Strategies for the retrofit of hollow piers under cyclic loads

P. Delgado & P. Rocha
Instituto Politécnico de Viana do Castelo, Portugal

A. Arêde, N. Vila Pouca & R. Delgado
Universidade do Porto, Portugal

A. Costa
Universidade de Aveiro, Portugal

SUMMARY:
The main objective of this paper is to present a strategy of retrofit RC hollow piers subjected to horizontal cyclic actions with axial load. Mainly, the retrofit technics aim to increase the shear strength and the ductility capacity through the establishment of principles and strategies applied in an experimental cyclic campaign of RC hollow piers, carried out in the Laboratory for Earthquake and Structural Engineering (LESE) of the Faculty of Engineering of University of Porto (FEUP). The evaluation and calibration of the efficiency of several retrofit solutions is also performed. The design criteria for the piers retrofit is presented, namely, the CFRP sheets for shear retrofit and CFRP base strip or steel bars applied inside the hollow box for ductility improvement, being the different types of retrofit grouped out in order to understand the main benefits of each one. Results of the experimental campaign allow to discuss and conclude about the efficiency of each retrofit solution.

Keywords: RC hollow piers, experimental tests, retrofit strategies

1. HOLLOW PIERS WITH SHEAR DAMAGE

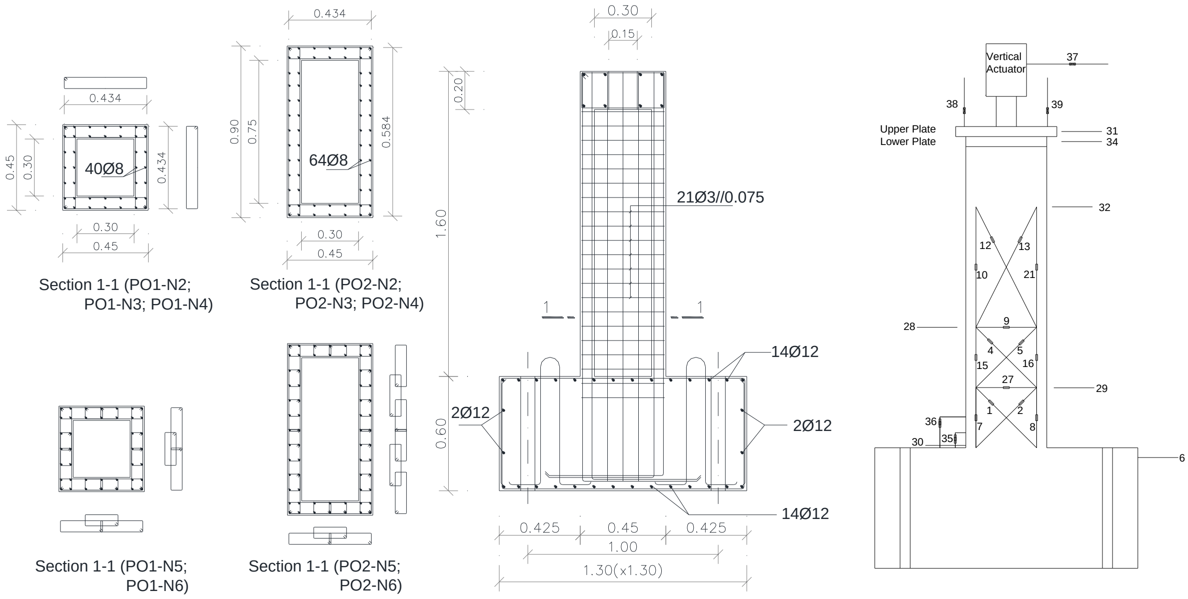
Reinforced concrete hollow section piers are usually one type of structures that sustain seriously damage, as clearly demonstrated in several reports of recent earthquakes. In comparative terms, these consequences of bridge piers vulnerability are found greater than those observed in building structures and, in most cases, the bridge safety is limited and conditioned by pier capacities. Several studies and works have been carried out on solid piers and can be applied to building structures; however, for hollow piers much less research is found in the literature. Usually, hollow piers have large section dimensions, with reinforcement bars spread along both wall faces, and unlike common solid section columns, quite often the shear effect has great importance on the pier behaviour. Thus, special attention should be given to this issue when the assessment and retrofit of such type of section piers is envisaged.

