Calibration of the numerical response of columns with cyclic loads

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SUMMARY:
The main purpose of this paper is to present the calibration of numerical tools based on the results of a experimental cyclic behavior campaign of RC columns, carried on the Laboratory LESE - FEUP, that involved a several specimens tested as build and after reinforcement or retrofitting techniques.
The numerical simulations also allowed the interpretations of experimental results and the understanding of the retrofitting efficiency. The numerical methodologies adopted were: concentrated non-linear on plastic hinges model and distributed non-linearity model.
The numerical modeling was carried out to reproduce the tests' laboratory conditions (axial loads and displacements imposed) that allowed to adjust the cycle and monotonic responses of the tested columns. In order to fully characterize the behavior under these conditions and to evaluate the effectiveness of the different retrofitting solutions, a comparison was performed between all results, namely: drift, energy dissipated; initial stiffness; relative stiffness corresponding to maximum and yield points.

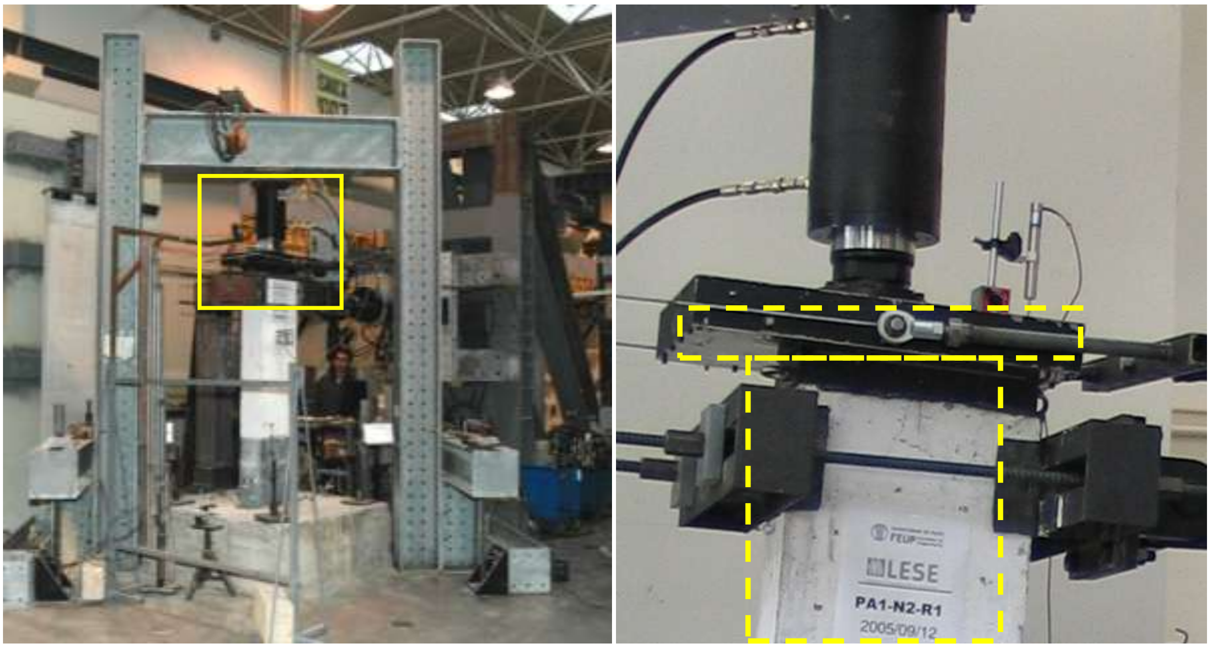
Keywords: RC Columns, non-linear cyclic behavior, experimental tests, RC retrofitting, numerical simulations

1. INTRODUCTION

In order to analyze and to assess different strategies for the seismic retrofit of RC columns, an experimental campaign was performed at the Laboratory of Earthquake and Structural Engineering (LESE) of the Faculty of Engineering of the University of Porto (FEUP), on with full-scale specimens, both undamaged and after reinforcement (or retrofit) with different techniques (Rocha (2011)).

The experimental campaign on RC columns was planned to represent typical buildings designed according standards codes prior to 1980, involving as built specimens and after retrofit columns (resorting to several applications techniques with CFRP sheets and steel bars) in order to improve the available capacity during seismic events and to assess the corresponding safety levels. To this aim, an increasing cyclic displacements law was applied associated with a constant axial load.

