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EXPERIMENTAL TESTS ON SEISMIC RETROFIT OF RC PIERS

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ABSTRACT

The main purpose of this paper is to present an experimental campaign of different strategies for the seismic retrofit of reinforced concrete piers, comparing the obtained results with analytical methodologies, and evaluating benefits concerning their structural behaviour under the cyclic loading. The setup of the RC piers experimental tests was specially designed to carry out biaxial bending with axial load, using two orthogonal and horizontal actuators and one vertical actuator (with a slide device to allow the pier top displacements).The numerical simulation of RC piers, representative of the typical bridge construction, as well as the application of a non-linear cyclic analysis methodology to evaluate the structural behaviour and the structural safety improvement of the various retrofit techniques adopted, are presented. The aim is, therefore, to contribute for developing and calibrating a procedure that enables the evaluation of the efficiency of the different retrofit solutions, their possibilities and fields of application.

Introduction

In order to analyze different strategies for seismic retrofit of RC piers, an experimental campaign has been initiated at LESE - Laboratory of Structural and Seismic Engineering at Faculty of Engineering of University of Porto (FEUP), where a set of RC piers with rectangular hollow section are analyzed. This set of piers will consist of fourteen specimens: a square hollow section RC pier (450mm x 450mm, with 75mm thick), tested experimentally at the laboratory of Pavia University in Italy – Pavese et al. (2004), a rectangular hollow section RC pier of 450mm x 900mm (with the same thick) and the last twelve specimens will be also rectangular with a intermediate dimensions (600mm x 450mm, maintaining the same thick). The choice of those dimensions closer to a square section is to try to understand and take some conclusions concerning the rectangular sections.

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Experimental Program

Specimens

This set of specimens consists of three different hollow rectangular cross sections RC piers: the first, a 450 mm square cross section with 75mm thick walls, is similar to some piers tested at the Laboratory of Pavia University, Italy (Pavese et al. 2004)(the results coming from this specimen tests will be compared with those obtained at Pavia); the second specimen, a rectangular section with 600mm x 900 mm, will be tested in order to understand the third group of twelve specimens of this category with intermediate dimensions (600 mm x 450 mm) which will be divided into two series: the first series will be constructed with the steel reinforcement as shown in Fig. 1 (cross section above), where the transverse reinforcement does not prevent the buckling of the longitudinal bars; and the second series detailed following the EC 8 code (cross section below). The thicknesses of the piers wall remain constant (75 mm).
The models are ¼ scale representations of bridge piers. The hollow piers are named as PO and the first two tested are: PO1 and PO2.

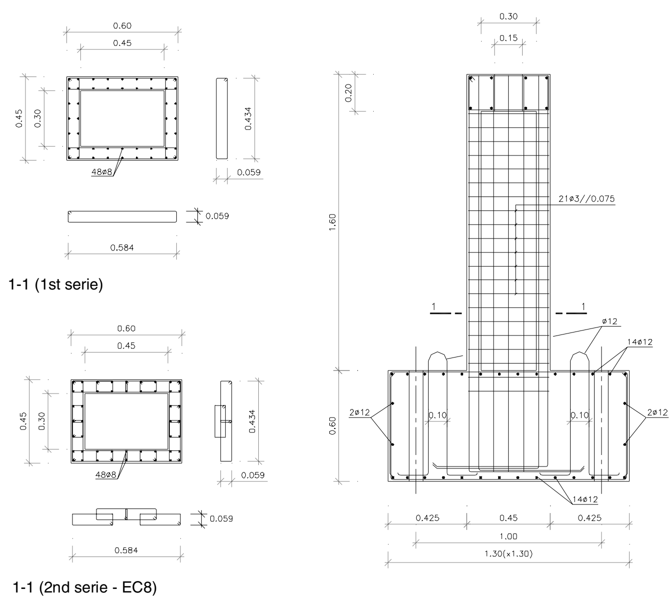


Figure 1: Hollow RC piers

Setup and instrumentations

The test setup, shown in Figs. 2 and 3, used a 200 kN actuator to apply lateral loads and a 700 kN actuator to apply axial loads. The specimen and frame were bolted to the floor with high strength prestressed rods. A constant axial load of 170kN was applied during the tests while the lateral loads were cycled, under lateral displacement control conditions.

