

REINFORCED CONCRETE COLUMNS JACKETED WITH FRP COMPOSITES AND SUBJECTED TO CYCLIC HORIZONTAL LOADS

Carlos CHASTRE ¹ and Manuel A. G. SILVA ²

UNIC-Centro de Investigação em Estruturas e Construção
Departamento de Engenharia Civil
Faculdade de Ciências e Tecnologia da Universidade Nova de Lisboa
Quinta da Torre, 2829-516 Caparica, Portugal

¹ e-mail: chastre@fct.unl.pt

² e-mail: mgs@fct.unl.pt

Key words: Retrofitting, Reinforced Concrete Columns, FRP, seismic design.

Summary: *An experimental program conducted in order to analyze the behaviour of reinforced concrete columns jacketed with FRP composites and subjected to axial compression and cyclic horizontal loads is described. The dimensions of the cylindrical columns were 1500 mm height by 250 mm diameter. The influence of various parameters on the response, including the type of FRP used on confinement is reported. The results of the tests are shown and interpreted.*

1 INTRODUCTION

The repair and strengthening of concrete structures is more and more needed. Among the causes are design and construction flaws, structural degradation or damage due to environmental and natural hazards such as earthquakes, accidents, fires or structural deterioration associated with lack of maintenance. Besides, if the structure is used for purposes other than the original ones, retrofitting may be a necessity to compensate the increase of the live load or the elimination of structural elements.

In order to study the behaviour of reinforced concrete columns strengthened with FRP composites an extensive research program was developed at *Universidade Nova de Lisboa* (UNL), which included 45 monotonic or cyclic tests on short columns and 12 tests on circular columns subjected to axial compression and to horizontal cyclic actions.

The program and the testing setup of the short columns have already been described elsewhere [1]. The main parameters varied in the study were the spacing of stirrups (0.05; 0.10 and 0.15m), the dimension of the short columns (ϕ 0.15m or ϕ 0.25m for 0.75m of height), the type of concrete (plain or reinforced) and the amount and the type of FRP composites: 1 to 4 layers of CFRP (Replark or Mbrace) and 2 to 3 layers of GFRP (Tyfo). In the short R.C. columns of ϕ 0.15m, the longitudinal reinforcement 6ϕ 6mm is equal for all the models, in steel S400. The stirrups are 3mm diameter bars spaced apart 0.05; 0.10 or 0.15m. The short ϕ 0.25m R.C. columns were built with materials having the same characteristics as the 12 columns (concrete type and reinforcement: 6ϕ 12mm – S400 longitudinal steel with ϕ 6mm stirrups spaced 0.15m).

The tested columns had a diameter of 0.25m and a headway of 1.5m above the foundation of 1.2m x 0.5m x 0.6m. Different retrofitting solutions with CFRP, GFRP and polymeric concrete were tested. The load history of the tests for each column started with the application of a constant vertical load according to the retrofit (400, 600 or 800kN) followed by sequences of 3 displacement cycles, multiples of the yield displacement until the column fails.

The columns selected for presentation here are P1, P4 and P8 as they allow the illustration of main results. P1 was not retrofitted and corresponds to the reference model. P4 was retrofitted with the application of two layers of CFRP (Mbrace) and P8 with two layers of GFRP (Tyfo), both in the plastic

hinge zone. Figure 3 shows columns P1, P4 and P8 during the tests.

2. MATERIALS

Two different FRP materials were used: glass fiber Tyfo SHE-51 and carbon fiber MBrace C1-30. The technical procedures and resins were proposed by the manufacturers. Tyfo SHE-51 was applied with Tyfo S Epoxy and MBrace C1-30 with resin MBrace Saturate.

Tests were made in accordance with standards (ASTM-39-86 1993; ASTM-D3039/D3039M 1995; NP-EN10002-1 1990). The average cylindrical compressive strength at the time of the concrete columns tests, was $f_{c0} = 35.2$ MPa. The yield strength of the steel reinforcing bars was 391 MPa for $\varnothing 6$ and 458 MPa for $\varnothing 12$ [1].

Mechanical tests of the carbon fibers MBrace C1-30 led to $E_j = 241$ GPa, $f_j = 3937$ MPa and strain for the maximum force equal to 1.54% for coupons with 2 plies of CFRP and $t_{ply} = 0.176$ mm.

Lab tests of glass fiber Tyfo SHE-51 led to $E_j = 21$ GPa, $f_j = 459$ MPa and strain for the maximum force equal to 2.17% for coupons with 2 plies of GFRP and $t_{ply} = 1.27$ mm [1].

