EXPERIMENTAL TESTS ON SEISMIC RETROFIT OF RC BRIDGE 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 bridge piers and evaluating benefits concerning their structural behaviour under the cyclic loading.

The setup of the RC pier experimental tests was specially designed to carry out bending with axial load, using a horizontal and a vertical actuator (with a slide device to allow top displacements of the pier). A square hollow section RC pier (450 mm x 450 mm, and 75 mm thick), similar to another one tested at the laboratory of Pavia University in Italy, and a rectangular hollow section RC pier of 450 mm x 900 mm (with the same thickness) were tested at LESE - Laboratory of Earthquake and Structural Engineering at Faculty of Engineering of University of Porto.

Besides numerical simulations of RC piers, which are not presented herein, the aim is 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

Bridges and viaducts, are amongst all the structures, those that sustain the most damage, as has been demonstrated through reports of recent earthquakes. Even for “moderate magnitude” earthquakes, the consequences in these structures have been very dramatic, in many occasions causing their partial destruction, and in some cases total collapse, with the corresponding heavy costs. When compared, these consequences of vulnerability have been greater than those observed in building structures and almost always the bridge safety is limited and conditioned by piers capacity.

Several studies and works have being carried out on solid piers and can be applied to building structures; however, for hollow piers much less research was performed. This type of piers has usually large dimensions, with reinforcement bars spread along both wall faces, and the shear effect can have great importance on the pier behaviour. Therefore, special attention must be given to the assessment and retrofit of RC pier, and mainly to the hollow piers.

In order to analyze different strategies for seismic retrofit of RC piers, an experimental campaign has been developing at LESE - Laboratory of Earthquake and Structural Engineering at Faculty of Engineering of University of Porto (FEUP), in which a set of RC piers with rectangular hollow section is being tested.

Within this framework and with the purpose of studying different retrofit strategies, this paper presents the basic description of the designed setup and its calibration, followed by the description of a set of tests already carried out involving a first set of original specimens and a second set of the retrofitted piers. Some results and conclusions are also presented.

EXPERIMENTAL CAMPAIGN

Setup

The test setup, shown in Figs. 1 and 2, makes use of a 500 kN actuator to apply lateral loads and a 700 kN actuator to apply axial loads. The specimen and reaction frame are bolted to the strong floor with high strength prestressed rods. A constant axial load of 250 kN was applied during the tests, herein described, while the lateral loads were cycled, under displacement controlled conditions.

