SEISMIC RETROFIT OF RC HOLLOW-SECTION PIERS WITH SHEAR FAILURE

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ABSTRACT :

The objective of this paper is to perform an experimental evaluation of seismic retrofit of reinforced concrete hollow piers with CFRP to prevent shear failure. The experimental campaign was carried out regarding the evaluation of: i) shear retrofit strategy efficiency and ii) the influence of different transverse reinforcement details on the shear capacity and concrete confinement. The experimental campaign has been carried out at LESE – FEUP (Laboratory of Earthquake and Structural Engineering of the Faculty of Engineering of University of Porto), on a set of four RC piers with hollow section. Within the scope of this work it is also the study of design procedures for CFRP shear retrofitting and the evaluation of the associated ductility capacity improvement. Efficiency assessment is pursued for different scenarios of shear failure prevention, and the corresponding seismic response enhancement is evaluated. The most relevant experimental information is presented in the paper, as for instance the evolution of shear cracks and the illustration of the outside and inside damage pattern.

KEYWORDS: RC hollow piers, CFRP retrofit, Shear failure, Experimental tests

1. EXPERIMENTAL CAMPAIGN

1.1. Testing setup

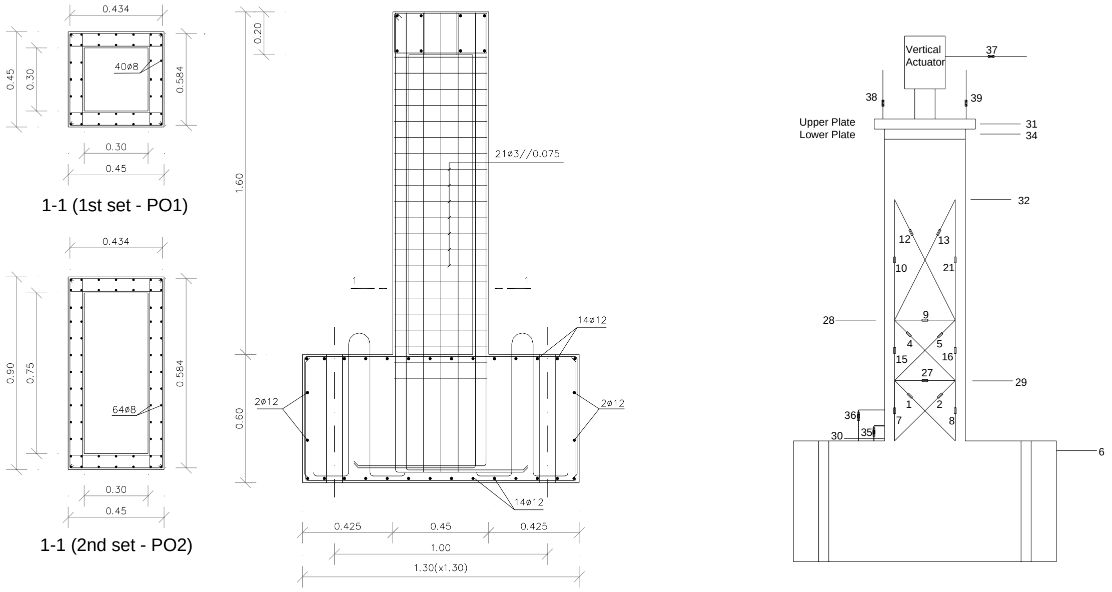
The test setup, shown in Figure 1, makes use of a 500 kN actuator to apply lateral loads and a 700 kN actuator to apply axial loads. The specimen and reaction frame are bolted to the strong floor with high strength prestressed rods. A constant axial load of 250 kN was applied during the tests, herein described, while the lateral loads were cycled, under displacement controlled conditions (Delgado (2007) and Delgado (2008)).