The test setup, as illustrated in Figure 1.1, is suitable to apply lateral loads using a hydraulic actuator attached to a reaction steel frame. Constant axial load of 170 kN was applied to the column supported on another independent steel portal frame. The specimen footing is bolted to the strong floor (600 mm thick). A special device was designed to apply a constant axial load in the column, while allowing lateral displacements and top-end rotations (Figure 1.1).



a) General View b) Axial special device detail

Figure 1.1. View of the test setup at LESE Laboratory

The specimens were named PA1-Nx or PB1-Nx, were A or B refer to the series, 1 the reference model and x the specimen number. They have a rectangular cross-section (200 x 400 mm) and half of the typical storey height (1700 mm), fully supported on a solid block bolted to the strong floor (1300 x 1300 x 500 mm) and heavily reinforced to avoid any premature failure during testing. As shown in Figure 1.2, the column specimen PA1 has six 12 mm diameter longitudinal bars of A400 steel grade with average yield strength of 460 MPa and it is transversely reinforced with 6 mm diameter stirrups spaced at 150 mm and made of A500 steel grade with average yield strength of 750 MPa. The average concrete compressive strength is 43 MPa.

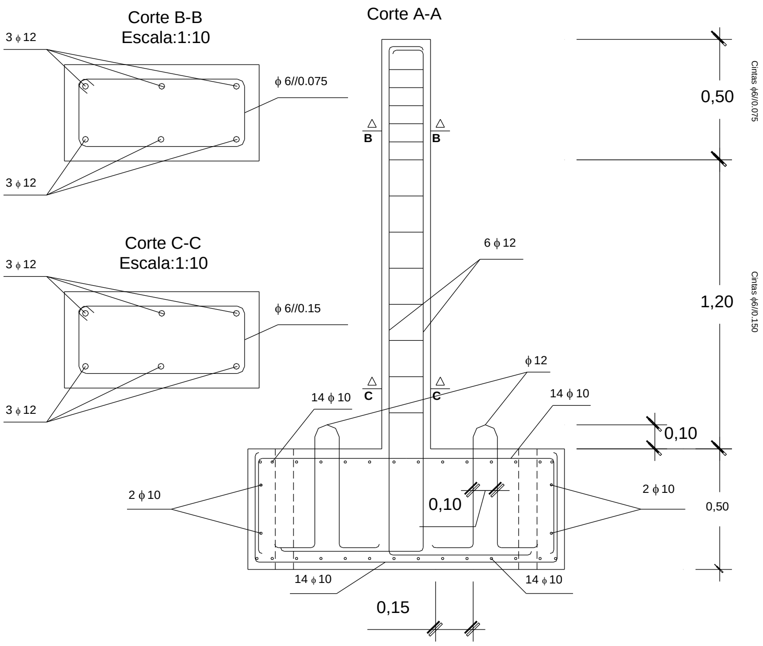


Figure 1.2. Specimen PA1

After the cyclic test of the as built specimens up to failure, they were repaired and retrofitted with three different techniques: CFRP jacketing; steel plates; and steel plates connected by L-shape steel bars. Before performing the retrofit, all specimens were prepared according to the following steps: i) delimitation of the repairing area (plastic hinge - from the footing up to 30 cm above the column height); ii) removal and cleaning of the damaged concrete; iii) alignment and replacement of the longitudinal reinforcement bars (Figure 1.3 a)); iv) application of formwork and new concrete (Microbeton, i.e. a pre-mixed micro concrete, modified with special additives to reduce shrinkage in the plastic and hydraulic phase).

All the adopted retrofits were designed according the Priestley et al. (1996) approach to calculate the jacket thickness for rectangular column aiming at achieving a target displacement of Δ = 50 mm at the point of horizontal force application, i.e. 1600 mm above the footing, while keeping the initial conditions (without ductility and strength upgrade). Inelastic deformation capacity of flexural plastic hinge regions can be increased by recourse to confinement of the column concrete with an advanced composite fiber jacketing system. The steel jacketing was calculated based on the same proposal, only changing the material characteristics (Rocha, et. al 2008). In this paper only the CFRP results will be presented.

a) reinforcement restored	d) CFRP sheets application	c) Final aspect

Figure 1.3. CFRP retrofitting

For the tests, the following conditions were considered: constant axial load of 170 kN and lateral loads imposed by controlled displacement in triple cycles with top displacements between 3 and 80 mm (drifts between 0,19% e 5,00%).