In line with this concern, an 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 RC piers. The test setup characteristics and more detailed results are available in previous reports (Delgado *et al.* (2007a); Delgado *et al.* (2007b); Delgado *et al.* (2009); Delgado (2009)). 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 behaviour, bearing in mind the purpose of assessing retrofitting solutions.

Ten prototypes with square and rectangular hollow section (see Table 1.1 and Fig 1) were tested with different transverse reinforcement details, namely: piers N2, N3 and N4 - simple stirrups with 2 legs (representative of the old bridge construction); piers N5 - more detailed transverse reinforcement with 2 legs, with EC8 type, and piers N6 - also detailed stirrups (like piers N5), but with 4 legs (twice cross section area of transverse reinforcement). The unconfined concrete compressive strength (f_c) is 28 MPa and the longitudinal and transversal reinforcement yielding strengths (f_{sy} and f_{swy}) are represented in Table 1.1. The model schemes shown in Fig. 1a correspond to ¼ scale representations of bridge piers with square and rectangular hollow section, herein referred to as PO1 and PO2. 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 Fig. 1b.

Table 1.1. Resume of hollow piers characteristics

Designation	Geometry	Concrete	Long. Reinf.		Transv. Reinf.		
		f_c (MPa)	area	f_{sy} (MPa)	ϕ (mm)	f_{swy} (MPa)	type
PO1-N2	Square	28	40ϕ8	435	2.6	440	2 legs
PO1-N3	Square	28	40ϕ8	435	2.6	440	2 legs
PO1-N4	Square	28	40ϕ8	560	2.6	440	2 legs
PO1-N5	Square	28	40ϕ8	560	2.6	440	2 legs (EC8)
PO1-N6	Square	28	40ϕ8	560	2.6	440	4 legs (EC8)
PO2-N2	Rectangular	28	64ϕ8	435	2.6	440	2 legs
PO2-N3	Rectangular	28	64ϕ8	435	2.6	440	2 legs
PO2-N4	Rectangular	28	64ϕ8	560	2.6	440	2 legs
PO2-N5	Rectangular	28	64ϕ8	560	2.6	440	2 legs (EC8)
PO2-N6	Rectangular	28	64ϕ8	560	2.6	440	4 legs (EC8)



a) b)

Figure 1. Hollow RC piers: a) geometry of specimen and b) lateral LVDT layout.

To define the values of Table 1.2, simple calculation were carried out for the cross section to compute the flexural capacity, while to evaluate the shear capacity the methodology suggested by Priestley (1996) was adopted, usually referred in the bibliography as original UCSD shear model (Kowalsky e Priestley (2000)), being the shear strength (V_d) obtain for these piers through Eqn. 1.1:

$$V_d = V_c + V_s + V_p \tag{1.1}$$

where V_c, V_s and V_p are the shear force components accounting, respectively, for the nominal strength of concrete (which depends on the pier displacement ductility), the transverse reinforcement shear resisting mechanism and the axial compression force. These values were obtained for an axial load of 250 kN (except pier PO2-N3 with 440kN) that corresponds to a normalized axial force of 0.08 and 0.05, respectively, for the square and rectangular pier cross sections, Delgado (2009).