Figure 1 General view of the test setup at LESE laboratory

1.2. Specimens and Instrumentation

The specimens presented in this paper correspond to one group of piers tested within this framework, being same results already shown in previous reports (Delgado *et al.* (2007a) and Delgado *et al.* (2007b)). This set of specimens was based on square piers tested at the Laboratory of Pavia University, Italy (Calvi *et al.* (2005);

Pavese *et al.* (2004)), consisting on hollow section RC piers with 450mm x 450mm exterior dimensions and 75mm thick walls, and is being tested in order to understand the influence of the cross section geometry of rectangular hollow piers on the cyclic behavior, bearing in mind the purpose of assessing retrofitting solutions. The unconfined concrete compressive strength is 35 MPa and for both longitudinal and transversal reinforcement the yielding strength is 450 MPa. The model schemes shown in Figure 2a correspond to ¼ scale representations of hollow section bridge piers, herein referred to as PO: PO1 for square section and PO2 for rectangular section. Instrumentation to measure curvature and shear deformations was included along the pier height, because important shear deformations were expected in these tests. The LVDT configuration used in both specimens is shown in Figure 2b.



a) b)

Figure 2 Hollow RC piers: a) geometry of specimen and b) lateral LVDT layout.

2. CYCLIC TESTS

In order to characterize the cyclic behavior of piers, three cycles were applied for each of the peak displacements shown in Figure 3 and Table 1. These values were applied under a constant axial load of 250 kN that corresponds to a normalized axial force of 0.08, except for pier PO2-N3 that was tested for 440kN axial load (normalized axial force of 0.05), Delgado (2008).

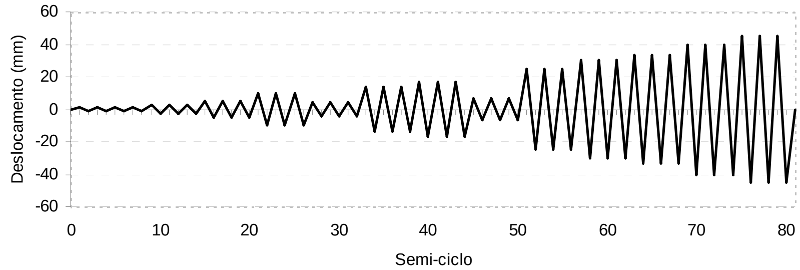


Figure 3 Top-displacements during the tests

Table 1 Top-displacements and drifts during the tests

Desloc. (mm)	1	3	5	10	4	14	17	7	25	30	33	40	45
Drift (%)	0.07	0.21	0.35	0.7	0.28	1.0	1.2	0.5	1.8	2.1	2.4	2.9	3.2

Concerning the results of the PO1-N2 original specimen test, the damage observed in the north and south sides was highly concentrated at the pier base with concrete crush and longitudinal reinforcement buckling, as shown in Figure 4a, and along the pier height well distributed cracks were visible. However, at the lateral sides, east and west (Figure 4b) strong damage was also found, where the concrete cover crushed within the entire pier height. At the final stage, shear and flexural damage was observed with significant concrete degradation due to the low efficiency of the transverse reinforcement. The damage of pier PO1-N3, illustrated in Figure 4c and 4d after the cyclic test, was quite similar to the previous square pier (PO1-N2), since both piers have identical geometry and materials. Therefore, these two piers also served to assess the reliability of the pier test setup.

(a) South side - flange (b) West side - web (c) North side - flange (d) West side - web

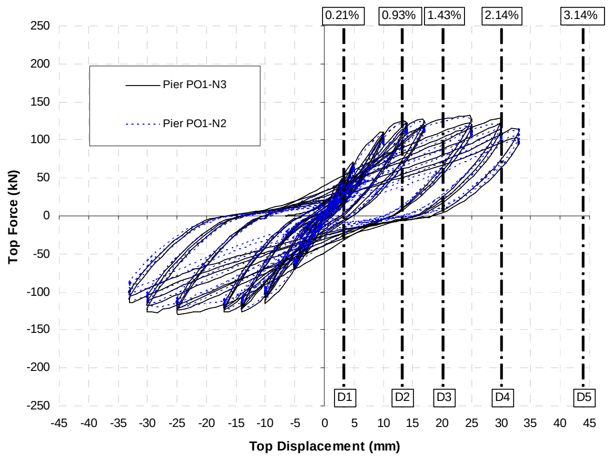
Figure 4 Piers damage for 2.4% drift, PO1-N2 [a) and b)] and PO1-N3 [c) and d)]