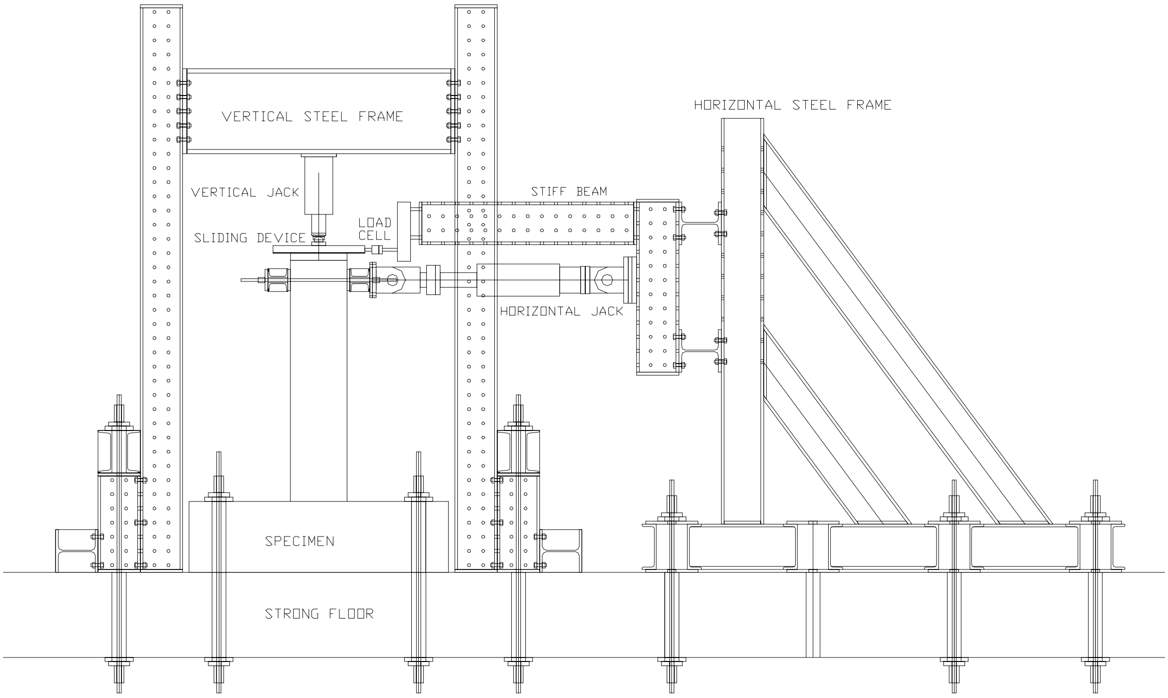


Fig. 1 Schematic layout of the test setup.

Fig. 2 View of the test setup at LESE laboratory.

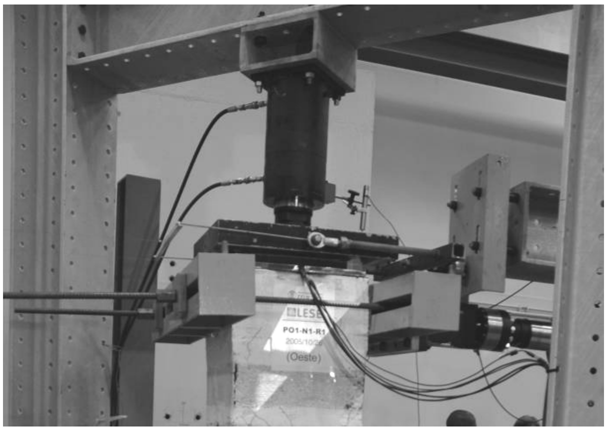


Fig. 3 The sliding device used to apply the axial load.

A special sliding device consisting of two steel plates, shown in Fig. 3, was used to minimize the friction created by the axial loads. The lower plate is bonded to the specimen top, whereas the upper is hinged to the vertical actuator, allowing top-end displacements and rotations on the specimens, when lateral loading is imposed during the test. The upper plate is also connected to a load cell to measure the residual frictional force between the two plates.

During the tests performed to date, an undesired problem occurred with the hydraulic system used to apply the axial load: the pressure that should been remain constant has in fact increased. Actually, the hydraulic system was designed to keep constant the oil pressure, in order to maintain constant the axial force, but a deficient performance of the circuit has blocked the return of the oil from the vertical actuator; therefore, the axial load increased during the cyclic displacement history, because the axial actuator was forced to remain in the same position when the top-end pier section was rotating and displacing.

A preliminary test to calibrate all the setup was carried out, by using a steel column hinged at the bottom, illustrated in Fig. 4, instead of the R/C specimen. The main purpose was therefore to accurately measure all the forces involved on the vertical reaction frame.

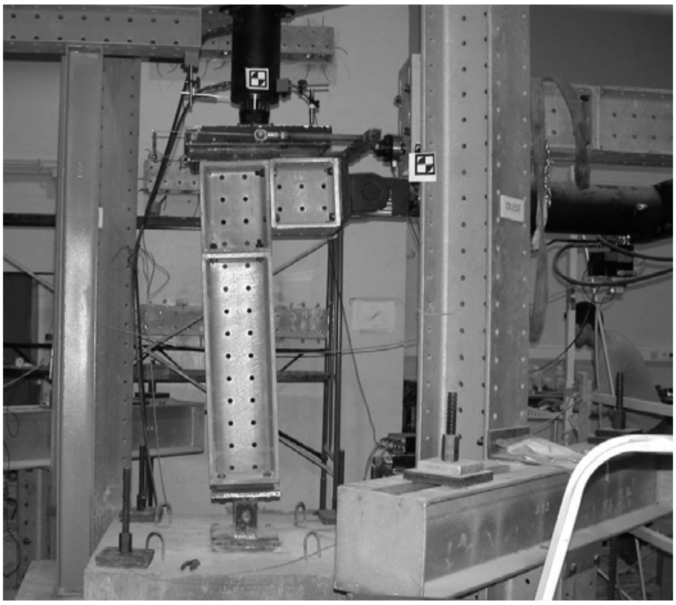


Fig. 4 Setup with a steel pier hinged at the base.