Table 1.2. Resume of flexural and shear capacities

Designation	Geometry	Flexural Capacity (kN)		Shear Capacity (kN)	
		yielding	ultimate	ductility of 2	ductility of 8
PO1-N2	Square	155	180	170	105
PO1-N3	Square	155	180	170	105
PO1-N4	Square	185	215	170	105
PO1-N5	Square	185	215	170	105
PO1-N6	Square	185	215	220	160
PO2-N2	Rectangular	230	265	170	105
PO2-N3	Rectangular	255	290	200	135
PO2-N4	Rectangular	280	320	170	105
PO2-N5	Rectangular	280	320	170	105
PO2-N6	Rectangular	280	320	220	160

The damage on the flange walls (face north and south) exhibited mainly horizontal cracking along the pier height. On the other hand it becomes clear that the pier webs (lateral faces east and west) were the most damaged zones of all specimens, exhibiting inclined cracking and concrete spalling on extensive zones (Fig. 2). Generally, these damage patterns are associated with shear mechanisms, revelling insufficient shear capacity provide by transverse reinforcement. However, some square piers have shear capacity slightly above the flexural capacity (see Table 1.2), which have resulted on a mixed collapse mechanism, bending/shear.

(a) square – flange (b) square – web (c) rectangular – flange (d) rectangular – web

Figure 2. Typical final damage on the webs and flange for square and rectangular cross sections piers.

For illustration purpose the cyclic response of two piers (PO1-N4 and PO1-N6) are shown in Fig. 3, where it is also included the shear capacity lines for both piers, Priestley (1996). Since both piers have the same maximum flexural force of about 200kN, associated with the yielding of the longitudinal reinforcement, premature shear failure was found for pier PO1-N4. On the other hand, pier PO1-N6 achieved the maximum flexural force, but with a moderately low ductility capacity and finally failing with shear mechanism. In the Table 1.3 is presented a summary of shear and flexural capacities, the values of experimental results, as well as the collapse mechanism. The definition of collapse displacement corresponds to the value when the horizontal force applied to the pier reached 80% of the maximum force. For almost all the piers the shear capacity is clearly below the flexural capacity, being the maximum experimental force closer to the numerical estimation of shear strength, although in some cases the experimental response has reached a peak force slightly above the expected values.

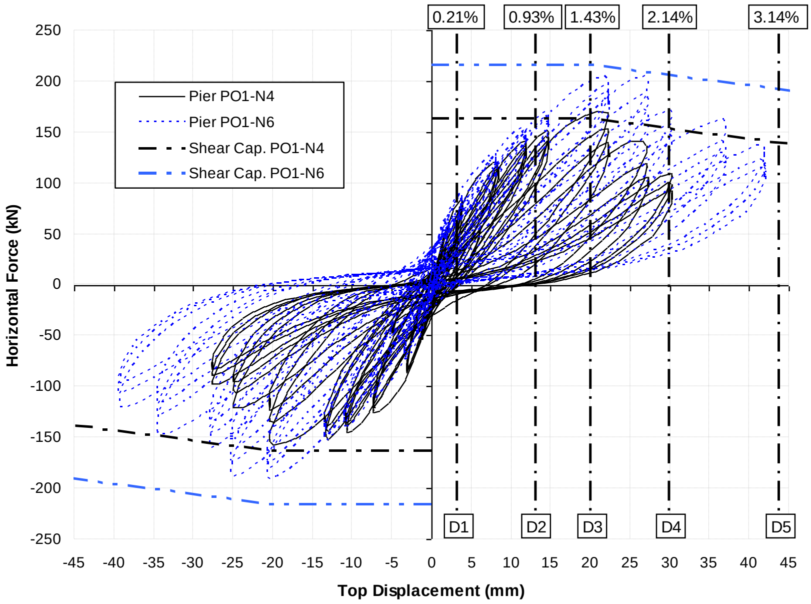


Figure 3. Experimental results comparison – PO1-N4 vs. PO1-N6.