As for the test results of original specimens PO2-N2 and PO2-N3, the greatest amount of damage was mainly observed at the lateral sides, east and west, where the concrete cover crushed within the entire pier height (see Figure 5) and strong shear damage was achieved due to concrete degradation caused by lack of transverse reinforcement efficiency. Little damage was observed in the north and south sides, with well distributed cracks. However, the cracks observed in those sides are not only horizontal, as for the square piers tested before, 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) and also found in other tests within this campaign, Delgado *et al.* (2006).

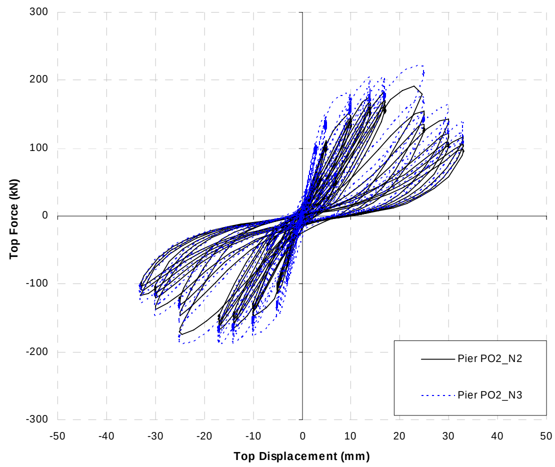
(a) North side - flange (b) West side - web (c) North side - flange (d) West side - web

Figure 5 Piers damage for 2.4% drift, PO2-N2 [a) and b)] and PO2-N3 [c) and d)]

The cyclic response of piers PO1-N2 and PO1-N3 are shown in Figure 6a, where very similar behavior can be found in terms of stiffness and maximum top force achieved. A mixed shear-flexure mechanism was observed on the pier response, since it seems to have a ductile behavior, but with an achieved horizontal force below its flexural capacity. Figure 6b shows the experimental responses of both piers PO2-N2 and PO2-N3 in terms of top force-displacement diagrams; as expected, these diagrams show larger initial stiffness and peak force for the pier with higher axial load (PO2-N3). However, failure of both piers was reached by the first cycles of 25mm amplitude (1.8% drift), with visible shear failure mode and the shear lag effect evidenced in the damage pattern shown in Figure 5.



a) PO1-N2 vs. PO1-N3



b) PO2-N2 vs. PO2-N3

Figure 6 Experimental results comparison

3. SEISMIC RETROFIT

After the original specimen cyclic tests, piers were repaired and retrofitted by an external contractor (S.T.A.P.) according to the following steps: 1) delimitation of the repair area; 2) removal and cleaning of the damaged concrete; 3) inside retrofit with transversal steel bars; 4) 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); 5) outside retrofit with the CFRP sheets. In order to provide a general idea of the pier damage and of the retrofit process, the following pictures show the piers during repair and after retrofit with CFRP sheet jacketing (Figure 7). The inside retrofit with transversal steel bars (only for pier PO2-N3) was designed taking in account the feasibility for future real retrofits; such bars were concentrated at the bottom, in correspondence with the outer CFRP jackets, for improving the plastic hinge confinement. In order to design the outside shear retrofit with CFRP jackets, the methodology suggested by Priestley (1996) was adopted to evaluate the thickness of the rectangular hollow pier jacket for increasing the shear strength above the maximum flexural force while keeping the initial section conditions. According to this methodology the shear strength can be conveyed by Eqn. 4.1 [11]:

$$V_d = V_c + V_s + V_p + V_{sj} \tag{4.1}$$

where V_c, V_s and V_p are the shear force components accounting, respectively, for the nominal strength of concrete, the transverse reinforcement shear resisting mechanism and the axial compression force; the term V_{sj} corresponds to the possible retrofit contribution with CFRP or metal jackets and can be estimated according to Eqn. 4.2 [11]

$$V_{sj} = \frac{A_j}{s} f_j \cdot h \cdot \cot\theta = \frac{A_j}{s} 0.004 E_j \cdot h \cdot \cot\theta \tag{4.2}$$

where h is the overall pier section dimension parallel to the applied shear force, f_j is the adopted design jacket stress level corresponding to a jacket strain of 0.004, A_j is the transverse section area of the jacket sheets spaced at distance s and inclined of the angle θ relative to the member axis. 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.