Simultaneously, numerical simulations were carried out in order to better interpret the results and to validate the adopted numerical tools. The comparison between experimental and numerical results was made considering some objective parameters and criteria, namely: drift, dissipated and accumulated energy and loss of stiffness.

2. NUMERICAL SIMULATIONS TOOLS

For this task two numerical tools were adopted for the seismic assessment of the tested RC columns, namely: i) a model with non-linear behavior concentrated in plastic hinges and ii) the fiber model for RC frame members, where the non-linear spread along the member length and across the section area is inherently considered.

2.1. Concentrated non-linear model

In the simulations performed with this procedure the PNL program (Non Linear Frames) was used; it was initially developed by researchers of Faculty of Engineering, University of Porto ((Costa (1989) e Varum (1995)) and later improved and tested by other elements of the same group (Rocha et al. (2004), Romão (2002), Rodrigues (2005), Delgado et al. (2009))

In this model the nonlinear behavior is concentrated at the ends of frame elements (plastic hinge) where cracking and yielding of reinforcement mainly develop. The nonlinear behavior of plastic hinges is characterized by moment curvature relationships obtained from the monotonic response approximated by trilinear curves. Based on these relationships, the hysteretic behavior for cycles in both loading directions is achieved by applying a set of rules that accounts for the stiffness degradation (α), the pinching effect (β) and the strength degradation (γ).

This model was applied to the simulation of the tested columns by adopting the procedures and parameters listed below:
- First, the non-linear behavior of the PA1 cross sections was characterized through the application of BIAX program to define the monotonic skeleton curve by imposing increasing curvatures, after which a trilinear approximation was fitted;
- The modified models of Kent and Park and Menegotto and Pinto were adopted, respectively, for the characterization of concrete and steel behavior;
- For the PNL application, displacements were imposed according to the same law as in experimental tests and hysteretic behavior parameters $\alpha = 0.3$, $\beta = 0.15$ and $\gamma = 150$ were used;
- Second order effects were not included.

2.2. Distributed non-linearity model

This model was applied through the open access program SeismoStruct (SeismoSoft (2006)), based on finite element modeling where sections are discretized by fibers (for concrete and steel) and the nonlinear behavior is distributed along its length. In this study, the following parameters and options were adopted:
- 200 fibers per cross-section;
- Although not significant, second-order effects were included;
- Concrete behavior model proposed by Mander (non-linear constant confinement concrete model);
- Steel behavior model proposed by Menegotto and Pinto and hardening rules improved by Filippou;
- The Baushinger and pinching effects in the steel model were adjusted to the experimental behavior by adopting $R_0 = 19.6$ (which represents the transition between the initial and the post- yielding stiffness).

3. COLUMNS RESULTS ANALYSIS

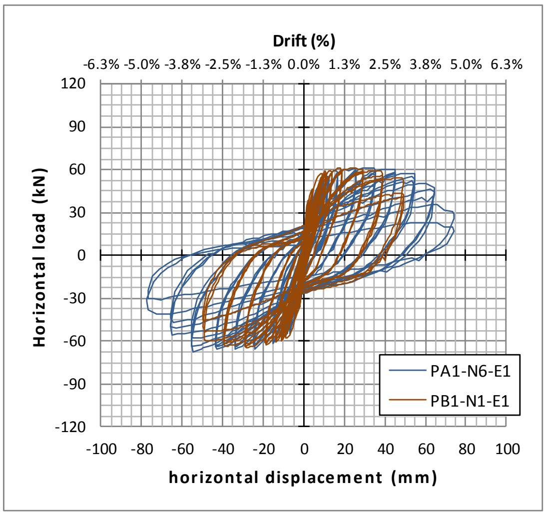
In order to facilitate the results analysis, this presentation is divided in two parts: first with the as built columns and then with retrofitted columns.