Table 1.3. Resume of forces and collapse mechanism for square and rectangular cross section piers

Piers	Numerical Flexural Cap. (kN)	Numerical Shear Cap. (kN)	Experimental Max. Force (kN)	Experimental Collapse Displ. (mm)	Collapse mechanism
PO1-N2	155 / 180	170 / 105	130	33	Flexural / Shear
PO1-N3	155 / 180	170 / 105	130	33	Flexural / Shear
PO1-N4	185 / 215	170 / 105	170	25	Shear
PO1-N5	185 / 215	170 / 105	170	25	Shear
PO1-N6	185 / 215	220 / 160	210	30	Shear
PO2-N2	230 / 265	170 / 105	190	25	Shear
PO2-N3	255 / 290	200 / 135	220	25	Shear
PO2-N4	280 / 320	170 / 105	190	30	Shear
PO2-N5	280 / 320	170 / 105	200	30	Shear
PO2-N6	280 / 320	220 / 160	250	40	Shear

2. SEISMIC RETROFIT PROCESS AND DESIGN

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) alignment or replacement of the longitudinal rebars; 5) 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); 6) 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 (Fig. 4). 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. 2.1:

$$V_d = V_c + V_s + V_p + V_{sj} \tag{2.1}$$

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. 2.2:

$$V_{sj} = \frac{A_j}{s} f_j \cdot h \cdot \cot\theta \tag{2.2}$$

where h is the overall pier section dimension parallel to the applied shear force, f_j is the adopted design jacket stress, 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.

Figure 4. Hollow piers before and after the shear retrofitting with inside steel bars and outside CFRP sheet.

Therefore, using Eqn 2.1 in order to increase the shear capacity of specimens, the number of 0.117mm thick CFRP strip layers was estimated. On some specimens, this retrofit was doubled near the foundation for improving the concrete confinement of the pier base and, therefore, the overall pier ductility.

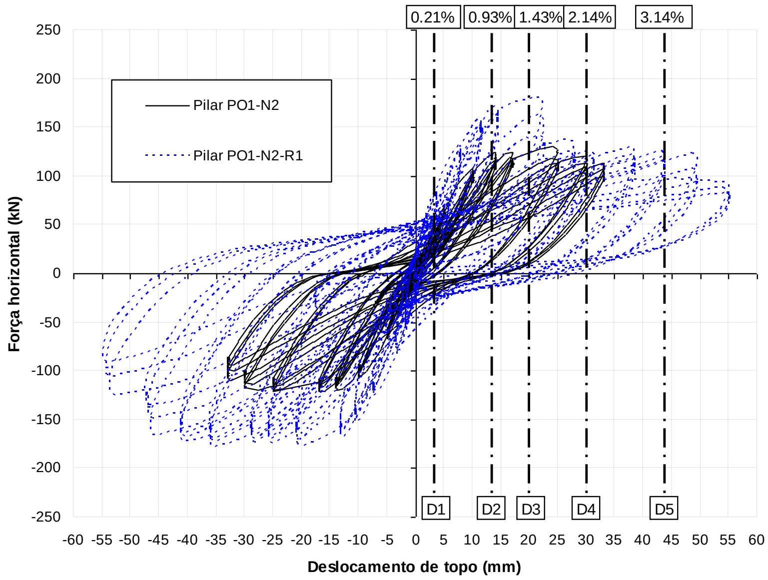
3. RESULTS OF RETROFIT SOLUTIONS UNDER CYCLIC LOADS

The retrofitted piers have been tested following the same cyclic displacement history of the original specimens, but with additional cycles when necessary. Experimental results for the piers are included in the following sections and fully described in Delgado (2009).