Figure 7 Hollow piers before and after the shear retrofitting with inside steel bars and outside CFRP sheet.

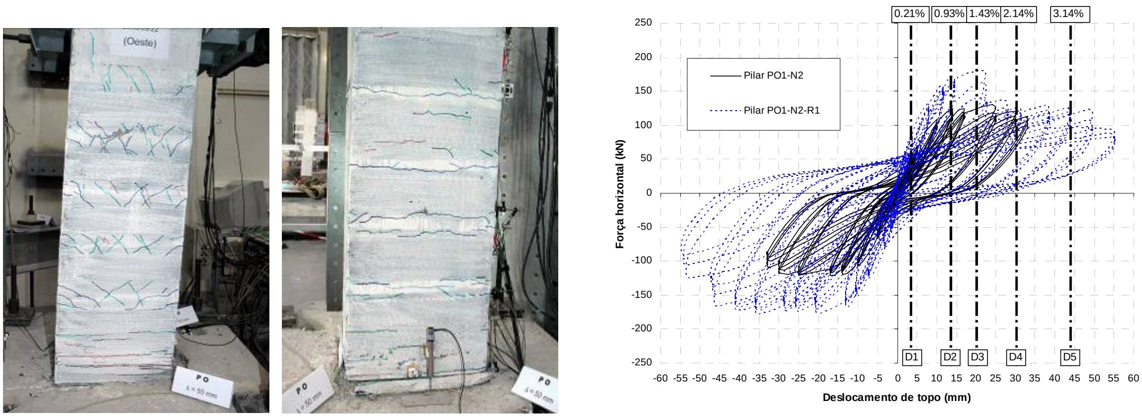
Therefore, using Eqn 4.1 in order to increase the shear capacity of specimens, the number of 0.117mm thick CFRP strip layers was estimated. This retrofit was doubled at the pier base for improving the concrete confinement and, therefore, the overall pier ductility.

4. CYCLIC TEST OF THE RETROFITTED SPECIMENS

The retrofitted piers have been tested following the same cyclic displacement history of the original specimens, but three additional cycles were performed with increased top displacement amplitude corresponding to 2.9%, 3.2% and 3.6% peak drift ratios. Results for both specimens are included in the following sections and fully described in Delgado (2008).

4.1. Pier PO1-N2-R1

For this specimen retrofitting two strip layers of CFRP sheet were adopted with 0.117mm thickness by 100mm width and spaced at 100mm along the pier height in order to increase the shear capacity. As above referred, this retrofit was doubled at the pier base, leading to a first strip layer 300mm wide. This CFRP retrofit design evidenced excellent benefits on the pier behavior (see Figure 8) since it avoided the shear collapse and allowed mobilizing a flexure mechanism with plastic hinge at the pier base (longitudinal reinforcement bucking). By comparing with the original ones, these retrofitted piers reached a significant improvement on the horizontal force capacity and maximum displacement achieved.

