tests damages five levels of damaged were reported (cracking, spalling, buckling, conventional rupture and 1st bar rupture) and plotted in Figure 16, and the following considerations can be performed:

- It was not found a variation of the drift demand for which occurred the beginning of the cracking, according to the repair strategy, or the type of test;
- The repaired columns subjected to biaxial bending with constant axial load suffered spalling and buckling for drift demand corresponding to 66% of the original non-damaged one, with variable axial load;
- The conventional rupture occurred in the repaired columns, generally, for lower drift demands when compared with the original ones, about 30% in the uniaxial tests and 10% for biaxial tests.

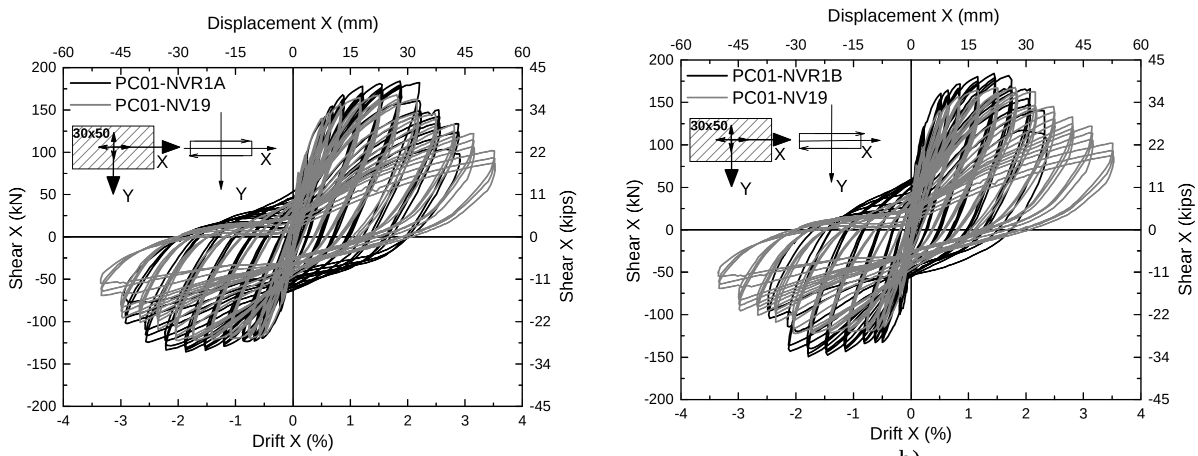


a) b)

Fig. 8 – Observed damage evolution: a) Uniaxial tests; and b) Biaxial tests.

3.2 Shear-drift hysteretic behaviour

From the analyses of the shear force-displacement hysteretic response of the uniaxial tests with variable axial load plotted in Fig.9 it can be observed that the column PC01NVR1A with the repair solution ST1A increased slightly the initial stiffness when compared with the original one. The same was not observed with the PC01NVR1B. Regarding the maximum strength both of the repaired tests reached 5% slightly less than the original column. However, it was observed that the column ductility of the specimens PC01NVR1A and PC01NVR1B were 10% and 20% higher than the original one respectively. Finally it was observed similar strength degradation between repaired and original columns.



a) b)

Fig. 9 – Shear-drift hysteretic response of uniaxial tests: a) PC01-NV19vsPC01NVR1A and b) PC01-NV19vsPC01NVR1B

Regarding the remaining biaxial tests with constant and variable axial load, plotted in Fig.10 and 11 respectively, the following conclusions can be drawn:

- In the biaxial tests, the plateau tends to be shorter and the softening is more pronounced, i.e., a more abrupt decay of the column strength is observed with increasing lateral deformation demands;
- The repair procedures caused a slight decrease of the initial stiffness in the columns subjected to biaxial bending with constant and variable axial load in the directions X+ and Y+. However no significant effects can be observed in other directions;

- From the comparison between the repaired columns subjected to variable axial load with the corresponding original ones, it is observed 10% higher maximum strength. In the case of the repaired columns subjected to constant axial load it was observed that when comparing with the results with the tests with variable axial load i) the maximum strength was lower about 10-15% for the repaired columns and about 5% with the original column.