3.1. Overestimated level of retrofit

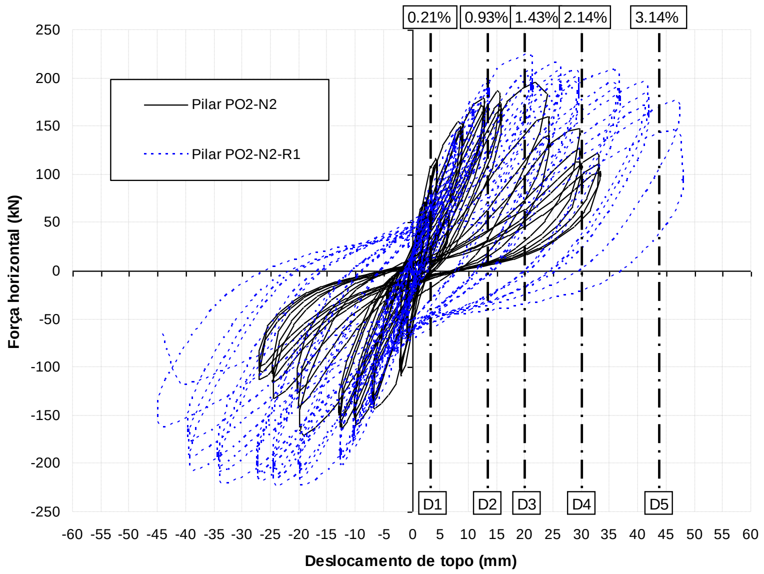
For these piers it was intended to obtain the shear retrofit with a significant level of safety, more than 100% in comparison to the maximum force that can be mobilized for bending. Therefore, for the piers PO1-N2 and PO2-N2, two strip layers of CFRP sheet were adopted with 0.117mm thickness by 100mm width and spaced at 100mm along the pier height in order to increase the shear capacity. The retrofit was doubled at the piers base, leading to a first strip layer 300mm wide.

For both piers, PO1-N2 and PO2-N2, this CFRP retrofit evidenced excellent benefits on the piers behaviour (see Fig. 5 and Fig. 6) 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, with level of displacement ductility of 4.



a) Comparison before and after retrofit b) Final damage

Figure 5. Experimental results of pier PO1-N2-R1.



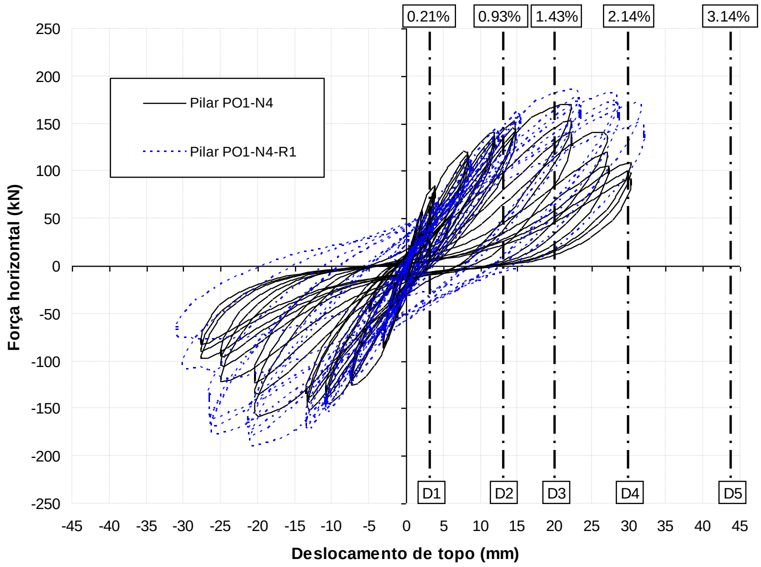
a) Comparison before and after retrofit b) Final damage - inside / base

Figure 6. Experimental results of pier PO2-N2-R1.

3.2. Optimized level of retrofit

For these piers it was intended to optimize the shear retrofit, corresponding to an increase of 40% in comparison to the maximum force that can be mobilized for bending. Therefore, for the square piers PO1-N4, PO1-N5 and PO1-N6, one strip layer 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.