3. SETUP, INSTRUMENTATION AND TEST METHOD

3.1 Test setup

The columns were tested at the Laboratory of Heavy Structures of UNL, which has two shear walls with 2.80 m height. The thickness of the top test floor slab is 0.60m. Tie down points consist of 50 mm holes which are 1.00m spaced, symmetrically, in both directions.

The test system was conceived having in consideration the characteristics of the laboratory and the type of test to perform. As observed on Figure 1, the system was made up, in general, of a mechanical actuator integrated in the shear wall and capable of applying horizontal loads of ± 500 kN or displacements until 400 mm (± 200 mm). The displacement applied by the actuator was transmitted to the column through a set of metallic frames, among which there was a 200 kN load cell (TML TCLP-20B) which could measure the applied horizontal load. The model was attached to the strong floor through *Dywidag* prestressing steel *Threadbars*.

The test system (Figure 1) for the vertical load was conditioned by the desired level of axial load (400 kN), which conditioned the diameter of the *Dywidag* prestressing steel *Threadbars* and the hydraulic cylinders used. This solution with the center of the hinge 400 mm below the column base has, especially for big displacements, the disadvantage of an axis of application of vertical load not coincident with the column axis. It is, thus, necessary to correct the moments applied at the base of the column so as to have into consideration this eccentricity (which is function of the horizontal displacement at the top of the column).

In every model, the vertical system of application of the load is, thus, constituted by two *Dywidag* prestressing steel *Threadbars* at the base of the column and connected at its top through a set of metallic frames. The intended axial load is exerted on the column by tensioning the *Dywidag* bars through two hydraulic cylinders (Enerpac RRH307). The load is kept stable during the application of the horizontal displacements due to the use of the hydraulic pump with load maintainer system (Enerpac RRH307). In order to measure the applied vertical load two load cells (Microtest MT KCM/300) of 300 kN were placed between the top of the hydraulic cylinder and the anchor of the *Dywidag* bars.

Figure 1 shows also the mechanical actuator and the platform for test control with the console of the actuator, the data logger, the laptop and the interface box used for the transducers and strain gauges connection to the data logger.

3.2 Instrumentation

The instrumentation was similar in every column and its objective was to get the values of the horizontal and vertical force throughout the test; the horizontal displacements on top of the column and on the area of the plastic hinge and the rotation of the cross section in different sections near the base of the column and the strains on the steel, concrete and FRP were also sought.

After placement of steel reinforcement, electrical strain gauges (TML-FLA-5-11) were used on some longitudinal bars and on some hoops. On the longitudinal bars eighteen strain gauges were placed, two by two, at four levels of height (0; 100; 250 and 550 mm) between the hoops, totalling eight strain gauges on the North (E1 to E8) and eight on the South (E9 to E16). At a longitudinal bar on the central zone of the cross section other two strain gauges were placed at the base of the column. On the hoops, six strain gauges were placed (E19 to E24): two per hoop, one on the North and another on the South, at three levels of the column height (25; 175 and 475 mm).

On the composite jacket of FRP six strain gauges (TML-BFLA-5-8) were placed at three levels of the column height (50, 200 e 350 mm), three on the North (H1 to H3) and three on the South (H4 to H6).

As previously stated, three load cells were used, one to measure the horizontal force (CC3) and two (CC1 and CC2) to measure the vertical forces acting on the *Dywidag* bars.

Regarding the displacement transducers, three were used to measure the horizontal displacement of the column (D5, D6 and D7); four to measure the rotation of the section (D1 to D4); and other six (i1 to i6) to measure the rotation in three sections in an alternative manner.

The transducers D1 to D6 (CDP100 of TML) had a maximum displacement of 100 mm and a sensibility of 100×10^{-6} /mm. The displacement transducer D7 had a maximum displacement of 500mm and a sensibility of 10×10^{-6} /mm. The displacement transducers i1 to i6 have a maximum displacement ± 20 mm and the sensibility (250 or 147×10^{-6} /mm) dependent on the transducer length (100 or 150 mm).

A HBM *Centipede 100* (UPM100) data logger from HBM with a maximum capacity of sixty channels was used together with a laptop and software *Catman 4.0*. Forty six channels were used, corresponding to three load cells, thirteen displacement transducers and thirty strain gauges.