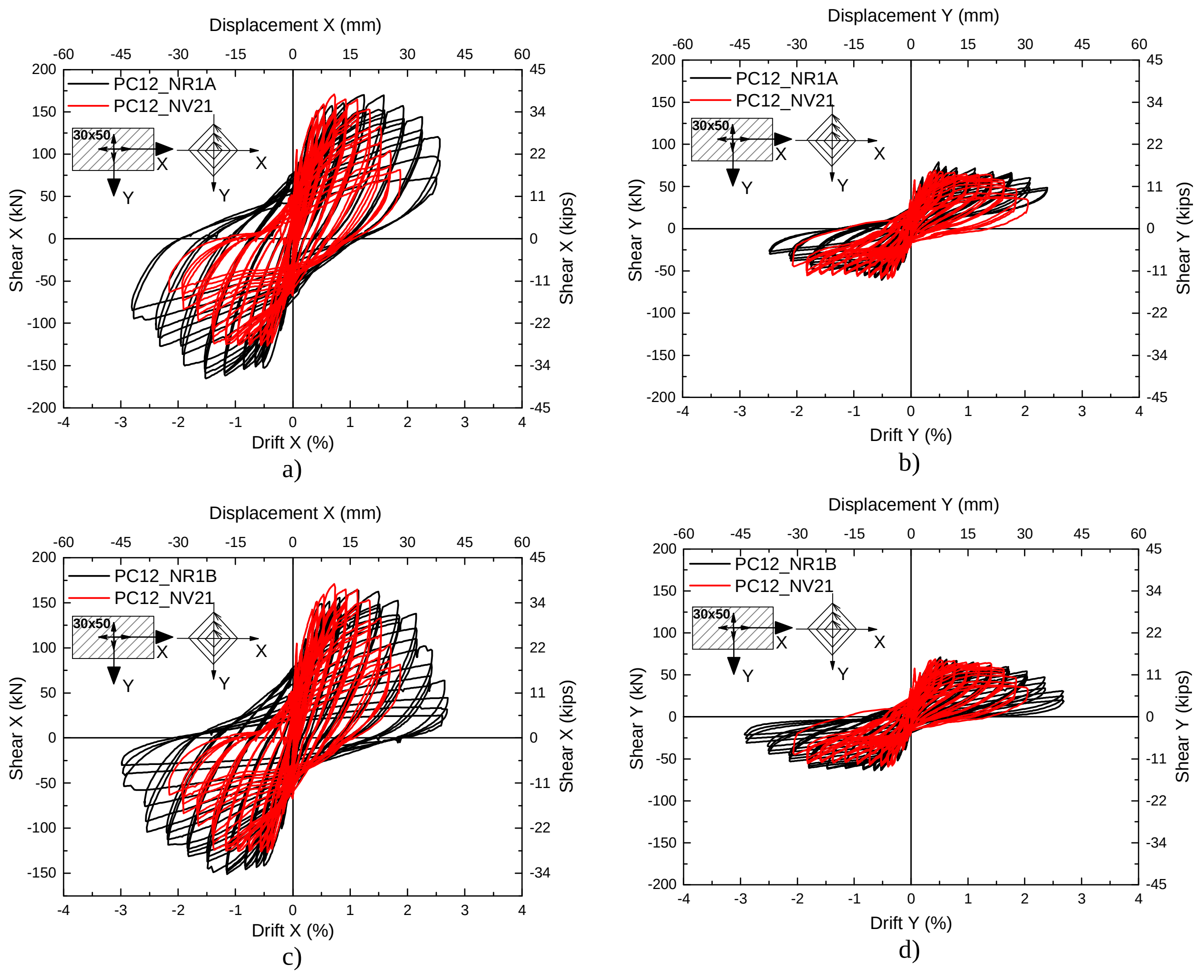
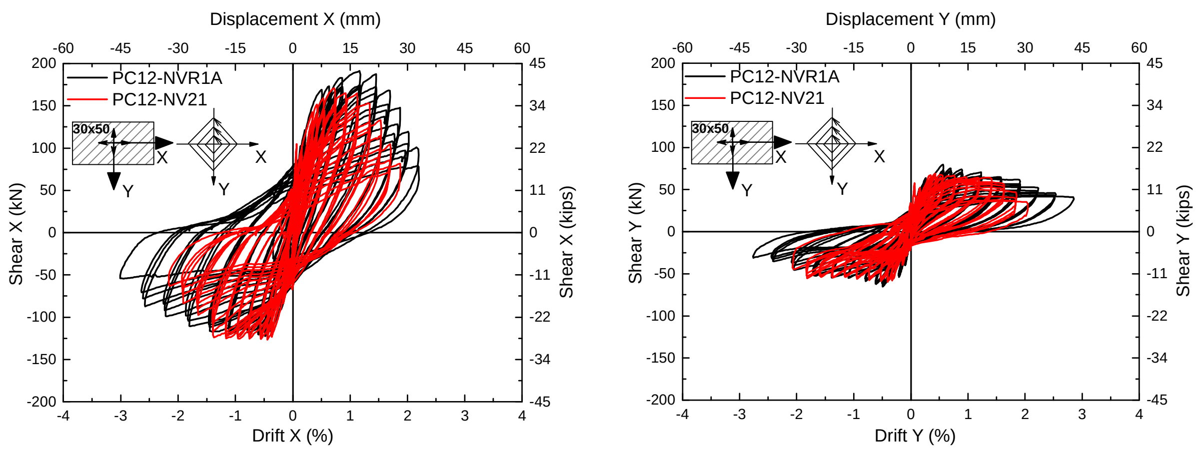


Fig. 10 – Shear-drift hysteretic response of biaxial tests with constant axial load: a) and b) PC12-NV21vsPC12NR1A, c) and d) PC12-NV21vsPC21NR1B.