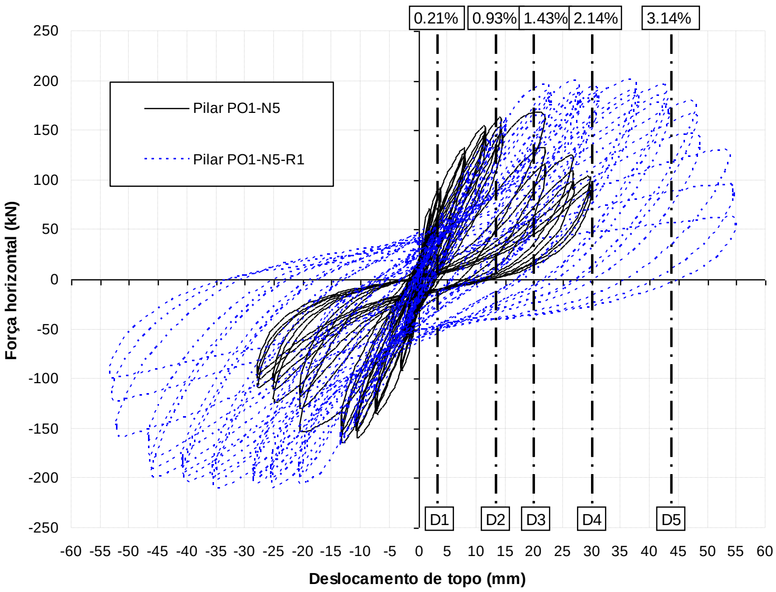
The comparison between the original and retrofitted pier (PO1-N4 and PO1-N4-R1) is illustrated in Fig. 7, where a slightly improvement on the maximum horizontal force was reached (approximately 10%) and also a slightly increase on the maximum displacement was achieved. This CFRP retrofit design evidenced benefits on the pier behaviour (see Fig. 7) since it allowed mobilizing a flexure mechanism with plastic hinge at the pier base. However the pier failure was achieved after the rupture of the first CFRP strip, revealing a premature shear failure.



a) Comparison before and after retrofit b) Final damage

Figure 7. Experimental results of pier PO1-N4-R1.

For the retrofit of pier PO1-N5, the first strip near the pier base which was performed with 200mm width. The comparison between the original and retrofitted pier (PO1-N5 and PO1-N5-R1) is illustrated in Fig. 8, where an improvement of approximately 20% on the maximum horizontal force was reached.

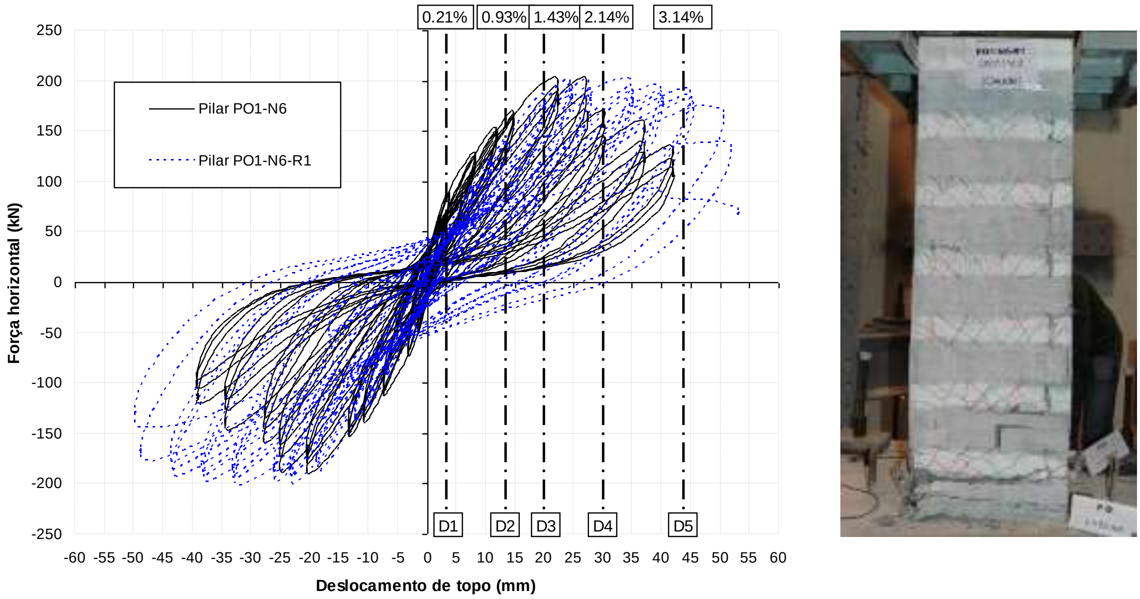


a) Comparison before and after retrofit b) Final damage

Figure 8. Experimental results of pier PO1-N5-R1.