Due to the column hinge the moment of all forces at the base has to be equal to zero. Thus, only three exterior forces are applied to the steel column, which produce moment at the base: the horizontal actuator force, the upper plate force (resulting from the displacement restrain at this point) and the horizontal force of the vertical reaction frame. The two first forces are measured from load cells and, from the null moment equilibrium condition at the base, the horizontal force of the vertical reaction frame was computed. Considering the same axial load, as for the specimen piers, the diagram shown in Fig. 5 was obtained whish relates the mention horizontal force with the horizontal displacement of the vertical frame.

These results allowed to determine the horizontal stiffness of the vertical reaction frame. As it can be seen from the Fig. 5, two parallel lines are defined on the envelop of the hysteretic curve, which leads to a horizontal stiffness of the vertical reaction frame of approximately 2.2 kN/mm. Probably due to the characteristics of the slide device, a small gap on the stiffness was observed when the direction of the movement is inverted.

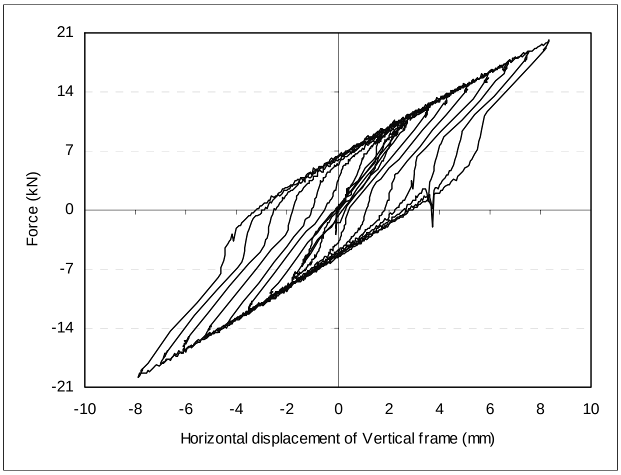


Fig. 5 Relationship between the horizontal force and displacement of the vertical steel frame.

<u>Instrumentation</u>

Instrumentation included strain gages attached to the reinforcement bars and LVDT's to measure curvature and shear deformations in the plastic hinge regions, as shown in Fig. 6. On the left side it is presented the LVDT's layout used during the pier PO1 testing while on the right side is shown the LVDT's scheme applied to pier PO2. This difference in the instrumentation layout was adopted after the first test which showed an important shear deformation, being therefore essential to distribute the LVDT's along the whole height of the pier, as used in pier PO2. Special software designed for data acquisition and for the hydraulic actuator control has been used, running in a LabVIEW computer program.

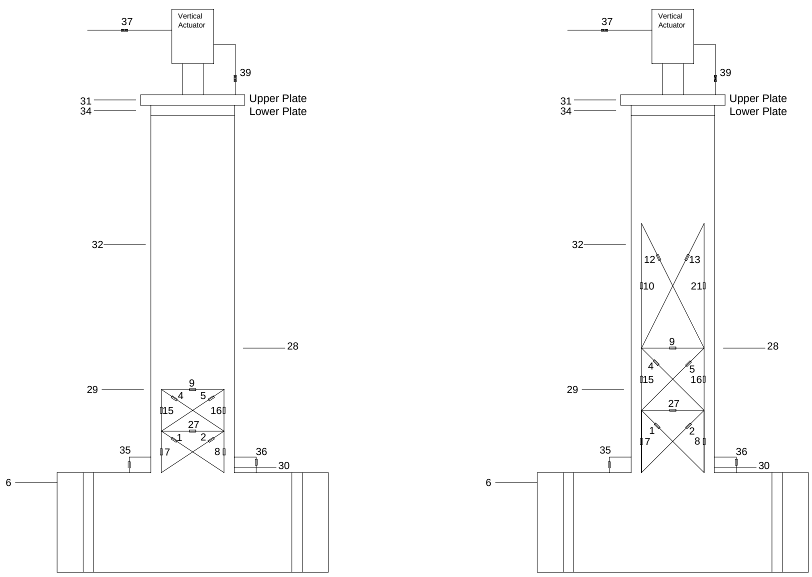


Fig. 6 LVDT's location in the test specimens: PO1 (left) and PO2 (right)

Specimens

This set of specimens consists of two 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) and the results coming from this specimen tests will be compared with those obtained at Pavia; the second specimen set, a rectangular section with 450mm x 900 mm, will be tested in order to understand the influence of the cross section geometry of rectangular hollow piers on the cyclic behaviour, bearing in mind the purpose of assessing retrofitting solutions. The thickness of the pier wall was kept constant, 75 mm.