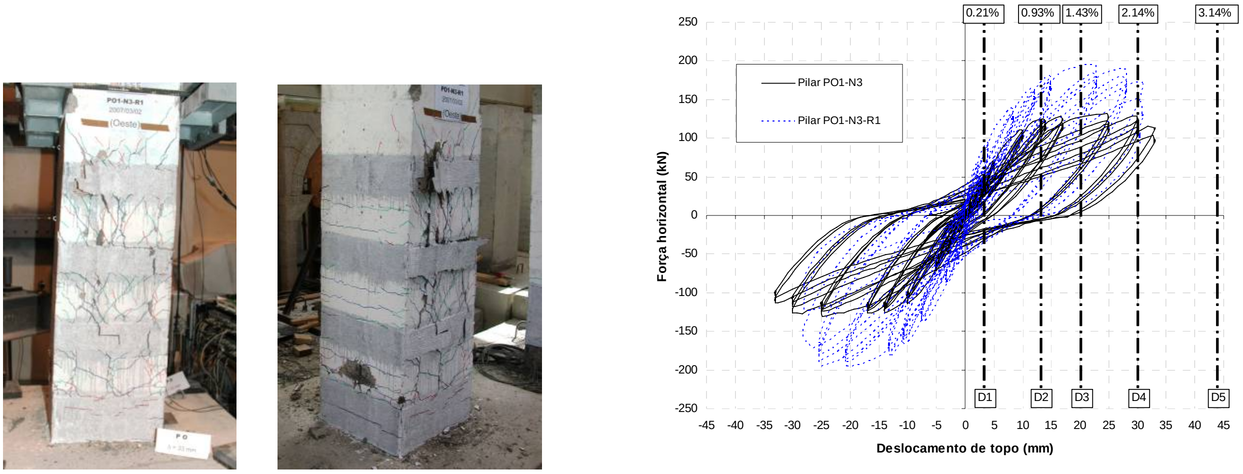


a) Final damage - west b) Final damage – south c) Comparison before and after retrofit

Figure 8 Experimental results of pier PO1-N2-R1.

4.2. Pier PO1-N3-R1

For this specimen retrofitting only one strip layer of CFRP sheet was adopted with 0.117mm thickness by 100mm width and spaced at 200mm along the pier height in order to increase the shear capacity; as before, this retrofit was increased at the pier base, with a 200mm wide first strip layer. This retrofit aimed at an ideal shear design, with only 15% increase over the maximum flexural force, but it was insufficient as illustrated further on.



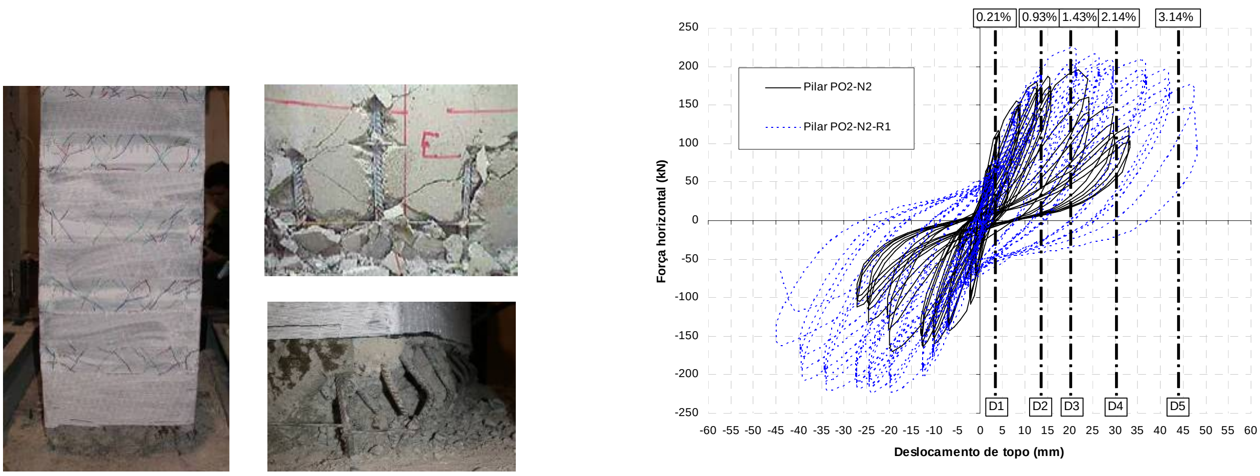
a) Final damage - west b) Final damage – northwest c) Comparison before and after retrofit

Figure 9 Experimental results of pier PO1-N3-R1.