Due to the shear failure prevention and improvement on the confinement at the pier base provided by the CFRP strips, significant increase on the maximum displacement was achieved, corresponding to a displacement ductility capacity of about 3. Therefore, this CFRP retrofit design evidenced significant benefits on the pier behaviour (see also final damage in Fig. 8) since it allowed mobilizing a flexure mechanism with plastic hinge at the pier base and satisfactory ductility achieved.

In Fig. 9, the comparison between the original and retrofitted pier (PO1-N6 and PO1-N6-R1) is illustrated, where the maximum horizontal force reached in both piers was almost identical, confirming that in the original pier the maximum yielding force associated with bending behaviour was achieved.



a) Comparison before and after retrofit b) Final damage

Figure 9. Experimental results of pier PO1-N6-R1.

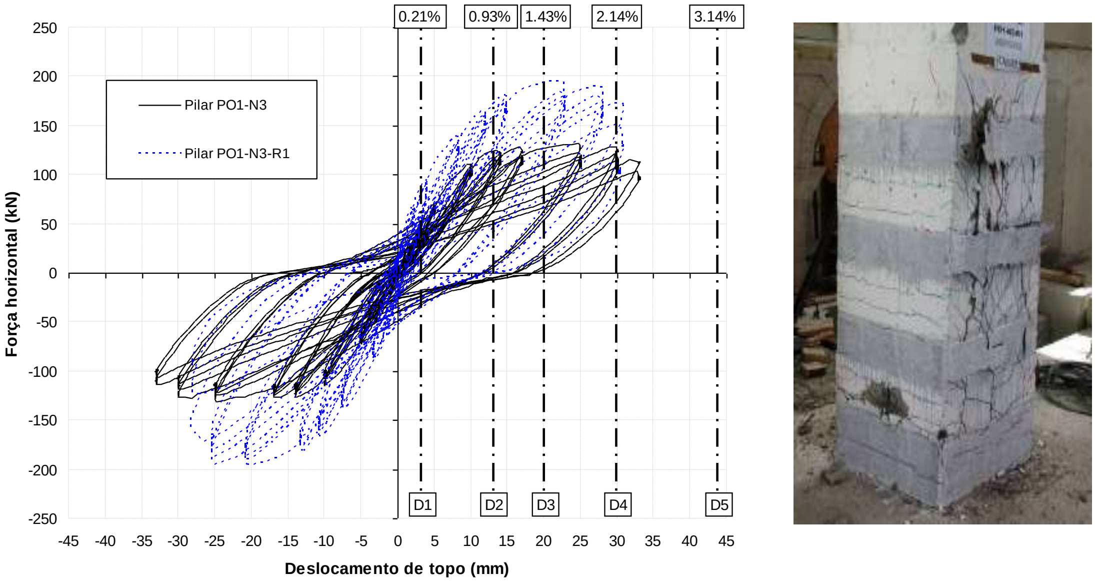
Due to the shear failure prevention provided by the CFRP strips, significant increase on the maximum displacement was achieved (see Fig. 9), corresponding to a displacement ductility capacity of about 3. Therefore, for this pier with double transverse reinforcement area, the CFRP retrofit carried out evidenced significant benefits on its behaviour since a flexure mechanism with plastic hinge at the pier base was mobilized and satisfactory ductility achieved.

3.3. Underestimated level of retrofit

The objective of these piers retrofit was to achieve the shear capacity with the same (or slightly above) the maximum force that can be mobilized for bending. Therefore, for the square pier PO1-N3 one strip layer of CFRP sheet were adopted with 0.117mm thickness by 100mm width and spaced at 200mm along the pier height, at the pier base the first strip layer was considered with a 200mm wide. The rectangular pier (PO2-N3) was also retrofitted with one strip layer of CFRP with 0.117mm thickness by 100mm width, except the first strip layer at the pier base with 400mm width, but with spacing of 100mm along the pier height. 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.