The models correspond to ¼ scale representations of bridge piers, illustrated in Fig. 7, and the hollow piers are named as PO: PO1 for the square section and PO2 the rectangular section.

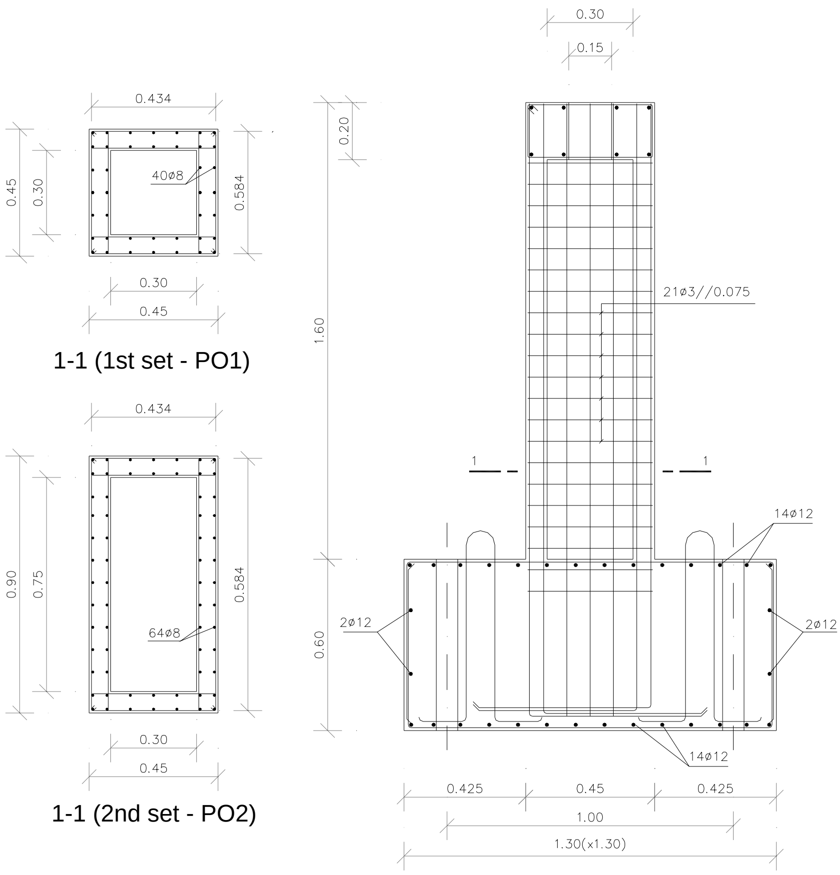


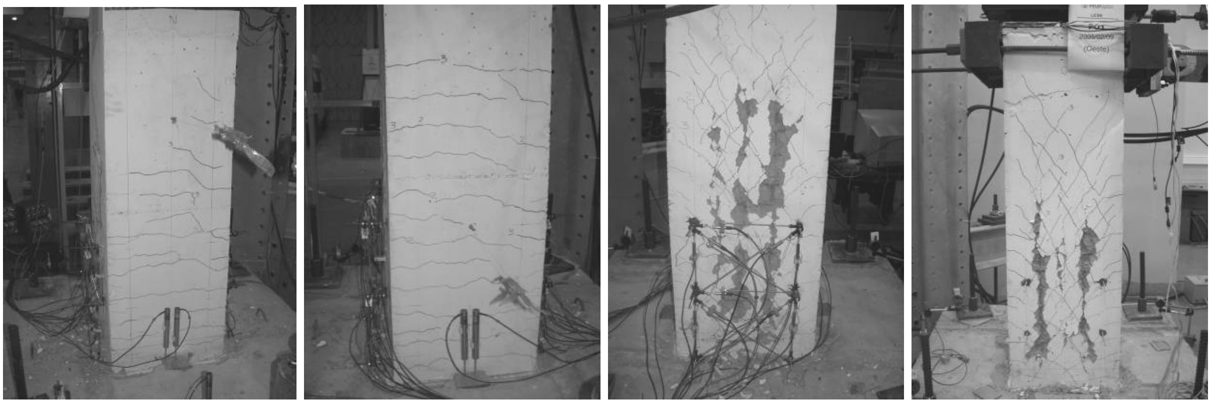
Fig. 7 Hollow RC piers

CYCLIC TESTS

In order to characterize the pier cyclic behaviour, 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%.