3.1. Analysis of as built columns

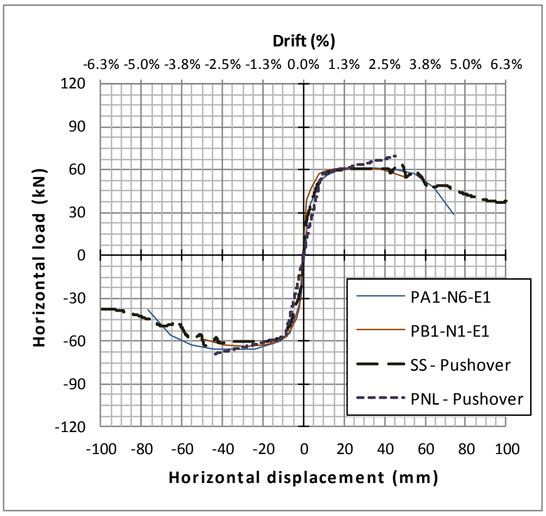
From the global set of specimens, the PA1-N6-E1 and PB1-N1-E1cases were chosen as representative of the results of the as tested columns before retrofitted.

The experimental results are first presented in Figure 3.1 a). The other plots in Figure 3.1 refer to the numerical modeling carried out, namely with the SeismoStruct and the PNL programs, for the simulation of both monotonic (pushover) and cyclic behavior.

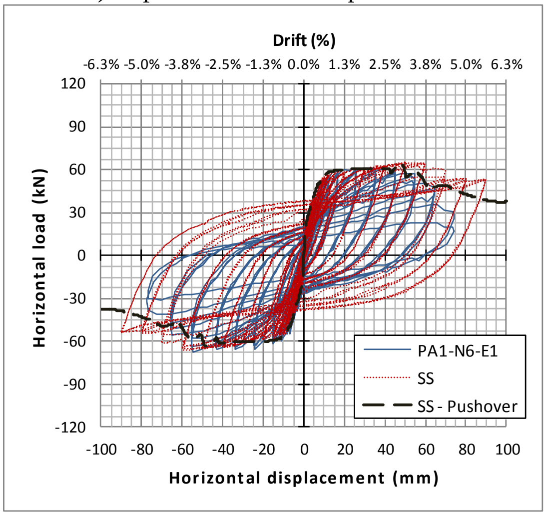
Then, Figure 3.2 shows the plots of the evolution of the following reference parameters: initial stiffness; accumulated dissipated energy and stiffness loss related to both the maximum stiffness and the yielding stiffness. For this group of parameters, the experimental results are similar between PA1-N6-E1 and PB1-N1-E1cases. Therefore the specimen PA1-N6 was considered as a reference for the comparison with numerical simulations and further comparisons with retrofitted columns.



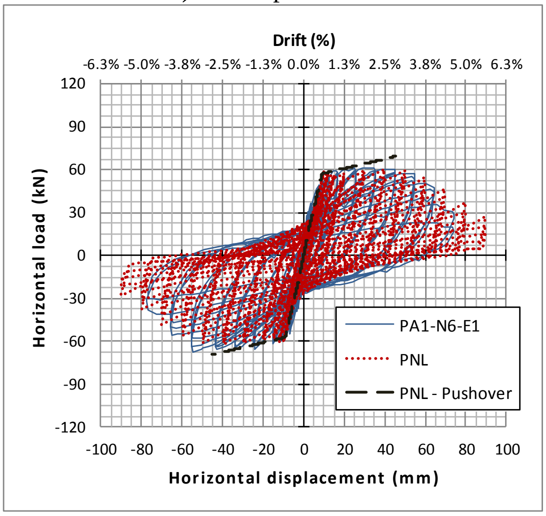
a) Experimental load/displacement



b) Envelope curves



c) Experimental and numerical (SeismoStruct)