3.3 Test procedure

i) Determination of the yield displacement

Normally, the reference displacement (d_0) to be applied in cyclic tests corresponds to the yield displacement (Δy) obtained in a similar column tested monotonically. The yield displacement is an important parameter to take into account in the analysis of the columns behaviour, specially the ductility coefficient ($\mu_{\Delta} = \delta / \Delta y$). However, its analytical determination is not totally accurate because it depends on the tensile strength of the concrete and on the deformability of several materials [2], especially for non rectangular cross sections.

Experimental determination of the yield displacement is an alternative. In the literature there are several references [2 to 6] with different methods to make this experimental determination in monotonic tests. Among these, the ECCS [3] recommendation for steel structures deserves to be referred. Here the yield displacement is determined on the basis of monotonic tests. After the completion of the tests, the force-displacement diagram is made and the tangent module of the curve in the origin (E_0) and the tangent module in the rupture ($E_r = 0.1 \times E_0$) determined. The intersection of these two straight lines, the first one passing by the point (0, 0) and second by the point (F_{max} , δ_{max}), corresponds to yield point (F_y , Δy).

As Gomes [2] explains, this procedure is impracticable in RC structures because it requires the use of one or even two tests (if the structure is not symmetrical) just to determine the yield displacement. Gomes [2] compared three different methods to determine the yield displacement. The first one starts

with the analytical determination of the yield moment (M_y) and during the tests forces that had produced moments to $0.6M_y$ and $0.75M_y$, are applied in the critical cross section, measuring the corresponding displacements, $\delta_{0.6}$ and $\delta_{0.75}$. The yield displacement is determined from the following expression:

$$\Delta_y = \delta_{0.75} + \frac{\delta_{0.75} - \delta_{0.60}}{0.6} \quad (1)$$

The second method consisted of applying horizontal displacements in the top of the column until the yield moment (M_y) is reached and, of this form, getting the yield displacement. The third method corresponded to the application of horizontal displacements in the top of the column until the deformation in the tensioned steel bars was equal to the yield deformation of these. For Gomes [2], it was observed a good correlation between the values obtained by the three methods.

Caltrans [4] suggests another approach based on the bi-linearization of the bending moment-curvature corresponding to the column response. Thus, according to this method, a first displacement corresponding to the yield displacement of the steel reinforcing bars is determined (δ_y - displacement for first analytical yield), the yield displacement (Δ_y) being given by the following expression:

$$\Delta_y = \frac{M_n}{M_y} \times \delta_y \quad (2)$$

where M_n corresponds to the moment that allows that the area below the idealized diagram bending moment-curvature (bilinear) is equal to the area below of the corresponding test response curve. According to Hose and Seible [6] the nominal yield bending moment could be estimated in the critical section for $\epsilon_c=0.4\%$.

To sum up, there is not much uniformity of methods in the determination of the yield displacement. The method of ECCS [3] is essentially used for steel structures; in the methods used by Gomes [2] the yield corresponds to the beginning of the yield of the reinforcing steel bars; in the method presented by Caltrans [4] the nominal yield bending moment lies between the yield moment and the moment at rupture, so the yield displacement will always be higher than the corresponding displacement to the yield of the reinforcing steel bars.

The yield determination is normally associated with the inflection occurred in the course of the curves force-displacement or bending moment-curvature in monotonic tests. This inflection is difficult to determine with accuracy in cyclical tests or in the case of the high levels of axial load, since it can occur due to yield of the tensioned or compressed reinforcing steel bars or be the consequence of concrete spalling.

To determine the yield displacement of column P1, displacement cycles of 3 mm, 6 mm, 10 mm, 15 mm and 20 mm were applied. The yield displacement ($\Delta_y = 20$ mm) was detected through the reading of the strain gauges placed in the longitudinal reinforcing steel bars near the column base. On the other columns the displacement history described in the following section was applied.

ii) Displacement history

As referred previously, the reference displacement (d_0) to apply in cyclical tests corresponds normally to the yield displacement (Δ_y) obtained in a similar column tested monotonically. In the present case, the aim was to apply an identical load history in all the tests. Although the columns are similar before the retrofitting, it was considered that the yield displacement was not equal in all the tests. This was due to the different levels of vertical loads applied in the columns tested in this

research program; however, in columns P1, P4 and P8 the vertical load was maintained constant and equal to 400kN. The option was to apply a reference displacement of $d_0=0.5\Delta y$ (of the column P1) that was equal for all the columns tested in this research program and could also be a common multiple of the yield displacement of all them.

The displacement history adopted in the test of each column was initiated with the application of a constant vertical load (400kN), followed by series of three displacement cycles multiple of the reference displacement (d_0) until the end of the test. The test finished when the horizontal force reached a value 50% inferior to the maximum force or in case the column collapsed.