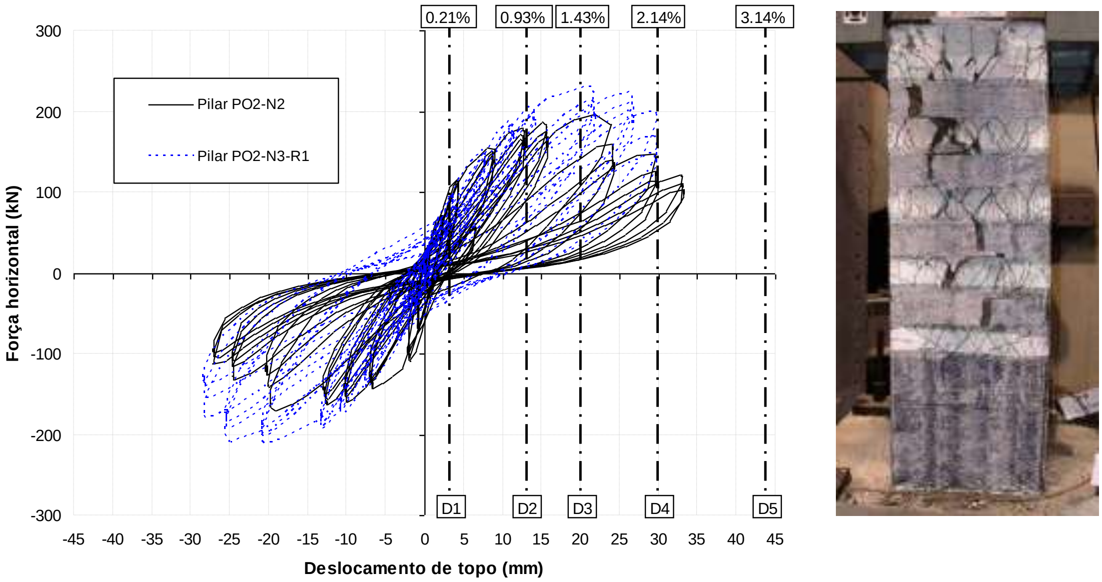
This retrofit exhibited earlier cracking between CFRP strips and the pier failure was achieved after the collapse of the first carbon strip (see Fig. 10 and Fig11). By comparing with the original piers, at least

25% increase was obtained for the maximum force, but without significant increase on the maximum displacement, therefore this retrofit strategy has demonstrated to be insufficient to allow a satisfactory ductility behaviour.



a) Comparison before and after retrofit b) Final damage

Figure 10. Experimental results of pier PO1-N3-R1.



a) Comparison before and after retrofit b) Final damage

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

5. CONCLUSIONS

For original piers, the damage observed in the flanges was considerably low, mainly horizontal cracking along the pier height, but significant damaged was achieved in the webs, exhibiting inclined cracking and concrete spalling on extensive zones. Therefore, generally, these damage patterns were associated with shear mechanisms, revelling insufficient shear capacity provided by transverse reinforcement, however, for some square piers a mixed shear-flexure mechanism was observed. It was also possible to verify and justify the mechanisms of shear failure, by means of simple evaluations on flexural and shear capacities.

The CFRP retrofit design with overestimated level showed excellent benefits on both piers behaviour 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 40% increase over the maximum flexural force, tests have shown that such strategy is adequate to allow satisfactory ductility behaviour. The failure of these piers was achieved after the first strip of CFRP collapse and for the pier which the strip near the pier base was carried out with 200mm width, only a partial collapse of this referred strip near the foundation was achieved.

For the retrofitting with shear capacity slightly above the maximum flexural force, tests have shown that such strategy is insufficient to allow satisfactory ductility behaviour, 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.

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