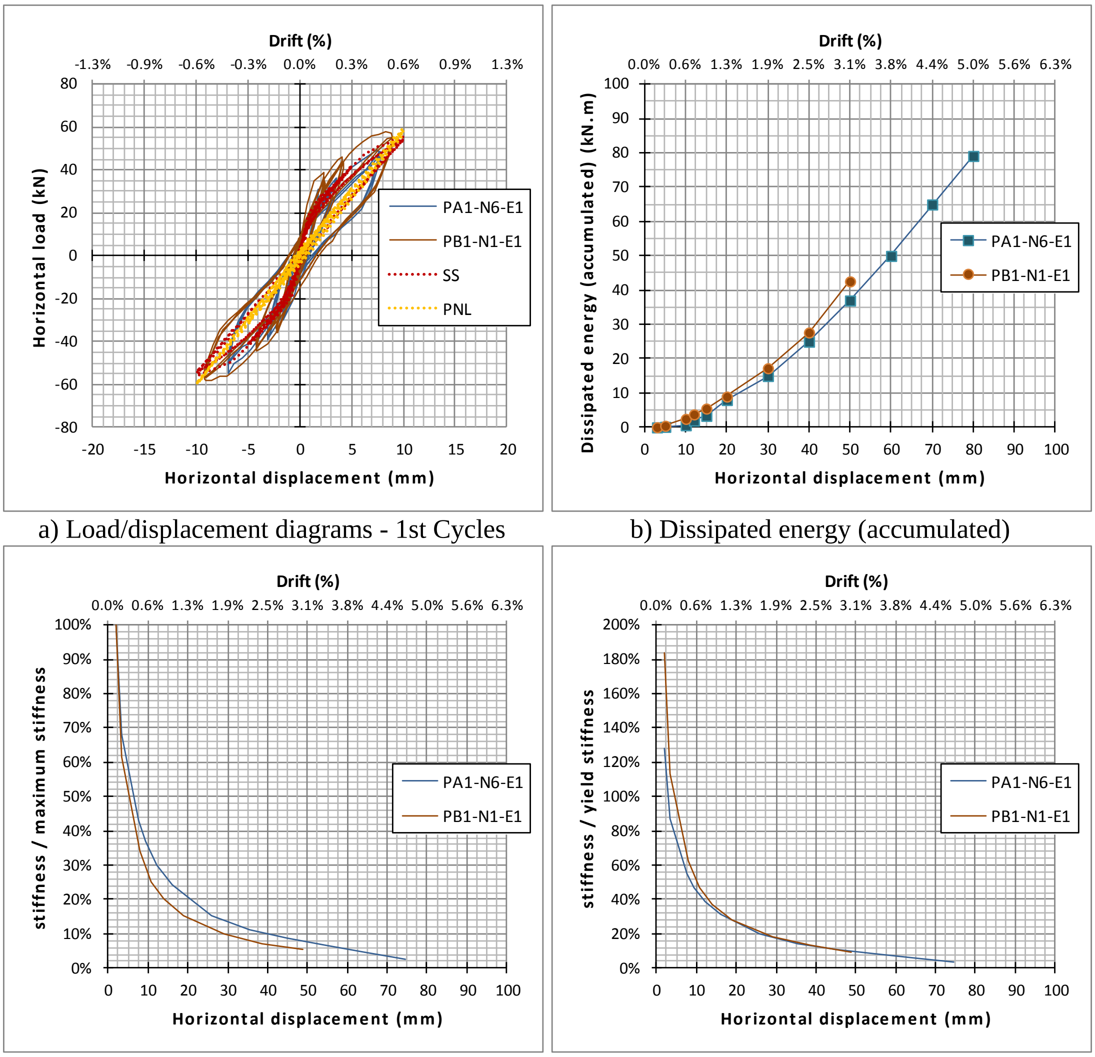
d) Experimental and numerical (PNL)

Figure 3.1. PA1 columns as built results – Part 1

Figure 3.1 b) shows that experimental results are very similar to the numeric ones and the monotonic modeling fits quite well. In particular, the SeismoStruct detected with more accuracy the onset of strength degradation for about a drift around 3.4%. By contrast, the PNL program could not detect this issue because it the non-linear behavior relies on a trilinear moment-curvature relationship, thus without explicitly considering the softening behavior phase.

Regarding the initial stiffness, again SeismoStruct simulation proved to be more accurate in predicting the first steps behavior. As shown in Figure 3.2 a) the PNL program follows a secant path directly to the yield stiffness while the SeismoStruct follows much more closely the experimental values with different stiffness values before and after concrete cracking.

The plots of dissipated energy and the stiffness degradation (Figure 3.2 b), c) and d)) confirm the similarity of the responses obtained with the two simulations of this group. Furthermore, the knowledge of these values progression is important for checking reinforcements' benefits.



a) Load/displacement diagrams - 1st Cycles

b) Dissipated energy (accumulated)

c) Relative loss of stiffness (to maximum stiffness)

d) Relative stiffness loss (to yield stiffness)

Figure 3.2. PA1 columns as built results – Part 2

3.2. Retrofitted columns analysis

This section presents a comparative analysis between the group of specimens retrofitted with CFRP and the as built column PA1-N6 taken as reference.