In the reference column, P1, after the completion of the first cycles for the determination of the yield displacement, the columns were placed in the initial position and a series of three displacement cycles multiples of the reference displacement ($1.0d_0$; $1.5d_0$; $2.0d_0$; $3.0d_0$; $4.0d_0$; . . . ; nd_0) were applied until the end of the test. Identical procedure was followed for the other columns.

In Figure 2 it is indicated the displacement history imposed to columns P1, P4 and P8, as well as the corresponding diagrams to the vertical load applied to them.

iii) Rupture criteria

For all the tests the rupture criteria was the moment when the force reached 85% of the maximum value occurred until then in the test. The values considered for the rupture correspond to the ones obtained in the cycle before the value of 0.85 of the maximum force was exceeded, whether in the North or South direction. All the tests proceeded beyond the post-rupture phase in order to evaluate the behaviour of the retrofitted columns. Hysteretic diagrams force-displacement are shown on Figure 4.

4. TESTS RESULTS

Figure 2 shows the diagrams of horizontal displacements and forces applied to columns P1, P4 and P8 during the tests and Figure 3 a general view and their corresponding curvature diagrams of the same columns. Hysteretic diagrams force-displacement of columns are shown on Figure 4.

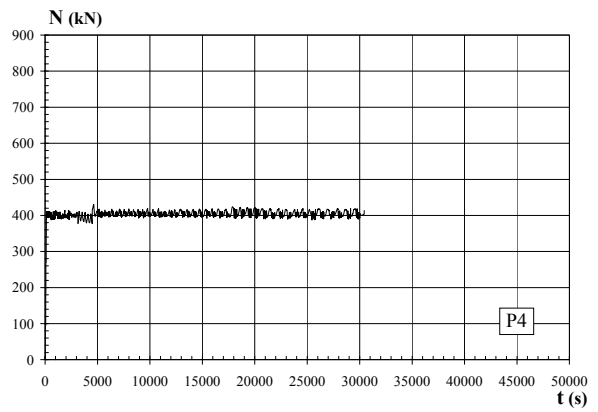
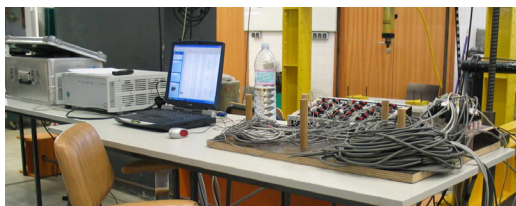
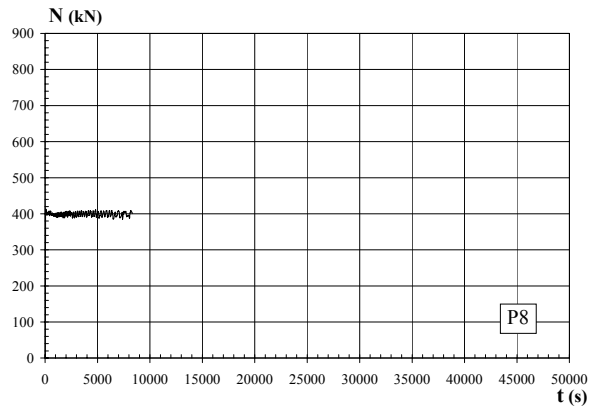
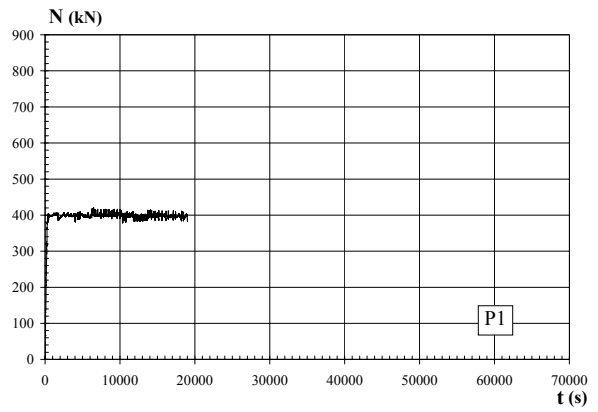
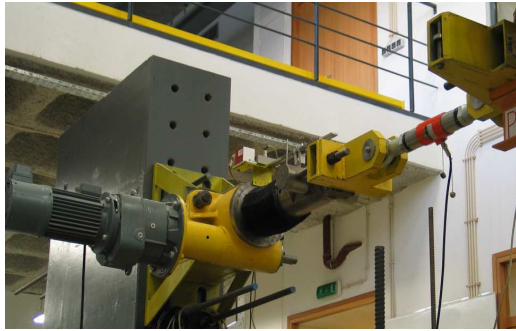
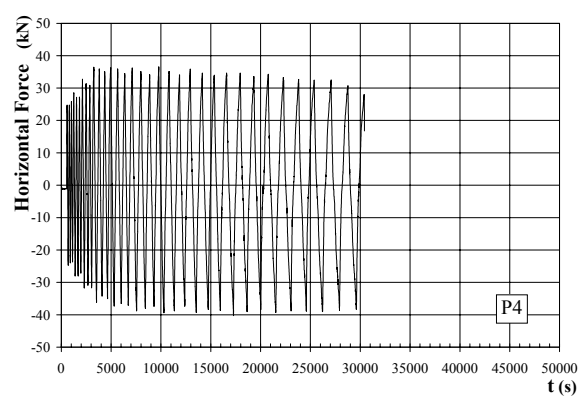
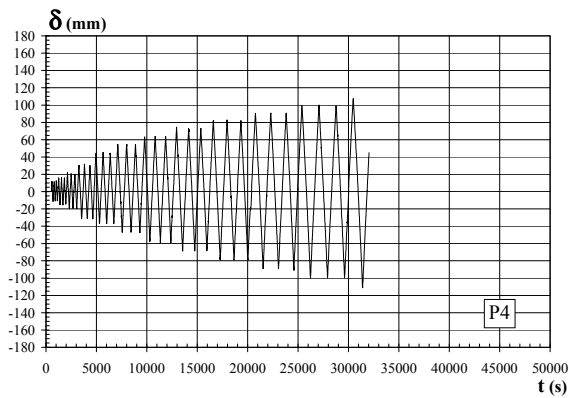
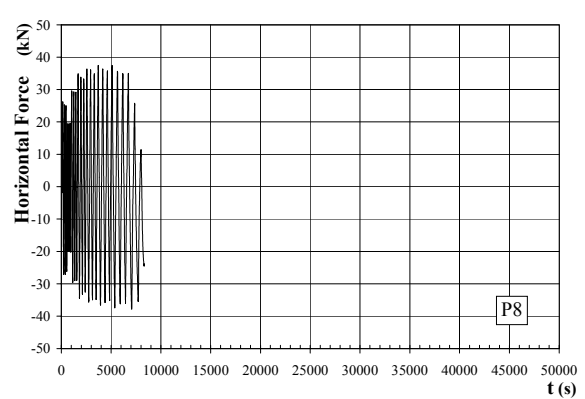
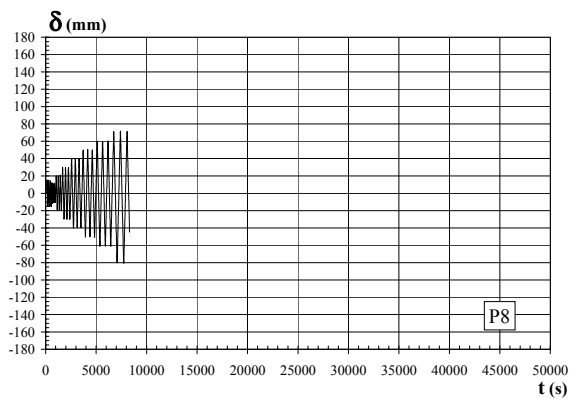
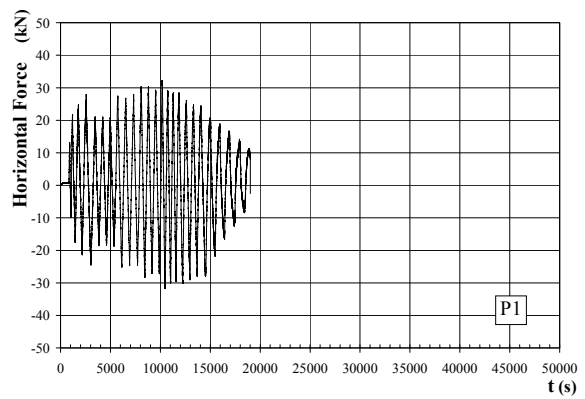
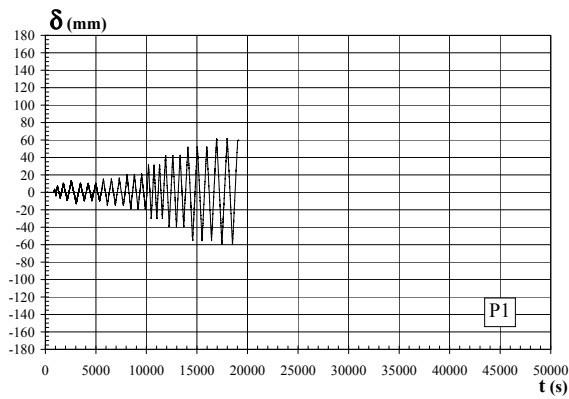


Figure 1: Testing setup and diagrams of vertical load vs. time.