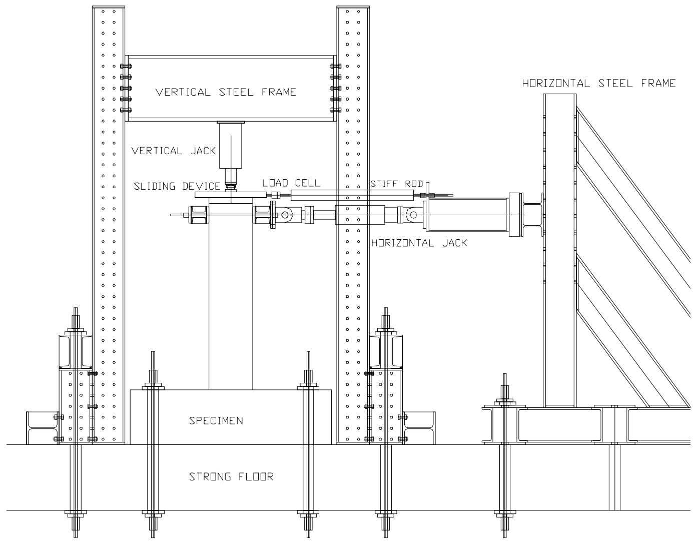


Figure 2: The test setup

Figure 3: View of the test setup at LESE lab

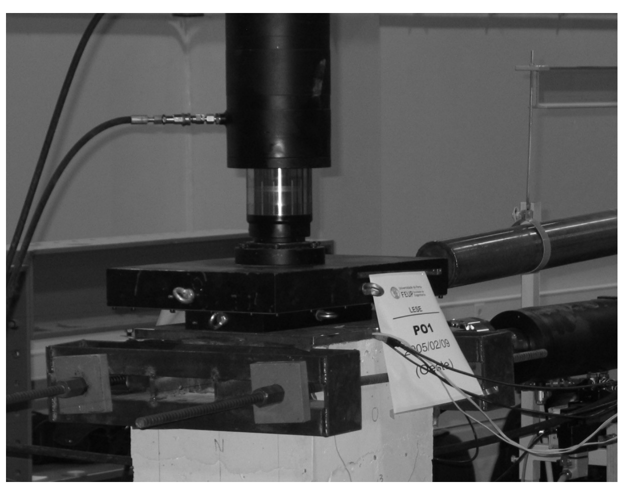


Figure 4: The slide device used to apply axial load

A special device consisting of two steel plates, shown in Fig. 4, was used to minimize the friction created by the axial loads. The lower plate is bonded to the specimen top and the upper is hinged to the vertical actuator, allowing top-end rotations on the specimens during the test, when lateral displacements are imposed, and it is also connected to a high stiff rod with a load cell to measure the residual frictional force, between the two plates.

Results of tests performed to determine the relationship between the applied axial load and maximum horizontal friction force are shown in Fig. 5. As it can be seen a frictional coefficient of about 0.06 has been obtained.

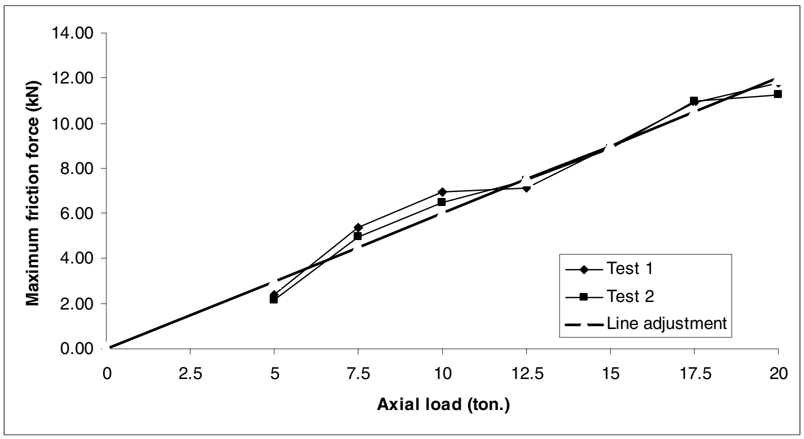


Figure 5: Relationship between the applied axial load and the maximum frictional force