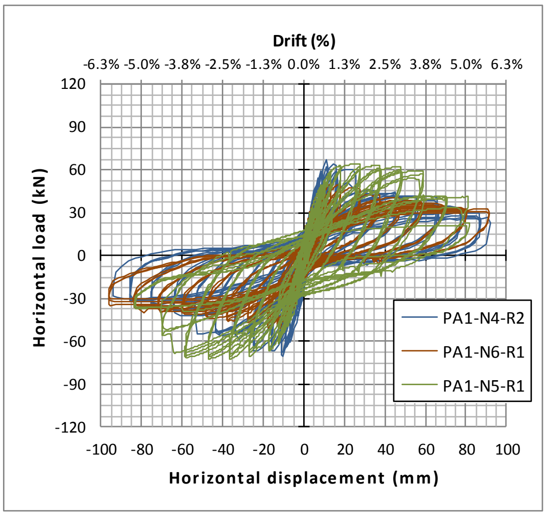
In retrofitting of the plastic hinge some variants were considered, namely:

i) PA1-N6-R1 – jacketing with CFRP
ii) PA1-N5-R1 – jacketing with CFRP with double thickness
iii) PA1-N4-R2 – jacketing with CFRP and laminates grounded in the base

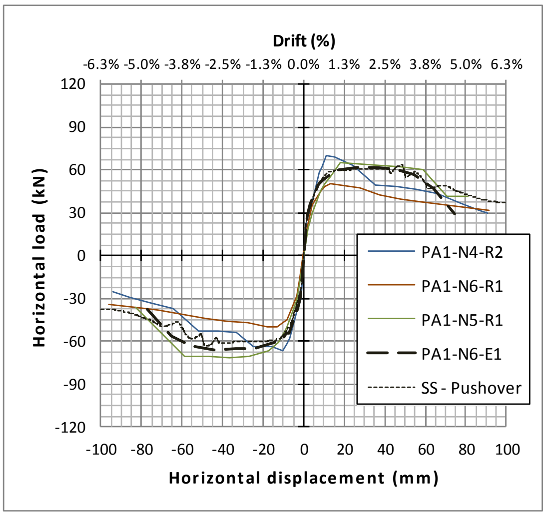
Concerning the initial stiffness, it can be concluded that the columns without retrofitting (Figure 3.3 c)) show values lower than the original columns (except for PA1-N4-R2). The adopted retrofitting technique allows recovering the concrete confinement but not the the steel characteristics throughout the length of the plastic hinge. Because of this, only in the case of longitudinal strengthening, the same stiffness could be observed as for the original column.

The behavior of the specimen PA1-N6-R1, simply reinforced with CFRP, was not as good as expected. For the initial stage, the ultimate strength was reached (Figure 3.3 c)) and a normal path was followed with a slight reduction of the stiffness. However, for drift values greater than 0.6% (Figure

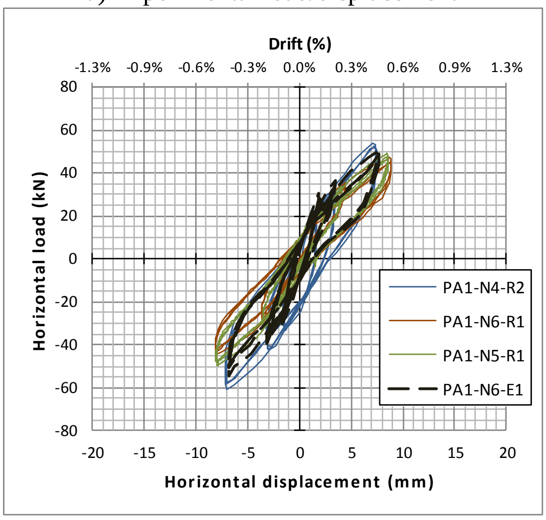
3.3 a) and b)) where the maximum force is achieved, sliding between the original and repair reinforced bars started, causing a strength drop by 20% (50 kN compared to 60 kN). As illustrated in Figure 3.3 d), initially the energy dissipation compared well with the reference case, but it drops after the point when the strength does not reach the expected values.