Horizontal displacement - time

Horizontal Force - time

Figure 2: Diagrams of horizontal displacements and forces applied during the tests

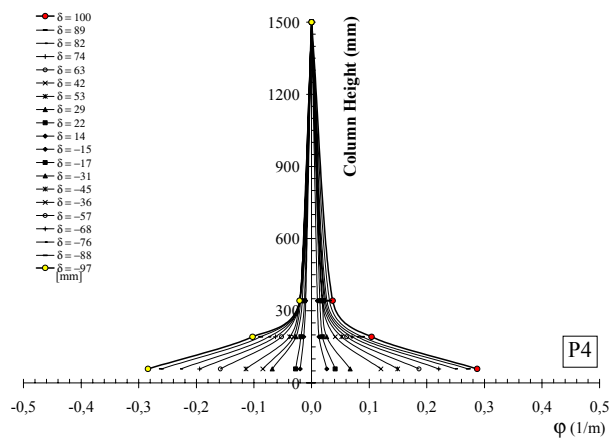
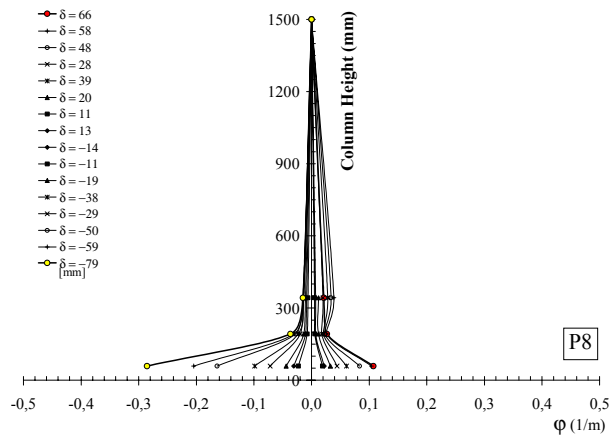
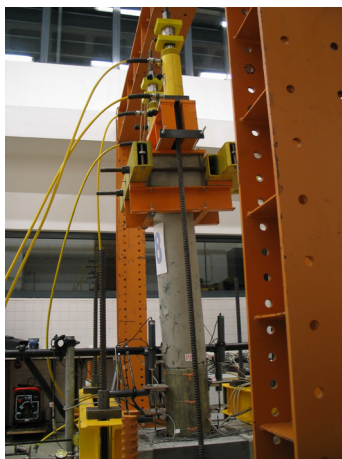
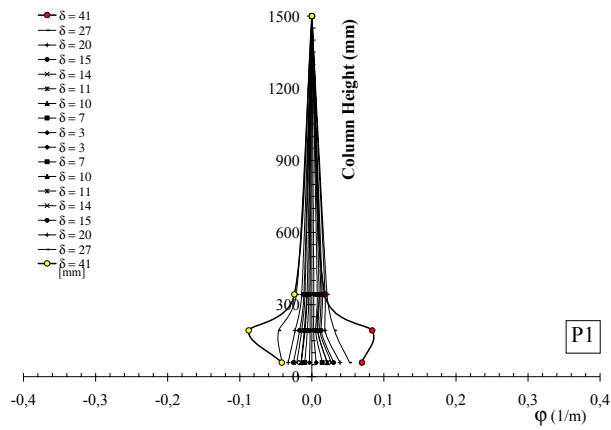


Figure 3: General view of columns P1, P8 and P4 during the tests and their corresponding curvature diagrams.