This retrofit exhibited earlier cracking between CFRP strips and the pier failure was achieved after the collapse of the first carbon strip (see Figure 9). By comparing with the original piers, at least 25% increase was obtained for the maximum force, but without significant increase on the maximum displacement.

4.3. Pier PO2-N2-R1

As for the square pier with the same number (PO1-N2-R1), this specimen retrofit aimed essentially at preventing the shear failure observed on the original specimen. Thus, two strip layers of CFRP sheet with 0.117mm thickness by 100mm width and spaced at 100mm were adopted along the pier height, though with 300mm width on the first strip.

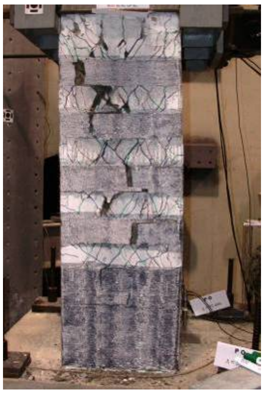


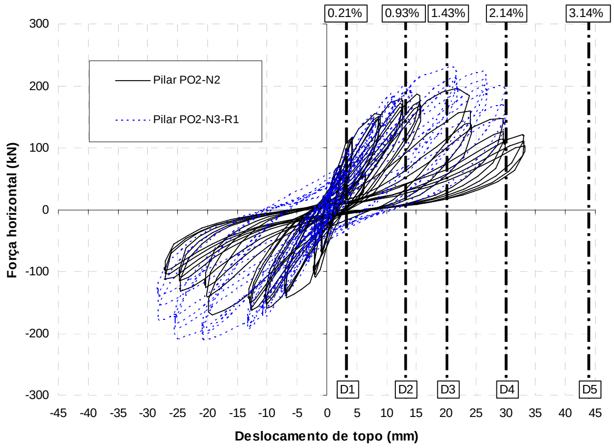
a) Final damage - west b) Final damage – inside / base c) Comparison before and after retrofit

Figure 10 Experimental results of pier PO2-N2-R1.

4.4. Pier PO2-N3-R1

This specimen retrofit included only one strip layer of CFRP with 0.117mm thickness by 100mm width, except the first strip layer at the pier base with 400mm width, and spaced at 100mm along the pier height. Again, this retrofit objective was to obtain an ideal shear design, with only 15% increase over the maximum flexural force, but it was insufficient as shown below. An inside retrofit was adopted in this pier using three 10mm thick transversal steel bars, 40mm wide and spaced at 70mm, bearing in mind the feasibility for future real retrofits. Such bars were concentrated at the bottom (for improving the plastic hinge confinement) and pos-tensioned after the application of the outer CFRP jackets with screw bars crossing the pier flange wall thickness.



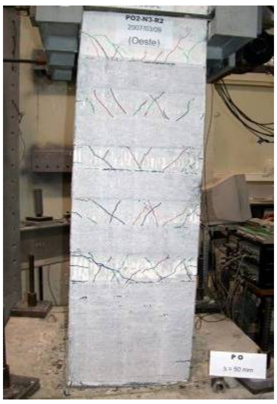


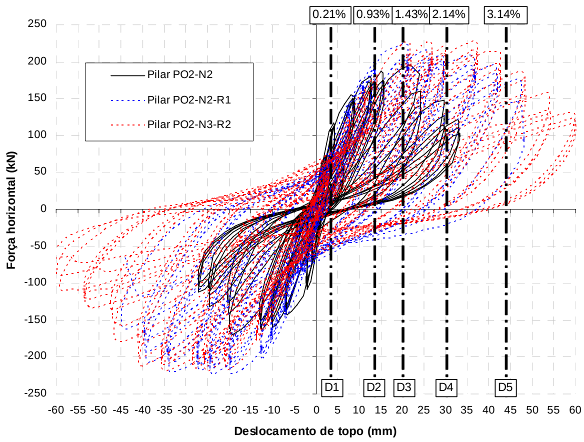
a) Final damage - west   b) Final damage – south   c) Comparison before and after retrofit

Figure 11 Experimental results of pier PO2-N3-R1.