Instrumentation included strain gages attached to the reinforcement and LVDTs to measure curvatures and shear deformations in the plastic hinge regions as sown in Fig. 6. Special software designed for data acquisition and for the hydraulic actuator control has been used, running in LABVIEW computer program

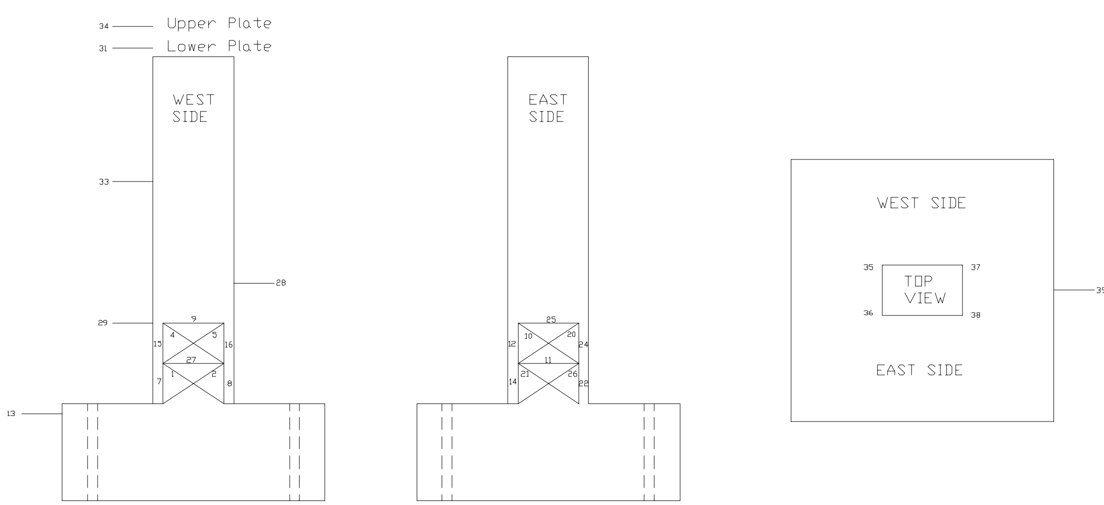


Figure 6: LVDTs location in the test specimen (PO1)

Cyclic Tests

Before the cyclic test of the specimens, a numerical prediction of monotonic curve was carried out to estimate the maximum force of horizontal actuator and the drift level of each cycle. Two numerical models were used for this prediction. The first one was freely obtained from internet: Seismostruct program and the second numerical model, PNL, was developed at FEUP.

Seismostruct is a fiber modeling finite element program for seismic analysis of framed structures, where the spread of the inelastic behavior along the member length and across the section area is explicitly represented through the employment of a fiber modeling approach, implicit in the formulation of the inelastic beam-column frame elements employed in the analyses.

PNL is based on a structural modeling that uses a bar element with a plastic behavior on its extreme zones, recently used by Delgado (2004a, b) and Rocha (2004). For the cross sections of the bar elements with plastic hinges, a global non-linear model for the sections must be used. In this case the moment-curvature loops for reinforced concrete behavior are obtained by a modified Takeda model (proposed at CEB (1996) by Costa & Costa). The laws of monotonic material behavior are numerically established using a procedure based on a cross section fiber model, (Vaz, 1996).

Using the methods above mentioned, monotonic numerical curves prediction for the pier top displacement is showed at Fig. 7. As can be seen the results obtained with Seismostruct shows a lower initial stiffness and higher yielding moment when compared with PNL model. Also computed in Eq. 1 and illustrated in Fig. 7, the shear capacity curve was predicted using the methodology proposed by Priestley et al (1996) for shear strength,

$$V_d = V_c + V_s + V_p = k(f'_c A_e)^{1/2} + A_v f_y . D'/s + P\tan\alpha \quad (1)$$

where V_c, V_s and V_p are the nominal strength of concrete, the transverse reinforcement shear resisting mechanism and axial compression component.