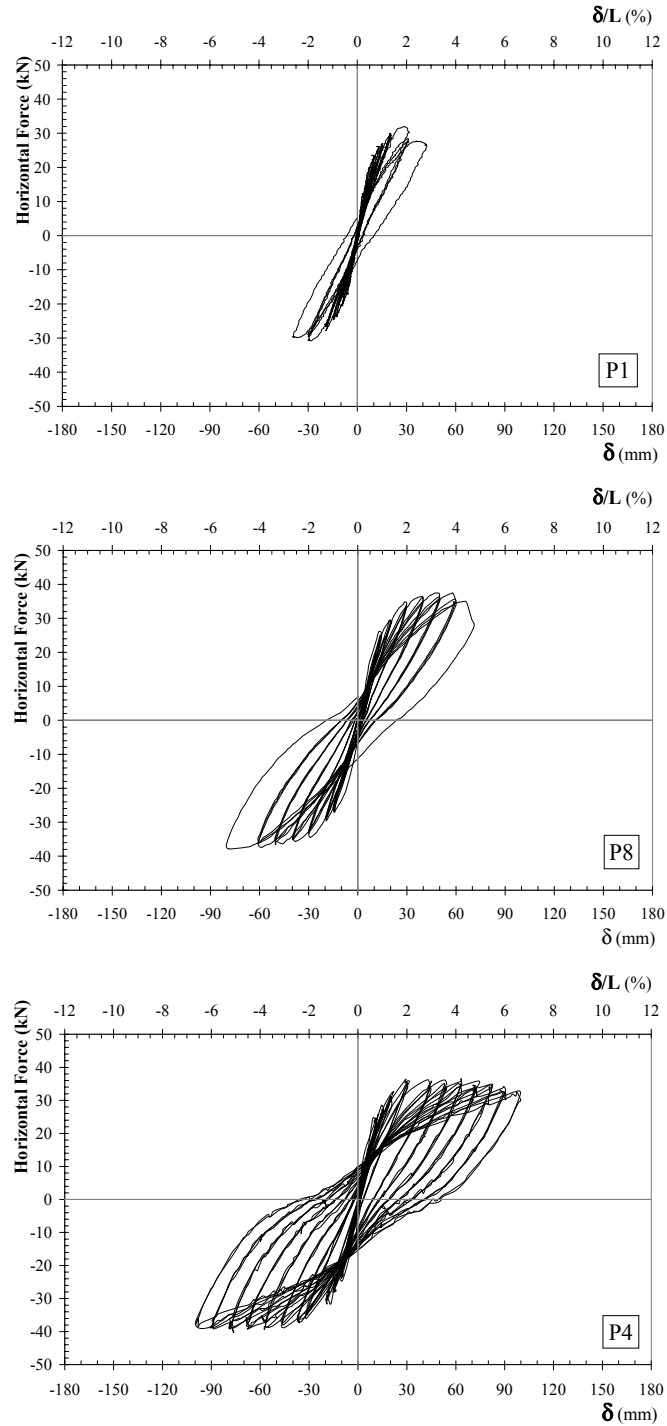


Figure 4: Hysteretic diagrams Force-Displacement of columns P1, P8 and P4.

5. ANALYSES OF RESULTS

In the previous sections the behaviour of the RC columns during the tests was presented. The next paragraphs analyse the results obtained in the tests.

Table 1 presents the values at rupture for the following performance parameters calculated for the columns P1, P4 and P8 [1]: drift (rate of horizontal displacement by column height); μ_{Δ} - ductility coefficient in displacement; μ_{ϕ} - ductility coefficient in curvature; θ_p - plastic rotation; W_d - dissipated energy by cycle of three; W_{acum} - dissipated energy accumulated by cycle; RDI - residual deformation index; ξ_{eq} - equivalent viscous damping rate and n_k - normalized effective stiffness.

Table 1 : Performance parameters of RC Columns P1, P8 (CFRP) and P4 (GFRP)

Parameters	P1	P8 (GFRP)	P4 (CFRP)
δ (mm)	41.2	65.6	97.4
δ/δ_{P1}	1	1.59	2.37
drift = δ/L (%)	2.75	4.37	6.5
μ_{Δ}	2.1	3.9	4.9
μ_{ϕ}	2.6	10.7	10.6
θ_p	0.014	0.039	0.052
F (kN)	32	37.7	38.4
F/F _{P1}	1	1.18	1.2
W_d (kNm)	1.55	2.72	4.06
W_{acum} (kNm)	10.5	13.9	51.3
RDI	0.53	1.06	2.09
ξ_{eq}	12.3	16.6	18.7
n_k	0.47	0.33	0.24

The yield displacement was 20.2 mm in the reference RC column P1 (non-retrofitted) for a vertical load of 400 kN. The columns retrofitted with GFRP (P8) or CFRP (P4) and subjected to the same vertical load presented a yield displacement 13% to 24% higher. It should be mentioned, however, that due to the involved variables and the reduced number of tests, the values of the yield displacement must not be seen as a precise value.