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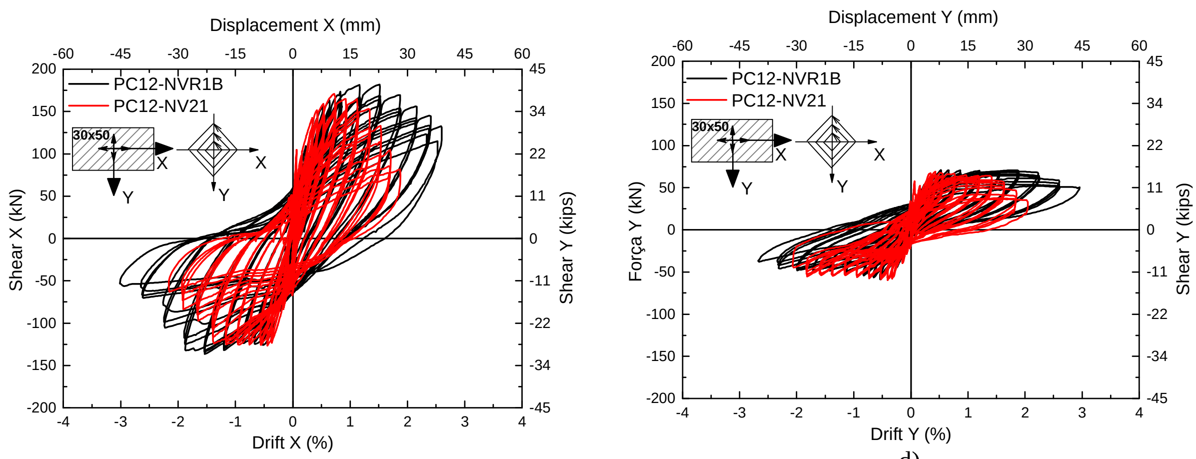
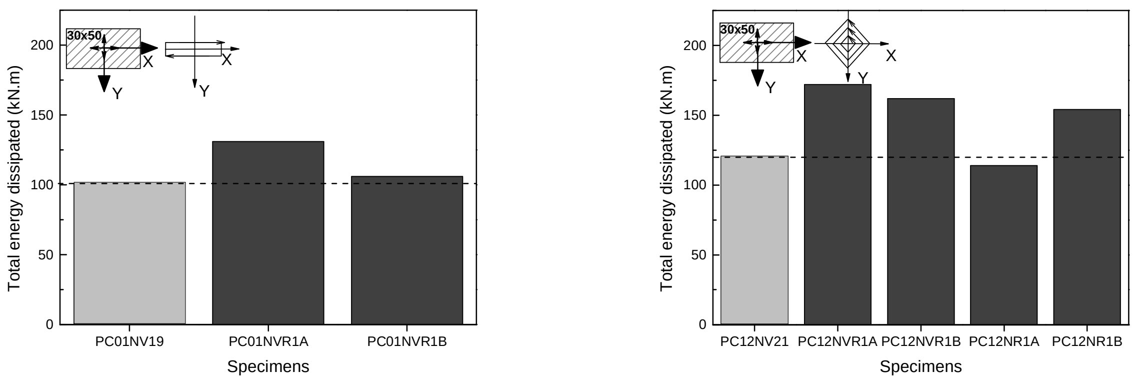


c) d)

Fig. 11 – Shear-drift hysteretic response of biaxial tests with variable axial load: a) and b) PC12-NV21vsPC12NVR1A, c) and d) PC12-NV21vsPC21NVR1B

3.3 Total energy dissipation

The energy dissipation is a fundamental structural property of RC elements when subjected to seismic demands. For RC structures designed to accommodate damage without collapse due to a seismic event, the input energy can be dissipated through RC elements' hysteretic response, without a significant reduction in strength. In this work the energy dissipation was a very important topic to be evaluated such one of the aims was to restore the structural capacity of the original specimen. The cumulative energy dissipation was evaluated for all the tests, considering the area of each loading cycle in the X and Y direction, and then the total energy was calculated as the sum of these two parts. Then the total dissipated energy was determined by each specimen and is illustrated in Fig 12. This total dissipated energy corresponds to the energy dissipated from the start of the test until conventional rupture is reached, to a strength decay of 20% relative to the maximum strength. From the results it can be observed the repaired columns reached the original dissipation capacity or even exceeded. It can be observed that the columns with the repair procedure ST1A obtained better results, except the column PC12NR1A, which dissipated 2% less energy. From the uniaxial tests results it can be observed that the column PC01NV1A dissipated 30% more energy than the original one, and from the biaxial ones it is the column PC12NVR1A that dissipate 40% more energy.



a) b)

Fig. 12 – Total energy dissipated: a) Uniaxial tests; and b) biaxial tests.