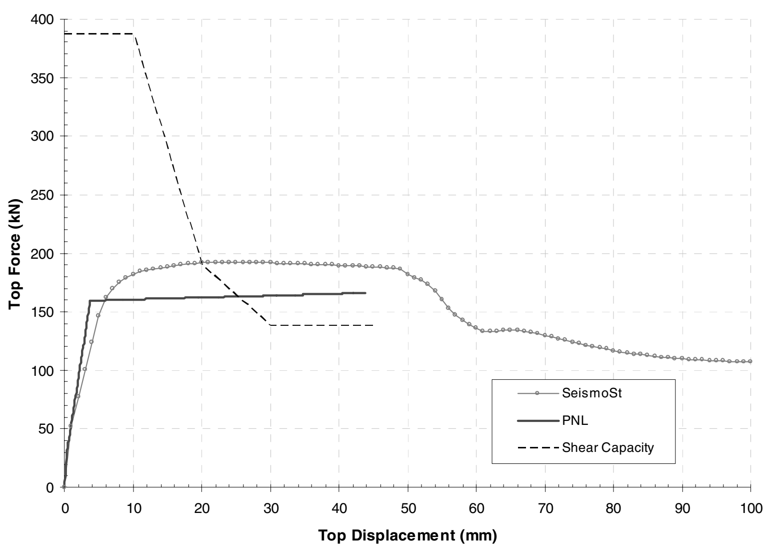
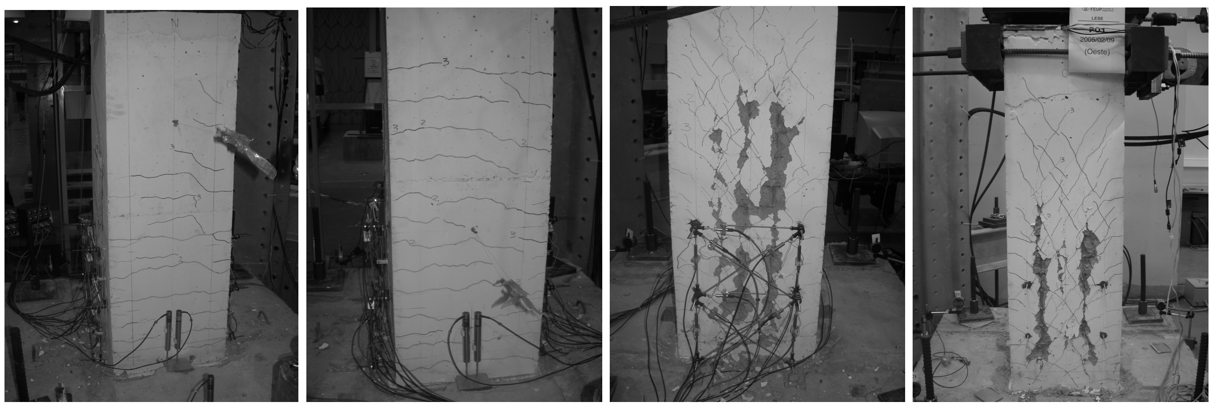


Figure 7: Monotonic numerical curves prediction.

Three cycles were applied for each of the following peak drift ratios: 0.1%, 0.2%, 0.35%, 0.7%, 0.3%, 1.0%, 1.2%, 0.5%, 1.8%, and 2.4%



(a) North side (b) South side (c) East side (d) West side

Figure 8: Pier PO1 damage for 2.4% drift.

As shown in Fig. 8, little damage was achieved on the north and south sides, where well distributed cracks were visible. However, the damage was highly concentrated at the lateral sides, east and west, where the cover concrete crushed within the entire pier high. As was expected, severe shear damage was verified during the test with significant concrete degradation due to the lower efficiency of the transverse reinforcement.