The difference that occurred in the displacements for the maximum force in the FRP RC retrofitted columns is not significant. In fact, the column response in this area levels as a large *plateau*, so that the small geometric or material variations can produce considerable changes in the displacement that occurs for the maximum force. Nevertheless, the value of the maximum force is important.

The jacketing of the RC columns increases the effective confined area in relation to the P1 column in 24%. The analysis of Table 1 reveals a force increment between 18 and 20% (columns P8 and P4), so that a correlation between the observed strength increase and the effective confined area of the retrofitted columns can be established.

The ductility coefficient in displacement, which is calculated, dividing the maximum displacement in the rupture for the yield displacement of the reference column, P1, shows values varying between 2.1 (P1) and 4.9 (P4). The described behaviour obtained from analysing the dissipated energy per cycle or the curvature ductility coefficient is similar.

If the normalized displacement in the rupture parameter is considered, the column P8 (2 GFRP) shows an increment of 59% relatively to the column P1 and the column P4 (2CFRP) shows an increment of 137% relatively to column P1.

The ductility gains expressed by the retrofitted columns must be balanced with the consequent

increment in the maximum strains in the longitudinal reinforcing steel bars, with values around 3.30% for P8 and 3.47% for P4. These increases, reaching sometimes 192% relatively to the strain observed on column P1 (1.19%), represent a clear decrease in the safety coefficient related to the steel rupture.

In the strengthened columns, the maximum strain observed in the jacket is 0.54% for column P4 with 2CFRP and 0.58% for column P8, with 2GFRP. These values are lower than 50% of the strain obtained in the flat coupons.

Regarding the strengthening, both retrofitted columns have a better performance than the non retrofitted ones; considering the type of FRP jacket, column P4 (2 CFRP), shows a better behaviour in terms of ductility coefficient (4.9) than the column P8 with 2 GFRP (3.3); the maximum force is similar (38.4 for column P4 and 37.7 for column P8). So, the retrofitted solution with CFRP is the most efficient one.

6 CONCLUSIONS

An experimental program was conducted in order to analyse the behaviour of reinforced concrete columns jacketed with CFRP or GFRP composites and subjected to axial cyclic compression and alternated cyclic horizontal loads and is described. The dimensions of the columns were 1500 mm height by 250 mm diameter and the models were subjected to a series of cyclic loadings. This allowed the study of the influence of various parameters in the response, including the type of confining material. The testing setup, the instrumentation and the test method are described and the experimental tests results are compared and discussed.

The jacketing of the RC columns increases the effective confined area in relation to the non-retrofitted column and the tests reveal a force increment. In this case, the ratio between the strength increase and the effective confined area increase of the retrofitted columns is 0.8.

The ductility gains expressed by the retrofitted columns must be balanced with the consequent increment in the maximum strains in the longitudinal reinforcing steel bars. These increments represent a clear decrease in the safety coefficient related to the steel rupture.

In the strengthened columns, the maximum strain values observed in the jacket are lower than 50% of the strain obtained in the flat coupons.

It was concluded that all the retrofitted solutions have a good performance (significant displacement ductility and strength increments) in relation to the non-retrofitted column. The retrofitted column with CFRP is the most efficient one.

REFERENCES

- [1] Chastre Rodrigues, C.; "Comportamento às Acções Cíclicas de Pilares de Betão Armado Reforçados com Materiais Compósitos". PhD Thesis in Structural Engineering, Universidade Nova de Lisboa, January 2005 (in Portuguese).
- [2] Gomes, A. M.; "Comportamento e Reforço de Elementos de Betão Armado Sujeitos a Acções Cíclicas". PhD Thesis, Universidade Técnica de Lisboa, Instituto Superior Técnico, July 1992 (in Portuguese).
- [3] ECCS; "Recommended Testing Procedure for Assessing the Behaviour of Structural Steel Elements Under Cyclic Loads", European Convention for Constructional Steelwork, N° 45, 1986.
- [4] CALTRANS. "Seismic Design Criteria (SDC)", Caltrans – California, Department of Transportation, Version 1.1, July 1999.
- [5] Priestley, M. J. N., and Seible, F.; Design of Seismic Retrofit Measures for Concrete and Masonry Structures. Construction and Building Materials, Elsevier 9 (1995).
- [6] Hose, Y., and Seible, F.; "Performance Evaluation Database for Concrete Bridge Components and Systems under Simulated Seismic Loads". Pacific Earthquake Engineering Research Center, PEER 1999-11, 1999.