4.5. Pier PO2-N3-R2

After the previous test that still exhibited shear failure due to the CFRP strip collapse, the same external retrofit was adopted but now with two strip layers of CFRP 0.117mm thick sheets, 100mm wide and the same 100mm spacing along the pier height. Again the first strip layer at the pier base was adopted with 400mm width (two layers also). And the internal retrofit was kept as before with three horizontal steel bars.





a) Final damage - west   b) Final damage – inside / base   c) Comparison before and after retrofit

Figure 12 Experimental results of pier PO2-N3-R2.

5. CONCLUSIONS

For original specimen tests with square cross section, the damage observed in the north and south sides was highly concentrated at the pier base with concrete crushing and longitudinal reinforcement buckling. The response seems to have a ductile behavior, but with an achieved horizontal force below its flexural capacity, leading to conclude that a mixed shear-flexure mechanism was observed. Concerning the response of rectangular cross section specimens, experiments evidenced that the collapse was achieved due to insufficient shear capacity at the first cycle of 25mm displacement (1.8% drift).

The CFRP retrofit design with two layers (PO1-N2-R1 e PO2-N2-R1) showed excellent benefits on both piers behavior since the shear collapse was avoided and a flexure mechanism with plastic hinge at the pier base was achieved. A significant improvement on the horizontal force capacity and maximum displacement was achieved for retrofitted piers.

Pursuing the objective of retrofitting for an ideal shear design, with only 15% increase over the maximum flexural force, tests have shown that such strategy is insufficient to allow a satisfactory ductility behavior, since cracks started developing earlier between CFRP strips and pier failure was achieved after the first strip collapse. Even so, the peak force increased to 25% over the flexural force.

Finally, by adopting a second retrofitting strategy, with two layers of CFRP and keeping internal retrofit, a good behavior and significant increase of maximum displacement was reached when compared with the previous retrofit option. Again, this shear retrofit design leads to an excellent response, since shear failure mechanism was avoided and the collapse was reached with a flexural mechanism. This occurred for a very large top displacement when the CFRP sheets lost the confinement capacity and allowed rebar buckling in the outer face of the pier wall; while the interior transversal steel bars were still effective in avoiding internal rebar buckling.

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REFERENCES

Calvi, G..M. Pavese, A. Rasulo, A. and Bolognini, D. (2005). Experimental and Numerical Studies on the Seismic Response of R.C. Hollow Bridge Piers, *Bulletin of Earthquake Engineering*, **3 (3)**, 267-297.

Delgado, P. Rodrigues, V. Rocha, P. Santos, M. Arêde, A. Pouca, N. V. Costa, A. and Delgado, R.. (2006). Experimental Cyclic Tests and Retrofit of RC Hollow Piers, 13th European Conference on Earthquake Engineering, 3-8 September, Geneva, Switzerland.

Delgado, P. Rocha, P. Pedrosa, J. Arêde, A. Pouca, N.V. Santos, M. Costa, A. and Delgado, R. (2007a). Retrofitting of Bridge Hollow Piers with CFRP. ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, 13-16 July Rethymno, Crete, Greece.

Delgado, P. Rocha, P. Santos, M. Pedrosa, J. Arêde, A. Pouca, N. V. Costa, A. and Delgado, R. (2007b) Experimental and Analytical Cyclic Tests of RC Hollow Section Piers Retrofitted with CFRP. FRPRCS-8, 8th International Symposium on Fiber Reinforced Polymer Reinforcement for Concrete Structures. Patras, Greece.

Delgado, P. (2008). Avaliação da Segurança Estrutural em Pontes. PhD Thesis, FEUP, Porto. (in Portuguese)

Pavese, A. Bolognini, D. Peloso, S. (2004). FRP seismic retrofit of RC square hollow section bridge piers, *Journal of Earthquake Engineering*, **1 SPEC. ISS.**, 225-250.

Priestley, M.J.N. Seible, F. Calvi, G..M. (1996). Seismic Design and Retrofit of Bridges, John Wiley & Sons, New York.