The experimental results and the Seismostruct and PNL models monotonic predictions are illustrated in Fig. 9. The envelopment of the experimental cyclic curve is less stiffer than both numerical predictions curves. However, the maximum values of the reached force are reasonable close. From the experimental response reproduced in Fig. 9 and the damage pattern presented in Fig. 8, it is visible that the shear mode failure of the pier was achieved on the 2nd cycle of 33mm amplitude (2.4% drift).

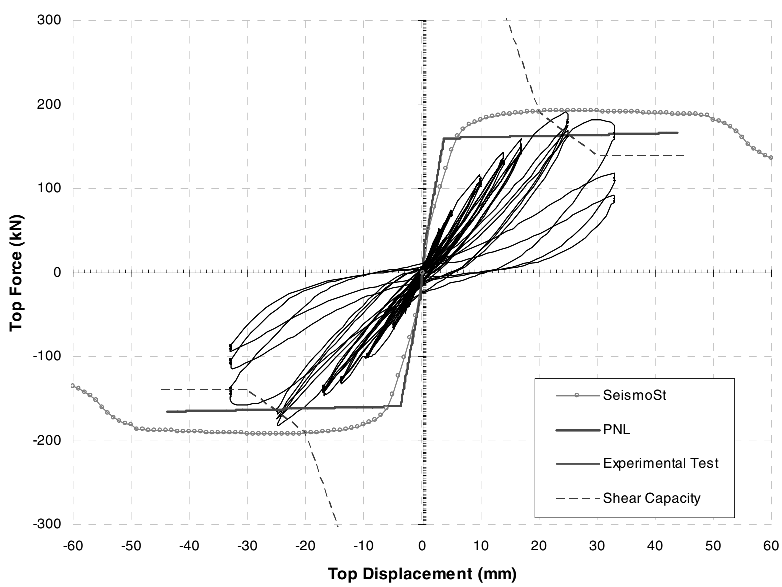


Figure 9: Experimental results and Seismostruct and PNL models monotonic predictions.

Retrofit

After the cyclic test of the "as built" specimen took place up to failure, it was repaired and retrofitted by STAP company as can be summarized in the following steps: 1) delimitation of the repairing area; 2) removal and cleaning of the damaged concrete; 3) application of formwork and new concrete (Microbeton, a pre-mixed micro concrete, modified with special additives to reduce shrinkage in the plastic and hydraulic phase); 4) retrofit with the CFRP sheets. To have an idea of the pier damage, the following photographs illustrate the pier during repairing and after being retrofitted with CFRP sheets jackets (Fig. 10).

Figure 10: Hollow square section pier (PO1) before and after the shear retrofitting with CFRP sheet.

To design the shear retrofit with CFRP jackets, the authors adopted the Priestley et al. (1996) approach to evaluate the thickness of the rectangular hollow pier jacket for increasing the shear strength (see Eq. 2), but keeping the initial section conditions.

$$V_{sj} = (A_j\, f_j\, h \cot \theta)/s = (A_j\, 0.004E_j\, h \cot \theta)/s \qquad (2)$$

In this equation, h is the overall pier dimension parallel to the shear force applied and f_j is the adopted design jacket stress level corresponding to a jacket strain of 0.004, to avoid large dilation strains and hence excessive degradation of the concrete, as well as to provide adequate safety against the possibility of jacket failure.

Resulting from Eq. 2, one layer strip of CFRP sheet of 0.117mm thickness by 100 mm width and 100 mm spaced along the height of the pier was applied, to retrofit the PO1 specimen, as can be seen in Fig. 10.

Cyclic Test of the Retrofitted Specimen

The retrofitted pier has been tested following the same cyclic displacement history of the "as built" specimen, but two additional cycles were performed with increase intensity top displacements: 2.9% and 3.2% peak drift ratios.