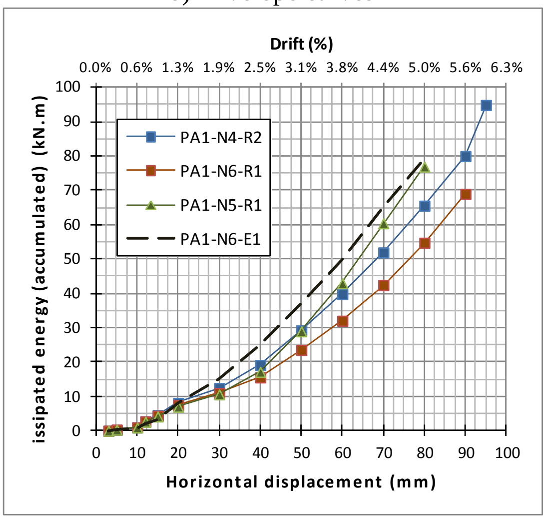
a) Experimental load/displacement



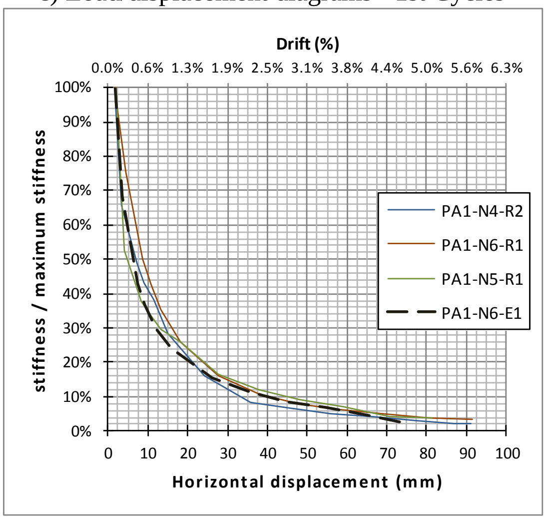
b) Envelope curves



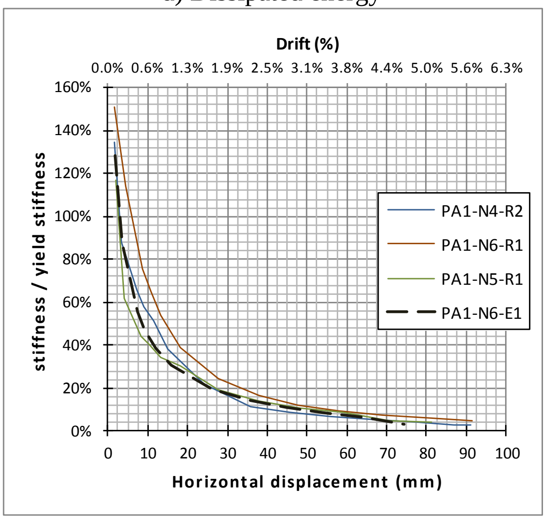
c) Load/displacement diagrams - 1st Cycles



d) Dissipated energy



e) Relative loss of stiffness
(to maximum stiffness)



f) Relative stiffness loss
(to yield stiffness)

Figure 3.3. PA1 columns retrofitted with CFRP

Concerning the reinforcement with double fiber thickness, i.e. the PA1-N5-R1 case, it was found to restore the original conditions. The envelope curves shown in Figure 3.3 b) followed the reference diagram without reinforcement and even exhibited a slight strength gain (approximately 5 kN); however, there was no change in ductility and energy dissipation (Figure 3.3 c)).

From the analysis of Figure 3.3 e), none of the cases show strong differences in the relative stiffness loss. Nevertheless, this conclusion does not mean that there are no stiffness variations between as built specimens and retrofitted ones. In fact, they are close because both stiffness values, initial and maximum, decreased almost the same amount. However, when the variation of stiffness relative to yielding stiffness is analyzed (see Figure 3.3 f)), the results of the typically retrofitted specimen shows early strength loss because reinforcement is not adequately restored (in almost all the way the line is over the rest).

4. CONCLUSIONS

The numerical modeling carried out with the programs SeismoStruct and PNL was found to simulate with good precision the experimental results, both in the case of the original and retrofitted columns.

In general, both programs represent well the aspects of initial stiffness and ultimate strength. In particular, SeismoStruct is better on simulating the onset of the strength degradation and PNL is better in representing the pinching effect.

For the analyzed retrofitted cases, the numerical modeling applied by simply changing the concrete confinement and keeping the remaining parameters, revealed to be a good strategy.

The parameters used for the comparison between as built and retrofitted columns (drift, energy dissipated, initial stiffness, relative stiffness related to maximum stiffness and yield stiffness) were appropriate to evaluate the major aspects that must be taken into account concerning capacity restoring by repairing RC columns.

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