Pier PO1

As shown in Fig. 8, at the end of the test 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 concrete cover crushed within the entire pier height. As expected, severe shear damage was observed during the test with significant concrete degradation due to the low efficiency of the transverse reinforcement.



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

Fig. 8 Pier PO1 damage for 2.4% drift.

From the experimental response illustrated in Fig. 9 and the damage pattern presented in Fig. 8, it is visible that the shear mode failure of the pier was achieved around the 1st or 2nd cycle of 33mm amplitude (2.4% drift).

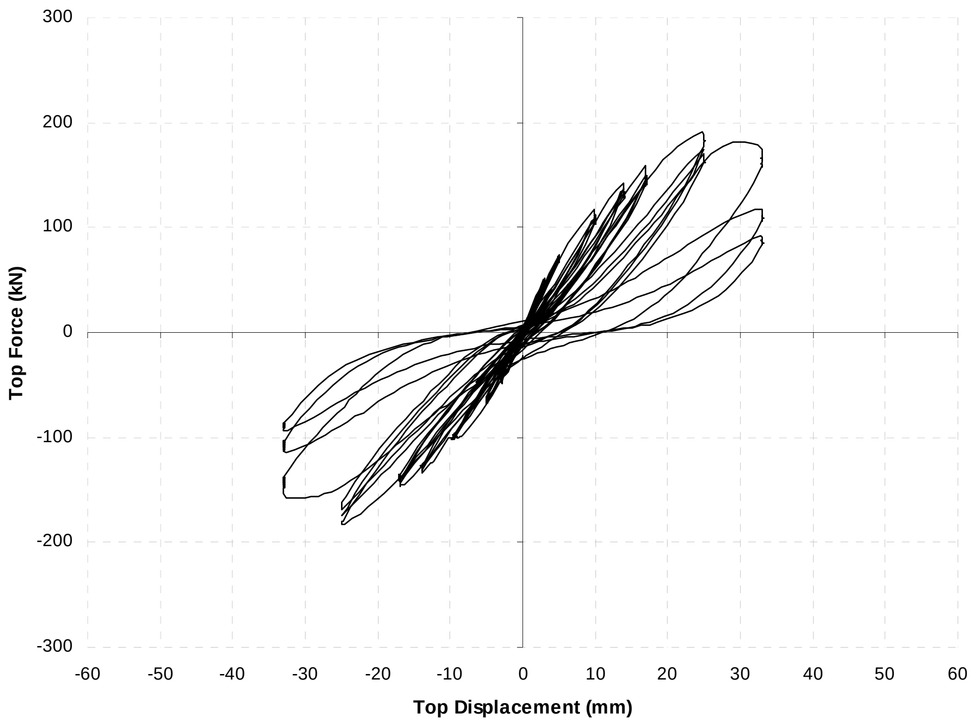


Fig. 9 Experimental results.

<u>Pier PO2</u>

As also described for the previous pier (PO1), the higher damage was mainly observed at the lateral sides, east and west, where the concrete cover crushed within the entire pier height (see Fig. 10) and strong shear damage was achieved due to concrete degradation caused by lack of transverse reinforcement efficiency. Again, little damage was observed on the north and south sides, with well distributed cracks. However, the cracks observed in those sides are not only horizontal, as for the square pier, but instead they show an angle that increases along the pier height, due the shear lag effect that occurs for this width / height ratio (2:1).

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

Fig. 10 Pier PO2 damage for 2.4% drift.

Fig. 11 show the experimental responses of both pier PO1 and PO2 where it is visible that, as expected, the rectangular section (PO2) achieved higher maximum forces, due to the significant increase of the longitudinal reinforcement and concrete area, when compared with the square pier.

From the experimental responses and damage patterns illustrated in Figs. 8 and 10, shear mode failure of both piers was achieved around the first cycles of 33mm amplitude (2.4% drift).