The evolution of the damages along the experimental test of the retrofitted specimen is illustrated in Fig. 11, for the east side of the pier. The first cracks, with dimension about 0.1mm, were visible at 0.35% drift, at the pier base. In the subsequence cycles the dimensions of theses cracks increased and new cracks were developed along the pier high (Fig. 11a). At the top displacement of 17mm (1.2% drift - Fig. 11b), only the base crack has growth to about 0.5mm. Shear cracks appeared in the next cycles of 1.8% drift and a generalized damage start to become visible. In the right hand side of Fig. 11c (2.14% drift), a vertical crack is visible over almost the entire pier height, which caused global damage to the CFRP sheets. Rupture of some of the fibers was audible. During the cycles of 40mm top displacement (2.9% drift - Fig. 11d) the bottom carbon sheets failed completely, leading to a drastic collapse of the pier on the compression side of the section, with buckling of the longitudinal reinforcement bars, due to a lack of confinement of the concrete (see also Figs. 12 and 13).

(a) 0.7% drift (b) 1.2% drift (c) 2.14% drift (d) 2.9% drift

Figure 11: Retrofitted pier PO1 damage from east side view.

(a) East side (b) South side

Figure 12: Final damage of the retrofitted pier PO1 for 3.2% drift.

As can be seen in Figs. 11 and 12, the retrofitted specimen showed a good behavior in comparison with the “as built”, exhibiting very distributed cracking along the CFRP spacing.

(a) West side (b) North side

Figure 13: Final damage details.

The shear retrofit design, applied on this pier, showed an excellent performance because the shear failure mechanism was partially prevented and a flexural collapse with the buckling of the reinforcement bars was achieved. In Fig. 14 the comparison between the original and retrofitted pier is illustrated.

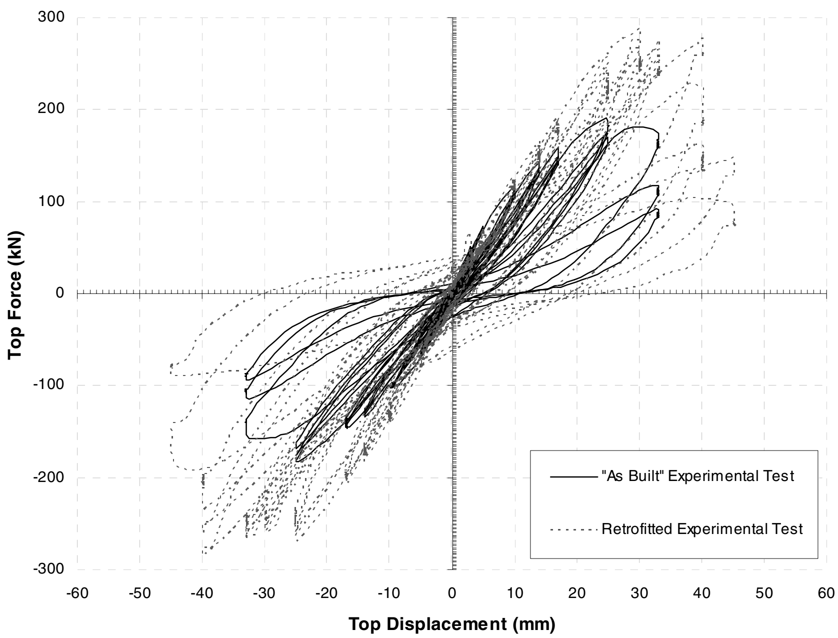


Figure 14: Experimental results of pier PO1 before and after retrofit.

Conclusions

The setup used within this framework showed a very good performance to carry out bending tests with axial load. The slide device, a steel plate system for the axial force transmission that allows the top displacements of the pier, performed satisfactorily and showed low values of frictional forces, which can be measured and considered on the final results.

Numerical prediction assessment was reasonably close to the experimental results, namely for maximum values of the reached force; however the initial stiffness was lower at experimental test. Some local deformation of the pier foundation can possibly justify these differences. The experimental test of the “as built” specimen showed severe shear damage with significant concrete degradation due to the lower efficiency of the transverse reinforcement.

The CFRP retrofit showed excellent benefits on the pier behavior once it avoid the shear collapse and a mixed of shear and flexure mechanism was achieved. The ultimate drift increased from 2.4% to 3.2% and an well cracking distribution was obtained. The retrofitting strategy adopted evidenced a good ability for the improvement of the seismic pier behavior both in ductility and strength.

Acknowledgments

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