5. Conclusions

The main objective of the present manuscript was to study the response of RC columns repaired after being previously tested, when subjected to uniaxial and biaxial horizontal loading and constant/variable axial load. One of the main issues was to give particular focus on the repair procedure of the longitudinal reinforcement repair through welding. Thus this study focus, initially, in the design and test of welding re-bars samples and subsequently in the application of this techniques in RC columns previously tested, with the aim of evaluate the efficiency of the repair procedures. The use of an adequate electrode and a suitable cord length well designed and executed for the pretended application guarantee the integrity of the welding re-bars, being the rupture conditioned by the resistance of the steel re-bars. Six RC columns were repaired with two different repair procedures and were tested for uniaxial and biaxial loading combined with constant and variable axial load. It was observed similar results between the repaired and the original columns results. The repair process reduced the initial stiffness in the columns subjected to biaxial bending, only for X+ and Y+ directions, while similar values to the original ones were found for the remaining directions and types of the test. The repaired columns when tested biaxially obtained similar results of the original column for the maximum strength, which not occur for the columns tested uniaxially where verified slight increase. The repaired columns shows an increase of the ductility, more pronounced in the biaxial tests, and otherwise it is observed lower stiffness degradation for same drift demands when compared with the original columns. Finally, the accumulative energy dissipated of the repaired columns was generally higher for than the original ones, mainly for the columns repaired with the solution ST1A. In general, the repair procedures were very efficient to restore the capacity of the columns, which was also improved in certain cases.

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References

[1] H. Rodrigues, A. Arêde, H. Varum, and A. Costa, "Behaviour of RC building columns under cyclic loading: experimental study," *Journal of Earthquake and Tsunami, JET, World Scientific Publishing* 2012.

[2] H. Rodrigues, A. Arêde, H. Varum, and A. Costa, "Experimental evaluation of rectangular reinforced concrete column behaviour under biaxial cyclic loading," *Earthquake Engineering and Structural Dynamics*, vol. 42, pp. 239-259, 2013.

[3] H. Rodrigues, A. Arêde, H. Varum, and A. Costa, "Damage evolution in reinforced concrete columns subjected to biaxial loading," *Bulletin of Earthquake Engineering*, vol. 11, pp. 1517-1540, 2013.

[4] CEB, "RC frames under earthquake loading," Lausanne Bulletin 220, 1996.

[5] H. Rodrigues, A. Furtado, and A. Arêde, "Behavior of Rectangular Reinforced-Concrete Columns under Biaxial Cyclic Loading and Variable Axial Loads," *Journal of Structural Engineering*, 2015.

[6] H. Rodrigues, A. Arêde, A. Furtado, and P. Rocha, "Seismic behavior of strengthened RC columns under biaxial loading: An experimental characterization," *Construction and Building Materials*, vol. 95, pp. 393-405, 2015.

[7] M. Priestley, F. Seible, and M. Calvi, "Seismic design and retrofit of bridges," Wiley, New York, 1996.

[8] G. Thermou and A. Elnashai, "Seismic retrofit schemes for RC structures and local-global consequences," *Prog. Struct. Eng. Mater.*, vol. 8, pp. 1-15, 2006.

[9] H. Rodrigues, A. Arêde, A. Furtado, and P. Rocha, "Seismic rehabilitation of RC columns under biaxial loading: an experimental characterization," *Structures*, vol. 3, pp. 43-56, 2015.

[10] F. Seible, M. Priestley, G. Hegemier, and D. Innamorato, "Seismic retrofit of RC columns with continuos carbon fiber jackets," *J. Compos. Constr.*, vol. 1, pp. 52-62, 1997.

[11] A. Gomes, "Comportamento e reforço de elementos de betão armado sujeitos a ações cíclicas, PhD Thesis," PhD, Instituto Superior Técnico, Universidade Técnica de Lisboa, Lisboa, 1992.

[12] *BS EN ISO 17660-1:2006 - Welding - Welding of reinforcing steel - Part 1: Load-bearing welded joints*, 2008.

[13] P. Riva, A. Franchi, and D. Tabeni, "Welded Tempcore reinforcement behaviour for seismic applications," *Materials and Structures/Matériaux et Constructions*, vol. 34, pp. 240-247, 2001.

[14] H. Rodrigues, A. Arêde, H. Varum, and A. G. Costa, "Experimental evaluation of rectangular reinforced concrete column behaviour under biaxial cyclic loading," *Earthquake Engineering Structural Dynamics*, 2012.