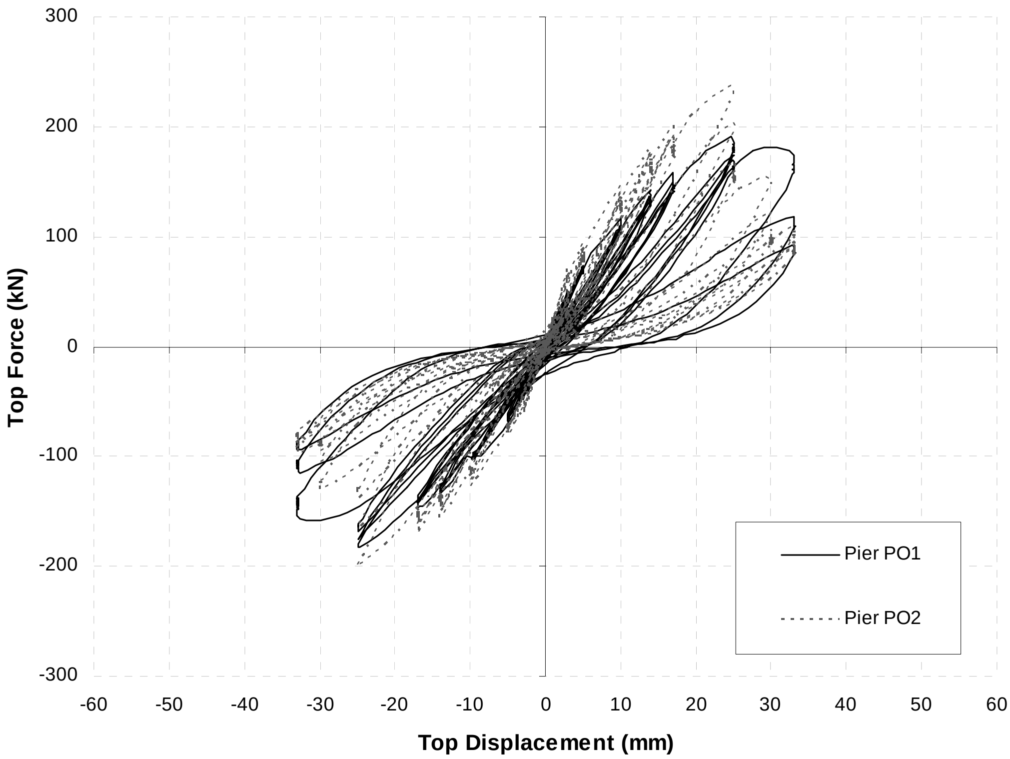


Fig. 11 Experimental results comparison of piers PO1 and PO2.

However, these higher forces in the pier PO2 (only about 1,2 times those of from PO1) is not proportion with the increase of longitudinal reinforcement of pier PO2, about 2 times more when compared with pier PO1, which is related with the shear lag effect quite apparent from the damage pattern really observed.

RETROFIT
After the cyclic test of the “as built” specimen up to failure, it was repaired and retrofitted by an external contractor (S.T.A.P.) 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. 12).

Fig. 12 Hollow square section pier (PO1) before and after the shear retrofitting with CFRP sheet

In order 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 above the maximum flexural force (see Eq. 1), but keeping the initial section conditions.

$$V_{sj} = (A_j\, f_j\, h \cot\theta)/s = (A_j\, 0.004E_j\, h \cot\theta)/s \qquad (1)$$

where h is the overall pier section 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. This condition is introduced 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. The shear capacity curve can therefore be predicted using the methodology proposed by Priestley et al (1996) for shear strength, according to Eq. 2.

$$V_d = V_c + V_s + V_p + V_{sj} \qquad (2)$$

where V_c, V_s and V_p are the shear force components accounting for the nominal strength of concrete, the transverse reinforcement shear resisting mechanism and the axial compression force. Thus using 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. 12.

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 increased intensity top displacements corresponding to 2.9% and 3.2% peak drift ratios.

The damage evolution during the experimental test of the retrofitted specimen is illustrated in Fig. 13, 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 subsequent cycles the dimensions of theses cracks increased and new cracks were developed along the pier height (Fig. 13a). For the top displacement of 17mm (1.2% drift - Fig. 13b), only the base crack has grown to about 0.5mm. Shear cracks appeared in the next cycles of 1.8% drift and generalized damage started to become visible. In the right hand side of Fig. 13c (2.14% drift), a vertical crack is visible over almost the entire pier height, which caused global damage to the CFRP sheets. Failure of some of the fibres was audible. During the cycles of 40mm top displacement (2.9% drift - Fig. 13d) 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 lack of concrete confinement (see also Figs. 14 and 15).

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

Fig. 13 Retrofitted pier PO1 damage from east side view.

(a) East side (b) South side

Fig. 14 Final damage of the retrofitted pier PO1 corresponding to a drift ratio of 3.2%.

As can be seen in Figs. 13 and 14, the retrofitted specimen showed a good behaviour in comparison with the “as built”, exhibiting very distributed cracking along the CFRP spacing.

(a) West side (b) North side

Fig. 15 Details of final damage.

The shear retrofit design, used on this pier, showed an excellent performance because the shear failure mechanism was partially prevented and the flexural collapse with buckling of the reinforcement bars was achieved. In Fig. 16 the comparison between the original and retrofitted pier is illustrated, where an increase on 30% of the top force and an increase of 20% at the maximum displacement obtained, without significant reduction of the resistant force.

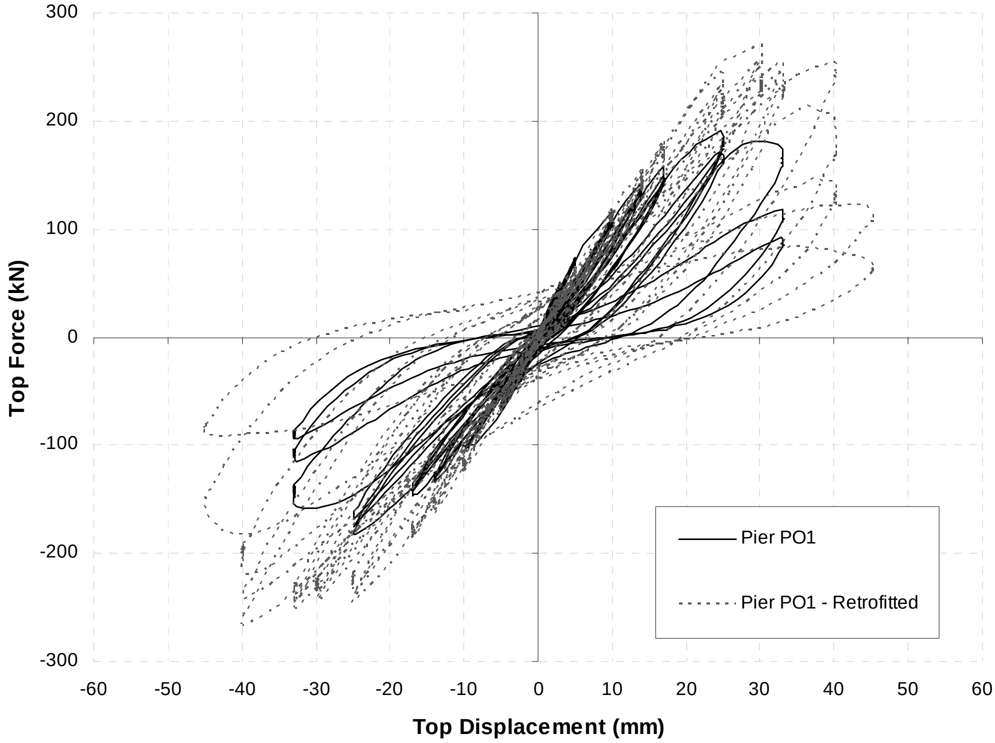


Fig. 16 Experimental results of pier PO1 before and after retrofit.

CONCLUSIONS

A setup for cyclic tests of piers maintaining the position of the axial load during the cyclic movement, with a slide device consisting of a steel plate system for the axial force transmission that allows the top displacements of the pier, is presented. A calibration test using a pin fixed steel column is described and the vertical reaction frame stiffness was evaluated. This and subsequent tests evidenced that the setup performed satisfactorily and showed low values of frictional forces, which can be measured and considered on the final results.

The PO1 and PO2 tests evidenced that the collapse was achieved because of an insufficient shear capacity. In both tests cracks were observed at lateral sides (east and west faces), where the concrete cover crushed within the entire pier height and severe shear damage was verified with significant concrete degradation due to the lower efficiency of the transverse reinforcement. For the PO2 pier it was also observed the shear lag effect evidenced by inclined cracks at the North and South sides, and the consequent reduction of the maximum predicable top horizontal force.

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

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